Geotechnical Evaluation

Chet F. Harritt School
Building Additions
8120 Arlette Street
Santee, California

Santee School District

9625 Cuyamaca Street | Santee, California 92071

July 11, 2019 | Project No. 108774001





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

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

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Geotechnical Evaluation

Chet F. Harritt School
Building Additions
8120 Arlette Street
Santee, California

Ms. Christina Becker
Santee School District
9625 Cuyamaca Street | Santee, California 92071

July 11, 2019 | Project No. 108774001

Christine M. Kuhns, PE (LA)
Project Engineer

Jeffrey T. Kent, PE, GE
Principal Engineer

Christina A. Tretinjak, PG, CEG
Senior Project Geologist

CMK/CAT/JTK/gg

Distribution: (1) Addressee (via e-mail)

CONTENTS

TABLES

FIGURES

APPENDICES

1 INTRODUCTION

In accordance with your authorization and Notice to Proceed dated March 8, 2019, we have performed an update geotechnical evaluation for the proposed building additions to the Chet F. Harritt School campus located at 8120 Arlette Street in Santee, California (Figure 1). The purpose of our services was to update our previous geotechnical evaluation reports from 2007 and 2009 (Ninyo & Moore, 2007 and 2009) to the current 2016 California Building Code (CBC) requirements and to update our infiltration testing report (Ninyo & Moore, 2017) to local guidelines for site infiltration to address storm water management. Our geotechnical evaluation was performed in general accordance with Chapter 18A of Title 24, Part 2, Volumes 1 and 2 of the 2016 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.

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), update geotechnical report (Ninyo & Moore, 2009), and infiltration testing report (Ninyo & Moore, 2017) for the school campus, 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 six (6) 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 including our previous geotechnical evaluation, this recent subsurface evaluation, 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 Chet F. Harritt School in Santee, California (Figure 1). In general, the campus is located on a trapezoidal-shaped parcel bound by single-family residential properties to the north and west, Arlette Street to the south, and Santee Pioneer Little League Fields and Mesa Road to the east. The school site generally consists of school buildings, facilities, parking lots in the central, south, and southeastern portions of the campus, with hardscape areas in the northwestern and northeastern portions of the property. The school site consists of a relatively level pad generally constructed over a north-trending drainage. The campus slopes very gently towards the northeast with elevations ranging from approximately 350 feet above mean sea level (MSL) at the school entrance in the southwestern portion of the campus to approximately 340 feet above MSL in the northeast portion of the campus. Review of historical topographic maps indicates that a drainage channel is present on the west side of the campus. The global coordinates of the project site are approximately 32.832°N Latitude and -117.0179°W Longitude.

Our office previously performed two geotechnical evaluations at the project site (Ninyo & Moore, 2007 and 2009). Based on our correspondence with the Project Architect, we understand that the project has been changed from the previous design described in our 2007 and 2009 geotechnical evaluation reports. The current project includes the construction of two, single-story buildings near the central and southeast portions of the campus. The two new buildings will be located east of the existing school buildings, in the central portion of the campus that is currently improved with modular classroom buildings, grass, and hardscape areas. A smaller rectangular building to be constructed in the central portion of the campus will have a footprint of approximately 2,900 square feet (sf) and a larger C-shaped building is to be constructed in the southeastern portion of the campus will have a footprint of approximately 13,500 sf. Additionally, we understand an approximately 2,000 sf single-story modular building is proposed east of the proposed C-shaped building. Per Division of the State Architect (DSA) Interpretation of Regulations (IR) A-4.13, this single-story modular building does not require a geotechnical report. Further improvements are anticipated to include new pavements, concrete flatwork, and underground utilities.

This update evaluation was performed in accordance with the 2016 CBC requirements and the CGS guidelines for geotechnical evaluations for public schools, which includes a minimum number of borings per school building based on the area of the ground-floor footprint of the building. Per CGS Note 48, one boring or exploration is required per 5,000 sf of building footprint, with a minimum of two borings per building. We have utilized some existing boring data from our previous geotechnical evaluations at the site (Ninyo & Moore, 2007 and 2009).

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 18 and 19, 2019 and included the drilling, logging, and sampling of six small diameter borings (NM-1 through NM-6). Borings NM-1 through NM-4 were drilled to depths up to approximately 19.4 feet using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Borings NM-5 and NM-6 were manually excavated to depths of approximately 5 feet using a 6-inch diameter hand auger. Our previous subsurface explorations were conducted in 2007, 2009, and 2017. Four small diameter borings (B-1 through B-4) were drilled on June 25 and June 26, 2007, another seven small diameter borings (B-5 through B-11) were drilled on January 19 and February 21, 2009, and four infiltration tests (IT-1 through IT-4) were excavated on April 17, 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 was performed on representative soil samples collected from our recent subsurface exploration. Testing for this recent study 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. Discussion regarding testing procedures used and the results of the other laboratory tests that we performed are presented in Appendix B. Also, the laboratory test results included our previous 2009 evaluation report are also presented in Appendix B.

6 INFILTRATION TESTING

As a means of evaluating the infiltration characteristics of near-surface materials, infiltration tests were performed on April 18 and 19, 2019 at two locations designated NM-5 and Ninyo & Moore-6 (Figure 2). Additionally, infiltration tests were performed at four locations designated IT-1 through IT-4 on April 17 and 18, 2017 as part of a previous evaluation (Ninyo & Moore 2017). Infiltration test borings were manually excavated to depths up to approximately 5 feet using a 6-inch diameter hand auger. Following excavation, the infiltration test locations were prepared by placing approximately 2 inches of gravel on the bottom, installing a 2-inch diameter perforated PVC pipe, and backfilling the annulus with pea gravel. As part of the test procedure, a presoak was performed on April 17, 2017 and April 18, 2019 for IT-1 through IT-4, and Ninyo & Moore-5 and NM-6, respectively, to represent adverse conditions for infiltration. The presoak consisted of maintaining approximately 1 to 2 feet of water in each test boring for approximately 4 hours. The water level was then allowed to drop overnight.

Infiltration testing at IT-1 through IT-4, and NM-5 and NM-6 was performed on April 18, 2017 and April 19, 2019, respectively, in general accordance with the City of Santee BMP Design Manual (2016). The infiltration test holes were filled with approximately 6 to 24 inches of water 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 NM-5, NM-6, and IT-3 indicated that the observed (i.e., unfactored) infiltration rates were 0.13 inches per hour or less. Infiltration tests IT-1 and IT-4 did not infiltrate. The infiltration rate of 8.64 inches per hour in IT-2 is not considered indicative of the site's infiltration characteristics. 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. Completed Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions and Worksheet D.5-1: Factor of Safety and Design Infiltration Rate Worksheet are presented in Appendix C. The rates presented in Table 1 are to be used for preliminary design purposes.

Table 1 – Infiltration Test Results Summary					
Infiltration Test	Approximate Test Depth (feet)	Description	Observed Infiltration Rate (in/hr)	Suitability Assessment Factor of Safety[1]	Reliable/Factored Infiltration Rate[2] (in/hr)
IT-1	2.5	Weathered GRANITIC ROCK	DNI	-	DNI
IT-2	3.0	Silty SAND (Fill)	8.64[3]	-	N/A
IT-3	2.0	Weathered GRANITIC ROCK	0.04	2.0	0.02
IT-4	2.0	Weathered GRANITIC ROCK	DNI	-	DNI
NM-5	5.0	Silty SAND (Fill)	0.13	2.0	0.07
NM-6	5.0	Silty, Clayey SAND (Fill)	0.01	2.0	<0.01

Notes:
in/hr = inches per hour
DNI = did not infiltrate
[1] Design safety factor to be determined by the design engineer in accordance with Appendix D of the City of Santee BMP Design Manual (2016)
[2] Factored infiltration rate shall be divided by the design safety factor to obtain the design infiltration rate.
[3]Testing may have been performed by an underground utility trench or another preferential pathway for water. The tested rate is not considered indicative of the site infiltration characteristics.

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, materials of the Tertiary-age Friars Formation, and Cretaceous-age Tonalite, hereafter referred to as “granitic rock”, and (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 Figure 4.

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 asphalt concrete (AC) over decomposed granite (DG) used as a base course or subgrade soils. 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 Base Course Thickness (inches)
NM-1	3	-*
NM-2	7	5

Notes:
* Did not encounter
AC – Asphalt concrete
DG – Decomposed granite

7.2.2 Fill

Fill material was encountered in all the borings we performed for our recent and previous evaluations from the ground surface or below the pavement sections to depths up to approximately 10 feet. As encountered, the fill material generally consisted of various shades of gray, brown, and red, moist, medium dense, silty sand, clayey sand, well graded sand, and sandy silt, and stiff to hard, sandy clay. Scattered amounts of gravel and cobble fragments were encountered in the fill. Documentation regarding fill placement was not available for our review.

7.2.3 Younger Alluvium

Younger alluvium was encountered in borings B-1, B-8 through B-10 from our previous evaluations and in boring NM-4 from our recent evaluation. The younger alluvium was encountered underlying the fill materials and extended to depths up to approximately 13 feet. As encountered, the material generally consisted of various shades of brown, moist, stiff to hard, sandy clay, and medium dense, sandy and clayey silt. Gravel and scattered cobble fragments were encountered in the younger alluvium.

7.2.4 Older Alluvium

Older alluvium was encountered in boring B-9 of our previous evaluation (Ninyo & Moore, 2009) underlying the younger alluvium and extending to a depth of approximately 14.5 feet. As encountered, the material generally consisted of dark reddish brown, moist, hard, silty clay. Gravel and cobble fragments were encountered in the older alluvium.

7.2.5 Friars Formation

Although not encountered in our recent or previous subsurface explorations, materials of the Tertiary-age Friars Formation have been mapped at the site. These materials are generally expected to consist of brown and gray siltstone, sandstone, and claystone.

7.2.6 Granitic Rock

Granitic rock was encountered in several of our borings for the recent and previous explorations. The granitic rock was encountered underlying the fill, younger alluvium, and older alluvium and extended to the total depths explored. As encountered, the material generally consisted of various shades of brown, fine to coarse grained, weathered, granitic rock.

7.3 Groundwater

Groundwater was not encountered during our recent subsurface evaluation. However, groundwater was encountered during our previous geotechnical evaluation in boring B-1 at a depth of approximately 13 feet below existing grade (Ninyo & Moore, 2007). Due to the close proximity of a drainage channel on the west side of the school, groundwater should be anticipated at depths of approximately 10 to 15 feet. 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 alluvium and granitic rock, within fractures in the granitic rock, 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 Hazards

Based on review of the 2012 map available on the Federal Emergency Management Agency (FEMA) Mapping Information Platform website, the site is not within a flood zone. Based on review of topographic maps, the site is located approximately 0.35 miles south 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 30 to 50 feet above the riverbed. Although a drainage exists less than 40 feet from the Building Option C site, this drainage does not pose a flood hazard according to the reviewed 2012 map on the FEMA Mapping Information Platform website. Based on this review and our site reconnaissance, the potential for significant flooding of the site is not a design consideration.

7.5 Landsliding

Per Tan (1995), the site is mapped as “most susceptible” to landsliding. Furthermore, a review of the geology map (Kennedy and Tan, 2008) indicates that a potential landslide is mapped approximately 1,800 feet east of the site. However, based on our review of referenced geologic maps, literature, topographic maps, and stereoscopic aerial photographs, no landslides or indications of deep-seated landsliding were noted underlying the project site. 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 11.4 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 United States Geological Survey (USGS) National Seismic Hazard Maps - Fault Parameters website (USGS, 2019).

Table 3 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (M_{max})
Rose Canyon	11.4 (18.4)	6.9
Coronado Bank	23.8 (38.3)	7.4
Elsinore (Julian Segment)	30.2 (48.6)	7.4
Newport-Inglewood (Offshore Segment)	32.7 (52.6)	7.0
Earthquake Valley	34.7 (55.8)	6.8
Elsinore (Temecula Segment)	35.1 (56.5)	7.1
Elsinore (Coyote Mountain Segment)	40.8 (65.7)	6.9
San Jacinto (Coyote Creek Segment)	51.1 (82.2)	7.0
San Jacinto (Borrego Segment)	53.8 (86.6)	6.8
San Jacinto (Clark Segment)	53.9 (86.7)	7.1
San Jacinto (Anza Segment)	53.9 (86.7)	7.3
Elsinore (Glen Ivy Segment)	56.8 (91.4)	6.9
Palos Verdes	60.7 (97.7)	7.3

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.

Table 4 – Historical Earthquakes that Affected the Site

Date	Magnitude (M)	Approximate Epicentral Distance miles (kilometers)
November 22, 1800	6.5	20 (32)
May 27, 1862	5.9	14 (22)
February 9, 1890	6.3	57 (91)
February 24, 1892	6.7	42 (68)
May 28, 1892	6.3	53 (86)
October 23, 1894	5.7	12 (20)
September 30, 1916	5.0	55 (88)
January 1, 1920	5.0	31 (50)

Table 4 – Historical Earthquakes that Affected the Site		
Date	**Magnitude (M)**	**Approximate Epicentral Distance miles (kilometers)**
November 25, 1934	5.0	55 (89)
March 25, 1937	6.0	59 (95)
June 4, 1940	5.1	35 (57)
October 21, 1942	6.5	60 (96)
August 15, 1945	5.7	57 (92)
November 4, 1949	5.7	51 (82)
March 19, 1954	6.2	57 (92)
May 26, 1957	5.0	60 (97)
September 23, 1963	5.0	57 (91)
April 9, 1968	6.4	60 (96)
April 28, 1969	5.8	52 (84)
January 12, 1975	5.1	55 (88)
February 25, 1980	5.6	54 (87)
July 13, 1986	5.8	50 (81)
October 31, 2001	5.2	55 (88)
June 12, 2005	5.2	54 (87)

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 relatively dense nature of the granitic rock encountered in our borings, 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 exploration, 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 areas of the proposed two new buildings are underlain by fill overlying younger alluvium, older alluvium, and granitic rock.

- Based on the laboratory testing presented in Appendix B, the existing fill and younger alluvium are potentially compressible. Remedial grading recommendations to address the compressibility of these soils are presented herein. However, due to the depths of the existing alluvium with respect to the groundwater table, practical remedial grading may result in leaving unmitigated alluvial soils beneath the building pad for the smaller rectangular building. Accordingly, this may result in additional static settlements for this building.

- The existing fill, and younger and older alluvium encountered onsite 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 in granitic rock may encounter very difficult excavation characteristics and additional efforts including heavy ripping or coring should be anticipated.

- Excavations that extend into the granitic rock are anticipated to generate oversize material and additional processing and handling of these materials, including screening and/or rock picking, should be anticipated.

- Onsite materials are generally considered suitable for reuse onsite as engineered fill, provided they are processed to meet the recommendations provided herein.

- Excavations that extend to near or below groundwater are anticipated to encounter caving conditions and yielding subgrade conditions.

- Groundwater was not encountered during this geotechnical evaluation. However, groundwater was encountered at a depth of approximately 13 feet below grade during a previous evaluation (Ninyo & Moore, 2007). Due to the close proximity of a drainage channel on the west side of the school, groundwater should be anticipated at depths of approximately 10 to 15 feet.

- 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 11.4 miles west of the project.

- Based on the results of our limited geotechnical laboratory testing performed for our recent evaluation, the onsite soils exhibit a very low expansion potential. However, our previous evaluations encountered soils that possessed a very low to medium potential for expansion. Our recommendations presented herein include consideration for the expansive nature of onsite soils.

- Based on the results of our limited geotechnical laboratory testing presented in Appendix B for our recent and previous evaluations as compared to the Caltrans (2018) corrosion guidelines, the onsite soils would not be considered corrosive.

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, and younger and older alluvium, granitic rock, and Friars Formation. The fill and alluvium 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 in granitic rock may encounter very difficult excavation characteristics and additional efforts including heavy ripping or coring should be anticipated. Excavations into the granitic rock may generate oversize material and additional processing and handling of these materials, including screening and/or rock picking, should be anticipated.

Drilling of holes within the granitic rock can also be expected to be difficult and the use of specialized equipment (such as core barrels or percussion drilling) may be needed to advance to design depths. Additionally, caving of open excavations (i.e., drilled excavations and trenches) should be anticipated, particularly where groundwater and/or cohesionless soils are encountered, or where excavations are not promptly backfilled.

9.1.3 Temporary Excavations

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

Fill, Younger and Older Alluvium	*Type C*
Granitic Rock, Friars Formation	*Type B*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 (horizontal to vertical) in fill, younger alluvium, and older alluvium, and 1:1 (horizontal to vertical) in granitic rock and materials of the 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 – C-Shaped Building

Borings NM-1 through NM-3 encountered sandy fill materials over granitic rock. The fill was encountered to depths of approximately 4.5 and 10.5 feet at the south and north ends of the building, respectively. In order to provide suitable support for the proposed new, single-story C-shape building, we recommend that the existing undocumented fills and younger alluvial soils within the building pad be overexcavated and replaced with compacted, engineered fill. 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.

We recommend that the existing fill materials and younger alluvial soils within the building pad be removed down to competent granitic rock or to a depth of 2 feet below the bottom of footings, whichever is deeper. This overexcavation should extend to the horizontal limits of the building pad as previously defined, where feasible. The lateral extents of the overexcavation may be modified in the field based on site constraints such as existing buildings 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.

Subsequent to removal, the resulting surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM Test Method D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50). Compacted fill materials with a medium potential for expansion may be placed at depths greater than 5 feet within the building pad. These materials are anticipated to consist of the soils derived from onsite excavations that have been processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

Additionally, some onsite soils possess a medium potential for expansion. These materials are not considered suitable for reuse as compacted fill within the upper 5 feet below the pad subgrade elevation for the building.

9.1.6 Remedial Grading – Rectangular Building

Boring NM-4 encountered sandy fill materials over younger alluvial soils that were further underlain by granitic rock at the southern end of the building. The fill was encountered to a depth of approximately 7 feet. The underlying clayey alluvium extended to a depth of approximately 13 feet. Boring B-3 from our previous evaluation (Ninyo & Moore, 2007) encountered sandy fill soils over granitic rock at the north end of the proposed building. The fill was encountered to a depth of approximately 8 feet. In order to provide suitable support for the proposed new, single-story rectangular building, we recommend that the existing undocumented fills and younger alluvial soils within the building pad be overexcavated and replaced with compacted, engineered fill. 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.

We recommend that the existing fill materials and younger alluvial soils within the building pad be removed down to competent granitic rock or to a depth of 2 feet below the bottom of footings, whichever is deeper. This overexcavation should extend to the horizontal limits of the building pad as previously defined, where feasible. The lateral extents of the overexcavation may be modified in the field based on site constraints such as existing buildings 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.

