Appendix F
Geotechnical Evaluation

Geotechnical Evaluation

Alpine Community Park
Alpine, California

MW Peltz & Associates

143 South Cedros Avenue, Suite B104 | Solana Beach, California 92075

December 30, 2020 | Project No. 109107001

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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Geotechnical Evaluation

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1 INTRODUCTION

In accordance with your request and authorization, we have prepared this geotechnical evaluation report for the proposed Alpine Community Park in Alpine, California (Figure 1). Presented in this report are the results of our background review, field exploration, and geotechnical laboratory testing along with our conclusions regarding the geotechnical conditions at the site and our recommendations for the design and earthwork construction aspects of this project.

2 SCOPE OF SERVICES

Ninyo & Moore's scope of services for this project included review of pertinent background data, performance of a geologic reconnaissance, subsurface exploration, and engineering analysis with regard to the proposed construction. These services generally follow the scope outlined in our proposal dated April 3, 2020. Specifically, we performed the following tasks:

- Reviewing readily available pertinent information including published in-house geotechnical literature, topographic maps, geologic maps, and fault maps, historic stereoscopic aerial photographs, and project conceptual drawings (MW Peltz + Associates, 2020).
- Performing a field reconnaissance to observe site conditions and to mark the locations of our exploratory test pits.
- Coordinating with Underground Service Alert (USA) for utility clearance at our test pit locations.
- Performing a subsurface exploration consisting of the excavating, logging, and sampling of 15 exploratory test pits. The test pits were excavated to depths of up to approximately 7.2 feet using a rubber-tire backhoe. Bulk samples of the materials encountered were collected at selected intervals from the test pits and transported to our in-house geotechnical laboratory for testing.
- Performing infiltration testing using the simple open pit test method within seven of the test pits. Infiltration testing was performed in general conformance with the guidelines presented in the 2020 County of San Diego BMP Design Manual.
- Performing geotechnical laboratory testing on representative samples to evaluate soil parameters for design and classification purposes.
- Performing engineering analyses of the site geotechnical conditions based on data obtained from our background review, field exploration, and laboratory testing.
- Preparing this geotechnical evaluation report describing the findings and conclusions of our study and providing recommendations for design and construction of the proposed improvements.

3 SITE AND PROJECT DESCRIPTION

The site is located on the western and northern sides of South Grade Road at its intersections with Calle de Compadres and Via Viejas in Alpine, California (Figure 1). The project site consists of undeveloped land with a gentle gradient down to the southeast, with a cross slope inclined toward the southeast. Onsite elevations range from approximately 2,030 feet above mean sea level (MSL) in the northeastern portion of the site to approximately 1,970 feet above MSL in the southwestern portion of the site. The project site is generally undeveloped and sparsely vegetated with grasses, shrubs, and trees. Several dirt roads and trails transect portions of the site.

Based on our review of project conceptual drawings (MW Peltz + Associates, 2020), we understand that the project will consist of the construction of a new County of San Diego park. The park improvements are to include new administration, restroom, and storage buildings, shade structures, a skate park, basketball courts, pickle-ball courts, sports fields, a bike park, a dog park, and a community garden. Additional improvements are anticipated to consist of parking and drive areas, picnic tables, underground utilities, American with Disabilities Act (ADA) access walkways and ramps, landscaping, and signage.

4 SUBSURFACE EXPLORATION

Our subsurface exploration was conducted on November 18 and 19, 2020, and included the excavating, logging, and sampling of 15 exploratory test pits (TP-1 through TP-15). Prior to commencing the subsurface exploration, USA was notified for marking of the existing site utilities. The purpose of the test pits was to evaluate subsurface conditions and to collect soil samples for laboratory testing.

The test pits were excavated to depths ranging from approximately 3 feet to 7.2 feet using a rubber-tire backhoe. 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 cuttings and bulk samples. Representative bulk and in-place soil samples were obtained from the test pits. The samples were then transported to our in-house geotechnical laboratory for testing. The approximate locations of the exploratory test pits are shown on Figure 2. Logs of the test pits are included in Appendix A.

5 INFILTRATION TESTING

Field infiltration testing was performed on November 18 and 19, 2020 in general accordance with the County of San Diego BMP Design Manual (2020) using the simple open pit test method. The infiltration tests IT-1 through IT-7 were performed within exploratory test pits TP-1, TP-2, TP-7, TP-9, TP-12, TP-14 and TP-15). An approximately 2 foot by 2 foot, square hole that was approximately 1-foot-deep was manually excavated within each of the noted test pits. For testing, the test holes were filled with 6 to 12 inches of water and the depth to the water was measured at 10 minute intervals for a span of 1 hour and then the test at each location was repeated another 2 times for total of 3 hours of testing per location. The test holes were refilled after the respective intervals as needed to restore the initial water level.

Infiltration rates were calculated using the Porchet method and an equivalent radius for the square holes. Infiltration tests IT-1 through IT-4 indicated that the observed (i.e., unfactored) infiltration rates ranged from a no infiltration condition to very slow variable infiltration rates. Per the County of San Diego BMP Design Manual (2020) Appendix D Section D.2-3 the safety factor must be between 2.0 and 9.0, and per Table D.2-3 a safety factor of 2.25 was selected. Completed Tables D.1-1: Consideration for Geotechnical Analysis of Infiltration Restrictions and D.2-3: Determination of Safety Factor are presented in Appendix D. Infiltration test results and calculations are included in Appendix C and summarized in Table 1 below.

Table 1 – Infiltration Test Results Summary

Infiltration Test	Approximate Test Depth (feet)	Description	Observed Infiltration Rate (in/hr)	Suitability Assessment Safety Factor[1]	Reliable/ Factored Infiltration Rate[2] (in/hr)
IT-1	3.3	Decomposed Granitic Rock	0.17[3]	2.25	0.07[3]
IT-2	3.8	Decomposed Granitic Rock	DNI	2.25	DNI
IT-3	3.2	Decomposed Granitic Rock	DNI	2.25	DNI
IT-4	4.2	Decomposed Granitic Rock	DNI	2.25	DNI
IT-5	3.8	Decomposed Granitic Rock	0.21[3]	2.25	0.09[3]
IT-6	3.6	Decomposed Granitic Rock	DNI	2.25	DNI
IT-7	4.9	Decomposed Granitic Rock	DNI	2.25	DNI

Notes:
DNI = did not infiltrate
in/hr = inches per hour
[1] Design safety factor to be determined by the design engineer in accordance with Appendix D of the County of San Diego BMP Design Manual (2019)
[2] Factored infiltration rate shall be divided by the design safety factor to obtain the design infiltration rate.
[3] Infiltration rates ranged from a no infiltration condition to very slow variable rates. The listed rates are approximations based on interpretation of the test results presented in Appendix C.

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.

6 LABORATORY TESTING

Geotechnical laboratory testing was performed on representative soil samples collected from our subsurface exploration. Testing included an evaluation of gradation (sieve) analysis, expansion index, soil corrosivity, and R-value. Descriptions of the geotechnical laboratory test methods and the results of the geotechnical laboratory tests performed are presented in Appendix B.

7 GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology and groundwater conditions are provided in the following sections.

7.1 Regional Geologic Setting

The project is situated in the coastal foothill section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles. 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 portion of the province in San Diego County that includes the project area consists generally of Cretaceous age sedimentary and granitic rock.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending approximately northwest. Several of these faults, shown on Figure 3, are considered active faults (Jennings, 2010). The Elsinore, San Jacinto, and San Andreas 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 fault located west of the project area. The Elsinore fault zone is the nearest active fault system and has been mapped approximately 21 miles east of the project site. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report

7.2 Site Geology

The geology of the site vicinity is shown on Figure 3. Geologic units encountered during our site reconnaissance and subsurface exploration included topsoil and decomposed granitic rock. Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration and mapped in the vicinity of the project site are provided in the subsequent sections (Todd, 2004). Additional descriptions of the subsurface units are provided on the test pit logs in Appendix A.

7.2.1 Topsoil

Topsoil was encountered in each of our test pits from the ground surface to depths of approximately 3.8 feet. As encountered, the topsoil generally consisted of dark brown, moist, stiff to very stiff, sandy clay and medium dense, clayey sand. Gravel, cobbles, and boulders were encountered in the topsoil.

7.2.2 Lusardi Formation

Although not encountered during our subsurface exploration, materials of the Cretaceous-age Lusardi Formation are mapped at the site. The Lusardi Formation generally consists of cobble and boulder conglomerate with thin lenses of sandstone.

7.2.3 Decomposed Granitic Rock

Decomposed granitic rock was encountered in each of our test pits underlying the topsoil and extended to the total depths explored. As encountered the decomposed granitic rock generally consisted of various shades of brown, yellow, and red, dry to moist, weathered, friable, fine- to medium-grained granitic rock with iron oxide staining. The granitic rock was observed to vary in degrees of weathering with the rock being less weathered with depth. Unweathered granitic rock corestones were encountered within out test pits and boulders were observed on the surface at numerous locations within the site.

7.3 Groundwater

Groundwater was not encountered during our evaluation and groundwater is not anticipated to be encountered during construction of the proposed improvements. However, perched groundwater or groundwater seepage may be encountered between the contact of topsoil and granitic rock or within fractures in the granitic rock. Fluctuations in groundwater typically occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, flooding, and other factors.

7.4 Faulting and Seismicity

The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey, active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years), but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active and potentially active faults in the vicinity of the site and their geographic relationship to the site are shown on Figure 4. Additionally, the site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2007).

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structures. Based on our review of the referenced geologic maps, as well as on our site reconnaissance, no faults are mapped as underlying the project site. The nearest known active fault is the Elsinore fault, located approximately 21 miles east of the site.

In general, hazards associated with seismic activity include strong ground motion, ground rupture, and liquefaction. These hazards, along with landsliding, are discussed in the following sections.

7.4.1 Ground Motion

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 0.39g using the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) (SEAOC and OSHPD, 2020) seismic design tool (web-based).

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 0.42g using the USGS (USGS, 2020) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.35g for the site and a site coefficient (F_{PGA}) of 1.2 for Site Class C.

7.4.2 Ground Rupture

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

7.4.3 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 60 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. Due to the dense nature of the underlying granitic rock at the site, liquefaction is not a design consideration.

7.4.4 Landsliding

Based on our review of referenced geologic maps, literature and topographic maps, and subsurface exploration, landslides or indications of deep-seated landsliding were not noted underlying the project site. In our opinion, the potential for significant large-scale slope instability at the site is not a design consideration.

7.5 Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM), the site is not located within mapped flood zones or floodplains. In our opinion, the potential for significant flooding at the site is not a design consideration.

8 CONCLUSIONS

Based on our review of the referenced background data, subsurface exploration, and geotechnical laboratory testing, it is our opinion that construction of the proposed improvements is feasible from a geotechnical standpoint. In general, the following conclusions were made:

- Based on the results of our field and laboratory evaluations, the subsurface soils at the project site consist of topsoil and decomposed granitic rock.
- The topsoil materials are loose and considered unsuitable for structural support in their present condition. Accordingly, recommendations are presented herein for remedial grading of these materials in preparation for new construction.
- Outcroppings of rock are exposed at the surface of the site. Excavations extending into granitic rock will encounter very difficult excavation conditions due to the presence of bedrock materials, boulders, and/or corestones. The contractor should be prepared for the use of heavy ripping, rock breaking, rock coring, and/or blasting techniques to perform onsite excavations.
- Groundwater was not encountered during our evaluation and is not anticipated to be a design and construction consideration.
- The project site is not located within a mapped floodplain or flood zone.
- No faults are mapped at the site. The closest known active fault, the Elsinore fault, has been mapped approximately 21 miles east of the site.
- Based on our field evaluation and a review of referenced maps, landslides are not present on the project site.
- Onsite excavations will generate oversized materials. Oversized materials should be screened, rock-picked, crushed, removed, or otherwise processed from the excavated materials prior to reuse as compacted fill.
- Based on the results of our laboratory testing, the topsoil possesses a medium to high potential for expansion. Accordingly, these expansive soils are not suitable for reuse as compacted fill beneath buildings, for retaining walls, or exterior concrete pedestrian flatwork. Remedial grading recommendations for these areas are presented herein.
- Based on the results of our soil corrosivity tests and Caltrans amended (2019) AASHTO (2017) corrosion criteria, the onsite soils would not be classified as corrosive.

9 RECOMMENDATIONS

The following recommendations are provided for the design and construction of the proposed project. These recommendations are based on our evaluation of the site geotechnical conditions and our assumptions regarding the planned development. 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. The geotechnical consultant should be contacted for questions regarding the recommendations or guidelines presented herein.

9.1.1 Pre-Construction Conference

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the Project Inspector, 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.

9.1.2 Site Preparation

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

9.1.3 Excavation Characteristics

During our subsurface evaluation, we observed outcroppings of rocks at the surface and encountered decomposed granitic rock with corestones in varying states of weathering. Onsite excavations will encounter very difficult excavation conditions due to the presence of bedrock materials, boulders, and/or corestones. The contractor should be prepared for the use of heavy ripping, rock breaking, rock coring, and/or blasting techniques to perform onsite excavations.

Additionally, onsite excavations will generate oversize materials that should be screened, rock-picked, crushed, removed, or otherwise processed from the excavated materials prior to reuse as compacted fill.

9.1.4 Temporary Excavations

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

Topsoil	*Type C*
Granitic Rock	*Type B*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field 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 a slope ratio no steeper than 1½:1 (horizontal to vertical) in topsoil and 1:1 in granitic rock. Temporary excavations that encounter seepage may require shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. Onsite safety of personnel is the responsibility of the contractor.

9.1.5 Remedial Grading – Structural Buildings

Based on the results of our laboratory testing presented in Appendix B, the existing topsoil possesses a medium to high potential for expansion. To mitigate for the effects of highly expansive onsite soils, we recommend the performance of the following remedial grading measures for buildings. Furthermore, recommendations to support the structures on deepened foundations, in conjunction with these remedial grading recommendations, are presented in following sections of this report.

We recommend that the existing near-surface topsoil within the building pad be removed down to competent decomposed granitic rock or 1 foot below the bottom of footings, whichever is deeper. This overexcavation should extend to the horizontal limits of the building pad. For the purposes of this report, the building pad is defined as the structural footprint (including foundations for attached overhangs, canopies, and other building appurtenances) plus a horizontal distance of 5 feet, where feasible. The lateral extents of the overexcavation may be modified in the field based on site constraints, such as property lines. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed.

Subsequent to performance of the overxcavation removal, the resulting surface should be scarified to a depth of approximately 6 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should then be backfilled with compacted fill soils placed in accordance with the recommendations herein.

We recommend that the upper 2 feet of compacted fill soils placed within the building pads possess a very low to low potential for expansion (i.e. an expansion index of less than 50). As noted earlier, the onsite topsoil possesses a medium to high potential for expansion and are not considered suitable for reuse within the upper 2 feet of compacted fill soils with building pads. Accordingly, the upper 2 feet of compacted fill soils within building pads may consist of import soils, soils derived from onsite excavations into the decomposed granitic rock, or lime-treatment of onsite soils.

9.1.6 Remedial Grading – Retaining Walls

We recommend that the existing near-surface topsoil beneath retaining walls be removed down to a depth of 1 foot below the bottom of the retaining wall foundations. This overexcavation should extend a lateral distance of 1 foot beyond the horizontal limits of the foundation. The lateral extents of the overexcavation may be modified in the field based on site constraints. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed.

Subsequent to performance of the overxcavation removal, the resulting surface should be scarified to a depth of approximately 6 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should then be backfilled with compacted fill soils placed in accordance with the recommendations herein.

We recommend that the upper 1 foot of compacted fill soils placed beneath retaining walls possess a very low to low potential for expansion (i.e., an expansion index of less than 50). As noted earlier, the onsite topsoil possesses a medium to high potential for expansion and are not considered suitable for reuse within the upper 1 foot of compacted fill soils beneath retaining walls. Accordingly, the upper 1 foot of compacted fill soils beneath retaining walls may consist of import soils, soils derived from onsite excavations into the decomposed granitic rock, or lime-treatment of onsite soils.

9.1.7 Remedial Grading – Exterior Pedestrian Concrete Flatwork

We recommend that the existing near-surface topsoil beneath exterior pedestrian concrete flatwork be removed down to a depth of 2 feet below the planned finished subgrade elevation. This overexcavation should extend a lateral distance of 1 foot beyond the horizontal limits of the flatwork. The lateral extents of the overexcavation may be modified in the field based on site constraints. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed.

Subsequent to performance of the overxcavation removal, the resulting surface should be scarified to a depth of approximately 6 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should then be backfilled with compacted fill soils placed in accordance with the recommendations herein.

We recommend that the upper 2 feet of compacted fill soils placed beneath exterior pedestrian flatwork possess a very low to low potential for expansion (i.e. an expansion index of less than 50). As noted earlier, the onsite topsoil possesses a medium to high potential for expansion and are not considered suitable for reuse within the upper 2 feet of compacted fill soils beneath exterior pedestrian flatwork. Accordingly, the upper 2 feet of compacted fill soils beneath exterior pedestrian flatwork may consist of import soils, soils derived from onsite excavations into the decomposed granitic rock, or lime-treatment of onsite soils.

9.1.8 Materials for Fill

Materials for fill may be processed from onsite excavations or may consist of import materials. Onsite soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) are suitable for reuse as general fill material. Fill soils should be free of trash, debris, roots, vegetation, organics, or other deleterious materials. Due to the shallow groundwater moisture conditioning of onsite materials, including drying and/or aerating, should be anticipated. Fill and utility trench backfill materials should not contain rocks or lumps over 3 inches, and not more than 30 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite.

