Geotechnical Evaluation

Pride Academy at Prospect Avenue School
Library Addition
9303 Prospect Avenue
Santee, California

Santee School District

9625 Cuyamaca Street | Santee, California 92071

May 30, 2019 | Project No. 108775001

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

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

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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1 INTRODUCTION

In accordance with your authorization and Notice to Proceed dated March 8, 2019, we performed an update geotechnical evaluation for the proposed library building addition to Pride Academy at Prospect Avenue School campus located at 9303 Prospect Avenue in Santee, California (Figure 1). The purpose of our services were to update our original geotechnical evaluation report (Ninyo & Moore, 2007) and infiltration testing report (Ninyo & Moore, 2017) to the current 2016 California Building Code (CBC) requirements and local guideless for site infiltration to address storm water management. Our geotechnical evaluation 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) and infiltration testing report for the school campus (Ninyo & Moore, 2017), topographic maps, geologic maps, fault maps, stereoscopic aerial photographs, and preliminary site plans (StudioWC, 2019).

- Performing a field reconnaissance to observe existing site conditions and to mark the locations of our exploratory borings.

- Coordinating with Santee School District personnel to locate underground utilities near our exploratory borings. Additionally, we notified Underground Service Alert (USA).

- Performing a subsurface exploration consisting of the drilling, logging, and sampling of two (2) 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 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 Pride Academy at Prospect Avenue School in Santee, California (Figure 1). In general, the campus is located on a rectangular-shaped parcel bounded by Prospect Avenue to the north, Ellsworth Lane to the west, Northview Lane to the east, and single-family residential properties to the south. The school site generally consists of school buildings, facilities, parking lots in the northern portion of the campus, with hardscape areas and athletic fields in the southern portions of the property. Elevations at the campus range from approximately 345 feet above mean sea level (MSL) at the northwest corner to approximately 365 feet above MSL in the southwest corner. The global coordinates of the project site are approximately 32.833°N Latitude and -116.995°W Longitude.

Our office previously performed a geotechnical evaluation for the proposed library addition (Ninyo & Moore, 2007). Based on our correspondence with the Project Architect, we understand that the project has been changed from the previous design described in our 2007 geotechnical evaluation report. The current project includes the construction of a single-story library building in the northeastern portion of the campus. The library building will have a footprint of approximately 5,700 square feet (sf). The location of the proposed building is currently improved with a modular classroom building and gravel areas. Further improvements, in addition to the new library building, 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. Additionally, due to our previous geotechnical design work at the site, we have utilized some existing boring data from our previous geotechnical evaluation (Ninyo & Moore, 2007).

Our office also previously performed an infiltration evaluation at the campus (Ninyo & Moore, 2017). This update evaluation includes the results of the four 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 24, 2019 and included the drilling, logging, and sampling of two small-diameter borings (NM-1 and NM-2). Borings NM-1 and Ninyo & Moore-2 were drilled to depths up to approximately 18.8 feet using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Our previous subsurface explorations were conducted in 2007 and 2017. Seven small-diameter borings (B-1 through B-7) were drilled on June 26 and July 2, 2007 and four infiltration tests (PA-1 through PA-4) were excavated on April 19, 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 subsurface exploration. Testing included an evaluation of in-situ dry density and moisture content, gradation, consolidation, shear strength, expansion index, soil corrosivity, and R-value. The results of the in-situ dry density and moisture content tests are presented on the boring logs presented in Appendix A. The results of the other laboratory tests that we performed are presented in Appendix B. Laboratory test results from our 2007 evaluation are also included in Appendix B.

6 INFILTRATION TESTING

As a means of evaluating the infiltration characteristics of near-surface materials, infiltration tests were previously performed on April 19 and 20, 2017 at four locations designated PA-1 through PA-4 (Figure 2, Ninyo & Moore, 2017). Borings PA-1 through PA-4 were manually excavated to depths up to approximately 3 feet using a 6-inch diameter hand auger. Following the 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 19, 2017 for to represent adverse conditions for infiltration. The presoak consisted of maintaining approximately 0.5 to 2 feet of water in each test boring for approximately 4 hours. The water level was then allowed to drop overnight.

Infiltration testing was performed on April 20, 2017, 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 at borings PA-1 through PA-4 indicated that the observed (i.e., unfactored) infiltration rates were 0.05 inches per hour or less. Infiltration test results and calculations are included in Appendix C and summarized in Table 1. Per the City of Santee BMP Design Manual Appendix D Section D.5.1, a suitability assessment factor of safety (FOS) of 2.0 was developed. A completed Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions with the appropriate geotechnical aspects is presented in Appendix C. The rates presented in Table 1 are to be used for preliminary design purposes.

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)
PA-1	3.0	Sandy CLAY (Fill)	0.05	2.0	0.03
PA-2	3.0	Sandy CLAY (Fill)	0.01	2.0	<0.01
PA-3	3.0	Sandy CLAY (Fill)	0.05	2.0	0.03
PA-4	3.0	Sandy CLAY (Fill)	0.05	2.0	0.03

Notes:
in/hr = inches per hour
[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.

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 older alluvial deposits, Cretaceous-age Tonalite, hereafter “granitic rock”, and materials of the Tertiary-age Friars Formation (Todd, 2004). Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the logs of the borings in Appendix A. The geology of the site is shown on Figure 3 and geologic cross sections are shown on Figures 4A and 4B.

7.2.1 Fill

Fill material was encountered in our exploratory borings from the ground surface to depths up to approximately 11 feet. As encountered, the material generally consisted of various shades of brown, moist, medium dense, silty sand, clayey sand, and firm to stiff, sandy clay. Scattered amounts of gravel were encountered in the fill. Documentation regarding fill placement was not available for our review.

7.2.2 Older Alluvium

Older alluvium was encountered in each our borings, except boring B-3, underlying the fill to a depth up to approximately 9.5 feet. As encountered, the material generally consisted of various shades of brown, moist, firm to very stiff, sandy clay and medium dense to dense, clayey sand. Gravel and cobble fragments were encountered in the older alluvium.

7.2.3 Friars Formation

Although not encountered in our recent subsurface exploration, 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.

7.2.4 Granitic Rock

Decomposed or weathered granitic rock was encountered in our borings underlying the other units to the total depth explored. As encountered, the decomposed granitic materials generally consisted of various shades of brown, moist, very dense, clayey silt and silty sand with remnant grain structures. Weathered granitic rock was generally various shades of brown and gray and slightly to highly weathered.

7.3 Groundwater

Groundwater was not encountered during our recent or previous subsurface evaluations. However, based on elevations of Forester Creek approximately 0.2 miles north of the project site, groundwater is estimated to be at approximately 30 feet below existing grade. 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 older 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 Federal Emergency Management Agency (FEMA) Mapping Information Platform website (2017), the site is not within a flood zone. Based on review of topographic maps, the site is located approximately 0.6 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 40 feet above the riverbed. 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. 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 12.5 miles west of the site. Table 2 lists selected principal known active faults that may affect the site and the maximum moment magnitude M_{max} calculated from the USGS National Seismic Hazard Maps - Fault Parameters website (USGS, 2008).

Table 2 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (M_{max})
Rose Canyon	12.5 (20.1)	6.9
Coronado Bank	24.9 (40)	7.4
Elsinore (Julian Segment)	29.1 (46.8)	7.4
Newport-Inglewood (Offshore Segment)	33.7 (54.2)	7.0
Earthquake Valley	33.7 (54.2)	6.8
Elsinore (Temecula Segment)	35 (56.3)	7.1
Elsinore (Coyote Mountain Segment)	39.4 (63.4)	6.9
San Jacinto (Coyote Creek Segment)	50.1 (80.6)	7.0
San Jacinto (Borrego Segment)	52.5 (84.5)	6.8
San Jacinto (Clark Segment)	53 (85.3)	7.1
San Jacinto (Anza Segment)	53.1 (85.5)	7.3
Elsinore (Glen Ivy Segment)	57 (91.7)	6.9

7.6.1 Strong Ground Motion

Based on our review of background information, data pertaining to the historical seismicity of the San Diego area are summarized in Table 3 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 3 – Historical Earthquakes that Affected the Site

Date	Magnitude (M)	Approximate Epicentral Distance miles (kilometers)
November 22, 1800	6.5	21 (33)
May 27, 1862	5.9	15 (24)
February 9, 1890	6.3	56 (90)
February 24, 1892	6.7	41 (66)
May 28, 1892	6.3	52 (84)
October 23, 1894	5.7	11 (18)
September 30, 1916	5.0	54 (87)
January 1, 1920	5.0	30 (49)
November 25, 1934	5.0	56 (88)
March 25, 1937	6.0	58 (93)
June 4, 1940	5.1	34 (55)
October 21, 1942	6.5	60 (96)
August 15, 1945	5.7	57 (91)
November 4, 1949	5.7	50 (81)
March 19, 1954	6.2	56 (90)
September 23, 1963	5.0	60 (97)
April 9, 1968	6.4	58 (94)
April 28, 1969	5.8	51 (82)
January 12, 1975	5.1	57 (91)
February 25, 1980	5.6	53 (86)
July 13, 1986	5.8	52 (83)
October 31, 2001	5.2	54 (87)
June 12, 2005	5.2	53 (86)

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 older alluvium and granitic rock encountered our borings and the depth to groundwater, it is our opinion that the potential for liquefaction to occur at the site is not a design consideration.

7.6.4 Tsunamis

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

8 CONCLUSIONS

Based on our review of the referenced background data, subsurface 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 site is underlain by fill materials overlying older alluvium and granitic rock.
- The existing fill 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 may generate oversize material and additional processing and handling of these materials, including screening and/or rock picking, should be anticipated.
- Fill materials encountered in our subsurface evaluation are not considered suitable for structural support in their current condition. Recommendations are presented herein for remedial grading of this material.
- Based on the geotechnical laboratory testing presented in Appendix B and our previous evaluation at the school campus (Ninyo & Moore, 2007), some of the onsite fill materials possess a medium potential for expansion. Materials that possess a medium to high potential for expansion are not suitable for reuse as compacted fill beneath buildings, behind retaining walls, or as subgrade soils beneath pedestrian concrete paving. The contractor should be prepared to perform selective grading techniques or removal and replacement if these materials are encountered.
- Groundwater was not encountered during our recent or previous geotechnical evaluations. However, groundwater is estimated to be approximately 30 feet below grade based on the elevation of Forester Creek north of the project site.
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). However, the closest known major active fault is the Rose Canyon Fault, which is located approximately 12.5 miles west of the project.

- Based on the results of our limited geotechnical laboratory testing, the onsite soils exhibit a low to medium expansion potential. Our recommendations presented herein include consideration for the expansive nature of onsite soils.
- Based on the results of our limited geotechnical laboratory testing the Caltrans (2018) corrosion guidelines, we consider the onsite soils to be corrosive due to the chloride contents from our 2007 evaluation.

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, older alluvium, granitic rock, and Friars Formation. The fill and older 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 and Friars Formation 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. Though primarily cohesive soils were encountered in our exploratory boring, caving of open excavations (i.e., drilled excavations) 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, 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 and older alluvium, and 1:1 (horizontal to vertical) in granitic rock and 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

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

9.1.5 Remedial Grading – Building Pads

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

This over excavation 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. Note, the recommended overexcavations should also be performed beneath the foundations associated with elevators for the buildings.

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). 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.

9.1.6 Remedial Grading – Retaining Wall

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

The resulting removal surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and 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.

9.1.7 Remedial Grading – Pedestrian Concrete Paving

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

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

9.1.8 Remedial Grading – Vehicular Pavements

In the proposed vehicular pavement areas, we recommend that the onsite soils be overexcavated to a depth of 1 foot below the planned subgrade elevation for the pavement. The proposed overexcavations should extend outward horizontally 2 feet from the exterior limits of the pavement, where feasible. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 6 inches, moisture conditioned, and 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.