Subsequent to removal, the resulting surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM Test Method D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50). Compacted fill materials with a medium potential for expansion may be placed at depths greater than 5 feet within the building pad. These materials are anticipated to consist of the soils derived from onsite excavations that have been processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

Based on the presence of deeper younger alluvial soils and a shallow groundwater table, full overexcavation of the younger alluvial soils may not be practical. Accordingly, we recommend that portions of the overexcavation that are deeper than 8 feet and that are encountering a shallow groundwater condition not be extended to further depths. The resulting removal surface should be stabilized and the overexcavation backfilled. However, if a shallow groundwater table is not encountered during remedial grading, the overexcavation should extend down to competent granitic rock as described above.

Due to the anticipated wet and yielding conditions, overexcavations that do not extend through the younger alluvial soils will need to be stabilized. For the stabilization efforts, we recommend stabilizing the removal surface by placing a 2-foot thick layer of aggregate base materials reinforced with geogrid. Once the resulting removal surface has been stabilized, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50). These materials are anticipated to consist of the soils derived from onsite excavations that have been processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

Additionally, some onsite soils possess a medium potential for expansion. These materials are not considered suitable for reuse as compacted fill within the upper 5 feet below the pad subgrade elevation for the building.

9.1.7 Remedial Grading – Site Retaining Walls

For site retaining walls that are not a part of or are not connected to a building, we recommend that the existing foundation subgrade soils at the site be removed down to a depth of 2 feet below the bottom of the 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 recompacted 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 recompacted, 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.

Note, some onsite soils possess a medium potential for expansion. These materials are not considered suitable for reuse as compacted fill within 2 feet below the bottom of the retaining wall footing.

9.1.8 Remedial Grading – Pedestrian Concrete Paving

In the proposed pedestrian concrete paving and exterior flatwork areas, we recommend that the onsite soils be overexcavated to a depth of 1 foot below the planned subgrade elevation for the surface improvement. 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 recompacted 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.

9.1.9 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 recompacted 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.

9.1.10 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 recent and previous evaluations at the site 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 in the upper 5 feet of building pads, in the upper 2 feet 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, if needed, should generally be granular soils with a very low to low expansion potential (i.e., an expansion index 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 greater 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, provided they are free from painted surfaces and rebar. 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. Additionally, the landscape architect should be consulted regarding the reuse of these materials within fill soils to be placed in landscaped areas.

9.1.11 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.

9.1.12 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.

9.1.13 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.

9.1.14 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 within the upper 5 feet of building pads, in the upper 2 feet beneath retaining wall footings, as retaining wall backfill, 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.

9.1.15 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.

9.1.16 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).

Table 5 – 2016 California Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.149
Site Coefficient, F_v	1.719
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.877g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.341g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.008g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.585g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.672g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.390g

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 with interior concrete slabs-on-grade. Recommendations for the shallow 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 - C-Shape Building

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 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.

9.3.1.1 Lateral Resistance - C-Shape Building

For resistance of footings to lateral loads, bearing on compacted fill, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 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 0.35 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.

9.3.1.2 Static Settlement - C-Shape Building

We estimate that the proposed structure, 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.

9.3.2 Shallow Foundations – Rectangular Building

Shallow, spread or continuous footings supported on compacted fill may be designed using an allowable bearing capacity of 2,500 psf. These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. We recommend that shallow foundations for the new 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.

9.3.2.1 Lateral Resistance - Rectangular Building

For resistance of footings to lateral loads, bearing on compacted fill, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 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 0.35 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.

9.3.2.2 Static Settlement – Rectangular Building

As described earlier, we understand that the remedial grading efforts for the building may not be able to overexcavate the younger alluvial soils down to competent granitic rock. Accordingly, some unmitigated and potentially compressible younger alluvial soils may remain in-place. We estimate that the proposed structure, designed and constructed as recommended herein, and founded in compacted fill over unmitigated younger alluvial soils will undergo total settlement on the order of 1½ inches. Differential settlement on the order of 1 inch over a horizontal span of 40 feet should be expected.

We understand that the settlements presented above are considered tolerable for the rectangular building. However, in the event that such settlements are not considered tolerable for the structure, consideration can be given to supporting the building on cast-in-drilled hole (CIDH) foundations founded on granitic rock or installing rammed aggregate piers to improve the subsurface ground conditions. If requested, such recommendations can be provided in a supplemental addendum.

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 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

We recommend that conventional, interior concrete slab-on-grade floors underlain by compacted fill materials of generally very low to low expansion potential (i.e. an expansion index of 50 or less) 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 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.

9.7 Preliminary Flexible Pavement Design

We understand that the project will include the construction of new pavements. Our laboratory testing of a near surface soil sample at the project site indicated an R-value of 19. This R-value, along with estimated design Traffic Indices (TI) of 5, 6, and 7 has been the basis of our preliminary flexible pavement design. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement 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)	19	3	8
6 (Drive Aisles)	19	3	11
7 (Fire Lanes and Bus	19	4	13

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, 8 inches of 600 pounds per square inch (psi) flexural strength Portland cement concrete reinforced with No. 4 bars, 18-inches on center, be placed over 4 inches or more of aggregate base materials compacted to a relative compaction of 95 percent.

9.8 Exterior 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.

9.9 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 recent and previous corrosivity testing of site soils indicated electrical resistivities between 1,400 and 16,080 ohm-cm, soil pH values between 5.5 and 7.2, chloride contents between 60 and 340 ppm, and sulfate contents between 0.001 and 0.019 percent (i.e., 10 and 190 ppm). Based on a comparison with the Caltrans corrosion (2018) criteria, the onsite soils would not 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 can be subject to premature chemical and/or physical deterioration. The soil samples tested in this and previous evaluations indicated water-soluble sulfate contents between 0.001 and 0.019 percent by weight (i.e., 10 and 190 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the site soils would correspond to exposure class S0. For this exposure class, ACI 318 recommends that normal weight concrete in contact with soil possess a compressive strength of 2,500 pounds per square inch (psi) or more. Furthermore, due to the potential for variability of site soils we also recommend that normal weight concrete in contact with soil use Type II, II/V, or V cement.

10 PERMANENT INFILTRATION DEVICES

Field testing of the site subsurface soils at the Chet Harritt School campus indicated factored infiltration rates between less than 0.01 and 0.09 inches per hour at borings NM-5 and NM-6, respectively. During our previous evaluation (Ninyo & Moore, 2017), in-situ infiltration rates ranged between 0.04 and 8.64 inches per hour at borings IT-3 and IT-2, respectively. The infiltration rate of within boring IT-2, 8.64 inches per hour, is not considered indicative of the site's infiltration characteristics. Additionally, borings IT-1 and IT-4 did not infiltrate. Based on our test results for IT-1, IT-3, IT-4, NM-5, and NM-6, 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.

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 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.

14 REFERENCES

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

American Society of Civil Engineers (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, ASCE 7-10.

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

California Building Standards Commission, 2016, California Building Code, Title 24, Part 2, Volumes 1 and 2.

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

California Emergency Management Agency, 2009, Tsunami Inundation Map for Emergency Planning, San Diego County, California: dated June 1.

California Geological Survey (CGS), 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: dated February.

California Geological Survey (CGS), 1999, Seismic Shaking Hazard Maps of California: Map Sheet 48.

California Geological Survey (CGS), 2008a, Earthquake Shaking Potential for California (revised): Map Sheet 48.

California Geological Survey (CGS), 2008b, Guidelines for Evaluating and Mitigating Seismic Hazards in California (revised).

California Geological Survey (CGS), 2010, Interactive Fault Activity Map of California; http://maps.conservation.ca.gov/cgs/fam/.

California Geological Survey (CGS), 2013, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings: Note 48: dated October.

City of Santee, 2016, Best Management Practices Design Manual: dated February 16.

County of San Diego, 1960, Topographic Survey, Sheet 242-1761, Scale 1:2,400.

County of San Diego, 1973, Topographic Survey (Orthotopo), Sheet 242-1761, Scale 1:2,400.

County of San Diego, 2009, Draft – Liquefaction Map, Hazard Mitigation Planning.

Department of Toxic Substances Control (DTSC), 2001, Information Advisory – Clean Import Fill Material, http://www.dtsc.ca.gov/Schools/index.cfm: dated October.

Geotracker website, 2019, www.geotracker.waterboards,ca.gov: accessed in March.

Google Earth, 2019, https://www.google.com/earth/: accessed in March.

Harden, D.R., 2004, California Geology, 2nd ed.: Prentice Hall, Inc.

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

Hartley and Duncan, 1987, E' and Its Variation with Depth, Journal of Transportation Engineering, Volume 113, Issue 5: dated September.

Historic Aerials website, 2019, www.historicaerials.com: accessed in April.

Kennedy, M.P. and Tan, S.S., 2008, Geologic Map of San Diego 30' by 60' Quadrangle, San Diego County, California, Scale 1:100,000.

Jennings, C.W., 2010, Fault Activity Map of California and Adjacent Areas: California Geological Survey, California Geological Map Series, Map No. 6.

Ninyo & Moore, In-house Proprietary Data.

Ninyo & Moore, 2007, Geotechnical Evaluation, Chet Harritt School, Santee, California, Project No. 106115001: dated July 5.

Ninyo & Moore, 2009, Update Geotechnical Evaluation, Building Location Option C, Chet Harritt School, Santee, California, Project No. 106115001: dated March 31.

Ninyo & Moore, 2017, Results of Infiltration Testing, Drops Grant Project at Nine School Campuses, Santee School District, Santee, California, Project No. 108356001: dated May 24.

Ninyo & Moore, 2019, Proposal for Update to Geotechnical Evaluation Report, Building Additions to Chet F. Harritt School, 8120 Arlette Street Santee, California, Proposal No. 106115001: dated February 21.

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

Structural Engineering Association of California (SEAOC), Office of Statewide Health Planning and Development (OSHPD), 2019, U.S. Seismic Design Maps website, https://seismicmaps.org/: Accessed in April.

StudioWC, 2019, Overall Site Plan, LRC and Classroom Building Additions, Chet F. Harritt School.

Tan, S.S., 1995, Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, San Diego County, California, Landslide Hazard Identification Map No. 33, DMG Open-File Report 95-03, Plate 33C.

United States Department of Agriculture (USDA), 1953, Flight AXN-10M, Numbers 15 and 16: dated April 14. Scale 1:24,000.

United States Department of the Interior, Bureau of Reclamation, 1989, Engineering Geology Field Manual.

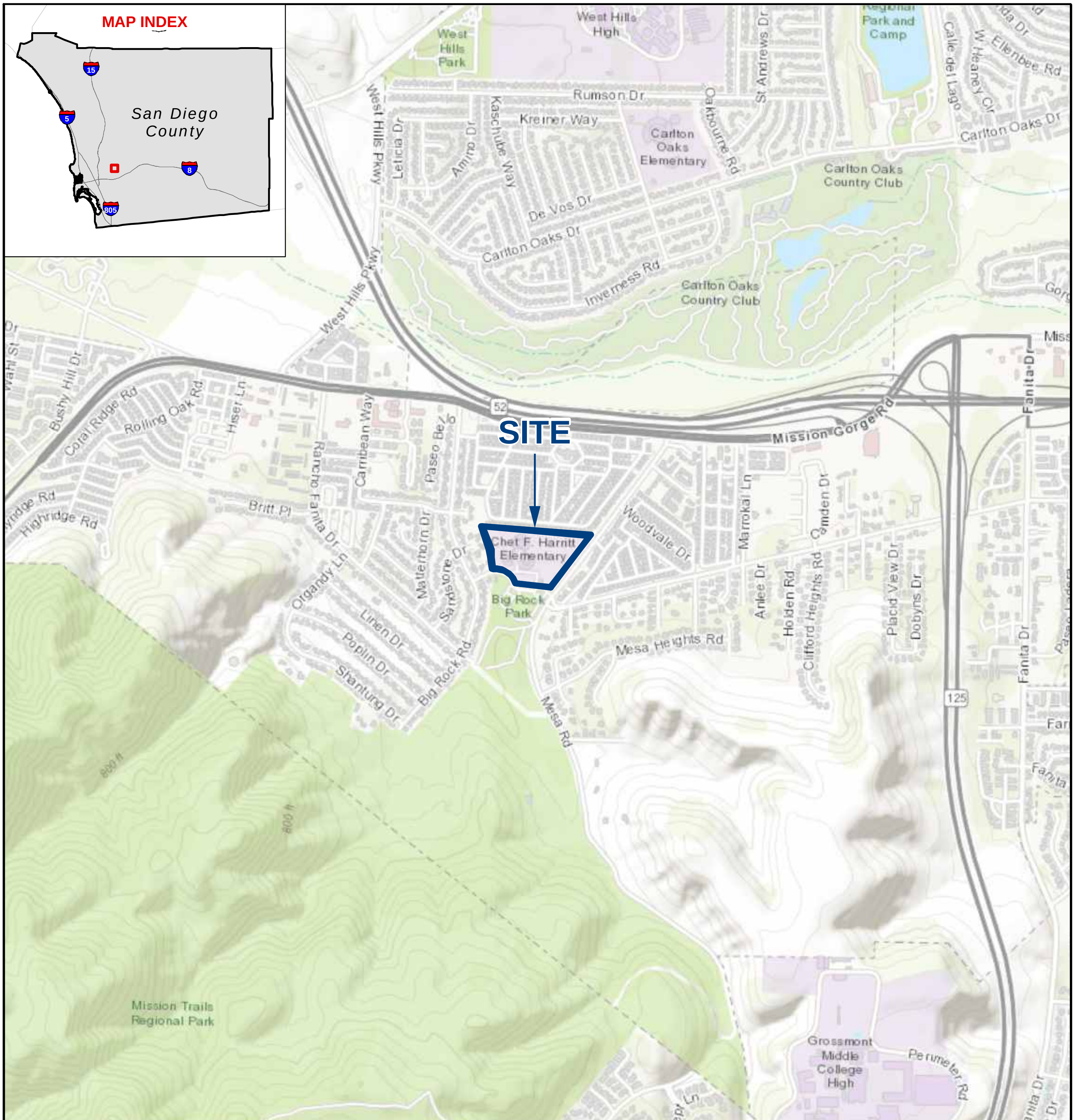
United States Federal Emergency Management Agency (FEMA), 2012, Flood Insurance Rate Map (FIRM), Map Number 06073C1632G, Panel 1634 of 2375: effective date May 16.

United States Geological Survey (USGS), 2018, Topographic Map of the La Mesa Quadrangle, California, 7.5-Minute Series: Scale 1:24,000.

United States Geological Survey (USGS), 2019, 2008 National Seismic Hazard Maps - Fault Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm: accessed April.

University of California, 2019, CalME: Caltrans Mechanistic-Empirical Tool (Version 3.0.0): dated June.

