DRAFT
GEOTECHNICAL ENGINEERING INVESTIGATION AND GEOLOGIC/SEISMIC HAZARDS EVALUATION PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO & SANTIAGO ROADS RIVERSIDE COUNTY, CALIFORNIA

Project Number: H17401.01

For:

DLR Group
1650 Spruce Street #300
Riverside, CA 92507

April 28, 2023

MOORE TWINING ASSOCIATES, INC.
Since 1898

April 28, 2023 ***DRAFT*** H17401.01

Mr. Andrew Thompson
DLR Group
1650 Spruce Street #300
Riverside, CA 92507

Subject: Geotechnical Engineering Investigation and
Geologic/Seismic Hazards Evaluation
Proposed Fire Station
Southwest of the Intersection of Morongo Road and Santiago Road
Riverside County, California

Dear Mr. Thompson,

We are pleased to submit this Geotechnical Engineering Investigation and Geologic/Seismic Hazard Evaluation report prepared for the fire station proposed at the southwest corner of Morongo Road and Santiago Road within the federal reservation of the Morongo Band of Mission Indians which is located northeast of the City of Banning in Riverside County, California. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, a geologic setting, tectonics and seismicity, evaluations (including geologic and seismic hazards), conclusions, and recommendations.

We recommend that those portions of the plans and specifications that pertain to earthwork, pavements, and foundations be reviewed by Moore Twining Associates, Inc. (Moore Twining) to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement, however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

We appreciate the opportunity to be of service to the DLR Group. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience at (800) 268-7021.

Sincerely,
MOORE TWINING ASSOCIATES, INC.

DRAFT
Scott W. Krauter, RGE
Assistant Manager
Geotechnical Engineering Division

EXECUTIVE SUMMARY

Moore Twining Associates, Inc. conducted this geotechnical engineering and geologic/seismic hazard investigation for a new fire station proposed at the southwest corner of Morongo and Santiago Roads, on the Morongo reservation located in Riverside County, California. It is our understanding the fire station building will be approximately 18,600 square feet and will include an apparatus bay. Appurtenant construction is also anticipated to include enclosed patios, concrete flatwork, underground utilities, and Portland cement concrete and asphalt concrete pavements.

On March 6 and 7, 2023, a total of eight (8) borings were drilled to depths of between 10 and 29 feet below site grade (BSG) within the building and apparatus bay areas. Many borings deeper than 10 feet BSG were terminated due to auger refusal on dense gravel/cobble/boulder materials. Also, a total of eight (8) borehole percolation tests were installed by drilling to depths of between 4 to 15 BSG within the leach field and storm water detention basin areas.

The near surface soils generally consisted of medium dense poorly graded and well graded gravel with varied sand and silt fractions. The medium dense condition was found to a depth of about 1 to 1½ feet BSG in each boring tested at the surface. Below this upper material, the gravel materials were found in a dense to very dense condition to the maximum depth explored, about 29 feet BSG. The soils throughout the depths explored contained larger cobble materials (greater than 3 inches) and some boulders (greater that 1 foot in size).

Due to the coarse gravel, cobble and boulder content anticipated for the onsite soils, it should be expected that significant amounts of the soils excavated will need to be processed in order to be used as engineered fill below the building pad, and as backfill in the pipe zone for installed utilities. Contractors should expect that soils will require equipment to process excavated materials to remove oversize gravel, cobbles and boulders through screening or crushing such that the materials retained on the 3/4-inch sieve are 30 percent or less prior to reuse as engineered fill. In addition, rock greater than 4 inches in the largest dimension should not be used within engineered fill soils.

The project site is located in a State of California surface fault rupture hazard zone (Alquist-Priolo Earthquake Fault Hazard Zone) associated with mapped fault traces along the active San Gorgonio Pass fault zone. Refer to Drawing No. 5 in Appendix A, which illustrates the limits of the fault rupture hazard zone in relation to the site location. Accordingly, there is a potential for fault rupture to occur at the site, which would be generally considered moderate to high. It was not the intent of this investigation to conduct fault trenching to evaluate potential surface fault rupture hazards. Due to the proximity of mapped active faults to the site and considering that the site is located in a State of California mapped fault rupture hazard zone, a surface fault rupture hazard investigation is needed to evaluate potential impacts associated with active faulting. All other geologic and seismic hazards evaluated were found to be have low risk to impact the site.

Due to the presence of larger cobble/boulder material, and the cut/fill conditions anticipated for the building pad, over-excavation and compaction of the upper 1.5 feet of the near surface soils and placement of a minimum of 2 feet of fill below the bottom of the foundations is recommended in the building pad area to reduce potential impacts with differential static settlement. When the building subgrade soils are prepared as recommended in this report, total and differential static settlements for the proposed structures are estimated to be 1 inch and ½ inch in 40 feet, respectively.

EXECUTIVE SUMMARY (Cont.)

The results of soil sample analyses indicate that the near-surface soils exhibit a "moderately corrosive" corrosion potential to buried metal objects. Chemical analyses indicated the soils exhibit an S0 Sulfate Exposure Class (per ACI 318) based on a water soluble sulfate in soil of less than 0.1 percent by mass.

The percolation tests conducted in the primary leach field and 100 percent expansion field suggest some areas/depths of the designated leach field may not be feasible for trench type disposal through infiltration. However, the results of the small borehole tests conducted in very dense soils, classified as well graded gravel with cobbles and boulders, may not accurately represent the infiltration rate of a larger leach trench sidewall. Therefore, it is suggested conduct supplemental percolation testing in larger area test pits to confirm or revise percolation testing rates to use for final leach field design.

The percolation tests conducted in the proposed basin area indicate infiltration rates of 1.5 and 2 inches per hour at depths of 10 and 15 feet BSG, respectively. This report recommends that the lowest unfactored infiltration rate of 1.5 inches per hour be used for preliminary design, with appropriate safety factors.

This executive summary should not be used for design or construction and should be reviewed in conjunction with the attached report.

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RIVERSIDE COUNTY, CALIFORNIA

Project Number: H17401.01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering and geologic/seismic hazard investigation for a new fire station to be located southwest of the intersection of Morongo Road and Santiago Road within the federal reservation of the Morongo Band of Mission Indians located northeast of the City of Banning in Riverside County, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by the DLR Group to conduct this investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides descriptions of general geologic conditions, an evaluation of the findings, an evaluation of geologic and seismic hazards, general conclusions, and related recommendations. The report appendices contain the drawings (Appendix A); the logs of borings (Appendix B); the results of laboratory tests (Appendix C); and the results of percolation testing (Appendix D).

The Geotechnical Engineering Division of Moore Twining performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of this investigation was to conduct a field exploration, a laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

2.1.1 A description of general subsurface soil and groundwater conditions encountered;

2.1.2 Soil profile type, site coefficients and mapped Maximum Considered Earthquake spectral response acceleration parameters in accordance with the current California Building Code;

2.1.3 Evaluation of seismic settlement and liquefaction potential;

2.1.4 Recommendations for earthwork construction, including site and subgrade preparation, and engineered fill;

2.1.5 Recommendations for temporary excavations, utility trench excavation and backfill, and excavation stability;

2.1.6 Foundation design parameters including allowable soil bearing capacity, foundation settlement, minimum foundation depth, and lateral resistance;

2.1.7 Recommendations for slab-on-grade floors and exterior concrete flatwork;

2.1.8 Evaluation of soil corrosion potential;

2.1.9 Recommendations for asphaltic concrete and Portland cement concrete pavements;

2.1.10 Final test boring logs and laboratory results; and

2.1.11 Review and discussion of potential geohazards in accordance with CGS Note 48 including.

As discussed in this report, the site is located in a State of California designated fault hazard zone. However, this investigation does not include a surface fault rupture hazard evaluation. Moore Twining has provided a separate estimate for a fault rupture hazard investigation. A separate report will be prepared to evaluate surface fault rupture hazard if this scope of work is authorized.

This report is provided specifically for the proposed improvements described in the Anticipated Construction Section 3.3 of this report. This investigation did not include in-place density tests, an environmental investigation, or an environmental audit.

2.2 **Scope:** Our proposal, dated February 27, 2023 (MTP 22-0731), outlined the scope of our services. The actions undertaken during the investigation are summarized as follows:

2.2.1 A site plan, prepared by the DLR Group (undated), was reviewed for general project information. A Topographic survey of the site, prepared by PBLA Surveying for Kimley-Horn (Orange Office), dated 2/08/2023, was provided and reviewed for an understanding elevation changes across the site.

2.2.2 A visual site reconnaissance and a subsurface exploration program, including test borings and percolation tests, was conducted.

2.2.3 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

2.2.4 Mr. Andy Thomson and Mr. Roberto Marquez with the DLR Group,and Mr. Geoff Rubin with Rick Engineering Company (project Civil engineer) were consulted during the investigation.

2.2.5 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and the engineering properties of the subsurface soils encountered.

2.2.6 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, geologic setting, tectonics and seismicity, evaluation, conclusions, and recommendations.

3.0 BACKGROUND INFORMATION

The site description, site history and previous studies, and the anticipated construction are summarized in the following subsections.

3.1 Site Description: The 5.23 acre site for the new fire station is located southwest of the intersection of Morongo Road and Santiago Road, within the federal reservation of the Morongo Band of Mission Indians. According to on-line sources, the Morongo reservation consists of 35,000 acres located at the base of the San Gorgonio and San Jacinto Mountains. The fire station site is located about 2 miles northeast of the City of Banning in Riverside County, California. A site location map is presented on Drawing No. 1 in Appendix A. The site was bound to the south by single family residences and undeveloped native lands; to the west by cleared lands possibly used for agriculture; and, to the north and east by the asphalt paved public roadways of Morongo and Santiago Roads. The proposed fire station and project area are shown on Drawing No. 2 in Appendix A of this report.

At the time of our field investigation, the site was vacant, undeveloped native desert land. The site was noted to have a gradual slope to the southwest from the high point at the intersection of Morongo and Santiago Roads to the low point just east of the southwest corner of the site.

At the time of our site observations, the ground surface was covered with native desert vegetation consisting of 1 to 2 foot tall bushes with underlying sparse native grasses and weeds. Native gravel and cobbles (up to 18 inches in diameter) and some larger boulders (over 2 feet in diameter) were noted across the site surface typical of this type of desert native environment. Some unpaved roads were noted around the south and west perimeters of the site, and some foot “trails” were noted in the interior areas. A few native small trees were also noted throughout the site. Also, during our March 2023 investigation, some scattered refuse/debris were noted mostly along the roadways, and some of the low grasses and small bushes were noted to be green, while other larger bushes appeared dry.

The Topographic survey indicates the site elevations (Datum NAVD 88) range from 2,320-½ feet at the northeast corner, to a elevation of 2,291-½ feet in the west portion of the south boundary of the site. This reflects an overall elevation change of 29 feet across the site.

3.2 Site History and Previous Studies: Based on our review of on-line aerial images of the site area for several years from 1996 to 2022, the site appears to have been vacant and undeveloped since at least 1996. However, some possible grading/clearing was noted near and along the north portion of Santiago Road in a 2004 image. Also, over the years (2006 to 2016), images show evidence of what appears to be dense vegetation (tree growth) at the low portion of the site (southwest corner). The images do show that the single family residence to the south was constructed prior to 2002, and a 2009 image shows the property to the west was cleared of native vegetation.

Moore Twining has conducted numerous geotechnical investigations for commercial developments near the Morongo Casino, about 1½ miles southeast of the site. However, these investigations were not conducted near the fire station site. No previous reports of geotechnical engineering investigations, compaction testing or environmental studies conducted for this site were provided for review during this investigation. If available, these reports should be provided for review and consideration for this project.

3.3 Anticipated Construction: Based on our review of the DLR Group site plan, the project site development will include a new fire station building with a fire apparatus bay, patios, concrete pads and various site improvements. These building improvements are planned in the north-central portion of the site. Also, landscape areas and frontage improvement are proposed adjacent to the pubic roadways along the northern and eastern portions of the site. In addition, a parking lot is proposed for the northeast portion of the site. The plan also designates an area for a leach field along the west boundary and southwest portion of the site, and a storm water detention basin is planned in the southeast portion of the site.

The fire station is anticipated to occupy about 18,600 +/- square feet in plan area (including the apparatus bay). It is anticipated that the fire station building will consist of concrete masonry unit (CMU) walls, a roof of structural steel or wood trusses and joists, and the floors will be concrete slabs on grade. The building will be supported on conventional continuous and isolated shallow spread foundations.

Information provided by Miyamoto International (project structural engineer) indicated maximum loading of foundations for apparatus bay walls of 4.5 kips per lineal foot for dead loads and 1.25 kips per lineal foot for live loads (5.75 kips per lineal foot total); and, maximum column loads of 32 kips dead load plus 34 kips live load (66 kips total).