As noted earlier, expansion index testing presented in Appendix B indicates that some of the onsite topsoil possesses a medium to high 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 upper 2 feet of building pads, in the upper 1 foot beneath retaining wall footings, as retaining wall backfill, or as the upper 2 feet of subgrade soils beneath pedestrian concrete flatwork.

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 amended (2019) AASHTO (2017) corrosion criteria. 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.

9.1.9 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 the 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.

9.1.10 Slopes

We anticipate that new cut and fill slopes will be constructed for the project. Unless otherwise recommended by our offices and approved by the regulating agencies, permanent cut and fill slopes should not be steeper than 2:1 (horizontal to vertical). Buildings, structures, and improvements should be set back from the top of slopes in accordance with the 2019 CBC. We recommend buildings and structures be set back 20 feet or more from the top of slopes.

Compaction of the face of fill slopes should be performed by backrolling at intervals of 4 feet or less in vertical slope height, or as dictated by the capability of the available equipment, whichever is less. Fill slopes should be overbuilt and cut back to finish grades. The placement, moisture conditioning, and compaction of fill slope materials should be done in accordance with the recommendations presented herein.

Site runoff should not be permitted to flow over the tops of slopes. Positive drainage should be established away from the top of slopes. This may be accomplished by utilizing brow ditches placed at the top of slopes to divert surface runoff away from the slope face where drainage devices are not otherwise available.

The onsite soils are susceptible to erosion. The project plans and specifications should contain design features and construction requirements to mitigate erosion of soils or contain a maintenance program to redress erosion features as they develop on a periodic basis.

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 greater.

Pipe bedding and pipe zone backfill should have a Sand Equivalent of 30 or more, and be placed around the sides and the crown of the pipe. In addition, the pipe zone backfill should extend 1 foot or more above the crown of the pipe. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the pipe zone backfill should proceed up both sides of the pipe.

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

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 1,200 pounds per square inch (psi) may be used for an excavation depth of up to approximately 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,500 psi may be used for trenches deeper than 5 feet.

9.1.12 Utility Trench Zone Backfill

Utility trench zone backfill should be free of organic material, clay lumps, debris, and meet the following recommendations. Trench backfill should not contain rocks or lumps over approximately 3 inches in diameter and not more than approximately 30 percent larger than ¾ inch. Backfill materials should be moisture-conditioned to generally at or slightly above the laboratory optimum. Trench 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 pavement areas which should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Wet soils should be allowed to dry to moisture contents near the optimum prior to their placement as backfill. 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.

Note, the upper 2 feet of utility trench backfill beneath building pads, in the upper 1 foot beneath retaining wall footings, or within the upper 2 feet of subgrade soils beneath exterior pedestrian concrete flatwork should possess a medium to high potential for expansion (i.e., an expansion index of 50 or more).

9.1.13 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 5. Thrust blocks should be backfilled following the recommendations presented in this report.

9.1.14 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 2 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 2 – 2019 California Building Code Seismic Design Criteria

Seismic Design Factors	Value
Seismic Design Category	D
Site Class	C
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.5
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.806g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.291g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.967g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.436g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.645g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.291g

9.3 Building Foundations

Based on our understanding of the project, site development will include the construction of various buildings. 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 or competent decomposed granitic rock may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. We recommend that shallow foundations for the new buildings be founded 30 inches below the lowest adjacent grade to mitigate for the effects of the highly expansive soils onsite. Continuous footings should have a width of 18 inches and spread footings should be 24 inches in width. The footings should be reinforced in accordance with the recommendations of the project structural engineer.

9.3.2 Lateral Resistance

For resistance of footings to lateral loads, 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 0.2 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 Interior Slabs-On-Grade

We recommend that interior concrete slabs-on-grade be underlain by 2 feet or more of compacted fill materials that generally possess a 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.

9.5 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 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 6. These pressures assume select backfill materials are used and free draining conditions. Select backfill materials should not contain rocks or lumps over 3 inches, and not more than 30 percent larger than ¾ inch, and possess a very low to low potential for expansion (i.e. an expansion index less than 50). 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 7. The drain should be connected to an appropriate outlet.

9.6 Shade Structure, Light Pole and Backstop Foundations

Improvements such as shade structures, light poles, and backstop fencing are structures that typically impose relatively light axial loads on foundations and have more structural demands for lateral and uplift loading. Typically, we recommend that such structures be supported on cast-in-drilled-hole (CIDH) pile foundations. However, due to potential difficulty with drilling of these foundations due to the presence of corestones at the surface and within the decomposed granite rock, we are also providing alternative recommendations to support these structures on shallow spread footings.

9.6.1 Cast-in-Drilled-Hole (CIDH) Piles

Shade structures, light poles, and backstop fencing 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. Furthermore, to mitigate for the potential effects of the highly expansive onsite soils, CIDH pile foundations should extend to depths for 6 feet 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.

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 300 psf per foot of depth, with an upper bound value of up to 3,000 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.

9.6.2 Shallow Spread Footings

Due to the potential drilling refusal on corestones that may be encountered within the decomposed granitic rock, we are providing alternative recommendations to support shade structures, light poles, and backstop fencing on shallow spread footings. To mitigate for the potential effects of the highly expansive onsite soils, spread footings for these types of structures should be embedded to depths for 3 feet or more. 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.

Shallow, spread footings for shade structures, light poles, and backstop fencing bearing on compacted fill or competent site soils may be designed using an allowable bearing capacity of 2,000 psf. These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. Spread footings should be 36 inches or more in width. The footings should be reinforced in accordance with the recommendations of the project structural engineer.

For resistance of spread footings to lateral loads, 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 0.2 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.

9.7 Preliminary Flexible Pavement Design

We understand that the project will include the construction of new flexible AC pavements. Our laboratory testing of near surface soil samples at the project site indicated an R-values of less than 5. We have used an R-value of less than 5, along with estimated design Traffic Indices (TI) of 4.5, 5, 6, and 7 as the basis of our preliminary flexible pavement design. These assumed TIs should be evaluated by the Civil Engineer based on anticipated traffic loading at the site. 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 3.

Table 3 – Recommended Preliminary Flexible Pavement Sections

Traffic Index (Pavement Usage)	Design R-Value	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
4.5 (Parking Stalls)	Less than 5	3	8
5 (Drive Aisles)	Less than 5	3	10
6 (Heavy Traffic Areas)	Less than 5	3	14
7 (Fire Lanes)	Less than 5	4	16

As indicated, these values assume TIs of 7.0 or less for site roads. 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.

9.8 Preliminary Gravel Road Design

As part of the new construction, we anticipate that new gravel vehicular roads and parking areas will be constructed. Our laboratory testing of near surface soil samples at the project site indicated R-values of less than 5. This R-value, along with assumed design Traffic Indices (TI) of 4.5, 5, 6, and 7 has been the basis of our preliminary road design. These assumed TIs should be evaluated by the Civil Engineer based on anticipated traffic loading at the site. Actual gravel road 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 4.

Table 4 – Recommended Preliminary Gravel Road Sections

Traffic Index (Pavement Usage)	Design R-Value	Aggregate Base Thickness (inches)
4.5 (Parking Stalls)	Less than 5	15
5 (Drive Aisles)	Less than 5	17
6 (Heavy Traffic Areas)	Less than 5	20
7 (Fire Access)	Less than 5	24

As indicated, these values assume TIs of 7.0 or less for site roads. 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. Gravel access roads will require periodic maintenance.

9.9 Rigid Concrete Pavements

We understand that rigid concrete pavements may be used for the American with Disabilities Act (ADA) parking stalls and their associated walkways. We recommend that these ADA parking stalls and walkways be 6 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 18 inches on-center both ways. The concrete for the ADA parking stall and walkway should be underlain by 2 feet of compacted fill that possesses a very low to low potential for expansion (i.e. an expansion index of less than 50). To reduce the potential manifestation of distress to rigid concrete pavements due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Positive drainage should be established and maintained adjacent to flatwork.

We also suggest that consideration be given to using Portland cement concrete pavements in areas where dumpsters will be stored and where refuse and delivery trucks will stop and load. Experience indicates that refuse and other heavy truck traffic can significantly shorten the useful life of AC or gravel road sections. We recommend that in these areas, a pavement section consisting of an 8-inch thickness of Portland cement concrete underlain by 8 inches of compacted aggregate base be placed. We recommend that the Portland cement concrete have a 600 pounds per square inch (psi) flexural strength and that it be reinforced with No. 4 bars that are placed 18 inches on center (both ways). The rigid pavement and aggregate base should be placed on compacted subgrade that is at a relative compaction of 95 percent as evaluated by ASTM D 1557.

9.10 Exterior Pedestrian Concrete Flatwork

We recommend that exterior pedestrian concrete flatwork be 5 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. Exterior pedestrian concrete flatwork should be underlain by 2 feet of compacted fill soils that possess a very low to low potential for expansion (i.e. an expansion index of less than 50). 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. Positive drainage should be established and maintained adjacent to flatwork.

9.11 Corrosion

Laboratory testing was performed on a select representative sample 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 630 to 1,600 ohm-cm, soil pH values of 7.4 to 7.5, chloride contents of 40 to 70 parts per million (ppm), and sulfate contents of 0.001 to 0.004 percent (i.e., 10 to 40 ppm). Based on a comparison with the California Department of Transportation amended (Caltrans, 2019) AASHTO (2017) corrosion criteria, the soils at the project site are classified as corrosive, which is defined as having earth materials with more than 500 ppm chlorides, more than 0.15 percent sulfates (i.e., 1,500 ppm), a pH less than 5.5, and/or an electrical resistivity less than 1,100 ohm-cm.

9.12 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 0.001 to 0.004 percent by weight (i.e., 10 to 40 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the site soils would correspond to exposure class S0. For this exposure class, ACI 318 recommends that normal weight concrete in contact with soil possess a compressive strength of 2,500 psi or more. Furthermore, due to the potential for variability of site soils, we also recommend that normal weight concrete in contact with soil use Type II, II/V, or V cement.

9.13 Storm Water BMPs

As previously discussed, the site subsurface soils at the project site had factored infiltration rates ranging from a no infiltration condition to very slow variable infiltration rates. Based on the geologic contact between the topsoil and the underlying granitic rock, attempts to infiltrate stormwater are anticipated to result in lateral movement, ponding, and/or mounding of stormwater and perched water conditions. Additionally, due to the presence of medium to highly expansive soils onsite, such conditions are anticipated to adversely affect surrounding improvements. Accordingly, we recommend that the project consider the use of pavement edge drains and cutoff curbs along the sides of infiltration devices to reduce the potential for lateral migration of water. Additionally, we recommend that permanent infiltration devices incorporate an overflow pipe that is connected to an appropriate outlet. 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 County of San Diego BMP Design Manual (2020) are met.

10 PLAN REVIEW AND CONSTRUCTION OBSERVATION

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory test pits. 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 letter 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.

11 LIMITATIONS

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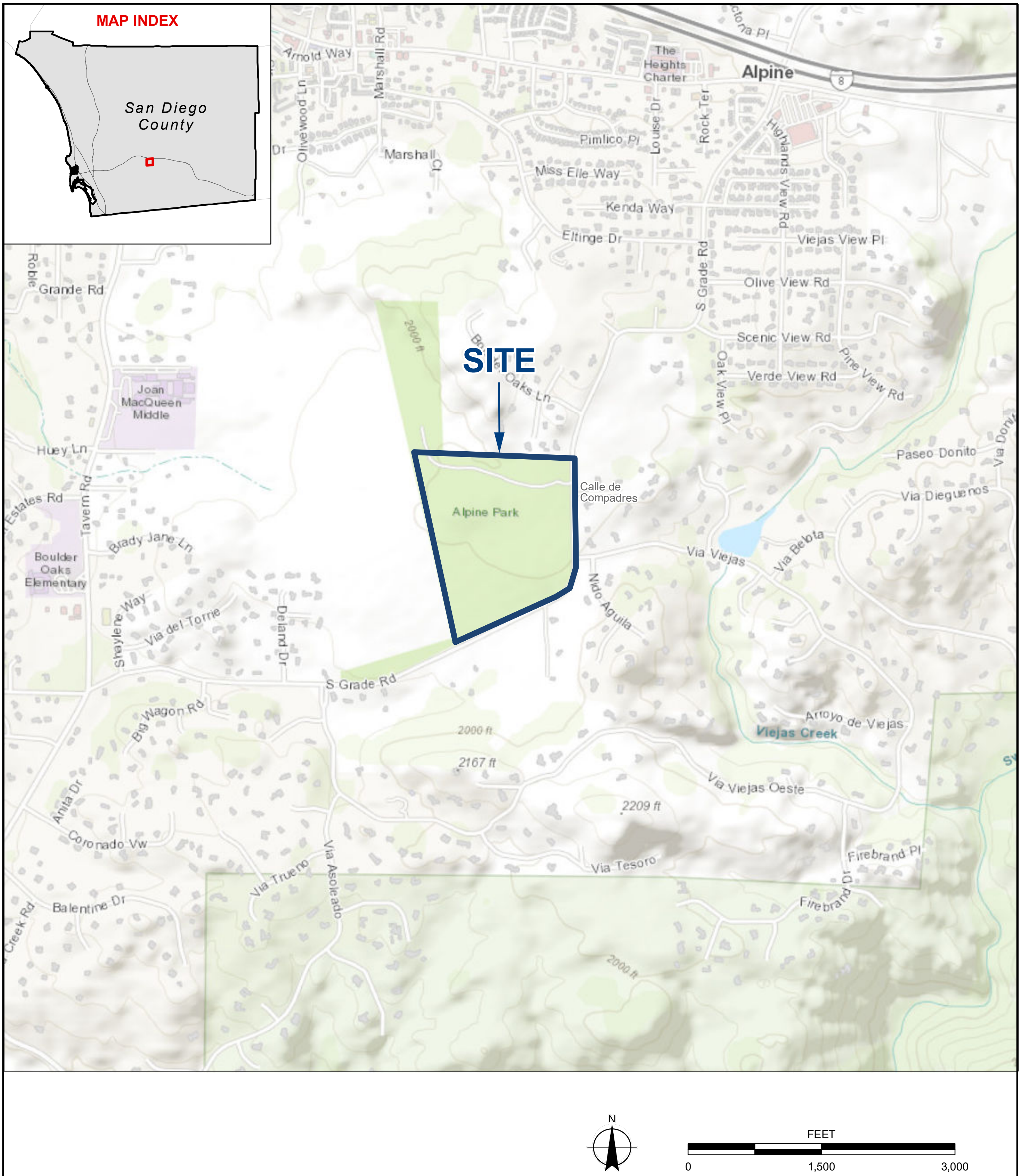
FIGURES



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

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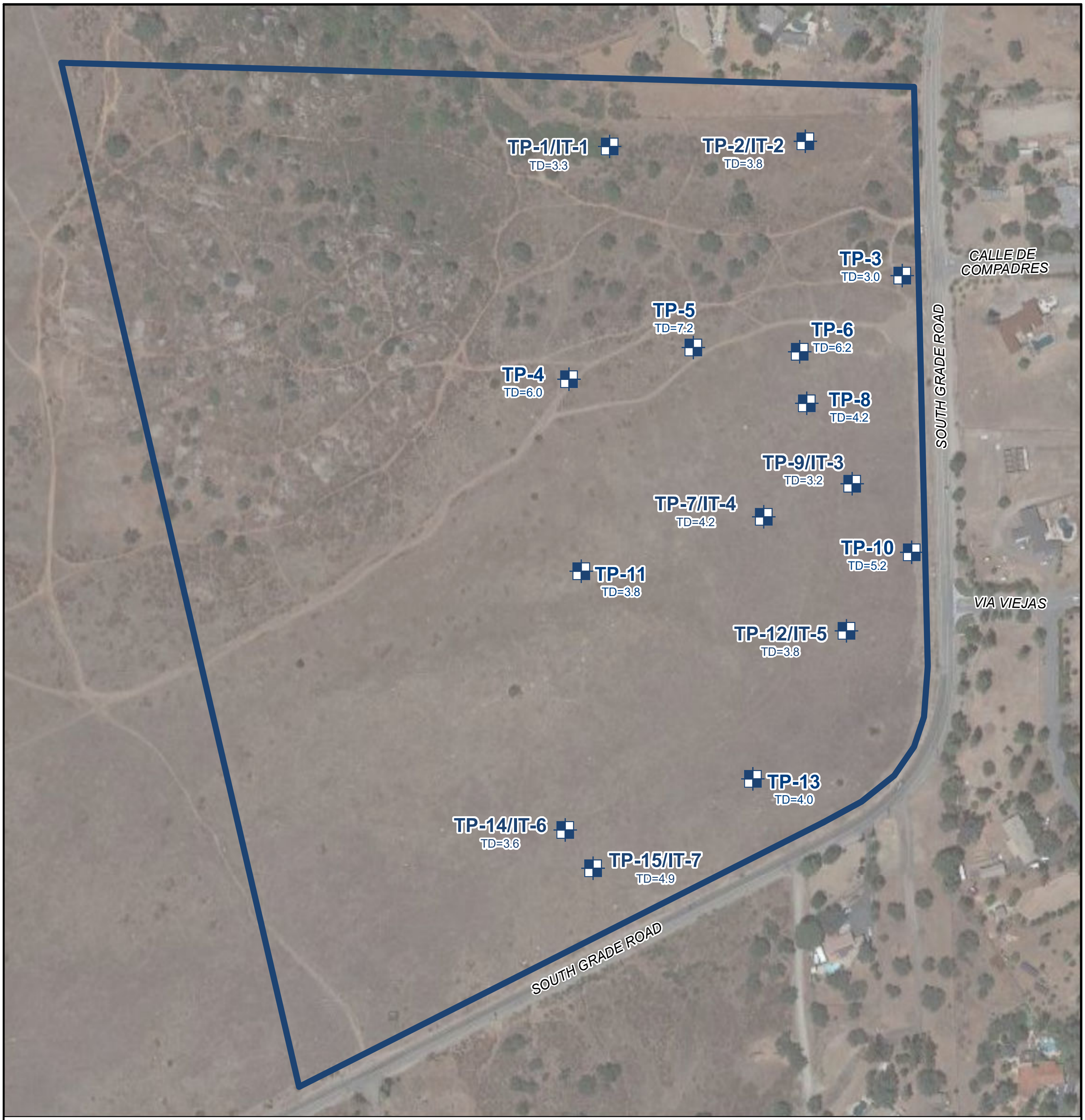
FIGURE 1