FIGURES



N

FEET

0 1,500 3,000

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

1_108774001_SL.mxd 6/27/2019 AOB

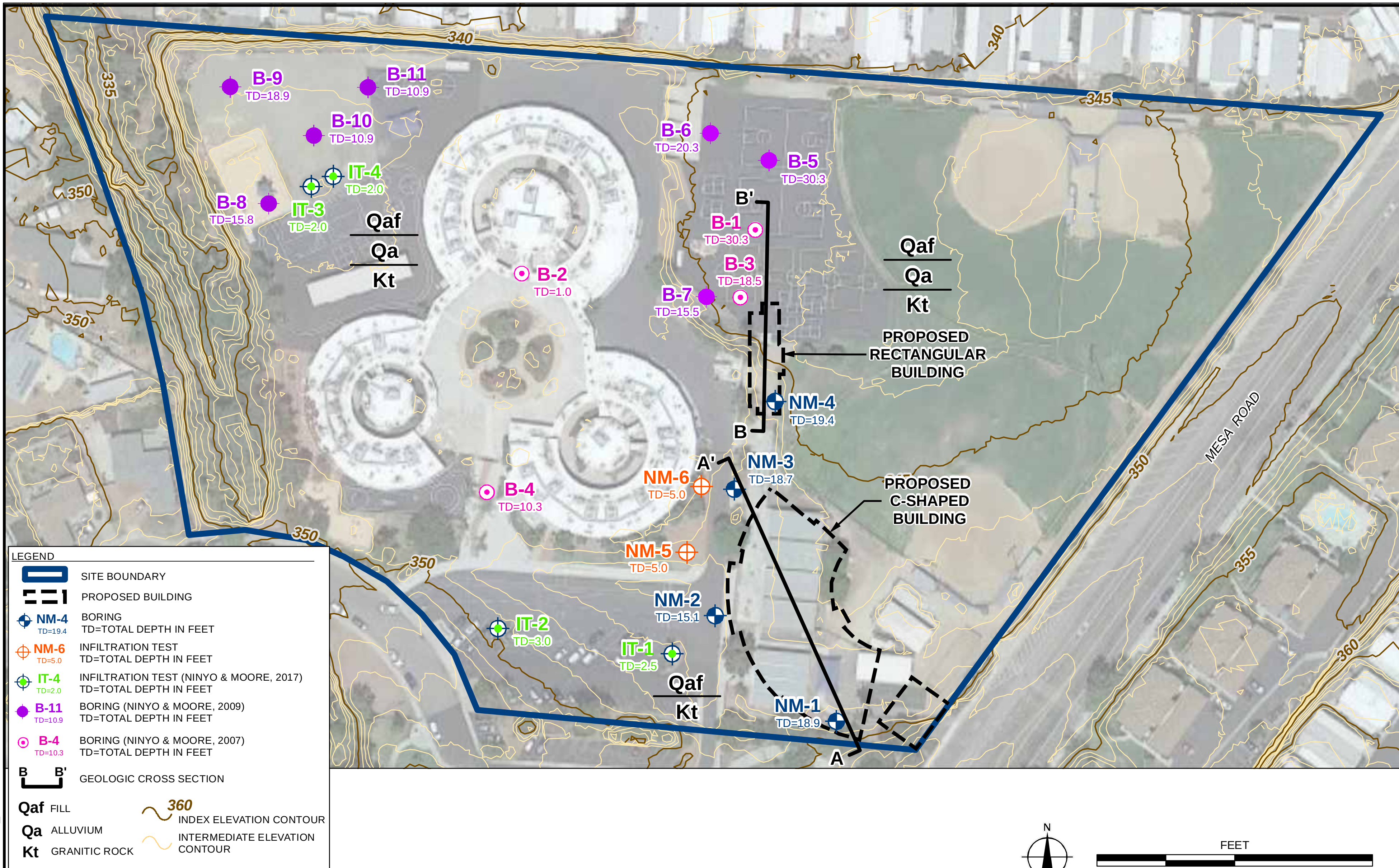
FIGURE 1

SITE LOCATION

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCES: TOPOTGRAPHIC CONTOURS - NOAA, 2019; GOOGLE EARTH, 2019

2_108774001_BL.mxd 7/10/2019 AOB

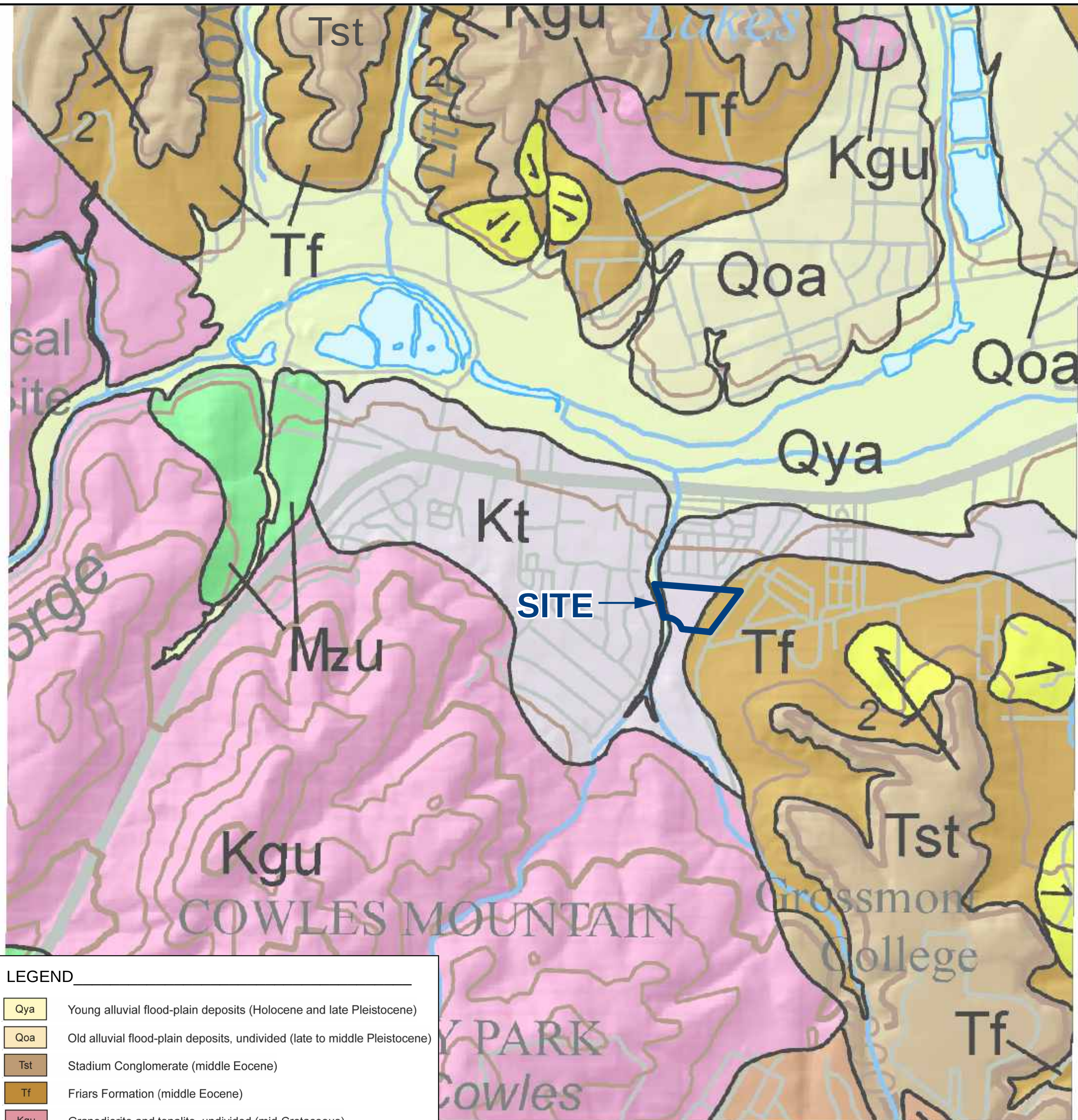
FIGURE 2

BORING LOCATIONS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108774001 | 7/19



LEGEND

Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qoa	Old alluvial flood-plain deposits, undivided (late to middle Pleistocene)
Tst	Stadium Conglomerate (middle Eocene)
Tf	Friars Formation (middle Eocene)
Kgu	Granodiorite and tonalite, undivided (mid-Cretaceous)
Kt	Tonalite, undivided (mid-Cretaceous)
Mzu	Metamorphosed and unmetamorphosed volcanic and sedimentary rocks, undivided (Mesozoic)
Qls?	Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.
70 U D	Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.
70	Strike and dip of beds

REFERENCE: KENNEDY, M.P., TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30 X 60-MINUTE QUADRANGLE, CALIFORNIA

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

3_108774001_G.mxd 7/10/2019 AOB

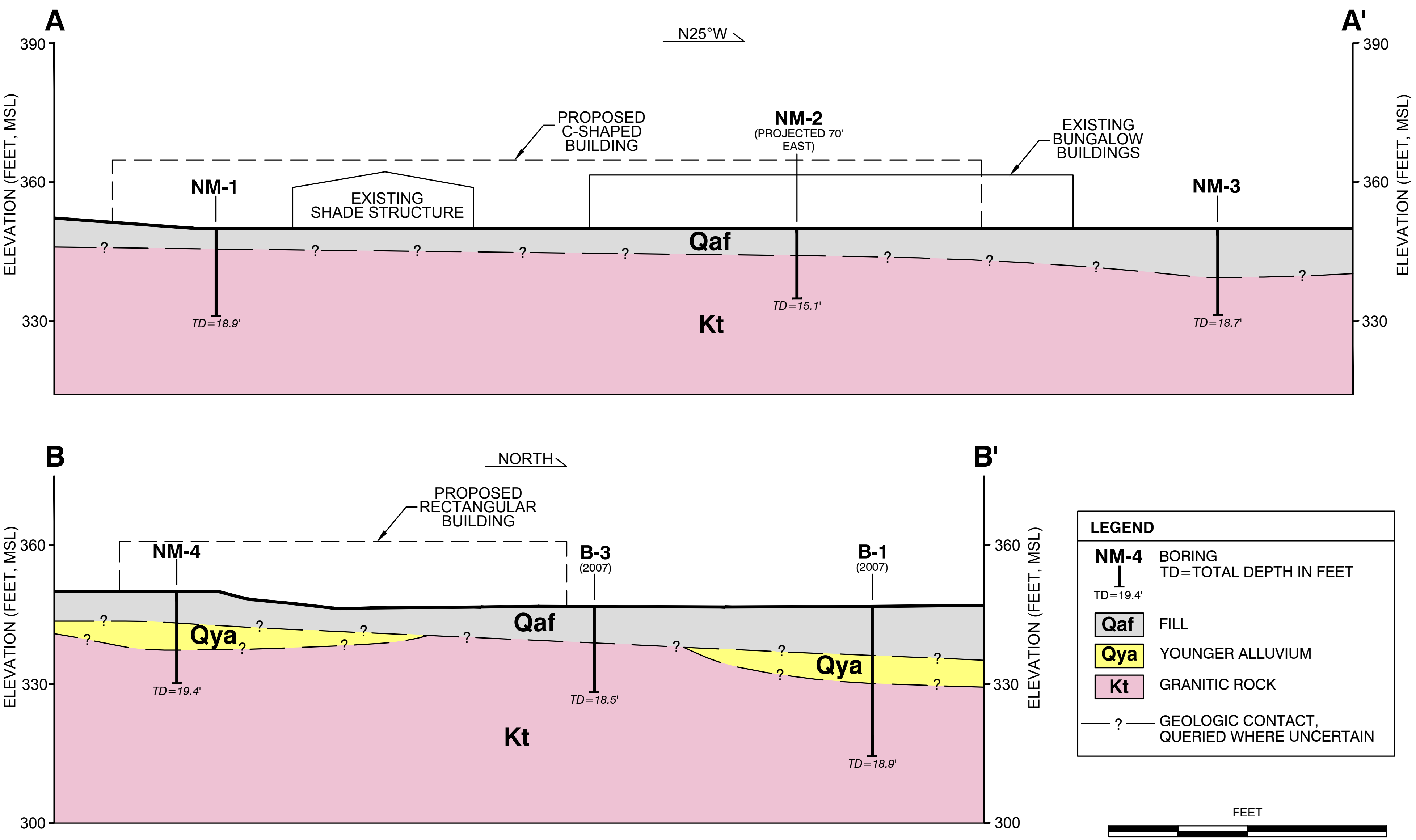
FIGURE 3

GEOLOGY

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

4 108774001 CS A-A' B-B'.DWG

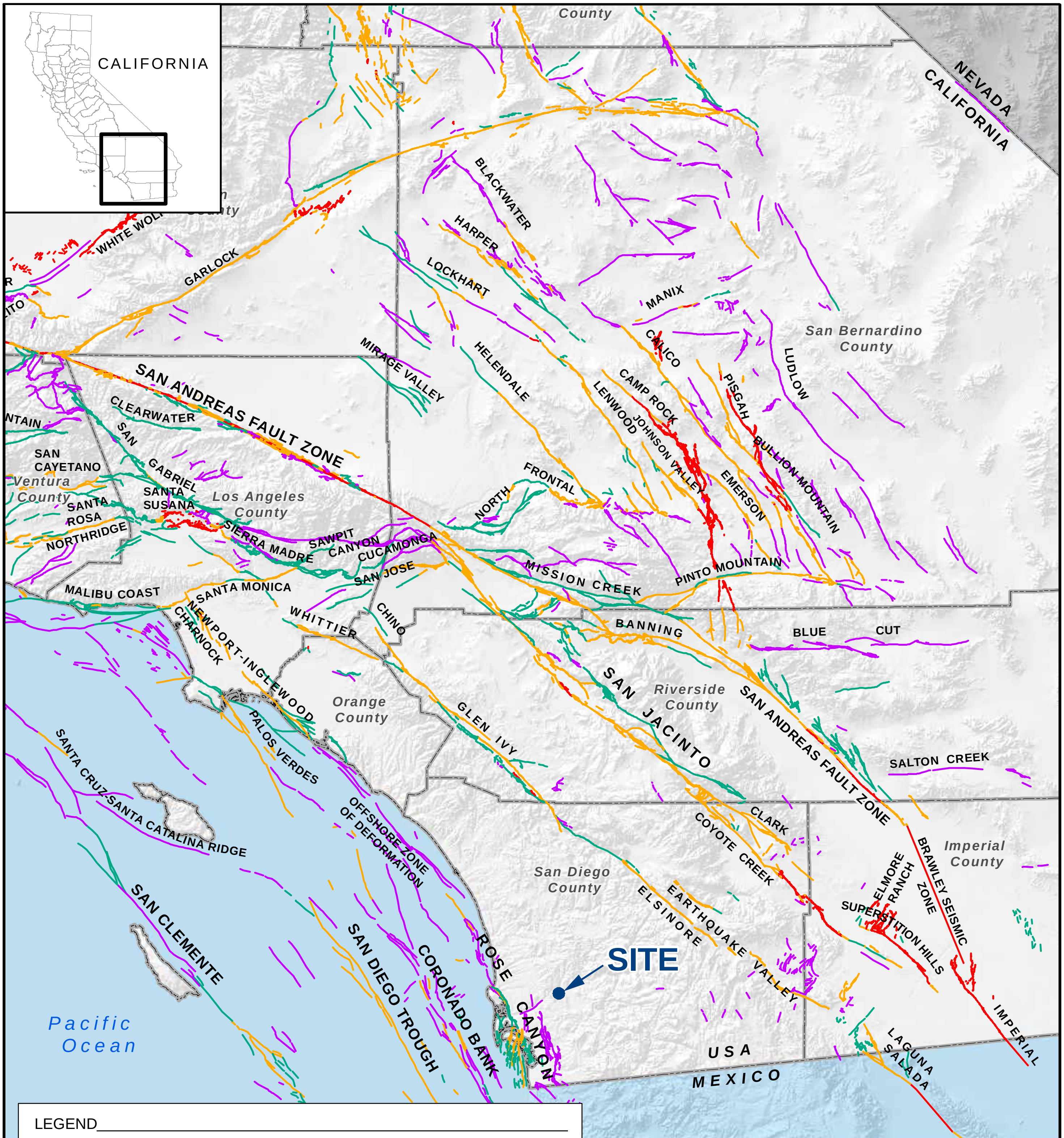
FIGURE 4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GEOLOGIC CROSS SECTIONS A-A' AND B-B'

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 I 7/19



LEGEND

CALIFORNIA FAULT ACTIVITY

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

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

5_108774001_FL.mxd 6/27/2019 AOB

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

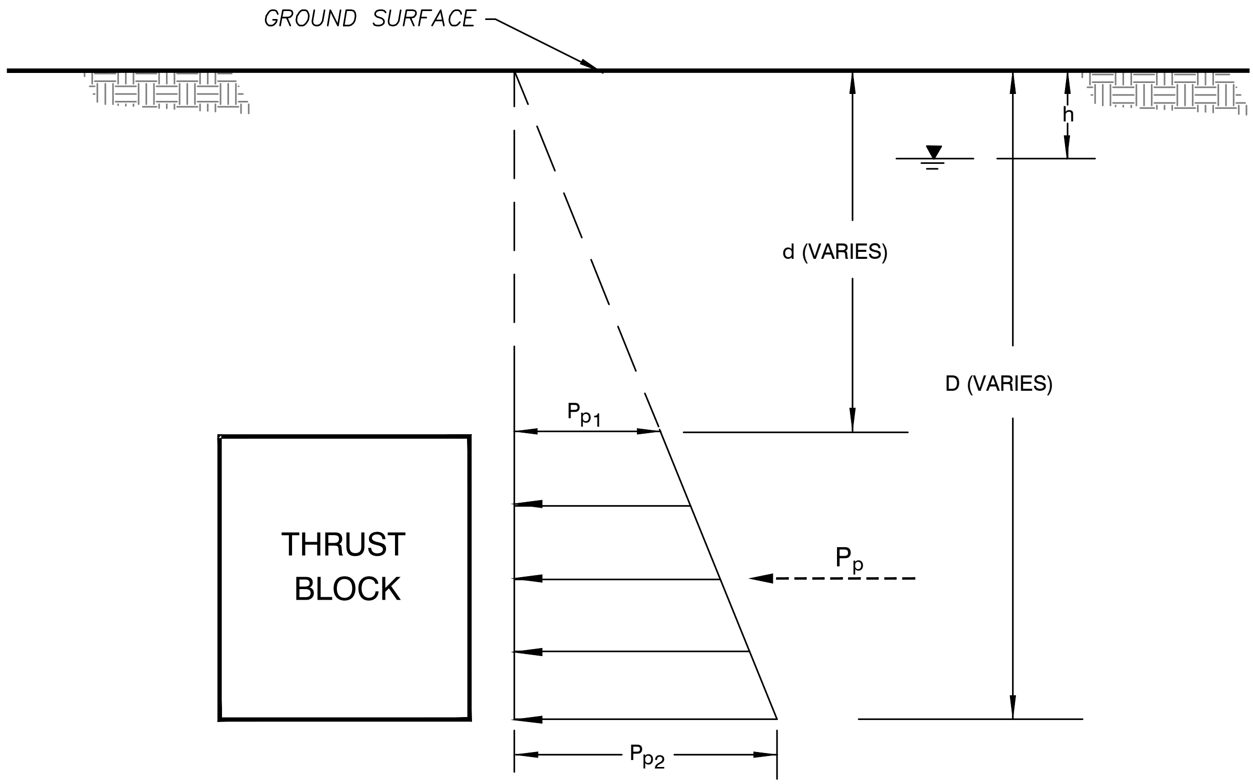
FIGURE 5

FAULT LOCATIONS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 175 \, (D^2 - d^2)$ lb/ft

2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.5 \, (D - d)[\, 124.8h + 58 \, (D + d)]$ lb/ft

3. ASSUMES BACKFILL IS GRANULAR MATERIAL

4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL

5. D, d AND h ARE IN FEET

6. GROUNDWATER TABLE

NOT TO SCALE

6 108774001 D-TB.DWG

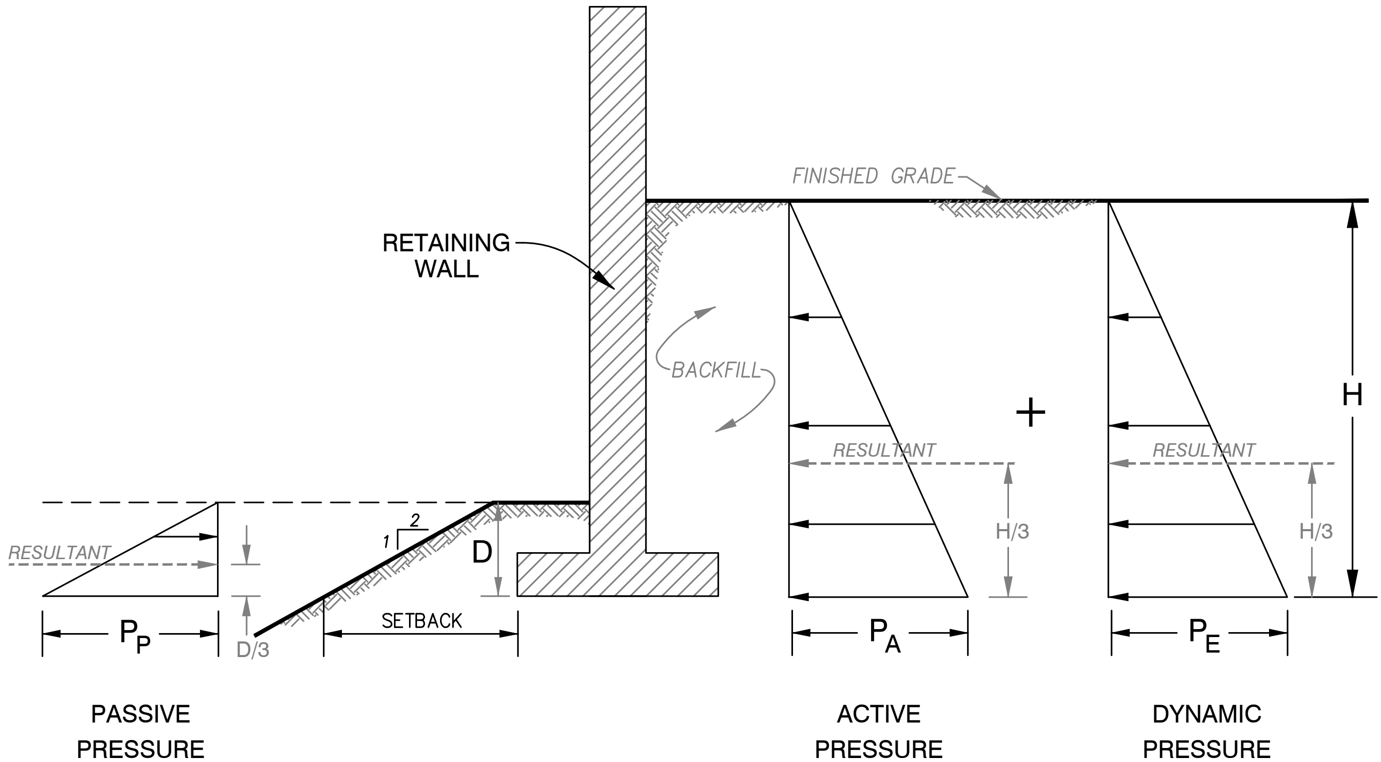
FIGURE 6

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108774001 I 7/19



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 0.40g
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.

