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GEOTECHNICAL INVESTIGATION

MOUNTAIN EMPIRE HIGH SCHOOL WHOLE SITE MODERNIZATION 3305 BUCKMAN SPRINGS ROAD PINE VALLEY, CALIFORNIA

PREPARED FOR

**MOUNTAIN EMPIRE UNIFIED SCHOOL DISTRICT
PINE VALLEY, CALIFORNIA**

**OCTOBER 15, 2021
PROJECT NO. G2820-42-01**

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Project No. G2820-42-01
October 15, 2021

Mountain Empire Unified School District
3305 Buckman Springs Road
Pine Valley, California 91962

Attention: Mr. Jacob Mann

Subject: GEOTECHNICAL INVESTIGATION
MOUNTAIN EMPIRE HIGH SCHOOL WHOLE SITE MODERNIZATION
3305 BUCKMAN SPRINGS ROAD
PINE VALLEY, CALIFORNIA

Dear Mr. Mann:

In accordance with your authorization, we herein submit our geotechnical investigation report for the subject project. The accompanying report presents the findings and conclusions pertinent to the project. Based on the results of our study, it is our opinion that the proposed improvements can be constructed as planned, provided the recommendations of this report are followed.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

Noel G. Borja Senior Staff Engineer	Rodney C. Mikesell GE 2533	Rupert S. Adams CEG 2561

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report contains the results of our geotechnical investigation for the proposed whole site modernization and improvements to the Mountain Empire High School at 3305 Buckman Springs Road in Pine Valley, California (see Topographic Vicinity Map, Figure 1). The purpose of our investigation was to evaluate subsurface soil and geologic conditions at the site and provide conclusions and recommendations pertaining to the geotechnical aspects of constructing the improvements as proposed.

The scope of our investigation included a site reconnaissance, drilling and logging six exploratory borings, performing three infiltration tests, and reviewing published and unpublished geologic literature and reports. The locations of the borings are shown on the Geologic Map, Figure 2. Logs of the borings and other details of the field investigation are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained from the exploratory borings to evaluate pertinent physical properties for engineering analyses. A discussion pertaining to the laboratory testing and results are presented in Appendix B.

The conclusions and recommendations presented herein are based on analysis of the data obtained during the field investigation, and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The Mountain Empire Jr./Sr. High School campus is located at 3305 Buckman Springs Road in Campo, California. The school campus is bounded by Buckman Springs Road to the west and open space to the north, south, and east. Interstate 8 is located approximately one-half mile east of the site.

We understand plans are to construct new exterior frontage to Buildings A and C, reconstruct the entry plaza, relocate the existing Book Room modular building, demolish and construct a new modular building in the area of Campo High #3, and new hardscape/landscape areas within the campus. Based on discussions with Davy Architecture, we understand the new exterior frontage to Buildings A and C will be supported on drilled piers embedded at least 10 feet below finish grade. The relocated Book Room building and the new modular buildings are expected to be lightly loaded structures supported by shallow footings.

The descriptions above are based on a review of the referenced plans. If development plans differ significantly from those described herein, Geocon Incorporated should be contacted for review and possible revisions to this report.

3. GEOLOGIC SETTING

The site is in the southern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges province extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by sedimentary rocks that thicken to the west and range in age from Upper Cretaceous through the Pleistocene. The sedimentary units are deposited on Jurassic to Cretaceous age igneous and metamorphic rocks. The coastal plain is characterized by a series of stair-stepped marine terraces (younger to the west). The coastal plain is dissected by faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone. A Regional Geologic Map and an explanation of the units [based on Kennedy & Tan (2008)], is presented on Figures 3 and 3A, respectively.

4. SOIL AND GEOLOGIC CONDITIONS

The site is underlain by alluvium overlying granitic rock. The geologic units are described below and shown on the Geologic Map, Figure 2.

4.1 Alluvium (Qal)

In the area of planned improvements, we encountered alluvium ranging from 5.5 feet to 10.5 feet. The alluvium consists of loose to medium dense, dry to damp, sandy silt with trace gravel. Laboratory tests indicate the alluvium possess a “very low” expansion potential (EI of 20 or less). Remedial grading should be performed to a depth of 1-foot below planned new footings that support the new modular buildings.

4.2 Granitic Rock (Kgr)

Cretaceous-aged Granitic Rock underlies the alluvium and is characterized as weak to moderately weak, completely weathered to weathered, rock. The granitic rock excavates as a silty, fine to coarse sand. We encountered hard drilling and refusal in boring B-2. The granitic rock may be encountered during pier drilling for the new Buildings A and C frontage. The granitic rock is suitable for the support of the planned improvements or additional fill.

5. GROUNDWATER

We did not encounter groundwater during our field investigation. Groundwater is not expected to significantly affect project development as presently proposed; however, it is not uncommon for groundwater or seepage conditions to develop where none previously existed. Proper surface drainage of irrigation and rainwater will be critical to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Ground Rupture

The USGS (2016) show that there are no mapped Quaternary faults crossing or trending toward the property. The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone (CGS, 2019). No active faults are known to exist at the site. The risk associated with ground rupture hazard is low.

6.2 Regional Faulting

Regional geologic information required to satisfy California Geological Survey (CGS) requirements for geology and seismology reports for California Public Schools is presented on Figures 3 through 5. Figure 3 shows the regional geologic structure for the site. Figure 4 is a regional fault map. Figure 5 is a seismicity map that depicts the historic seismicity with respect to the site.

The Elsinore Fault zone is located approximately 16.5 miles northeast of the site and is the closest known "active fault." The CGS considers a fault seismically active when evidence suggests seismic activity within roughly the last 11,700 years. Based upon a review of available geologic data and published reports, the site is not located within a State of California Alquist-Priolo Earthquake Fault Zone.

6.3 Local Faulting

Based on the results of our field investigation and our review of aerial photographs, published geologic maps, and previous geotechnical reports, it is our opinion that the site is not located on any active or potentially active fault trace as defined by the CGS.

6.4 Seismicity

Considerations important in seismic design include the frequency and duration of motion and the soil conditions underlying the site. Seismic design of structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency. The risk associated with strong ground motion due to earthquake at the site is high; however, the risk is no greater than that for the region.

6.5 Liquefaction and Seismically Induced Settlement

Due to the lack of near surface groundwater and formational bedrock at the site, the risk associated with seismically induced soil liquefaction hazard is low.

6.6 Landslides

We did observe evidence of landslide at the site during the geotechnical investigation. The risk associated with ground movement hazard due to landslide is low.

6.7 Subsidence

Based on the subsurface soil conditions encountered during grading, the risk associated with ground subsidence hazard is low.

6.8 Seiche and Tsunami

The site is not located within a tsunami inundation zone as defined by California Geological Survey (2009). Elevation at the site is approximately 3140 feet MSL. There are no lakes or reservoirs located near the site. The risk associated with inundation hazard due to tsunami or seiche is low.

6.9 Flooding

The site is designated as a Zone D – Area of Undetermined Flood Hazard (FEMA, 2019).

6.10 Expansive Soil

Based on the results of our laboratory testing, the on-site materials possess a "very low" expansion potential (EI of 20 or less).

6.11 Erosion

The site is not located adjacent to the Pacific Ocean coast or a free-flowing drainage where active erosion is occurring. We do not expect erosion to impact to site development. In addition, we expect the proposed development would not increase the potential for erosion if properly designed.

6.12 Naturally Occurring Asbestos

The geologic units and existing fills are not conducive for the presence of naturally occurring asbestos. Therefore, the risk associated with naturally occurring asbestos is considered negligible.

7. CONCLUSIONS

7.1 General

7.1.1 We did not encounter soil or geologic conditions during our exploration that would preclude constructing the proposed improvements, provided the recommendations presented herein are followed and implemented during design and construction. The project Geotechnical Engineer should provide supplemental recommendations if variable or undesirable conditions are observed during construction, or if the proposed construction will differ from that anticipated herein.

7.1.2 The site is underlain by alluvium overlying granitic rock. The alluvium extended to depths of 5.5 feet to 10.5 feet below existing grade at the boring locations. Removal and recompaction of the alluvium should be performed to a depth of 1-foot below the bottom of new footings that support the modular buildings. In surface improvement areas, the upper 12 inches of existing soil should be scarified, moisture conditioned and compacted.

7.1.3 With the exception of possible moderate to strong seismic shaking, we did not observe or know of significant geologic hazards to exist on the site that would adversely affect the proposed project.

7.1.4 Based on our research, no active, potentially active, or activity unknown faults are known to cross the site or are trending toward the site.

7.1.5 The risks associated with liquefaction, ground rupture, landslides, and flooding hazards are low.

7.1.6 We did not encounter groundwater during our subsurface exploration, and we do not expect it to be a constraint to project development. However, seepage may be encountered during construction, especially during the rainy seasons.

7.1.7 The proposed structures can be supported on a shallow foundation system founded in properly compacted fill. We understand building A and C frontage improvements will be supported on drilled piers.

7.1.8 Proper drainage should be maintained. Recommendations for site drainage are provided herein.

7.1.9 Based on the results of our infiltration tests, full or partial infiltration is feasible. A discussion of the infiltration testing and storm water management recommendations are provided in Appendix D.

7.1.10 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions; however, some variations in subsurface conditions between trench locations should be anticipated.

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the alluvium should be possible with moderate effort using conventional heavy-duty equipment. Moderately weathered granitic may require a very heavy effort to excavate.

7.2.2 The soil encountered in the field investigation are considered to be "non-expansive" (expansion index [EI] of 20 or less) as defined by 2019 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil will possess a "very low" expansion potential (EI of 20 or less) in accordance with ASTM D 4829. The following table presents soil classifications based on the expansion index.

EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2019 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

7.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0	SO_4<0.10	No Type Restriction	n/a	2,500
S1	0.10≤SO_4<0.20	II	0.50	4,000
S2	0.20≤SO_4≤2.00	V	0.45	4,500
S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

[1] Maximum water to cement ratio limits do not apply to lightweight concrete

7.2.4 We performed laboratory tests on selected soil samples to check the corrosion potential to subsurface metal structures. A site is considered corrosive if the chloride ion concentration is 500 parts per million (ppm) or greater, water-soluble sulfate concentration is 2,000 ppm (0.2%) or greater, or the pH is 5.5 or less according to Caltrans *Corrosion Guidelines* (Caltrans, 2015). The laboratory test results are presented in Appendix B.

7.2.5 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be needed if improvements susceptible to corrosion are planned.

7.3 Grading

7.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix E and the applicable agency's grading ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during fill placement.

7.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the project architect, DSA inspector of record, city inspector, grading and underground contractors, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

7.3.3 Site preparation should begin with the removal of deleterious material, debris, and vegetation. The depth of vegetation removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

7.3.4 Abandoned utilities should be removed and the resulting depressions and/or trenches backfilled with properly compacted soil as part of the remedial grading.

7.3.5 Within the area of the proposed modular buildings, existing soil should be removed to a depth of at least 1 foot below the bottom of proposed footings and replaced with properly compacted fill. The removals should extend 5 feet outside the building structure footprint, where possible. The actual extent of unsuitable soil removals should be determined in the field by the soil engineer and/or engineering geologist.

7.3.6 Within surface improvement areas (parking lot, hardscape, etc.) we recommend the upper 1-foot below existing grade be scarified, moisture conditioned, and compacted prior to constructing new improvements.

7.3.7 Prior to fill being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM Test Procedure D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

7.3.8 Imported fill, if necessary, should consist of the characteristics presented in the following table. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to "Low" (Expansion Index of 50 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

7.4 Site-Specific Ground Motion Hazard Analysis

7.4.1 We performed a site-specific ground motion hazard analyses in accordance with ASCE 7-16 Chapter 21 and Section 1613A of the 2019 CBC using the online applications developed by USGS.

7.4.1 Probabilistic Seismic Hazard Analysis

7.4.1.1 The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping.

7.4.1.2 We evaluated the mean spectral response accelerations having a 2 percent chance of exceedance in 50 years at 5 percent damping using the USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled as a Site Class "C" with a corresponding average shear wave velocity (V_S30) of 537 meters per second. The site class definition is based on Standard Penetration Test blow count data.

7.4.1.3 The web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

7.4.1.4 The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second (Z2.5) be defined. Additionally, the GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require that the depth to where the shear wave velocity reaches 1 kilometer per second (Z1.0) be defined. The values of Z2.5 and Z1.0 are internally calculated by the Uniform Hazard Tool.

7.4.1.5 The MCE uniform hazard response spectra was adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the USGS Risk-Targeted Ground Motion Calculator and following ASCE 7-16 Section 21.2.1.2 Method 2.

7.4.1.6 The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 6.

7.4.1.7 In accordance with ASCE 7-16, Supplement 1, Section 21.2.2, the largest spectral response acceleration of the probabilistic response spectrum is less than 1.2Fa, with Fa determined from Table 11.4.1 with Sa taken as 1.5; therefore, a deterministic analysis of the ground motion was not required.

7.4.2 Site-Specific Response Spectrum

7.4.2.1 The lesser of the probabilistic and deterministic MCE_R response spectrums is the Site-Specific MCE_R. Two thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the modified General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with Fa and Fv determined as specified in Section 21.3.

7.4.2.2 Graphical representations of the analyses are presented on Figures 7 and 8. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 7 and in tabular form on Figure 8.

7.4.3 Mapped Acceleration Parameters

7.4.3.1 The following table summarizes the mapped acceleration parameters obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16A Structural Design, Section 1613A Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second.

MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613A.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.899g	Figure 1613A.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.316g	Figure 1613A.2.1(2)
Site Coefficient, F_A	1.2	Table 1613A.2.3(1)
Site Coefficient, F_V	1.5	Table 1613A.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.078g	Section 1613A.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.474g	Section 1613A.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.719g	Section 1613A.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.316g	Section 1613A.2.4 (Eqn 16-39)
T_S	0.44 sec	ASCE 7-16 Chapter 11
Site Latitude	32.733649	--
Site Longitude	-116.492244	--

7.4.4 Site-Specific Seismic Design Criteria

7.4.4.1 Based the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of thesite-specific ground motion hazard analysis.

7.4.4.2 The parameter S_{DS} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods from 1 to 2 seconds, inclusive. The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} and S_{D1}. The site-specific design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

7.4.4.3 The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.149g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.447g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.766g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.298g

7.4.5 Site-Specific Peak Ground Acceleration

7.4.5.1 The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5.

7.4.5.2 The probabilistic geometric mean peak ground acceleration and the deterministic 84th percentile geometric mean peak ground acceleration were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake.