SITE LOCATION

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

109107001 | 12/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



LEGEND

▬	SITE BOUNDARY
TP-15/IT-7 TD=4.4 ⊞	TEST PIT/INFILTRATION TEST TD=TOTAL DEPTH IN FEET

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

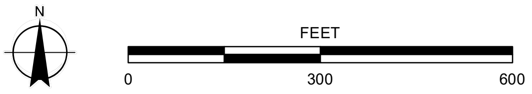


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FIGURE 2

TEST PIT AND INFILTRATION TEST LOCATIONS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

109107001 | 12/20

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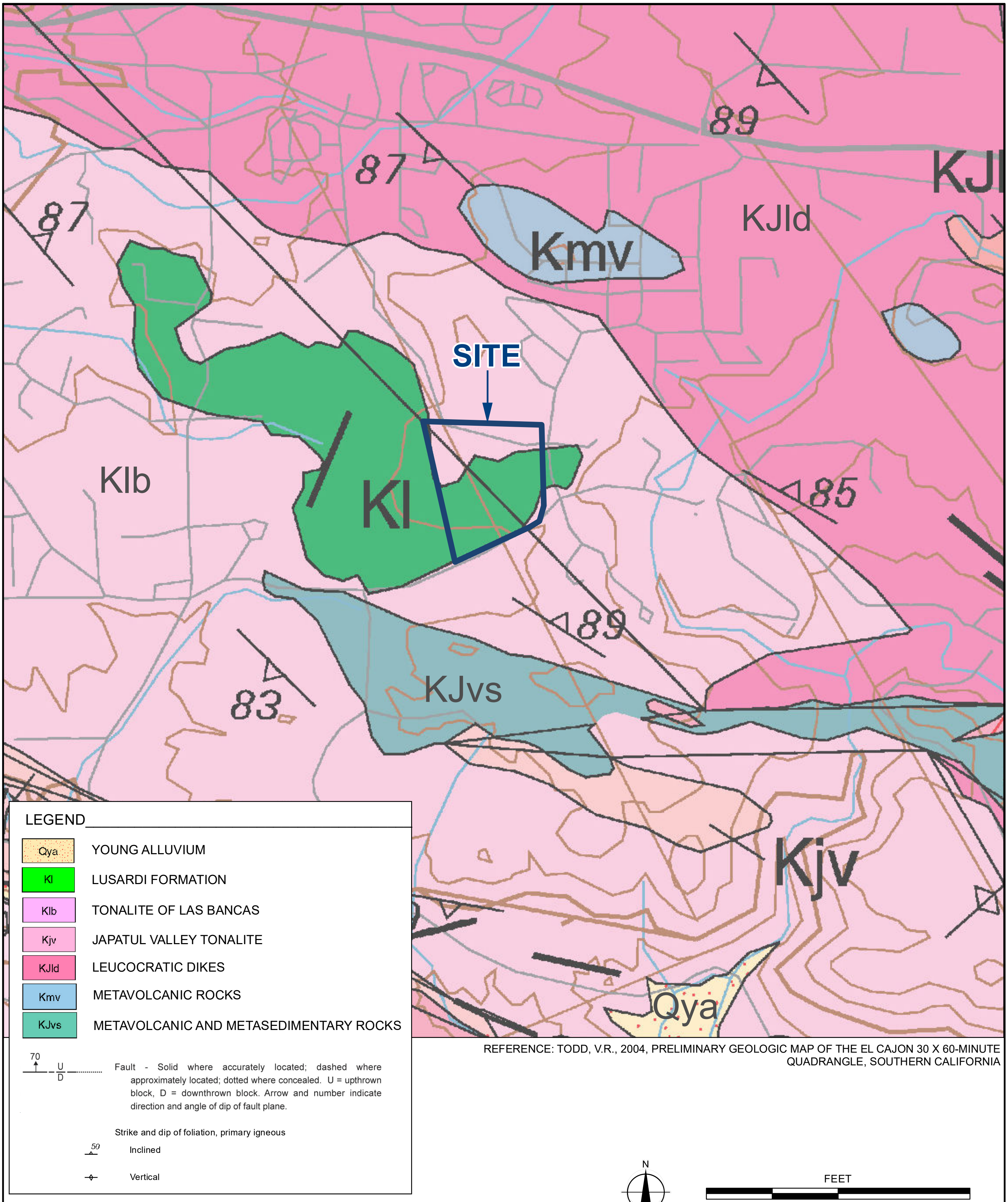

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

GEOLOGY

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

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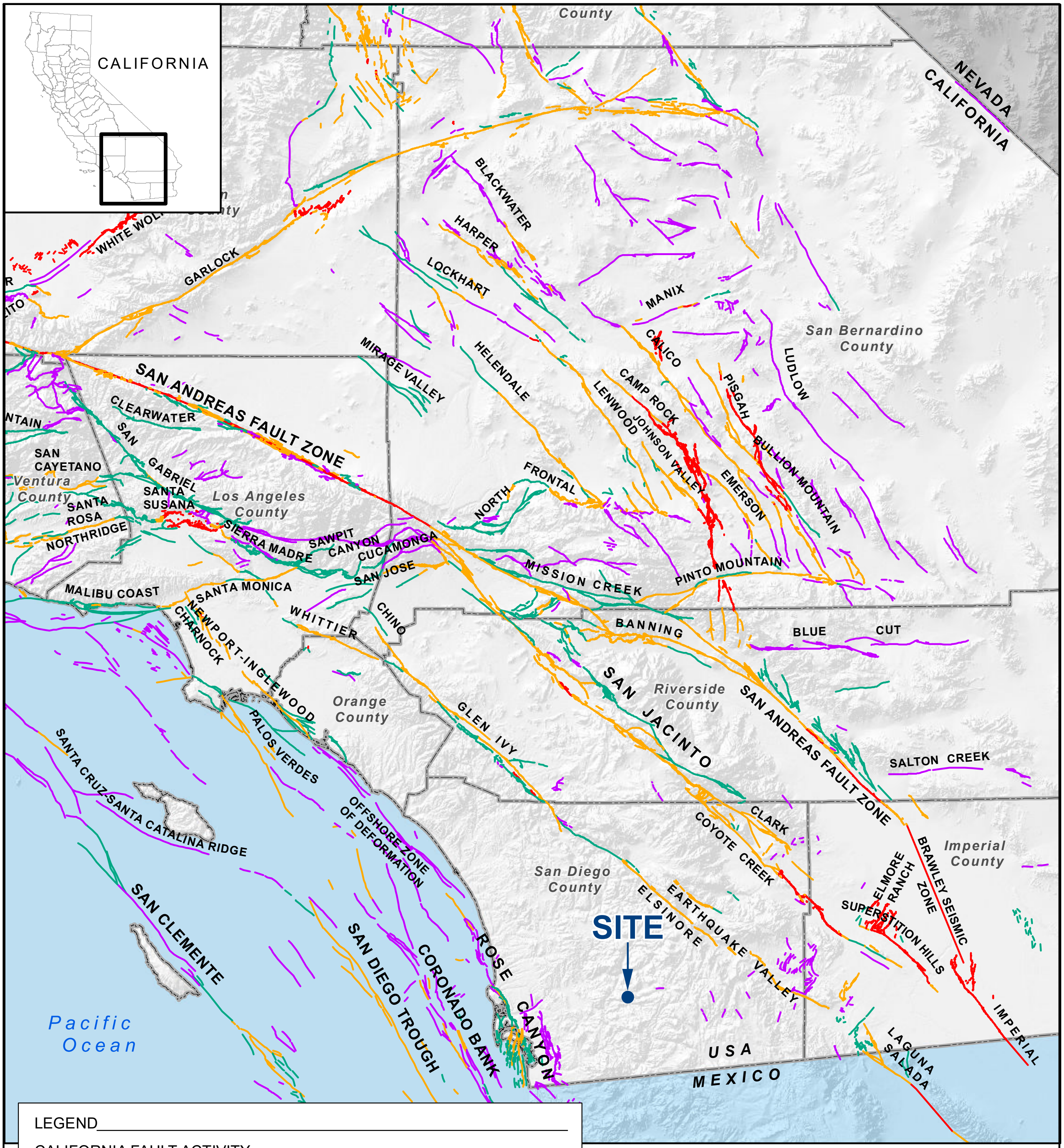

LEGEND

CALIFORNIA FAULT ACTIVITY

🟥 HISTORICALLY ACTIVE	🟪 QUATERNARY (POTENTIALLY ACTIVE)
🟧 HOLOCENE ACTIVE	STATE/COUNTY BOUNDARY
🟩 LATE QUATERNARY (POTENTIALLY ACTIVE)	

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

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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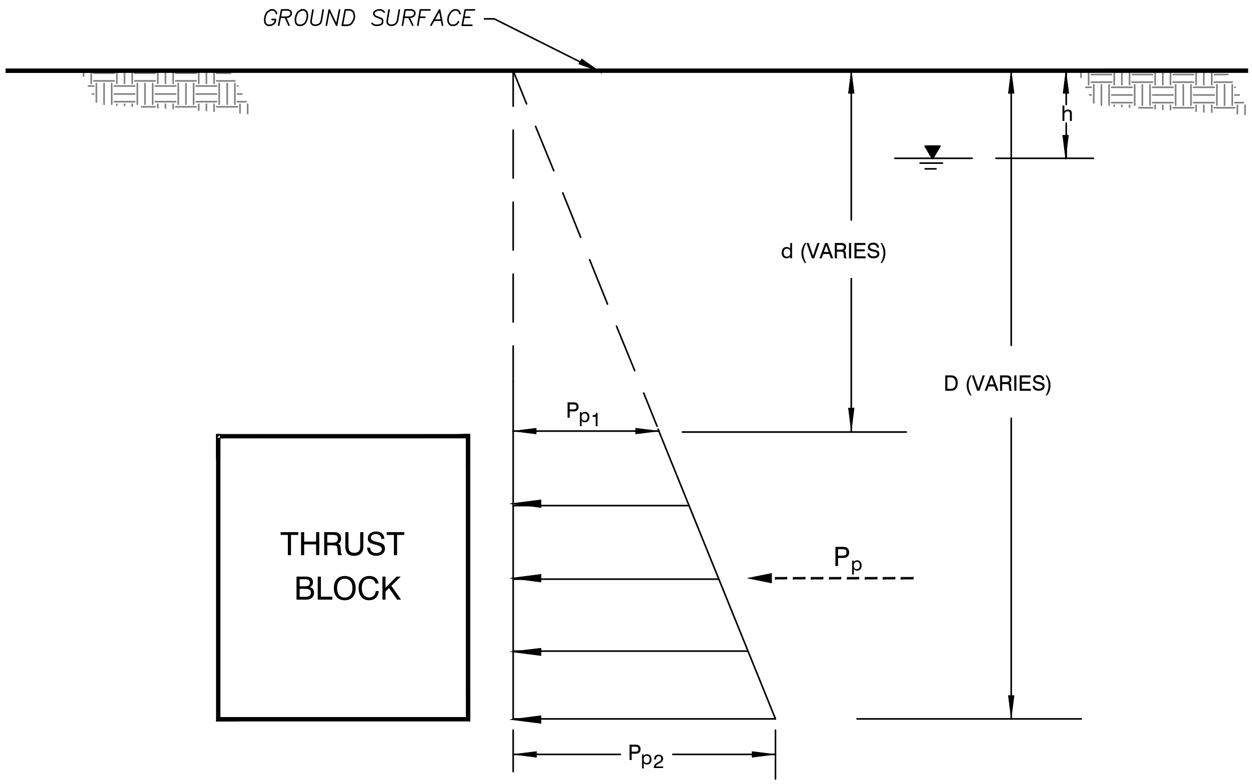
FIGURE 4

FAULT LOCATIONS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTES:

1. GROUNDWATER BELOW BLOCK

   $P_p = 150\ (D^2 - d^2)$ lb/ft

2. ASSUMES BACKFILL IS GRANULAR MATERIAL
3. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
4. D, d AND h ARE IN FEET

109107001 DETAIL FIGS.DWG AOB

NOT TO SCALE

FIGURE 5

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

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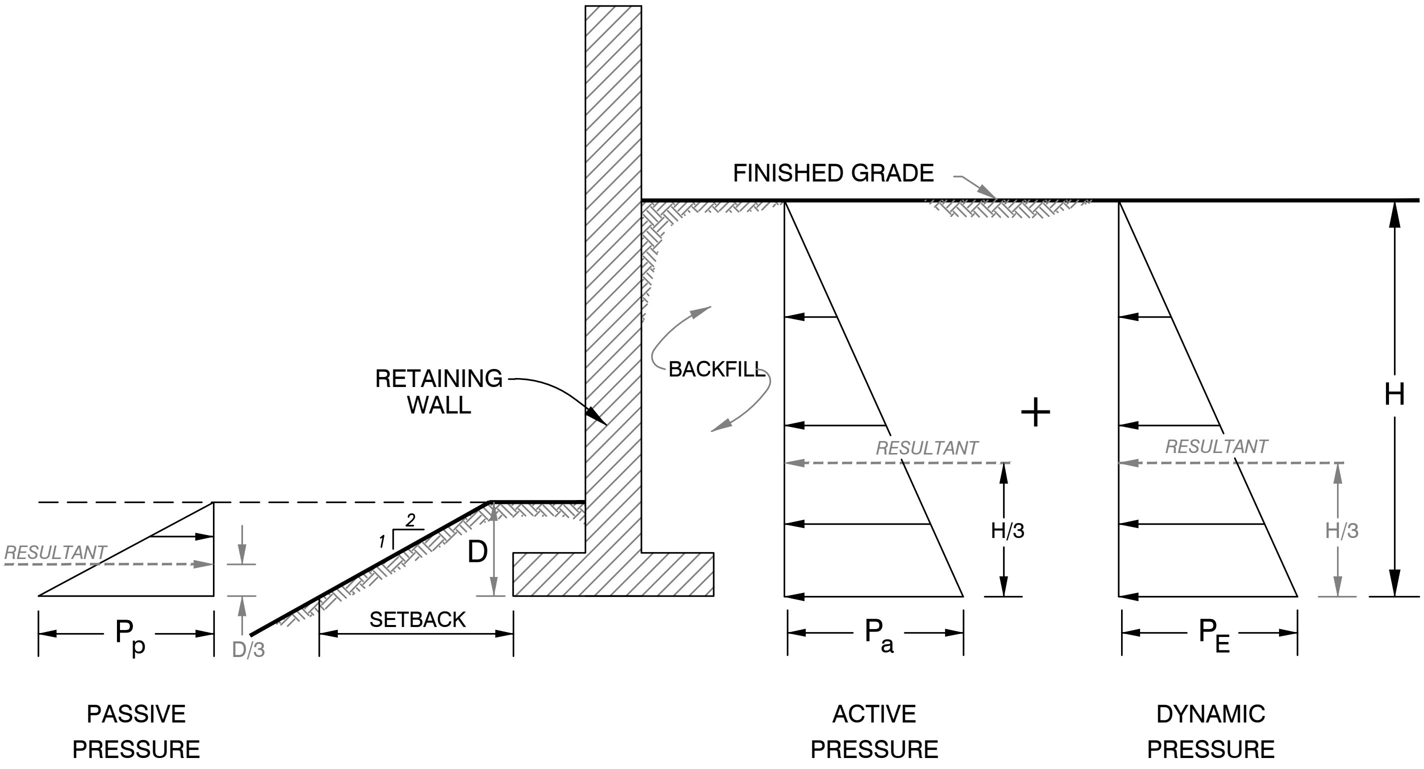
ALPINE, CALIFORNIA

109107001 I 12/20



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS AS SPECIFIED IN SECTION 9.5 OF THE REPORT 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

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 $^{(2)}$
	40 H	65 H
P_p	Level Ground	2H:1V Descending Ground
	300 D	130 D