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_A	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	36 H	56 H
P_E	16 H	
P_P	Level Ground	2H:1V Descending Ground
	350 D	150 D

NOT TO SCALE

7 108774001 D-YRW.DWG

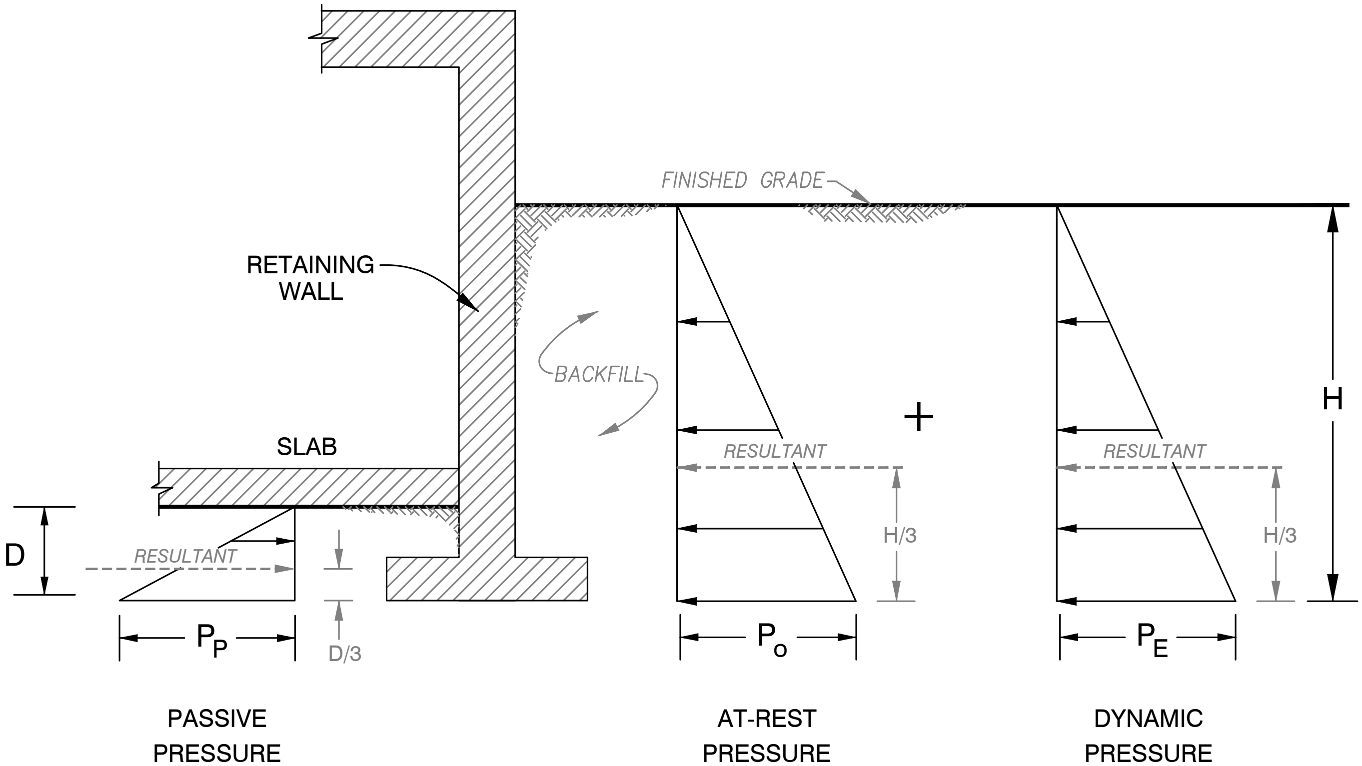
FIGURE 7

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 I 7/19



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 0.40g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)[1]	
P_O	Level Backfill with Granular Soils [2]	2H:1V Sloping Backfill with Granular Soils [2]
	56 H	81 H
P_E	16 H	
P_P	Level Ground	2H:1V Descending Ground
	350 D	150 D

NOT TO SCALE

8 108774001 D-RRW.DWG

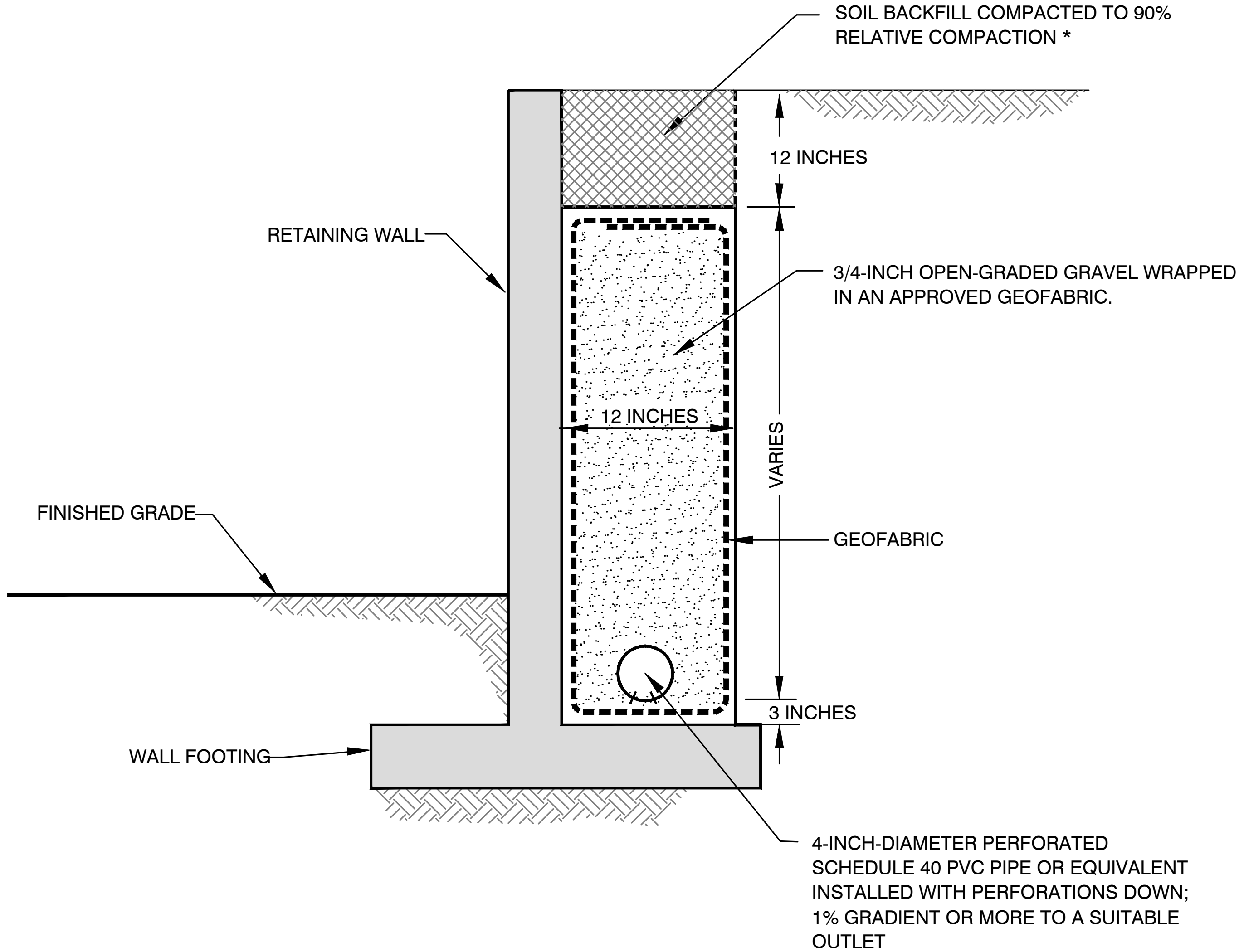
FIGURE 8

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



*BASED ON ASTM D1557

NOT TO SCALE

FIGURE 9

RETAINING WALL DRAINAGE DETAIL

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108774001 I 7/19

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

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

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

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1⅜ inches. The sampler was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

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

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

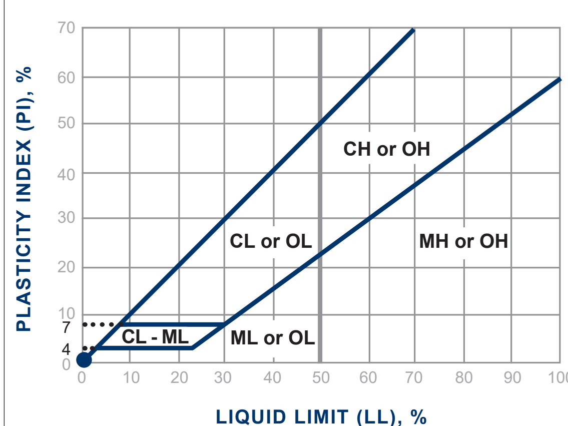
Soil Classification Chart Per ASTM D 2488

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

Grain Size

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

Plasticity Chart



Apparent Density - Coarse-Grained Soil

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

Consistency - Fine-Grained Soil

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

BORING LOG EXPLANATION SHEET

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

DATE DRILLED 4/18/19 BORING NO. NM-1

GROUND ELEVATION 351'± (MSL) SHEET 1 OF 1

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

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

SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ASPHALT CONCRETE: Approximately 3 inches thick.
								FILL: Brown, moist, medium dense, silty fine to coarse SAND; scattered gravel and cobble fragments.
			50/3"	13.5	122.4			GRANITIC ROCK: Reddish brown, moist, fine- to coarse-grained GRANITIC ROCK; highly fractured; weathered.
10			50/6"	8.6	106.5			
			50/4"					
			50/5"					
20								Total Depth = 18.9 feet. Groundwater not encountered during drilling. Backfilled with approximately 6.6 cubic feet of bentonite grout, and patched with black-dyed concrete shortly after drilling on 4/18/19. Note: Ground water, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

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

DATE DRILLED 4/18/19 BORING NO. NM-2

GROUND ELEVATION 347'± (MSL) SHEET 1 OF 1

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

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

SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT

Depth	Blows/Foot	Moisture (%)	Dry Density (PCF)	U.S.C.S.	Description/Interpretation
0					ASPHALT CONCRETE: Approximately 7 inches thick.
					BASE: (DG) Approximately 5 inches thick.
	50/4"	5.9	115.1	SM	FILL: Red and brown, moist, medium dense, silty fine to coarse SAND with clay pockets; scattered gravel; scattered cobble fragments.
10	50/3"	7.2	116.3		GRANITIC ROCK: Grayish brown, moist, fine- to coarse-grained GRANITIC ROCK; highly fractured; weathered.
	50/1"				Total Depth = 15.1 feet. (Refusal)
20					
30					
40					

Total Depth = 15.1 feet. (Refusal)
Groundwater not encountered during drilling.
Backfilled with approximately 5.3 cubic feet of bentonite grout, and patched with black-dyed concrete shortly after drilling on 4/18/19.

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

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

DATE DRILLED	4/18/19	BORING NO.	NM-3
GROUND ELEVATION	348'± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-75) (Baja)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	CMK	LOGGED BY CMK	REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								MULCH/PLANTER TOPSOIL: Approximately 6 inches thick.
	X						SC	FILL: Reddish brown, moist, medium dense, clayey fine to medium SAND; clay pockets.
	X	X	20	17.9	109.4		SM	Reddish brown, moist, medium dense, silty fine to coarse SAND.
10	X	X	78/9"	12.0	122.4			
								GRANITIC ROCK: Brown, moist, fine- to coarse-grained GRANITIC ROCK; highly fractured; weathered.
		X	50/3"	5.7	121.1			
		X	50/5"					
		X	50/2"	8.2	101.4			
20								Total Depth = 18.7 feet. Groundwater not encountered during drilling. Backfilled with approximately 6.5 cubic feet of bentonite grout shortly after drilling on 4/18/19. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

DATE DRILLED	4/18/19	BORING NO.	NM-4
GROUND ELEVATION	350'± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-75) (Baja)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	CMK	LOGGED BY CMK	REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	GRASS/PLANTER TOPSOIL: Approximately 3 inches thick.
								FILL: Brown, moist, medium dense, silty fine to coarse SAND; scattered gravel and cobble fragments.
			33	13.2	116.8			
							CL	YOUNGER ALLUVIUM: Dark brown, moist, stiff, sandy lean CLAY; few gravel; scattered cobble fragments.
10			23	18.6	110.3			
								@ 13': Hard drilling.
								GRANITIC ROCK: Brown, moist, fine- to coarse-grained GRANITIC ROCK; highly fractured; weathered.
			50/3"					
			95/10"					
20								Total Depth = 19.4 feet. Groundwater not encountered during drilling. Backfilled with approximately 6.8 cubic feet of bentonite grout shortly after drilling on 4/18/19. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 4/18/19 BORING NO. NM-5 GROUND ELEVATION 349'± (MSL) SHEET 1 OF 1 METHOD OF DRILLING Hand Auger DRIVE WEIGHT N/A DROP N/A SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT **DESCRIPTION/INTERPRETATION**
0								MULCH/PLANTER TOPSOIL: Approximately 6 inches thick.
							SM	FILL: Brown, moist, medium dense, silty fine to medium SAND.
								Total Depth = 5 feet. Groundwater not encountered during drilling. Infiltration test set on 4/18/19. Backfilled after infiltration testing on 4/19/19. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
20								
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 4/18/19 BORING NO. NM-6 GROUND ELEVATION 349'± (MSL) SHEET 1 OF 1 METHOD OF DRILLING Hand Auger DRIVE WEIGHT N/A DROP N/A SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT **DESCRIPTION/INTERPRETATION**
0								MULCH/PLANTER TOPSOIL: Approximately 6 inches thick.
							SM	FILL: Reddish brown, moist, medium dense, silty fine to coarse SAND with clay.
								Total Depth = 5 feet. Groundwater not encountered during drilling. Infiltration test set on 4/18/19. Backfilled after infiltration testing on 4/19/19. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
20								
30								
40								

DATE DRILLED 6/25/07 BORING NO. B-1

GROUND ELEVATION Existing Grade SHEET 1 OF 2

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

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

SAMPLED BY BTM LOGGED BY BTM REVIEWED BY RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							SM	FILL: Light brown, damp, medium dense, slightly clayey, silty SAND; micaceous.
			17					
5			26	14.8	105.8			Moist.
10			14	11.3	115.4			
							CL	ALLUVIUM: Light to dark brown, damp, very stiff, sandy silty CLAY; scattered gravel.
				▼				Saturated.
15			50/5"					
								GRANITIC ROCK: Light to dark gray, weathered, GRANITIC ROCK; highly fractured.
20								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-1

DATE DRILLED	6/25/07	BORING NO.	B-1
GROUND ELEVATION	Existing Grade	SHEET	2 OF 2
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)		
DRIVE WEIGHT	140 lbs. (Auto-Trip Hammer)	DROP	30"
SAMPLED BY	BTM	LOGGED BY	BTM
REVIEWED BY	RI		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		■	50/6"					GRANITIC ROCK: (Continued) Light to dark gray, weathered, GRANITIC ROCK; highly fractured.
25		■	50/4"					
30			50/3"					Total Depth = 30.3 feet (Refusal). Groundwater was measured at a depth of approximately 13.0 feet in the borehole at about 10 minutes after completion of drilling. 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 approximately 11 cubic feet of bentonite shortly after drilling on 6/25/07.
35								
40								

DATE DRILLED	6/26/07	BORING NO.	B-2
GROUND ELEVATION	Existing Grade	SHEET	1 OF 1
METHOD OF DRILLING	6" Diameter Coring Machine (Manual)		
DRIVE WEIGHT	N/A	DROP	N/A
SAMPLED BY	DLP	LOGGED BY	DLP
REVIEWED BY	RI		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
	X		10.0				SM	FILL: Brown, moist, medium dense, silty SAND; trace gravel.
								Total Depth = 1.0 feet (Refusal on rock). Groundwater not encountered. 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 concrete shortly after drilling on 6/26/07.
5								
10								
15								
20								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-3

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							SM	FILL: Light brown, damp, medium dense, clayey silty SAND; micaceous.
			21					
5			27	10.8	115.2			Damp to moist.
								GRANITIC ROCK: Light brown, weathered, GRANITIC ROCK; highly fractured.
10			92/8"					
15			50/4"					
			50/6"					
20								Total Depth = 18.5 feet (Refusal). Groundwater not encountered. 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 approximately 6.0 cubic feet of bentonite shortly after drilling on 6/25/07.