7.4.5.3 The deterministic MCE_G shall not be less than $0.5F_{PGA}$, where F_{PGA} is determined from ASCE 7-16 Table 11.8-1 with the value of PGA taken as 0.5g. The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G, provided the value is not less than 80 percent of the value of PGA_M as determined by ASCE 7-16 Equation 11.8.1.

ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE_G Peak Ground Acceleration, PGA_M	0.461g	Section 21.5

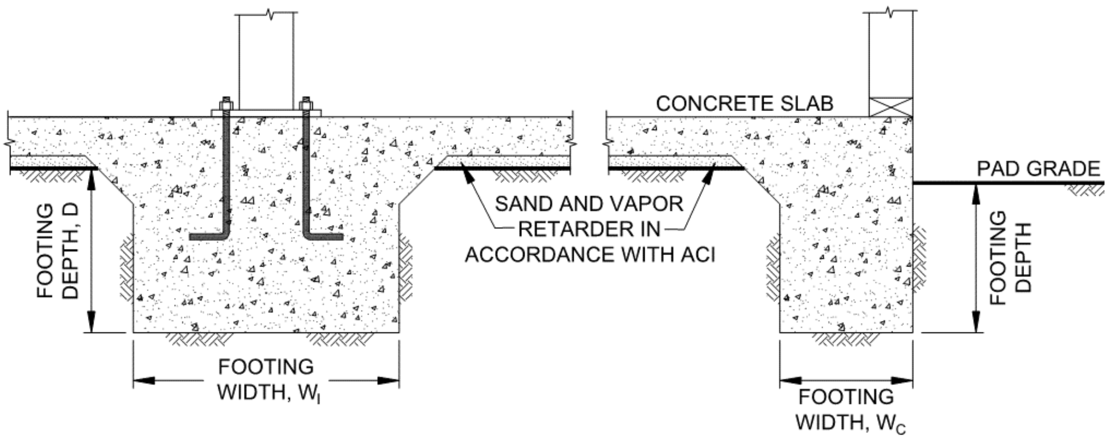
7.5 Shallow Foundations

7.5.1 The proposed modular buildings can be supported on a shallow foundation system founded in properly compacted fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. The following table provides a summary of the foundation design recommendations.

SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	12 Inches Below Lowest Adjacent Grade
Minimum Concrete Reinforcement	4 No. 4 steel Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity	1,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement*	½ Inch in 40 Feet
Footing Size Used for Settlement	6-Foot Square
Design Expansion Index	50 or less

7.5.2 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings.



Wall/Column Footing Dimension Detail

7.5.3 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

7.5.4 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected soil conditions are encountered.

7.5.5 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

7.6 Bearing Pressure Validation

7.6.1 We performed an analysis in the area of modular buildings P102 through P110 to evaluate if the existing soil has an allowable bearing capacity of at least 1,000 pounds per square foot (psf). We collected samples at borings B-3 through B-5 and subjected the samples to direct shear strength laboratory tests in accordance with ASTM D 3080. Based on the laboratory test results, as well as penetration resistance (blow counts) obtained during the field investigation, we opine that the soils at buildings P102 through P110 have an allowable bearing capacity that meets or exceeds the required 1,000 psf bearing pressure for the modular buildings. A summary of the bearing pressure calculations is presented in Appendix C.

7.7 Exterior Concrete Flatwork

7.7.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in the following table. The recommended concrete reinforcement would help reduce the potential for cracking.

MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Concrete Reinforcement* Options	Minimum Thickness
EI $\leq$ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	5 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

7.7.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557.

7.7.3 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

7.7.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

7.7.5 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stem wall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

7.7.6 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. Even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

7.8 Retaining Walls

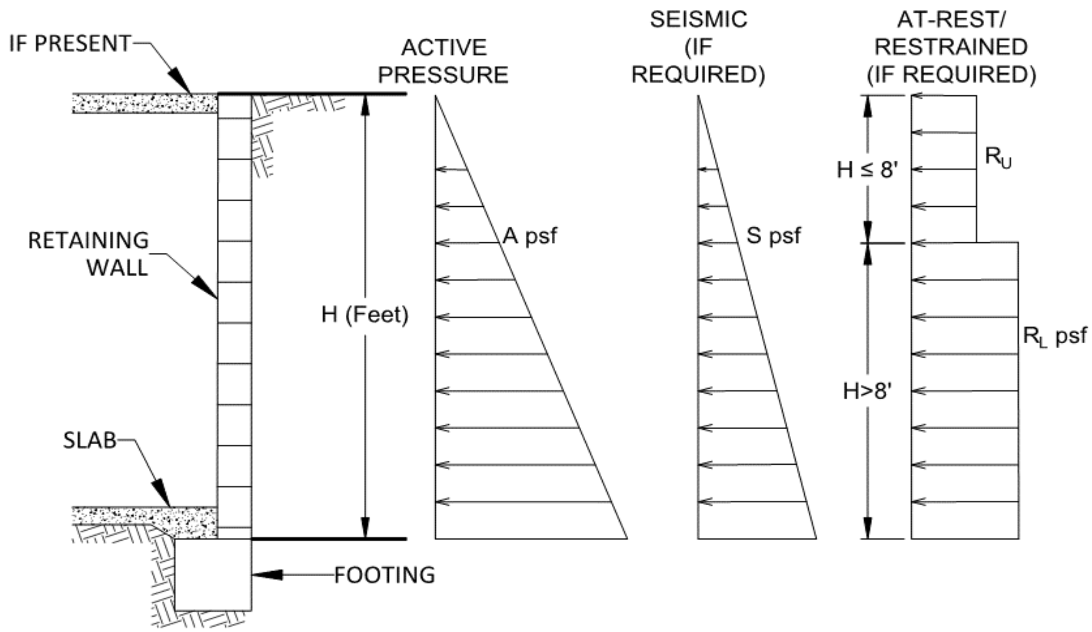
7.8.1 Retaining walls should be designed using the values presented in the following table. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	14H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI$\leq$50

H equals the height of the retaining portion of the wall

7.8.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



Retaining Wall Loading Diagram

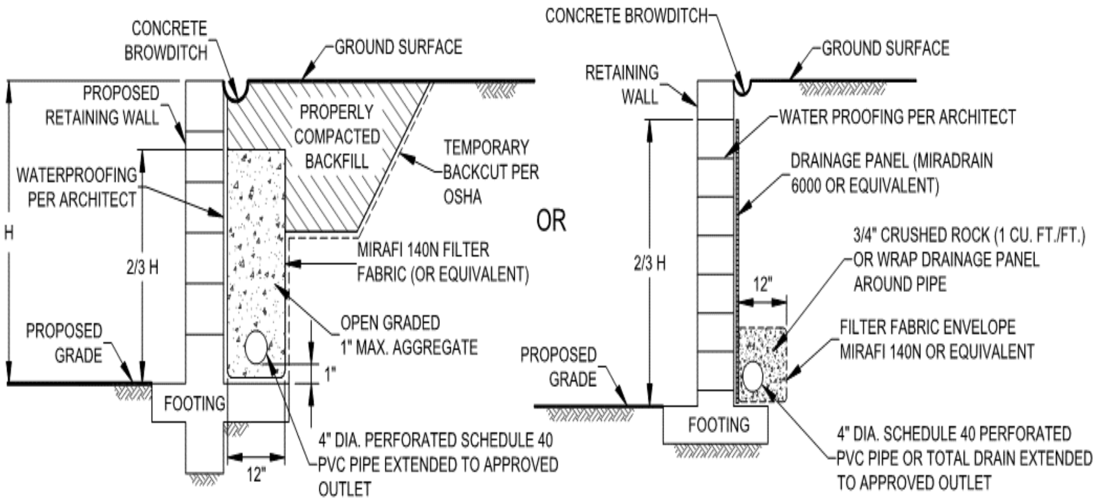
7.8.3 Where walls are restrained from movement at the top, an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

7.8.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2019 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

7.8.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

7.8.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load.

The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

7.8.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

7.8.8 In general, wall foundations should be designed in accordance with the following table. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure.

SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Concrete Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	1,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

7.8.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

7.8.10 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

7.8.11 Soil contemplated for use as retaining wall backfill, including imported soil, should be identified in the field prior to backfill. At that time, Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

7.9 Lateral Loading

7.9.1 The following table should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	300 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

7.9.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

7.10 Storm Water Management

7.10.1 If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and property located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water being detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff into the subsurface occurs, downstream improvements may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

7.10.2 We performed three infiltration tests at the locations shown on Figure 2. The tests were performed in 6-inch-diameter boreholes excavated by a limited access drill rig. The calculation sheets are presented in Appendix D.

7.10.3 We used the guidelines presented in the Riverside County Low Impact Development BMP Design Handbook. Based on this widely accepted guideline, the saturated hydraulic conductivity (Ksat) is equivalent to the infiltration rate. Therefore, the Ksat value determined from our testing is assumed to be the unfactored infiltration rate.

UNFACTORED, FIELD-SATURATED, INFILTRATION TEST RESULTS

Test No.	Depth (inches)	Geologic Unit	Field Infiltration Rate, I (in/hr)
A-1	47	Alluvium	0.199
A-2	47.5	Alluvium	0.729
A-3	45	Alluvium	0.701

7.11 Site Drainage and Moisture Protection

7.11.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

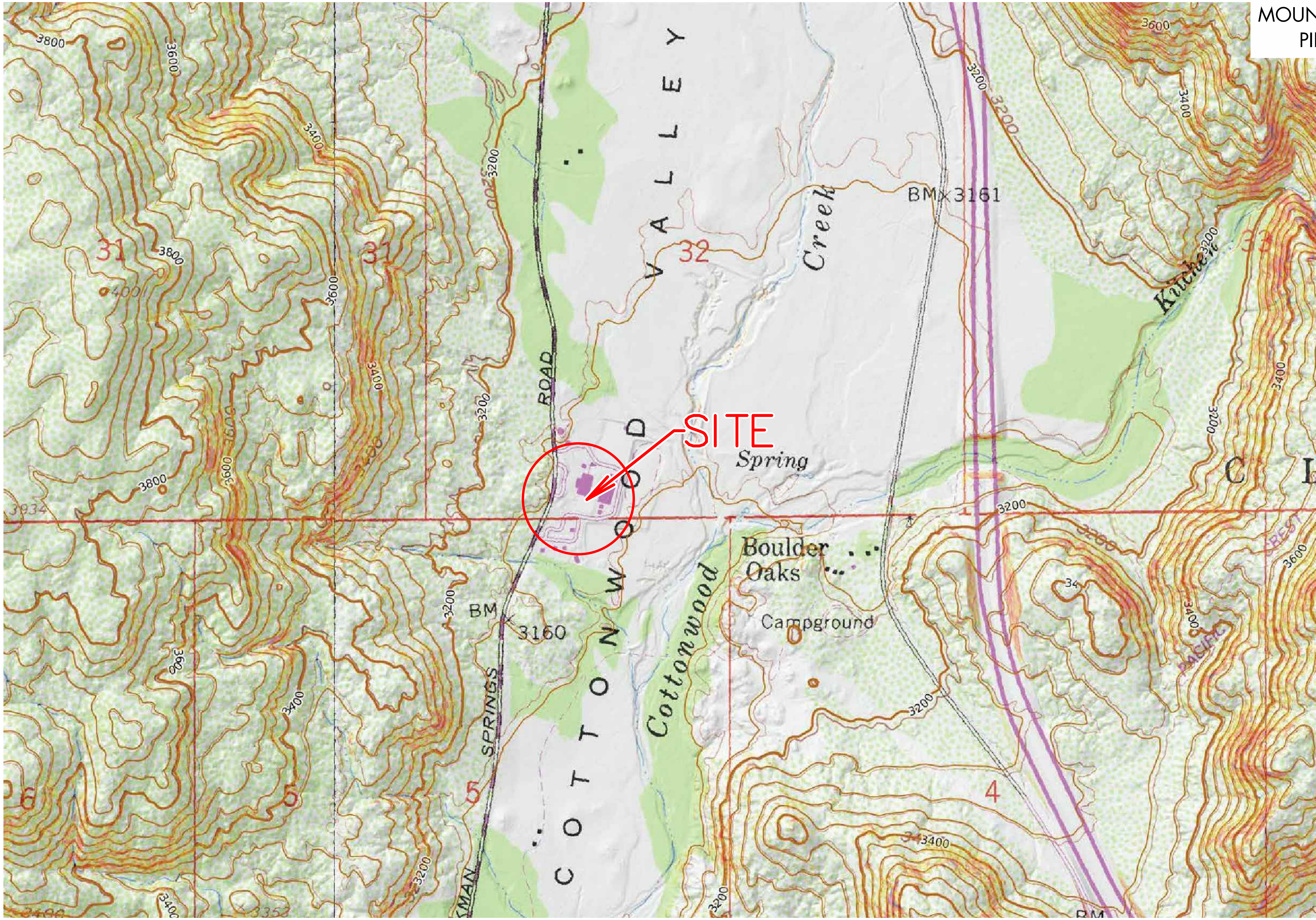
7.11.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

7.11.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

7.11.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

7.12 Geotechnical Engineer of Record

7.12.1 Geocon Incorporated should be retained as the geotechnical engineer during construction of site improvements such that the Geotechnical Engineer of Record is maintained. If a new geotechnical engineer is retained for compaction testing and observation during grading and construction of improvements, then the replacement geotechnical company will become the new Geotechnical Engineer of Record and will be responsible for providing geotechnical consultation and recommendations for the construction phase based on their field observations and testing during grading and improvements.