The site plan shows concrete flatwork for patios and walkways, Portland cement concrete for fire apparatus areas and asphalt concrete pavements for auto parking.

The details of the planned onsite sewage disposal system (leach field) were not available at the time of this investigation, so it was assumed that leach lines could range from about 4 to 7 feet deep. Also, the site plans shows a storm water detention basin will be cut in the southeast portion of the site. It is expected that this basin will be used to dispose of on-site surface water. Details of the basin were not provided, so it was assumed that the basin would be about 5 to 10 feet deep.

The project civil engineer (Rick Engineering) reported that preliminary grading plans were not available at the time this report was prepared. However, given the existing topography, it is expected that fill materials will be excavated from the south detention basin to fill the northern portion of the site to grade a flat building pad area.. Based on the 6 feet of elevation change across the building area, it is expected the building pad could be filled by as much as about 5 to 7 feet to achieve pad grade. Thus, it is expected that fill depths near the northern edge of the building pad may range from about 0 to 1 foot (or a possible slight cut condition) and fill depths near the southern edge of the building pad may range from about 5 to 7 feet.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing program conducted for this investigation are summarized in the following subsections.

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, soil sampling, standard penetration tests, and percolation tests.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by a Moore Twining staff engineer on March 6 and 7, 2023. The features noted are described in the "Background Information" section of this report.

4.1.2 Drilling Test Borings: The number of soil borings drilled for the proposed buildings were based upon the minimum requirements of the 2019 CBC and CGS Note 48 (frequency of at least one (1) test boring per 5,000 square feet of building area, with a minimum of two borings for any one building). The depths of the borings were selected based on the type of construction, the depth of influence of the anticipated foundation loads and the subsurface soil conditions.

On March 6 and 7, 2023, a total of eight (8) borings (B-1 through B-8) were drilled with a CME-75 drill rig equipped with 6⅝-inch outside diameter (O.D.) hollow-stem drilling augers. Borings B-1 through B-5 were drilled to depths of between 10 to 29 feet below site grade (BSG) within the building and apparatus bay areas. Most of these boring were terminated due to auger refusal on dense gravel/cobble/boulder materials prior to the intended 51½ feet to 21½ feet BSG.

Also, borings B-6, B-7 and B-8 were drilled in the locations proposed for the parking lot, leach field and detention basin areas, respectively. These borings were drilled to depths of 10 to 15 feet BSG which were the intended depths of exploration at these locations. The approximate locations of the borings are depicted on Drawing No. 2 in Appendix A of this report.

The borings were drilled and soils logged during drilling. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of the test borings.

Test boring locations were determined by pacing with reference to the existing site features. The locations, as described, should be considered approximate. The elevations of the borings were estimated from the Topographic survey of the site, prepared by PBLA Surveying for Kimley-Horn (Orange Office), dated 2/08/2023. The boreholes were loosely backfilled with material excavated during the drilling operations. Due to the loose nature of the test boring backfill, some settlement of the backfill should be anticipated.

4.1.3 <u>Soil Sampling:</u> During drilling of the hollow-stem auger borings, standard penetration tests were conducted, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜ inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches, or portion thereof, and the number of blows required to advance the sampler an additional 12 inches, or portion thereof, is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in stainless steel rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing. In addition, bulk samples of soil were obtained for laboratory testing.

4.1.4 <u>Percolation Testing:</u> Percolation tests were conducted at eight (8) locations in borings drilled within the leach field and detention basin areas (see locations of P-1 through P-8 on Drawing No. 2 in Appendix A of this report). The procedures used for installation and conducting the tests was programmed in general conformance with the Percolation Testing and Exploratory Boring Procedures; Chapter 2 of the Riverside County Local Agency Management Program for On-Site Wastewater Treatment Systems, dated November 17, 2022 (referred to hereinafter as the Riverside County LAMP).

The percolation test borings were installed to depths of approximately 4 to 12 feet BSG in the six (6) locations installed for testing in the leach field area, and to depths of about 10 and 15 feet BSG in two (2) locations installed for testing in the detention basin area. Percolation test holes were drilled with a truck-mounted CME-75 drill rig equipped with 8-inch outside diameter (O.D.) hollow-stem augers. The percolation tests were conducted within the boreholes for use in estimating percolation rates for the leach field area and infiltration rates for the storm water basin area.

The test holes were cylindrical with a diameter of about 8 inches. Gravel packing was used to protect the sidewalls of the holes from washout during refilling. A 2-inch diameter perforated PVC pipe was placed in the boreholes and used to transmit poured water to the bottom of the holes. Prior to the start of the percolation testing, the percolation holes were presoaked with water a minimum of two (2) times.

Percolation testing included adding water to the test holes periodically and measuring the drop in water level over time to the nearest 0.1 inch. Measurements of water levels and the time of each reading were recorded during testing. The rates of water level decline near the end of the test period (generally stabilized) were used to determine the average stabilized percolation rates.

Details of the percolation test installation, and water depth (head) versus time readings are presented in Appendix D of this report.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils sampled and tested. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface materials encountered.

The results of laboratory tests conducted on samples obtained from the test borings are summarized on the figures in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

5.1 Soil Profile: The near surface soils generally consisted of gravel material, both poorly graded and well graded gravel with varied sand and silt fractions, extending to the maximum depth explored, about 29 feet BSG. The soils contained larger cobble materials (greater than 3 inches) and some boulders (greater that 1 foot in size) throughout the depths explored as evidenced by auger refusal (hollow stem augers and soil samplers typically cannot penetrate larger cobbles/boulders) and the presence of cobbles and occasional boulders at the surface.

The foregoing is a general summary of the soil conditions encountered in the test borings drilled for this investigation. Detailed descriptions of the soils encountered at each test boring are presented on the logs of borings in Appendix B. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual. General soil profiles are also included as cross-sections A-A' and B-B' on Drawing Nos. 8 and 9 in Appendix A of this report.

5.2 Soil Engineering Properties: The following is a description of the soil engineering properties as determined from our field exploration and laboratory testing.

The gravel materials encountered were described as medium dense to very dense, as determined by standard penetration test (SPT), N-values, ranging from 12 to over 50 blows per foot (sampler refusal). Medium dense conditions were generally noted in the upper 18 inches from the surface, and dense to very dense conditions were found below about 1½ feet BSG.

The moisture contents of the gravel materials tested ranged from about 2 to 8 percent. Testing of relatively undisturbed gravel samples indicated dry densities ranging from 122.6 to 131.7 pounds per cubic foot. The results of sieve analysis conducted on several poorly graded and well graded gravel samples indicated sand fractions ranging from 35.7 to 42.4 percent while the fines fraction (silt and clay) ranged from 6.3 to 13.8 percent.

An expansion index test conducted on a near surface sample indicated an expansion index value of 0.

Maximum Density-Optimum Moisture Determination: A maximum density-optimum moisture determination conducted on a sample composited from several borings from depths of 0 to 5 feet BSG indicated a maximum dry density of 138.4 pounds per cubic foot at an optimum moisture content of 6.7 percent.

Shear Strength Determination - Since relatively undisturbed samples contained coarse sands and gravel materials, direct shear tests were conducted on samples prepared by screening out the gravel materials and remolding the sand and fines fraction to a relative compaction of about 90 percent based on the non-rock corrected maximum dry density. The results of this remolded shear strength testing indicated an angle of internal friction of 34 degrees with 500 pounds per square foot cohesion.

R-Value: The results of two R-value tests conducted on samples collected from depths of 0 to 5 feet BSG indicated R-values of 73 and 78.

Chemical Tests: The results of chemical tests performed on a near surface soil sample indicated a pH value of 7.9, a minimum resistivity value of 9,100 ohm-centimeters, 0.0082 percent by weight concentration of sulfate, and none detected (less than 0.0040 percent by weight) concentration of chloride.

5.3 <u>Groundwater Conditions</u>: During our March 7 and 8, 2023 field exploration, free groundwater was not encountered in the borings drilled to the maximum depth explored of about 29 feet BSG. Based on our review of water well data on the Department of Water Resources SGMA Data Viewer web site, a well located about 1.6 miles west of the site indicated that groundwater has ranged from about 580 to 607 feet BSG between the years 2002 and 2021.

It should be recognized, that water table elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

5.4 <u>Percolation Test Results</u>: The results of the percolation tests are summarized in Table No. 1 below. Since the details and anticipated depths of the leach field system and on-site stormwater retention basin were not known, the percolation tests were conducted at varying depths between 4 and 12 feet BSG, and at depths of 10 and 15 feet BGS in the basin area. The results of the percolation tests are presented in Appendix D.

It should be noted that the field tests do not take into account the long term effects of subgrade saturation, silt accumulation, groundwater influence, nor vegetation. In general, the infiltration rate of the soils will decrease when the soils are saturated and the reduction in the infiltration rate increases the longer the soils are saturated. Published studies indicate field infiltration rates can significantly overestimate the saturated permeability. In addition, soil bed consolidation, sediment, suspended soils, etc. in the discharge water can result in clogging of the pore spaces in the soil. This clogging effect can also reduce the long term infiltration rate. Numerous other factors, such as variations in soil type and soil density across the entire area of the system can influence the infiltration rate, both short and long term.

Table No. 1
Results of Percolation Testing

Location and Depth	Percolation Rate (Minutes per Inch)[1]	Unfactored Infiltration Rate (Inches per Hour)[1]	Soil Type
P-1 at 12 Feet BSG	NP*	NI	Dense Poorly Graded Gravel
P-2 at 7 Feet BSG	90*	<0.1	Dense Poorly Graded Gravel
P-3 at 9 feet BSG	14	1.2	Very Dense Poorly Graded Gravel
P-4 at 5 feet BSG	107*	0.1	Very Dense Poorly Graded Gravel
P-5 at 4 feet BSG	30	0.5	Dense Poorly Graded Gravel
P-6 at 5 feet BSG	50	0.2	Medium Dense Silty Gravel with Sand
P-7 at 15 feet BSG	3.5	1.5	Very Dense Silty Gravel with Sand
P-8 at 10 feet BSG	2.5	2.0	Dense Silty Gravel with Sand

Notes:
BSG - Below site grade
[1] - results based on 1 foot of water, includes no factor of safety
* result is slower than 60 min/in which is the lowest percolation rate allowed in the Riverside County LAMP
NP/NI - No Significant Percolation/Infiltration (less than 0.1 inch over ½ hour or more than 200 min/inch)

6.0 GEOLOGIC SETTING

The site is located in an elongated narrow valley (pass) between the San Bernardino and San Jacinto Mountains located in the southern geographic portion of California. The site is situated on the north portion of the San Gorgonio Pass, between the San Bernardino Valley to the west and the Coachella

Valley to the east. The pass forms the border between the San Gorgonio Mountains (southern portion of the San Bernardino Mountains) on the north and San Jacinto Mountains to the south.

Small, intermittent streams enter the valley from both arid ranges that flow into the intermittent San Gorgonio River which flows and drains to the east into the larger Coachella Valley. Large coalescing alluvial fans have developed along each side of the pass/valley. Specifically, the site for this investigation is located on alluvial fan deposits from Potrero Creek Canyon with has a mouth about ½ mile north of the site.

The Geologic Map of The Cabazon Quadrangle, prepared by the Division of Mines and Geology, dated 2004 (see Drawing No. 3 in Appendix A) indicates that the site is underlain by Older Surficial Sediments described as Quaternary (Recent) alluvial fan deposits of the San Gorgonio Pass. The sediments are sand and gravel of plutonic and gneissic detritus derived from rising San Bernardino Mountains to the north; slightly dissected by steam channels; including small alluvial fans at the base of and derived from the San Jacinto Mountains in the south area.

Regional geologic and site geologic maps are included in Appendix A as Drawing Nos. 3 and 7, respectively, and geologic (soil profile) cross sections through the proposed building area are included as Drawing Nos. 8 and 9 in Appendix A of this report.

7.0 TECTONICS AND SEISMICITY

Numerous active faults are located throughout the site region and contribute to design seismic ground motion estimates. An "active fault" is defined, for the purpose of this evaluation, as a fault that has had surface displacement within Holocene time (about the last 11,700 years). A widely accepted definition of a potentially active is a fault showing evidence of displacement older than 11,700 years and younger than 1.6 million years (Pleistocene). Faults showing evidence of displacement older than 1.6 million years are usually classified as "inactive.”

The site is situated along the active San Gorgonio Pass fault zone which is a complex zone of faulting controlled by the San Andreas fault system as it attempts to accommodate the southern “Big Bend”segment which is a part of the Mojave Desert and Coachella Valley segments within southern California (Treiman, 1994). Within the San Gorgonio Pass area, strain appears to be transferred, from the Coachella Valley segment to the Mojave Deseret segment, in a very complex manner. The change in strike of the fault system imposes a component of compression in addition to the strike slip displacement. These stresses are accommodated by displacement along the San Gorgonio Pass Fault Zone as well as the Banning and Garnet Hill Faults.