DATE DRILLED 6/25/07 BORING NO. B-3

GROUND ELEVATION Existing Grade SHEET 1 OF 1

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

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

SAMPLED BY BTM LOGGED BY BTM REVIEWED BY RI

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-4

DATE DRILLED 6/25/07	BORING NO. B-4	
GROUND ELEVATION Existing Grade	SHEET 1 OF 1	
METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)		
DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer)	DROP 30"	
SAMPLED BY BTM	LOGGED BY BTM	REVIEWED BY RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							SM	FILL: Light brown, damp, medium dense, silty SAND.
			53					GRANITIC ROCK: Light brown, weathered, GRANITIC ROCK; highly fractured.
5			90/8"					
10			50/4"					Total Depth = 10.3 feet (Refusal). Groundwater not encountered. 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 approximately 4 cubic feet of bentonite shortly after drilling on 6/25/07.
15								
20								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-6

DATE DRILLED	1/19/09	BORING NO.	B-5
GROUND ELEVATION	345' ± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (A-300) (Scott's Drilling)		
DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"
SAMPLED BY	BTM	LOGGED BY	BTM
REVIEWED BY	RI		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 6 inches thick.
	Bulk					SM	FILL: Brown, damp to moist, medium dense, clayey, silty, fine to medium SAND.
	Driven	28					Fine to coarse sand; trace clay; scattered dark brown and reddish brown mottled.
5						ML	Dark brown, damp, medium dense, fine to medium sandy SILT.
	Driven	48				SC	Dark grayish brown and yellowish brown (mottled), damp, medium dense, gravelly, clayey SAND; gravel approximately 1 inch in diameter.
	Driven	95					Brown and gray (mottled). Gravel layer from approximately 10 to 11 feet; difficult drilling.
10						CL	Dark brown and dark gray (mottled), damp, hard, fine to coarse sandy CLAY; scattered roothairs.
	Driven	45	▼				
15	Driven	47					GRANITIC ROCK: Brown and grayish brown, damp to moist, fine- to medium-grained decomposed GRANITIC ROCK; (resembled soil- clayey fine SAND); scattered caliche-filled high-angle fractures; micaceous.
	Driven	50/3"					Reddish brown to yellowish brown; damp; weathered; (looks less like soil).
20							

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-7

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20								<u>GRANITIC ROCK:</u> (Continued) Reddish brown to yellowish brown; damp, fine- to medium-grained, weathered GRANITIC ROCK; micaceous.
								Difficult drilling.
25		◤	50/2"	▽				Light and dark gray; scattered; fresh; hard.
30		◤	50/3"					Same as above.
								Total Depth = 30.3 feet. Groundwater encountered at approximately 25 feet during drilling. Groundwater measured at approximately 12.5 feet approximately 3 hours after drilling. Backfilled with approximately 8.8 cubic feet of bentonite grout shortly after drilling on 1/19/09. <u>Note:</u> Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
35								
40								

DATE DRILLED 1/19/09 BORING NO. B-5

GROUND ELEVATION 345' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (A-300) (Scott's Drilling)

DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"

SAMPLED BY BTM LOGGED BY BTM REVIEWED BY RI

Ninyo & Moore

BORING LOG		
CHET HARRITT SCHOOL SANTEE, CALIFORNIA		
PROJECT NO.	DATE	FIGURE
106115001	3/09	A-8

DATE DRILLED	1/19/09	BORING NO.	B-6	
GROUND ELEVATION	345' ± (MSL)	SHEET	1	OF 2
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (A-300) (Scott's Drilling)			
DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"	
SAMPLED BY	BTM	LOGGED BY	BTM	REVIEWED BY RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 6 inches thick.
	X						SC	FILL: Brown, damp to moist, medium dense, clayey fine to medium SAND.
	X							Dark bluish gray; scattered gravel up to approximately 1 inch.
5		X	54					
10		X	54					Brown; micaceous.
								GRANITIC ROCK: Brown and gray, damp, fine- to medium-grained, weathered GRANITIC ROCK; micaceous.
15		X	87/9"					Light gray with reddish brown staining.
				▼				
								Difficult drilling.
20								

![Ninyo & Moore]

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-9

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
								DATE DRILLED 1/19/09 BORING NO. B-6
								GROUND ELEVATION 345' ± (MSL) SHEET 2 OF 2
								METHOD OF DRILLING 8" Diameter Hollow Stem Auger (A-300) (Scott's Drilling)
								DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"
								SAMPLED BY BTM LOGGED BY BTM REVIEWED BY RI
20			50/3"					GRANITIC ROCK: (Continued) Light gray, damp, fine to medium-grained, weathered GRANITIC ROCK; scattered reddish brown staining.
								Total Depth = 20.3 feet. Groundwater not encountered during drilling. Groundwater measured at approximately 14 feet approximately 2 hours after drilling. Backfilled with approximately 5.9 cubic feet of bentonite grout shortly after drilling on 1/19/09.
								Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
25								
30								
35								
40								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-10

DATE DRILLED	1/19/09	BORING NO.	B-7
GROUND ELEVATION	345' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (A-300) (Scott's Drilling)		
DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"
SAMPLED BY	BTM	LOGGED BY	BTM
REVIEWED BY	RI		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 6 inches thick.
	Bulk					SC	FILL: Brown, damp, medium dense, clayey fine to medium SAND; scattered gravel.
							Gravelly from approximately 4 to 5 feet.
5	Bulk / Driven	15				CL	Dark brownish gray, damp, stiff, fine to coarse sandy CLAY; micaceous; few gravel.
							GRANITIC ROCK: Reddish brown, damp to moist, fine- to medium-grained, weathered to decomposed GRANITIC ROCK; micaceous.
10	Bulk	77					
							Difficult drilling.
							Less weathered.
15	Driven	50/6"					Moderately hard to hard.
							Total Depth = 15.5 feet. Groundwater not encountered during drilling. Backfilled with bentonite grout on 1/19/09.
							Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
20							

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-11

DATE DRILLED	2/21/09
BORING NO.	B-8
GROUND ELEVATION	348' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-55) (Scott's Drilling)
DRIVE WEIGHT	140 lbs. (Auto-Trip Hammer)
DROP	30"
SAMPLED BY	CAT
LOGGED BY	CAT
REVIEWED BY	RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 5 inches thick.
	X						SW	FILL: Grayish brown, damp, medium dense, silty gravelly fine to coarse well-graded SAND.
	X						CL	YOUNGER ALLUVIUM: Reddish brown, damp, hard, fine to medium sandy CLAY.
5		X	50/5"	12.8	94.7			GRANITIC ROCK: Reddish brown and dark gray, moist, fine- to medium-grained GRANITIC ROCK.
10		X	50/6"					Dark brown and dark gray; fine-grained.
								Difficult drilling.
15		X	83/9"	♀				Wet.
								Total Depth = 15.8 feet. Seepage encountered at approximately 15 feet during drilling. Backfilled with approximately 4.5 cubic feet of bentonite grout and patched with concrete immediately after drilling on 2/21/09. Note: Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage. Please refer to the report for groundwater monitoring recommendations.
20								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-12

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown, damp, medium dense, silty fine to medium SAND.
							SW	Dark reddish brown, damp, medium dense, silty, clayey, gravelly fine to coarse well-graded SAND.
5			28					On rock.
			10					Yellowish brown and dark brown (mottled).
							ML	YOUNGER ALLUVIUM: Dark brown, damp, medium dense, fine to medium sandy SILT; trace clay. Gravelly from approximately 8.5 to 9 feet.
							CL	OLDER ALLUVIUM: Dark reddish brown, damp, hard, silty CLAY; trace fine gravel and coarse sand.
10			82					
								Difficult drilling; gravel and cobbles.
15			50/6"	12.5	110.2			GRANITIC ROCK: Dark gray and brown, moist, fine- to medium-grained GRANITIC ROCK.
			50/5"					Damp to moist.
								Total Depth = 18.9 feet. Groundwater not encountered during drilling. Backfilled with approximately 5.5 cubic feet of bentonite grout shortly
20								

DATE DRILLED 2/21/09 BORING NO. B-9

GROUND ELEVATION 348' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-55) (Scott's Drilling)

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

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY RI

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-13

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/21/09 BORING NO. B-9 GROUND ELEVATION 348' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-55) (Scott's Drilling) DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30" SAMPLED BY CAT LOGGED BY CAT REVIEWED BY RI DESCRIPTION/INTERPRETATION
20								after drilling on 2/21/09. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
25								
30								
35								
40								

Ninyo & Moore

BORING LOG		
CHET HARRITT SCHOOL SANTEE, CALIFORNIA		
PROJECT NO.	DATE	FIGURE
106115001	3/09	A-14

DATE DRILLED	2/21/09	BORING NO.	B-10
GROUND ELEVATION	348' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-55) (Scott's Drilling)		
DRIVE WEIGHT	140 lbs. (Auto-Trip Hammer)	DROP	30"
SAMPLED BY CAT	LOGGED BY CAT	REVIEWED BY	RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown, damp, medium dense silty fine to coarse SAND.
							SC	Dark reddish brown, damp, medium dense, clayey fine to medium SAND; scattered gravel.
							ML	YOUNGER ALLUVIUM: Dark brown, damp, medium dense, fine sandy, clayey SILT.
5								Coarse gravel layer.
			94/10"	8.6	111.2			GRANITIC ROCK: Dark gray and reddish brown, damp, fine- to medium-grained GRANITIC ROCK.
10			82/11"					
								Total Depth = 10.9 feet. Groundwater not encountered during drilling. Backfilled with approximately 3 cubic feet of bentonite grout shortly after drilling on 2/21/09. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15								
20								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-15

DATE DRILLED: 2/21/09	BORING NO.: B-11	
GROUND ELEVATION: 348' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING: 8" Diameter Hollow Stem Auger (CME-55) (Scott's Drilling)		
DRIVE WEIGHT: 140 lbs. (Auto-Trip Hammer)	DROP: 30"	
SAMPLED BY: CAT	LOGGED BY: CAT	REVIEWED BY: RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						SM	FILL: Grayish brown, moist to wet, medium dense silty fine to coarse SAND; trace fine gravel.
								GRANITIC ROCK: Dark gray and white, damp, fine-grained GRANITIC ROCK; scattered reddish brown staining.
5		X	50/6"	8.4	112.7			
10		X	82/11"					
								Total Depth = 10.9 feet. Groundwater not encountered during drilling. Backfilled with approximately 3 cubic feet of bentonite grout shortly after drilling on 2/21/09. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15								
20								

Ninyo & Moore

BORING LOG

CHET HARRITT SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106115001	3/09	A-16

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-4 from our recent evaluation and Figures B-1 (2009) through B-4 (2009) from our previous evaluations (Ninyo & Moore, 2007 and 2009). 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, 2009) 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-5 (2009) from our previous evaluation (Ninyo & Moore, 2009).

Consolidation
Consolidation tests were performed on selected relatively undisturbed soil samples from our recent evaluation in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-5 and B-6.

Direct Shear Test
Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-7 and B-8 from our recent evaluation and on Figures B-6 (2009) through B-8 (2009) from our previous evaluation (Ninyo & Moore, 2009).

Expansion Index Test
The expansion index of selected material was 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 these tests are presented on Figure B-9 from our recent evaluation and on Figure B-9 (209) from our previous evaluations (Ninyo & Moore, 2007 and 2009).

Proctor Density Test

The maximum dry density and optimum moisture content of a selected representative soil sample were evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of this test are summarized on Figure B-10 (2009) from our previous evaluations (Ninyo & Moore, 2007 and 2009).

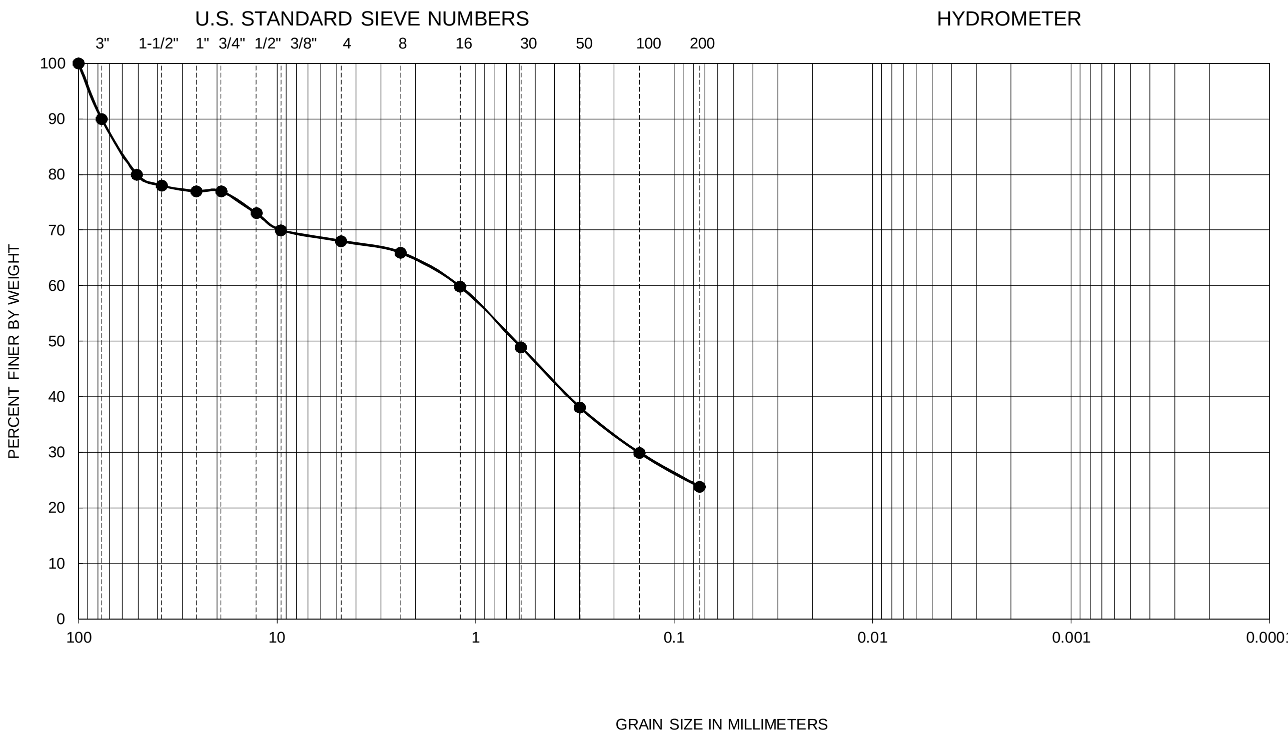
Soil Corrosivity Tests

Soil pH and electrical resistivity tests were performed on representative samples 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-10 from our recent evaluation and on Figure B-11 (2009) from our previous evaluations (Ninyo & Moore, 2007 and 2009).

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-11.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NM-2	1.0-5.0	--	--	--	--	--	--	--	--	24	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

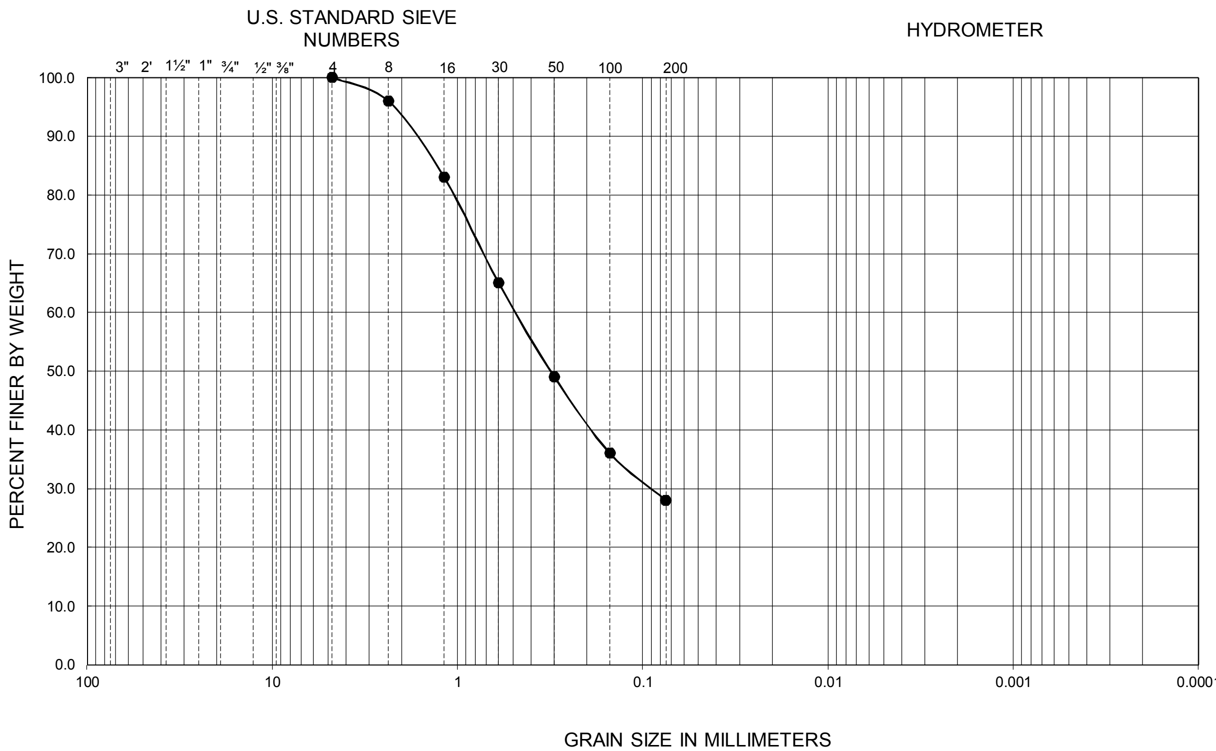
GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SAN DIEGO, CALIFORNIA

108774001 | 7/19

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NM-4	0.3-5.0	--	--	--	--	--	--	--	--	28	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

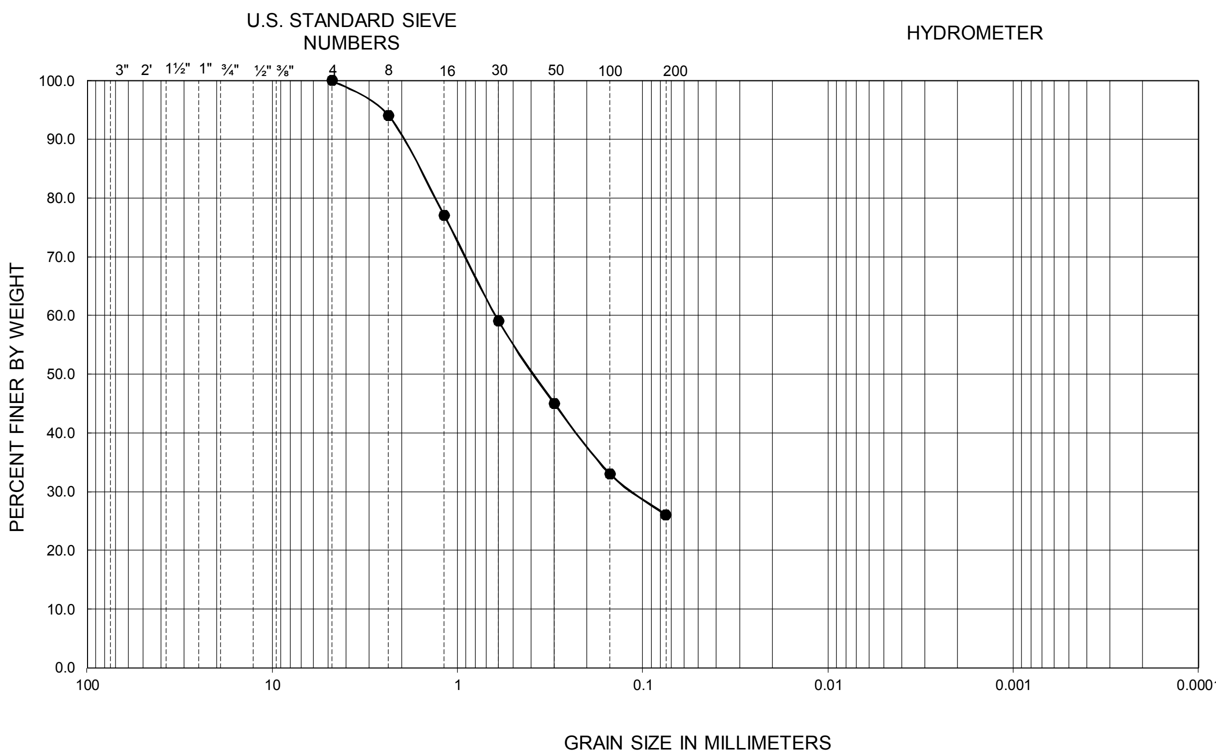
GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NM-5	0.5-5.0	--	--	--	--	--	--	--	--	26	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