9.1.9 Materials for Fill

Materials for fill may be obtained from onsite excavations or import sources. However, 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 evaluation 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 within the building pad, in the upper 1 foot beneath retaining wall footings, as retaining wall backfill, or as the upper 1 foot of subgrade soils beneath pedestrian concrete paving. These materials should be selectively graded for use in non-structural areas, may be blended with other soils to create a mixture that possesses an EI of less than 50, or removed and replaced.

Imported fill material should generally be granular soils with a very low to low expansion potential (i.e., an 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 less than 5.5. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing. To reduce the potential of importing contaminated materials to the site, prior to delivery, soil materials obtained from off-site sources should be sampled and tested in accordance with standard practice (DTSC, 2001). Soils that exhibit a known risk to human health, the environment, or both, should not be imported to the site.

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

9.1.10 Compacted Fill

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

Fill materials should be moisture conditioned to generally at or slightly above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

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

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

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

It is our recommendation that new pipelines (pipes), where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or more.

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,400 pounds per square inch (psi) may be used for an excavation depth of up to approximately 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557.

9.1.12 Pipe Zone Backfill

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

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

9.1.13 Utility Trench Zone Backfill

Onsite excavations may generate clayey soils that possess a medium potential for expansion. These expansive clay soils are generally not considered suitable for trench backfill within the building pads, in the upper 1 foot beneath retaining wall footings, behind retaining walls, or as the upper 1 foot of subgrade soils beneath pedestrian concrete paving. Trench zone backfill material should be generally free of trash, debris, roots, vegetation, or deleterious materials. Additionally, onsite excavations may generate oversized materials. Trench zone backfill should generally be free of rocks or hard lumps of material in excess of 3 inches in diameter. Rocks or hard lumps larger than about 3 inches in diameter should be broken into smaller pieces or should be removed from the site. Oversize materials should be separated from material to be used as trench backfill. Moisture conditioning (including drying and/or mixing) of existing onsite materials is anticipated if reused as trench backfill. Trench zone backfill should be moisture-conditioned to generally at or slightly above the laboratory optimum. Trench zone backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557, except for the upper 12 inches of the backfill beneath vehicular pavements that should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

9.1.14 Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the magnitude and distribution of passive lateral earth pressures presented on Figure 6. Thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

9.1.15 Drainage

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

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

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

9.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 4 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 4 – 2016 California Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.152
Site Coefficient, F_v	1.723
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.871g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.339g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.003g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.583g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.669g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.389g

9.3 Foundations

Based on our understanding of the project, the proposed new buildings will be constructed at the site following demolition of the existing improvements. The new buildings are anticipated to be supported on shallow foundations. Recommendations for the shallow building foundations are presented in following sections. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

9.3.1 Shallow Foundations

Shallow, spread or continuous footings supported on compacted fill may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. To mitigate for the expansive nature of the onsite soils, we recommend that shallow, spread footings be founded 36 inches below the lowest adjacent grade and be 36 inches of width. The footings should be reinforced in accordance with the recommendations of the project structural engineer.

9.3.2 Lateral Resistance

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

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.20 be used between soil and concrete. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.3.3 Static Settlement

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

9.4 Site Retaining Walls

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

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

9.5 Interior Slabs-on-Grade

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

9.6 Concrete Flatwork

We recommend that exterior concrete flatwork be 4 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. A vapor retarder is not needed for exterior flatwork. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the civil engineer. Before placement of concrete, the subgrade soils should be scarified to a depth of 6 inches, moisture conditioned to generally at or slightly above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Positive drainage should be established and maintained adjacent to flatwork.

9.7 Light Pole and Canopy Foundations

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

The drilled pile construction should be observed by Ninyo & Moore during construction to evaluate if the piles have been extended to the design depths. It is the contractor's responsibility to (a) take appropriate measures for maintaining the integrity of the drilled holes, (b) see that the holes are cleaned and straight, and (c) see that sloughed loose soil is removed from the bottom of the hole prior to the placement of concrete. Drilled piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by 1 percent of the depth of the pile. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile. If the CIDH piles extend into groundwater or seepage, the contractor should consider appropriate measures during construction to reduce the potential for caving of the drilled holes, including the use of steel casing and/or drilling mud. In addition, we recommend concrete be placed by tremie method, to see that the aggregate and cement do not segregate during concrete placement, on the same day the CIDH piles are drilled.

For resistance of light pole footings to lateral loads, we recommend an allowable passive pressure of 300 psf per foot of depth be used, with an upper bound value of up to 3,000 psf. This value assumes that the light poles are designed to tolerate ½ inch of deflection at the surface and that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.30 be used between soil and concrete. The allowable lateral resistance values may be increased by 1/3 during short term loading conditions, such as wind or seismic loading.

9.8 Preliminary Flexible Pavement Design

We understand that the project will include the construction of new pavements. Our laboratory testing of a near surface soil sample at the project site indicated an R-value of 9. 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 5.

Table 5 – Recommended Preliminary Flexible Pavement Sections

Traffic Index (Pavement Usage)	Design R-Value	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5 (Parking Stalls)	9	3	9
6 (Drive Aisles)	9	3	11
7 (Fire Lanes and Bus Lanes)	9	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, 7 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.

9.9 Corrosion

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

The results of the corrosivity testing indicated electrical resistivities of 1,000 and 3,790 ohm-cm, soil pH of 7.3 and 8.1, chloride contents of 225 and 260 ppm, and sulfate contents of 0.006 and 0.068 percent (i.e., 60 and 680 ppm). Based on a comparison with the Caltrans corrosion (2018) criteria, the onsite soils would be classified as corrosive. Corrosive soils are defined as soil with an electrical resistivity less than 1,100 ohm-cm, a chloride content more than 500 ppm, more than 0.15 percent sulfates (1,500 ppm), and/or a pH less than 5.5.

9.10 Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates that can be subject to premature chemical and/or physical deterioration. As noted, the soil sample tested in this evaluation indicated a water-soluble sulfate content of 0.006 and 0.068 percent by weight (i.e., 60 and 680 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soils less than about 0.10 percent by weight. Therefore, the site soils may be considered to have a negligible potential for sulfate attack. However, due to the potential variability of site soils, consideration should be given to using Type II, II/V, or Type V cement for normal weight concrete in contact with soil.

10 PERMANENT INFILTRATION DEVICES

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

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

11 PRE-CONSTRUCTION CONFERENCE

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

12 PLAN REVIEW AND CONSTRUCTION OBSERVATION

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

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client and Ninyo & Moore with a Division of The State Architect (DSA) 109 form indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

13 LIMITATIONS

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

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

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

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

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

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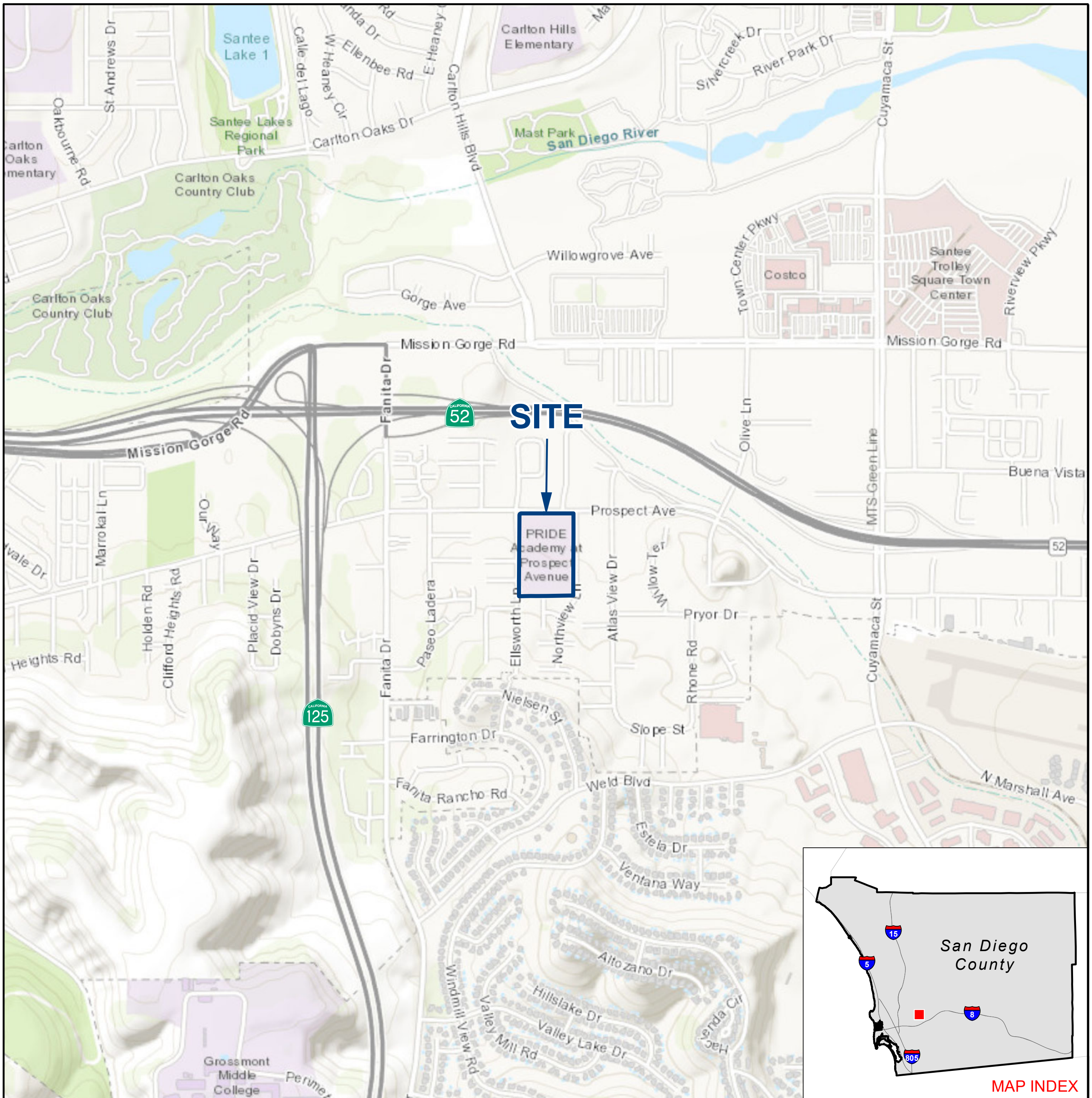
FIGURES