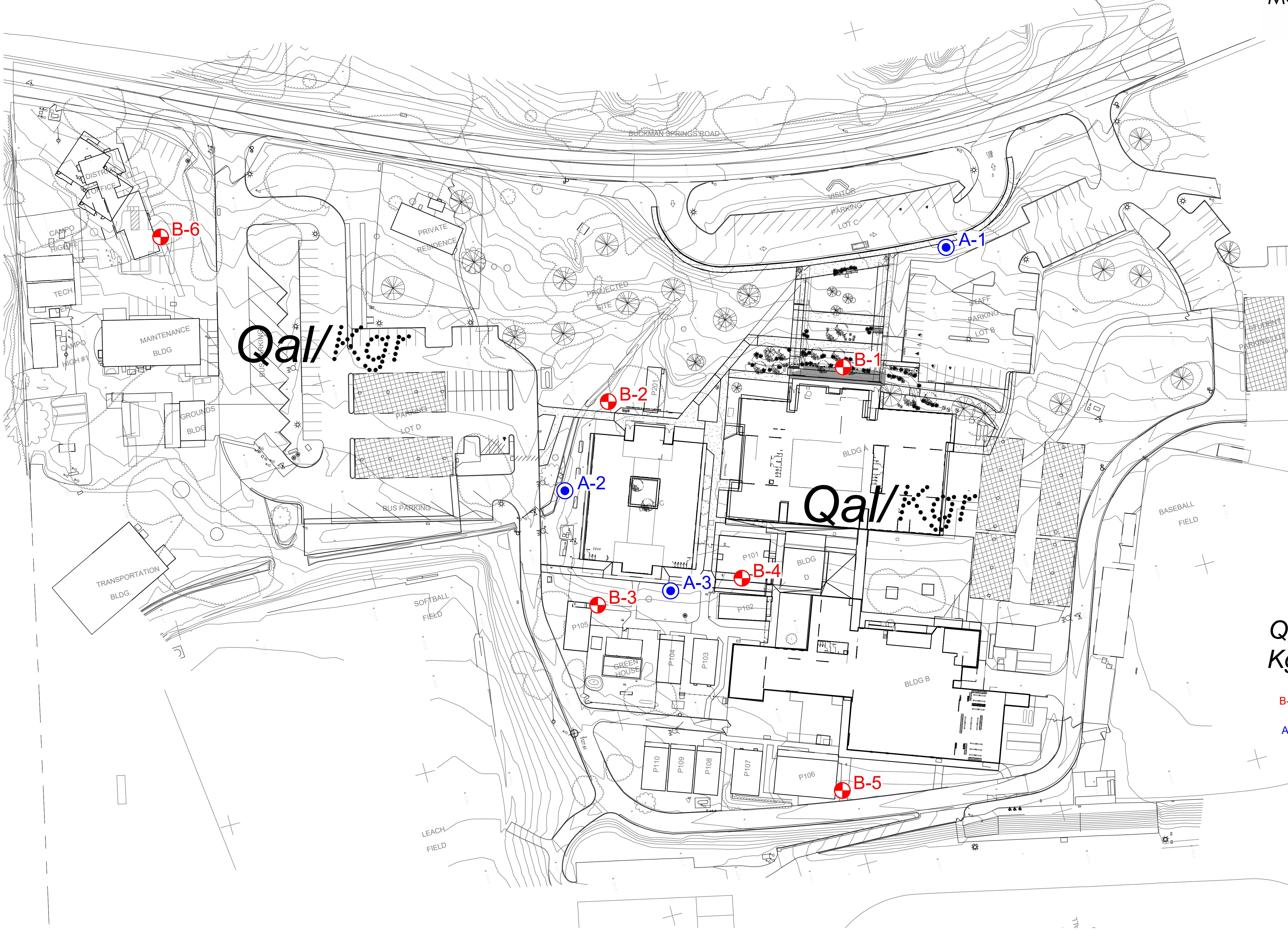
SITE LOCATION - LATITUDE: 32.733513, LONGITUDE: -116.492465

TOPOGRAPHIC VICINITY MAP

GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PROJECT NO. G2820 - 42 - 01
FIGURE 1
DATE 10 - 15 - 2021

MOUNTAIN EMPIRE HIGH SCHOOL
PINE VALLEY, CALIFORNIA



GEOCON LEGEND

Qal ....ALLUVIUM

Kgr ....GRANITIC ROCK (Dotted Where Buried)

B-6 ....APPROX. LOCATION OF BORING

A-3 ....APPROX. LOCATION OF INFILTRATION TEST

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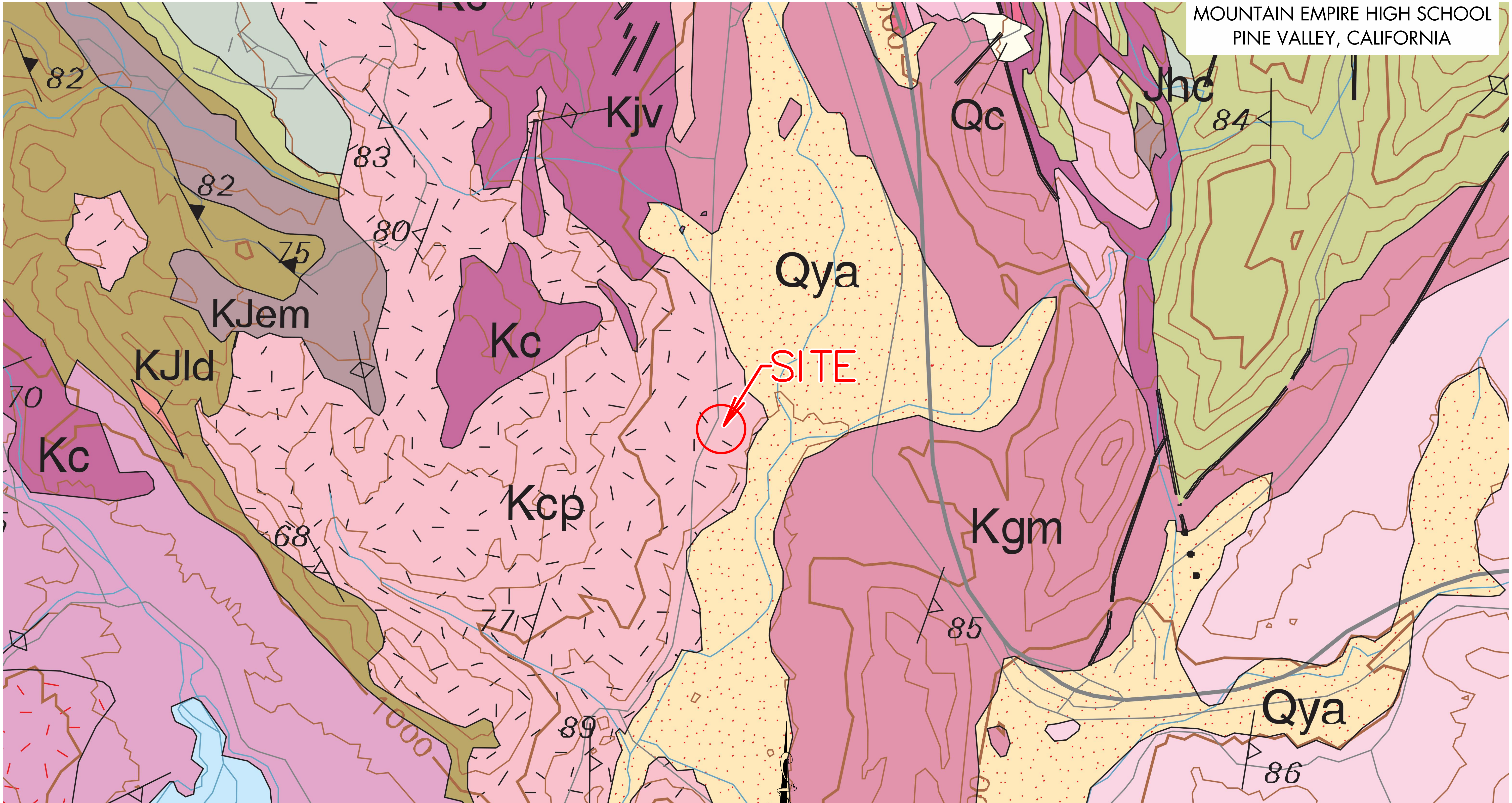
GEOLOGIC MAP

PROJECT NO. G2820 - 42 - 01
FIGURE 2
DATE 10 - 15 - 2021



SOURCE: Victoria R Todd, 2004, *Geologic Map of El Cajon 30'x60' Quadrangle, California*
U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

0' 2,000' 4,000'

SCALE 1"=2,000'(on 11x17)

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REGIONAL GEOLOGIC MAP

PROJECT NO. G2820 - 42 - 01
FIGURE 3
DATE 10 - 15 - 2021

Plotted:10/15/2021 2:41PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2820-42-01 (Mountain Empire HS)\SHEETS\G2820-42-01 RegionalGeologicMap&Explanation.dwg

MOUNTAIN EMPIRE HIGH SCHOOL
PINE VALLEY, CALIFORNIA

EXPLANATION

Contact—Solid where accuracy of location ranges from well located to approximately located; dashed where very poorly located or inferred. Color change without a contact shown is a scratch boundary

Fault—Solid where accurately located, dashed where approximately located, dotted where concealed. Arrow and number indicate direction and amount of dip.

Anticline—Solid where accurately located, dashed where approximately located, dotted where concealed.

Syncline—Solid where accurately located, dotted where concealed, dashed where inferred.

Strike and dip of bedding

Horizontal

Inclined

Strike and dip of foliation, primary igneous

Inclined

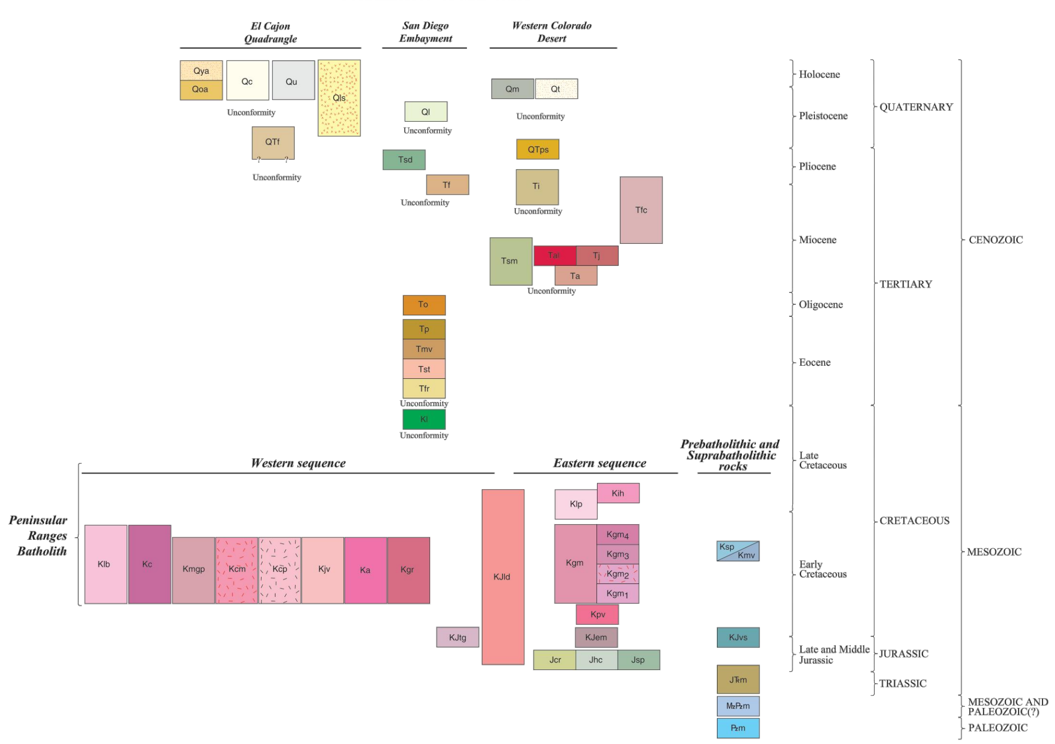
Vertical

Strike and dip of foliation, metamorphic

Inclined

Vertical

CORRELATION OF MAP UNITS



LIST OF MAP UNITS

El Cajon quadrangle

Qya — **Young alluvium (Holocene)**—Sand, silt, and gravel in modern streambeds and washes. Includes recent material accumulated on active alluvial fans

Qc — **Colluvium (Holocene and Pleistocene)**—Sand and gravel of slopewash, debris-flow, and talus deposits. Grades locally into younger alluvium (Qya) and older alluvium (Qoa)

Qu — **Alluvium and colluvium, undivided (Holocene and Pleistocene)**—Younger and older alluvium and colluvium not mapped separately

Qls — **Landslide deposits (Quaternary)**—Localized deposits of unconsolidated to consolidated earth and rock materials that moved downslope as landslides

Qoa — **Older alluvium (Holocene and Pleistocene)**—Sand, silt, and gravel; moderately dissected terraces in stream valleys. Well to poorly bedded, unconsolidated. In places, modern streams incise older alluvium to as much as 15 m. In some areas, older alluvium grades into younger alluvium

QTf — **Fanglomerate (Pleistocene and Tertiary?)**—Conglomeratic sand and gravel fanglomerate; locally derived. Scattered deposits, poorly sorted, weakly indurated. Unit also includes debris-flow deposits and small landslides. Some deposits mark courses of ancient drainages

San Diego Embayment

Ql — **Lindavista Formation (Pleistocene or Pliocene)**—Reddish-brown interbedded sandstone and conglomerate. Ferruginous cement, mainly hematite, gives formation characteristic color and resistance to erosion. Near-shore marine and nonmarine deposit. Molluscan fauna suggests early Pleistocene or late Pliocene age

Tsd — **San Diego Formation (Pliocene)**—Marine sandstone and subaerial conglomerate (Kennedy, 1975). Sandstone is typically fine to medium grained, yellowish-brown, poorly indurated, locally containing limy cement; interfingers with conglomerate. Conglomeratic part of unit consists of pebbles, cobbles, and boulders in coarse-grained sandstone matrix. Maximum thickness is 75 m

Tf — **Fanglomerate (Pliocene and Miocene)**—Boulder fanglomerate. Rests nonconformably on low-grade metavolcanic rocks; clasts locally derived. Matrix is medium- and coarse-grained, light-brown sandstone and bentonite

To — **Otay Formation (Oligocene)**—Massive sandstone and claystone. Light gray and light brown, moderately well sorted, poorly indurated. Claystone is waxy, composed almost exclusively of bentonite. Correlated with Miocene-Pliocene rocks in Baja California, Mexico (Kennedy and Tan, 1977)

Poway Group

Tp — **Pomerado Conglomerate (Eocene)**—Massive cobble conglomerate. Uppermost unit of Poway Group; maximum thickness is 55 m. Contains sparse beds and lenses of light-brown sandstone