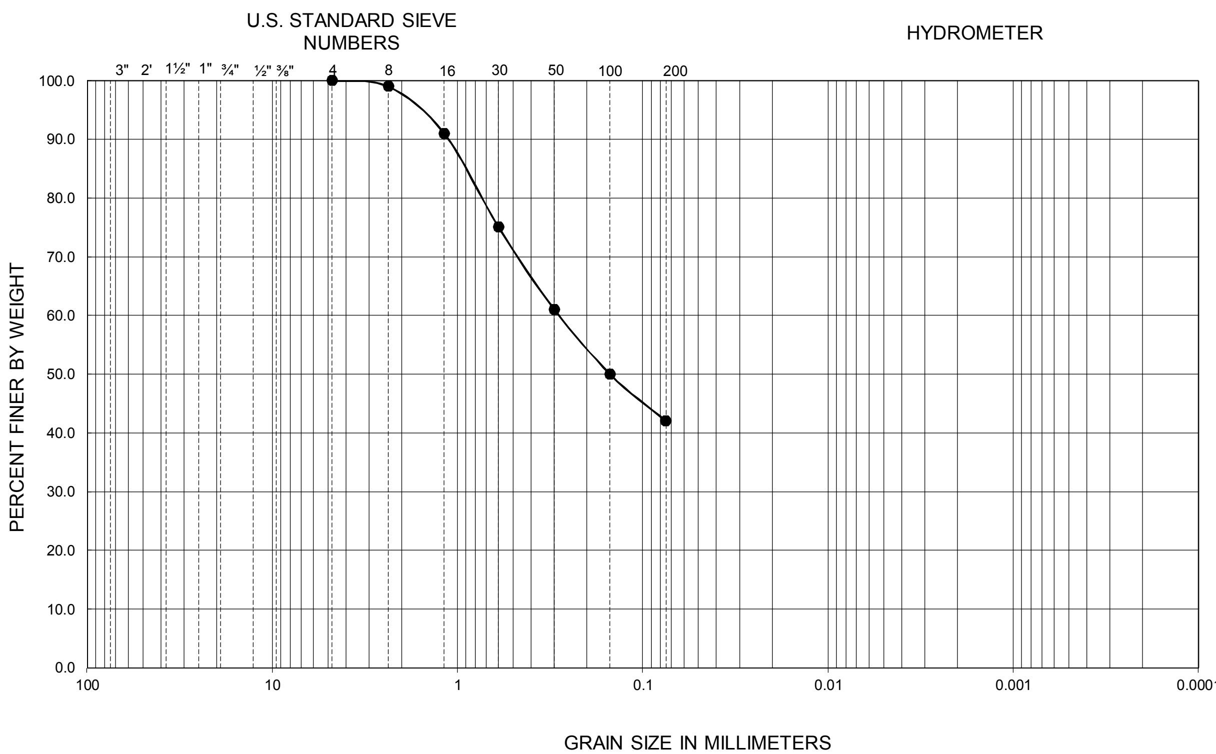
GRADATION TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NM-6	0.5-5.0	--	--	--	--	--	--	--	--	42	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

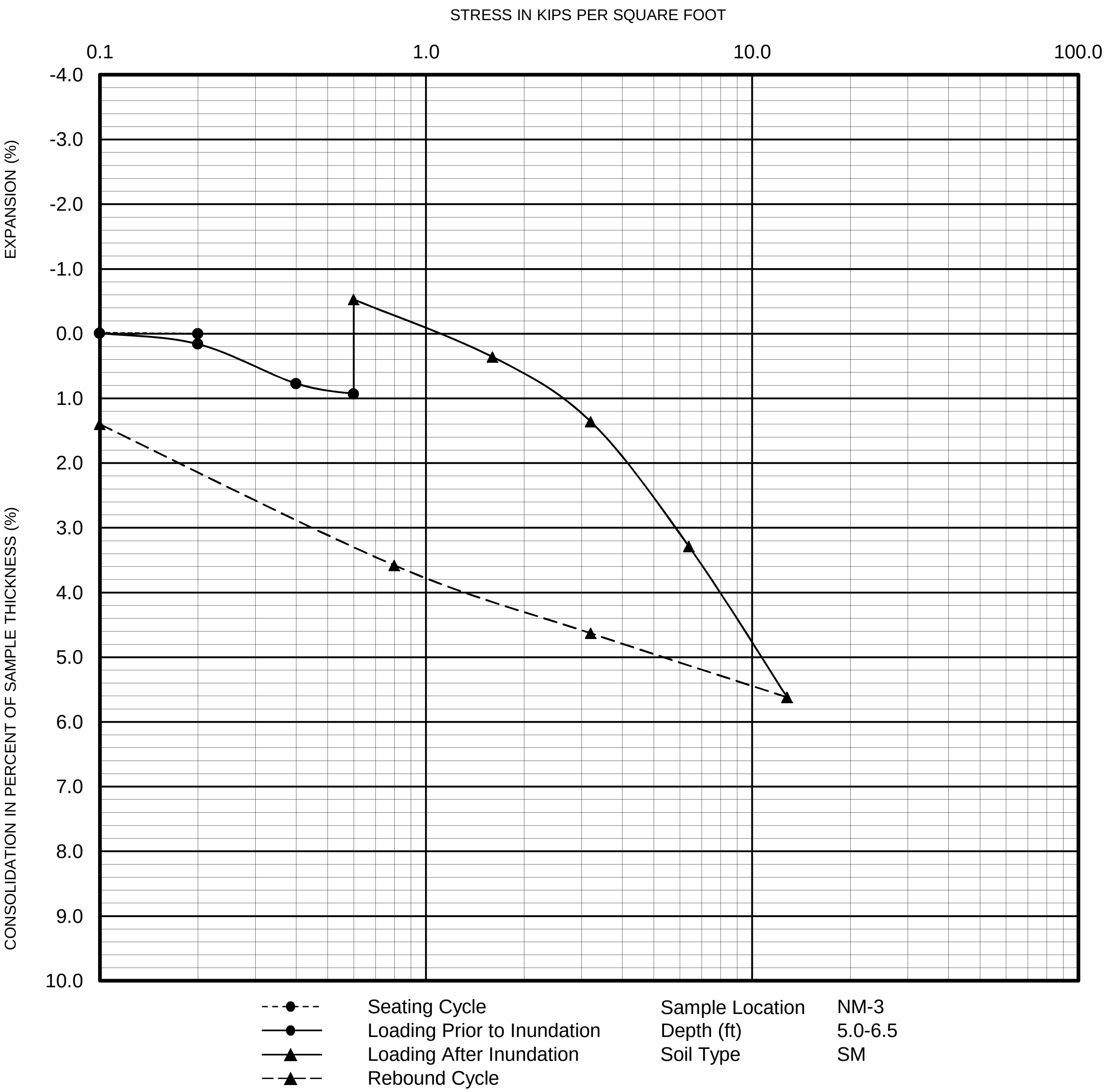
FIGURE B-4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRADATION TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



Sample Location	NM-3
Depth (ft)	5.0-6.5
Soil Type	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

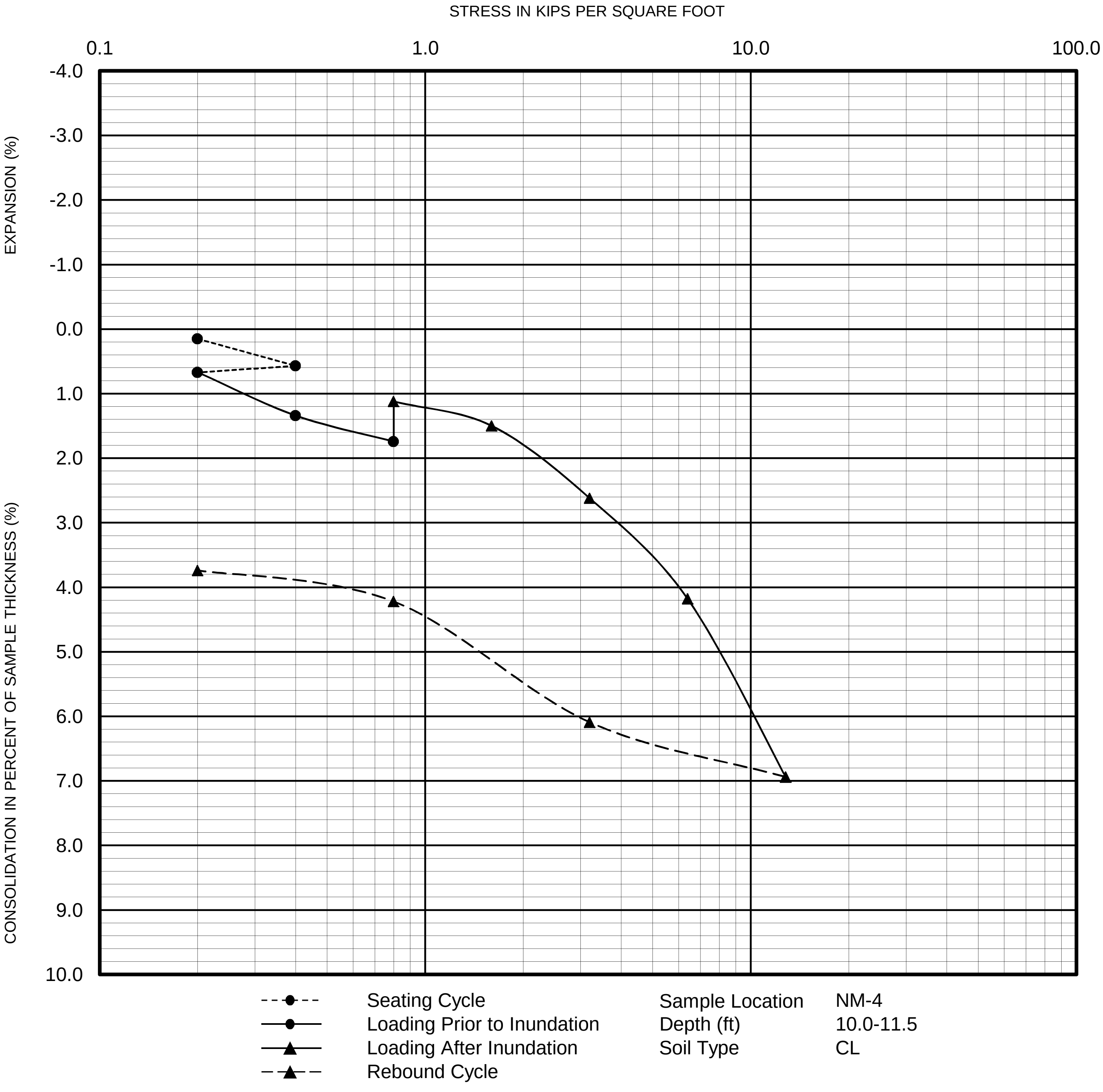
FIGURE B-5

CONSOLIDATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



Sample Location	NM-4
Depth (ft)	10.0-11.5
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

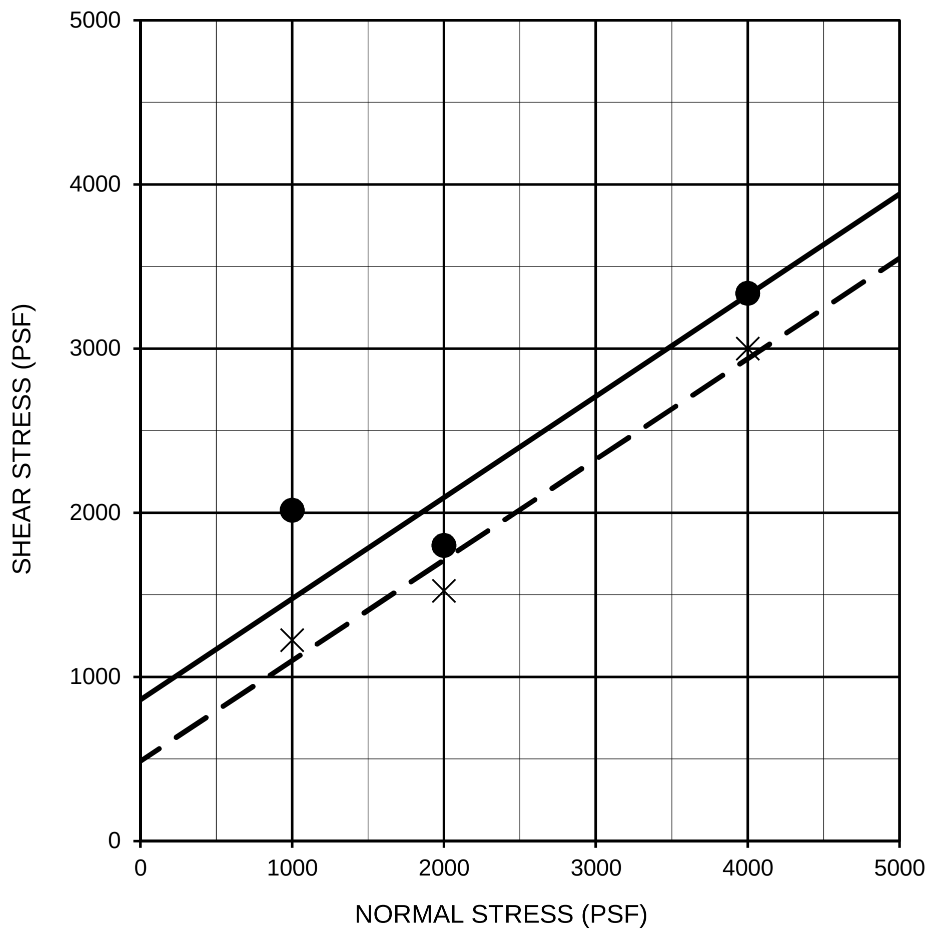
FIGURE B-6

CONSOLIDATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
GRANITIC ROCK	——●——	NM-1	5.0-5.8	Peak	860	32	Bedrock
GRANITIC ROCK	– – X – –	NM-1	5.0-5.8	Ultimate	490	32	Bedrock

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

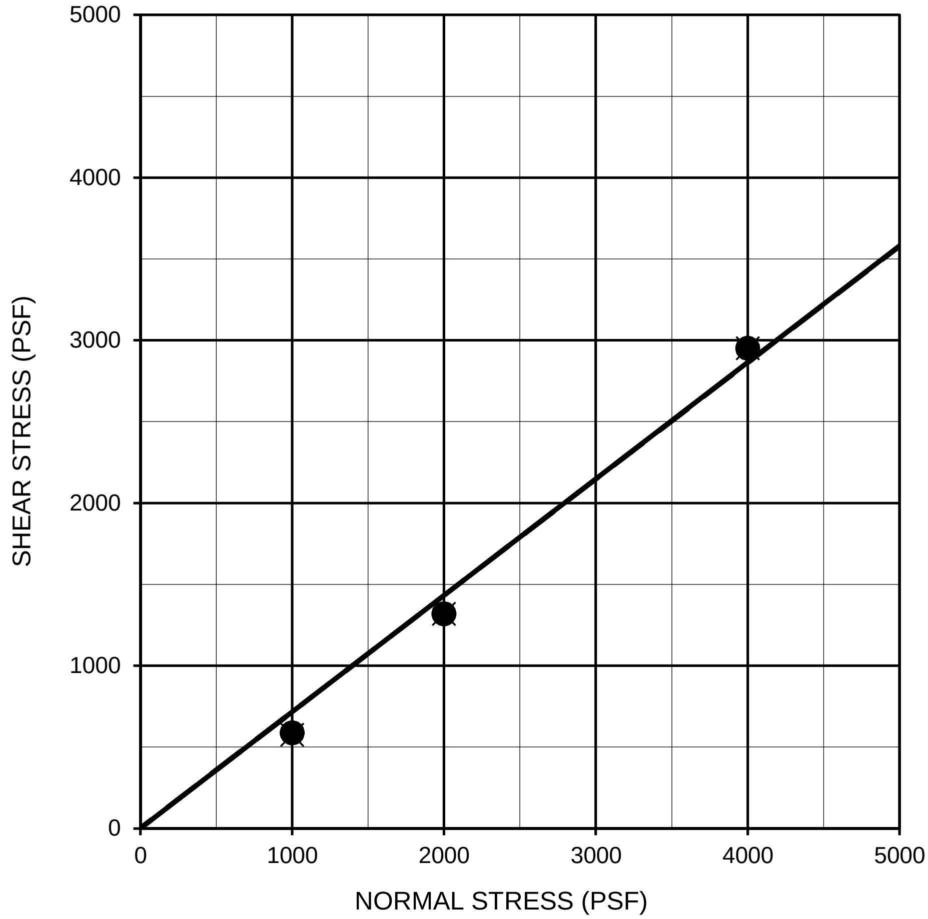
FIGURE B-7

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Silty SAND	——●——	NM-4	5.0-6.5	Peak	0	36	SM
Silty SAND	– – X – –	NM-4	5.0-6.5	Ultimate	0	36	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-8

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
NM-3	0.5-5.0	8.5	115.2	19.7	0.019	19	Very Low

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

FIGURE B-9

EXPANSION INDEX TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

108774001 | 7/19

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
NM-3	0.5-5.0	7.2	1,500	10	0.001	210
NM-4	0.3-5.0	7.0	1,400	20	0.002	230

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

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CORROSIVITY TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
NM-3	0.5-5.0	Clayey SAND (SC)	19

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-11

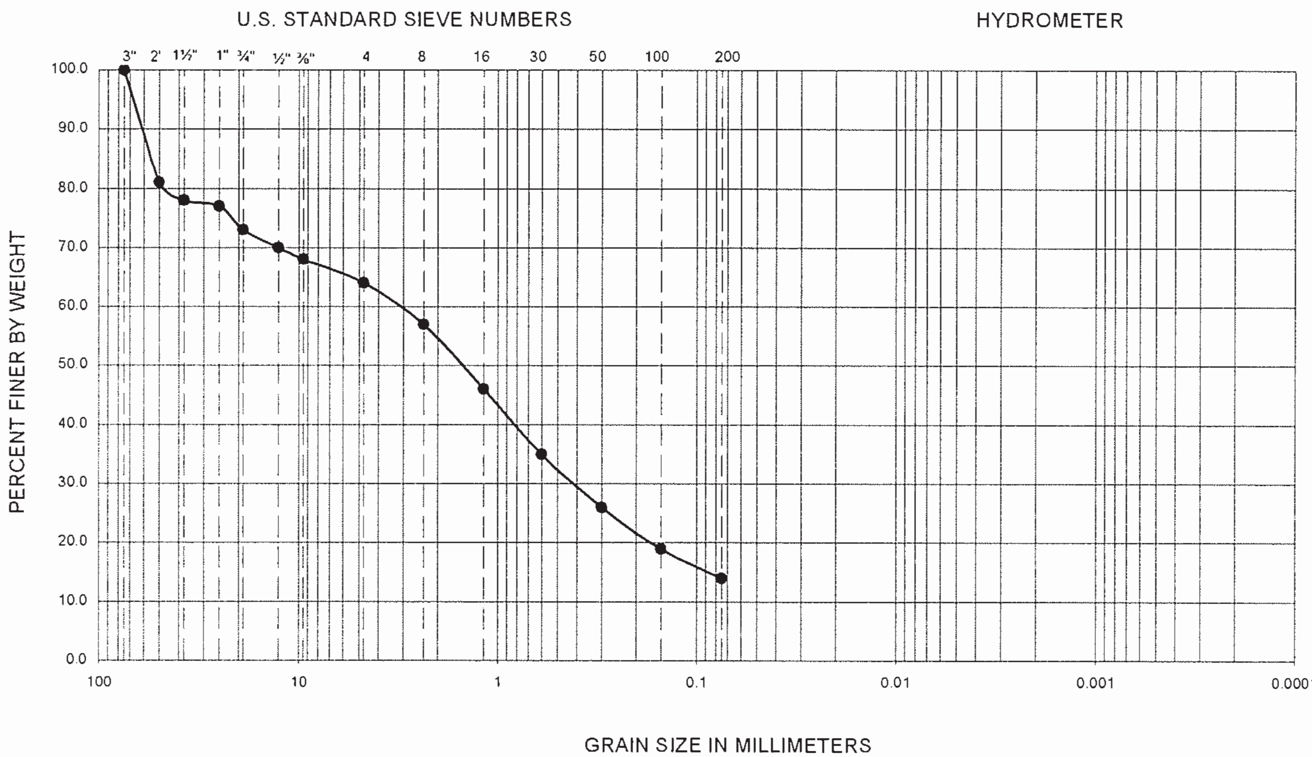
R-VALUE TEST RESULTS

CHET F. HARRITT SCHOOL BUILDING ADDITIONS
8120 ARLETTE STREET, SANTEE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108774001 | 7/19