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

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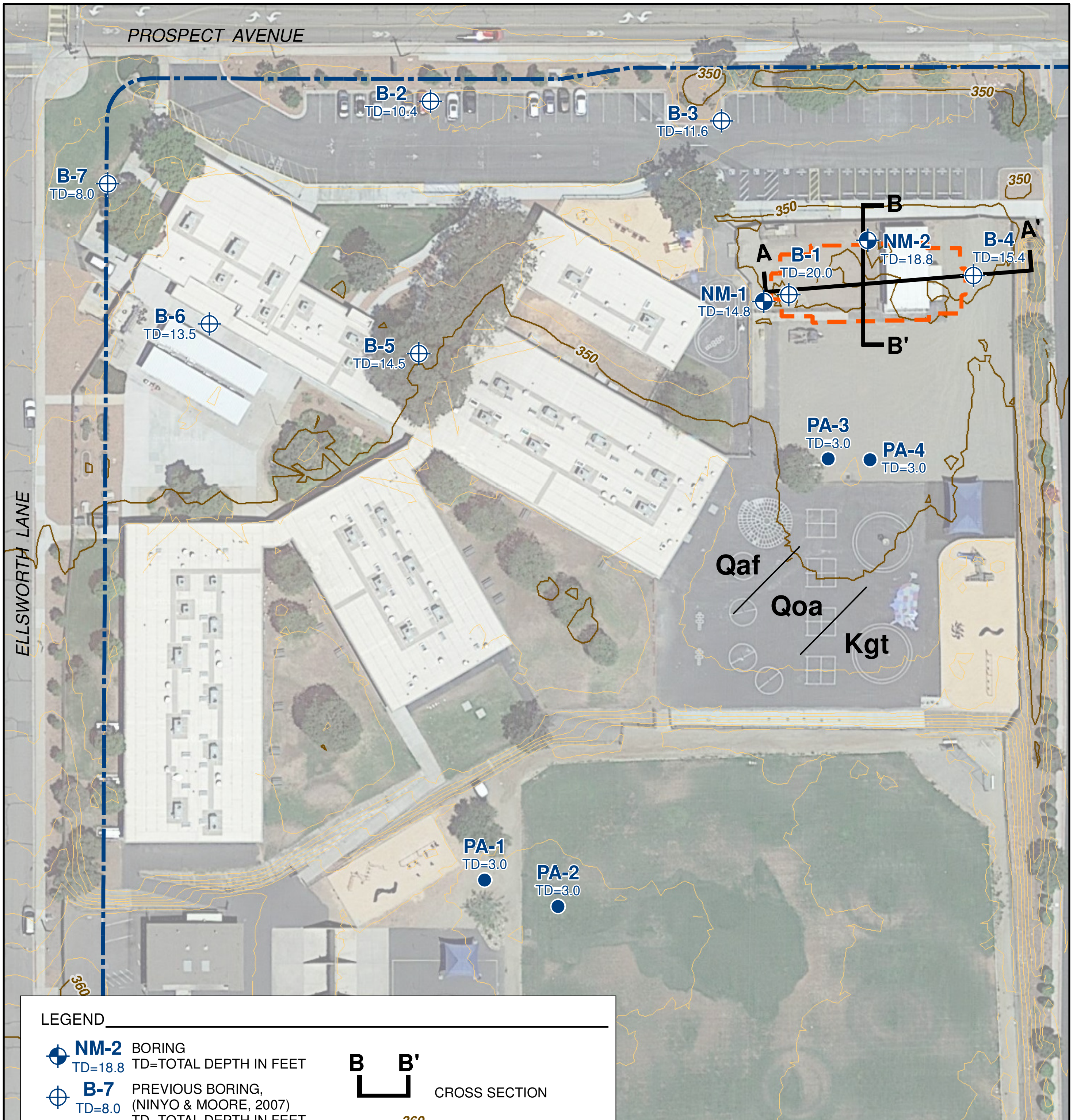
FIGURE 1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE LOCATION

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19



LEGEND

NM-2 TD=18.8	BORING TD=TOTAL DEPTH IN FEET	B B'	CROSS SECTION
B-7 TD=8.0	PREVIOUS BORING, (NINYO & MOORE, 2007) TD=TOTAL DEPTH IN FEET	360	INDEX/INTERMEDIATE TOPOGRAPHIC CONTOUR
PA-4 TD=3.0	PREVIOUS INFILTRATION TEST, (NINYO & MOORE, 2007) TD=TOTAL DEPTH IN FEET	Qaf	FILL
	PROPOSED BUILDING ADDITION	Qoa	ALLUVIUM
	SITE BOUNDARY	Kgt	GRANITIC ROCK

N

FEET

0 80 160

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

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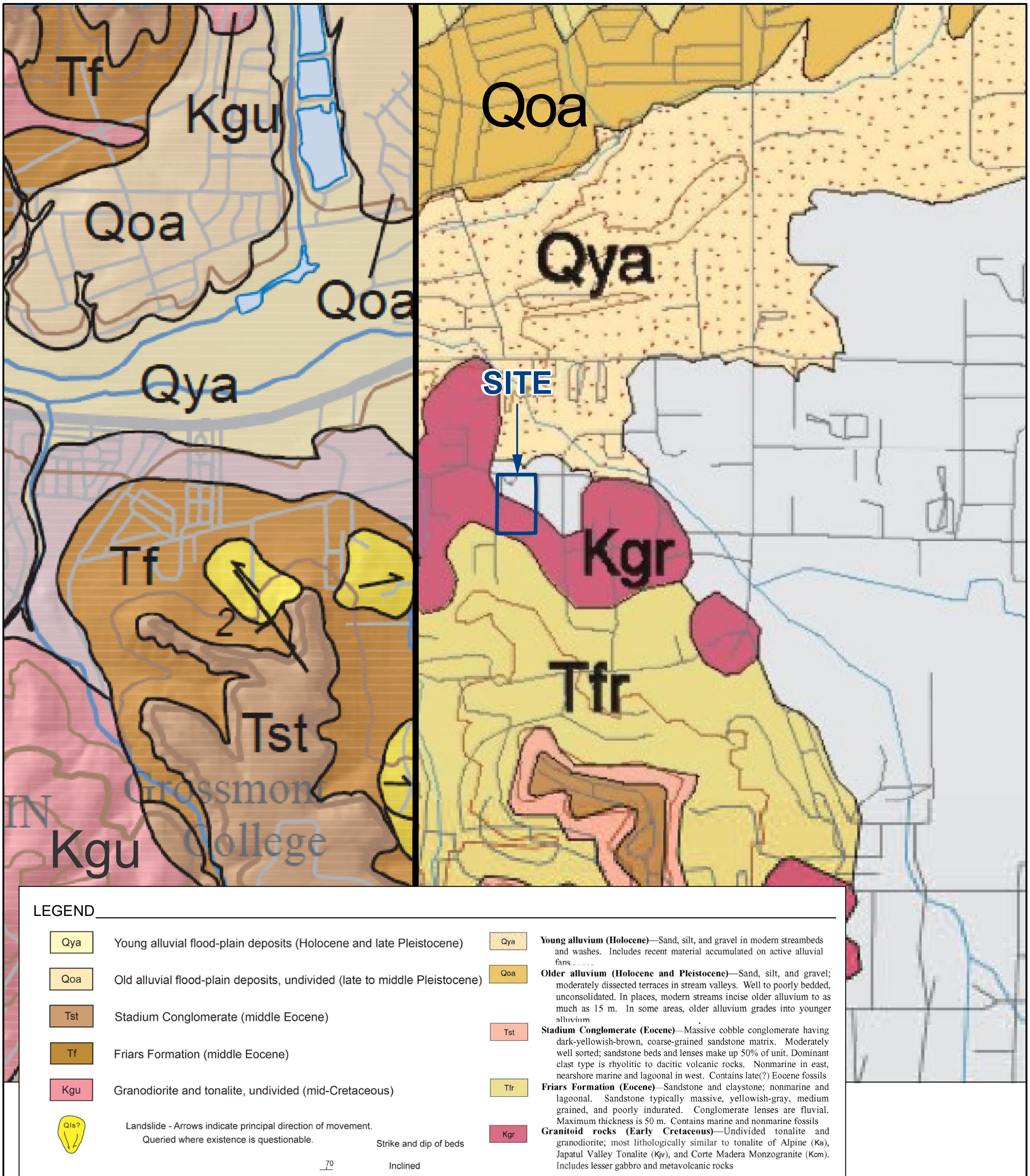
FIGURE 2

BORING LOCATIONS

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108775001 | 5/19



3_108775001_G.mxd 5/14/2019 JDL

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCES: KENNEDY, M.P., AND TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30' X 60' QUADRANGLE, CALIFORNIA; TODD, V.R., 2004, PRELIMINARY GEOLOGIC MAP OF THE EL CAJON 30' X 60' QUADRANGLE, SOUTHERN CALIFORNIA,

N

FEET

0 2,000 4,000

FIGURE 3

GEOLOGY

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

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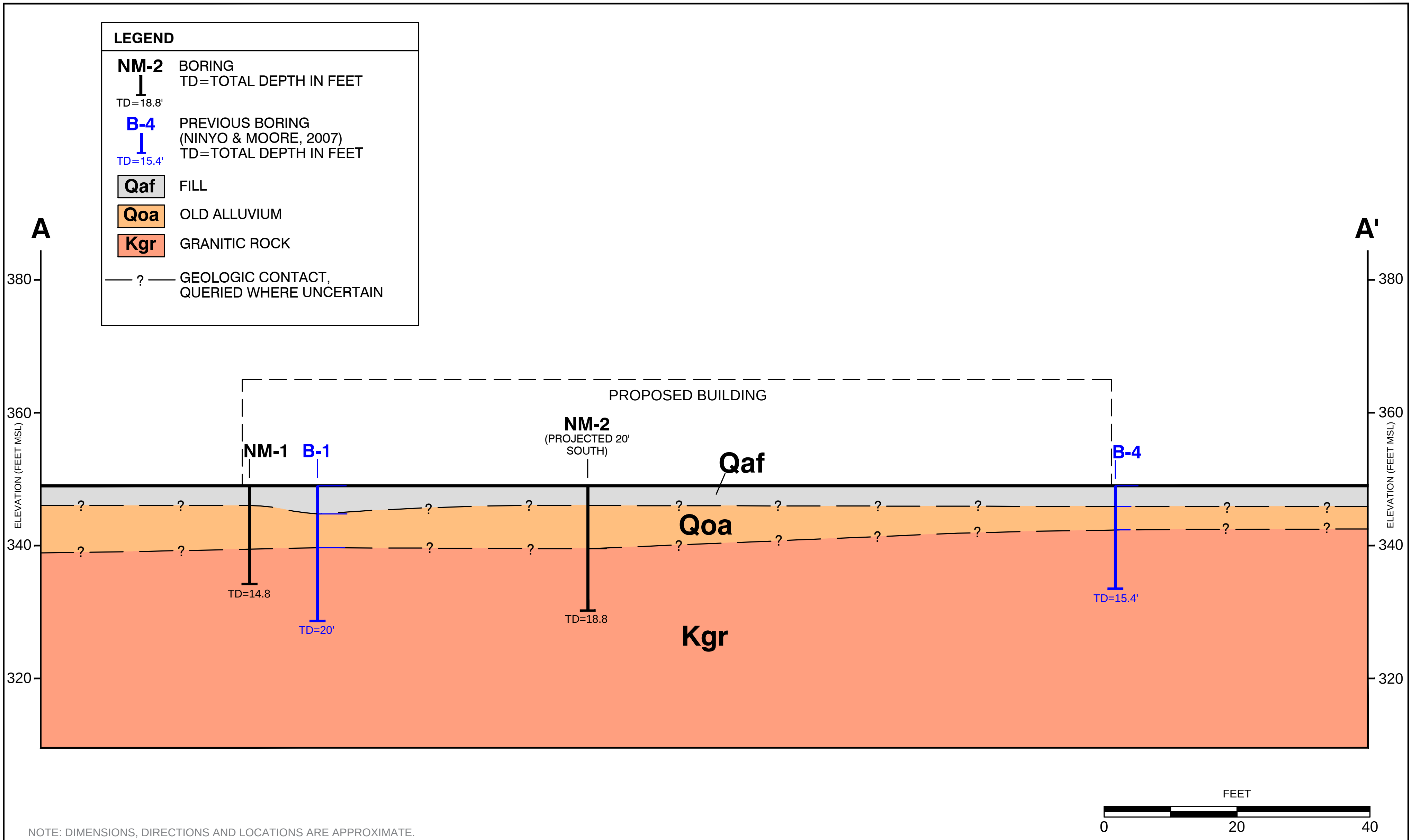
108775001 | 5/19