Tmv — **Mission Valley Formation (Eocene)**—Marine sandstone; soft, friable, light-olive-gray, fine- to medium-grained. Mostly of quartz and potassium feldspar. Maximum thickness is 60. Has interbeds of claystone. Contains molluscan fauna in western and central exposures and land-mammal fauna in eastern exposures

Tst — **Stadium Conglomerate (Eocene)**—Massive cobble conglomerate having dark-yellowish-brown, coarse-grained sandstone matrix. Moderately well sorted; sandstone beds and lenses make up 50% of unit. Dominant clast type is rhyolitic to dacitic volcanic rocks. Nonmarine in east, nearshore marine and lagoonal in west. Contains late(?) Eocene fossils

Tfr — **Friars Formation (Eocene)**—Sandstone and claystone; nonmarine and lagoonal. Sandstone typically massive, yellowish-gray, medium grained, and poorly indurated. Conglomerate lenses are fluvial. Maximum thickness is 50 m. Contains marine and nonmarine fossils

Kl — **Lusardi Formation (Late Cretaceous)**—Cobble and boulder fanglomerate derived solely from crystalline rocks of the Peninsular Ranges batholith. Conglomerate contains thin lenses of medium-grained sandstone; clasts as much as 10 m in diameter. Restricted to northwest part of quadrangle. Maximum thickness about 125 m. Late Cretaceous because it is overlain by Late Cretaceous Point Loma Formation

Western Colorado Desert

Qm — **Mesa Conglomerate (Pleistocene)**—Poorly stratified to unstratified sand and gravel of extensively dissected alluvial-fan and terrace deposits. Massively bedded. Characterized by nearly horizontal beds paved with cobbles and small boulders having well developed desert varnish. Grain size decreases away from mountains

Qt — **Terrace deposits (Pleistocene)**—Sand, silt, and gravel of highly dissected alluvial terraces at elevations substantially higher than modern stream terraces; may be partly or wholly equivalent to Mesa Conglomerate (Woodard, 1967) (Qm). Terraces are capped by desert pavement. Poorly sorted, angular clasts derived from nearby mountains

QTps — **Palm Spring Formation (Pleistocene and Pliocene)**—Nonmarine sandstone, siltstone, and claystone commonly containing pebble and cobble interbeds and minor marine interbeds (Woodring, 1932; Woodard, 1974); grades laterally and downward into basal boulder to cobble fanglomerate assigned by Dibblee (1954) to his Canebrake conglomerate. Sandstone commonly arkosic; contains lesser fresh-water limestone. Fluvial and alluvial-fan deposits and minor lacustrine deposits. Represents ancient alluvial floodplain deposits marginal to the retreating Gulf of California

Ti — **Imperial Formation (Pliocene and Miocene)**—Massive, poorly bedded, gray, feldspathic arenite in lower part; rhythmically bedded, gray, silty mudstone and very fine quartz arenite in middle part; siltstone and sandstone interbedded with massive biostromal limestone and calcerous arenite in upper part (Woodring, 1932). Two lower parts are marine; part of upper part is nonmarine

Tfc — **Fish Creek Gypsum (Pliocene and Miocene)**—Extremely pure gypsum and anhydrite as much as 60 m thick. Rests unconformably on basement or conformably above transitional marine mudstone. Records earliest marine incursion into Salton Trough (Dean, 1988). Thin claystone interbeds contain marine microfossils; intertidal

Tsm — **Split Mountain Formation (Miocene)**—Nonmarine conglomerate and sandstone containing intercalated megabreccias composed of crystalline rocks of Peninsular Ranges batholith. Lower part of unit is dark-gray, boulder and cobble fanglomerate; middle part is interlensing quartz arenite and olive-green micaceous shale, which contains lateral interbeds of Fish Creek Gypsum; upper part is massive gray fanglomerate or megabreccia similar to lower part of formation

Tal — **Alverson Andesite (Miocene)**—Andesite; dark-purple-gray plagioclase-hornblende andesite interbedded with gray andesitic tuff. As much as 120 m thick. In northern Jacumba Mountains unit is flow rock, breccia, volcaniclastic rock, and air-fall deposits; flow rock is basaltic. Basalt yielded K-Ar whole-rock age of 16.9 ± 0.5 Ma (Hoggatt, 1979)

Tj — **Jacumba Volcanics (Miocene)**—Alkalic and tholeiitic basalt flows, breccia, and pyroclastic rocks; andesite and andesitic breccia. Parts of unit record remnants of five cinder cones and two hypersthene andesite plugs (Minch and Abbott, 1973). K-Ar ages average about 19 Ma

Ta — **Anza Formation (Miocene)**—Nonmarine arkosic sandstone and conglomerate; equivalent to basal conglomerate member of Split Mountain Formation (Tsm). Coarse conglomeratic sandstone. About 540 m thick at type locality, but only about 5 m thick in quadrangle; preserved only where covered by flows of Alverson Andesite

Peninsular Ranges Batholith

Western sequence

Klb — **Tonalite of Las Bancas (Early Cretaceous)**—Hypersthene-biotite tonalite, quartz diorite, granodiorite, and lesser diorite, quartz monzodiorite, and quartz norite. Medium grained, equigranular; weak foliation, but protomylonitic at margins of some plutons. Color index ranges from 22 to 32. Poikilitic, having potassium feldspar and biotite oikocrysts

Kc — **Cuyamaca Gabbro (Early Cretaceous)**—Troctolite, anorthositic gabbro, gabbronorite, hornblende gabbro; minor hornblende diorite and leucodiorite. Inner parts of bodies are hornblende-bearing troctolite; anorthositic gabbro ±amphibole ±orthopyroxene ±olivine; and amphibole-olivine gabbronorite. Margins and smaller bodies are mainly fine- to medium-grained hornblende gabbro ±orthopyroxene ±clinopyroxene ±biotite. Moderately to strongly foliated

Kmgp — **Monzogranite of Mother Grundy Peak (Early Cretaceous)**—Hornblende-biotite leucomonzogranite, leucogranodiorite, and tonalite. Medium to coarse grained, locally very coarse-grained; strongly foliated. Characterized by subhedral K-feldspar phenocrysts as much as 4 cm long and hornblende prisms from 0.5 to 1 cm in length. Basaltic and gabbroic dikes abundant near contacts with gabbro plutons

Kcm — **Corte Madera Monzogranite (Early Cretaceous)**—Biotite leucomonzogranite, leucogranodiorite, and syenogranite; trace hornblende. Medium to coarse grained; weakly to strongly foliated, locally protomylonitic. Forms lensoid plutons and fringing dikes. Color index varies from 1 to 11

Kcp — **Chiquito Peak Monzogranite (Early Cretaceous)**—Hornblende-biotite monzogranite and granodiorite and lesser tonalite, leucogranite, alaskite, and pegmatite. Color index 2-16. Forms lenticular plutons and narrow, sheet-like bodies. Medium grained; moderately to strongly foliated. Variable from one body to another; partly dependent on lithology of nearby units

Kjv — **Japatul Valley Tonalite (Early Cretaceous)**—Biotite-hornblende tonalite containing relict pyroxene; hornblende-biotite tonalite; and lesser hornblende-biotite granodiorite. Average color index about 22. Medium to coarse grained; equigranular but much is moderately to strongly foliated. Grades into tonalite of Alpine (Ka) and Chiquito Peak Monzogranite (Kcp)

Ka — **Tonalite of Alpine (Early Cretaceous)**—Biotite-hornblende tonalite, lesser quartz diorite, and scarce granodioritic tonalite. Medium to coarse grained; moderately to strongly foliated; mafic inclusions. Average color index 30. Unit is heterogeneous in outcrop and hand specimen

Kgr — **Granitoid rocks (Early Cretaceous)**—Undivided tonalite and granodiorite; most lithologically similar to tonalite of Alpine (Ka), Japatul Valley Tonalite (Kjv), and Corte Madera Monzogranite (Kcm). Includes lesser gabbro and metavolcanic rocks

KJtg — **Tonalite and gabbro (Early Cretaceous and Jurassic)**—Mixed tonalite and gabbro of specifically defined units, undifferentiated. Includes parts of Tonalite of Las Bancas (Klb), Cuyamaca Gabbro (Kc), Jurassic gneiss of Stephenson Peak (Jsp), tonalite of Granite Mountain (Kgm), and tonalite of La Posta (Klp)

KJld — **Leucocratic dikes (Late Cretaceous and Late Jurassic)**—Leucogranite, granophyre, alaskite, pegmatite, and aplite; found cutting plutonic units in quadrangle. Includes dikes of at least three ages

Eastern sequence

Kih — **Indian Hill granodiorite of Parrish and others (1986) (Late Cretaceous)**—Garnetiferous muscovite-biotite leucogranodiorite and leucomonzogranite. Fine- to medium-grained, weakly foliated. Color index less than 7. Extensive, large, fine-grained muscovitic leucocratic dikes in southern parts of unit

Klp — **Tonalite of La Posta (Early and Late Cretaceous)**—Hornblende-biotite trondhjemite in western part, and biotite trondhjemite and granodiorite in eastern part. Unit is leucocratic, homogeneous, largely undeformed, and inclusion-free, but locally, pluton margins are moderately to strongly foliated. Color index from 6 to 15

Kgm — **Tonalite of Granite Mountain (Early Cretaceous)**—Biotite-hornblende tonalite; hornblende-biotite tonalite, lesser granodiorite; and minor quartz diorite. Medium- to coarse-grained; weak to very strong foliation. Color index from 17 to 27. Divided into four subunits in Morena Reservoir 7.5' quadrangle:

Kgm_4 — **Tonalite of Granite Mountain, Unit 4**—Mafic biotite-hornblende tonalite having subidiomorphic texture, scattered poikilitic biotite grains, moderate to well developed foliation, and relatively high color index

Kgm_3 — **Tonalite of Granite Mountain, Unit 3**—Relatively leucocratic hornblende-biotite tonalite and granodiorite having moderate to faint magmatic foliation and large, oval biotite grains ± small acicular hornblende grains

Kgm_2 — **Tonalite of Granite Mountain, Unit 2**—Biotite-hornblende tonalite having idiomorphic texture, moderate to faint magmatic foliation, and lower color index than marginal phase (Kgm_1)

Kgm_1 — **Tonalite of Granite Mountain, Unit 1**—Marginal biotite-hornblende tonalite that is finer-grained, has well developed solid-state foliation (overprinting magmatic foliation), and has higher color index than average rock of interior parts of pluton

Kpv — **Monzogranite of Pine Valley (Early Cretaceous)**—Hornblende-biotite leucomonzogranite, leucogranodiorite, and minor biotite-hornblende tonalite. Medium- to coarse-grained, subporphyritic, moderately to strongly foliated. Color indexfrom 4 to 10. Voluminous leucogranite, alaskite, granophyre, and pegmatite-aplite dikes associated with body

KJem — **Quartz Diorite of East Mesa (Cretaceous and Jurassic)**—Fine- to medium-grained, gneissic biotite-hornblende tonalite and quartz diorite and fine-grained, locally porphyritic biotite-hornblende quartz diorite and tonalite. Texturally and compositionally heterogeneous. Strongly foliated to mylonitic. Some rocks contain hypersthene ±clinopyroxene

Jcr — **Granodiorite of Cuyamaca Reservoir (Late and Middle Jurassic)**—Biotite and hypersthene-biotite granodiorite and tonalite; also contains actinolitic amphibole. Fine to medium grained, strongly foliated, locally mylonite gneiss. Average color index is 25

Jhc — **Gneiss of Harper Creek (Late and Middle Jurassic)**—Gneissic to mylonitic biotite granodiorite and tonalite, and lesser monzogranite. Fine- to medium-grained; strongly foliated. Average color index is 22. Contains muscovite, cordierite, sillimanite, and garnet, and abundant, inclusions. Isoclinal folded in places

Jsp — **Migmatitic schist and gneiss of Stephenson Peak (Late and Middle Jurassic)**—Granodiorite orthogneiss, paragneiss; migmatitic pelitic schist, and lesser calcsilicate-bearing quartzite, marble, and amphibolite. Layers range from several centimeters to tens of meters

Prebatholithic and Suprabatholithic rocks

Ksp — **Santiago Peak Volcanics (Early Cretaceous)**—Dacitic and andesitic breccia, tuff, and flows, and lesser basalt and rhyolite. Unmetamorphosed to slightly metamorphosed

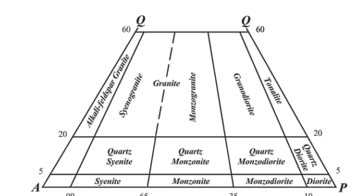
Kmv — **Metavolcanic rocks (Early Cretaceous)**—Amphibolite-facies tuff, tuff-breccia, and volcanic flow rock of andesitic, dacitic, and basaltic composition. Also includes rare feldspathic metaquartzite, pelitic schist, and granitoid-cobble metaconglomerate. Typically forms screens between and within plutons in the western part of the El Cajon quadrangle

KJvs — **Metavolcanic and metasedimentary rocks (Cretaceous and Jurassic?)**—Amphibolite, calcsilicate rocks, felsic tuff-breccia, biotite-rich schist, and quartzite

JTRm — **Metasedimentary and metavolcanic rocks (Jurassic and Triassic)**—Interlayered semi-pelitic, pelitic, and quartzitic schists; calcsilicate-bearing feldspathic metaquartzite; and minor small-pebble metaconglomerate. Includes layers of sandstone, quartz-pebble conglomerate, mudstone, and amphibolite. Interpreted to be metamorphosed submarine fan deposits and intercalated volcanic rocks; equivalent to the Julian Schist of Hudson (1922)

MzPzm — **Rocks of Jacumba Mountains (Mesozoic and Paleozoic?)**—Marble, schist, and metaquartzite. Metasedimentary and metavolcanic rocks forming screens within Jurassic granitoids and plutons of middle to Late Cretaceous tonalite. Interlayered with minor metachert and abundant hornblende schist. Metamorphosed sedimentary and volcanic rocks may be of oceanic affinity

Pzm — **Metasedimentary rocks (Paleozoic)**—Greenschist, marble, schist, metaquartzite, and metaconglomerate. Mainly occurs as metamorphic screens, but some rocks preserved well enough to contain fossils. Interpreted to be metamorphosed shelf-type sedimentary strata containing thick carbonate sequences



Classification of plutonic rock types (from IUGA, 1973, and Streckeisen, 1973). A, alkali feldspar; P, plagioclase feldspar; Q, quartz.