The site location, relative to known nearby fault systems within 50 miles of the site, is depicted on Drawing No. 4 in Appendix A of this report.

The following subsections briefly describe the major fault systems contributing to the seismicity of the site area.

7.1 San Gorgonio Pass Fault Zone: The Fault Evaluation Report FER-235 (Treiman 1994) indicates the San Gorgonio Pass Fault is actually a zone of discontinuous reverse and thrust faults and associated tear faults. An Earthquake Fault Zone was established around portions of the fault zone based on Smith's work (CDMG, 1980), and Matti and others (1985 & 1992) subsequently mapped a much greater extent to the fault zone and clarified its relation to the regional tectonics. This work characterized the fault as a zone of northeasterly trending thrust faults connected by northwesterly trending tear faults. Drawing No. 5 in Appendix A shows the portion of plate IIIc (CDMG FER-235, Treiman 1994) which shows the segments of this fault zone with respect to the site.

The segment of the fault zone that is located near the site is the San Gorgonio River to Millard Canyon segment (Plates Ic, IIc, and IIIc of CDMG FER-235, Treiman 1994). Drawing No. 5 in Appendix A of this report shows the portion of plate IIIc (CDMG FER-235, Treiman 1994) which shows the segments of this fault zone with respect to the site. The west portion of this segment (near the site) is noted to have weak and discontinuous faulting which then steps to the southeast to a more prominent and continuous thrust fault. The FER-235 report concludes: "This section of the fault zone is apparently active based on several scarp segments in Holocene alluvial fans. Fault expression is notable at and east of Potrero Creek. This is roughly where the San Andreas Fault sweeps southward into the Pass and probably reflects the introduction of stress from that fault zone. The larger displacement of Qyf_3 across the Millard fan suggests a vertical slip rate of about 2.6-3.6 mm/yr."

7.2 San Andreas Fault: The San Andreas fault is the major strike slip-slip fault dominating the tectonics of southern California in general, and the San Gorgonio Pass in particular. According to the Fault Activity Map of California, prepared by the California Geological Survey (see Drawing No. 4 in Appendix A), the nearest segment of the San Andreas fault zone is identified as the Banning fault (Strand B) which lies approximately 0.6 miles north of the site.

The local San Bernardino strand of the San Andreas fault (also referred to as the South Branch) is clearly active as it approaches the region near the site from the north, and based on topography, is a well defined Holocene-active fault in the Potrero Creek area near the site. It has been suggested by various geologists that the surface displacement dies out as the San Andreas fault approaches the San Gorgonio Pass, with slip transferred to other faults or continuing along a deeply buried San Andreas structure. Subtle geomorphic evidence, as well as the more obvious offsets approaching Millard Canyon suggest that the surface trace of the San Andreas Faults does continue southeast along Potrero Creek from Burro Flats until it is joined by the Gandy Ranch Fault and then merges with, or is truncated by, the Banning Fault.

7.3 Banning Fault Zone : In the San Gorgonio Pass area, the Banning Fault Zone is mostly an older strike-slip structure which as been reactivated in response to local compression. According to the Fault Activity Map (see Drawing No. 4 in Appendix A), the nearest segment of the Banning Fault (Strand B) lies approximately 0.6 miles south of the site.

The eastern segment of the Banning Fault, as it exits the Pass, is the main expression of the strike-slip of the San Andreas Fault Zone while the western segment near Potrero Creek (and the site) nearly intersects the San Gorgonio Pass Fault Zone. The segment of the fault that trends south of the site and eastward towards the Coachella Valley may be the most active segments.

8.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface soil conditions determined from the field exploration and laboratory testing program and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report.

8.1 Geologic Hazards: The potential geologic hazards of flooding, landslides, and volcanic activity are described in the following subsections.

8.1.1 Flooding: The site is located partially in Community Panel number 06065C0829G, effective date August 28, 2008, which denotes a Zone D "areas in which flood hazards are undetermined, but possible."

Also, the California Department of Water Resources, Division of Safety of Dams online Dam Breach Inundation Map Viewer was utilized to determine if the site could be flooded from dam failure. The data shows that the site and Banning areas are not within an inundation zone for catastrophic failure of any known dam. Thus, the potential for flooding at the site appears to be low.

8.1.2 Landslides: The California Department of Conservation, California Geological Survey online database of Earthquake Zones of Required Investigation was reviewed to identify if the site is within, near, or downslope of any known areas for landslides to occur that pose a risk for damage. The data did not show the site was within any landslide hazard zones; and, the mountain areas ½ mile north of the site show a low risk for landslide susceptibility. Therefore, landslide hazard is not anticipated to be a factor for the project site.

8.1.3 Volcanic Activity: California includes six regions with a history of late Pleistocene volcanic eruptions, that are subject to hazards from future eruptions (Miller, 1989). The area that is closest to the site is the Salton Buttes area located on the south shore of the Salton Sea about 88 miles southeast of the site. Based on the distance of volcanic hazards from the site, the prospect for volcanic hazards to impact the site during the design life of the facility is considered low.

8.1.4 <u>Conditional Geologic Hazards</u>: Conditional geologic hazards, as identified in section 31 of California Geological Survey Note 48 are discussed in the following subsections.

8.1.4.1 <u>Hazardous Materials</u>: Hazardous materials such as methane gas, hydrogen-sulfide gas and tar seeps are not known to be present in the project area and are not considered to be a concern at the subject site.

8.1.4.2 <u>Radon Gas</u>: Naturally occurring radon gas is known to occur in some areas of California. Radon gas can accumulate in buildings and breathing air with elevated radon concentrations results in an increased risk of developing lung cancer. Radon gas forms from radioactive decay of small amounts of the elements uranium and thorium, naturally present in rocks and soils. Rock types, such as black shales, marine phosphatic rocks, and certain igneous rocks, are associated with relatively higher levels of radon gas than other rock types. These rock types are not known to be present near the ground surface at the project site.

Our review of a database maintained by the Department of Health Services (DHS), California Indoor Radon Levels Sorted By Zip Code, last updated February, 2016, indicates that only one (1) of the ten (10) radon tests conducted within the same zip code (92220) as the proposed fire station reported levels of radon gas exceeding 4 picocuries per liter (14.8 picocuries per liter). The U.S. EPA recommends that individuals avoid long-term exposure to radon concentrations above 4 picocuries per liter.

Based on our review of the geologic conditions at the site and the referenced data reported by the DHS, potential hazards associated with radon gas appears to be low.

8.1.4.3 <u>Naturally Occurring Asbestos</u>: Asbestos occurs in soil and rock naturally in certain geologic settings in California. It has been documented that inhalation of asbestos fibers may cause negative health effects. Most commonly, asbestos is associated with serpentinite and partially serpentized ultramafic rocks. Ultramafic rocks are scattered throughout much of the Sierra Nevada mountain and Coast Ranges regions. Review of Map Sheet 59, dated 2011, titled *"Reported Historic Asbestos Mines, Historic Asbestos Prospects, and Other Natural Occurrences of Asbestos in California,"* prepared by U.S.G.S. indicates the closest occurrence of asbestos is located in Santa Barbara 170 miles northwest of the site. However, the maps indicate some spot occurrence of asbestos have been noted in the San Jacinto Mountains south of the site, but these occurrences are not represented as a major deposit.

Ultramafic rocks, which commonly contain asbestos, were not encountered at the site and are not common to the geologic environment of the site. Accordingly, the potential to encounter surface or near surface naturally occurring asbestos containing rock is very low.

8.1.4.4 Hydrocollapse: Collapsible soils typically consist of loose, dry, low-density soils that exhibit significant consolidation with the addition of water. Collapsible soils can be found in many areas of the southwestern United States, specifically in areas of young alluvial fans, debris flow sediments, and wind-blown deposits. Collapsible soils can possess significant strength in a dry state; however, the saturation of collapsible soils can break the bonds holding the soil grains together and subsequent collapse of the soils can cause damage to overlying structures. Soils susceptible to hydrocollapse are common in arid desert environments with young alluvial fans. Based on the mapped surficial geology (older sediments) and the data obtained from this investigation, including high dry densities and the coarse granular nature of the soils, it is not expected that the subsurface soils at the site have a significant potential for hydrocollapse.

8.1.4.5 Regional Subsidence: An online map “Areas of Land Subsidence in California,” on a USGS website, indicates that the Banning area is not located in a mapped subsidence area for groundwater pumping, peat loss or oil extraction. Based on the relatively older alluvial fan deposits of the San Gorgonio Pass, land subsidence is not expected to impact the project.

8.2 Seismic Hazards: The potential for fault ground rupture, seismic groundshaking and seismic coefficients/earthquake spectral response acceleration design values, seiche, tsunamis, and liquefaction and seismic settlement are described in the following subsections.

8.2.1 Faulting and Ground Rupture: Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it manifests in abrupt relative ground displacement. Damage resulting directly from fault rupture ground displacement occurs only where structures are located near or astride the fault traces that move.

The project site is located in a State of California surface fault rupture hazard zone (Alquist-Priolo Earthquake Fault Hazard Zone) due to mapped fault traces along the active San Gorgonio Pass fault zone. Refer to Drawing No. 5 in Appendix A, which illustrates the limits of the fault hazard zone in relation to the site location, including nearby mapped active fault traces (concealed and inferred). The locations of active and potentially active faults relative to the site were identified by the California Department of Conservation, California Geological Survey online database of Earthquake Zones of Required Investigation. One mapped active fault trace segment is immediately to the west (500 feet west of the western edge of the site and trending towards the site) and a second mapped active fault segment trends roughly east to west and is mapped about 550 feet north of the north edge of the site. Accordingly, there is a potential for fault rupture to occur at the site, which would be generally considered moderate to high. It was not the intent of this investigation to conduct fault trenching to evaluate potential surface fault rupture hazards. Due to the proximity of mapped active faults to the site and considering that the site is located in a State of California mapped fault rupture hazard zone, a surface fault rupture hazard investigation is needed to evaluate potential impacts. In the event surface fault rupture were to occur below, or adjacent to the building, significant, potentially catastrophic damage could occur. It should also be noted that surface fault rupture hazard is a potential impact to underground utilities that would be servicing the site.

We understand that a surface fault rupture hazard investigation will be conducted to evaluate the potential for surface fault displacement. If active faults are discovered, the building should be setback in accordance with recommendations of the future investigation.

8.2.2 **<u>Groundshaking</u>**: For any given earthquake, the rock in the immediate vicinity will respond with a certain maximum acceleration and with a predominant period that depends on the nature of the rock and the source mechanism. Away from the focus of the earthquake, the ground motions begin to attenuate. The way in which the earthquake wave is altered depends to a great degree on source characteristics and to a lesser degree on the travel path.

A summary of our review of historic seismic activity relative to the site is included below.

8.2.2.1 **<u>Historic Seismic Activity</u>**: The U.S. Geological Survey's earthquake database system identified approximately 149 historical earthquakes with magnitude 4.0 or greater have been recorded from 1900 to present (April 2023) within a 50 mile radius of the site. A map showing the location of the project site with relation to the approximate historical earthquake epicenter locations is presented on Drawing No. 6 in Appendix A of this report. The data presented on Drawing No. 6 includes the locations and magnitudes of the historical earthquakes.

The nearest earthquake event (estimated magnitude of 4.9) found during the search occurred on September 28, 1946, approximately 2.1 miles southwest of the site (in the City of Banning). The largest magnitude earthquake identified within the 50 mile radius search was the 7.3 magnitude Landers earthquake which occurred on June 28, 1992, approximately 28.6 miles northeast of the site. The estimated peak horizontal ground acceleration at the site from the Landers event (based on published Shake Maps in the USGS database), is about .015g. The highest peak horizontal ground acceleration of 0.18g estimated from the USGS database appears to have occurred at the site on July 8, 1986 as a result of the Morongo Valley Earthquake (M=6.0) located 13.3 miles east/northeast of the site.

8.2.2.2 **<u>Design Seismic Ground Motion Parameters and Site Class:</u>** Seismic coefficients and spectral response acceleration values were developed for design of the building as required by the 2019 California Building Code (CBC). Based on the 2019 CBC, the site is classified as a class D site (stiff soil profile type) with standard penetration resistance, N-values averaging between 15 and 50 blows per foot in the upper 100 feet below site grade.

A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 1.101g was determined for the site using the Ground Motion Parameter Calculator provided by the Structural Engineers Association of California website (https://seismicmaps.org/).

A table providing the recommended seismic coefficients and earthquake spectral response acceleration values for the project site is included in the Foundations recommendations section of this report.

8.2.3 Liquefaction and Seismic Settlement: Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing usually results. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction.