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 4A

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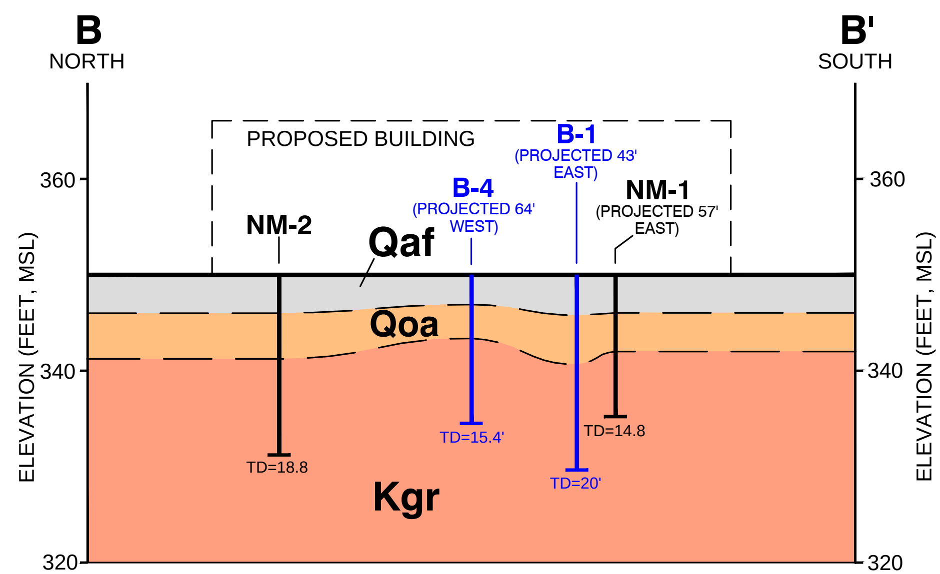
GEOLOGIC CROSS SECTION A-A'

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 I 5/19



LEGEND

NM-2 TD=18.8'	BORING TD=TOTAL DEPTH IN FEET
B-4 TD=15.4'	PREVIOUS BORING (NINYO & MOORE, 2007) TD=TOTAL DEPTH IN FEET
Qaf	FILL
Qoa	OLD ALLUVIUM
Kgr	GRANITIC ROCK
—— ? ——	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

4B 108775001 CS B-B'.DWG

FIGURE 4B

GEOLOGIC CROSS SECTION B-B'

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

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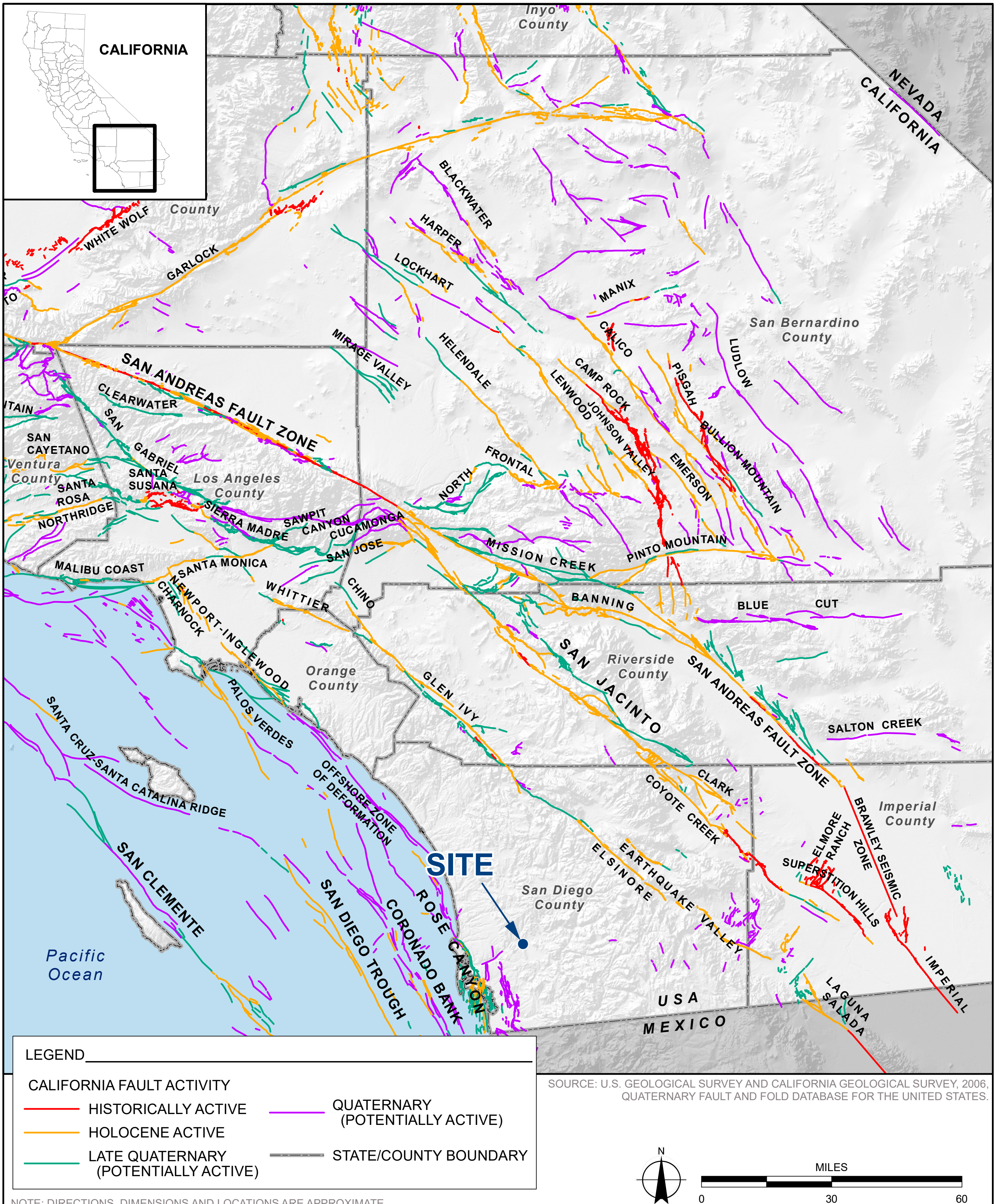
108775001 I 5/19



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

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 5

FAULT LOCATIONS

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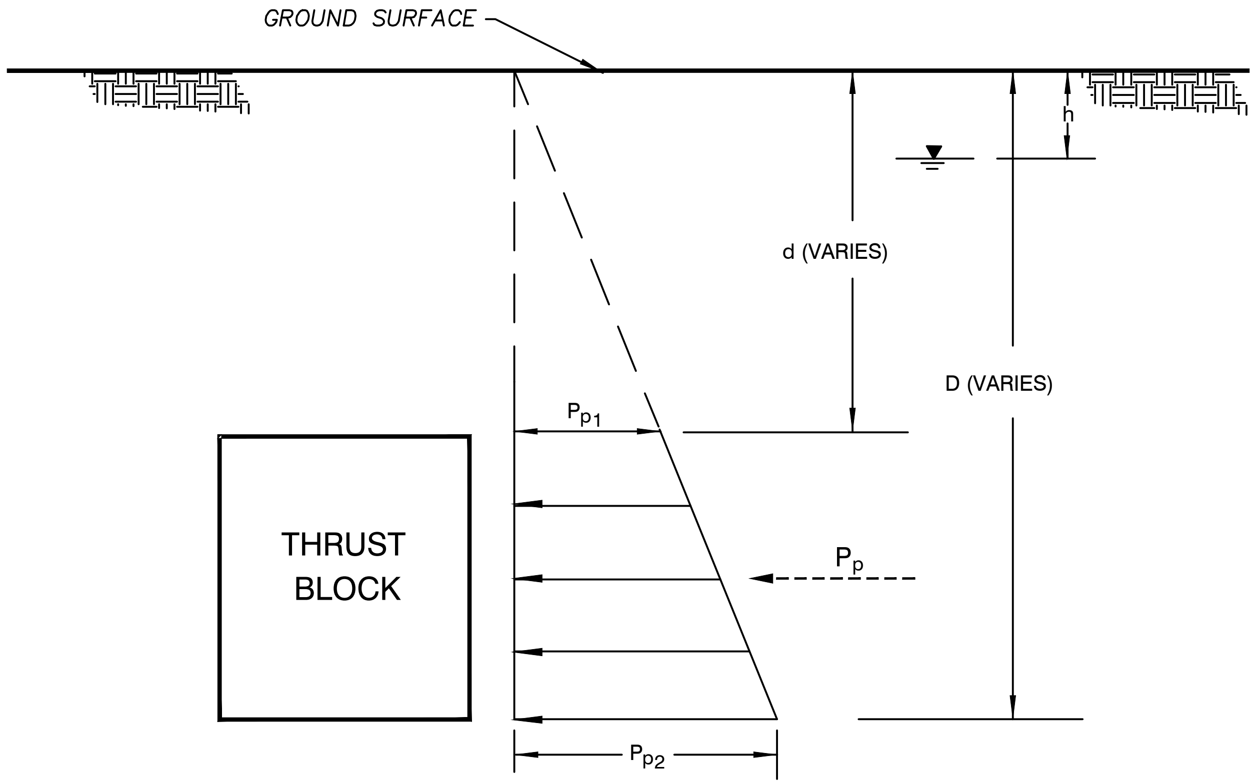
PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19



NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 150\ (D^2 - d^2)$ lb/ft
2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.5\ (D - d)[124.8\ h + 57.6\ (D+d)]$ 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

FIGURE 6

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

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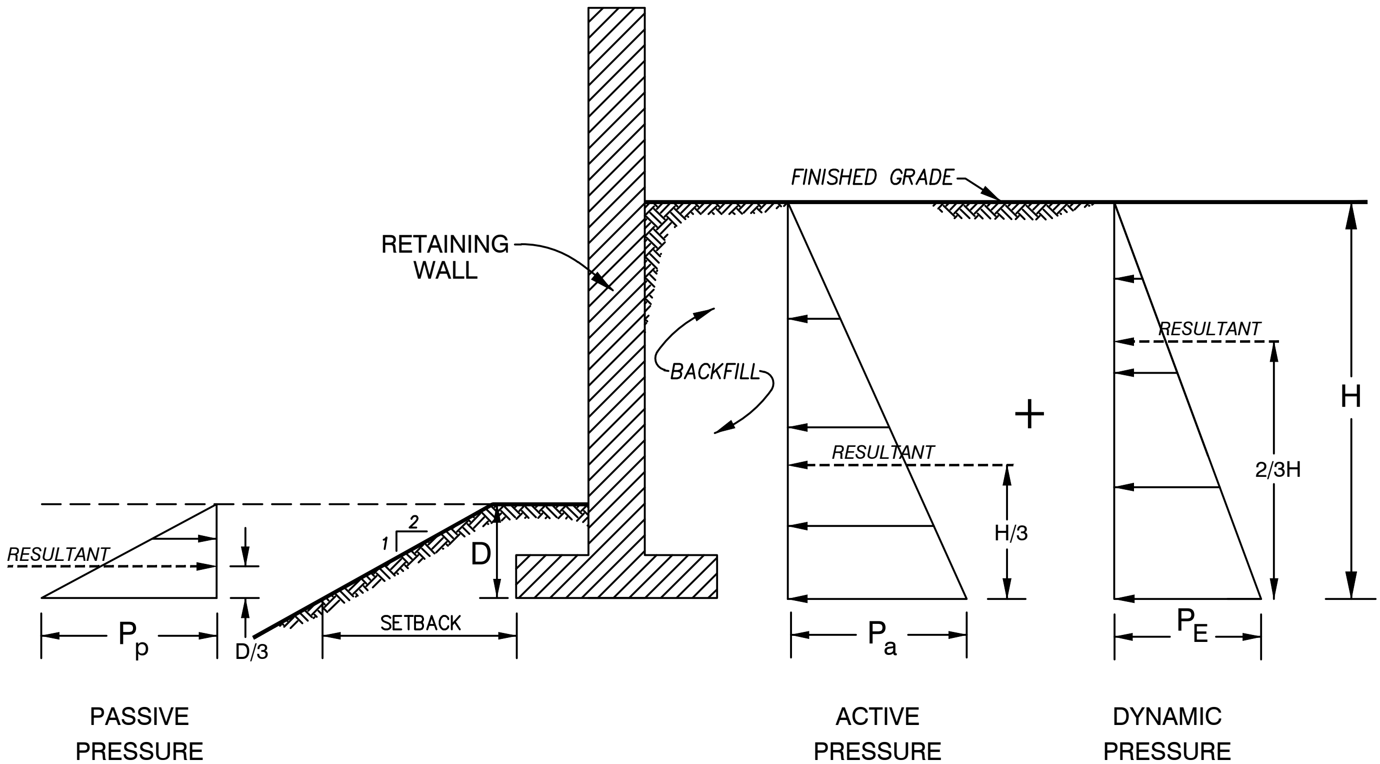
PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19



