Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

April 17, 2020
Project No. 108464009

Mr. Eric Berg
San Diego County Office of Education
6401 Linda Vista Road, #506
San Diego, California 92111

Subject: Addendum to Geotechnical Evaluation
Joan MacQueen Middle School Field Renovation
2001 Tavern Road
Alpine, California

Reference: Ninyo & Moore, 2020, Geotechnical Evaluation, Joan MacQueen Middle School Field Renovation, 2001 Tavern Road, Alpine, California, Project No. 108464009: dated February 17.

Dear Mr. Berg:

In accordance with your request, we have prepared this addendum letter to our referenced geotechnical evaluation report (Ninyo & Moore, 2020). The purpose of this addendum is to provide updated recommendations for Section 10 - Turf Field Infiltration of the referenced report. Based on our discussions with you, we understand that the design documents are to include the performance of an overexcavation of the upper 1 foot of the existing subgrade soils beneath the planned artificial turf field. The overexcavation will include the removal of the upper 1 foot of existing subgrade soils and replacement with quarry-derived Caltrans Class 2 aggregate base materials. The described overexcavation and replacement of subgrade soils with Class 2 aggregate base materials meets and exceeds the remedial grading recommendations presented in Section 9.1.4 - Remedial Grading – Turf Field, Flatwork, and ADA Ramp of the referenced report. Based on the noted use of Caltrans Class 2 aggregate base materials, we are providing the following updated section for the referenced report. The other geotechnical conclusions and recommendations in the referenced report remain valid and applicable.

TURF FIELD INFILTRATION

As discussed in the geotechnical evaluation report (Ninyo & Moore, 2020), the onsite soils in the field area possess poor infiltration characteristics with factored infiltration rates of 0.02 inches per hour, or less. In addition, onsite soils possess a very low to medium potential for expansion. Such conditions may adversely impact the proposed artificial turf field. To mitigate the potential effects of the very low to medium potential of the underlying soils, the upper 1 foot of subgrade soils are to be overexcavated and replaced with compacted Caltrans Class 2 aggregate base materials. Subsequent to placement and compaction of the aggregate base materials, a non-woven filter fabric (such as Mirafi 140N or equivalent) should be placed on top of the compacted aggregate base in accordance with the manufacturer's recommendations. Additionally, we recommend that the drainage for the artificial turf field system incorporate an overflow pipe that is connected to an appropriate outlet.

We appreciate the opportunity to be of service.

Respectfully submitted,
NINYO & MOORE

Nissa Morton, PG, CEG
Project Geologist

Jeffrey T. Kent, PE, GE
Principal Engineer

NMM/JTK/gg

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

Geotechnical Evaluation

Joan MacQueen Middle School Field Renovation
2001 Tavern Road
Alpine, California

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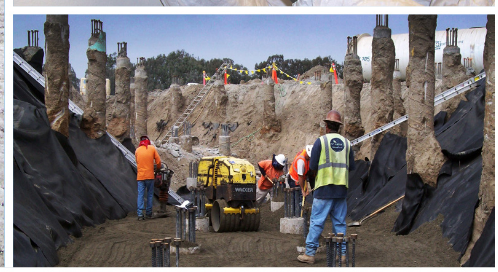

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CERTIFIED ENGINEERING GEOLOGIST
NISSA M. MORTON
No. 2655
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JEFFREY T. KENT
No. 2817
GEOTECHNICAL
STATE OF CALIFORNIA

Nissa M. Morton, PG, CEG
Project Geologist

Jeffrey T. Kent, PE, GE
Principal Engineer

NMM/JTK/gg

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

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1 INTRODUCTION

In accordance with your approval and our proposal dated December 11, 2019, we have performed a geotechnical evaluation for the proposed Field Renovation project at the Joan MacQueen Middle School campus. The existing Joan MacQueen Middle School campus is located at 2001 Tavern Road in Alpine, California (Figure 1). 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 included the following:

- Reviewing readily available published and in-house geotechnical literature, topographic maps, geologic maps, fault maps, and stereoscopic aerial photographs.
- Performing a field reconnaissance to observe existing site conditions and to mark the locations of our exploratory borings.
- Reviewing available maps to locate underground utilities near our exploratory borings. Additionally, we notified Underground Service Alert (USA).
- Performing a subsurface exploration consisting of the excavating, logging, and sampling of three exploratory borings using a truck-mounted drill rig and manual techniques. Bulk soil samples were obtained at selected intervals from the borings. The collected samples were transported to our in-house geotechnical laboratory for testing.
- Performing infiltration tests in two of our borings within the field area to evaluate the infiltration rates of the underlying near-surface soils.
- Performing geotechnical laboratory testing on representative soil samples to evaluate design parameters and soil characteristics.
- Compiling and performing an engineering analysis of the data obtained from our background review, field activities, and geotechnical laboratory testing.
- Preparing this geotechnical report presenting our findings, conclusions, and recommendations regarding the geotechnical aspects of the design and construction of this project.

3 SITE AND PROJECT DESCRIPTION

The project site is situated within the existing Joan MacQueen Middle School campus in Alpine, California (Figure 1). The campus is located on a generally rectangular-shaped parcel bounded by Tavern Road to the west, White Oak Drive to the south, undeveloped open space to the east, and residential properties to the north. The school site generally consists of various classroom and administrative buildings, athletic fields, landscaping, and parking lots. The northern portion of the campus primarily houses the school buildings and parking lots. The southern portion of the school campus is primarily asphalt concrete (AC) covered play courts and athletic fields, including an approximately 2-acre decomposed granite (DG) field (Figure 2). Topography at the southern portion of the school campus generally steps down to the west between the relatively level field and playcourt areas. Elevations in these areas generally range from approximately 1,820 feet above mean sea level (MSL) in the southwest corner of the campus adjacent to Tavern Road up to approximately 1,860 feet above MSL on the northeast corner of the athletic fields (Snipes-Dye Associates, 2000). The global coordinates of the project site are approximately 32.8236°N Latitude and 116.7744°W Longitude.

Based on our correspondence with the client, we understand that the field renovation project will include the installation of artificial turf within the DG field on the southern portion of the school campus. The project will also include the construction of an American with Disabilities Act (ADA) access ramp connecting the DG field to the adjacent play court. The ADA access ramp is planned at the northwest end of the DG field.

4 SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on January 16 and 17, 2020 and included excavating, logging, and sampling of three small-diameter borings (B-1, IT-1, and IT-2). Prior to commencing the subsurface exploration, the locations were cleared of underground utility conflicts by Underground Service Alert. Borings B-1 and IT-2 were excavated using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers to depths of approximately 11 ½ and 4 feet, respectively. Boring IT-1 was manually excavated using a 6-inch diameter hand auger to a depth of approximately 4 feet. Drilling refusal was encountered with the drill rig in boring IT-2. 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 soil 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 exploratory borings are shown on Figure 2. Logs of the 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 moisture content and dry density, shear strength, expansion index, and soil corrosivity. The results of the in-situ dry density and moisture content tests are presented at the corresponding depths on the boring logs in Appendix A. The results of the other laboratory tests that we performed and a description of the test procedures used are presented in Appendix B.