NOT TO SCALE

109107001 DETAIL FIGS.DWG AOB

FIGURE 6

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

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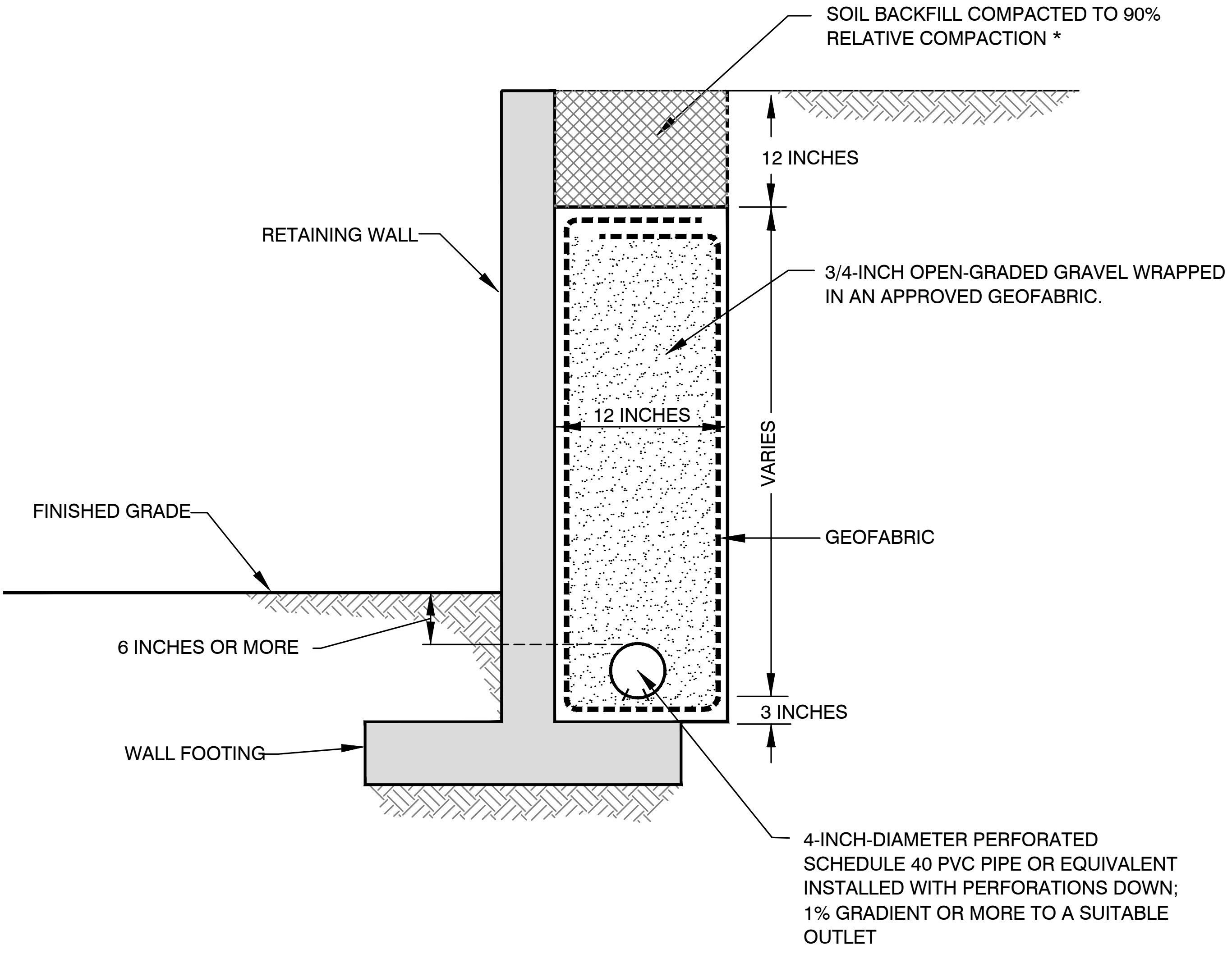

*BASED ON ASTM D1557

NOT TO SCALE

109107001 DETAIL FIGS.DWG AOB

FIGURE 7

RETAINING WALL DRAINAGE DETAIL

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APPENDIX A

Test Pit Logs

APPENDIX A

TEST PIT 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 test pits. The samples were bagged and transported to the laboratory for testing.

DATE EXCAVATED ____________________

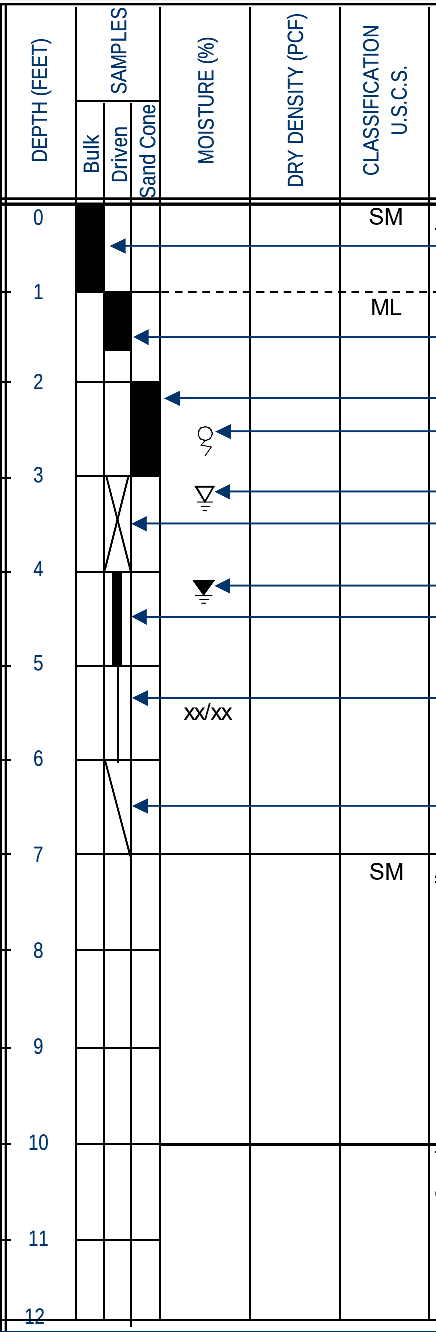
GROUND ELEVATION ____________________

METHOD OF EXCAVATION ____________________

TEST PIT DIAGRAM

EXCAVATION LOG EXPLANATION SHEET

DESCRIPTION



FILL:

Bulk sample.

Dashed line denotes material change.

Drive sample.

Sand cone performed.

Seepage.

Groundwater encountered during excavation.

No recovery with drive sampler.

Groundwater encountered after excavation.

Sample retained by others.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

ALLUVIUM:

Solid line denotes unit change.

Attitude: Strike/Dip

b: Bedding	s: Shear
c: Contact	bss: Basal Slide Surface
j: Joint	sf: Shear Fracture
f: Fracture	sz: Shear Zone
F: Fault	sbs: Sheared Bedding Surface
cs: Clay Seam	

The total depth is a solid line that is drawn at the bottom of the excavation log.

SCALE 1 inch = 2 feet

FIGURE B-1

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TEST PIT LOG

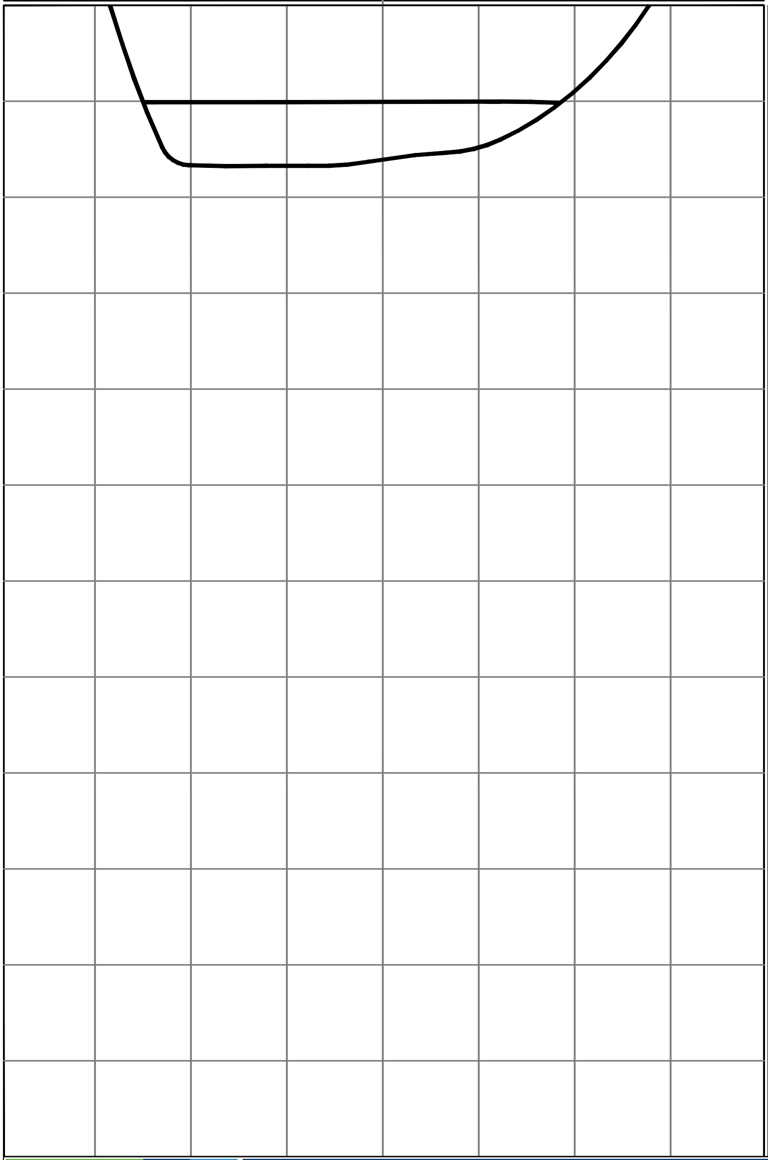
EXPLANATION OF TEST PIT, CORE, TRENCH AND HAND AUGER LOG SYMBOLS

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020
TEST PIT NO. TP-1 / IT-1
GROUND ELEVATION 2,002' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION Northwest Portion of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SC	TOPSOIL: Dark brown, moist, medium dense, clayey SAND; with fine gravel.
	■						DECOMPOSED GRANITIC ROCK : Brownish/yellowish red, moist, weathered, friable, unconsolidated DECOMPOSED GRANITIC ROCK; iron oxide staining.
4							Total Depth = 3.3 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020.
8							
12							
16							
20							
24							

Notes: Groundwater, though not encountered at the time of excavation, 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.

SCALE = 1 in./4 ft.

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TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

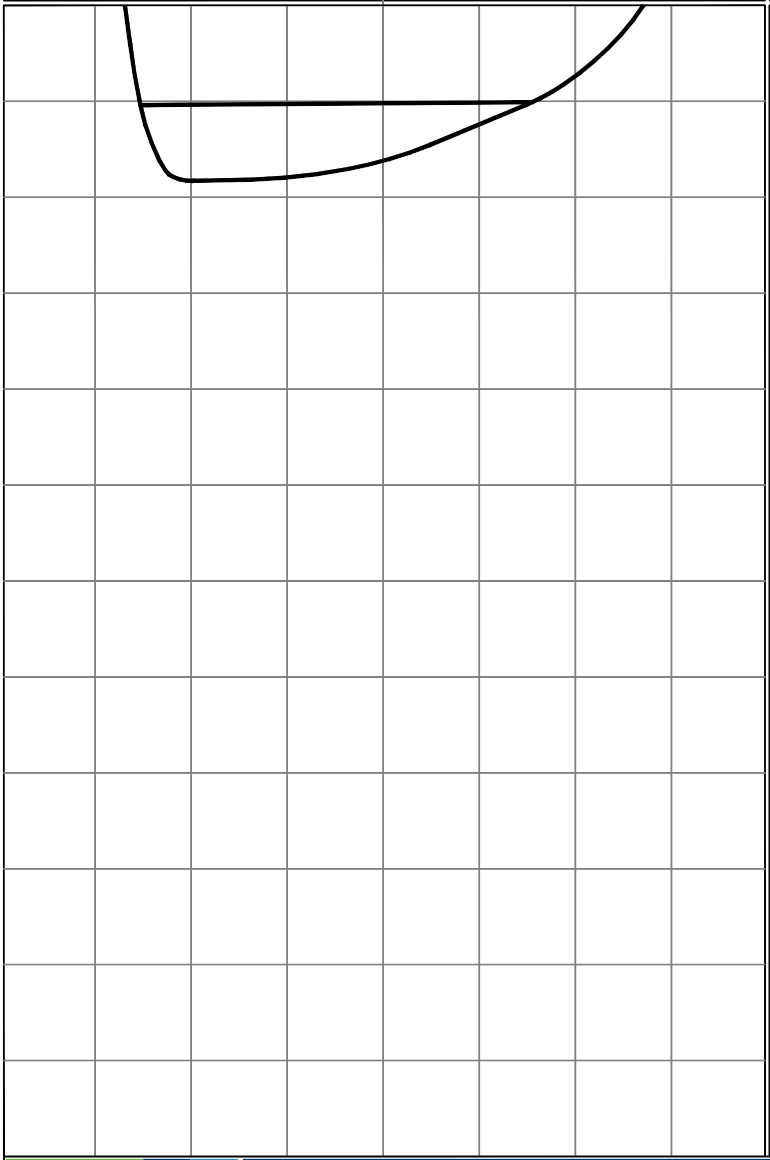
PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020 TEST PIT NO. TP-2 / IT-2

GROUND ELEVATION 2,020' ± (MSL) LOGGED BY BAB

METHOD OF EXCAVATION Backhoe

LOCATION Northeast Portion of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SM	TOPSOIL: Dark brown, moist, medium dense, silty SAND; micaceous.
	■						DECOMPOSED GRANITIC ROCK : Brownish, moist, DECOMPOSED GRANITIC ROCK; iron oxide staining.
4							Total Depth = 3.8 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020. Notes: Groundwater, though not encountered at the time of excavation 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.
8							
12							
16							
20							
24							

SCALE = 1 in./4 ft.

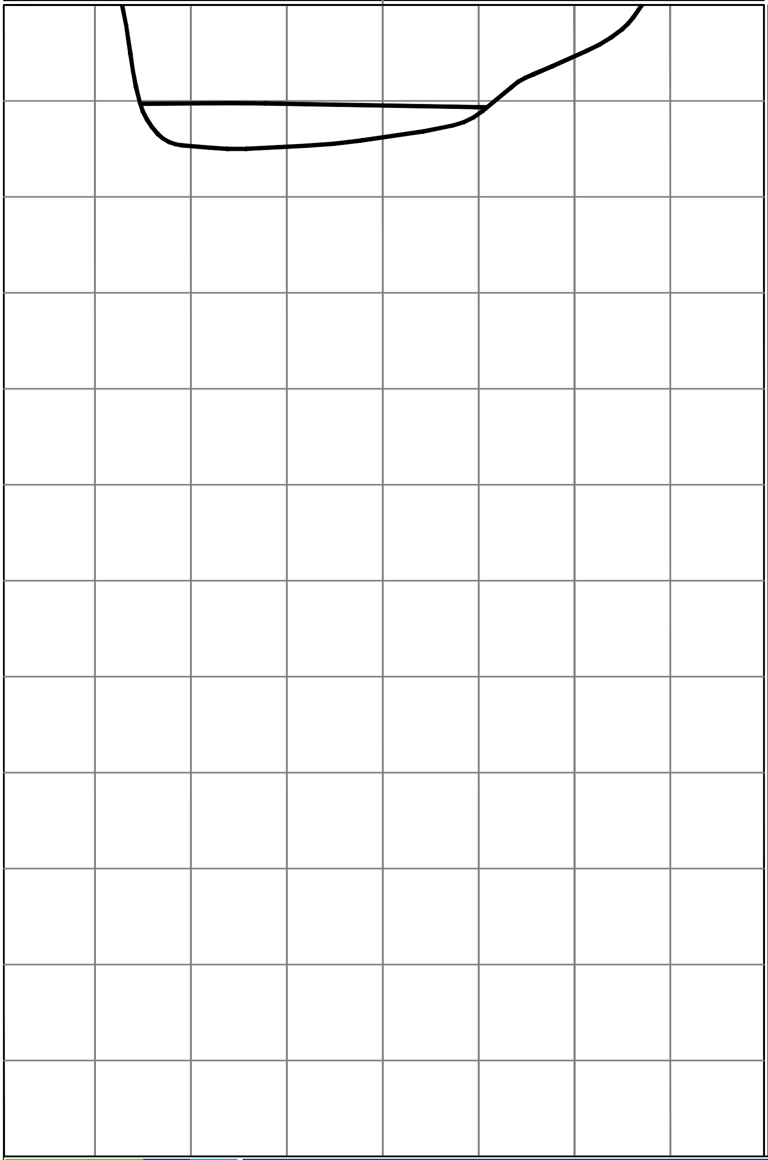
Ninyo & Moore

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020
TEST PIT NO. TP-3
GROUND ELEVATION 2,032' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION Northeast Portion of Site



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					CL	TOPSOIL: Dark brown, moist, stiff to very stiff, sandy CLAY; with fine gravels and rootlets.
	■						DECOMPOSED GRANITIC ROCK: Mottled light brownish yellowish red, moist, unconsolidated, friable, DECOMPOSED GRANITIC ROCK.
4 8 12 16 20 24							Total Depth = 3.0 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020. Notes: Groundwater, though not encountered at the time of excavation 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.

SCALE = 1 in./4 ft.