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FIGURE 3A

REGIONAL GEOLOGIC MAP EXPLANATION

DATE 10 - 15 - 2021

Plotted:10/15/2021 2:42PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2820-42-01 (Mountain Empire HS)\SHEETS\G2820-42-01 RegionalGeologicMap&Explanation.dwg

Fault Areas

- Class B
- historic
- late Quaternary
- latest Quaternary
- middle and late Quaternary

National Database

- Historic (< 150 years), well constrained location
- Historic (< 150 years), moderately constrained location
- Historic (< 150 years), inferred location
- Latest Quaternary (<15,000 years), well constrained location
- Latest Quaternary (<15,000 years), moderately constrained location
- Latest Quaternary (<15,000 years), inferred location
- Late Quaternary (< 130,000 years), well constrained location
- Late Quaternary (< 130,000 years), moderately contrained location
- Late Quaternary (< 130,000 years), inferred location
- Middle and late Quaternary (< 750,000 years), well constrained location
- Middle and late Quaternary (< 750,000 years), moderately constrained location
- Middle and late Quaternary (< 750,000 years), inferred location
- Undifferentiated Quaternary (< 1.6 million years), well constrained location
- Undifferentiated Quaternary (< 1.6 million years), moderately constrained location
- Undifferentiated Quaternary (< 1.6 million years), inferred location

National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp. | USGS |

0' 5 10

SCALE 1"=5 miles (on 11x17)

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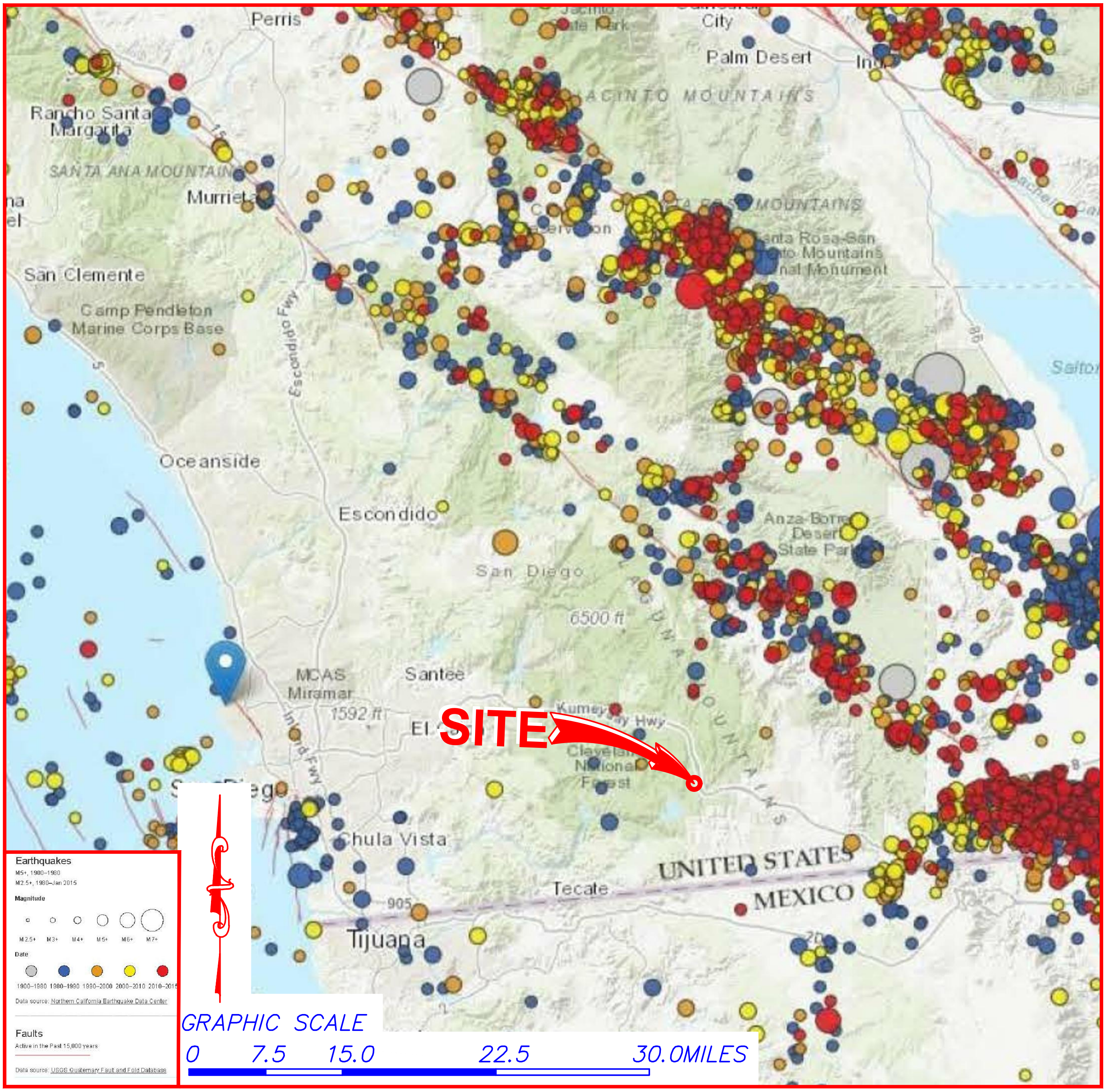
REGIONAL FAULT MAP

PROJECT NO. G2820 - 42 - 01
FIGURE 4
DATE 10 - 15 - 2021

Plotted:10/15/2021 2:40PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2820-42-01 (Mountain Empire HS)\SHEETS\G2820-42-01 RegionalFaultMap.dwg



SOURCE: Northern California EarthQuake Data Center

REGIONAL SEISMICITY MAP

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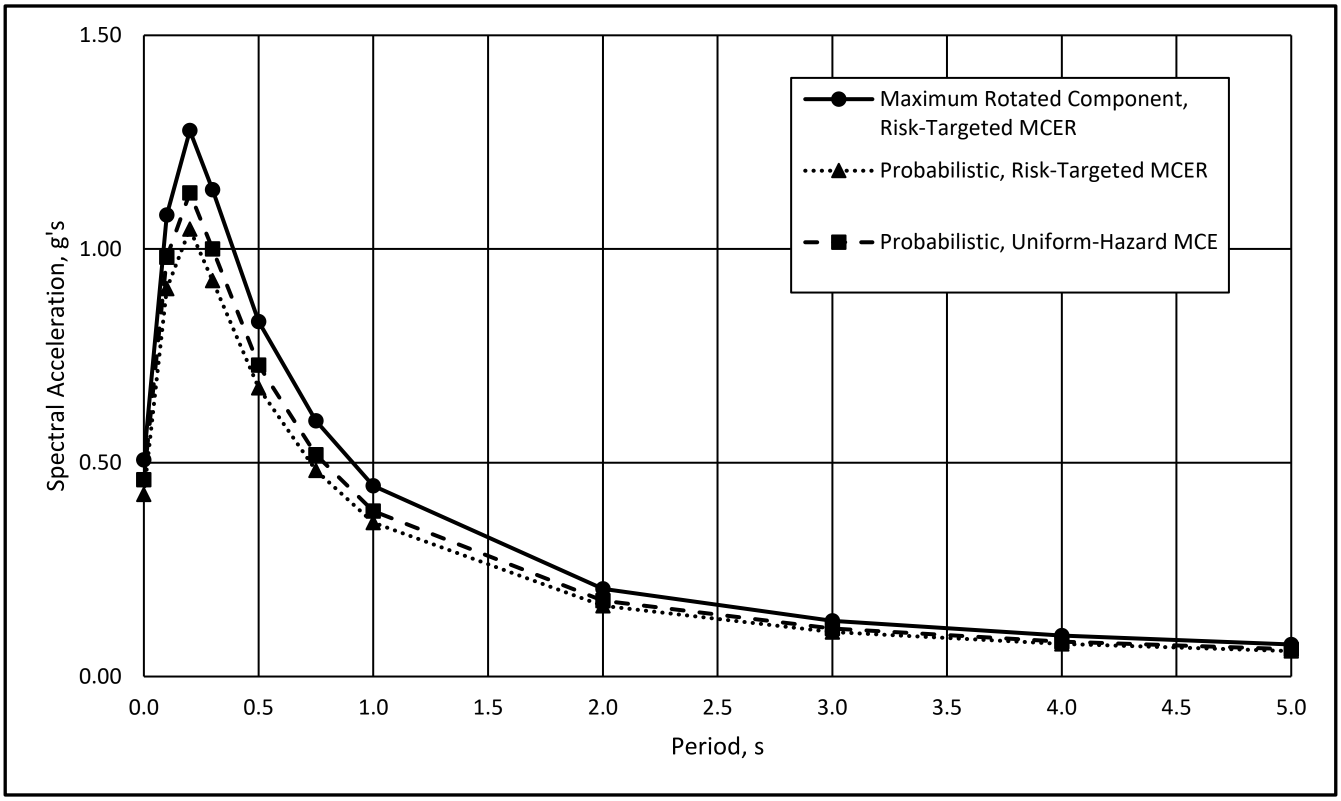
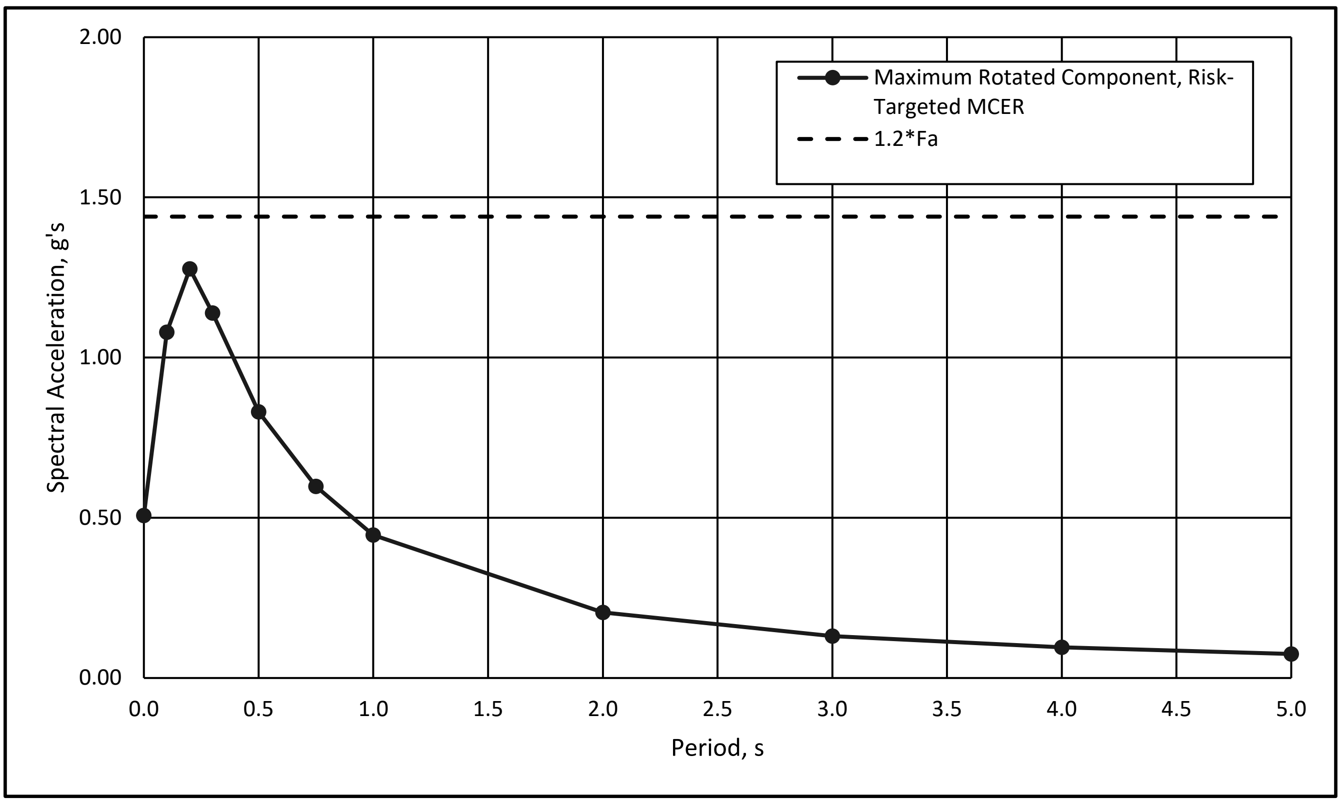
MOUNTAIN EMPIRE HIGH SCHOOL
PINE VALLEY, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 10 - 15 - 2021	PROJECT NO. G2820 - 42 - 01	FIG. 5