The California Department of Conservation, California Geological Survey online database of Earthquake Zones of Required Investigation was reviewed to identify if the site is within or near an area mapped for any susceptibility to liquefaction. The database indicated the site is not within or near a zone known to be susceptible to liquefaction. Furthermore, given the depth to groundwater and the dense soil conditions encountered, the potential for liquefaction to impact the project is insignificant.

The results of the test borings drilled for this investigation were considered for potential dry seismic settlement. Due to the dense to very dense nature of the gravel/sand/cobbles and boulders encountered, significant seismic settlement is not expected at the site.

8.2.4 Seiches and Tsunamis: A seiche is a wave generated by the periodic oscillation of a body of water whose period is a function of the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. The site is not near any large bodies of water, so seiches are not considered a significant hazard at the site.

Tsunamis are waves generated in oceans from seismic activity. Due to the inland location of the site, tsunamis are not considered a significant hazard for the site.

8.3 Soil Engineering: The following sections of this report include evaluations of potential geotechnical engineering impacts to the proposed site development.

8.3.1 Existing Surface and Subsurface Conditions: At the time of our field investigation, the site was vacant, undeveloped native desert land. The ground surface was covered with native desert vegetation of low bushes with sparse native grasses and weeds. Native gravel, cobbles and boulders (up to 18 inches in diameter) were noted across the site surface. Where existing vegetation is to be removed, these areas should be stripped of all vegetation and top soil, and removal of trees and vegetation should remove all root balls and roots greater than ¼ inch in diameter.

Given the undeveloped condition of the site, it is not expected that fill soils are present within the site. However, given the construction of improvements (roadways/residences) around the perimeter of the site, some fill soils may be present and could be encountered during grading. As part of the site preparation, all fill soils encountered during site preparation should be over-excavated and placed back as engineered fill in accordance with the recommendations of this report.

8.3.2 Processing Onsite Soils with Gravel, Cobbles and Boulders for Use As Engineered Fill: The near surface and deeper soils contain coarse gravel, cobbles and some boulders. Due to the size of the rock, special consideration is anticipated to be required for equipment selection for excavation, and the excavated materials are anticipated to require special processing such as crushing and screening the materials prior to use as fill.

In order to provide uniform support of foundations, allow compaction testing of the on-site soils for maximum density/optimum moisture determination in accordance with ASTM D1557 and allow determination of the relative compaction of compacted fill soils, the percentage of rock material retained on the 3/4-inch sieve (i.e., coarse gravel, cobbles and boulders) is required to be not more than 30 percent. Thus, the oversize gravel, cobbles and boulders would need to be removed or processed by some methods such that the materials retained on the 3/4-inch sieve are 30 percent or less. In addition to the requirements described above, this report also recommends that rock greater than 4 inches in the largest dimension not be used within engineered fill soils below the building pad or as backfill in the pipe zone of utility lines.

Based on the subsurface soil conditions with excessive coarse gravel and cobbles and boulders, the contractor will need to determine the methods they will use to achieve the specified requirements for engineered fill. These may include screening, crushing and blending the materials to achieve the gradation requirements for engineered fill. The particle-size recommendations for engineered fill are included in Engineered Fill Recommendations Section 10.5 of this report.

Due to the small diameter of the geotechnical borings and considering the many borings that encountered drilling refusal, in order to provide additional information for use in estimating the screening and processing requirements of the cobbles and boulders in the onsite materials, it is recommended pits be excavated at the site as part of the bid process, to allow the exposed soil, cobble and boulder conditions to be directly observed by the contractor's bidding the work.

8.3.3 Static Settlement and Bearing Capacity of Shallow Foundations: The potential for excessive total and differential static settlement of foundations and slabs-on-grade is a geotechnical concern that was evaluated for this project. The increases in effective stress to underlying soils which can occur from new foundations and structures, placement of fill, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structures and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

Although the site soils are noted to be medium dense to very dense, it is expected that the fire station structure would be mostly filled to grade such that the wall foundations and adjacent column foundations may be supported on both engineered fill and coarse native soils. This report recommends foundations be supported on similar materials, not a mix of native and engineered fill soils. Therefore, this report recommends that the footings for the proposed Fire Station building be supported on a minimum thickness of screened/processed engineered fill soils in order to limit total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet. The building is planned in a fill area, thus, the site should be prepared in accordance with the recommendations of this report prior to fill placement. Provided the recommendations of this report are followed for site preparation, a net allowable soil bearing pressure of 3,000 pounds per square foot, for dead-plus-live loads, may be used for design.

The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and weight of the footing may be neglected. The net allowable soil bearing pressure presented was selected using the Terzaghi bearing capacity equations for foundations considering a minimum factor of safety of 3.0 and based on the anticipated static settlements noted in this report.

A structural engineer experienced in foundation and slab-on-grade design should determine the thickness, reinforcement, design details and concrete specifications for the proposed building foundations and slabs-on-grade based on the anticipated settlements estimated in this report.

8.3.4 <u>Proposed Graded Slopes</u>: Although a grading plan was not available during this investigation, based on the elevation change across the building area, and considering that a drainage basin will be cut, it is expected that some graded slopes will be required to provide a flat building pad . It is expected that cut and fill slopes with a repose ranging from 2H:1V to 3H:1V, and heights of 10 feet or less will be required. Given the dense, granular nature, and the shear strength of the site soils, engineered fill slopes graded at 2H to 1V or flatter should remain stable. However, due to surficial erosion of cohesionless sands on exposed slopes at the water line of the basin localized slope instability could be experienced. Due to this condition, it is recommended the basin slopes be planned at not steeper than 3H:1V.

In general, the top of proposed slopes should be developed and maintained to rapidly drain surface and roof runoff away from cut or fill slopes - both during and after construction. To accomplish this, use brow ditches, berms or other measures to intercept and safely redirect flow. In addition, upslope drainage such as brow ditches / interceptor drains should be used to divert water away from graded slopes and to reduce erosion potential. Drainage should be directed into natural swales and energy dissipaters such as gravel or rip-rap to minimize erosion.

Graded slopes and areas of slopes which are disturbed during construction should be planted with ground cover vegetation or other methods to reduce erosion potential. Shallow rooted ground cover, as well as deeper rooted trees or bushes, should be planted on the disturbed or reconstructed portions of the slopes to reduce the potential for erosion and aid in surficial slope stability.

8.3.5 Percolation/Infiltration: The project will utilize percolation/infiltration in two areas: 1) an on-site waste water disposal system leach field to be located in the western and southwestern portions of the site; and 2) the storm water basin proposed in the southeast portion of the site. To provide design infiltration rates for these systems, eight percolation tests were conducted at various depths. The following evaluates these results for use in design.

Six (6) percolation tests were conducted in the area designated for the primary leach field and 100 percent expansion field. The depths of the tests ranged from 4 to 12 feet BSG, which should be within and below typical disposal trench depths estimated to be 4 to 7 feet deep. The native soil materials encountered throughout these depths was mostly gravel materials with varied silt and sand fractions in a dense to very dense condition. Also, cobbles and possible boulder sized material were noted based on the resistance to drilling with hollow stem augers.

Field testing indicated percolation rates in the leachfield area ranging from 14 to greater than 60 minutes per inch (slowest rate allowed for system design in the Riverside County LAMP). Furthermore, the depths of the slow percolation results ranged from 4 to 12 feet (across the range tested). These results suggest some areas/depths of the designated leach field may not be feasible for trench type disposal through infiltration. Typically, coarse grained soils would support higher infiltration rates due to porous nature of these strata. However the percolation rates were likely influenced by the dense to very dense condition of the soils which reduces porosity, and potentially natural cementation. It should also be noted that small diameter borehole tests such as percolation tests may not accurately represent the conditions within the larger absorption area of a trench/excavation, and thus testing conducted in a larger, excavated pit may provide a more accurate percolation rate to for use in system design. Thus, after preliminary design of the primary and secondary leach field, it is recommended to conduct supplemental percolation testing in test pits to confirm or update the recommended percolation rates to use for final design.

The two percolation tests in the proposed storm water basin were installed to depths of 10 and 15 feet below site grade, in a silty gravel material. The results of the percolation tests indicated estimated infiltration rates of 1.5 and 2 inches per hour. As noted above, these rates are less than expected, but the soils in the basin area appear to be more porous compared to the leach field. As such, the preliminary design of the basin can utilize an un-factored infiltration rate of 1.5 inches per hour. An appropriate factor of safety should be applied to this field result to account for reduction in the long term effects of subgrade saturation, silt accumulation, and vegetation.

8.3.6 Asphaltic Concrete (AC) Pavements: Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report for proposed asphaltic concrete (AC) pavements. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highway Design Manual. The analysis was based on traffic index values ranging from 5.0 to 8.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

It should be noted that if pavements are constructed prior to the construction of the buildings, the additional construction truck traffic should be considered in the selection of the traffic index value. If more frequent or heavier traffic (see fire apparatus discussion in the following section) is anticipated and higher Traffic Index values are needed, Moore Twining should be contacted to provide additional pavement section designs.

Based on the results of the R-value tests of 73 and 78, the procedures of the Caltrans Highway Design Manual and considering the extent of grading planned for the project, a design R-value of 50 was used to determine the pavement section thickness recommendations.

8.3.7 Portland Cement Concrete (PCC) Pavements: Recommendations for Portland cement concrete (PCC) pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the characteristics of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an a range of average dailty truck traffic (ADTT) of the expected fire engine apparatus and other heavy trucks. The recommendations provided in this report for PCC pavements are based on maximum truck axle loads and singe and tandem axle configurations for loading and the design procedures contained in the American Concrete Institute (ACI) 330R Guide for the Design and Construction of Concrete Parking Lots using the www.pavementdesigner.org application for parking project design.

The design assumes that all fire engine/apparatus vehicles will have axle loads not exceeding typical Department of Transportation (Caltrans) vehicle maximums of 18 kips for single axles and 36 kips for tandem axles. However, certain fire engine apparatus can receive a Caltrans exception to exceed these vehicle axle weight limits. Moore Twining should be notified to revise the pavement designs if fire apparatus exceeding (Caltrans) maximums of 18 kips for single axles and 36 kips for tandem axles are to be used at this facility.

The pavement sections were prepared based on traffic loadings expected for auto parking to the main fire apparatus driveway ranging from average daily truck traffic of 1 to 12 trucks per day, respectively.

8.3.8 Soil Corrosion: The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 2 below.

Table No. 2
Soil Resistivity and Corrosion Potential Ratings

Soil Resistivity (ohm cm)	Corrosion Potential Rating
>20,000	Essentially non-corrosive
10,000 - 20,000	Mildly corrosive
5,000 - 10,000	Moderately corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly corrosive
<1,000	Extremely corrosive

The results of soil sample analyses indicate that the near-surface soils exhibit a “moderately corrosive” corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the “moderately corrosive” corrosion potential. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

8.3.9 <u>Sulfate Attack of Concrete</u>: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil, groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with structural concrete is to perform laboratory testing to determine the concentrations of sulfates present in the soils. The test results are then compared with the exposure classes in Table 19.3.1.1 of ACI 318 to provide guidelines for concrete exposed to soils containing sulfates. It should be noted that other exposure conditions such as the presence of seawater, groundwater with elevated concentrations of dissolved sulfates, or materials other than soils can result in sulfate exposure categories to concrete that are higher than the concentrations of sulfate in soil. The design engineer will need to determine whether other potential sources of sulfate exposure need to be considered other than exposure to sulfates in soil. The sulfate exposure classes for soils from Table 19.3.1.1 are summarized in the below table.

Table No. 3
ACI Exposure Categories for Water Soluble Sulfate in Soils

Sulfate Exposure Class (per ACI 318)	Water Soluble Sulfate in Soil (Percent by Mass)
S0	Less than 0.10 Percent
S1	0.10 to Less than 0.20 Percent
S2	0.20 to Less than or Equal to 2.00 Percent
S3	Greater than 2.00 Percent

Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to, the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The laboratory test results for sulfates are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

9.0 CONCLUSIONS

Based on the data collected during the field exploration and laboratory testing programs, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following general conclusions are presented.

9.1 The site is considered suitable for support of the proposed improvements provided the recommendations contained in this report from a geotechnical engineering standpoint. However, as noted in this report, the project site is located in a State of California surface fault rupture hazard zone (Alquist-Priolo Earthquake Fault Hazard Zone) associated with mapped fault traces along the active San Gorgonio Pass fault zone. Refer to Drawing No. 5 in Appendix A, which illustrates the limits of the fault rupture hazard zone in relation to the site location. Accordingly, there is a potential for fault rupture to occur at the site, which would be generally considered moderate to high. It was not the intent of this investigation to conduct fault trenching to evaluate potential surface fault rupture hazards. Due to the proximity of mapped active faults to the site and considering that the site is located in a State of California mapped fault rupture hazard zone, a surface fault rupture hazard investigation is needed to evaluate potential impacts associated with active faulting.