6 INFILTRATION TESTING

Field infiltration testing was performed on January 16 and 17, 2020 at two locations within the DG field. On January 16, 2020, the infiltration test holes (borings IT-1 and IT-2) were drilled with a truck-mounted drill rig equipped with 8-inch diameter augers and manually excavated with a 6-inch diameter hand auger to depths up to approximately 4 feet at the locations shown on Figure 2. The infiltration tests were performed in general accordance with the County of San Diego BMP Design Manual (2019). Approximately 2 inches of gravel was placed on the bottom of each prepared boring. A 2-inch diameter, perforated PVC pipe was installed in the boring and the annulus was then backfilled with pea gravel. As part of the test procedure, presoaking of each hole was performed on January 16, 2020 to represent adverse conditions for infiltration. The presoak was accomplished by percolating approximately 5 gallons of water through the test hole. The water level was allowed to drop overnight and testing commenced the following morning. Infiltration testing was then performed in the presoaked test borings on January 17, 2020. The infiltration test holes were filled with 24 inches of water or more, if needed. Measurements of the water depth after infiltration were recorded every thirty minutes. As necessary, the borings were refilled to maintain the water level until the infiltration rate stabilized.

6.1 Infiltration Test Results

Infiltration rates were calculated using the Porchet method. Infiltration test results and calculations are included in Appendix C and summarized in Table 1 below. Per the County of San Diego BMP Design Manual Appendix D Tables D.2-3 and D.2-4, and on the results of our evaluation, a suitability factor of safety (FOS), of 2 is appropriate for the site. The estimated reliable infiltration rates presented in Table 1 are to be used for preliminary design purposes only. The rates should be corrected for the design infiltration rate after applying the design safety factor determined by the design engineer.

Table 1 – Infiltration Test Results Summary					
Infiltration Test	Test Depth (feet)	Description (Geologic Unit)	Observed In-Situ Infiltration Rate (in/hr)	Factor of Safety[1]	Estimated Reliable/Factored Infiltration Rate[1] (in/hr)
IT-1	4.0	Clayey Sand (Fill)	0.04	2.0	0.02
IT-2	4.0	Clayey Sand and Sandy Clay (Fill and Colluvium)	0.01	2.0	≤ 0.01

Notes:
in/hr = inches per hour
[1] Factor of safety of 2.0 used in accordance with Appendix D of the County of San Diego BMP Design Manual (2019).

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 (Figure 3). 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 encountered during our subsurface exploration included fill soils and colluvium. The site is mapped as being underlain by the Cretaceous-age Lusardi Formation (Todd, 2004; Figure 4). Generalized descriptions of the earth units encountered during our subsurface exploration and those mapped at the site are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the boring logs in Appendix A.

7.2.1 Fill

Fill materials were encountered in our borings at the ground surface or underlying the AC pavement and extending to depths of approximately 5½ feet. As encountered, the fill material generally consisted of various shades of brown, moist, medium dense, silty and clayey sand. Scattered amounts of gravel and cobbles were encountered in the fill soils.

7.2.2 Colluvium

Colluvium was encountered underlying the fill materials in borings B-1 and IT-2 and extended the total depths explored. As encountered, the colluvium generally consisted of dark gray to black, moist, very stiff sandy clay. Scattered amounts of organic material, gravel, and cobbles were encountered in the colluvium. Refusal to further drilling due to the presence of cobbles was encountered within the colluvium in boring IT-2.

7.2.3 Lusardi Formation

While not encountered in our subsurface exploration, the site is mapped as being underlain by the Cretaceous-age Lusardi Formation. The Lusardi Formation is anticipated to consist of brown and gray, moderately to strongly cemented, silty and clayey sandstone and cobble conglomerate.

7.3 Groundwater

Groundwater was not encountered in our borings. Based on review of available data and topographic maps, groundwater is anticipated to be at depths greater than 50 feet. Fluctuations in the groundwater level and perched water conditions may occur due to variations in ground surface topography, subsurface geologic conditions and structure including the geologic contact between fill and underlying materials, rainfall, irrigation, and other factors. Additionally, perched water conditions may be present at the site due to the presence of trench backfill and bedding materials for underground utilities, as these materials tend to act as a conduit for water and perched water conditions.

7.4 Flood Hazard

Based on review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map for the area (2012), the site is not located within mapped floodplains, flood zones, or active floodways. The site is also not located within a mapped dam inundation area (CDWR, 2020). Based on this review and our reconnaissance, the potential for inundation from dam releases and significant flooding at the site are not design considerations.

7.5 Landsliding

Based on our review of referenced geologic maps, literature, topographic maps, and stereoscopic aerial photographs, as well as our subsurface evaluation, 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, 2007). However, like the majority of Southern California, the site is located in a seismically active area and the potential for strong ground motion is considered significant during the design life of the proposed structure. Figure 3 shows the approximate site location relative to the major faults in the region. The nearest known active fault is the Julian segment of the Elsinore fault, located approximately 21 miles northeast 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, 2020).

Table 2 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)[1]	Maximum Moment Magnitude (M_{max})
Elsinore (Julian Segment)	21 (34)	7.4
Rose Canyon	24 (38)	6.9
Earthquake Valley	26 (41)	6.8
Elsinore (Coyote Mountain Segment)	28 (44)	6.9
Coronado Bank	35 (57)	7.4
Elsinore (Temecula Segment)	38 (61)	7.1
San Jacinto (Coyote Creek Segment)	42 (67)	7.0
San Jacinto (Borrego Segment)	42 (68)	6.8

Table 2 – Principal Active Faults		
Fault	**Approximate Fault-to-Site Distance miles (kilometers)[1]**	**Maximum Moment Magnitude (M_{max})**
Newport-Inglewood (Offshore Segment)	44 (71)	7.0
San Jacinto (Clark Segment)	46 (75)	7.1
San Jacinto (Anza Segment)	48 (78)	7.3
San Jacinto (Superstition Mountain Segment)	50 (80)	6.7

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 50 miles (80 kilometers) or the site with a magnitude 6.0 or greater.

Table 3 – Historical Earthquakes that Affected the Site		
Date	**Magnitude (M)**	**Approximate Epicentral Distance miles (kilometers)**
May 27, 1862	6.2	29 (46)
February 9, 1890	6.8	48 (78)
May 28, 1892	6.5	42 (68)
October 23, 1894	6.1	2 (4)
October 21, 1942	6.4	46 (74)
March 19, 1954	6.3	47 (75)
April 9, 1968	6.6	45 (73)

The 2019 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the site was calculated as 0.38g using a web-based seismic design tool (SEAOC/OSHPD, 2020).