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DESIGN RESPONSE SPECTRUM

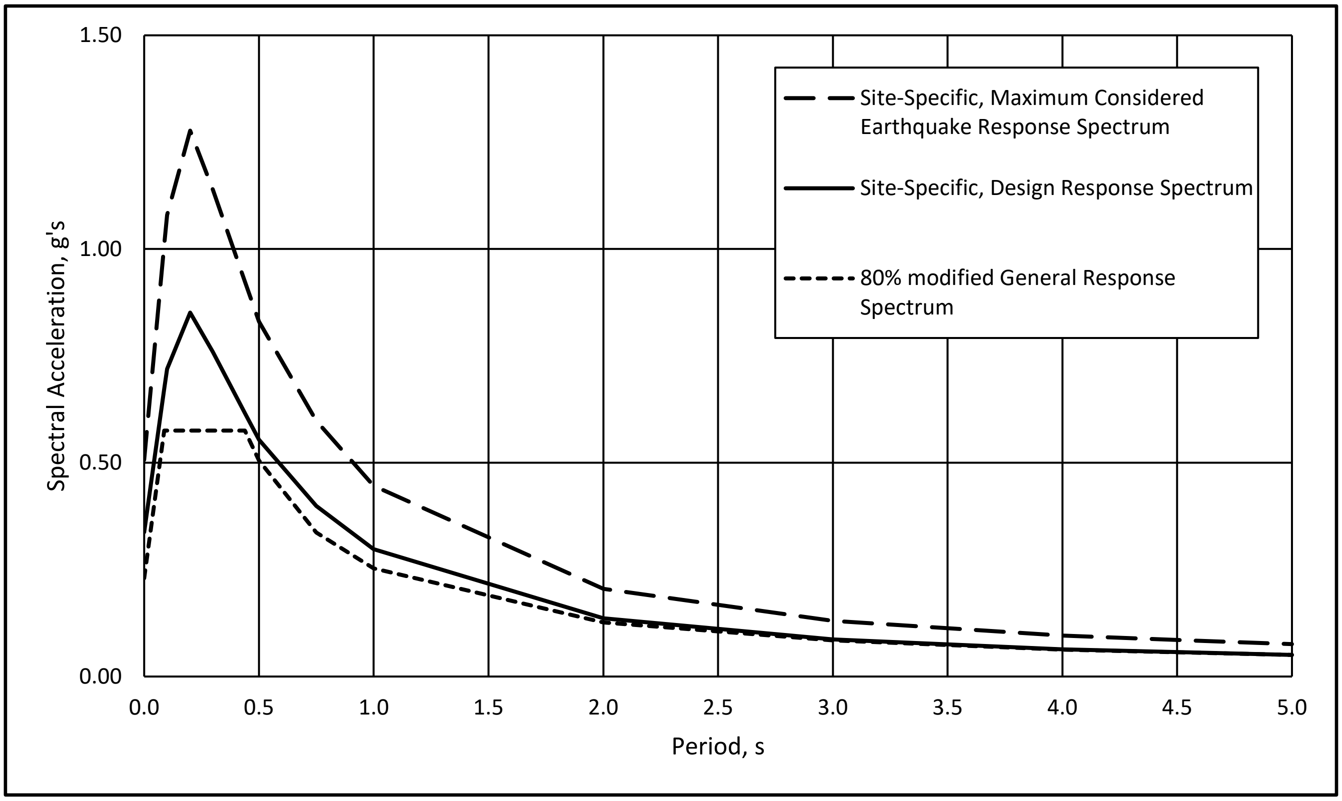
Checked by: NGB/LR

Project No.: G2820-42-01

MEHS-SITE MODERNIZATION
3305 BUCKMAN SPRINGS ROAD
PINE VALLEY, CA

Oct 21

Figure 6



GEOCON

DESIGN RESPONSE SPECTRUM

Checked by: NGB/LR

Project No.: G2820-42-01

MEHS-SITE MODERNIZATION
3305 BUCKMAN SPRINGS ROAD
PINE VALLEY, CA

Oct 21

Figure 7

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modifed General Response Spectrum	Site-Specific Maximum Considered Earthquake
0.00	0.461	0.426	0.932	1.190	0.507	--	0.338	0.230	0.507
0.09	--	--	--	--	--	--	0.673	0.575	1.010
0.10	0.981	0.907	0.925	1.190	1.079	--	0.720	0.575	1.079
0.20	1.132	1.047	0.929	1.220	1.277	--	0.851	0.575	1.277
0.30	1.000	0.926	0.926	1.230	1.139	--	0.759	0.575	1.139
0.44	--	--	--	--	--	--	0.616	0.575	0.924
0.50	0.728	0.675	0.932	1.230	0.831	--	0.554	0.506	0.831
0.75	0.519	0.482	0.932	1.240	0.598	--	0.399	0.337	0.598
1.00	0.387	0.360	0.929	1.240	0.447	--	0.298	0.253	0.447
2.00	0.177	0.165	0.922	1.240	0.205	--	0.137	0.126	0.205
3.00	0.112	0.104	0.918	1.250	0.130	--	0.087	0.084	0.130
4.00	0.082	0.076	0.908	1.260	0.096	--	0.064	0.063	0.096
4.58	--	--	--	--	--	--	0.056	0.056	0.084
5.00	0.064	0.060	0.906	1.260	0.075	--	0.051	0.051	0.076

SM_S = 1.149 g

SM_1 = 0.447 g

SD_S = 0.766 g

SD_1 = 0.298 g

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30}$ > 1,200 ft/s ($v_{s,30}$ > 365.76 m/s) and for periods from 1 to 5 s for sites with $v_{s,30} \leq$ 1,200 ft/=s ($v_{s,30} \leq$ 365.76 m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

"--" Indicates that spectral period was not used at that calculation step

GEOCON

DESIGN RESPONSE SPECTRUM

Checked by: NGB/LR

Project No.: G2820-42-01

MEHS-SITE MODERNIZATION
3305 BUCKMAN SPRINGS ROAD
PINE VALLEY, CA

Oct 21

Figure 8

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on September 20 and September 21, 2021. Our investigation consisted of drilling and logging six exploratory borings and performing three infiltration tests. The borings and infiltration tests were drilled to depths ranging from 4 feet to 15.5 feet using a limited access drill rig. The approximate locations of the borings and infiltration tests are shown on the Geologic Map, Figure 2.

The soil conditions encountered in the borings were visually examined, classified, and logged in general conformance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). Exploratory boring logs are presented on Figures A-1 through A-6. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

PROJECT NO. s

BORING B 1

ELEV. (MSL.) **3142'** DATE COMPLETED **09-20-2021**

EQUIPMENT **LIMITED ACCESS RAD (MOLE)** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIUM (Qal)** Loose, damp, dark brown, Silty, fine to medium SAND; trace gravel			
2	B1-1							
4								
	B1-2					14	113.7	6.7
6								
					GRANITIC ROCK (Kgr) Weak, completely weathered, mottled brown, tan brown, and black, GRANITIC ROCK; excavates as Silty, fine to medium SAND			
8								
10	B1-3				-Poor recovery; disturbed sample (slough)	50/2"		
12					-Becomes weathered to moderately weathered; harder drilling			
14								
	B1-4				-No recovery	50/1"		

BORING TERMINATED AT 15.5 FEET
Groundwater not encountered
Backfilled on 09-20-2021

Figure A-1,
Log of Boring B 1, Page 1 of 1

G2820-42-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. s

BORING B 2

ELEV. (MSL.) **3140'** DATE COMPLETED **09-20-2021**

EQUIPMENT **LIMITED ACCESS RAD (MOLE)** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIUM (Qal)** Loose, damp, brown to dark brown, Silty, fine to coarse SAND; few gravel and cobble			
2					-Becomes medium dense			
4	B2-1					22		
6	B2-2				**GRANITIC ROCK (Kgr)** Moderately weak, moderately weathered, mottled brown, dark brown, and black, GRANITIC ROCK; excavates as Silty, fine to coarse SAND; very hard drilling below 5.5 feet	50/5"		

BORING TERMINATED AT 6.5 FEET
Groundwater not encountered
Backfilled on 09-20-2021

Figure A-2,
Log of Boring B 2, Page 1 of 1

G2820-42-01.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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BORING B 3

ELEV. (MSL.) **3136'** DATE COMPLETED **09-20-2021**

EQUIPMENT **LIMITED ACCESS RAD (MOLE)** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIUM (Qal)** Loose, damp, dark brown, Silty, fine to coarse SAND; trace gravel			
2								
4	B3-1					15	114.2	8.0
					BORING TERMINATED AT 5 FEET Groundwater not encountered Backfilled on 09-20-2021			

Figure A-3,
Log of Boring B 3, Page 1 of 1

G2820-42-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. s

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 4 ELEV. (MSL.) 3141' DATE COMPLETED 09-20-2021 EQUIPMENT LIMITED ACCESS RAD (MOLE) BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B4-1			SM	**ALLUVIUM (Qal)** Loose to medium dense, dry to damp, dark brown, Silty, fine to medium SAND; trace gravel			
2								
					Becomes medium dense, damp			
4	B4-2					19	118.7	7.0
6								
8								
10	B4-3					27		
					GRANITIC ROCK (Kgr) Weak, completely weathered, mottled tan brown and black, GRANITIC ROCK; excavates as Silty, fine to coarse SAND			

BORING TERMINATED AT 11 FEET
Groundwater not encountered
Backfilled on 09-20-2021

Figure A-4,
Log of Boring B 4, Page 1 of 1

G2820-42-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. s

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 5 ELEV. (MSL.) 3134' DATE COMPLETED 09-20-2021 EQUIPMENT LIMITED ACCESS RAD (MOLE) BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					2" **ASPHALT** Over **SUBGRADE**			
				SM	**ALLUVIUM (Qal)** Medium dense, damp, dark brown, Silty, fine to coarse SAND; trace gravel			
2								
4	B5-1					26	118.0	8.4
					BORING TERMINATED AT 5 FEET Groundwater not encountered Backfilled on 09-20-2021			

Figure A-5,
Log of Boring B 5, Page 1 of 1

G2820-42-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. s

BORING B 6

ELEV. (MSL.) **3141'** DATE COMPLETED **09-20-2021**

EQUIPMENT **LIMITED ACCESS RAD (MOLE)** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B6-1			SM	**ALLUVIUM (Qal)** Loose to medium dense, damp, light brown, Silty, fine to medium SAND			
2								
4	B6-2				-Rock encountered at ~3.5 feet	50/5"	119.3	1.9
6					**GRANITIC ROCK (Kgr)** Weak, weathered, light brown, GRANITIC ROCK; excavates as Silty, fine to medium SAND; hard drilling below 6 feet			
8	B6-3					50/3"		

BORING TERMINATED AT 8.5 FEET
Groundwater not encountered
Backfilled on 09-20-2021

Figure A-6,
Log of Boring B 6, Page 1 of 1

G2820-42-01.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR ... SEEPAGE

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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected samples for in-place density and moisture content, compaction characteristics, gradation, direct shear, and expansion characteristics, water-soluble sulfate content, and chloride content. The results of our laboratory tests are presented on the following tables and figures.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS (ASTM D 1557)

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B4-1	Dark gray, Silty, fine to coarse SAND; trace gravel	134.5	8.1

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS (ASTM D 4829)

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2019 CBC Classification
	Before Test	After Test			
B4-1	7.7	14.0	119.0	1	Very Low
B6-1	8.0	13.8	118.6	2	Very Low

TABLE B-III
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Sulfate Exposure
B1-1	0.005	S0
B4-1	0.010	S0
B6-1	0.001	S0

TABLE B-IV
SUMMARY OF LABORATORY CHLORIDE ION CONTENT TEST RESULTS AASHTO TEST NO. T 291

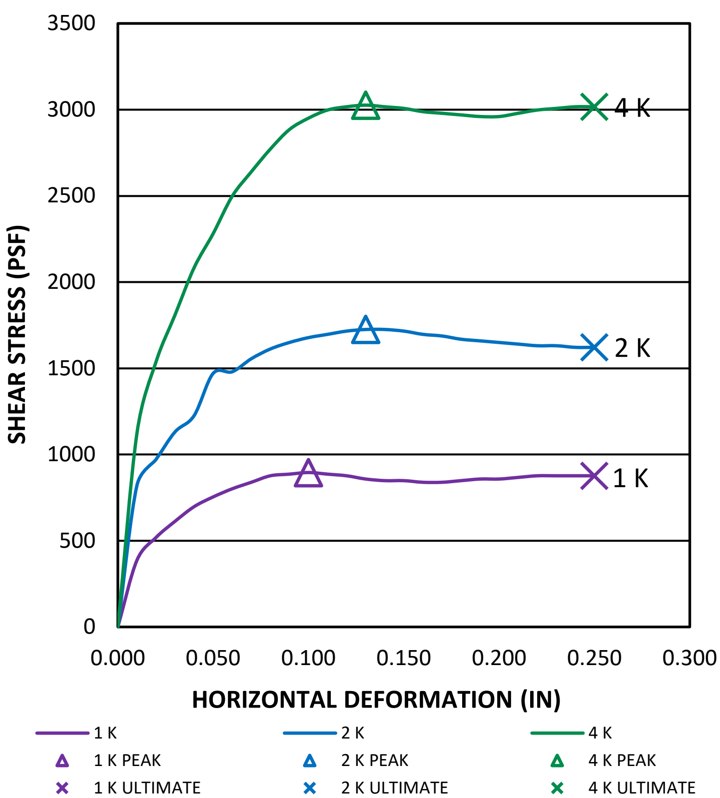
Sample No.	Chloride Ion Content ppm (%)
B1-1	80 (0.008)
B4-1	151 (0.015)
B6-1	70 (0.007)

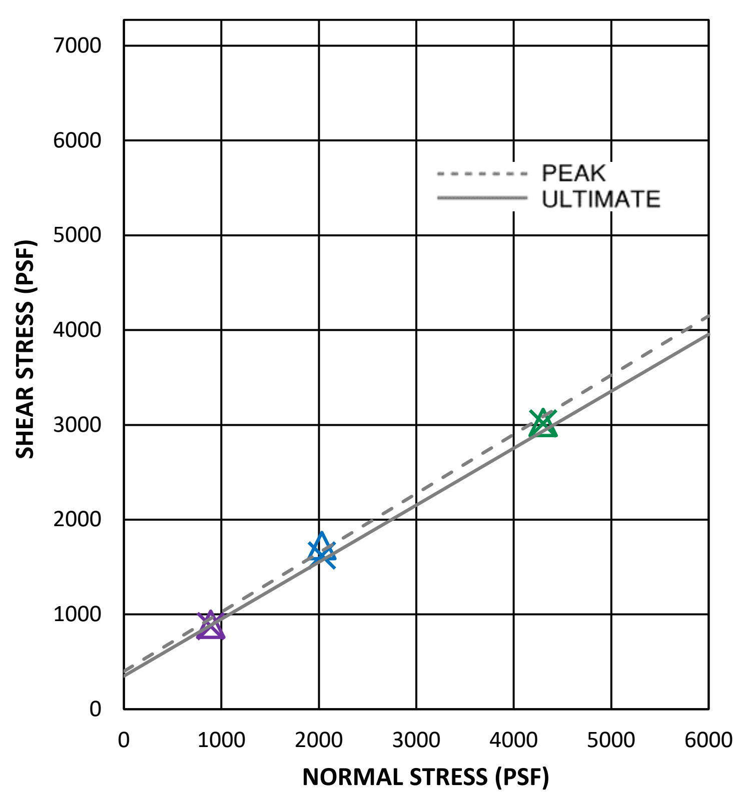
SAMPLE NO.:	B1-2	GEOLOGIC UNIT:	Qal
SAMPLE DEPTH (FT):	4'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	**1 K**	**2 K**	**4 K**	**AVERAGE**
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	6.5	7.0	6.5	6.7
DRY DENSITY (PCF):	110.8	115.1	115.2	113.7