9.2 The near surface soils generally consisted of medium dense poorly graded and well graded gravel with varied sand and silt fractions. The medium dense condition was found to extend to a depth of about 1 to 1½ feet below the surface in each boring tested at the surface. Below this upper material, the gravel materials were found to be dense to very dense to the maximum depth explored, about 29 feet BSG. The soils throughout the depths explored contained larger cobble materials (greater than 3 inches) and some boulders (greater that 1 foot in size).

9.3 The near surface gravel materials exhibited a "very low" expansion potential, good shear strength and excellent support characteristics for pavements when compacted as engineered fill.

9.4 Due to the presence of larger cobble/boulder material, and the cut/fill conditions anticipated for the building pad, over-excavation and compaction of the upper 1.5 feet of the near surface soils and placement of a minimum of 2 feet of fill below the bottom of the foundations is recommended in the building pad area to reduce potential impacts with differential static settlement. When the building subgrade soils are prepared as recommended in this report, total and differential static settlements for the proposed structures are estimated to be 1 inch and ½ inch in 40 feet, respectively.

9.5 Due to the coarse gravel, cobble and boulder content anticipated within the onsite soils, it should be expected that significant amounts of the soils excavated will need to be processed in order to be used as engineered fill below the building pad, and as backfill in the pipe zone on installed utilities. Thus, contractors should expect that soils will require equipment to process excavated materials to remove oversize gravel, cobbles and boulders through screening or crushing such that the materials retained on the 3/4-inch sieve are 30 percent or less prior to reuse as engineered fill. In addition, rock greater than 4 inches in the largest dimension should not be used within engineered fill soils.

9.6 Groundwater was not encountered in the test borings drilled at the time of our March 2023 field exploration to the maximum depth explored, about 29 feet BSG. Based on our review of water well data on the Department of Water Resources website, groundwater is anticipated to be deeper than 500 feet.

9.7 The results of soil sample analyses indicate that the near-surface soils exhibit a "moderately corrosive" corrosion potential to buried metal objects. Chemical analyses indicated a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils. The soils exhibit a Sulfate Exposure Class (per ACI 318) S0 based on a water soluble sulfate in soil of less than 0.1 percent by mass.

9.8 The percolation tests conducted in the primary leach field and 100 percent expansion field indicated percolation rates ranging from 14 minutes per inch to more than 60 minutes per inch (slowest rate allowed for system design in the Riverside County LAMP). These results suggest some areas/depths of the designated leach field may not be feasible for trench type disposal through infiltration. However, the results of these tests conducted in small borehole tests for soils classified as well graded gravel, cobbles, and boulders were in a very dense condition and may not accurately represent the infiltration rate of a larger trench sidewall. Therefore, it is recommended to conduct supplemental percolation testing in larger area test pits to confirm or update the percolation testing rates to use for final leach field design.

9.9 The percolation tests conducted in the proposed storm water basin indicate infiltration rates of 1.5 and 2 inches per hour at 10 and 15 feet BSG. This report recommends that the lowest unfactored infiltration rate of 1.5 inches per hour from be used for preliminary design, with appropriate safety factors.

10.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, we present the following recommendations for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and observation of clearing, demolition activities and earthwork operations by Moore Twining are integral to the proper application of the recommendations.

Where the requirements of a governing agency or utility agency differ from the recommendations of this report, the more stringent recommendations should be applied to the project.

10.1 General

10.1.1 Plans were not available at the time this report was prepared. Moore Twining should be provided the opportunity to review the final grading and foundation plans before the plans are released for bidding purposes so that any relevant recommendations can be presented. If proposed foundation loading or the planned construction is different from that described in the Anticipated Construction section of this report, the recommendations in this report may not be appropriate. Moore Twining should be notified and requested to provide supplemental recommendations for the proposed construction if changes are planned.

10.1.2 A surface fault rupture hazard investigation is needed to evaluate potential impacts associated with active faulting. The project design and development should be conducted in accordance with the recommendations of all future fault rupture hazard investigations.

10.1.3 The onsite soils contain significant fractions of coarse gravel, cobbles and boulders that will need to be processed to be used as engineered fill. Contractors should expect that soils will require equipment to process excavated materials to remove oversize gravel, cobbles and boulders through screening or crushing to meet the gradation requirements in the Engineered Fill recommendations Section 10.5 of this report. Although this report provides logs of auger borings and gradations of soil samples collected, such information is likely not sufficient to estimate the actual fraction of oversized material, nor strength and deposition of the boulder and cobble material to allow contractors to anticipate excavation and processing means and methods. In order to provide additional information for use in estimating the processing requirements (i.e., crushing, screening), contractors, as part of the bid process, should conduct subsurface exploration (such as backhoe pits) to observe the amount and an evaluate an effective methods to both excavate and process soils to meet the particle-size recommendations for engineered fill of this report.

10.1.4 The Contractor(s) bidding on this project should determine if the information included in the construction documents are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect information as required to prepare accurate bids.

10.2 Site Grading and Drainage

10.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least ten feet away from the structures, or as necessary to preclude ponding of water adjacent to foundations, whichever is more stringent. Adjacent exterior grades which are paved should be sloped at least 1 percent away from the foundations.

10.2.2 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. Trees should be setback from the proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

10.2.3 Landscaping after construction should direct rainfall and irrigation runoff away from the structures and should establish positive drainage of water away from the structures. Care should be taken to maintain a leak-free sprinkler system.

10.2.4 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the aggregate base soils and reducing the life of the pavements.

10.2.5 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with low water requirements are recommended.

10.2.6 Rain gutters and roof drains should be provided, and connected directly to the site storm drain system. As an alternative, the roof drains should extend a minimum of 5 feet away from the structures and the resulting runoff directed away from the structures at a minimum of 2 percent.

10.2.7 In the event subsurface storm water infiltration systems are planned near structures or below pavements, the proposed locations and details of these features should be provided to Moore Twining for review and comment. If these types of features are required, sufficient setbacks to existing improvements should be maintained, and/or specific measures such as deepened curbs, cutoffs, liners, etc. should be incorporated in the designs to reduce the potential for excessive settlement of improvements due to moisture and freewater migration from storm water disposal systems.

10.3 Cut/Fill Slope Gradients, Building-to-Slope Setbacks, and Slope Drainage

10.3.1 In general, cut and fill slopes up to 10 feet in height may be graded at a repose of 2H to 1V; however, slopes associated with the storm water basin and slopes near to structures and improvements which are sensitive to movement should be graded at a 3H:1V repose to reduce impacts due to erosion potential and surficial slope movement. Refer to Section 10.12 for additional information regarding slopes at the storm water basin area.

10.3.2 Due to the presence of cobbles and boulders, there will be a potential for downslope movement of oversized rock from the face of graded slopes, primarily cut slopes. The current development plans for structures and improvements below cut slopes have not been developed. Future plans for development of the slopes should incorporate appropriate provisions to reduce potential impacts from this potential hazard, such as constructing fences at the base of cut slopes to retain oversized rock and allow for removal of these materials over time. Maintenance plans should be developed to periodically observe the site slopes and remove rocks to reduce potential impacts.

10.3.3 Structures, foundations and improvements should be setback from native, cut and fill slopes with a repose of 5H:1V or steeper to provide adequate foundation support and protection against erosion. Greater setbacks may be required for drainage design purposes. For slopes up to 10 feet high structures, foundations and improvements above the top of a descending native, cut or fill slope should be setback a minimum distance from the top of the slope equal to one-third of the height (H/3) of the slope, and not less than 5 feet (measured to the face of the slope), whichever is the most stringent. The minimum structural setback from the structures to the toe of an ascending slope should be 5 feet or ½ the slope height (H/2), whichever is greater.

10.3.4 Setbacks should be designed anticipating that some slope erosion will occur and that sediment will have to be removed periodically from the base of the slope. A higher frequency of slope maintenance should be expected for the first few seasons after slope grading.

10.3.5 Improvements such as pavements, sidewalks, flatwork, etc. constructed adjacent to descending slopes or within the setback zone would have an increased potential for damage due to slope movement or erosion. Therefore, at a minimum, improvements such as these are recommended to be setback a distance of at least one-half the above setback recommended for building foundations.

10.3.6 Develop and maintain site grades which will rapidly drain runoff away from cut or fill slopes - both during and after construction. To accomplish this, use brow ditches, berms or other measures to intercept and safely redirect flow.

In addition, upslope drainage such as lined brow ditches should be used to divert water away from graded slopes and to reduce erosion potential. Drainage should be directed into natural swales and energy dissipaters such as gravel or rip-rap should be used to minimize erosion.

10.3.7 Graded slopes should be planted with ground cover vegetation or other methods to reduce erosion potential. Deeper rooted trees or bushes, should be planted on the disturbed or reconstructed portions of the slopes to reduce the potential for erosion and aid in surficial slope stability.

10.3.8 Irrigation in the areas of manufactured slopes should be of a drip type system without surface runoff.

10.3.9 Irrigation lines between the structures and on slopes should not be pressurized when not in use (i.e., main supply lines). All irrigation lines and sprinklers should be monitored for leaks. All leaks and damage should be repaired promptly.

10.3.10It is recommended that the contractor be required to maintain the slopes and drainage facilities such as swales, gutters, and repair erosion damage for a minimum of one year from completion of the project, or until the surface erosion cover is fully established, whichever occurs later.

10.3.11 Where erosion or surficial slope movements occur, Moore Twining should be notified and requested to observe the conditions and provide recommendations for repair. There will be a higher potential for slope impacts until the erosion cover is fully established on slopes and all drainage facilities are in place.

10.4 <u>Site Preparation</u>

10.4.1 Stripping should be conducted in all areas of existing improvements to remove surface vegetation and root systems. The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. The actual depth of stripping should be reviewed by our firm at the time of construction. Deeper stripping may be required in localized areas. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner.

10.4.2 Where shrubs/trees are to be removed, all roots larger than ¼ inch in diameter and any accumulation of organic matter that will result in an organic content more than 3 percent by weight should be removed and not used as engineered fill. The bottom of the excavation should be scarified to a minimum depth of 8 inches and compacted as engineered fill prior to backfilling operations. Moore Twining should be contacted to observe removal of the bushes and roots.

10.4.3 **Building Pad Area:** After site stripping, removal of root systems and removal of existing surface and subsurface improvements (if any), prior to placement of fill to achieve the finished grades for these structures, the area of the proposed building pad should be over-excavated to at least 18 inches below preconstruction site grades, to the depth to remove any unsuitable, loose, disturbed or fill soils, and to at least 2 feet below the bottom of proposed foundations, whichever is greater. The over-excavation should include the entire footprint of the structures, including all foundations, a minimum of 5 feet beyond the foundations, a horizontal distance beyond the foundations equal to the depth of proposed fill, and a minimum of 3 feet beyond all perimeter concrete slabs directly adjacent to the buildings such as walkways, patios, cleaning slabs, etc., whichever is greater. The bottom of the excavation should be scarified 8 inches in depth, oversized rock should be removed, the soils should be uniformly moisture conditioned to within optimum to three (3) percent above optimum moisture content and compacted as engineered fill.

10.4.4 The grading plans should show the limits of over-excavation for the building pad as described above in sections 10.4.3.

10.4.5 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for measuring and verifying the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). It is recommended that this verification be performed by a licensed surveyor. This verification should be provided prior to requesting pad certification from Moore Twining or excavating for foundations.

10.4.6 **Areas to Receive Fill, Pavements and Exterior Slabs-on Grade Outside the Building Pad Limits:** Following stripping and removal of surface and subsurface improvements, areas to receive fill outside the building pad over-excavation limits described (refer to Section 10.4.3 of this report), pavement areas, and exterior slab-on-grade areas (not directly attached to buildings) should be prepared by over-excavation to the depth required to remove any unsuitable, loose, disturbed or fill soils, whichever is greater. The bottom of the over-excavation should be scarified to a minimum depth of 12 inches, oversized material should be removed, the soils should be moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted as engineered fill. The upper 12 inches of subgrade beneath the pavement areas should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

10.4.7 **Areas to Receive Miscellaneous Lightly Loaded Foundations:** Following stripping and removal of existing surface and subsurface improvements, areas to receive miscellaneous lightly (less than 1 kip per foot) loaded foundations such as site walls, trash enclosure walls and retaining walls, should be over-excavated to the bottom of foundations; to at least 12 inches below preconstruction site grades; to the depth required to remove any unsuitable, loose, disturbed or fill soils; and to at least 12 inches below subsurface improvements (structures, utilities, etc.) to be removed, whichever is greater. The over-excavation should extend to at least 3 feet beyond the edge of the foundations. If site walls are planned along property lines and over-excavation cannot extend beyond the property line, then the over-excavation should extend up to the property line. The bottom of the over-excavation should be scarified to a depth of at least 8 inches, oversized material should be removed, the soils should be moisture conditioned and compacted as engineered fill.