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 0.39g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH THE CBC

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)
	45 H	85 H
P_E	18 H	
P_p	Level Ground	2H:1V Descending Ground
	300 D	140 D

NOT TO SCALE

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

7_108775001_D-LEPY.DWG

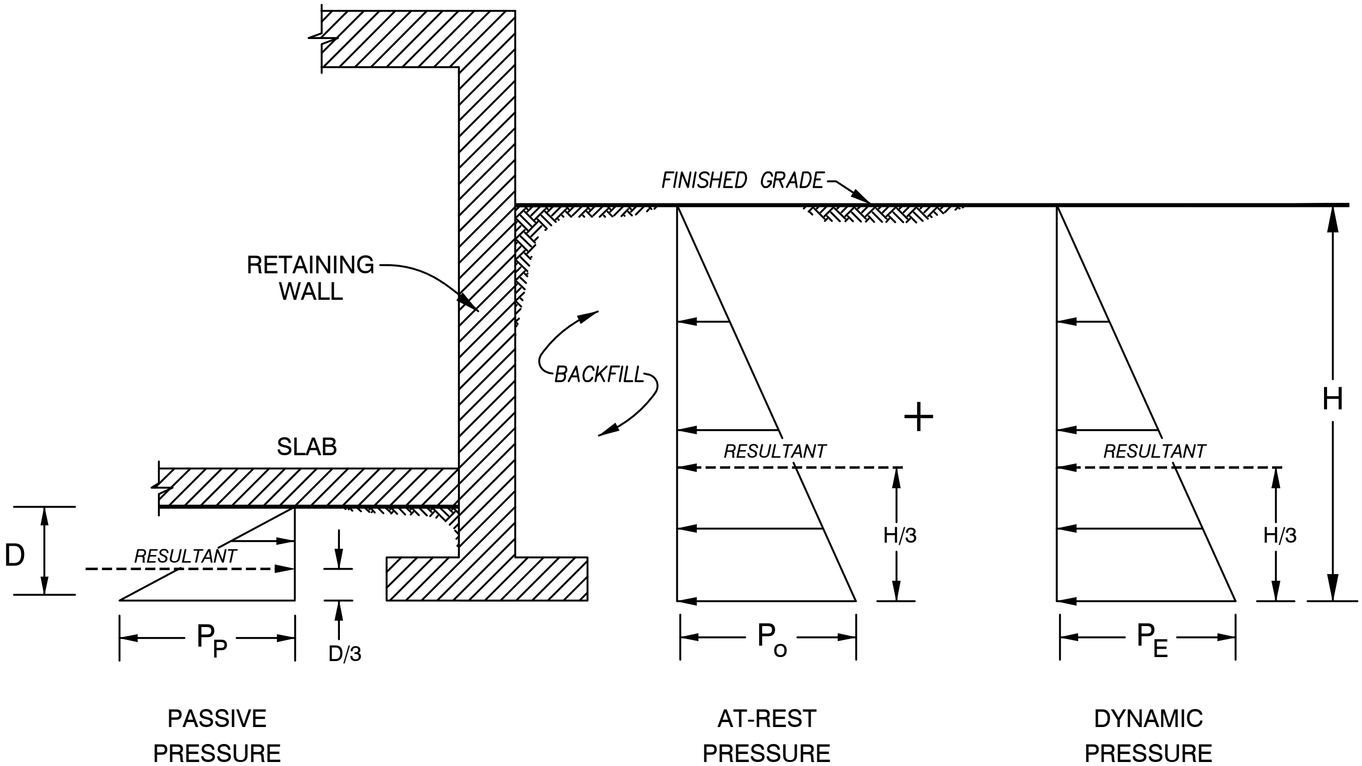
FIGURE 7

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108775001 I 5/19



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 0.39g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)[1]	
P_O	Level Backfill with Granular Soils [2]	2H:1V Sloping Backfill with Granular Soils [2]
	65 H	110 H
P_E	18 H	
P_P	Level Ground	2H:1V Descending Ground
	300 D	140 D

NOT TO SCALE

8_108775001_D-LEPR.DWG

FIGURE 8

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

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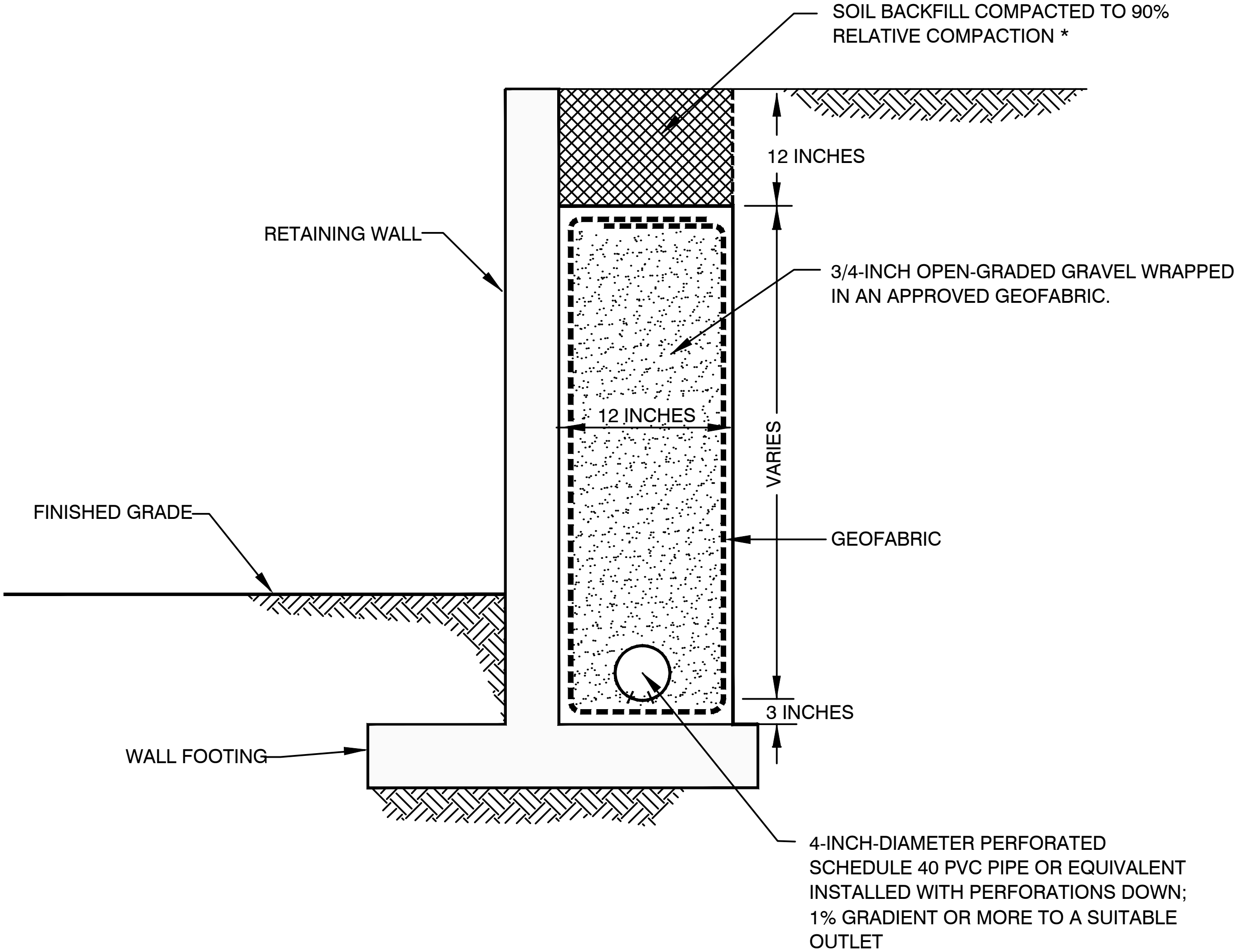
PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 I 5/19



*BASED ON ASTM D1557

9_108775001_D-RW.DWG

NOT TO SCALE

FIGURE 9

RETAINING WALL DRAINAGE DETAIL

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

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108775001 I 5/19

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.

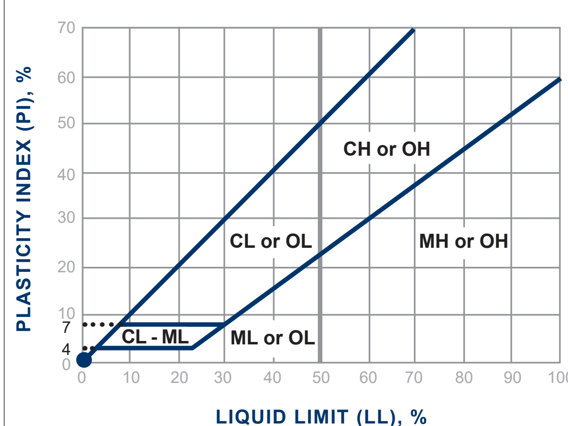
Soil Classification Chart Per ASTM D 2488

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

Grain Size

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

Plasticity Chart



Apparent Density - Coarse-Grained Soil

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

Consistency - Fine-Grained Soil

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

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USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

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

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BORING LOG

DATE DRILLED 4/24/2019 BORING NO. NM-1

GROUND ELEVATION 350' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)

DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"

SAMPLED BY GSW LOGGED BY GSW REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Brown to dark brown, moist, stiff, sandy CLAY; few gravel.
			18	20.6	100.4		CL	OLDER ALLUVIUM: Light to dark brown, moist, stiff, sandy CLAY. At 8 feet; harder drilling.
10			50/5" 50/3"					GRANITIC ROCK: Olive brown to yellowish brown, moist, moderately to highly weathered, GRANITIC ROCK, micaceous. Hard drilling at 14 feet. Grayish brown.
20								Total Depth = 14.8 feet. (Refusal) Groundwater not encountered during drilling. Backfilled shortly after drilling on 4/24/19. 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 the evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

DATE DRILLED	4/24/2019	BORING NO.	NM-2
GROUND ELEVATION	350' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)		
DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"
SAMPLED BY GSW	LOGGED BY GSW	REVIEWED BY	CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown to dark brown, moist, medium dense, clayey SAND; few gravel.
			23	19.5	103.9		CL	OLDER ALLUVIUM: Gray to light brown, moist, very stiff, sandy CLAY; slightly micaceous. Increase in moisture content.
10			50/4"					GRANITIC ROCK: Olive brown, moist, moderately to highly weathered, GRANITIC ROCK; iron-oxide staining present.
			70					Yellowish brown to gray; slightly weathered.
			50/3"					No recovery with SPT.
20								Total Depth = 18.8 feet. (Refusal) Groundwater not encountered during drilling. Backfilled shortly after drilling on 4/24/19. 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 the 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
0							CL	FILL: Reddish brown, moist, firm to stiff, silty, sandy CLAY.
							CL	OLDER ALLUVIUM: Brown, damp, stiff, sandy CLAY.
5			13	10.2	98.5			
							SM	DECOMPOSED GRANITIC ROCK: Brown, moist, very dense, silty, fine to coarse SAND; remnant grain structure.
10			50/5"					
15			50					
			40					
20								