The 2019 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) 7-16 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 0.41g using a web-based seismic design tool (SEAOC/OSHPD, 2020) that yielded a mapped MCE_G peak ground acceleration of 0.34g for the site and a site coefficient (F_{PGA}) of 1.20 for Site Class C.

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 of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils and non-plastic silts that are saturated by a relatively shallow groundwater table are susceptible to liquefaction. The performance of deep subsurface exploration to evaluate liquefaction and seismically induced settlement is not a part of this scope of work. However, as the school campus is not located in a mapped liquefaction hazard zone (SANGIS, 2009) and is generally underlain by relatively dense geologic units, it is our opinion that the potential for liquefaction and seismically induced settlement to occur at the subject site is low.

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. Based on the inland location and elevation of the site, the potential for a tsunami 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 is 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 project site is underlain by fill soils, colluvium, and Lusardi Formation.
- The existing fill soils and colluvium 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 within the Lusardi Formation are anticipated to encounter strongly cemented zones, concretions, cobbles, boulders, and other difficult excavation conditions. These conditions may result in the need for heavy ripping, rock wheel/saw, or core barrels for excavation/drilling efforts.
- Onsite excavations are anticipated to generate oversize material. Additional processing and handling of these materials, including screening or rock picking, should be anticipated prior to reuse as engineered fill.
- Onsite materials are generally considered suitable for reuse as engineered fill, provided they are processed to meet the recommendations provided herein.
- Groundwater was not encountered in our borings. However, perched water conditions may be encountered in such areas as existing utility trenches.
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). The closest known major active fault is the Elsinore Fault, which is located approximately 21 miles northeast of the project.
- Based on the results of our geotechnical laboratory testing presented in Appendix B, the onsite soils possess a very low to medium potential for expansion.
- Based on the results of our infiltration testing presented in Appendix C, the onsite soils in the field area possess poor infiltration characteristics.
- Based on the results of our geotechnical laboratory testing presented in Appendix B compared to the Caltrans (2019) corrosion guidelines, the onsite soils are considered corrosive.

9 RECOMMENDATIONS

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

9.1 Earthwork

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

9.1.1 Site Preparation

Site preparation should begin with the removal of flatwork, vegetation, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area.

9.1.2 Excavation Characteristics

The results of our field exploration program indicate that the project site, as presently proposed, is underlain by fill soils, colluvium, and Lusardi Formation. The fill materials and colluvium 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 extending into materials of the Lusardi Formation will encounter very difficult excavation conditions and the contractor should be prepared to utilize heavy ripping, rock wheel/saw, and/or core barrels for drilling efforts. Excavations (including utility trenches) are anticipated to generate oversize material. Additional processing and handling of these materials, including screening or rock picking, should be anticipated prior to reuse of these materials as engineered fill.

9.1.3 Temporary Excavations

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

Fill and Colluvium	*Type C*
Lusardi 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 Type C soils and 1:1 in Type B soils. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

9.1.4 Remedial Grading – Field Turf, Flatwork, and ADA Ramp

Due to the variability of soils encountered at the site, we recommend remedial grading be performed in areas where new pavements and/or flatwork is proposed. The intent of this remedial grading is to reduce differential vertical offsets and resulting trip hazards within proposed paving and flatwork areas. In the proposed field turf, concrete flatwork, and the ADA ramp areas, we recommend that the on-site soils be overexcavated to a depth of 1 foot below the planned finished surface elevation. The proposed overexcavations should extend outward horizontally 2 feet from the horizontal limits of the pavement and/or flatwork. 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 overexcavation should then be filled with engineered fill soils that possess a very low to low expansion potential (i.e., expansion index less than 50). The engineered fill should be moisture conditioned to generally above optimum moisture content and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557.

The overexcavations may generate materials with an expansion index greater than 50. These soils are not suitable for reuse within the upper 1-foot of subgrade soils beneath the proposed turf field, concrete flatwork, and the ADA ramp.

9.1.4.1 Remedial Grading Alternative - Lime-Treatment of Onsite Soils

As an alternative to removal and replacement of expansive soils under the proposed improvements, the lime treatment method may be used to improve the expansive characteristics of the existing soils to create a less expansive soil subgrade for support of the new improvements. For this operation, we recommend that the upper 1 foot of subgrade soil below the turf field, concrete flatwork, and ADA ramp be treated with lime. The following lime treatment criteria are preliminary. If this method is selected, additional laboratory testing should be performed to aid in preparation of construction specifications.

The soils to be treated should not contain rocks or clods larger than 1 inch in dimension. Each lift of soil to be treated should not exceed 12 inches in thickness and may be further limited by the mixing equipment.

High calcium quicklime with the physical and chemical properties in accordance with ASTM C 977 with the noted exception that the available lime index shall be 90 percent or more available calcium oxide (CaO) when tested in accordance with ASTM C 25-95 should be utilized. The quicklime should be added to the prepared exposed surface of the remedial excavation. Based on our laboratory testing and for preliminary design purposes, an estimate for the addition of 6 percent lime by weight of the dry material may be added to the existing site soils to produce subbase materials with plasticity index less than 20. Lime should be applied in separate 3 percent applications and allowed to “mellow” between applications. However, further laboratory testing should be performed to during construction.

The treated subgrade shall be mixed while introducing water into the soil through the metering/pump device on the mixer. Water shall be added to the subgrade during mixing to provide a moisture content of 3 percent or more above the optimum moisture of the soil-lime mixture to chemical action of the lime and soil. The soil-lime mixture shall be allowed to cure or "mellow" above the optimum moisture content in an uncompacted state prior to secondary mixing, pulverization, and compaction.

The moisture content of the mixture should be 2 percent or more above over the optimum moisture content at the time of compaction. The field dry density of the compacted mixture should be 95 percent or more of relative compaction in accordance with ASTM D 1557. The corrosive characteristics of the onsite soil may limit the effectiveness of the soil treatment. The lime treatment option may require additional processing and multiple applications to achieve desired treatment results.

9.1.5 Materials for Fill

Materials for fill may be derived from onsite excavations or import sources provided they meet the following recommendations. 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. Note, onsite excavations are anticipated to generate oversize materials that are not suitable for reuse as compacted fill. The contractor should anticipate additional processing, including screening or rock picking of onsite materials prior to reuse as compacted fill. Oversize materials should be removed and disposed of offsite.

Materials with an expansion index greater than 50 are present onsite (Appendix B). These soils are not suitable for reuse within the upper 1-foot of subgrade soils beneath the proposed turf field, concrete flatwork, and the ADA ramp.