10.4.8 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

10.4.9 The contractor should locate all on-site water wells (if any). All wells scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Moore Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters) should have their casings removed to a depth of at least 8 feet below preconstruction site grades or finished pad grades, whichever is deeper. In parking lot or landscape areas, the casings should be removed to a depth of at least 5 feet below site grades or finished grades. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill.

10.4.10 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

10.4.11 Final grading shall produce building pads ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. The finished pad (before aggregate base is placed) shall not depress more than one-half (½) inch under the wheels of a fully loaded water truck, or equivalent loading. If depressions more than one-half (½) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the owner.

10.4.12 The Contractor should be responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. (if any) that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor should be responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal.

10.5 Engineered Fill

10.5.1 The on-site near surface soils encountered are predominantly gravel with minor fractions of cobbles, boulders, silts and sands. Due to the coarse gravel, cobbles and boulders, the onsite soils are anticipated to require processing such as by screening, and/or crushing and blending in order to meet the particle size requirements for engineered fill for this project. Thus, the on-site near surface soils should be processed to meet the specified gradation for engineered fill, or imported fill meeting the recommendations of this report should be used. The onsite soils may be used as engineered fill below the recommended aggregate base section, provided the onsite soils are free of organics (less than 3 percent by weight), free of debris and can be processed to meet the below specified gradation. Processing of the onsite materials by rock removal, screening, crushing and blending or other acceptable methods should be anticipated to achieve the following recommended gradations for engineered fill:

Table No. 4
Allowable Particle Size Requirement for On-Site Processed Soils

Sieve Size	Percent Material Passing Required
4 inch sieve	100
3/4 inch sieve	70-100
No. 4 sieve	40 - 100
No. 200 sieve	10 - 40

10.5.2 Oversized material may be placed as fill in non-structural areas beyond building, pavement and fill slopes which require engineered fill, or used a rip/rap for erosion slope protection or landscape cover at the discretion of the design team and owner.

10.5.3 The interior building slab on grade and exterior slabs should be supported on a minimum of 4 inches of aggregate base, over subgrade soils prepared as recommended in the Site Preparation section of this report.

10.5.4 The compactability of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, it is recommended that they be evaluated by the contractor during preparation of bids and construction of the project.

10.5.5 Import fill soil (if any) should be non-recycled, non-expansive and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing 3/4 Inch Sieve	75-100
Percent Passing No. 4 Sieve	40 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 15
Organics	Less than 3 percent by weight
R-Value	Minimum 50*
Sulfates	< 0.05 percent by weight
Min. Resistivity	> 5,000 ohms-cm

* for pavement areas only

10.5.6 Prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. In addition, Moore Twining should be requested to sample and test the material to determine compliance with the above geotechnical criteria. Contractors should provide a minimum of 7 working days to complete the testing

10.5.7 Processed native soils and imported engineered fill soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum moisture content and three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that fills placed at a depth of greater than 10 feet below finished grade and the upper 12 inches of the pavement subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

10.5.8 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at a frequency of at least:

Table No. 5
Minimum In-Place Density Testing Frequency

Area	Minimum Test Frequency
Building Pad	1 test per 5,000 square feet per compacted lift, but not less than two tests per building pad per lift
Fill Slopes	1 test per 100 lineal feet of fill slope per compacted lift
Pavement Subgrade and Mass Grading Outside Building Pads	1 test per 10,000 square feet per compacted lift
Utility Lines	1 test per 150 feet per lift

10.5.9 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Crushed rock should be placed in thin (less than 8 inch) lifts and densified with a minimum of three (3) passes using a vibratory compactor.

10.5.10 Aggregate base below the building slabs should comply with State of California Department of Transportation requirements for a non-recycled Class 2 aggregate base or Crushed Aggregate Base (CAB) from the Standard Specifications for Public Works Construction. Alternatively, Crushed Miscellaneous Base (CMB), or a recycled Class 2 aggregate base, may be used for pavement areas outside the building and overbuild zones, provided that the recycled materials are accepted by the Owner and adequate quality control testing is conducted. Aggregate base should be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all the quality requirements (i.e., gradation, R-value, sand equivalent, durability, etc.) for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

10.6 Shallow Spread Foundations

10.6.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on the estimated settlements. The following settlements should be anticipated for design a total static settlement of 1 inch and a differential static settlement of ½-inch in 40 feet.

10.6.2 Foundations supported on subgrade soils prepared as recommended in the Site Preparation section of this report may be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

10.6.3 Perimeter foundations should have a minimum depth of 18 inches below the top of the slab and a minimum depth of 12 inches below the lowest adjacent finished exterior ground surface, whichever is greater. All footings should have a minimum width of 15 inches, regardless of load.

10.6.4 The foundations should be continuous around the perimeter of the structures to reduce moisture migration beneath the structures. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

10.6.5 The following seismic factors were developed for the site using the Ground Motion Parameter Calculator available from SEAOC and OSHPD (http://seismicmaps.org) in accordance with the 2019 CBC, using a site latitude of 33.946657 degrees, and a longitude of -116.832373 degrees. The data provided in Table No. 6 are based upon the procedures of ASCE 7-16 and were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 6 and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

TABLE NO. 6

Seismic Factor	2019 CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	1.101g
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration (PGA)	1.001 g
Spectral Response At Short Period (0.2 Second), S_s	2.334
Spectral Response At 1-Second Period, S_1	0.989
Site Coefficient (based on Spectral Response At Short Period), F_a	1.0
Site Coefficient (based on spectral response at 1-second period) F_v	See Note
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	2.334

TABLE NO. 6

Seismic Factor	2019 CBC Value
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	See Note
Five percent damped design spectral response accelerations for short period, S_{DS}	1.556
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	See Note

*Note: Requires ground motion hazard analysis per ASCE Section 21.2 (ASCE 7-16, Section 11.4.8), unless an Exception of Section 11.4.8 of ASCE 7-16 is applicable for the building design.

10.6.6 Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement and concrete to verify conformance with the intent of the recommendations of this report. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of this observation prior to placement of steel reinforcement.

10.6.7 Structural loads for lightly (less than 1.5 kips per lineal foot) loaded miscellaneous foundations (such as screen walls) should be supported on subgrade soils prepared in accordance with the "Site Preparation" Section 10.4.7 of this report. Lightly loaded foundations may be supported by footings extending to a minimum depth of 12 inches below the lowest adjacent finished grade and a minimum width of 12 inches. These improvements may be designed for a maximum allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads for footings. This value may be increased by one-third for short duration wind or seismic loads.

10.6.8 Site lighting and pylon signs (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 250 pounds per square foot may be used to resist axial loads. The allowable passive resistance of the native soils may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per square foot per foot of depth to a maximum of 3,500 pounds per square foot. The passive pressure may be assumed to act over twice the pier diameter. The passive resistance of the surface soils to a depth of 12 inches, or to the depth where the horizontal setback from the foundation to a descending slope is less than 3 feet, whichever is greater, should be neglected.

10.6.9 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.40 can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10 can be used for design.

10.6.10The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot. The upper 6 inches of subgrade in landscaped areas should be neglected in determining the total passive resistance.

10.7 Interior Slabs-on-Grade

The slabs on the project that should be prepared as interior slabs include: the interior floor slab and all concrete slabs on grade directly adjacent to the buildings.

10.7.1 Interior slabs-on-grade should be constructed over 4 inches of non-recycled aggregate base over engineered fill placed for the building pad preparation in accordance with the Site Preparation section 10.4.3 of this report.

10.7.2 The recommendations provided herein are intended only for the design of interior concrete slabs-on-grade and their proposed uses, which do not include construction traffic (i.e., cranes, cement mixers, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

10.7.3 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

10.7.4 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarder when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The vapor retarder should overlay the compacted aggregate base. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the aggregate base and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier selection and installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

10.7.5 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.

10.7.6 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacture's recommendations.

10.7.7 The moisture retarding membrane is not required beneath exposed concrete floors, such as warehouses and garages, provided that moisture intrusion into the structures are permissible for the design life of the structures.

10.7.8 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.52 or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structures, 4) providing adequate drainage away from the structures, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structures.

10.7.9 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity, etc., at a frequency and method as specified by the flooring manufacturer or as required by the plans and specifications, whichever is most stringent. The results of vapor transmission tests, pH tests, internal relative humidity tests, ambient building conditions, etc. should be within floor manufacturer's and adhesive manufacturer's specifications at the time the floor is placed. It is recommended that the floor manufacturer and subcontractor review and approve the test data prior to floor covering installation.

10.7.10 To reduce the potential for damaging slabs during construction the following recommendations are presented: 1) design for a differential slab movement of ½ inch relative to interior columns; and 2) the construction equipment which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

10.7.11 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

10.8 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc. outside the overbuild zone.

10.8.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.

10.8.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Preparation" section of this report. Upon completion of the over-excavation and compaction of subgrade soils, the exterior slabs should be supported on 4 inches of aggregate base overlying subgrade soils prepared in accordance with the recommendations provided in the "Site Preparation" section of this report. The recommended 4 inch aggregate base layer may be omitted if a higher risk of shrinkage cracking of exterior slabs-on-grade is acceptable to the owner.

10.8.3 The moisture content of the subgrade soils should be verified to be slightly above optimum moisture content within 48 hours of placement of the slab-on-grade. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

10.8.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to at least a depth of 6 inches below the bottom of the slabs-on-grade.

10.8.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

10.9 Asphaltic Concrete (AC) Pavements

Recommendations are provided below for new asphaltic concrete pavements planned as part of the new construction.

10.9.1 The subgrade soils for asphaltic concrete pavements should be over-excavated and compacted as recommended in the "Site Preparation" section of the recommendations in this report.

10.9.2 The following pavement sections are based on an R-value of 50 and traffic index values ranging from 5.0 to 8.0 and a minimum aggregate base thickness of 4 inches. It should be noted that if pavements are constructed prior to construction, the traffic index value should account for construction traffic.

The actual traffic index values applicable to the site should be determined by the project civil engineer.

Table No.7
Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	2.5	4.0	12
5.5	3.0	4.0	12
6.0	3.0	4.0	12
6.5	3.5	4.5	12
7.0	4.0	4.5	12
8.0	5.0	5.0	12

AC - Asphaltic Concrete compacted as recommended in this report
AB - Class II Aggregate Base, Crushed Aggregate Base (CAB), or Crushed Miscellaneous Base (CMB) with minimum R-value of 78 and compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D1557)

10.9.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should extend at least to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

10.9.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.

10.9.5 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

10.9.6 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

10.9.7 Pavement materials and construction method should conform to the State of California Standard Specifications.

10.9.8 It is recommended that the base 2 inch thick course of asphaltic concrete consist of a ¾ inch maximum medium gradation. The top course or wear course should consist of a ½ inch maximum medium gradation.

10.9.9 The asphaltic concrete, including the joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to ASTM D2041.

10.9.10 The asphalt concrete should comply with the requirements for a Type A asphalt concrete in accordance with the current State of California Department of Transportation (Caltrans) Standard Specification, or the requirements of the governing agency, whichever is more stringent.

10.10 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural section thicknesses are presented in the following subsections. The PCC pavement design thickness assumes a minimum modulus of rupture of 500 psi. The design professional should specify where Portland cement concrete pavements are used based on the anticipated type and frequency of traffic.

10.10.1The subgrade soils for Portland cement concrete pavements should be over-excavated and compacted as recommended in the "Site Preparation" section of the recommendations in this report.

10.10.2 The following preliminary Portland cement concrete pavement sections have been prepared for traffic including auto parking to the main fire apparatus driveway ranging from average daily truck traffic estimated to be 1 to 12 trucks per day, respectively. The design pavement sections should be selected by the civil engineer based on the anticipated traffic loading. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic. The design thicknesses were prepared based on the procedures outlined in the American Concrete Institute (ACI) 330R Guide for the Design and Construction of Concrete Parking Lots assuming the following: 1) minimum modulus of rupture of 500 psi for the concrete, 2) a design life of 20 years, 3) load transfer by aggregate interlock or dowels, 4) concrete shoulder, 5) a reliability of 95% and percent of slabs cracked at the end of design life of 5%, 6) a five axle truck with one single axle load of 18 kips and two tandem axle loads of 36 kips each and 7) a 4 inch aggregate base section.