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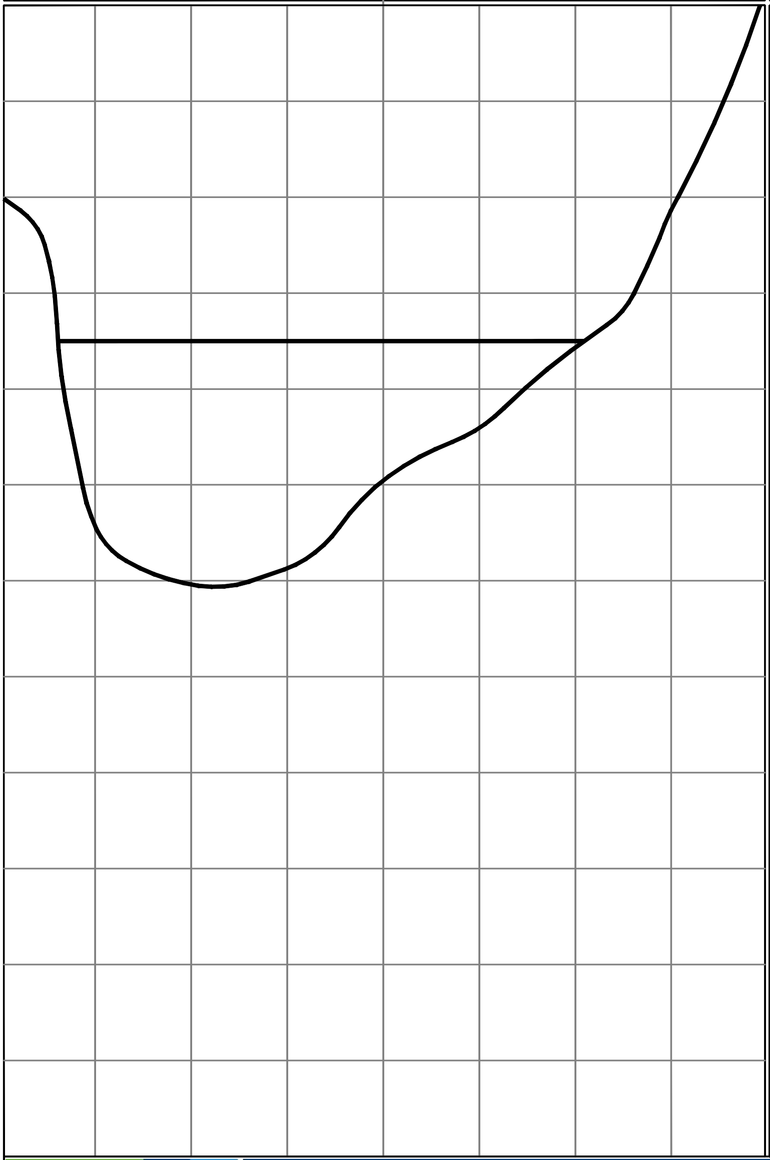
TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020
TEST PIT NO. TP-4
GROUND ELEVATION 2,028' ± (MSL)
LOGGED BY SJQ
METHOD OF EXCAVATION Backhoe
LOCATION Northwest Portion of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					CL	<u>TOPSOIL</u>: Dark brown, moist, stiff to very stiff, silty sandy CLAY; with scattered gravels and rootlets.
4	■						<u>DECOMPOSED GRANITIC ROCK</u>: Yellowish brown, dry, moderately weathered, weakly to moderately friable, medium-grained DECOMPOSED GRANITIC ROCK; micaceous.
6							Total Depth = 6.0 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.
8							
10							
12							

SCALE = 1 in./2 ft.

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TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

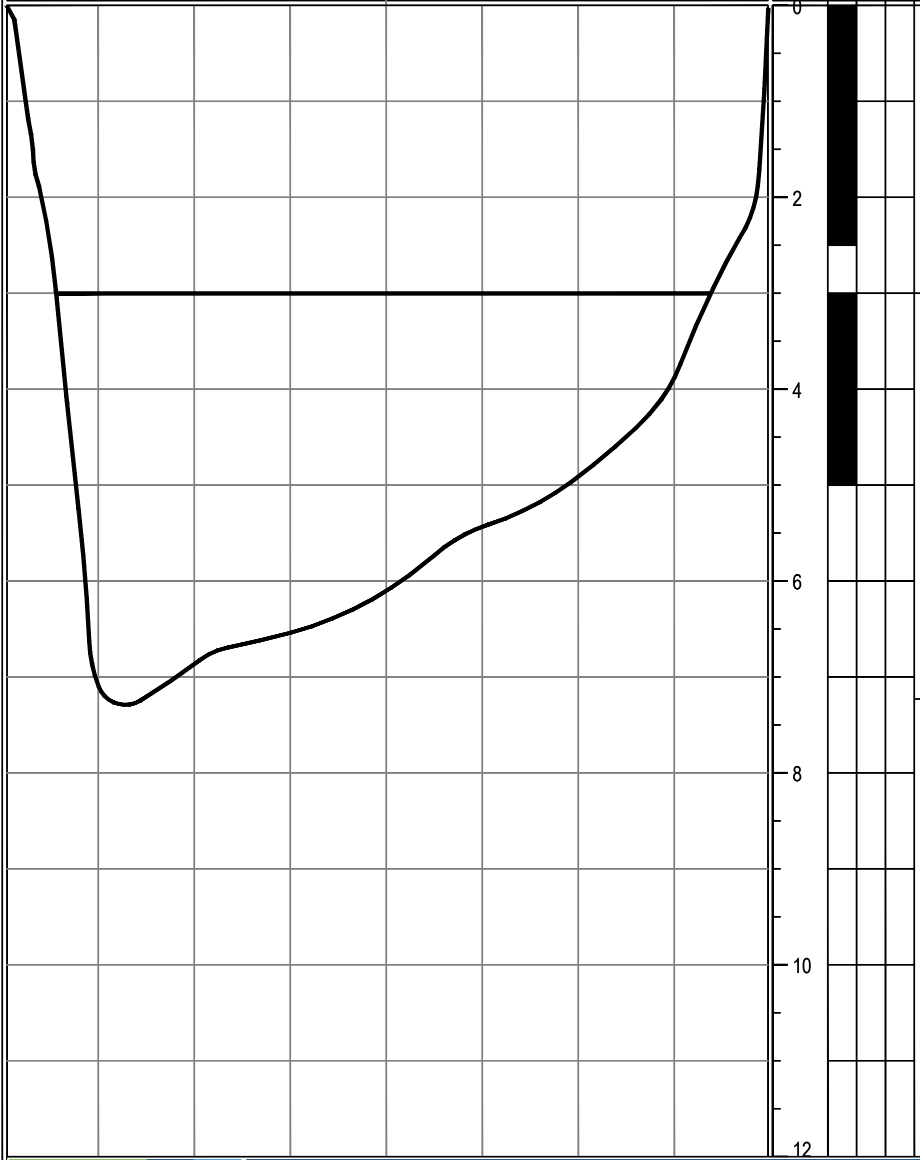
PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020 TEST PIT NO. TP-5

GROUND ELEVATION 2,026' ± (MSL) LOGGED BY SJQ

METHOD OF EXCAVATION Backhoe

LOCATION North Portion of Site



MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
		CL	TOPSOIL: Dark brown, moist, stiff to very stiff, silty sandy CLAY; with scattered gravel and rootlets.
			DECOMPOSED GRANITIC ROCK : Yellowish brown to reddish brown, moist, highly weathered, highly friable, DECOMPOSED GRANITIC ROCK.
			Total Depth = 7.2 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020. Notes: Groundwater, though not encountered at the time of excavation 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.

SCALE = 1 in./2 ft.

FIGURE A-5

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TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

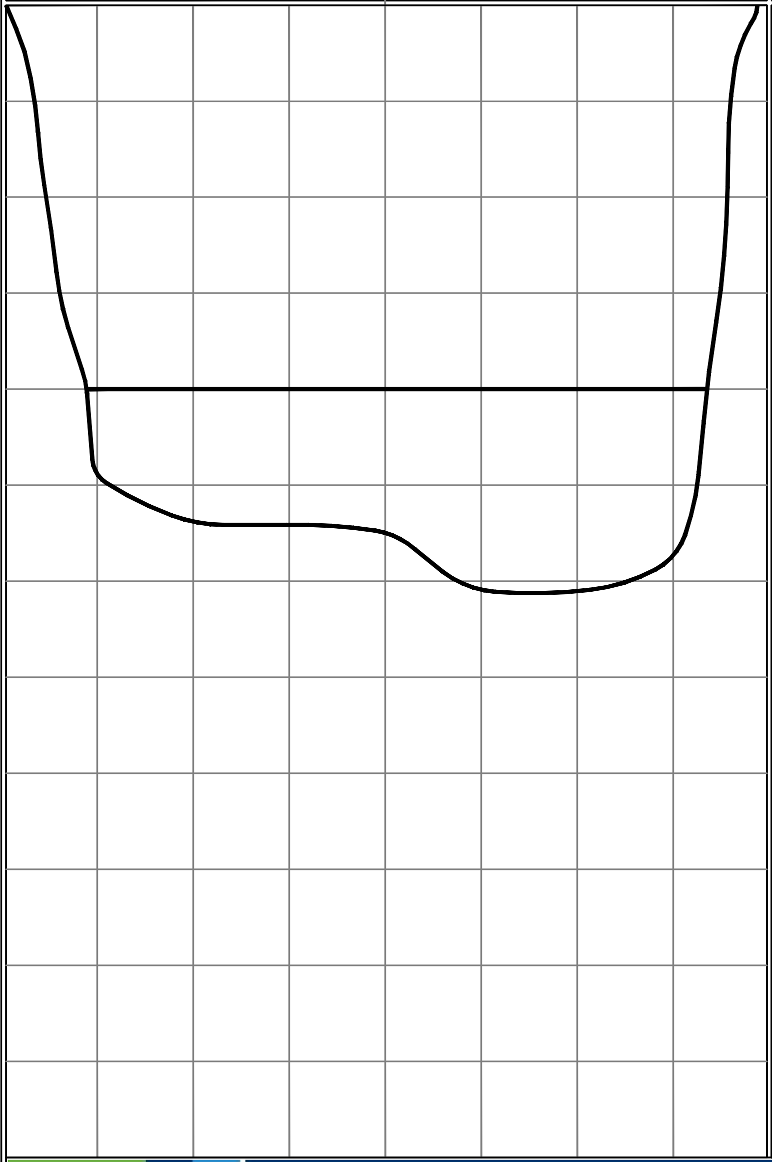
PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020 TEST PIT NO. TP-6

GROUND ELEVATION 2,024' ± (MSL) LOGGED BY BAB

METHOD OF EXCAVATION Backhoe

LOCATION North Portion of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					CL	<u>TOPSOIL</u>: Dark brown, moist, stiff to very stiff, silty sandy CLAY; with scattered gravel and rootlets.
2	■						
4	■						<u>DECOMPOSED GRANITIC ROCK</u>: Reddish brown to light yellowish brown, dry, moderately weathered, moderately friable, coarse-grained DECOMPOSED GRANITIC ROCK; micaceous.
6							Total Depth = 6.2 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.
8							
10							
12							

SCALE = 1 in./2 ft.

FIGURE A-6

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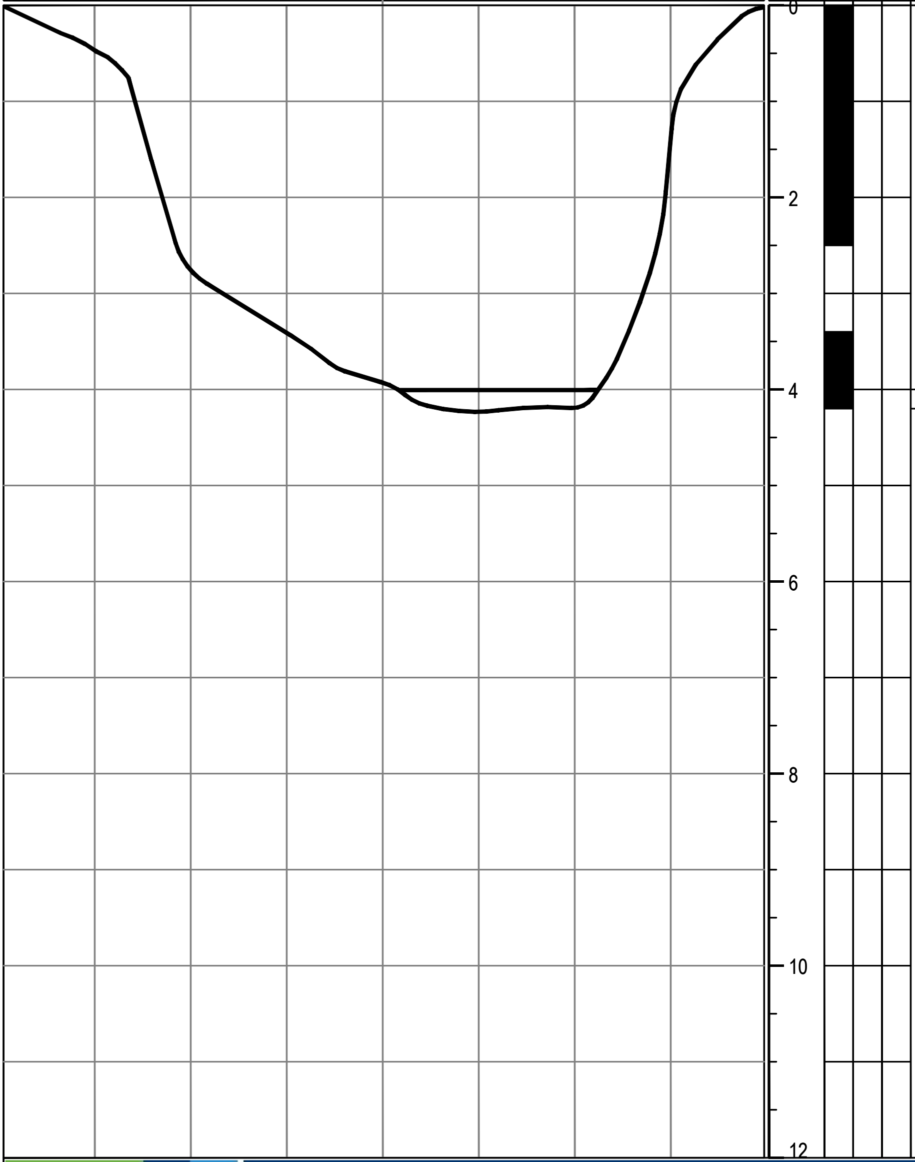
TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020 TEST PIT NO. TP-7 / IT-4
GROUND ELEVATION 2,010' ± (MSL) LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION Central Portion of Site



DEPTH (FEET)	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0			CL	<u>TOPSOIL</u>: Dark brown, moist, very stiff to hard, silty sandy CLAY; with scattered gravel.
4				<u>DECOMPOSED GRANITIC ROCK</u>: Yellowish to reddish brown, dry, moderately to highly weathered, moderately to highly friable, medium to coarse-grained GRANITIC ROCK.
				Total Depth = 4.2 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020.

<u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.

SCALE = 1 in./2 ft.

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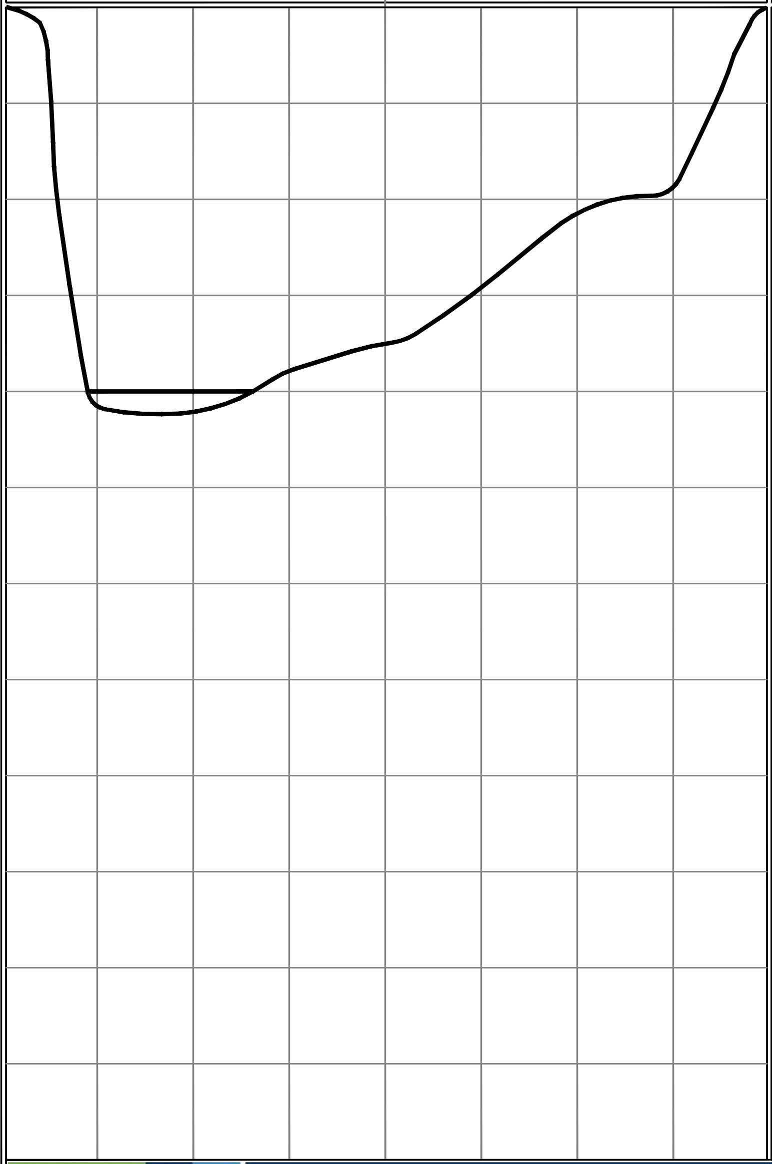
TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020
TEST PIT NO. TP-8
GROUND ELEVATION 2,020' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION Central Portion of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SC	<u>TOPSOIL</u>: Dark brown, moist, medium dense, clayey SAND; with scattered gravel and coarse sand; scattered rootlets.
2	■						
4							<u>DECOMPOSED GRANITIC ROCK</u>: Yellowish brown to reddish brown with green chlorite staining, dry, moderately to highly weathered, medium-grained DECOMPOSED GRANITIC ROCK.
							Total Depth = 4.2 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020.
6							<u>Notes:</u> Groundwater, though not encountered at the time of excavation may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
8							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							
12							

SCALE = 1 in./2 ft.