Imported fill material, if needed, should generally be granular soils with a very low expansion potential (i.e., an expansion index of 20 or less). Import fill material should not be considered corrosive as defined by Caltrans (2019) corrosion guidelines. Corrosive soils are defined as soil with an electrical resistivity equal to or less than 1,100 ohm-centimeters (ohm-cm), a chloride content more than 500 parts per million (ppm), more than 0.15 percent sulfates (1,500 ppm), and/or 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, , the landscape architect should be consulted regarding the reuse of these materials within fill soils to be placed in landscaped areas.

9.1.6 Compacted Fill

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

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

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

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

9.1.7 Utility 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.

9.1.8 Utility Trench Zone Backfill

Based on our subsurface evaluation, the onsite materials should be generally suitable for reuse as trench zone backfill provided they are free of organic material, clay lumps, debris, and rocks more than approximately 3 inches in diameter and meet the other recommendations for fill materials presented herein. Due to the presence of gravel and cobbles within the onsite soils, the contractor should anticipate additional processing, including screening or rock picking of onsite materials prior to reuse as compacted 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 backfill 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.

9.1.9 Drainage

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 (2019) guidelines and adjusted MCE_R spectral response acceleration parameters (SEAOC/OSHPD, 2020).

Table 4 – 2019 California Building Code Seismic Design Criteria	
Seismic Design Factors	**Value**
Seismic Design Category	D
Site Class	C
Site Coefficient, F_a	1.200
Site Coefficient, F_v	1.500
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.800g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.289g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.960g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.434g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.640g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.289g

9.3 Site Retaining Walls

If proposed, site retaining walls that are under 4 feet in height and are not a part of or are not connected to buildings may be supported on continuous footings bearing on compacted fill. The continuous footing should have a width of 24 inches or more and be embedded a depth of 18 inches or more. An allowable bearing capacity of 2,500 pounds per square foot (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 5. 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 6. The drain should be connected to an appropriate outlet.

9.4 Light Pole and Backstop Foundations

We recommend that posts/poles be supported on cast-in-drilled-hole (CIDH) foundations. Posts/poles typically impose relatively light axial loads on foundations. We recommend that CIDH foundations supporting posts/poles be evaluated and designed by the project structural engineer based on the geotechnical recommendations provided below.

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.

Due the variable nature and depth of the existing fill materials at the site, we recommend CIDH foundations be designed using an allowable passive pressure of 350 psf per foot of depth, with an upper bound value of up to 3,500 psf. This value assumes that the posts/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 0.3 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.

9.5 Exterior Concrete Flatwork

We recommend that exterior concrete flatwork underlain by compacted fill materials that generally possess a very low expansion potential (i.e., an EI of 20 or less) 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 concrete 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. Positive drainage should be established and maintained adjacent to exterior concrete flatwork.

9.6 Corrosion

Laboratory testing was performed on representative samples of the on-site 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 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 an electrical resistivity of 750 ohm-cm, a soil pH value of 7.7, a chloride content of 115 parts ppm, and a sulfate content of 0.008 percent (i.e., 80 ppm). Based on a comparison with the Caltrans corrosion (2019) criteria and our experience with similar soils, the onsite soils would be classified as corrosive. Corrosive soils are defined as soil with an electrical resistivity equal to or 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.

9.7 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 samples tested in this evaluation indicated water-soluble sulfate contents of 0.008 percent by weight (i.e., 80 ppm). Based on the American Concrete Institute (ACI) 318 criteria, water-soluble sulfate contents in soils less than about 0.10 percent by weight would indicate a S0 Exposure Class. Per ACI, it is recommended that concrete in contact with soil possess a 28-day compressive strength of 2,500 pounds per square inch (psi). Furthermore, due to the potential variability of site soils, we also recommend that Type II, II/V, or Type V cement be used for normal weight concrete in contact with soil.

10 TURF FIELD INFILTRATION

As discussed previously in this report, the onsite soils in the field area possess poor infiltration characteristics with factored infiltration rates of 0.02 inches per hour, or less. In addition, onsite soils possess a very low to medium potential for expansion. Such conditions may adversely impact the proposed artificial turf field. Accordingly, we recommend that a 20 mil or thicker impermeable liner be placed on the prepared subgrade soils beneath the turf field. Additionally, drainage for the artificial turf field system should incorporate an overflow pipe that is connected to an appropriate outlet.

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.

14 REFERENCES

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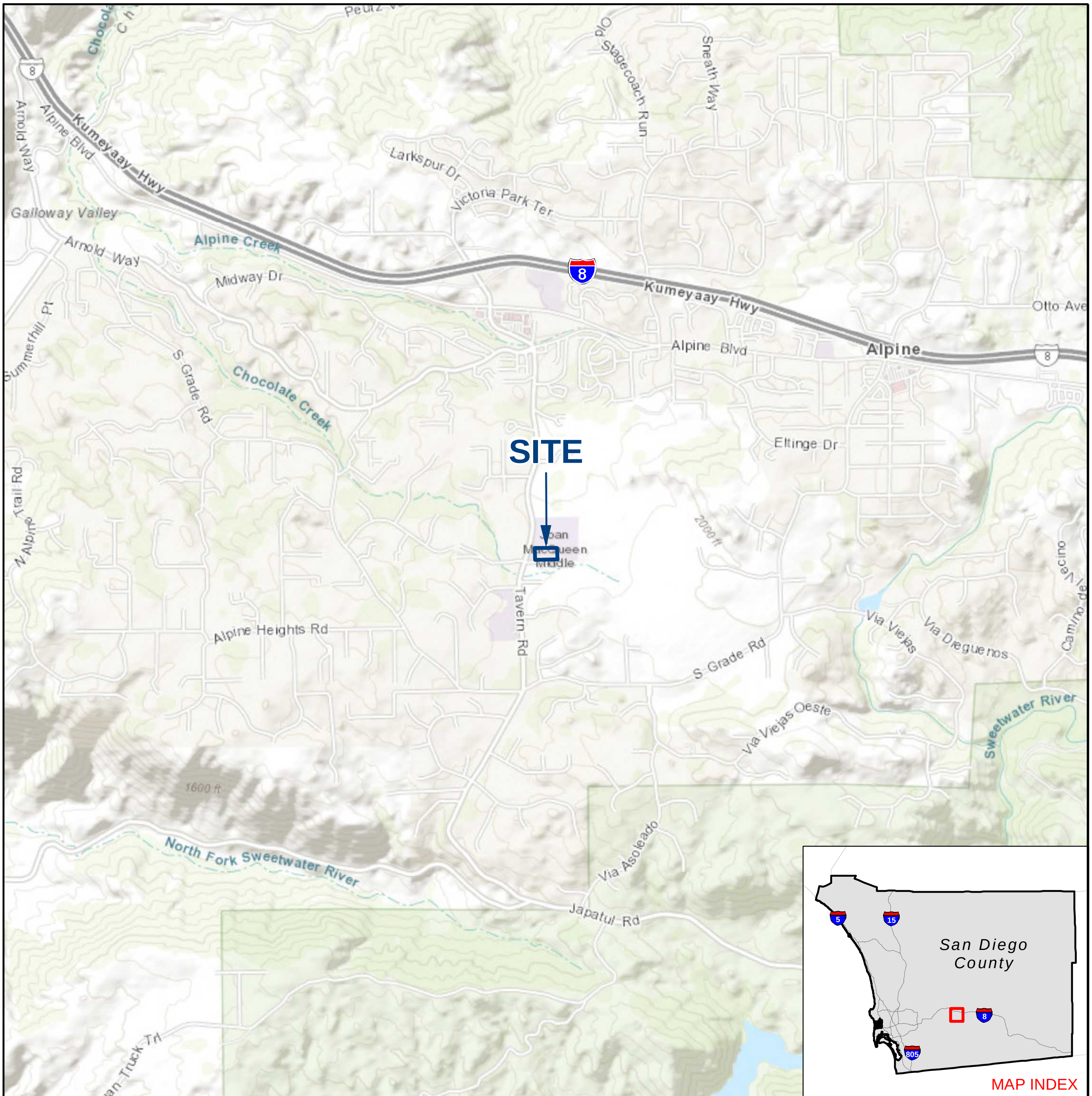
FIGURES