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	**1 K**	**2 K**	**4 K**	**AVERAGE**
WATER CONTENT (%):	18.1	16.0	16.6	16.9
PEAK SHEAR STRESS (PSF):	896	1725	3026	--
ULT.-E.O.T. SHEAR STRESS (PSF):	877	1622	3017	--

RESULTS		
PEAK	COHESION, C (PSF)	400
	FRICTION ANGLE (DEGREES)	32
ULTIMATE	COHESION, C (PSF)	350
	FRICTION ANGLE (DEGREES)	31





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MEHS-SITE MODERNIZATION

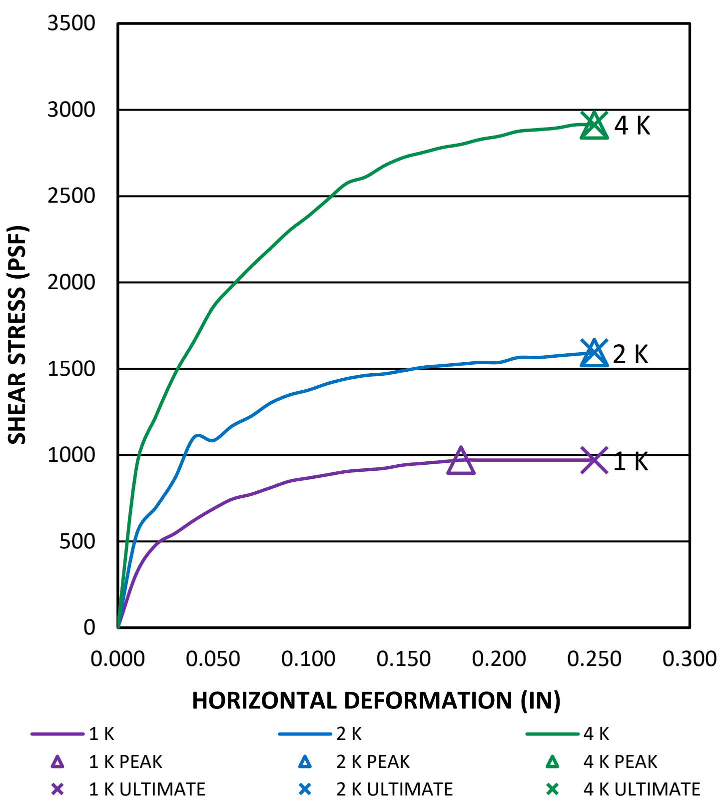
PROJECT NO.: G2820-42-01

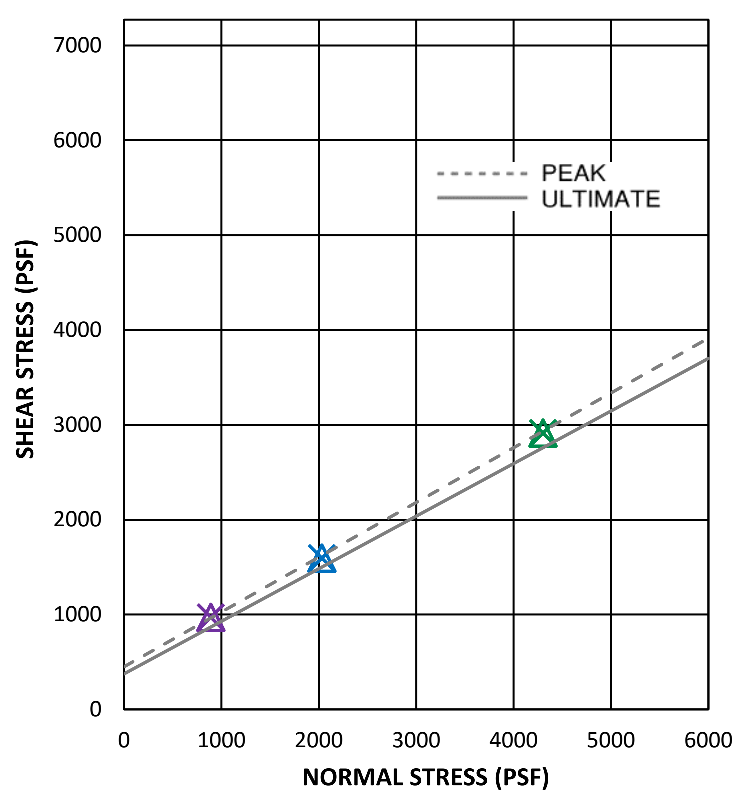
SAMPLE NO.:	B3-1	GEOLOGIC UNIT:	Qal
SAMPLE DEPTH (FT):	4'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	7.9	8.2	8.0	8.0
DRY DENSITY (PCF):	116.3	112.8	113.3	114.2

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	15.5	16.9	16.5	16.3
PEAK SHEAR STRESS (PSF):	971	1593	2913	--
ULT.-E.O.T. SHEAR STRESS (PSF):	971	1593	2913	--

RESULTS		
PEAK	COHESION, C (PSF)	450
	FRICTION ANGLE (DEGREES)	30
ULTIMATE	COHESION, C (PSF)	375
	FRICTION ANGLE (DEGREES)	29





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DIRECT SHEAR - ASTM D 3080

MEHS-SITE MODERNIZATION

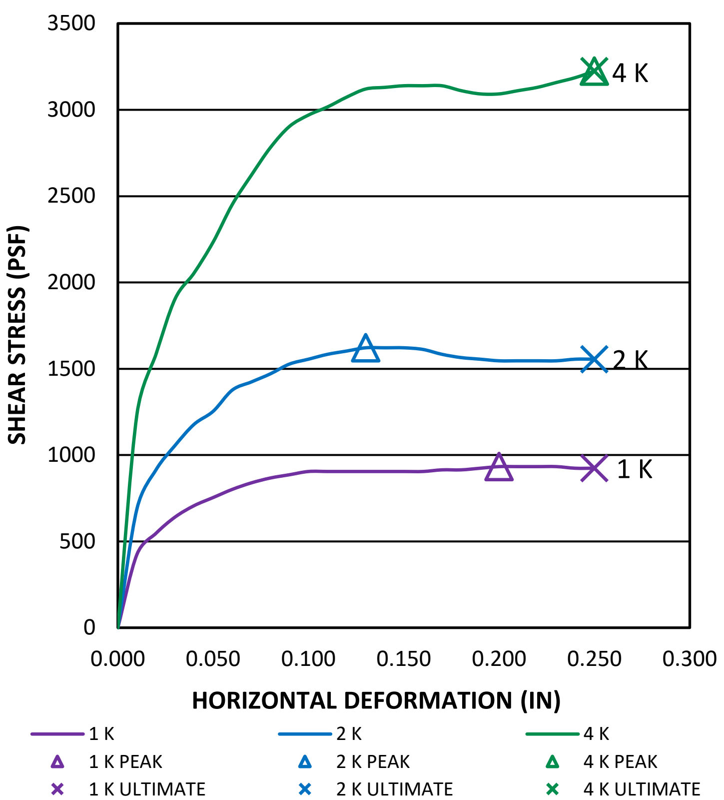
PROJECT NO.: G2820-42-01

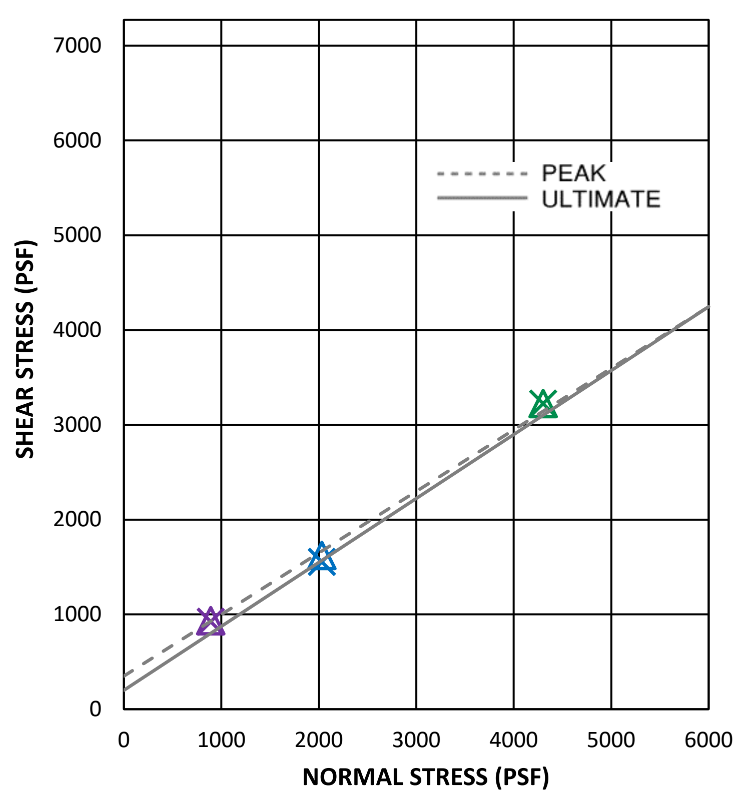
SAMPLE NO.:	B4-2	GEOLOGIC UNIT:	Qal
SAMPLE DEPTH (FT):	4'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	7.1	6.9	6.9	7.0
DRY DENSITY (PCF):	116.8	119.4	120.0	118.7

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	15.5	15.4	14.1	15.0
PEAK SHEAR STRESS (PSF):	933	1622	3224	--
ULT.-E.O.T. SHEAR STRESS (PSF):	924	1556	3224	--

RESULTS		
PEAK	COHESION, C (PSF)	350
	FRICTION ANGLE (DEGREES)	33
ULTIMATE	COHESION, C (PSF)	200
	FRICTION ANGLE (DEGREES)	34





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DIRECT SHEAR - ASTM D 3080

MEHS-SITE MODERNIZATION

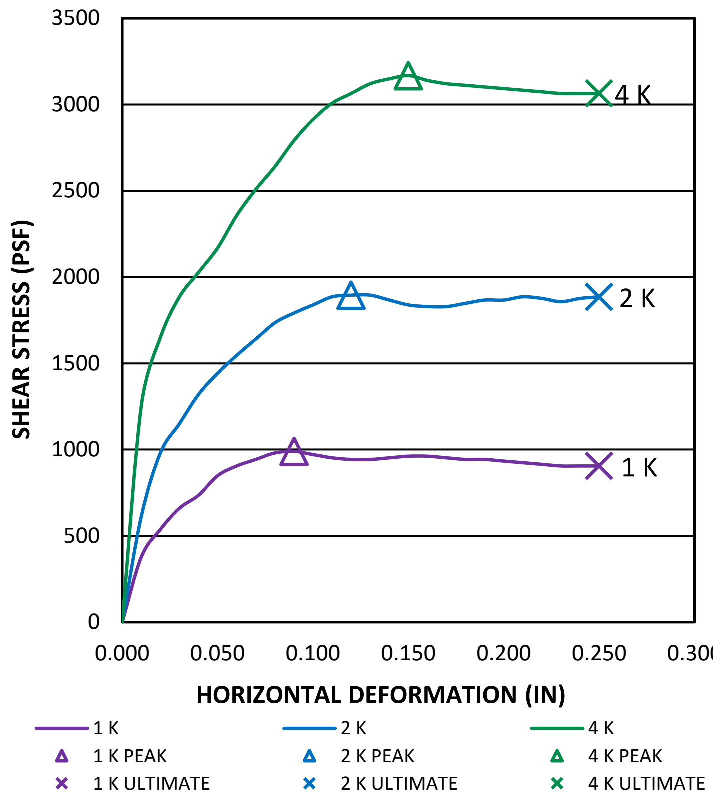
PROJECT NO.: G2820-42-01

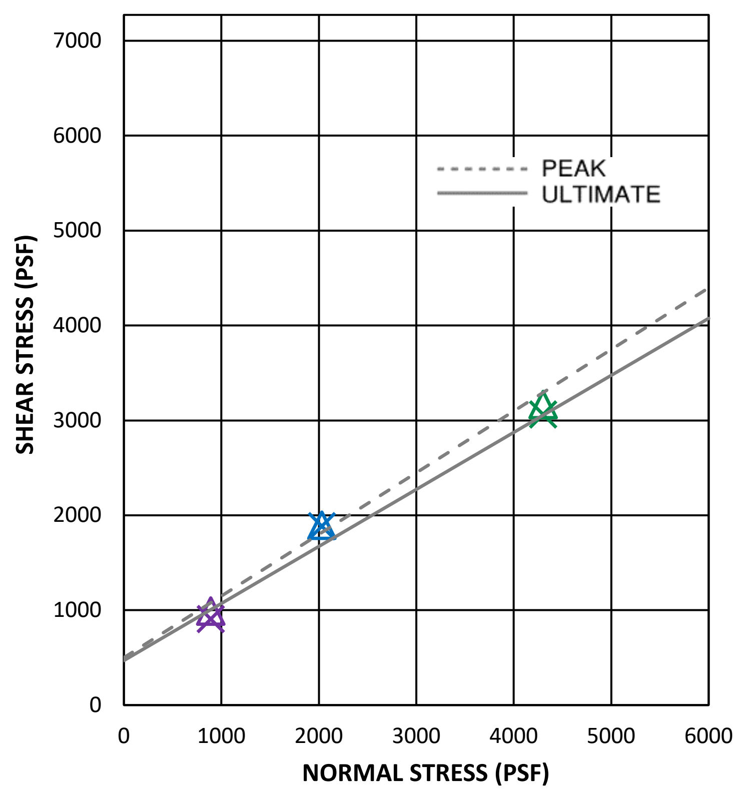
SAMPLE NO.:	B5-1	GEOLOGIC UNIT:	Qal
SAMPLE DEPTH (FT):	4'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	8.1	8.2	8.9	8.4
DRY DENSITY (PCF):	118.2	117.3	118.5	118.0

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	15.3	14.1	14.7	14.7
PEAK SHEAR STRESS (PSF):	990	1895	3168	--
ULT.-E.O.T. SHEAR STRESS (PSF):	905	1885	3064	--

RESULTS		
PEAK	COHESION, C (PSF)	500
	FRICTION ANGLE (DEGREES)	33
ULTIMATE	COHESION, C (PSF)	470
	FRICTION ANGLE (DEGREES)	31