DATE DRILLED 7/02/07 BORING NO. B-1

GROUND ELEVATION 350' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS)

DRIVE WEIGHT 140 LBS. (AUTO-TRIP HAMMER) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-1

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 7/02/07 BORING NO. B-1 GROUND ELEVATION 350' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS) DRIVE WEIGHT 140 LBS. (AUTO-TRIP HAMMER) DROP 30" SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI **DESCRIPTION/INTERPRETATION**
20								Total Depth = 20 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with hydrated bentonite shortly after drilling on 7/02/07.
25								
30								
35								
40								

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-2

DATE DRILLED	7/02/07	BORING NO.	B-2	
GROUND ELEVATION	350' ± (MSL)	SHEET	1	OF 1
METHOD OF DRILLING	8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS)			
DRIVE WEIGHT	140 LBS. (AUTO-TRIP HAMMER)	DROP	30"	
SAMPLED BY	MAH	LOGGED BY	MAH	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, moist, medium dense, silty SAND; trace clay.
	■						CL	OLDER ALLUVIUM: Brown, moist, firm to stiff, silty, sandy CLAY; few scattered gravel.
5		■	32					
							SM	DECOMPOSED GRANITIC ROCK: Brown, moist, very dense, clayey, silty fine to coarse SAND; micaceous.
10		■	50/5"					
								Total Depth = 10.4 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with hydrated bentonite shortly after drilling on 7/02/07.
15								
20								

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	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.
							CL	FILL: Reddish brown, damp, stiff, sandy CLAY.
5			20	9.7	114.0			
10			50					
							SM	DECOMPOSED GRANITIC ROCK: Brown, moist, very dense, silty, fine to coarse SAND.
								Total Depth = 11.6 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with hydrated bentonite and soil shortly after drilling on 7/02/07.
15								
20								

DATE DRILLED 7/02/07 BORING NO. B-3

GROUND ELEVATION 350' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS)

DRIVE WEIGHT 140 LBS. (AUTO-TRIP HAMMER) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-4

DATE DRILLED 7/02/07 BORING NO. B-4

GROUND ELEVATION 350' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS)

DRIVE WEIGHT 140 LBS. (AUTO-TRIP HAMMER) DROP 30"

SAMPLED BY MAH LOGGED BY MAH 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: Light brown, damp, medium dense, clayey, silty SAND.
			20	17.0	109.5		CL	OLDER ALLUVIUM: Reddish brown, moist, very stiff, sandy CLAY; few scattered gravel.
5								
			50/6"				SM	DECOMPOSED GRANITIC ROCK: Reddish brown, damp, very dense, silty fine to coarse SAND; remnant grain structure.
10			50/11"					
								Tough drilling.
15			95/10"					
								Total Depth = 15.4 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with hydrated bentonite shortly after drilling on 7/02/07.
20								

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-5

DATE DRILLED 7/02/07	BORING NO. B-5
GROUND ELEVATION 350' ± (MSL)	SHEET 1 OF 1
METHOD OF DRILLING 8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS)	
DRIVE WEIGHT 140 LBS. (AUTO-TRIP HAMMER)	DROP 30"
SAMPLED BY MAH LOGGED BY MAH	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: Reddish brown, damp, medium dense, silty, fine to medium SAND; trace clay; micaceous.
	X						CL	OLDER ALLUVIUM: Grayish brown, moist, firm to stiff, CLAY; trace fine sand.
5		X	25					
								Sandy.
10		X	50/6"				SM	DECOMPOSED GRANITIC ROCK: Brown, moist, very dense, silty fine to coarse SAND; remnant grain structure.
								Tough drilling.
15								Total Depth = 14.5 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with hydrated bentonite shortly after drilling on 7/02/07.
20								

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-6

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

DATE DRILLED 6/26/07 BORING NO. B-6

GROUND ELEVATION 350' ± (MSL) 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

DESCRIPTION/INTERPRETATION

Depth (feet)	Classification U.S.C.S.	Description/Interpretation
0		CONCRETE: Approximately 3.5 inches thick.
	SM	FILL: Light brown, moist, medium dense, silty SAND; micaceous; trace clay.
	CL	OLDER ALLUVIUM: Brown to dark brown, moist, firm to stiff, silty CLAY.
5	SC	DECOMPOSED GRANITIC ROCK: Light brown to brown, moist, medium dense to dense, clayey SAND.
	SM	Light brown, moist, medium dense to dense, silty SAND.
10		Brown.
		GRANITIC ROCK: Light brown to brown, weathered, GRANITIC ROCK.
15		Total Depth = 13.5 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with hydrated bentonite and patched with concrete shortly after drilling on 6/26/07.
20		

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-7

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 7/02/07 BORING NO. B-7 GROUND ELEVATION 350' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 8" HOLLOW STEM AUGER (CME 75 - BAJA EXPLORATIONS) DRIVE WEIGHT 140 LBS. (AUTO-TRIP HAMMER) DROP 30" SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI **DESCRIPTION/INTERPRETATION**
0							SM	FILL: Brown, damp to moist, medium dense, clayey, silty, fine to coarse SAND.
			11	14.9	110.9			Moist.
5								
							SC	OLDER ALLUVIUM: Brown, moist, medium dense to dense, clayey, fine to medium SAND.
								Total Depth = 8 feet. Groundwater was not encountered during drilling. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled shortly after drilling on 7/02/07.
10								
15								
20								

Ninyo & Moore

BORING LOG

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

PROJECT NO.	DATE	FIGURE
106116001	7/07	A-8

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 Figure B-1 from our recent evaluation and Figures B-1 and B-2 from our previous evaluation (Ninyo & Moore, 2007). The test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples from our previous evaluation (Ninyo & Moore, 2007) to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classifications in accordance with the USCS. The test results and classifications are shown on Figure B-3 from our previous evaluation (Ninyo & Moore, 2007).

Consolidation
A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test is summarized on Figure B-2.

Direct Shear Test
A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-3.

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

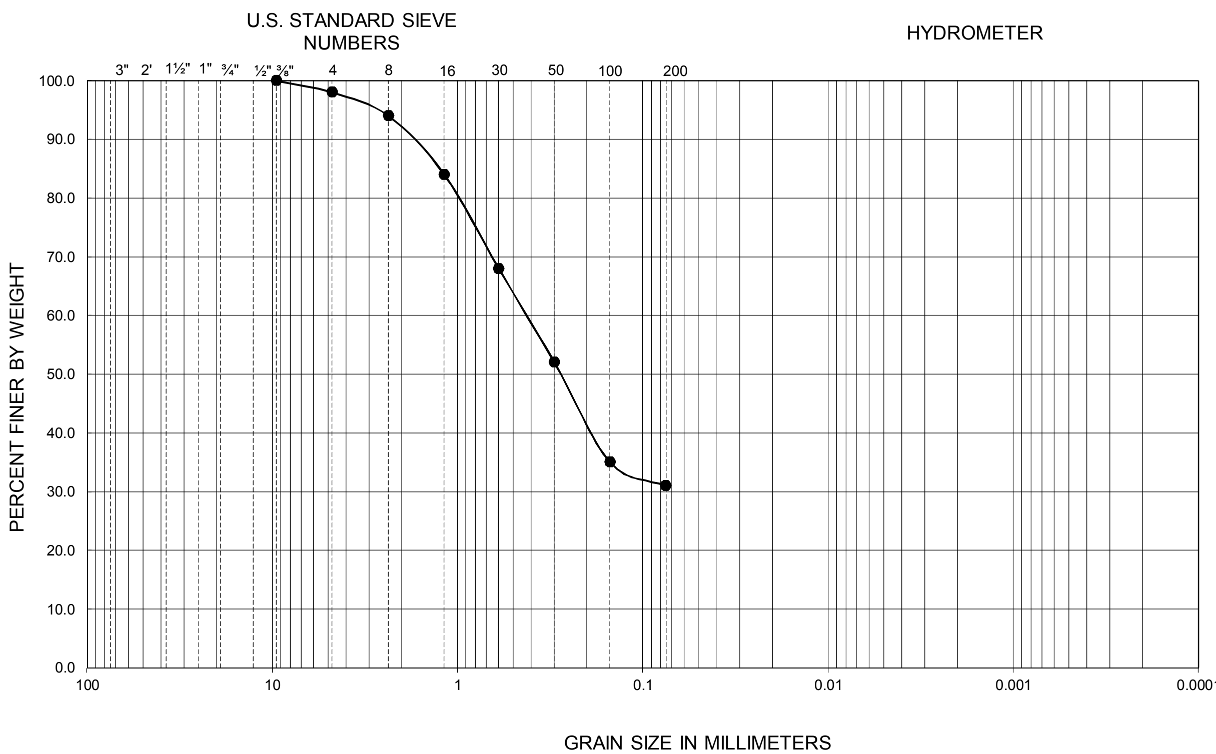
Soil Corrosivity Tests

Soil pH and electrical resistivity tests were performed on a representative sample in general accordance with CT 643. The sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-5 from our recent evaluation and Figure B-5 from our previous evaluation (Ninyo & Moore, 2007).

R-Value

The resistance value, or R-value, for site soils were evaluated in general accordance with CT 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-6.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NM-2	0.0-5.0	--	--	--	--	--	--	--	--	31	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