LEGEND

SITE BOUNDARY

N

FEET

0 3,000 6,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2020

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FIGURE 1

SITE LOCATION

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE, CALIFORNIA

108464009 | 2/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2020

2_108464009_BL.mxd 2/12/2020 JDL

FIGURE 2

BORING LOCATIONS

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE, CALIFORNIA

108464009 | 2/20

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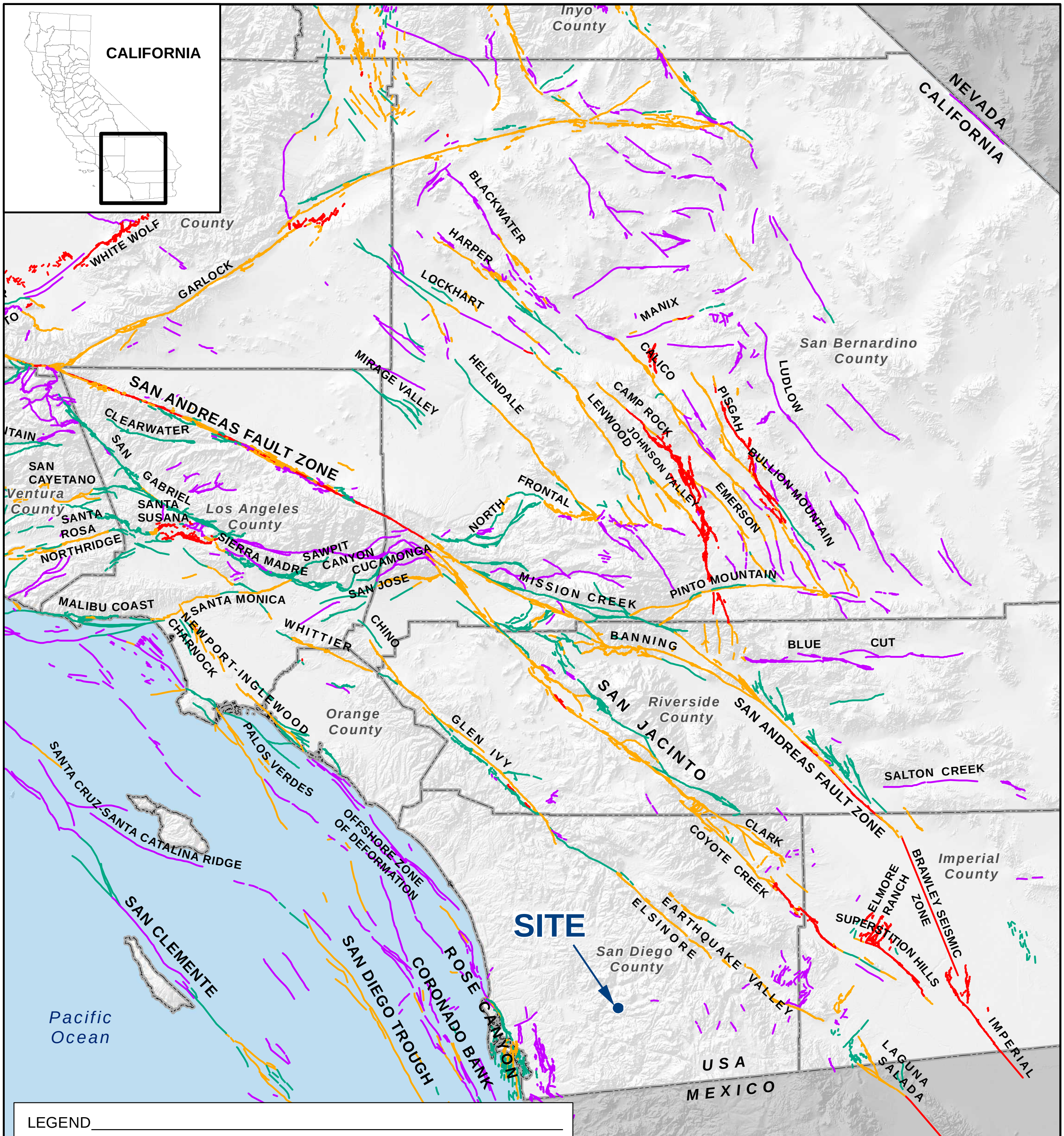