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DIRECT SHEAR - ASTM D 3080

MEHS-SITE MODERNIZATION

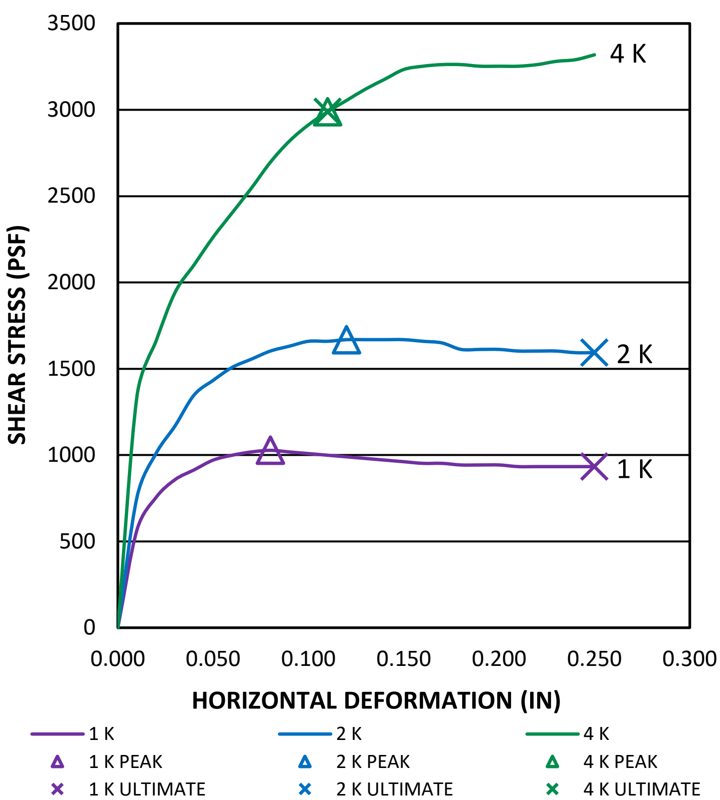
PROJECT NO.: G2820-42-01

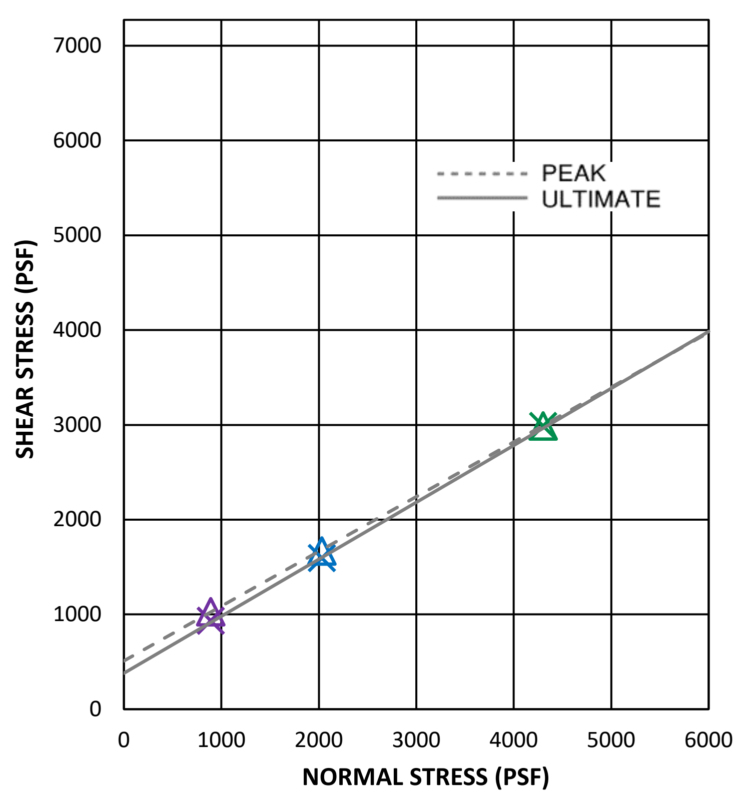
SAMPLE NO.:	B6-2	GEOLOGIC UNIT:	Qal
SAMPLE DEPTH (FT):	4'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	1.8	2.0	1.8	1.9
DRY DENSITY (PCF):	121.1	118.2	118.3	119.2

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	13.1	14.8	14.6	14.2
PEAK SHEAR STRESS (PSF):	1028	1669	2988	--
ULT.-E.O.T. SHEAR STRESS (PSF):	933	1593	2988	--

RESULTS		
PEAK	COHESION, C (PSF)	510
	FRICTION ANGLE (DEGREES)	30
ULTIMATE	COHESION, C (PSF)	380
	FRICTION ANGLE (DEGREES)	31





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DIRECT SHEAR - ASTM D 3080

MEHS-SITE MODERNIZATION

PROJECT NO.: G2820-42-01

APPENDIX C

APPENDIX C

BEARING CAPACITY CALCULATION SHEET

FOR

MOUNTAIN EMPIRE HIGH SCHOOL
WHOLE SITE MODERNIZATION
3305 BUCKMAN SPRINGS ROAD
PINE VALLEY, CALIFORNIA

PROJECT NO. G2820-42-01

Bearing Capacity

Reference: French, Samuel E., *Design of Shallow Foundations* ,ASCE Press, Chapter 6, pp. 143-169.

Insert in highlighted fields			Calculated by:	N. BORJA
Project Name:	MEHS-SITE MODERNIZATION			
Project Number:	G2820-42-01			
Date:	10/12/2021			

Geologic Unit =	Qal			
Cohesion, c (psf) =	450			
Friction Angle, ϕ (deg.) =	30		tan(45+ϕ/2) =	1.73
Soil Density, γ (pcf) =	125		cot(45+ϕ/2) =	0.58
Width of Ftg., B (ft.) =	1			
Depth of Ftg, D_f (ft.) =	1.5			
Length of Ftg., L (ft) =	2	L > B		
Depth of Water Below Ftg. (ft.) =	500			
Vertical Load, Q (lbs) =	0			
Factor of Safety =	3			
Pressure Under Ftg. (psf) =	0.00			

Rectangular Footing

Bearing Capacity Factors (6-13):	N_γ =	25.86	Bearing Capacity Factors (6-14):	N_γ =	22.40
	N_q =	14.93	(Better Fit - Suggested)	N_q =	18.40
	N_c =	25.86		N_c =	30.14

Terzahi Modified

Factors? 1 = (6-13), 2 = (6-14)	1			q_{ULT} (psf) =	26811.94
Shape Factors (6-16):	s_γ =	0.80	(gross)	q_{ALL} (psf) =	8937.31
	s_q =	1.29	(net)	q_{ALL} (psf) =	8874.81
	s_c =	1.29			
Depth Factors (6-17):	d_γ =	1.00			
	d_q =	1.28			
	d_c =	1.39			
Water Table Factors (Table 6-4):	w_γ =	1.00			
	w_q =	1.00			
	w_c =	1.00			

APPENDIX D

APPENDIX D

INFILTRATION TEST SHEETS

FOR

MOUNTAIN EMPIRE HIGH SCHOOL
WHOLE SITE MODERNIZATION
3305 BUCKMAN SPRINGS ROAD
PINE VALLEY, CALIFORNIA

PROJECT NO. G2820-42-01

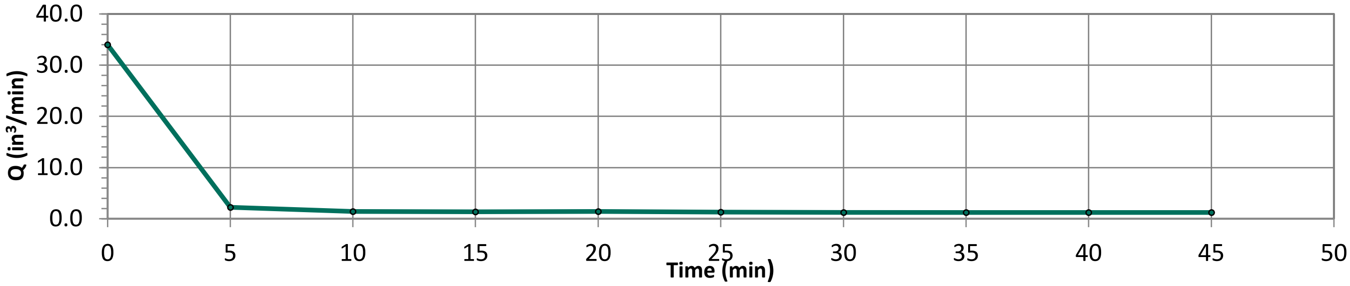
TEST NO.: **A-1**

GEOLOGIC UNIT: **Qal**

EXCAVATION ELEVATION (MSL, FT): **3146**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	6
BOREHOLE DEPTH (FT):	3.9
TEST/BOTTOM ELEVATION (MSL, FT):	3142
MEASURED HEAD HEIGHT (IN):	5.3
CALCULATED HEAD HEIGHT (IN):	5.0
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**1.218**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.199**
FACTORED INFILTRATION RATE (IN/HR):	**0.100**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	6.135	169.89	33.978
3	5.00	0.400	11.08	2.215
4	5.00	0.260	7.20	1.440
5	5.00	0.240	6.65	1.329
6	5.00	0.255	7.06	1.412
7	5.00	0.235	6.51	1.302
8	5.00	0.225	6.23	1.246
9	5.00	0.220	6.09	1.218
10	5.00	0.220	6.09	1.218
11	5.00	0.220	6.09	1.218

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AARDVARK PERMEAMETER TEST RESULTS

MEHS-SITE MODERNIZATION

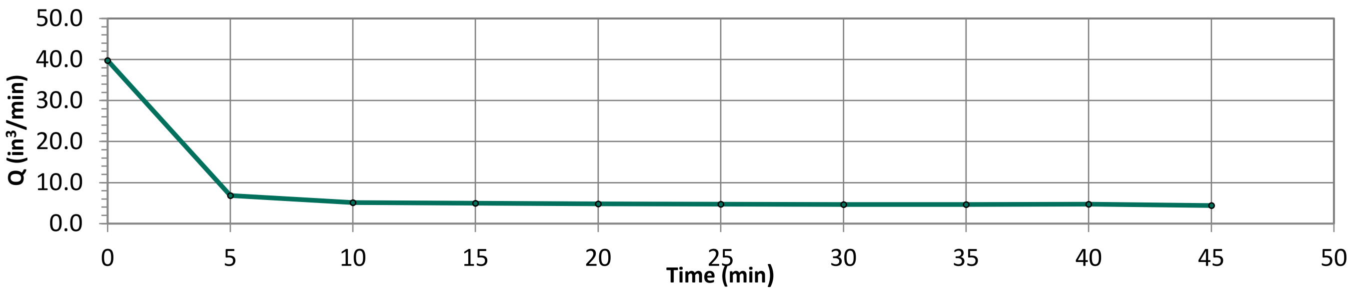
PROJECT NO.: G2820-42-01

TEST NO.: **A-2** GEOLOGIC UNIT: **Qal**

EXCAVATION ELEVATION (MSL, FT): **3137**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	6
BOREHOLE DEPTH (FT):	4.0
TEST/BOTTOM ELEVATION (MSL, FT):	3133
MEASURED HEAD HEIGHT (IN):	5.3
CALCULATED HEAD HEIGHT (IN):	5.2
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**4.458**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.729**
FACTORED INFILTRATION RATE (IN/HR):	**0.365**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	7.180	198.83	39.766
3	5.00	1.230	34.06	6.812
4	5.00	0.930	25.75	5.151
5	5.00	0.895	24.78	4.957
6	5.00	0.865	23.95	4.791
7	5.00	0.855	23.68	4.735
8	5.00	0.835	23.12	4.625
9	5.00	0.845	23.40	4.680
10	5.00	0.855	23.68	4.735
11	5.00	0.795	22.02	4.403
12	5.00	0.815	22.57	4.514
13	5.00	0.805	22.29	4.458

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AARDVARK PERMEAMETER TEST RESULTS

MEHS-SITE MODERNIZATION

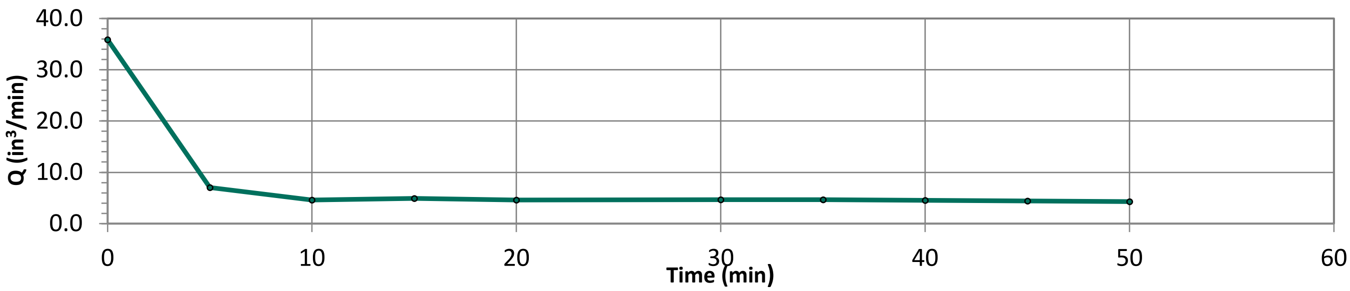
PROJECT NO.: G2820-42-01

TEST NO.: **A-3** GEOLOGIC UNIT: **Qal**

EXCAVATION ELEVATION (MSL, FT): **3136**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	6
BOREHOLE DEPTH (FT):	3.8
TEST/BOTTOM ELEVATION (MSL, FT):	3132
MEASURED HEAD HEIGHT (IN):	5.3
CALCULATED HEAD HEIGHT (IN):	5.1
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**4.283**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.701**
FACTORED INFILTRATION RATE (IN/HR):	**0.350**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	6.475	179.31	35.862
3	5.00	1.270	35.17	7.034
4	5.00	0.830	22.98	4.597
5	5.00	0.885	24.51	4.902
6	10.00	1.670	46.25	4.625
7	5.00	0.840	23.26	4.652
8	5.00	0.840	23.26	4.652
9	5.00	0.820	22.71	4.542
10	5.00	0.800	22.15	4.431
11	5.00	0.775	21.46	4.292
12	5.00	0.770	21.32	4.265
13	5.00	0.775	21.46	4.292

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AARDVARK PERMEAMETER TEST RESULTS

MEHS-SITE MODERNIZATION

PROJECT NO.: **G2820-42-01**

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