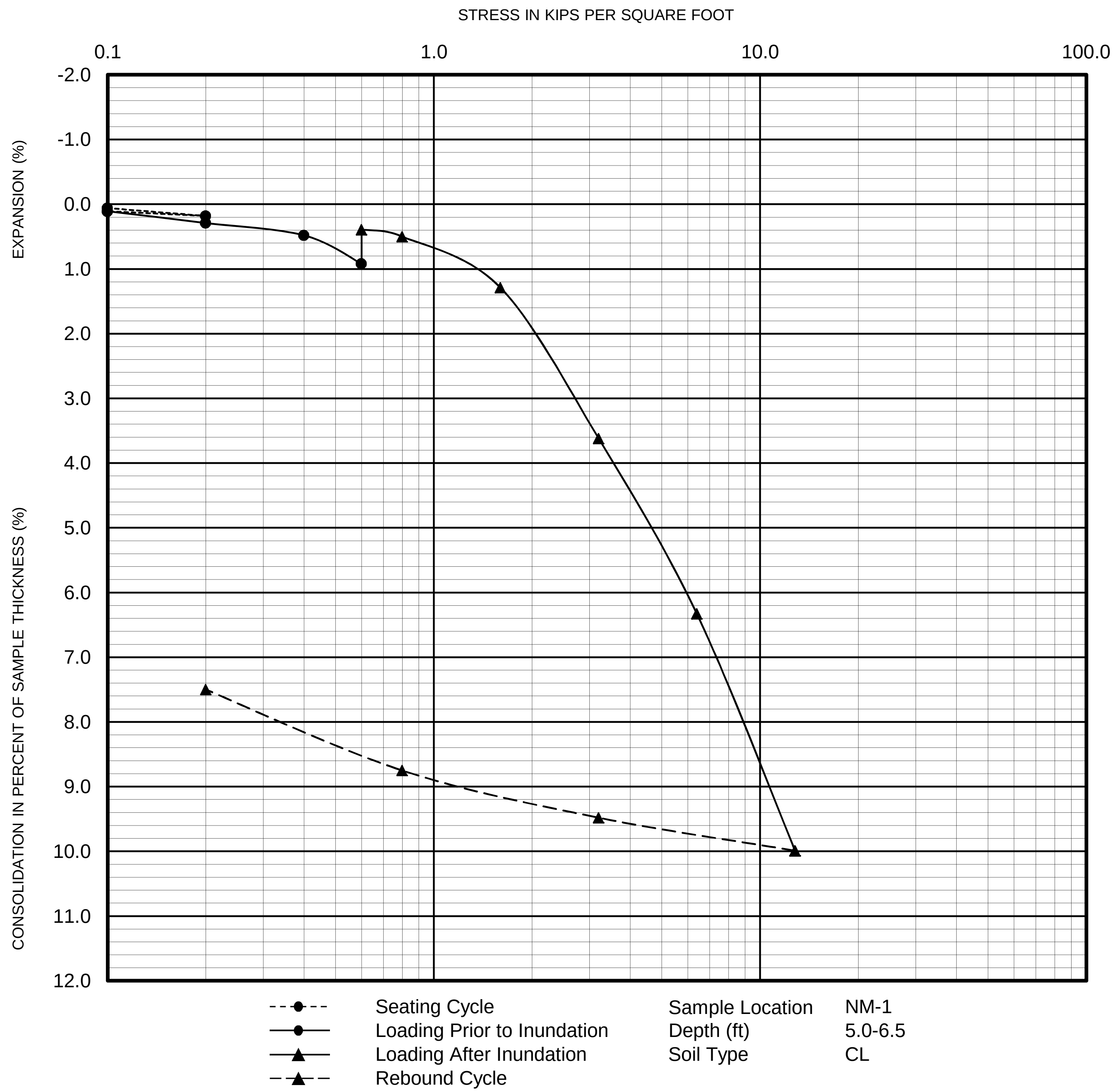
FIGURE B-1

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE , SANTEE, CALIFORNIA

108775001 | 5/19



Sample Location	NM-1
Depth (ft)	5.0-6.5
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

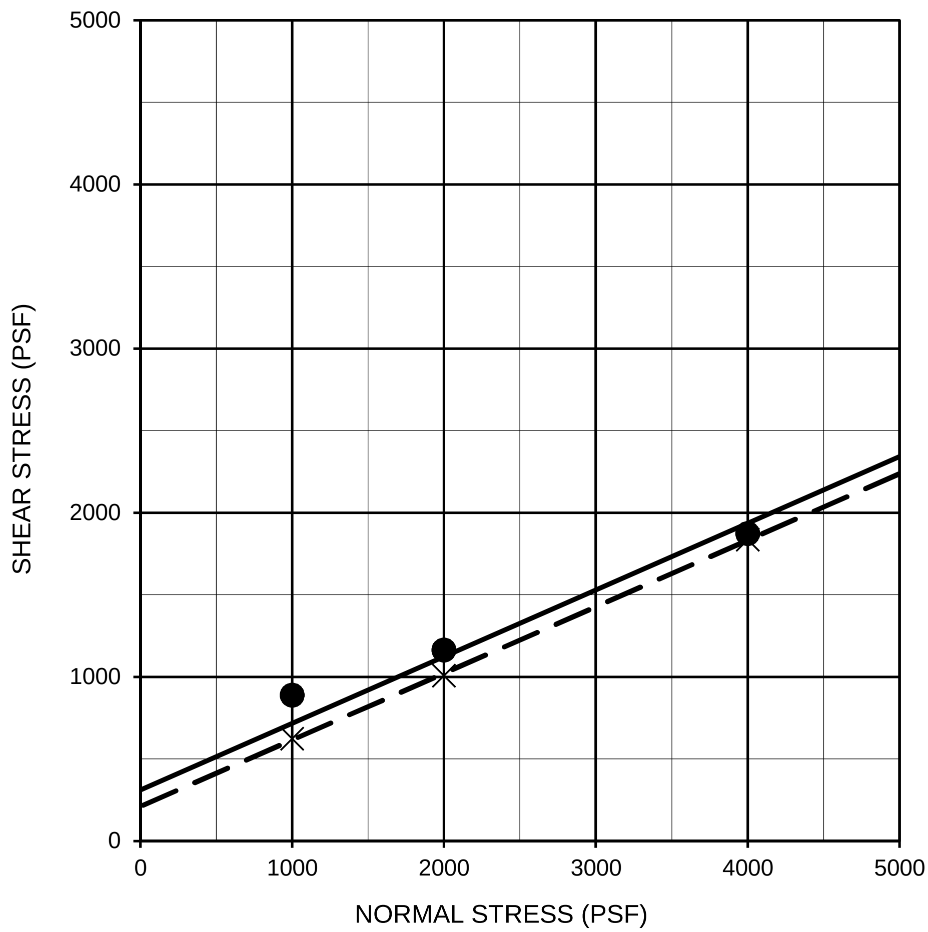
FIGURE B-2

CONSOLIDATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Sandy CLAY	——●——	NM-2	5.0-6.5	Peak	310	22	CL
Sandy CLAY	– – X – –	NM-2	5.0-6.5	Ultimate	210	22	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-3

DIRECT SHEAR TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
NM-1	0.0-5.0	11.0	106.2	22.4	0.041	41	Low
NM-2	0.0-5.0	10.5	108.3	19.1	0.053	53	Medium

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

FIGURE B-4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

EXPANSION INDEX TEST RESULTS

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
NM-1	0.0-5.0	8.1	1,000	60	0.006	225

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

FIGURE B-5

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CORROSIVITY TEST RESULTS

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
NM-2	0.0-5.0	Clayey SAND (SC)	9

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-6

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

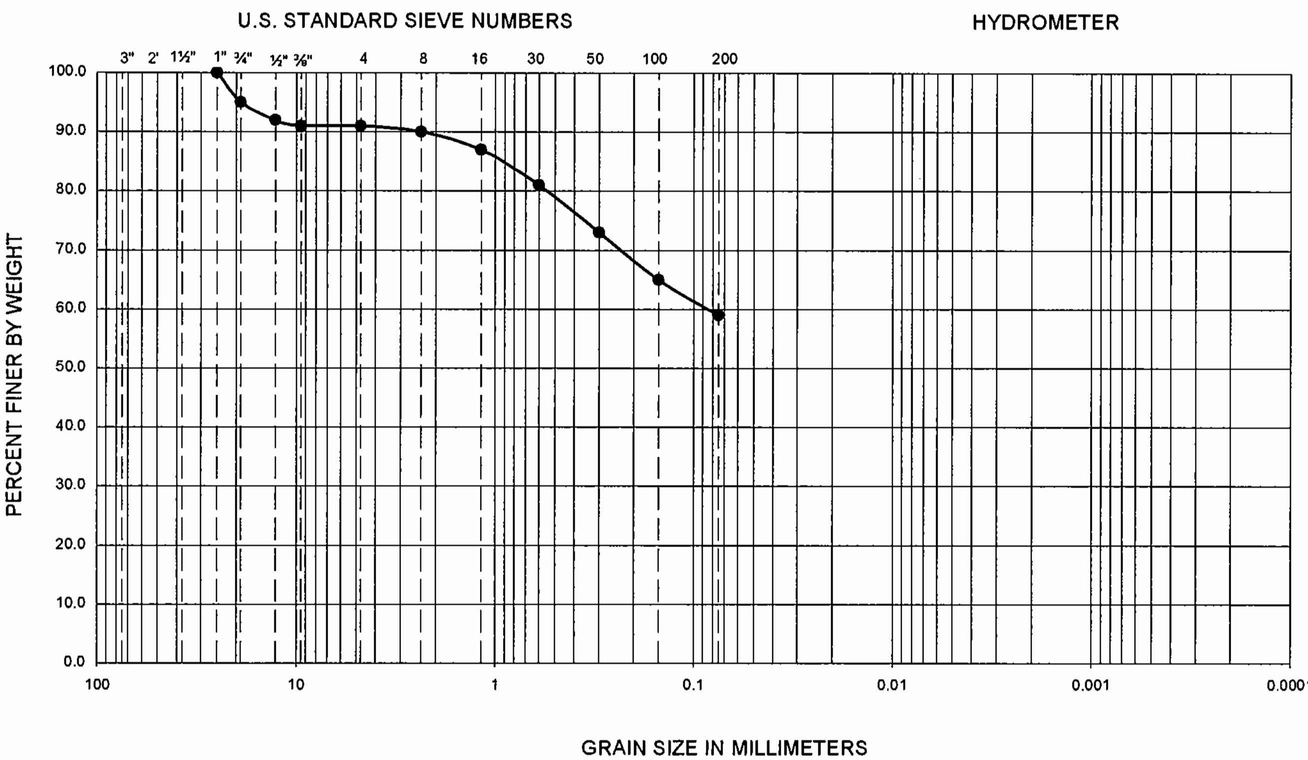
R-VALUE TEST RESULTS

PRIDE ACADEMY AT PROSPECT AVENUE SCHOOL LIBRARY ADDITION
9303 PROSPECT AVENUE, SANTEE, CALIFORNIA

108775001 | 5/19

108775001_RVTABLE1.xlsx

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



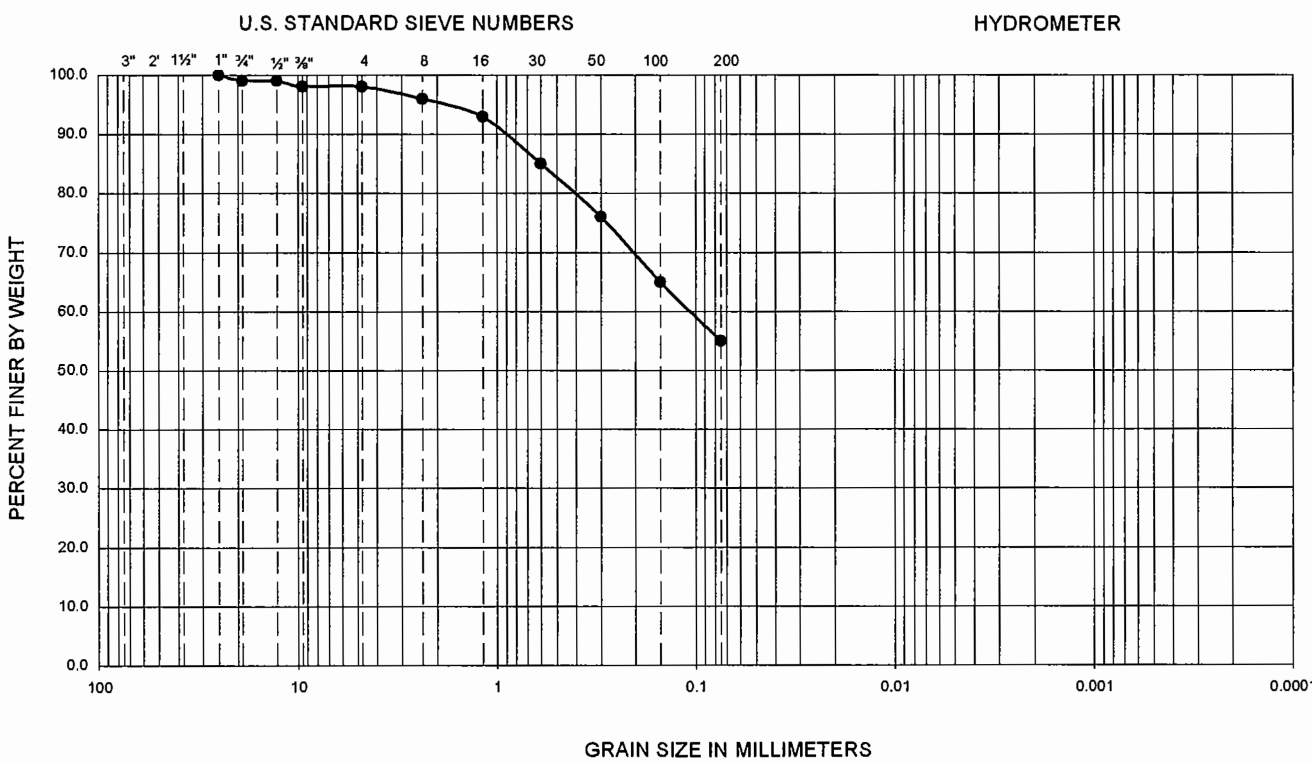
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-2	5.0-6.5	--	--	--	--	--	--	--	--	59	CL