LEGEND

CALIFORNIA FAULT ACTIVITY

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

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

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

FAULT LOCATIONS

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE, CALIFORNIA

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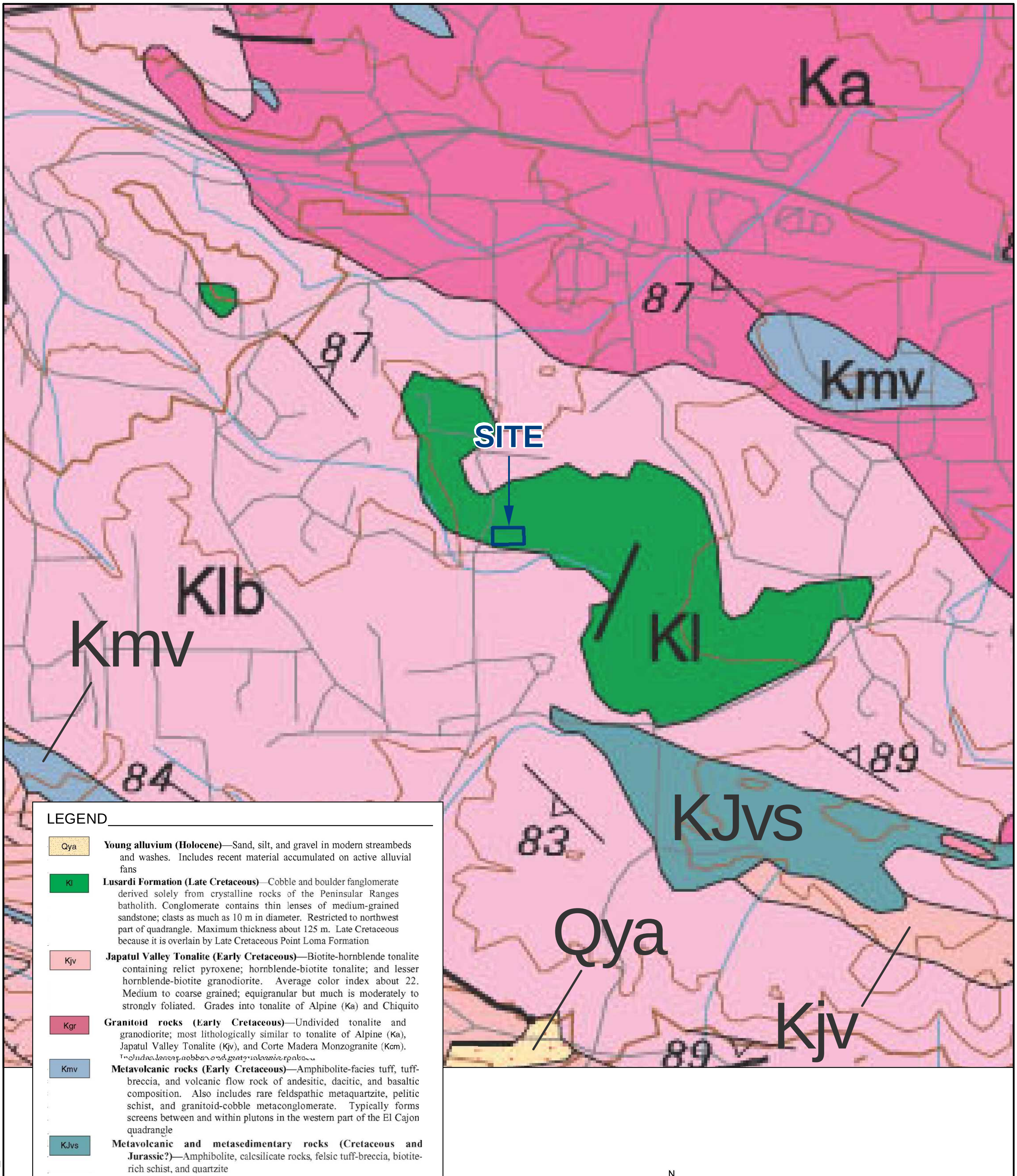

4_108464009_G.mxd 2/12/2020 JDL

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: 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 4

GEOLOGY

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE, CALIFORNIA

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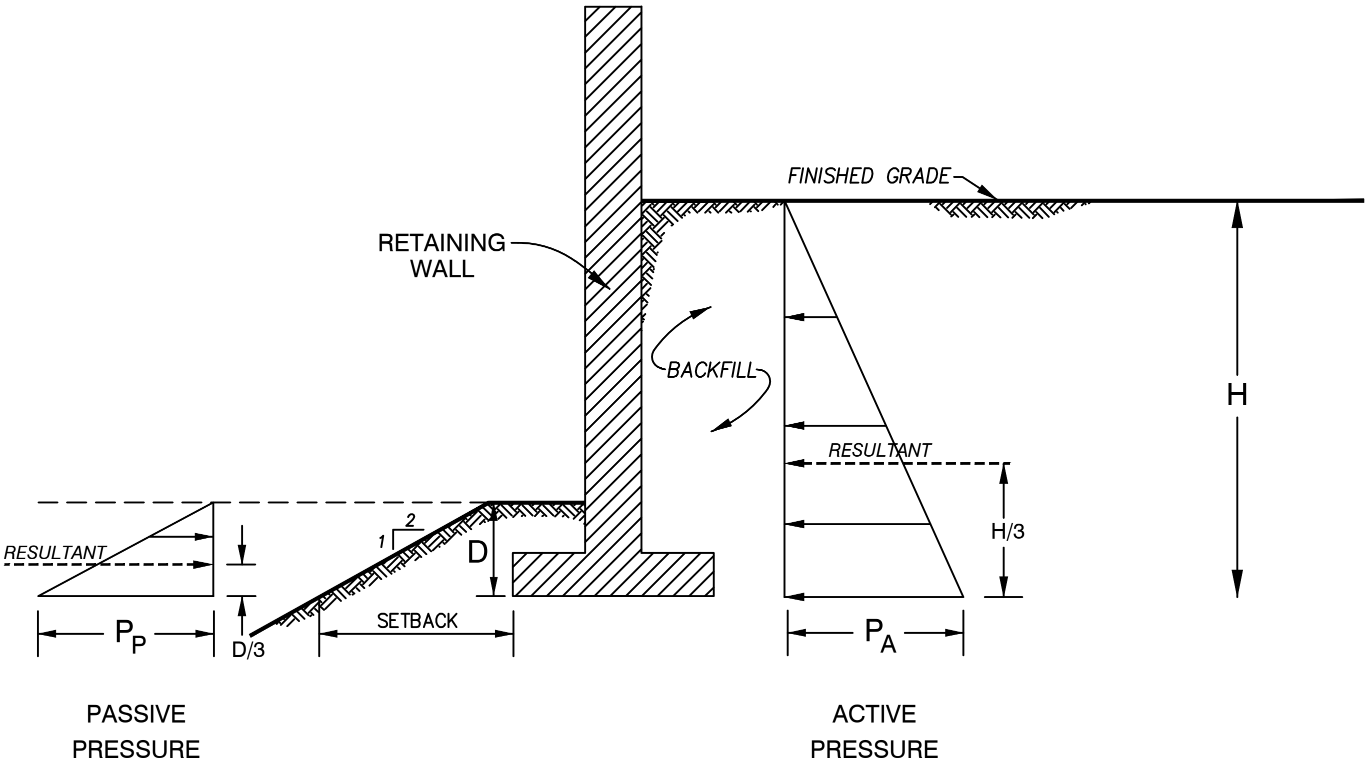

NOTES:

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. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET
6. SETBACK SHOULD BE IN ACCORDANCE WITH THE CBC
7. COEFFICIENT OF FRICTION, $\mu = 0.2$

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure[1] ($lb/ft^2/ft$)
P_A	Level Backfill with Granular Soils[2]
	55 H
P_P	Level Ground
	350 D