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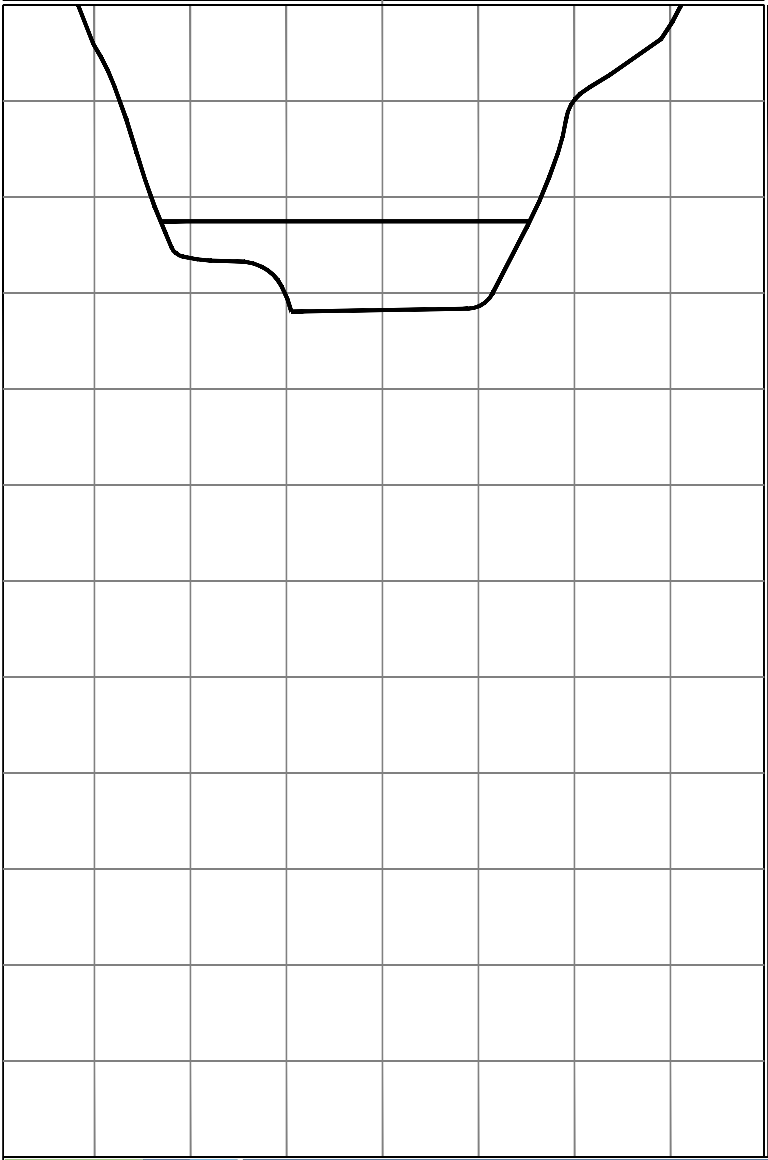
TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020
TEST PIT NO. TP-9 / IT-3
GROUND ELEVATION 2,011' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION East End of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					CL	<u>TOPSOIL</u>: Dark brown, moist, very stiff, silty sandy CLAY; with scattered gravel and rootlets.
2	■						<u>DECOMPOSED GRANITIC ROCK</u>: Yellowish brown to reddish brown, dry, highly weathered, strongly friable, GRANITIC ROCK; scattered iron oxide staining; scattered friable core stones approximately 3 inches diameter.
							Total Depth = 3.2 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.
4							
6							
8							
10							
12							

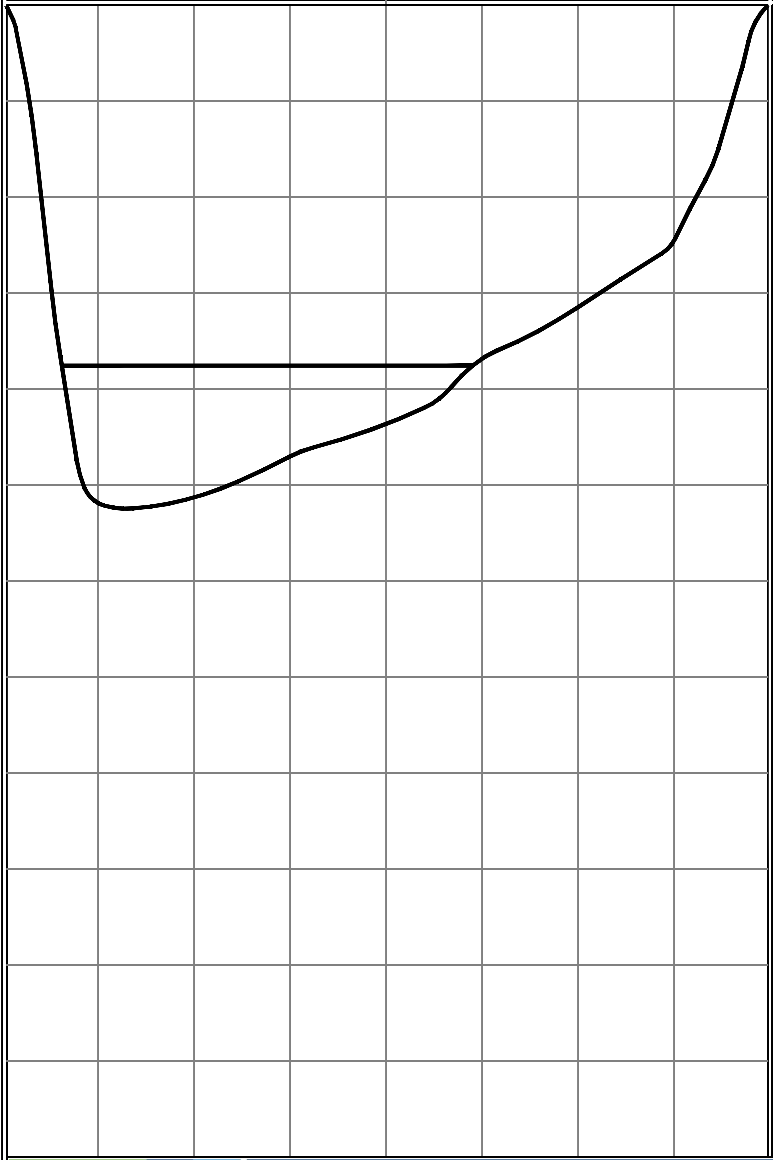
SCALE = 1 in./2 ft.

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020 TEST PIT NO. TP-10
GROUND ELEVATION 2,002' ± (MSL) LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION East End of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0 2	■					CL	<u>TOPSOIL</u>: Dark brown, very stiff, silty sandy CLAY; with scattered gravel, rootlets and caliche nodules and stringers.
4	■						<u>DECOMPOSED GRANITIC ROCK</u>: Yellowish brown to whitish yellow, dry, moderately to strongly weathered, moderately friable, GRANITIC ROCK; scattered iron oxide and greenish cholorite staining; occasional tonalite corestones.
6 8 10 12							Total Depth = 5.2 feet. Groundwater not encountered during excavation. Backfilled on 11/18/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.

SCALE = 1 in./2 ft.

FIGURE A-10

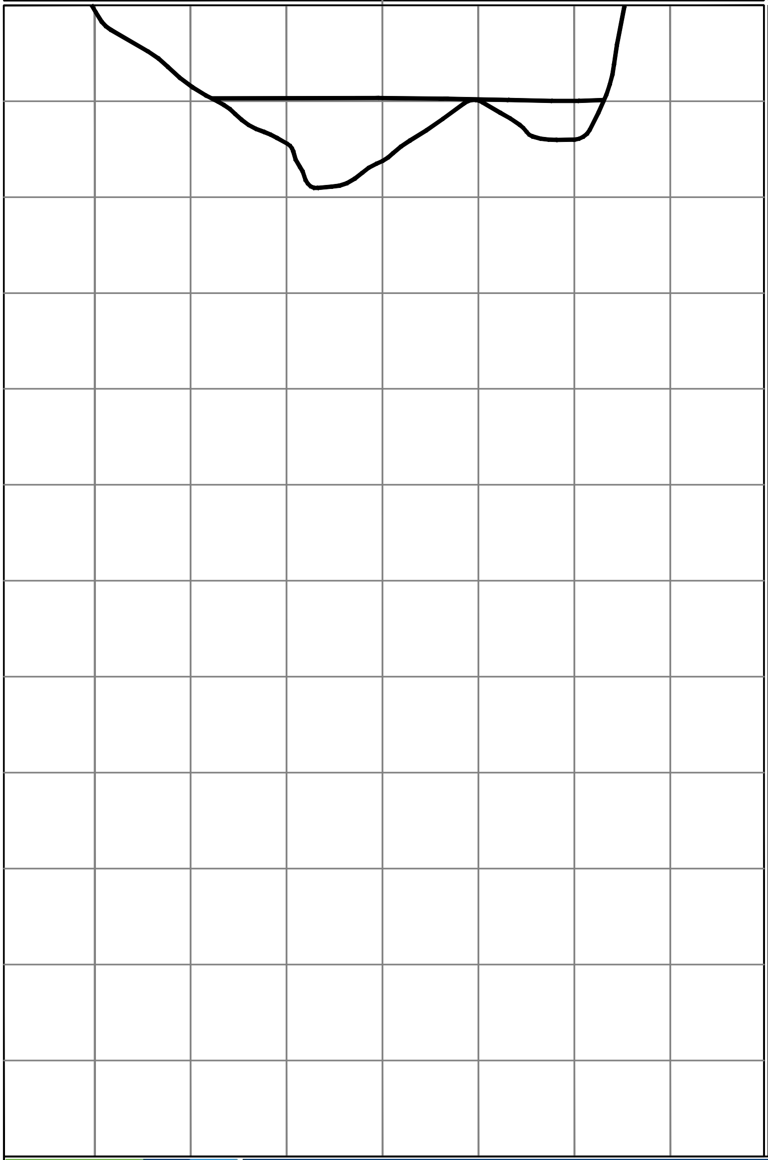
Ninyo & Moore

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020
TEST PIT NO. TP-11
GROUND ELEVATION 2,015' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION West Portion of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SC	<u>TOPSOIL</u>: Dark brown, moist, medium dense, clayey SAND; with fine gravel and numerous large angular boulders of granitic rock; approximately 1 to 2-1/2 feet in size.
	■						<u>DECOMPOSED GRANITIC ROCK</u> : Mottled, brownish yellow to reddish brown, moist, unconsolidated, friable, DECOMPOSED GRANITIC ROCK; weathered; iron oxide staining; with boulder size corestones.
4							Total Depth = 3.8 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.
8							
12							
16							
20							
24							

SCALE = 1 in./4 ft.

Ninyo & Moore

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/18/2020 TEST PIT NO. TP-12 / IT-5
GROUND ELEVATION 1,994' ± (MSL) LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION Southeast Portion of Site

DEPTH (FEET)	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0			CL	TOPSOIL: Dark brown, moist, stiff to very stiff, silty sandy CLAY; with scattered gravel and rootlets; occasional boulder.
2.5				DECOMPOSED GRANITIC ROCK: Light grayish brown to yellowish brown, dry, moderately to highly weathered, moderately friable, medium-grained, DECOMPOSED GRANITIC ROCK.

Total Depth = 3.8 feet.
Groundwater not encountered during excavation.
Backfilled on 11/18/2020.

Notes: Groundwater, though not encountered at the time of excavation 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.

SCALE = 1 in./2 ft.

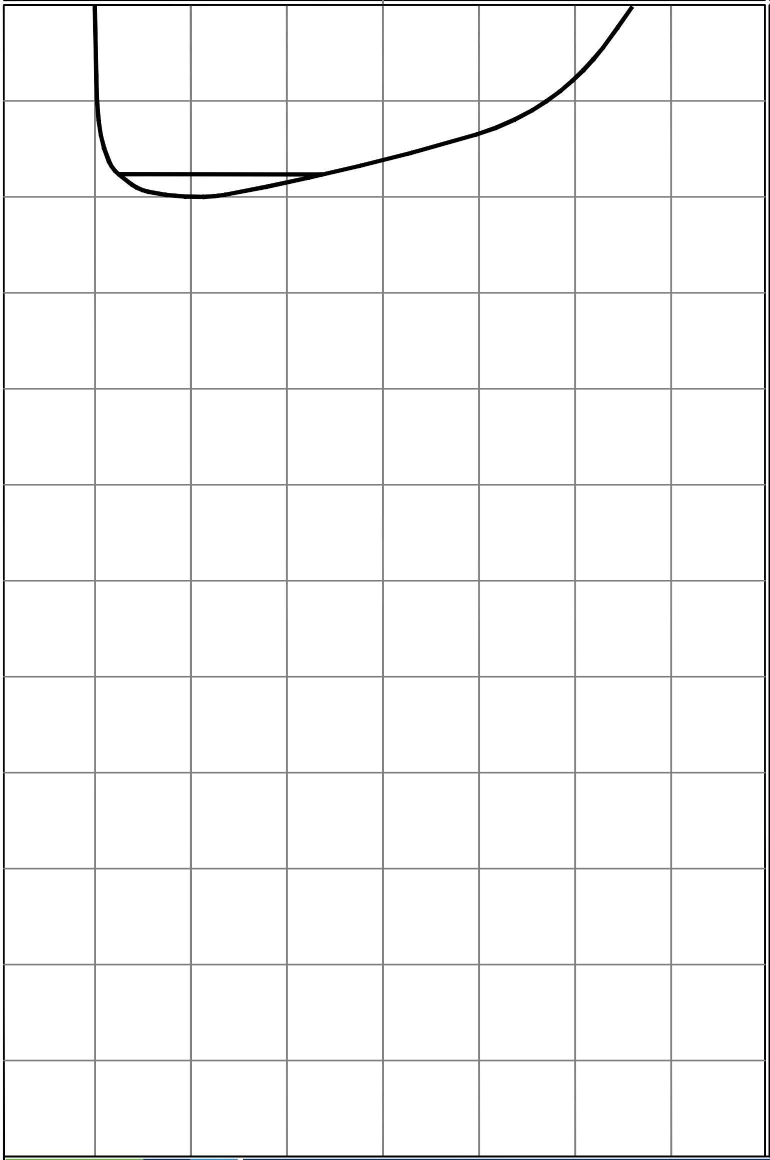
Ninyo & Moore

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020
TEST PIT NO. TP-13
GROUND ELEVATION 1,985' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION South End of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					CL	<u>TOPSOIL</u>: Dark brown to dark gray brown, moist, firm to very stiff, sandy CLAY; with fine gravel.
	■						<u>DECOMPOSED GRANITIC ROCK</u> : Mottled yellowish brown to reddish brown, moist, friable, unconsolidated to partially consolidated, DECOMPOSED GRANITIC ROCK.
4							Total Depth = 4.0 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020.
8							<u>Notes:</u> Groundwater, though not encountered at the time of excavation may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
12							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.
16							
20							
24							

SCALE = 1 in./2 ft.

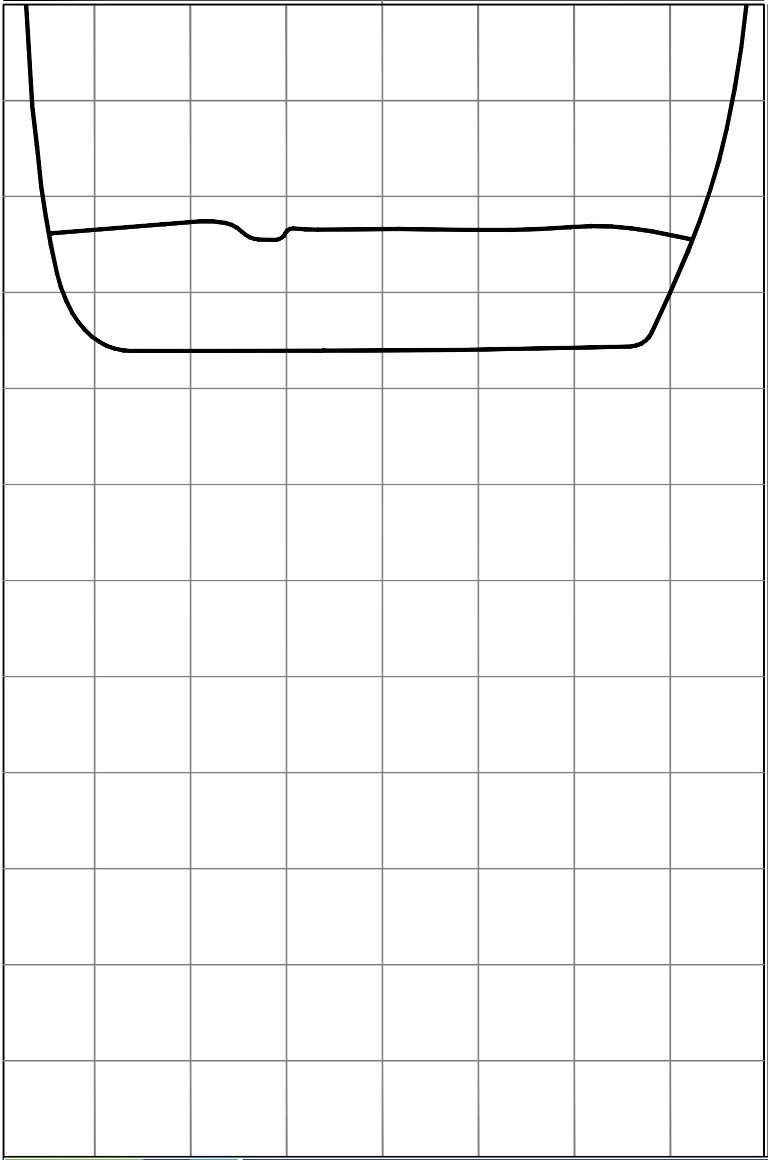
Ninyo & Moore

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020 TEST PIT NO. TP-14 / IT-6
GROUND ELEVATION 1,989' ± (MSL) LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION South End of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SC	<u>TOPSOIL</u>: Dark brown, moist, medium dense, clayey SAND; with fine gravel; several large boulders and rootlets.
2	■						<u>DECOMPOSED GRANITIC ROCK</u>: Light brownish yellow, moist, friable, unconsolidated, DECOMPOSED GRANITIC ROCK; highly altered; iron oxide staining.
4							Total Depth = 3.6 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.
6							
8							
10							
12							

SCALE = 1 in./2 ft.