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

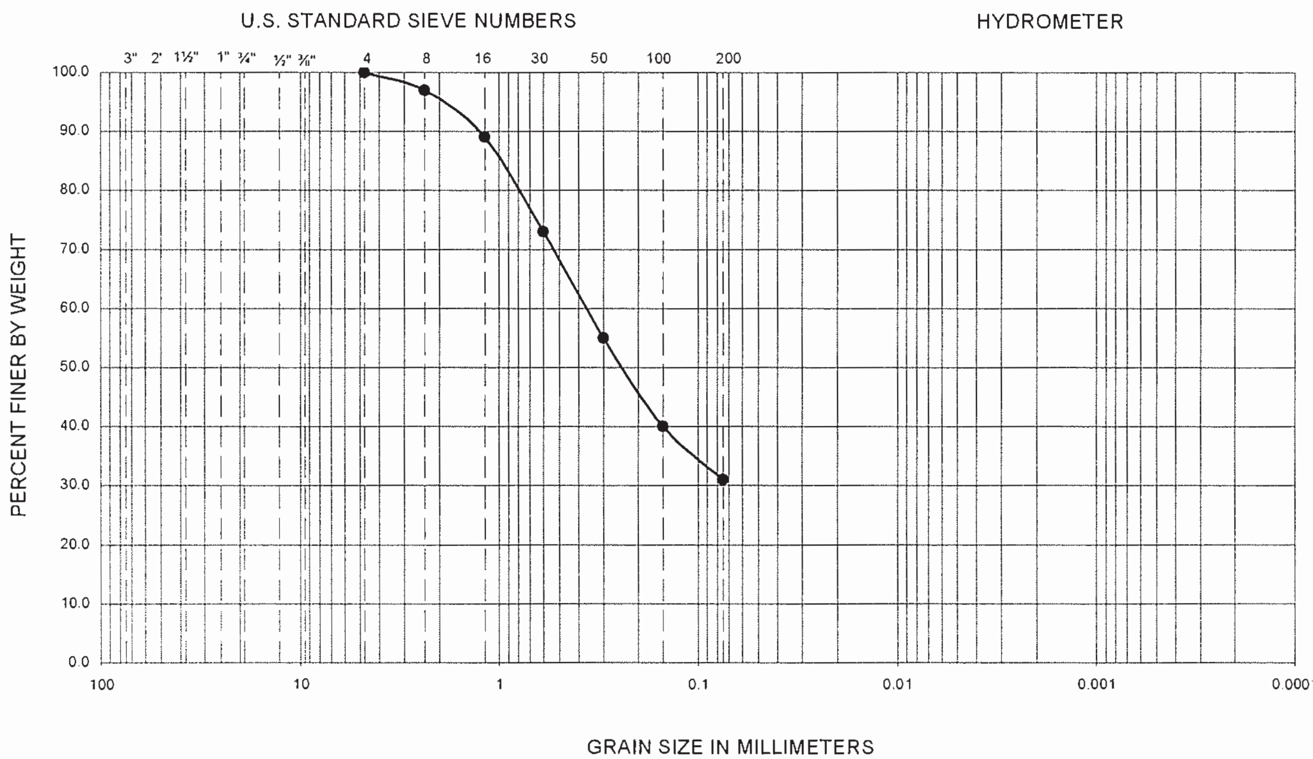


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-3	5.0-6.5	--	--	--	--	--	--	--	--	14	SM

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

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-1
106115001	3/09		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



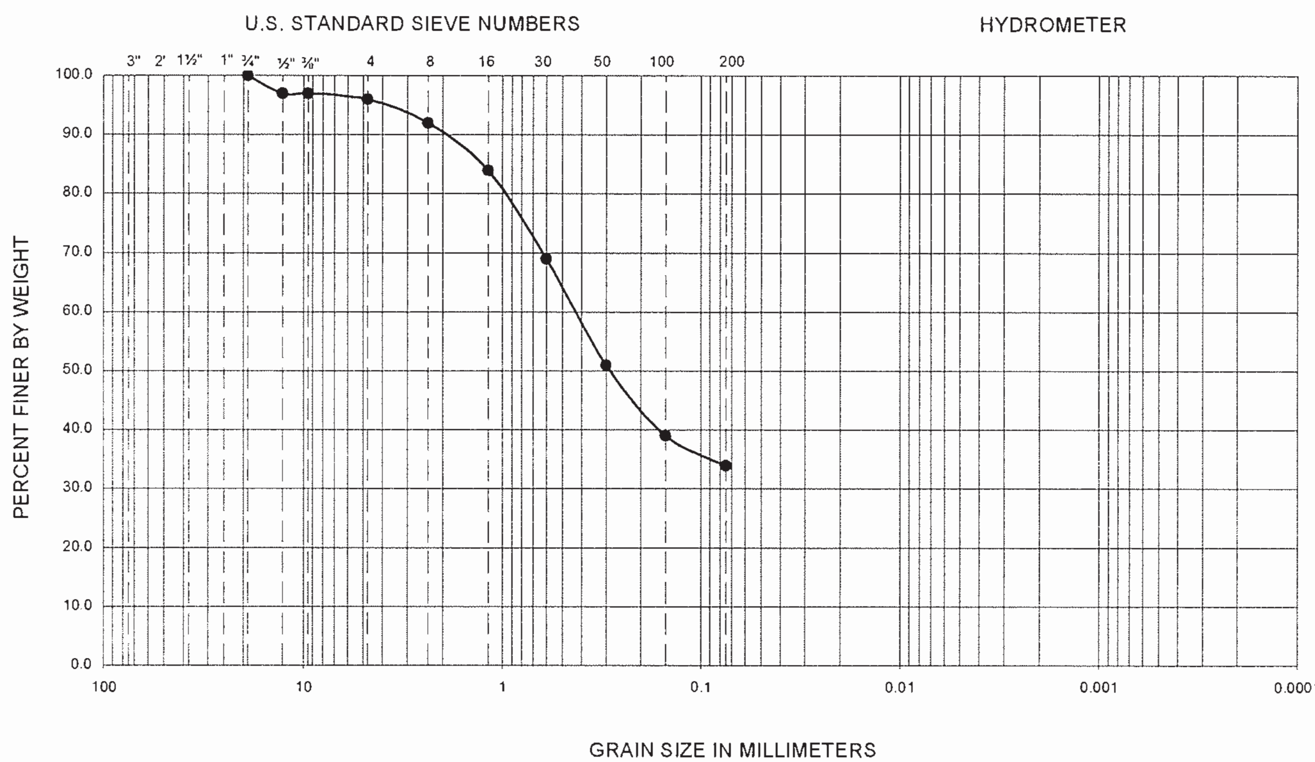
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-5	0.5-3.0	--	--	--	--	--	--	--	--	31	SM

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

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-2
106115001	3/09		

106115001 SIEVE B-5 @ 0.5-3.0.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



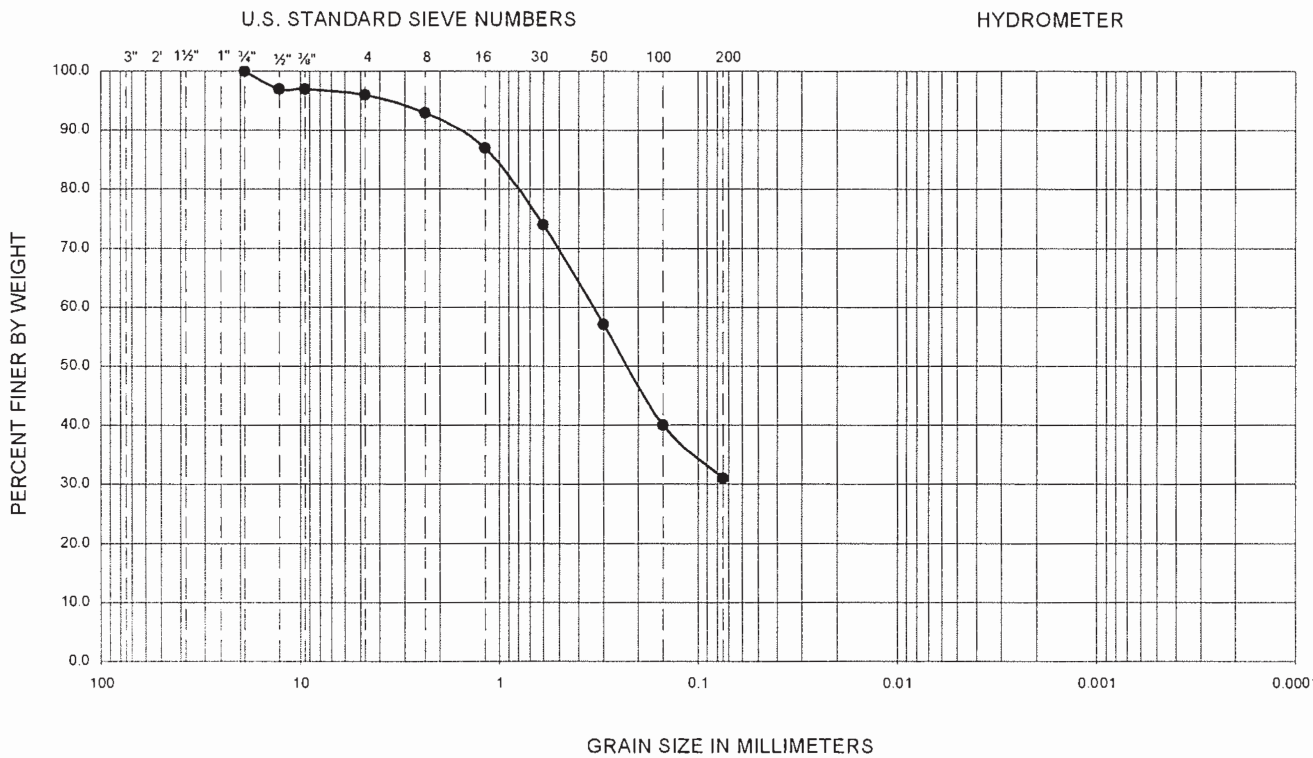
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-7	1.0-4.0	--	--	--	--	--	--	--	--	34	SC

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

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-3
106115001	3/09		

106115001 SIEVE B-7 @ 1.0-4.0.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-9	0.0-2.0	--	--	--	--	--	--	--	--	31	SM

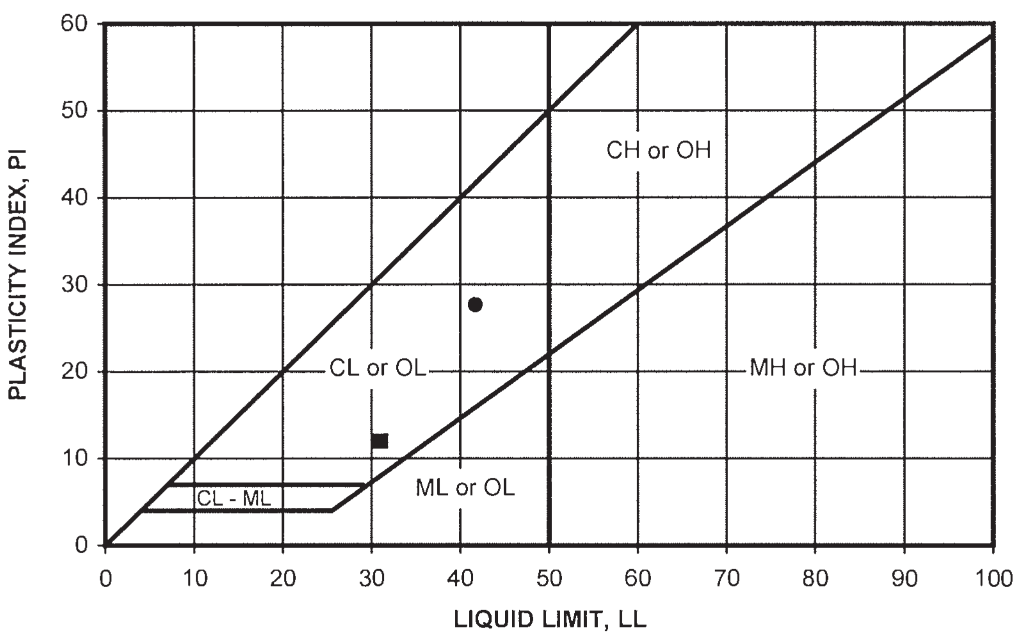
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE		
106115001 / 3/09	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-4

106115001 SIEVE B-9 @ 0.0-2.0.xls

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-5	12.0-13.5	42	14	28	CL	CL
■	B-9	10.0-11.5	31	19	12	CL	CL

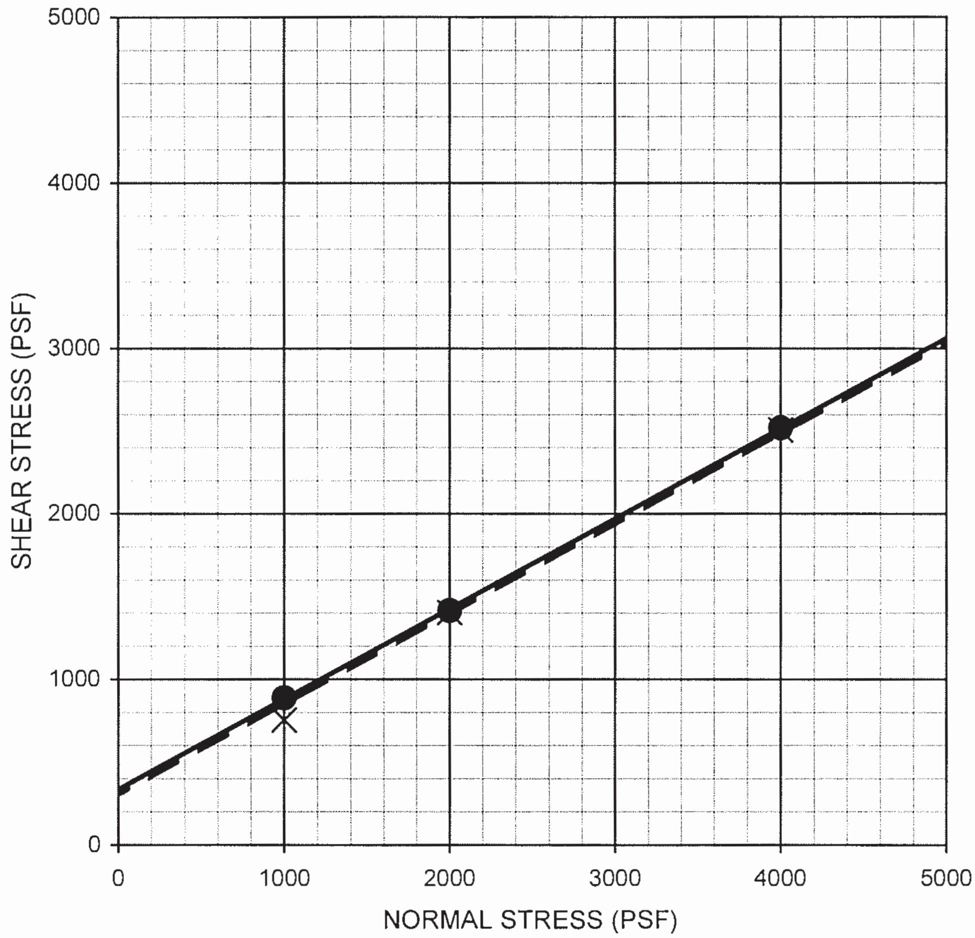
NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-05

Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-5
106115001	3/09		

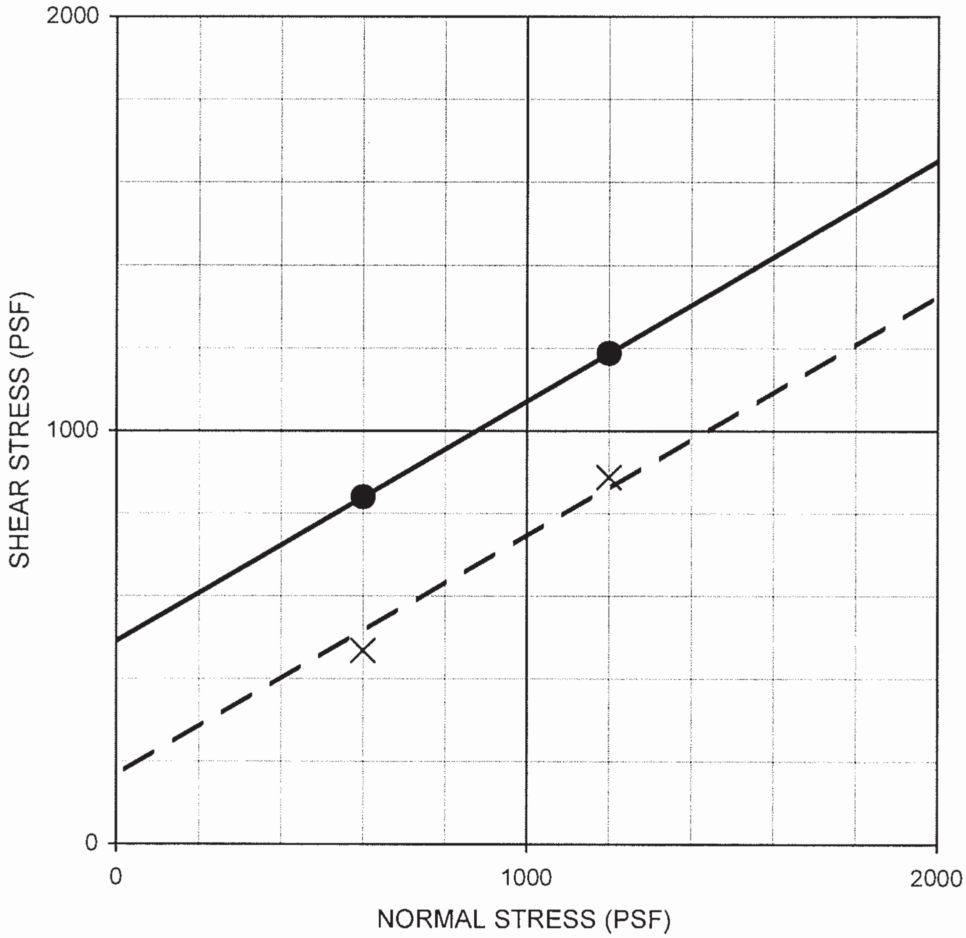
106115001 ATTERBERG.xls



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, φ (degrees)	Soil Type
Remolded @ 90% Relative Compaction	—●—	B-4	0.3-5.0	Peak	340	29	SM
	- - x - -	B-4	0.3-5.0	Ultimate	300	29	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