Table No. 8
Portland Cement Concrete Pavements

Traffic Index/ADTT	PCC thickness (inches)	Aggregate Base (inches)	Compacted Subgrade (inches)
Auto Parking 1 truck per day	5.5	4.0	12.0
12 trucks per day	6.0	4.0	12.0
50 trucks per day	6.5	4.0	12.0

ADTT - Average Daily Truck Traffic based on a loaded five axle truck
PCC - Portland Cement Concrete (minimum Modulus of Rupture=500 psi)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

10.10.3 The above pavement designs assumes that all fire engine/apparatus vehicles will have axle loads not exceeding typical Department of Transportation (Caltrans) vehicle maximums of 18 kips for single axles and 36 kips for tandem axles. However, certain fire engine apparatus can receive a Caltrans exception to exceed these vehicle axle weights. Moore Twining should be notified to revise the above pavement design thicknesses (or material strengths) if fire apparatus exceeding (Caltrans) maximums of 18 kips for single axles and 36 kips for tandem axles are to be used at this facility.

10.10.4 Jointing is one of the most critical aspects of the PCC pavement design and construction. Joint spacing, joint type and load transfer devices have significant impacts on the pavement design and performance. Thus, the detailing of joints needs to be considered carefully and applied with clear details on the project plans by the pavement designer/detailer. Guidelines for jointing within ACI 330R are recommended to be used for development of project details. Positive load transfer devices such as dowels are commonly used at contraction joints whenever the designer cannot be assured aggregate interlock will be maintained.

10.10.5 Specifications for the concrete mixtures used in the PCC pavement to reduce the effects of excessive shrinkage (such as curling and excessive shrinkage at joints), including maximum water requirements for the concrete mix, allowable shrinkage limits, curing methods, etc. should be provided by the designer/detailer of the PCC slabs. In addition, as noted in Section 10.9.3, contraction joint requirements should be detailed by the designer/detailer of the PCC pavement to maintain stability. The minimum PCC thickness noted in this report assumes aggregate interlock occurs at contraction joints. However, curling and excessive shrinkage can disengage aggregate interlock and allow greater pavement deflection at free edges.

10.10.6 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 500 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC slabs.

10.10.7 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels or load transfer devices are recommended for construction joints to transfer loads. The joint details should be detailed by the pavement design engineer and provided on the plans.

10.10.8 Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.

10.10.9 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

10.10.10Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

10.10.11 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.

10.10.12 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

10.10.13 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

10.10.14 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

10.11 On-Site Sewage Disposal Field

10.11.1 The percolation tests conducted in the primary leach field and 100% expansion field indicated some percolation rates were more than 60 minutes per inch, which is slowest rate allowed for use of an application rate of 1.1 gallons per square foot of sidewall per day for seepage trenches pits in the Riverside County LAMP. Since these tests were conducted in small boreholes, the rates may not be representative of the actual percolation rate of the dense to very dense gravel, cobbles, and boulders. Therefore, it is recommended supplemental percolation testing in larger area test pits to confirm or update percolation testing rates to use for design of the sewage disposal field. If excessively slow percolation rates are still applicable, alternative systems such as mounds could be a consideration.

10.11.2 The septic system design and construction should be conducted in accordance with the Riverside County LAMP Chapter 5 - Design Requirements for Conventional Onsite Wastewater Treatment Systems.

10.11.3 Since the soils within the primary and secondary leach field were known to include large cobbles and boulders which can restrict infiltration, it is recommended to remove all material larger than 12 inches from excavated trench or pit sidewalls; or increase the area to compensate for zones of larger material.

10.11.4 All excavated leach trench/pit sidewalls and bottoms should be inspected by Moore Twining to confirm the materials exposed are consistent with the material tested to determine the design percolation rate, and confirm cobbles/boulders are removed.

10.12 Proposed Storm Water Basin

10.12.1 To reduce the potential for erosion, shallow soil movements and related maintenance requirements for the basin side slopes, on a preliminary basis, a maximum repose of 3 Horizontal (H) to 1 Vertical (V) is recommended for the interior basin side slopes. Where steeper slopes are used, a higher potential for erosion and soil movements should be anticipated. Erosion control measures should be applied for all slopes within the basin.

10.12.2 The area above the top of the side slopes of the basin should be graded to prevent concentrated runoff from flowing over the top of the basin side slopes.

10.12.3 This report recommends that the lowest unfactored infiltration rate of 1.5 inches per hour be considered for the storm water basin. Since field percolation testing is a small-scale test method and does not take into account the long term effects of subgrade saturation, silt/fines accumulation, or mechanical densification of the soils as a result of the construction process, etc., a safety factor should be applied to the recommended infiltration rate for design of the basin. A safety factor ranging from 3 to 10 is generally recommended; however, the final safety factor should also meet the requirements of the governing agency and should consider such factors as the sediment load of the stormwater, whether pretreatment of stormwater is planned, the consequences of failure, the degree of maintenance that can be relied upon and the uncertainties in the estimated inflow volume.

10.12.4 During construction of the improvements to the stormwater basin, the excavation(s) should be observed by Moore Twining to confirm the soils exposed are consistent with those encountered and tested herein.

10.12.5 Construction/excavation of the basins should be conducted so as to limit the impacts from construction equipment that may reduce the permeability of the soils. Excavation work conducted near the base of the basin should therefore be conducted using lower pressure equipment, such as tracked equipment that limit artificial densification of the soils.

10.12.6 It is not recommended to use the basin for collection of stormwater during construction of the project to reduce impacts from sediment loads, which can clog the pore spaces in the soils and further reduce the infiltration capacity.

10.12.7 Discing or ripping and cross ripping of the bottom of the basin should be conducted just prior to completion of the basin to loosen the soils at the bottom.

10.12.8 It should be noted that gravel, cobbles and potentially boulders will be exposed on the sidewalls of the basin. If these materials creep and slide in the future, the sidewalls of the basin will require periodic maintenance and possibly stabilization.

10.12.9 In order to reduce the potential for erosion of the side slopes, erosion protection should be established and maintained on the slopes.

10.12.10 Our experience with infiltration basins is that they have a limited life span. Thus, regular maintenance should be expected to maximize the useful life of these facilities and future expansion or modification of these systems should be anticipated to maintain functionality. Periodic maintenance of the basin should be conducted which would be anticipated to include removal of debris, vegetation, siltation and fine particles (e.g. silts and organic matter) from the bottom of the basin. Lightweight equipment should be used to minimize compaction of the basin surfaces.

10.12.11Cutoffs should be included at inlet and outlet pipes/structures (if any) to reduce erosion and to reduce seepage from migrating along trenches.

10.13 Temporary Slopes and Excavations

10.13.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades, classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

10.13.2 Temporary excavations should be constructed in accordance with OSHA requirements. Temporary cut slopes up to 10 feet should not be steeper than 1.5:1, horizontal to vertical, and not steeper than 1H:1V for slopes greater than 10 feet high. If excavations cannot meet these criteria, the temporary excavations should be shored. In addition, temporary slopes will expose rock such as cobbles and boulders which may become dislodged from the face of excavations. These conditions will also need to be addressed to reduce impacts at the base of slopes such as by excavating flatter slopes, clearing "loose" rock from slope faces, or other measures.

10.13.3 In no case should excavations extend below a 2H to 1V zone below utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs.

10.13.4 Shoring should be designed by an engineer with experience in designing shoring systems and registered in the State of California.

10.13.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structures occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

10.14 Utility Trenches

10.14.1 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable, the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 92 percent relative compaction prior to placement of bedding material. The Contractor is responsible for notifying Moore Twining when these conditions occur and arrange for Moore Twining to observe and test these areas prior to placement of pipe bedding. The Contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The Contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of bedding, pipe and backfill of the trench.

10.14.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of

30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted to a minimum of 92 percent relative compaction. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

10.14.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in Table No. 9 (minimum manufacturer requirements), or as necessary to provide sufficient space to achieve the required compaction, whichever is greater. As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 9
Minimum Trench Widths for HDPE Pipe with
Sand Bedding Initial Backfill

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321
12	14.2	30
18	21.5	39
24	28.4	48

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321
36	41.4	64
48	55	80

10.14.4 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Crushed rock should be placed in thin (less than 8 inch) lifts and densified with a minimum of three (3) passes using a vibratory compactor.

10.14.5 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned to between optimum and three (3) percent above the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

10.14.6 On-site soils and approved imported engineered fill may be used as final backfill (12 inches above the pipe to the ground surface) in trenches

10.14.7 Jetting of trench backfill is not allowed to compact the backfill soils.

10.14.8 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

10.14.9 Storm drains and/or utility lines should be designed to be “watertight.” If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil movement causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. The Contractor is required to video inspect or pressure test the wet utilities prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are "watertight." The Contractor shall provide the Owner a copy of the results of the testing. The Contractor is required to repair all noted deficiencies at no cost to the owner.

10.14.10The plans should note that all utility trenches, including electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 92 percent per ASTM D-1557 except for the upper 12 inches below pavements which should be compacted to at least 95 percent relative compaction.

10.14.11Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of building foundations.

10.15 <u>Corrosion Protection</u>

10.15.1Based on the resistivity values and the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in Table No. 3 of this report, the analytical results of sample analyses indicate the soils had a resistivity value of 9,100 ohm-centimeter, with a pH value of 7.1. Based on the resistivity value, the soils exhibit an "moderately corrosive" corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "moderately corrosive" corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

10.15.2 Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318, the sulfate concentration from chemical testing of soil samples falls in the S0 classification (less than 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or strength of the concrete used for foundations and slabs due to the sulfate content.

10.15.3 We recommend that these soil corrosion data be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturer's or supplier's cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

11.0 DESIGN CONSULTATION

11.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

11.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

11.3 If Moore Twining is not retained for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

12.0 CONSTRUCTION MONITORING

12.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

12.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

12.3 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

12.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

12.5 If Moore Twining is not retained to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance that is satisfactory to the Owner and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

12.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

13.0 NOTIFICATION AND LIMITATIONS

13.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of subsurface variations between borings may not become evident until construction.

13.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

13.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

13.4 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

13.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for other structures or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

13.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

13.7 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

13.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

13.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to the DLR Group. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Respectfully Submitted,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

DRAFT

Scott W. Krauter RGE
Assistant Manager

DRAFT

Allen H. Harker, CEG
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Geotechnical Engineering Division

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Read L. Andersen, RGE
Manager

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State of California Department of Water Resources Division of Safety of Dams - Dam Breach Inundation Map Web Viewer application https://fmds.water.ca.gov/webgis/?appid=dam_prototype_v2

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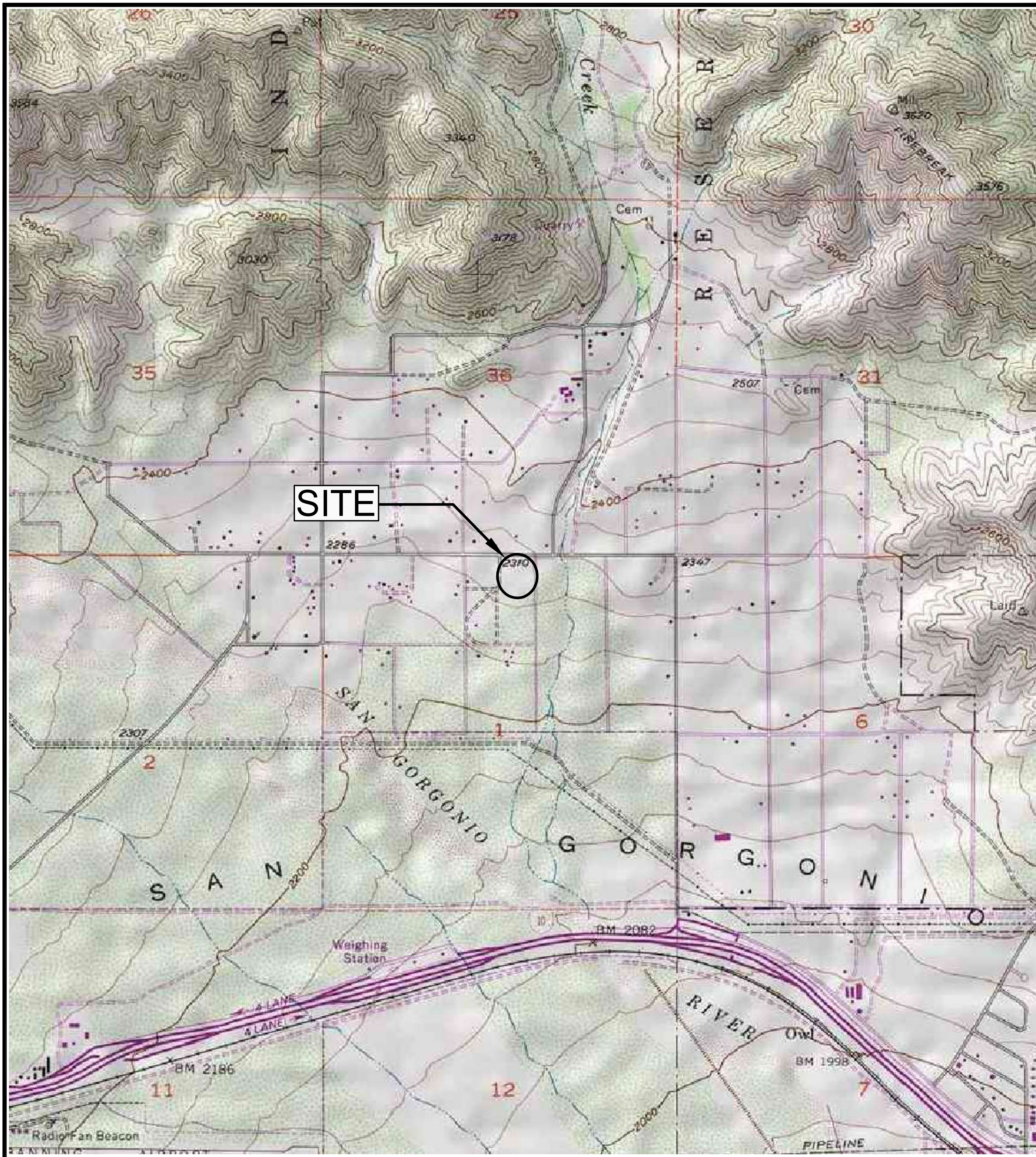
APPENDIX A