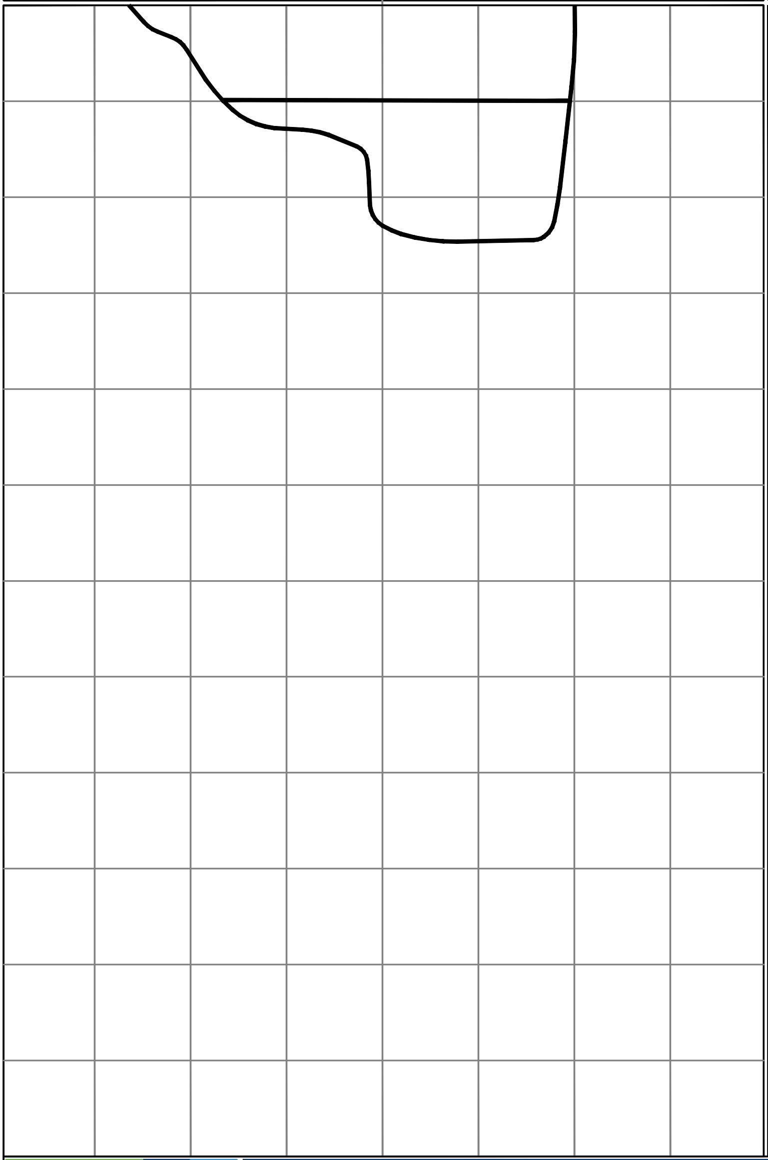
Ninyo & Moore

TEST PIT LOG

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

PROJECT NO.	DATE
109107001	12/20

DATE EXCAVATED 11/19/2020
TEST PIT NO. TP-15 / IT-7
GROUND ELEVATION 1,983' ± (MSL)
LOGGED BY BAB
METHOD OF EXCAVATION Backhoe
LOCATION South End of Site



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SC	<u>TOPSOIL</u>: Dark brown, moist, medium dense, clayey SAND; with fine gravel; several boulders up to 15 inches in size.
4	■						<u>DECOMPOSED GRANITIC ROCK</u> : Light brownish yellow, moist, friable, unconsolidated, DECOMPOSED GRANITIC ROCK.
8 12 16 20 24							Total Depth = 4.9 feet. Groundwater not encountered during excavation. Backfilled on 11/19/2020. <u>Notes:</u> Groundwater, though not encountered at the time of excavation 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.

SCALE = 1 in./4 ft.

FIGURE A-15

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory test pits 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 curves are shown on Figures B-1 through B-5. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Expansion Index Tests

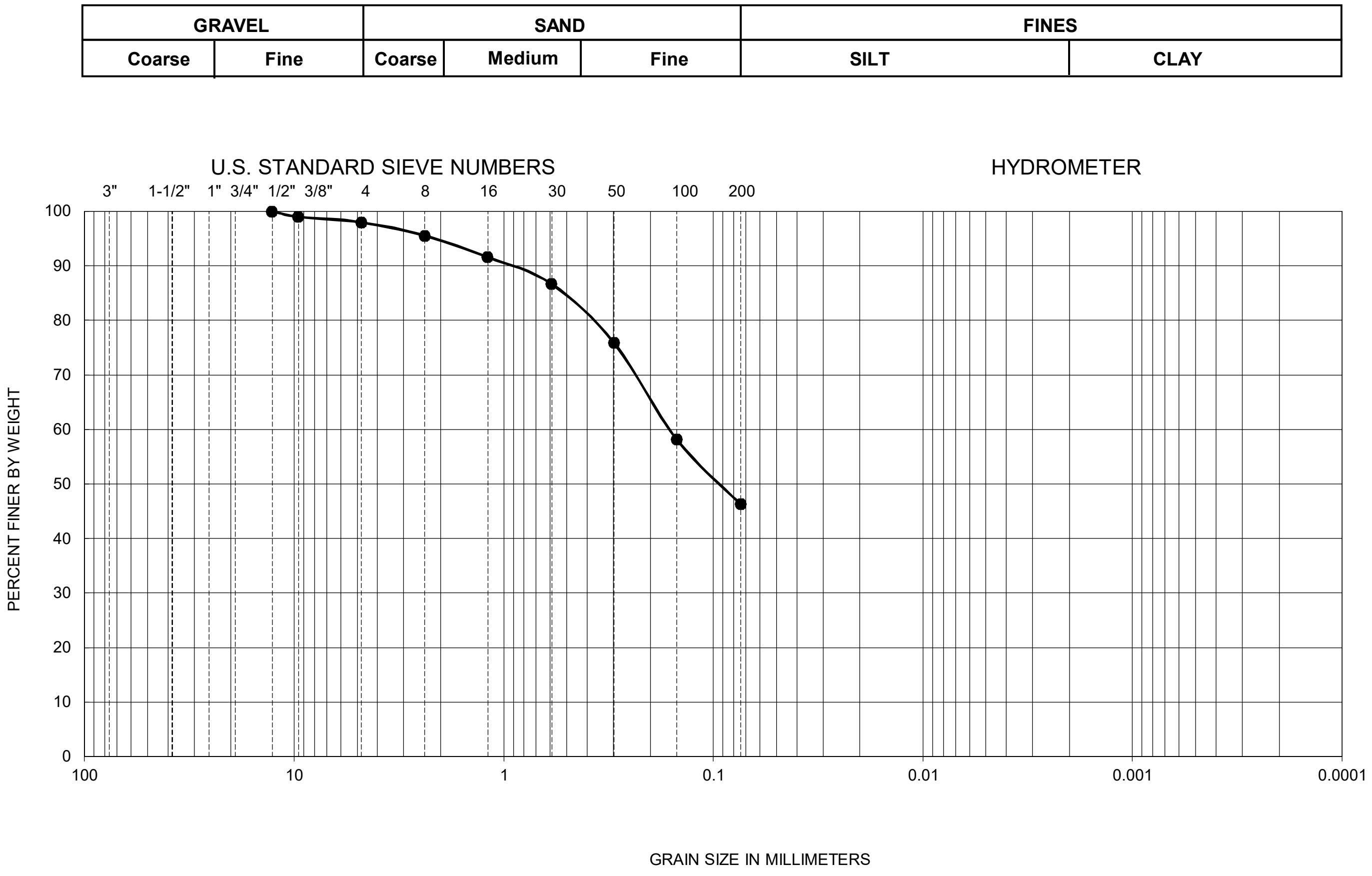
The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square 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-6.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on representative samples in general accordance with CT 643. The soluble sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-7.

R-Value

The resistance value, or R-value, for site soils was 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-8.



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
●	TP-8	0.0-2.5	--	--	--	--	--	--	--	--	46	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

GRADATION TEST RESULTS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

109107001 | 12/20

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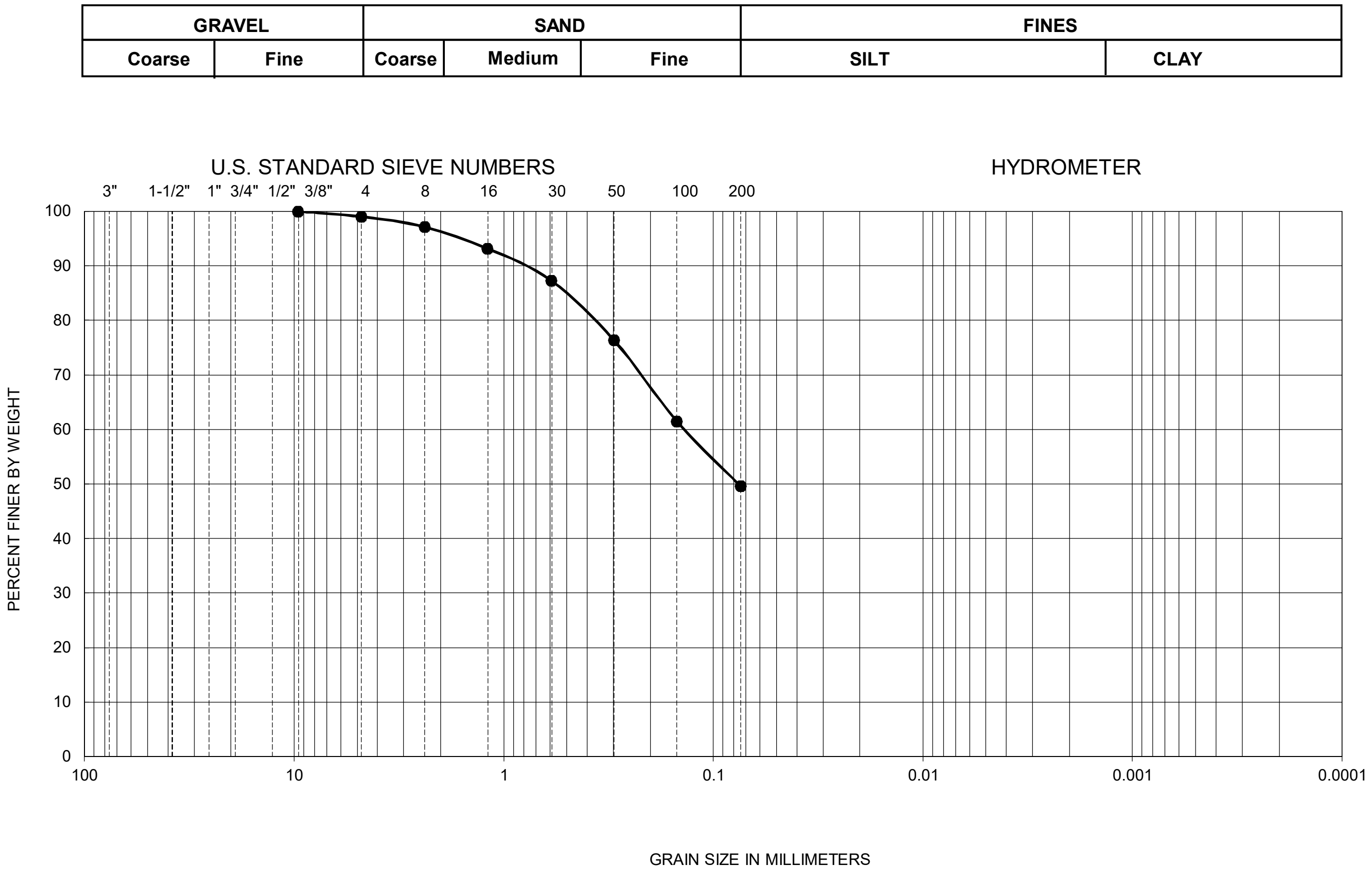


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	TP-10	0.0-3.3	--	--	--	--	--	--	--	--	50	CL

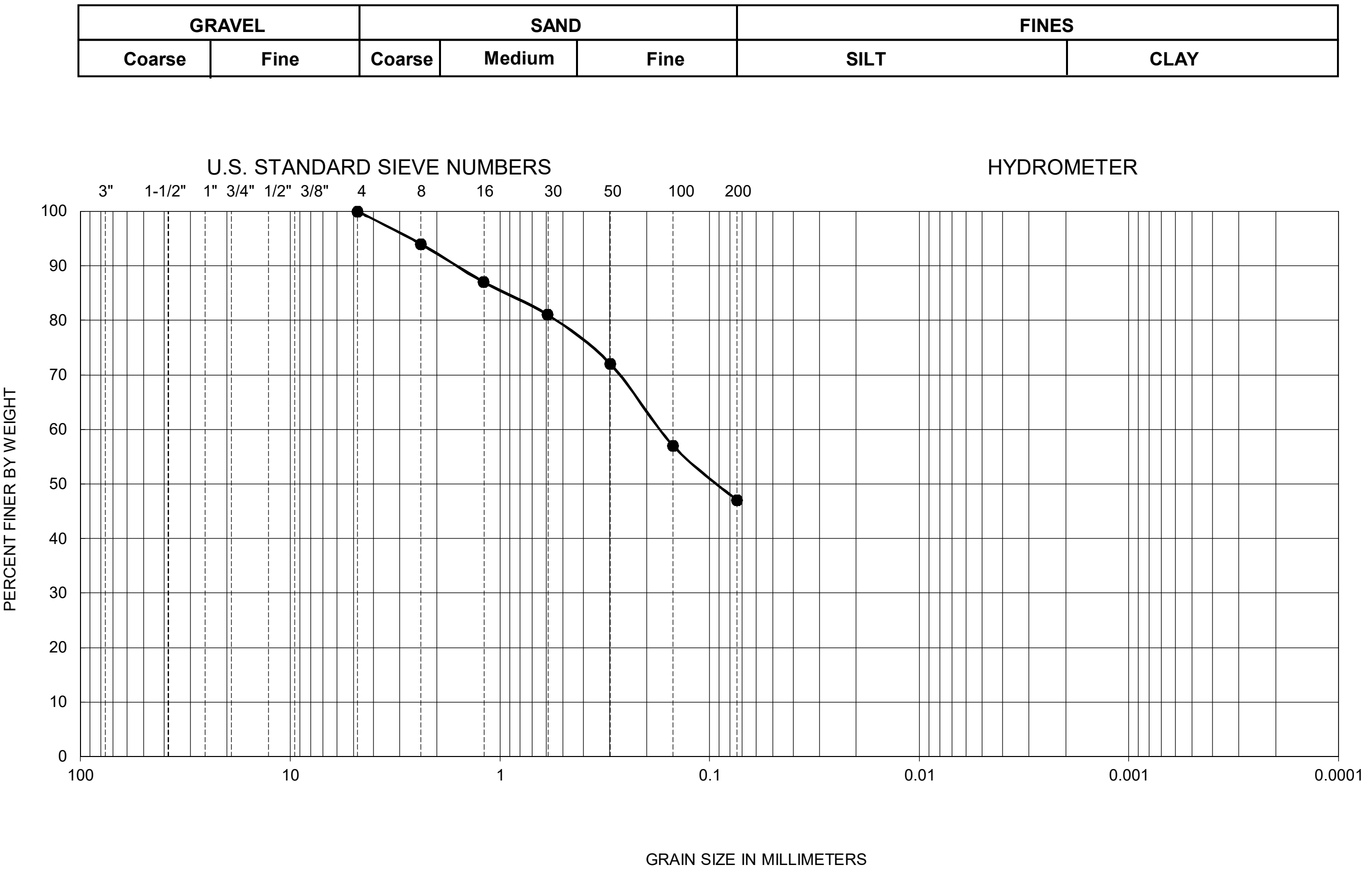
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

GRADATION TEST RESULTS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



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
●	TP-11	0.0-1.9	--	--	--	--	--	--	--	--	47	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-4