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

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	PROSPECT AVENUE SCHOOL SANTEE, CALIFORNIA	B-1
106116001	7/07		

106116001 SIEVE B-2 @ 5.0-6.5.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

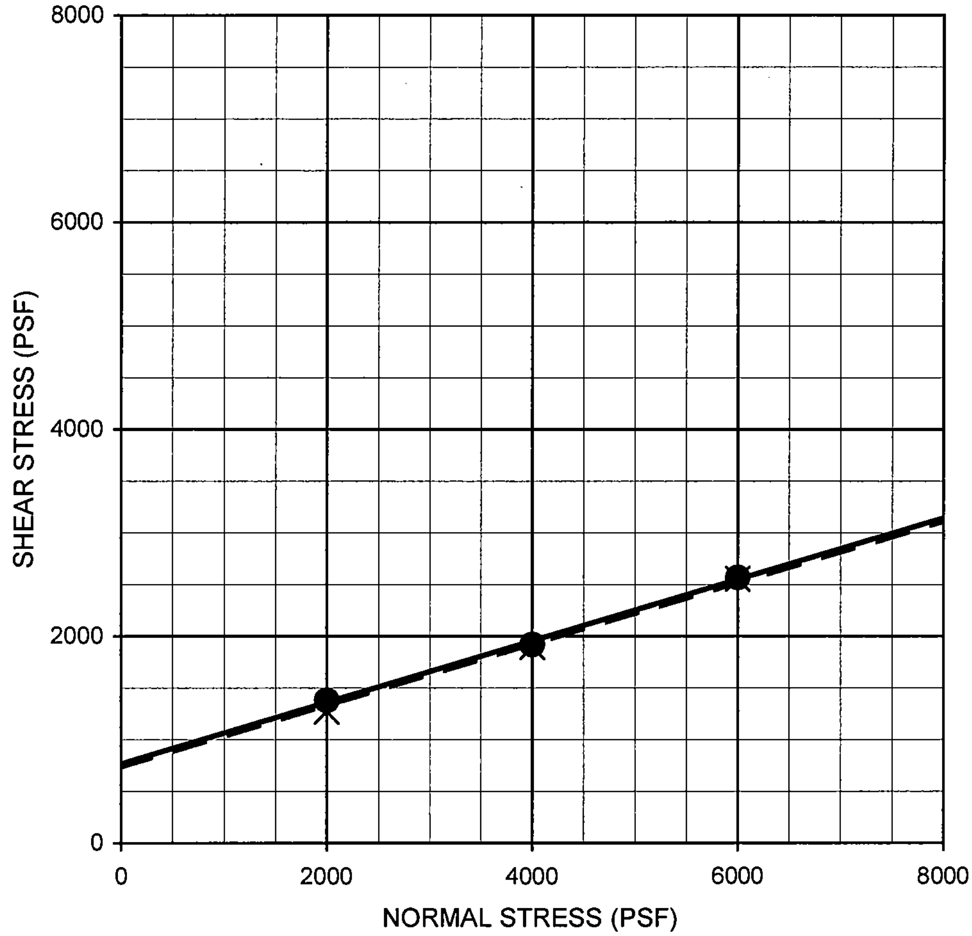


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-4	3.5-5.0	--	--	--	--	--	--	--	--	55	CL

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

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	PROSPECT AVENUE SCHOOL SANTEE, CALIFORNIA	B-2
106116001	7/07		

106116001 SIEVE B-4 @ 3.5-5.0.xls



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
CLAY	—●—	B-5	5.0-6.5	Peak	770	17	CL
CLAY	- - x - -	B-5	5.0-6.5	Ultimate	730	17	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore	DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO. 106116001 / DATE 7/07	PROSPECT AVENUE SCHOOL SANTEE, CALIFORNIA	B-3

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.0-5.0	9.5	110.8	22.6	0.033	33	Low

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

Ninyo & Moore		EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT NO.	DATE	PROSPECT AVENUE SCHOOL SANTEE, CALIFORNIA	B-4
106116001	7/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	0.0-5.0	7.4	240	900	0.090	1130

[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 NO.	DATE	PROSPECT AVENUE SCHOOL SANTEE, CALIFORNIA	B-5
106116001	7/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-3	0.0-5.0	CL	19

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844-01/CT 301

Ninyo & Moore

R-VALUE TEST RESULTS

FIGURE B-6

PROJECT NO.	DATE
106116001	7/07

PROSPECT AVENUE SCHOOL
SANTEE, CALIFORNIA

APPENDIX C

Infiltration Testing

Test Date:	4/20/2017						Infiltration Test No.:	PA-1
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	3.0
Test performed and recorded by:			GSW				Pipe Length (feet):	3.00
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)
8:25	2.50	8:50	2.52	25	0.02	104.17	0.49	0.12
8:50	2.50	9:15	2.50	25	0.00	---	0.50	DNI
9:15	2.50	9:45	2.51	30	0.01	250.00	0.50	0.05
9:45	2.50	10:15	2.51	30	0.01	250.00	0.50	0.05
10:15	2.50	10:45	2.51	30	0.01	250.00	0.50	0.05
10:45	2.50	11:15	2.51	30	0.01	250.00	0.50	0.05
11:15	2.50	11:45	2.51	30	0.01	250.00	0.50	0.05
11:45	2.50	12:15	2.51	30	0.01	250.00	0.50	0.05
12:15	2.50	12:45	2.51	30	0.01	250.00	0.50	0.05
12:45	2.50	1:15	2.51	30	0.01	250.00	0.50	0.05
1:15	2.50	1:45	2.51	30	0.01	250.00	0.50	0.05
1:45	2.50	2:15	2.51	30	0.01	250.00	0.50	0.05

Test Date:	4/20/2017						Infiltration Test No.:	PA-2
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	3.0
Test performed and recorded by:			GSW				Pipe Length (feet):	3.0
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:27	1.02	8:52	1.02	25	0.00	---	1.98	DNI
8:52	1.02	9:17	1.03	25	0.01	208.33	1.98	0.02
9:17	1.02	9:47	1.03	30	0.01	250.00	1.98	0.01
9:47	1.02	10:17	1.03	30	0.01	250.00	1.98	0.01
10:17	1.02	10:47	1.03	30	0.01	250.00	1.98	0.01
10:47	1.02	11:17	1.03	30	0.01	250.00	1.98	0.01
11:17	1.02	11:47	1.03	30	0.01	250.00	1.98	0.01
11:47	1.02	12:17	1.03	30	0.01	250.00	1.98	0.01
12:17	1.02	12:47	1.03	30	0.01	250.00	1.98	0.01
12:47	1.02	1:17	1.03	30	0.01	250.00	1.98	0.01
1:17	1.02	1:47	1.03	30	0.01	250.00	1.98	0.01
1:47	1.02	2:17	1.03	30	0.01	250.00	1.98	0.01

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

ΔH = change in depth to water or change in height of water column (i.e., $d_2 - d_1$)

in/hr = inches per hour

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

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:	4/20/2017						Infiltration Test No.:	PA-3
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	3.0
Test performed and recorded by:			GSW				Pipe Length (feet):	3.0
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:30	2.50	8:55	2.52	25	0.02	104.17	0.49	0.12
8:55	2.50	9:20	2.50	25	0.00	---	0.50	DNI
9:20	2.50	9:50	2.51	30	0.01	250.00	0.50	0.05
9:50	2.50	10:20	2.51	30	0.01	250.00	0.50	0.05
10:20	2.50	10:50	2.51	30	0.01	250.00	0.50	0.05
10:50	2.50	11:20	2.51	30	0.01	250.00	0.50	0.05
11:20	2.50	11:50	2.51	30	0.01	250.00	0.50	0.05
11:50	2.50	12:20	2.51	30	0.01	250.00	0.50	0.05
12:20	2.50	12:50	2.51	30	0.01	250.00	0.50	0.05
12:50	2.50	1:20	2.51	30	0.01	250.00	0.50	0.05
1:20	2.50	1:50	2.51	30	0.01	250.00	0.50	0.05
1:50	2.50	2:20	2.51	30	0.01	250.00	0.50	0.05

Test Date:	4/20/2017						Infiltration Test No.:	PA-4
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	3.0
Test performed and recorded by:			GSW				Pipe Length (feet):	3.0
t_1	d_1 (feet)	t_2	d_2 (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H_{avg} (feet)	Infiltration Rate (in/hr)
8:31	2.50	8:56	2.53	25	0.03	69.44	0.49	0.18
8:56	2.50	9:21	2.51	25	0.01	208.33	0.50	0.06
9:21	2.50	9:51	2.51	30	0.01	250.00	0.50	0.05
9:51	2.50	10:21	2.51	30	0.01	250.00	0.50	0.05
10:21	2.50	10:51	2.51	30	0.01	250.00	0.50	0.05
10:51	2.50	11:21	2.52	30	0.02	125.00	0.49	0.10
11:21	2.50	11:51	2.51	30	0.01	250.00	0.50	0.05
11:51	2.50	12:21	2.52	30	0.02	125.00	0.49	0.10
12:21	2.50	12:51	2.51	30	0.01	250.00	0.50	0.05
12:51	2.50	1:21	2.51	30	0.01	250.00	0.50	0.05
1:21	2.50	1:51	2.51	30	0.01	250.00	0.50	0.05
1:51	2.50	2:21	2.51	30	0.01	250.00	0.50	0.05

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

ΔH = change in depth to water or change in height of water column (i.e., $d_2 - d_1$)

in/hr = inches per hour

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

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 x v
A	Suitability Assessment	Soil assessment methods	0.25	2	0.5
		Predominant soil texture	0.25	3	0.75
		Site soil variability	0.25	2	0.5
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			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:

Four infiltration tests were performed at the site, PA-1 through PA-4 . In-situ rates (i.e., unfactored rates) ranged from 0.01 to 0.05 inches per hour for PA-1 through PA-4.
PA-1 through PA-4 were each performed at a depth of approximately 3 feet below existing grade in fill material consisting of sandy clay.
Additional information regarding infiltration testing performed at the site 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: Four infiltration tests were performed at the site, PA-1 through PA-4 . In-situ rates (i.e., unfactored rates) ranged from 0.01 to 0.05 inches per hour for PA-1 through PA-4. PA-1 through PA-4 were each performed at a depth of approximately 3 feet below existing grade in fill material consisting of sandy clay. 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 0.03 inches per hour or less. 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.			

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Criteria	Screening Question	**Yes**	**No**
3	**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.		X
Provide basis: Reliable infiltration rates (i.e., factored) of 0.03 inches per hour or less. 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.			
4	**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.		X
Provide basis: Reliable infiltration rates (i.e., factored) of 0.03 inches per hour or less, 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.			
Part 1 Result*	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	NO	

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

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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 0.03 inches per hour or less. 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 may be located 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.			

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Criteria	Screening Question	**Yes**	**No**
7	**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.	X	
Provide basis: Groundwater is anticipated to be approximately 30 feet below existing grade. 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.			
8	**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.		X
Provide basis: Santee Lakes are located approximately 0.20 miles northeast of the project site. Pride Academy is located up-gradient from the Forester Creek. 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.			
Part 2 Result*	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.**	No infiltration	

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

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