NOT TO SCALE

5 108464009 D-YRW.DWG

FIGURE 5

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION

2001 TAVERN ROAD, ALPINE, CALIFORNIA

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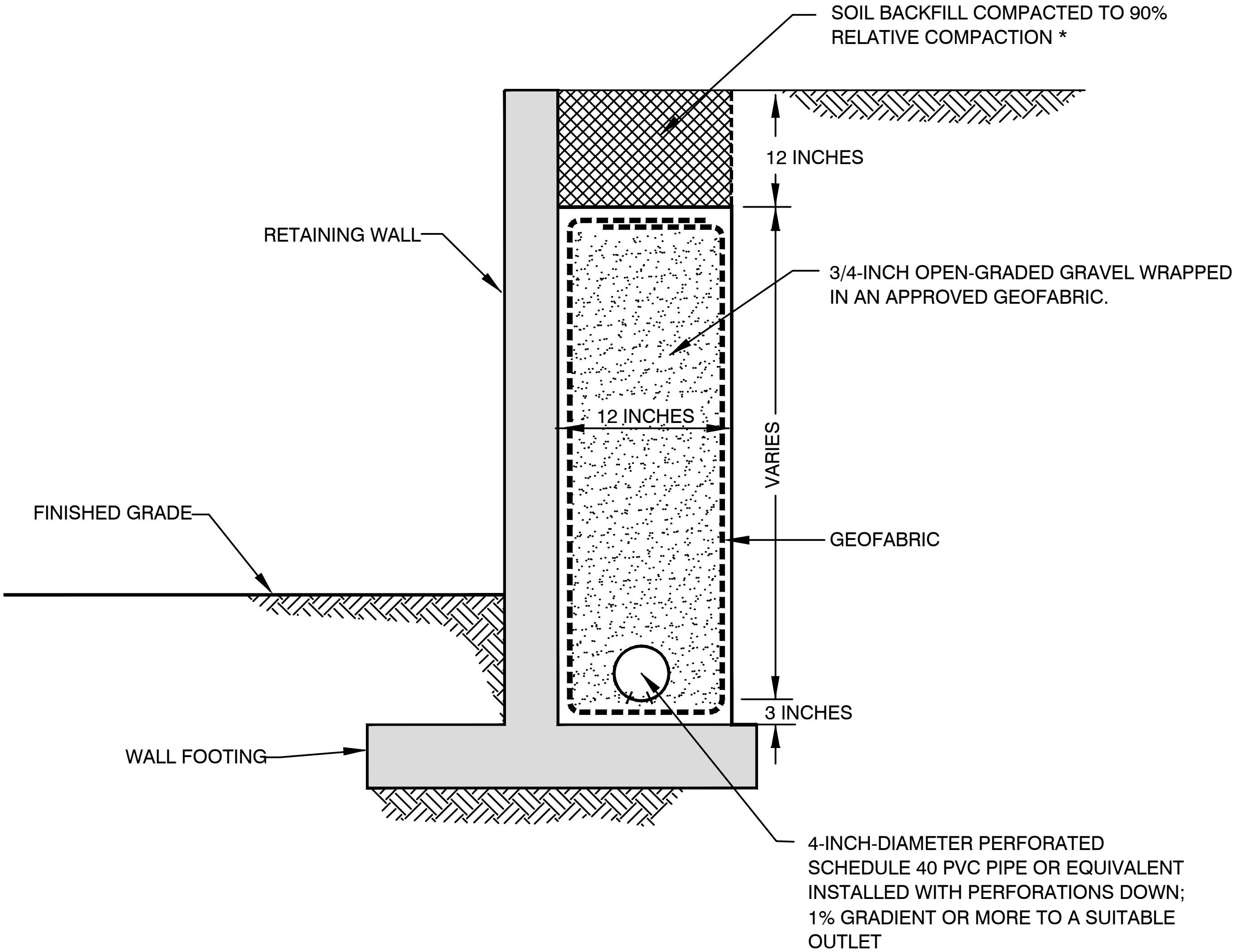

*BASED ON ASTM D1557

6_10464009_D-RW.DWG

NOT TO SCALE

FIGURE 6

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RETAINING WALL DRAINAGE DETAIL

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION

2001 TAVERN ROAD, ALPINE, CALIFORNIA

108464009 I 2/20

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

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

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

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

The Modified Split-Barrel Drive Sampler
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.

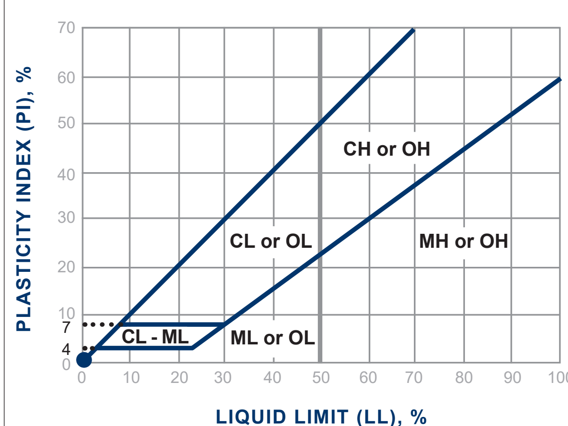
Soil Classification Chart Per ASTM D 2488

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

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

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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	<u>MAJOR MATERIAL TYPE (SOIL)</u>: 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								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 2 inches thick.
							SC	FILL: Dark brown, moist, medium dense, clayey fine to medium SAND; scattered gravel.
								Light brown; scattered cobbles.
5								
			22	21.5	100.2		CL	COLLUVIUM: Dark gray to black, moist, very stiff, fine to medium sandy CLAY; scattered organic material; organic odor.
								Scattered gravel.
10								
			22	19.8	105.2			
								Total Depth = 11.5 feet. Groundwater not encountered during drilling. Backfilled and patched shortly after drilling on 1/16/20. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15								
20								

DATE DRILLED 1/16/20 BORING NO. B-1

GROUND ELEVATION 1,837' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja Exploration)

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

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 1/16/20 BORING NO. IT-1 GROUND ELEVATION 1,837' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 6" Diameter Hand Auger DRIVE WEIGHT N/A DROP N/A SAMPLED BY CAT LOGGED BY CAT REVIEWED BY NMM DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown, moist, medium dense, clayey fine to coarse SAND; scattered gravel and cobbles. Light brown.
								Total Depth = 4 feet. Groundwater not encountered during drilling. Infiltration test set on 1/16/20. Backfilled after testing on 1/17/20. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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15								
20								

DATE DRILLED 1/16/20 BORING NO. IT-2

GROUND ELEVATION 1,841' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-95) (Baja Exploration)