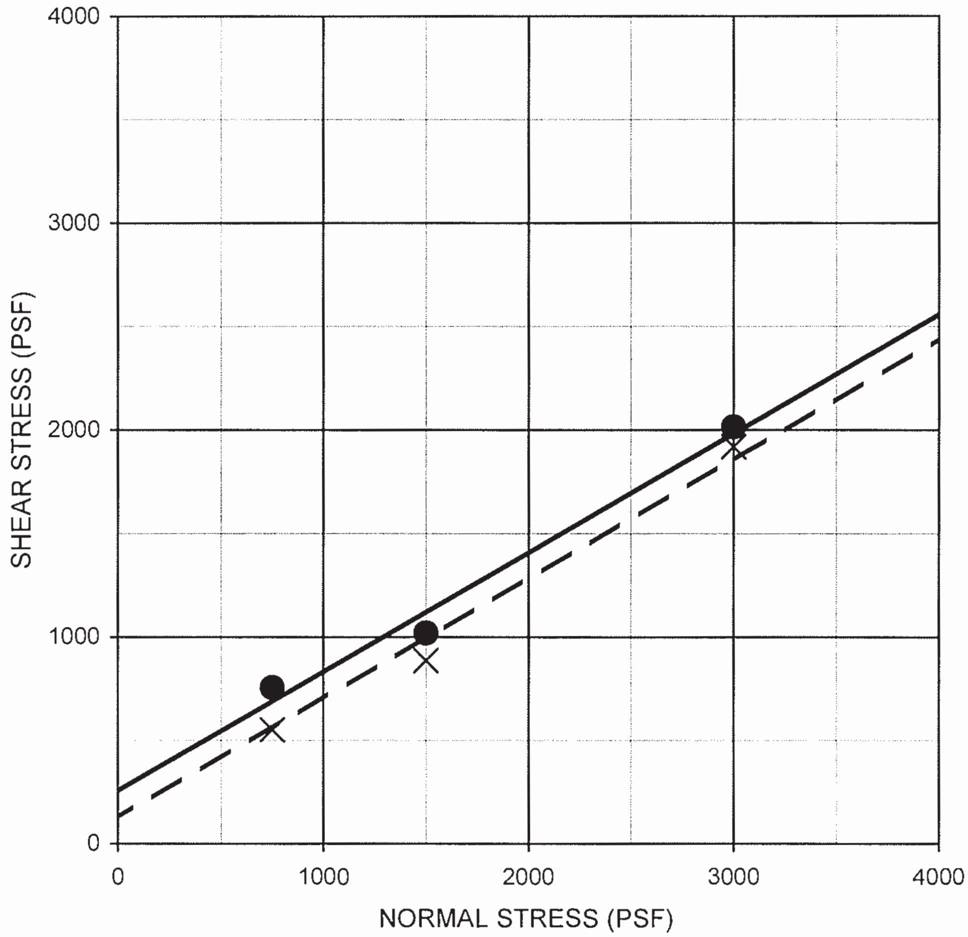
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-6
106115001	3/09		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Clayey SAND	——●——	B-5	6.5-7.5	Peak	490	30	SC
Clayey SAND	- - X - -	B-5	6.5-7.5	Ultimate	170	30	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-7
106115001	3/09		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Clayey SAND	—●—	B-5	12.0-13.5	Peak	260	30	SC
Clayey SAND	- - X - -	B-5	12.0-13.5	Ultimate	130	30	SC

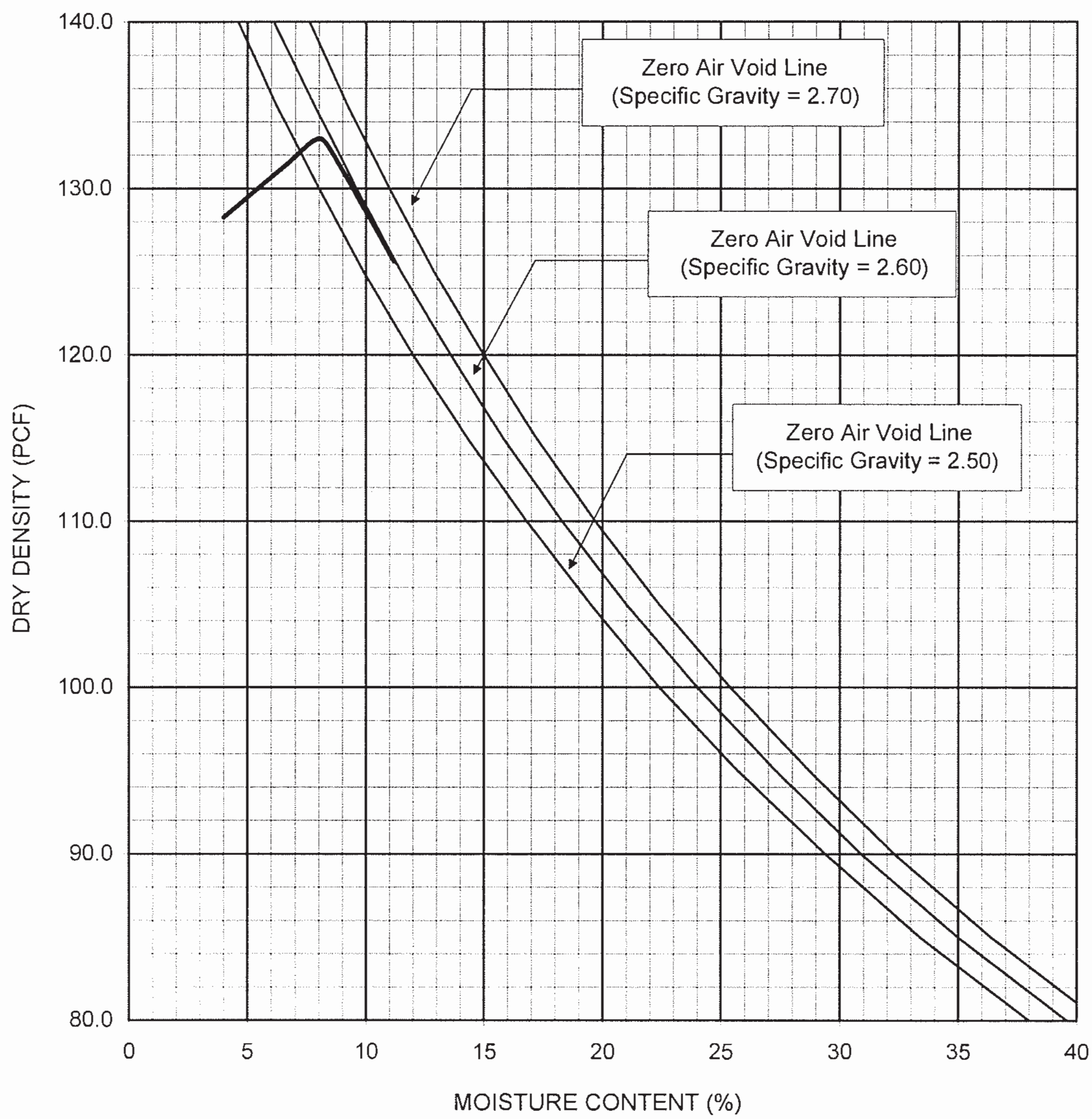
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore	DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO. 106115001 / DATE 3/09	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-8

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-3	3.0-4.5	9.3	111.7	19.2	0.012	12	Very Low
B-6	0.5-3.0	10.0	108.4	20.7	0.039	39	Low
B-7	5.0-8.5	11.0	106.5	21.8	0.056	56	Medium
B-10	0.0-5.0	8.5	116.3	16.2	0.005	5	Very Low

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

Ninyo & Moore		EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-9
106115001	3/09		



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-4	0.3-5.0	Light Brown Silty SAND (SM)	133.0	8.0
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718-87)			N/A	N/A

PERFORMED IN GENERAL ACCORDANCE WITH ☑ ASTM D 1557-02 ☐ ASTM D 698-00a METHOD ☑ A ☐ B ☐ C

Ninyo & Moore	PROCTOR DENSITY TEST RESULTS	FIGURE
PROJECT NO. 106115001	DATE 3/09	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-10

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	2.0-3.5	6.2	16,080	110	0.011	60
B-7	5.0-8.5	6.3	1,500	190	0.019	340
B-10	0.0-5.0	5.5	7,370	50	0.005	320

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT	DATE	CHET HARRITT SCHOOL SANTEE, CALIFORNIA	B-11
106115001	3/09		

APPENDIX C

Infiltration Testing

Test Date:	4/19/2019						Infiltration Test No.:	NM-5
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	5.0
Test performed and recorded by:			CMK				Pipe Length (feet):	5.0
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
6:59	2.88	7:29	2.99	30	0.11	23	2.07	0.15
7:29	2.99	7:59	3.08	30	0.09	28	1.97	0.13
7:59	2.92	8:29	3.01	30	0.09	28	2.04	0.13
8:29	2.78	8:59	2.89	30	0.11	23	2.17	0.14
8:59	2.89	9:29	2.97	30	0.08	31	2.07	0.11
9:29	2.87	9:59	2.96	30	0.09	28	2.09	0.12
9:59	2.84	10:29	2.94	30	0.10	25	2.11	0.13
10:29	2.94	10:59	3.02	30	0.08	31	2.02	0.11
10:59	2.80	11:29	2.91	30	0.11	23	2.15	0.15
11:29	2.91	11:59	2.99	30	0.08	31	2.05	0.11
11:59	2.84	12:29	2.92	30	0.08	31	2.12	0.11
12:29	2.92	12:59	3.01	30	0.09	28	2.04	0.13

Test Date:	4/19/2019						Infiltration Test No.:	NM-6
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	5.0
Test performed and recorded by:			CMK				Pipe Length (feet):	5.0
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
7:01	2.27	7:31	2.29	30	0.02	125	2.72	0.02
7:31	2.29	8:01	2.30	30	0.01	250	2.71	0.01
8:01	2.30	8:31	2.32	30	0.02	125	2.69	0.02
8:31	2.32	9:01	2.33	30	0.01	250	2.68	0.01
9:01	2.33	9:31	2.35	30	0.02	125	2.66	0.02
9:31	2.35	10:01	2.36	30	0.01	250	2.65	0.01
10:01	2.30	10:31	2.31	30	0.01	250	2.70	0.01
10:31	2.31	11:01	2.32	30	0.01	250	2.69	0.01
11:01	2.32	11:31	2.33	30	0.01	250	2.68	0.01
11:31	2.33	12:01	2.35	30	0.02	125	2.66	0.02
12:01	2.35	12:31	2.36	30	0.01	250	2.65	0.01
12:31	2.36	1:01	2.37	30	0.01	250	2.64	0.01

Notes:

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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_2 - d_1)

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion [1]

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

I_t = 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_{avg} = average head over the time interval, inches

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

Test Date:	4/18/2017						Infiltration Test No.:	IT-1 (2017)
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	2.5
Test performed and recorded by:			GSW				Pipe Length (feet):	2.5
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:42	2.00	9:07	2.00	25	0.00	---	0.50	DNI
9:07	2.00	9:32	2.00	25	0.00	---	0.50	DNI
9:32	2.00	10:02	2.01	30	0.01	250	0.50	0.05
10:02	2.00	10:32	2.00	30	0.00	---	0.50	DNI
10:32	2.00	11:02	2.00	30	0.00	---	0.50	DNI
11:02	2.00	11:32	2.00	30	0.00	---	0.50	DNI
11:32	2.00	12:02	2.01	30	0.01	250	0.50	0.05
12:02	2.00	12:32	2.00	30	0.00	---	0.50	DNI
12:32	2.00	1:02	2.00	30	0.00	---	0.50	DNI
1:02	2.00	1:32	2.00	30	0.00	---	0.50	DNI
1:32	2.00	2:02	2.00	30	0.00	---	0.50	DNI
2:02	2.00	2:32	2.00	30	0.00	---	0.50	DNI

Test Date:	4/18/2017						Infiltration Test No.:	IT-2 (2017)
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	3.0
Test performed and recorded by:			GSW				Pipe Length (feet):	3.0
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:40	2.20	9:05	3.00	25	0.80	3	0.40	5.49
9:05	2.20	9:30	3.00	25	0.80	3	0.40	5.49
9:30	2.20	9:40	2.90	10	0.70	1	0.45	10.96
9:40	2.20	9:50	2.85	10	0.65	1	0.48	9.75
9:50	2.20	10:00	2.83	10	0.63	1	0.49	9.30
10:00	2.20	10:10	2.80	10	0.60	1	0.50	8.64
10:10	2.20	10:20	2.80	10	0.60	1	0.50	8.64
10:20	2.20	10:30	2.80	10	0.60	1	0.50	8.64

Notes:

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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_2 - d_1)

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion [1]

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

I_t = 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_{avg} = average head over the time interval, inches

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

Test Date:	4/18/2017					Infiltration Test No.:		IT-3 (2017)
Test Hole Diameter, D (inches):			6.0			Excavation Depth (feet):		2.0
Test performed and recorded by:			GSW			Pipe Length (feet):		2.0

t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:47	1.45	9:12	1.45	25	0.00	---	0.55	DNI
9:12	1.45	9:37	1.45	25	0.00	---	0.55	DNI
9:37	1.45	10:07	1.45	30	0.00	---	0.55	DNI
10:07	1.45	10:37	1.46	30	0.01	250	0.55	0.04
10:37	1.45	11:07	1.46	30	0.01	250	0.55	0.04
11:07	1.45	11:37	1.46	30	0.01	250	0.55	0.04
11:37	1.45	12:07	1.46	30	0.01	250	0.55	0.04
12:07	1.45	12:37	1.46	30	0.01	250	0.55	0.04
12:37	1.45	1:07	1.46	30	0.01	250	0.55	0.04
1:07	1.45	1:37	1.46	30	0.01	250	0.55	0.04
1:37	1.45	2:07	1.46	30	0.01	250	0.55	0.04
2:07	1.45	2:37	1.46	30	0.01	250	0.55	0.04

Test Date:	4/18/2017					Infiltration Test No.:		IT-4 (2017)
Test Hole Diameter, D (inches):			6.0			Excavation Depth (feet):		3.0
Test performed and recorded by:			GSW			Pipe Length (feet):		3.0

t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:48	1.21	9:13	1.21	25	0.00	---	1.79	DNI
9:13	1.21	9:38	1.22	25	0.01	208	1.79	0.02
9:38	1.21	10:08	1.21	30	0.00	---	1.79	DNI
10:08	1.21	10:38	1.21	30	0.00	---	1.79	DNI
10:38	1.21	11:08	1.21	30	0.00	---	1.79	DNI
11:08	1.21	11:38	1.21	30	0.00	---	1.79	DNI
11:38	1.21	12:08	1.22	30	0.01	250	1.79	0.02
12:08	1.21	12:38	1.21	30	0.00	---	1.79	DNI
12:38	1.21	1:08	1.21	30	0.00	---	1.79	DNI
1:08	1.21	1:38	1.21	30	0.00	---	1.79	DNI
1:38	1.21	2:08	1.21	30	0.00	---	1.79	DNI
2:08	1.21	2:38	1.21	30	0.00	---	1.79	DNI

Notes:

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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., $d_2 - d_1$)

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion[1]

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

I_t = 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_{avg} = average head over the time interval, inches

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

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	2	0.5
		Site soil variability	0.25	2	0.5
		Depth to groundwater / impervious layer	0.25	2	0.5
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.0
B	Design	Level of pretreatment/ expected sediment loads	0.5		
		Redundancy/resiliency	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
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, $K_{design} = K_{observed} / S_{total}$					
Supporting Data					

Briefly describe infiltration test and provide reference to test forms:
Six infiltration tests were performed at the site (IT-1 through IT-4, NM-5, and NM-6). In-situ rates (i.e., unfactored rates) of 0.04, 0.16, and 0.01 inches per hour for IT-3, NM-5 and NM-6, respectively. Infiltration tests IT-1 and IT-4 did not infiltrate. The infiltration rate of 8.64 inches per hour in IT-2 is not considered indicative of the site's infiltration characteristics.
IT-1 was performed at a depth of 2.5 feet below existing grade in weathered granitic rock.
IT-2 was performed at a depth of 3 feet below existing grade in fill material consisting of silty sand.
IT-3 was performed at a depth of 2 feet below existing grade in weathered granitic rock.
IT-4 was performed at a depth of 2 feet below existing grade in weathered granitic rock.
NM-5 was performed at a depth of 5 feet below existing grade in fill material consisting of silty sand.
NM-6 was performed at a depth of approximately 5 feet below existing grade in fill material consisting of silty, clayey sand.
Additional information regarding infiltration testing is included in Ninyo & Moore's geotechnical evaluation (2019).

Worksheet 0-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		Worksheet C.4-1	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** 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: Six infiltration tests were performed at the site, IT-1 through IT-4, NM-5, and NM-6 . In-situ rates (i.e., unfactored rates) of 0.04, 0.16, and 0.01 inches per hour for IT-3, NM-5 and NM-6 , respectively. Infiltration tests IT-1 and IT-4 did not infiltrate. The infiltration rate of 8.64 inches per hour in IT-2 is not considered indicative of the site's infiltration characteristics. IT-1 was performed at a depth of 2.5 feet below existing grade in weathered granitic rock. IT-2 was performed at a depth of 3 feet below existing grade in fill material consisting of silty sand. IT-3 was performed at a depth of 2 feet below existing grade in weathered granitic rock. IT-4 was performed at a depth of 2 feet below existing grade in weathered granitic rock. NM-5 was performed at a depth of 5 feet below existing grade in fill material consisting of silty sand. NM-6 was performed at a depth of approximately 5 feet below existing grade in fill material consisting of silty, clayey sand. 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.			
2	**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?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: Reliable infiltration rates (i.e., factored) of 0.09 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 & 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.			

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>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? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Reliable infiltration rates (i.e., factored) of 0.09 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.</td></tr>
<tr><td>4</td><td>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? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Reliable infiltration rates (i.e., factored) of 0.09 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.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is “No”, 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</td><td>NO</td></tr>
</table>

*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.

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	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** 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 0.09 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	**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**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
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.			

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>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)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Groundwater was encountered approximately 13 feet below existing grade. Due to the close proximity of a drainage channel on the west side of the school, groundwater should be anticipated at depths of approximately 10 to 15 feet. 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.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
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.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No infiltration</td></tr>
</table>

*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

Geotechnical & Environmental Sciences Consultants

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

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

www.ninyoandmoore.com