GRADATION TEST RESULTS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

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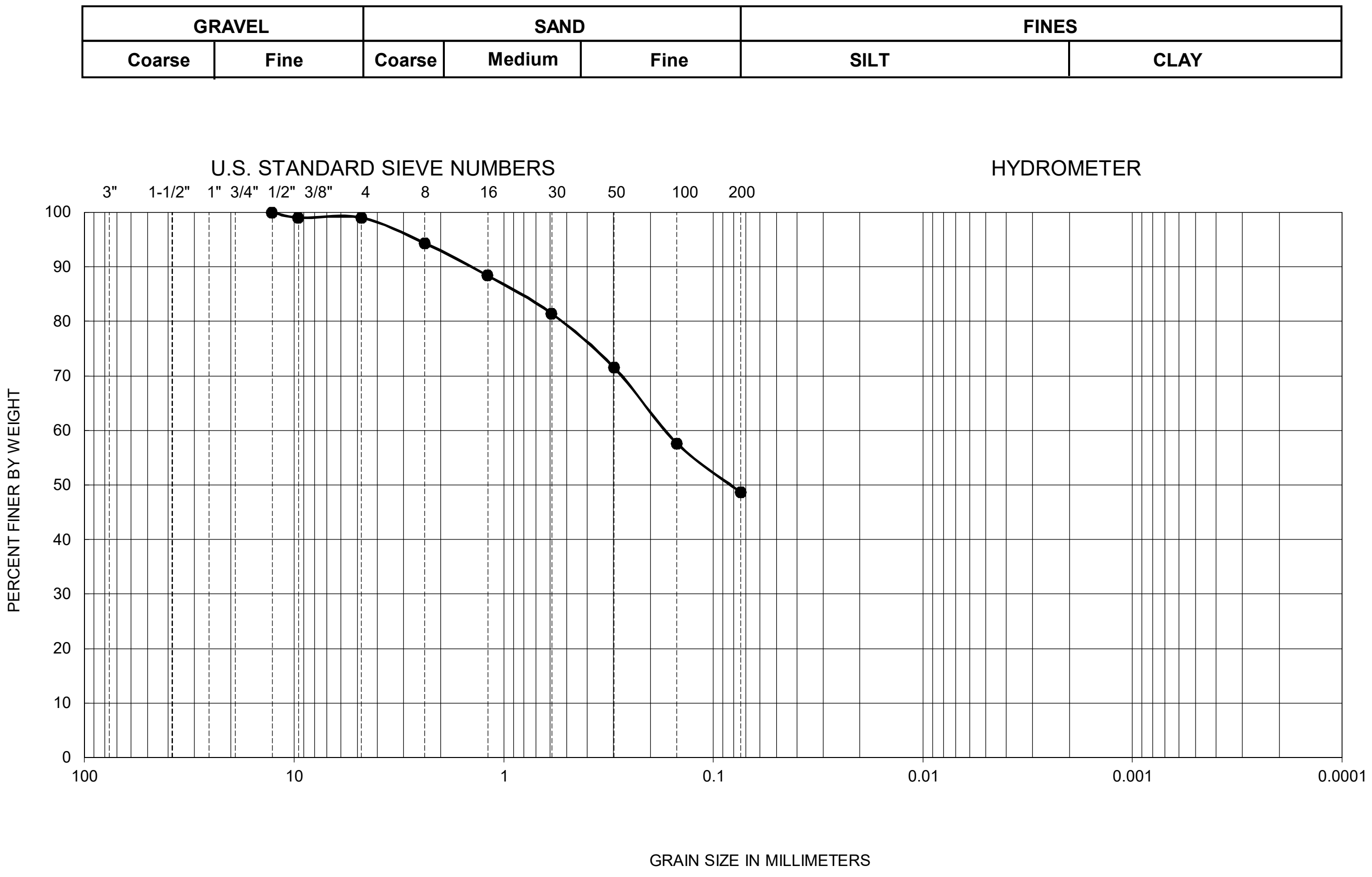
109107001 | 12/20



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
●	TP-14	0.0-2.4	--	--	--	--	--	--	--	--	49	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-5

GRADATION TEST RESULTS

ALPINE COMMUNITY PARK
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
TP-4	0.0-2.5	12.0	100.9	23.3	0.094	94	High
TP-6	0.0-2.7	13.0	98.8	28.1	0.097	97	High
TP-11	0.0-1.9	12.0	103.3	23.3	0.088	88	Medium
TP-13	1.0-3.3	11.0	106.8	31.9	0.105	105	High
TP-15	0.0-2.0	11.0	107.1	23.5	0.061	61	Medium

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

FIGURE B-6

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EXPANSION INDEX TEST RESULTS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

109107001 | 12/20

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
TP-1	0.0-2.0	7.4	1,600	10	0.001	40
TP-14	0.0-2.4	7.5	630	40	0.004	70

[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-7

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CORROSIVITY TEST RESULTS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

109107001 | 12/20

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
TP-3	0.0-2.0	Sandy CLAY (CL)	LESS THAN 5
TP-13	1.0-3.3	Sandy CLAY (CL)	LESS THAN 5

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-8

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R-VALUE TEST RESULTS

ALPINE COMMUNITY PARK
ALPINE, CALIFORNIA

109107001 | 12/20

APPENDIX C

Infiltration Test Data

Test Date: 11/19/2020

Infiltration Test No.: IT-1

Test Hole Dimensions, W x L (feet): 2 x 2

Excavation Depth (feet): 3.3

Test Area (ft^2 4

Test performed and recorded by: DP

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)
9:45 AM	2.75	9:55 AM	2.79	10	0.04	20.83	0.53	1.00
9:55 AM	2.79	10:05 AM	2.81	10	0.02	41.67	0.50	0.52
10:05 AM	2.81	10:15 AM	2.81	10	0.00	*DNI*	0.49	*DNI*
10:15 AM	2.81	10:25 AM	2.81	10	0.00	*DNI*	0.49	*DNI*
10:25 AM	2.81	10:35 AM	2.83	10	0.02	36.23	0.48	0.61
10:35 AM	2.83	10:45 AM	2.83	10	0.00	*DNI*	0.47	*DNI*
End of Test #1								
10:50 AM	2.79	11:00 AM	2.79	10	0.00	*DNI*	0.51	*DNI*
11:00 AM	2.79	11:10 AM	2.79	10	0.00	*DNI*	0.51	*DNI*
11:10 AM	2.79	11:20 AM	2.81	10	0.02	41.67	0.50	0.52
11:20 AM	2.81	11:30 AM	2.81	10	0.00	*DNI*	0.49	*DNI*
11:30 AM	2.81	11:40 AM	2.81	10	0.00	*DNI*	0.49	*DNI*
11:40 AM	2.81	11:50 AM	2.83	10	0.02	36.23	0.48	0.61
End of Test #2								
11:52 AM	2.79	12:02 PM	2.79	10	0.00	*DNI*	0.51	*DNI*
12:02 PM	2.79	12:12 PM	2.79	10	0.00	*DNI*	0.51	*DNI*
12:12 PM	2.79	12:22 PM	2.79	10	0.00	*DNI*	0.51	*DNI*
12:22 PM	2.79	12:32 PM	2.81	10	0.02	41.67	0.50	0.52
12:32 PM	2.81	12:42 PM	2.81	10	0.00	*DNI*	0.49	*DNI*
12:42 PM	2.81	12:52 PM	2.83	10	0.02	41.67	0.48	0.53
End of Test #3								

Notes:

DNI = did not infiltrate

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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

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

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion[1]

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

I_t = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

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

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

Test Date: 11/19/2020

Infiltration Test No.: IT-2

Test Hole Dimensions, W x L (feet): 2 x 2

Excavation Depth (feet): 3.8

Test Area (ft^2 4

Test performed and recorded by: DP

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)
9:30 AM	3.33	9:40 AM	3.35	10	0.02	40.00	0.46	0.57
9:40 AM	3.35	9:50 AM	3.38	10	0.02	40.00	0.44	0.59
9:50 AM	3.38	10:00 AM	3.38	10	0.00	*DNI*	0.43	*DNI*
10:00 AM	3.38	10:10 AM	3.38	10	0.00	*DNI*	0.43	*DNI*
10:10 AM	3.38	10:20 AM	3.38	10	0.00	*DNI*	0.43	*DNI*
10:20 AM	3.38	10:30 AM	3.38	10	0.00	*DNI*	0.43	*DNI*
End of Test #1								
10:32 AM	3.31	10:42 AM	3.31	10	0.00	*DNI*	0.49	*DNI*
10:42 AM	3.31	10:52 AM	3.33	10	0.02	40.00	0.48	0.56
10:52 AM	3.33	11:02 AM	3.33	10	0.00	*DNI*	0.47	*DNI*
11:02 AM	3.33	11:12 AM	3.35	10	0.02	40.00	0.46	0.57
11:12 AM	3.35	11:22 AM	3.35	10	0.00	*DNI*	0.45	*DNI*
11:22 AM	3.35	11:32 AM	3.35	10	0.00	*DNI*	0.45	*DNI*
End of Test #2								
11:35 AM	3.29	11:45 AM	3.29	10	0.00	*DNI*	0.51	*DNI*
11:45 AM	3.29	11:55 AM	3.29	10	0.00	*DNI*	0.51	*DNI*
11:55 AM	3.29	12:05 PM	3.29	10	0.00	*DNI*	0.51	*DNI*
12:05 PM	3.29	12:15 PM	3.29	10	0.00	*DNI*	0.51	*DNI*
12:15 PM	3.29	12:25 PM	3.29	10	0.00	*DNI*	0.51	*DNI*
12:25 PM	3.29	12:35 PM	3.29	10	0.00	*DNI*	0.51	*DNI*
End of Test #3								

Notes:

DNI = did not infiltrate

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.

Test Date: 11/18/2020

Infiltration Test No.: IT-3

Test Hole Dimensions, W x L (feet): 2 x 2

Excavation Depth (feet): 3.2

Test Area (ft^2 4

Test performed and recorded by: DP

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)
9:09 AM	2.67	9:19 AM	2.75	10	0.08	10.42	0.49	2.10
9:19 AM	2.75	9:29 AM	2.79	10	0.04	20.83	0.43	1.14
9:29 AM	2.79	9:39 AM	2.82	10	0.03	27.78	0.40	0.90
9:39 AM	2.82	9:49 AM	2.83	10	0.01	83.33	0.38	0.31
9:49 AM	2.83	9:59 AM	2.83	10	0.00	*DNI*	0.37	*DNI*
9:59 AM	2.83	10:09 AM	2.83	10	0.00	*DNI*	0.37	*DNI*
End of Test #1								
10:17 AM	2.65	10:27 AM	2.66	10	0.01	83.33	0.55	0.25
10:27 AM	2.66	10:37 AM	2.67	10	0.01	83.33	0.54	0.25
10:37 AM	2.67	10:47 AM	2.68	10	0.01	83.33	0.53	0.25
10:47 AM	2.68	10:57 AM	2.68	10	0.00	*DNI*	0.52	*DNI*
10:57 AM	2.68	11:07 AM	2.68	10	0.00	*DNI*	0.52	*DNI*
11:07 AM	2.68	11:17 AM	2.68	10	0.00	*DNI*	0.52	*DNI*
End of Test #2								
11:28 AM	2.68	11:38 AM	2.68	10	0.00	*DNI*	0.52	*DNI*
11:38 AM	2.68	11:48 AM	2.68	10	0.00	*DNI*	0.52	*DNI*
11:48 AM	2.68	11:58 AM	2.69	10	0.01	83.33	0.52	0.25
11:58 AM	2.69	12:08 PM	2.69	10	0.00	*DNI*	0.51	*DNI*
12:08 PM	2.69	12:18 PM	2.69	10	0.00	*DNI*	0.51	*DNI*
12:18 PM	2.69	12:28 PM	2.69	10	0.00	*DNI*	0.51	*DNI*
End of Test #3								

Notes:

DNI = did not infiltrate

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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

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

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion [1]

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

I_t = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

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

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

Test Date: 11/18/2020
Infiltration Test No.: IT-4
Test Hole Dimensions, W x L (feet): 2 x 2
Excavation Depth (feet): 4.2
Test Area (ft^2 4
Test performed and recorded by: DP

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)
1:40 PM	3.17	1:50 PM	3.19	10	0.02	41.67	1.02	0.31
1:50 PM	3.19	2:00 PM	3.21	10	0.02	41.67	1.00	0.32
2:00 PM	3.21	2:10 PM	3.21	10	0.00	*DNI*	0.99	*DNI*
2:10 PM	3.21	2:20 PM	3.23	10	0.02	41.67	0.98	0.32
2:20 PM	3.23	2:30 PM	3.23	10	0.00	*DNI*	0.97	*DNI*
2:30 PM	3.23	2:40 PM	3.23	10	0.00	*DNI*	0.97	*DNI*
End of Test #1								
2:41 PM	3.15	2:51 PM	3.17	10	0.02	41.67	1.04	0.31
2:51 PM	3.17	3:01 PM	3.17	10	0.00	*DNI*	1.03	*DNI*
3:01 PM	3.17	3:11 PM	3.19	10	0.02	41.67	1.02	0.31
3:11 PM	3.19	3:21 PM	3.19	10	0.00	*DNI*	1.01	*DNI*
3:21 PM	3.19	3:31 PM	3.19	10	0.00	*DNI*	1.01	*DNI*
3:31 PM	3.19	3:41 PM	3.19	10	0.00	*DNI*	1.01	*DNI*
End of Test #2								
3:46 PM	3.15	3:56 PM	3.15	10	0.00	*DNI*	1.05	*DNI*
3:56 PM	3.15	4:06 PM	3.15	10	0.00	*DNI*	1.05	*DNI*
4:06 PM	3.15	4:16 PM	3.15	10	0.00	*DNI*	1.05	*DNI*
4:16 PM	3.15	4:26 PM	3.15	10	0.00	*DNI*	1.05	*DNI*
4:26 PM	3.15	4:36 PM	3.17	10	0.02	41.67	1.04	0.31
4:36 PM	3.17	4:46 PM	3.17	10	0.00	*DNI*	1.03	*DNI*
End of Test #3								

Notes:

DNI = did not infiltrate
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.

Test Date: 11/18/2020

Infiltration Test No.: IT-5

Test Hole Dimensions, W x L (feet): 2 x 2

Excavation Depth (feet): 3.8

Test Area (ft^2 4

Test performed and recorded by: DP

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)
11:50 AM	3.33	12:00 PM	3.38	10	0.05	16.67	0.45	1.40
12:00 PM	3.38	12:10 PM	3.40	10	0.02	41.67	0.41	0.59
12:10 PM	3.40	12:20 PM	3.42	10	0.02	41.67	0.39	0.60
12:20 PM	3.42	12:30 PM	3.42	10	0.00	*DNI*	0.38	*DNI*
12:30 PM	3.42	12:40 PM	3.44	10	0.02	41.67	0.37	0.62
12:40 PM	3.44	12:50 PM	3.44	10	0.00	*DNI*	0.36	*DNI*
End of Test #1								
1:35 PM	3.42	1:45 PM	3.42	10	0.00	*DNI*	0.38	*DNI*
1:45 PM	3.42	1:55 PM	3.44	10	0.02	41.67	0.37	0.62
1:55 PM	3.44	2:05 PM	3.44	10	0.00	*DNI*	0.36	*DNI*
2:05 PM	3.44	2:15 PM	3.44	10	0.00	*DNI*	0.36	*DNI*
2:15 PM	3.44	2:25 PM	3.44	10	0.00	*DNI*	0.36	*DNI*
2:25 PM	3.44	2:35 PM	3.46	10	0.02	41.67	0.35	0.64
End of Test #2								
2:38 PM	3.42	2:48 PM	3.44	10	0.02	41.67	0.37	0.62
2:48 PM	3.44	2:58 PM	3.44	10	0.00	*DNI*	0.36	*DNI*
2:58 PM	3.44	3:08 PM	3.44	10	0.00	*DNI*	0.36	*DNI*
3:08 PM	3.44	3:18 PM	3.46	10	0.02	41.67	0.35	0.64
3:18 PM	3.46	3:28 PM	3.46	10	0.00	*DNI*	0.34	*DNI*
3:28 PM	3.46	3:38 PM	3.48	10	0.02	41.67	0.33	0.66
End of Test #3								

Notes:

DNI = did not infiltrate

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.

Test Date: 11/19/2020

Infiltration Test No.: IT-6

Test Hole Dimensions, W x L (feet): 2 x 2

Excavation Depth (feet): 3.6

Test Area (ft^2 4

Test performed and recorded by: DP

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)
2:13 PM	3.13	2:23 PM	3.15	10	0.02	40.00	0.46	0.57
2:23 PM	3.15	2:33 PM	3.15	10	0.00	*DNI*	0.45	*DNI*
2:33 PM	3.15	2:43 PM	3.17	10	0.02	40.00	0.44	0.58
2:43 PM	3.17	2:53 PM	3.17	10	0.00	*DNI*	0.43	*DNI*
2:53 PM	3.17	3:03 PM	3.19	10	0.02	40.00	0.42	0.60
3:03 PM	3.19	3:13 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
End of Test #1								
3:13 PM	3.19	3:23 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
3:23 PM	3.19	3:33 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
3:33 PM	3.19	3:43 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
3:43 PM	3.19	3:53 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
3:53 PM	3.19	4:03 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
4:03 PM	3.19	4:13 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
End of Test #2								
4:13 PM	3.19	4:23 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
4:23 PM	3.19	4:33 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
4:33 PM	3.19	4:43 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
4:43 PM	3.19	4:53 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
4:53 PM	3.19	5:03 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
5:03 PM	3.19	5:13 PM	3.19	10	0.00	*DNI*	0.41	*DNI*
End of Test #3								

Notes:

DNI = did not infiltrate

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.

Test Date: 11/19/2020

Infiltration Test No.: IT-7

Test Hole Dimensions, W x L (feet): 2 x 2

Excavation Depth (feet): 4.9

Test Area (ft^2: 4

Test performed and recorded by: DP

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)
2:07 PM	4.46	2:17 PM	4.48	10	0.02	40.00	0.43	0.59
2:17 PM	4.48	2:27 PM	4.50	10	0.02	40.00	0.41	0.61
2:27 PM	4.50	2:37 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
2:37 PM	4.50	2:47 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
2:47 PM	4.50	2:57 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
2:57 PM	4.50	3:07 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
End of Test #1								
3:07 PM	4.50	3:17 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
3:17 PM	4.50	3:27 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
3:27 PM	4.50	3:37 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
3:37 PM	4.50	3:47 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
3:47 PM	4.50	3:57 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
3:57 PM	4.50	4:07 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
End of Test #2								
4:07 PM	4.50	4:17 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
4:17 PM	4.50	4:27 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
4:27 PM	4.50	4:37 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
4:37 PM	4.50	4:47 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
4:47 PM	4.50	4:57 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
4:57 PM	4.50	5:07 PM	4.50	10	0.00	*DNI*	0.40	*DNI*
End of Test #3								

Notes:

DNI = did not infiltrate

t_1 = initial time when filling or refilling is completed

d_1 = initial depth to water in hole at t_1

t_2 = final time when incremental water level reading is taken

d_2 = final depth to water in hole at t_2

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

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

H_0 = Initial height of water column

in/hr = inches per hour

Percolation Rate to Infiltration Rate Conversion[1]

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

I_t = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

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

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

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