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY NMM

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 fine to coarse SAND.
							SC	Brown, moist, medium dense, clayey fine to medium SAND.
							CL	COLLUVIUM: Dark brown, moist, very stiff, sandy CLAY; scattered gravel and cobbles.
5								Total Depth = 4 feet. (Refusal on cobbles) Groundwater not encountered during drilling. Infiltration test set on 1/16/20. Backfilled after testing on 1/17/20. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

APPENDIX B

Geotechnical Laboratory Testing

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

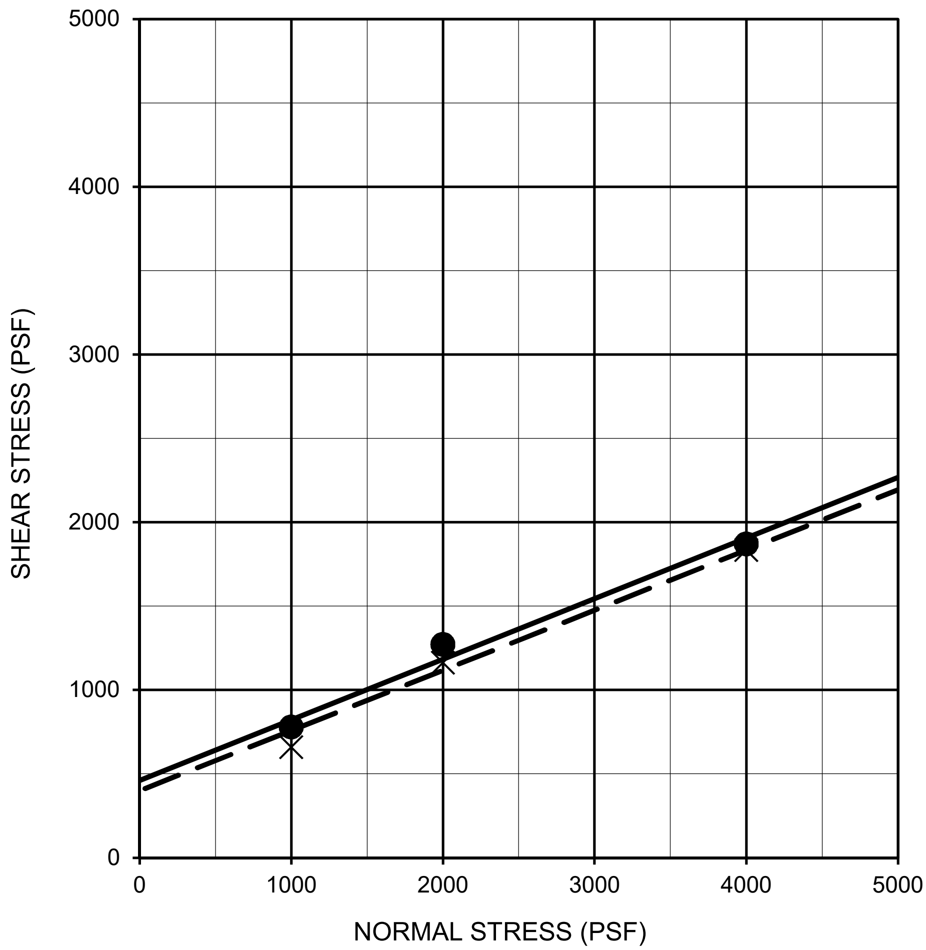
Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

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.

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-1.

Expansion Index Tests
The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-2.

Soil Corrosivity Tests
Soil pH and electrical resistivity tests were performed on representative samples in general accordance with CT 643. The sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-3.



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
CLAY	——●——	B-1	5.0-6.5	Peak	460	20	CL
CLAY	– – X – –	B-1	5.0-6.5	Ultimate	400	20	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-1

DIRECT SHEAR TEST RESULTS

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE,CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.2-3.0	12.5	101.9	28.4	0.050	50	Low
IT-1	0.0-3.0	9.0	113.8	20.5	0.009	9	Very Low
IT-2	2.5-4.0	11.0	106.4	25.8	0.068	68	Medium

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

FIGURE B-2

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EXPANSION INDEX TEST RESULTS

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE,CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
IT-2	2.5-4.0	7.7	750	80	0.008	115

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

FIGURE B-3

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CORROSIVITY TEST RESULTS

JOAN MACQUEEN MIDDLE SCHOOL FIELD RENOVATION
2001 TAVERN ROAD, ALPINE,CALIFORNIA

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APPENDIX C

Infiltration Testing

Test Date:	1/17/2020						Infiltration Test No.:	IT-1
Test Hole Diameter, D (inches):			6.0				Excavation Depth (feet):	4.0
Test performed and recorded by:			TJT				Pipe Length (feet):	4.0
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)
8:25	1.00	8:50	1.21	25	0.21	10	2.90	0.25
8:55	1.21	9:20	1.40	25	0.19	11	2.70	0.24
9:20	1.40	9:50	1.51	30	0.11	23	2.55	0.12
9:50	1.51	10:20	1.60	30	0.09	28	2.45	0.11
10:20	1.60	10:50	1.66	30	0.06	42	2.37	0.07
10:50	1.66	11:20	1.75	30	0.09	28	2.30	0.11
11:20	1.75	11:50	1.81	30	0.06	42	2.22	0.08
11:50	1.81	12:20	1.90	30	0.09	28	2.15	0.12
12:20	1.90	12:50	1.96	30	0.06	42	2.07	0.08
12:50	1.96	13:20	2.01	30	0.05	50	2.02	0.07
13:20	2.01	13:50	2.09	30	0.08	31	1.95	0.12
13:50	2.09	14:20	2.12	30	0.03	83	1.90	0.04

Test Date:	1/17/2020						Infiltration Test No.:	IT-2
Test Hole Diameter, D (inches):			8.0				Excavation Depth (feet):	4.0
Test performed and recorded by:			TJT				Pipe Length (feet):	4.0
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	1.00	8:55	1.11	25	0.11	19	2.95	0.13
8:55	1.10	9:25	1.12	25	0.02	104	2.89	0.02
9:25	1.12	9:55	1.14	30	0.02	125	2.87	0.02
9:55	1.14	10:25	1.20	30	0.06	42	2.83	0.06
10:25	1.20	10:55	1.25	30	0.05	50	2.78	0.05
10:55	1.25	11:25	1.32	30	0.07	36	2.72	0.07
11:25	1.32	11:55	1.34	30	0.02	125	2.67	0.02
11:55	1.34	12:25	1.38	30	0.04	63	2.64	0.04
12:25	1.38	12:55	1.40	30	0.02	125	2.61	0.02
12:55	1.40	13:25	1.42	30	0.02	125	2.59	0.02
13:25	1.42	13:55	1.43	30	0.01	250	2.58	0.01
13:55	1.43	14:25	1.44	30	0.01	250	2.57	0.01

Notes:

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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

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

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion [1]

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

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.

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

www.ninyoandmoore.com