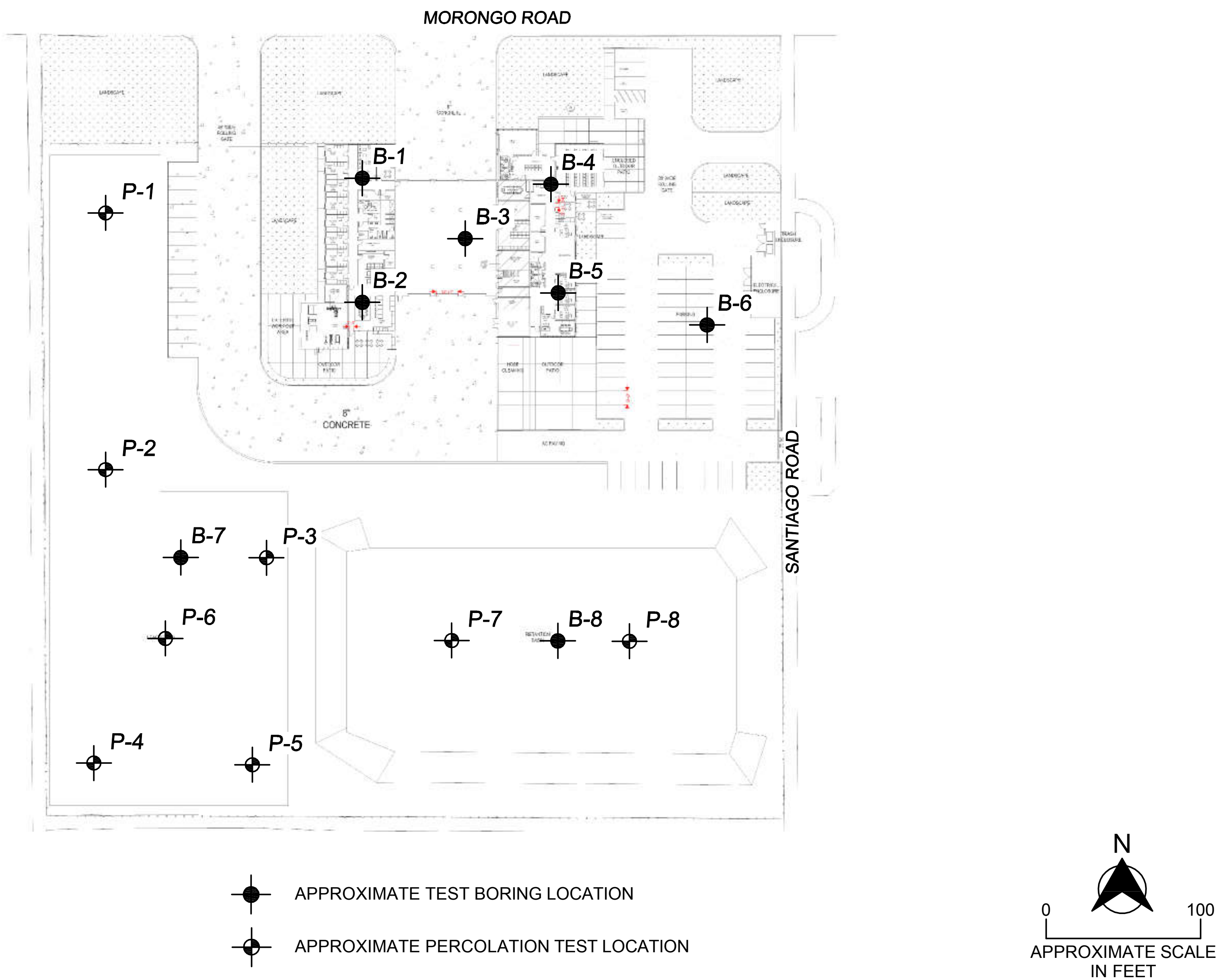
DRAWINGS



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
CABAZON, CALIFORNIA QUADRANGLE 1996

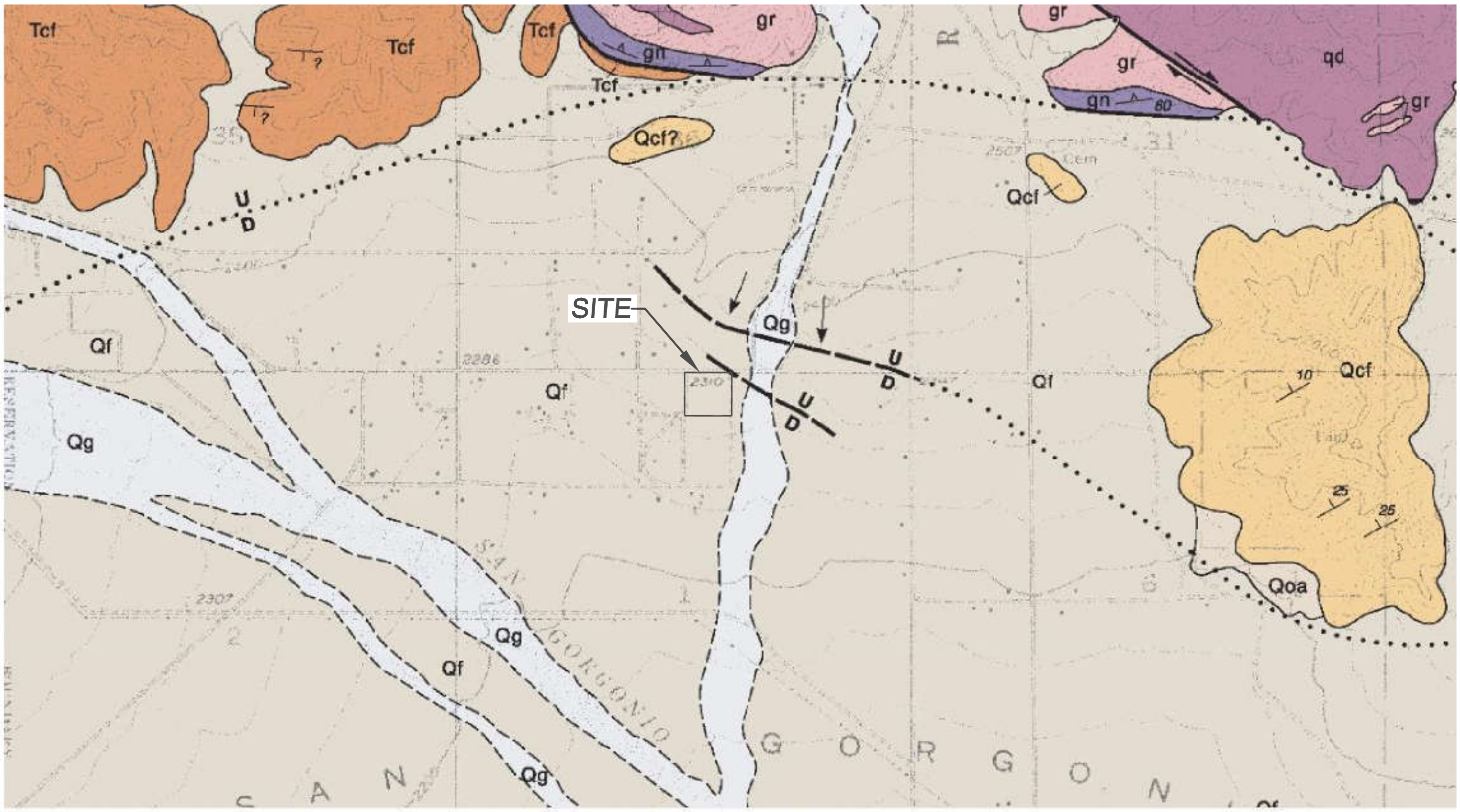
0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO.: 17401-01-01	DATE: 04/27/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H17401.01	DRAWING NO. 1	



TEST BORING LOCATION MAP PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/27/2023
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. H17401.01	DRAWING NO. 2

MOORE TWINING ASSOCIATES, INC.



CABAZON MAP (DF-119)

LEGEND

QUATERNARY

Holocene

Qa | Qg

SURFICIAL SEDIMENTS

Unindurated, undissected alluvial sediments

Qg *Alluvial gravel and sand of stream channels*

Qa *Alluvial sand and gravel of flat flood plains and small valleys mostly near and in San Jacinto Mountains*

— UNCONFORMITY —

?

Pleistocene

Qls

LANDSLIDE DEBRIS

Qls *Gravel rubble in* **Tsf**

Qoa | Qf | Qof

OLDER SURFICIAL SEDIMENTS

Slightly indurated, partly dissected alluvial sediments

Qf *Alluvial fan of San Gorgonio Pass, sand and gravel of plutonic and gneissic detritus derived from rising San Bernardino Mountains to north; slightly dissected by stream channels; includes small alluvial fans at base of and derived from San Jacinto Mountains in south area*

Qoa *Alluvial fans of gneissic rubble in NW area, much dissected*

Qof *Alluvial fan of quartz diorite detritus as small erosional remnants in east central area*

TERTIARY

?

Miocene

Ti

IMPERIAL FORMATION

(of Allen, 1954, 1957, Dibblee, 1954, 1981a) shallow marine sediments of Gulf of California, moderately lithified; age late Miocene or early Pliocene

Ti *Light gray claystone, weathered tan, and sandstone, tan to rusty brown, semi-friable, arkosic; contains shallow marine molluscan shell fragments; exposed only at east border of quadrangle in an upended section where it is about 200m (700 ft.) thick; conformable between Palm Spring and Coachella Formations; not present in areas to the west*

Tb | Tcf | Tcs

COACHELLA FANGLOMERATE

(of Allen, 1954, Dibblee, 1981 a, b) included in Hathaway Formation of Allen 1954 in this quadrangle; stream laid sedimentary deposits, including a thin basalt unit at top, moderately lithified; age probably upper Miocene

Tb *Basalt, black olivine bearing, locally vesicular, about 50m (150 ft.) thick*

Tcf *Fanglomerate, gray-brown, massive to crudely bedded, of unsorted detritus of plutonic and gneissic rocks derived from San Bernardino Mountains*

Tcs *Sandstone, lithified, light gray, arkosic, and interbedded cobble conglomerate and some silty greenish to reddish claystone*

— UNCONFORMITY —

CRETACEOUS

gr | qd

PLUTONIC ROCKS

Medium grained holocrystalline rocks of Transverse Ranges; age, Cretaceous

gr *Leucocratic granitic rocks of mostly quartz monzonite composition, of quartz potassic feldspar, and sodic plagioclase feldspar in nearly equal amounts, and small amounts of biotite in small flakes; light tan, massive, intrusive into all other crystalline rocks, complexly in some areas*

qd *Quartz diorite, ranges to diorite, same as Wilson Diorite of Miller 1934, in San Gabriel Mountains; composed of 1/3 or less of quartz, 2/3 of sodic plagioclase feldspar, little or no potassic feldspar and about 10-15% biotite and hornblende; gray, massive to faintly gneissoid, moderately coherent to incoherent; includes small engulfed remnants of gneiss, many too small to map, only few larger remnants mapped*

gn

GNEISS

Severely metamorphosed rocks, medium grained, holocrystalline from sedimentary and igneous (?) petrolith of Precambrian (?) age

gn *Gneiss, gray, composed of white and light gray quartz-feldspar rich laminae alternating with dark gray biotite-rich laminae, with somewhat wavy or undulent structure in many places; commonly migmatized with quartz diorite* (**qd**)*, in places includes hornblende-biotite gneiss, and augen gneiss with large augen of potassic feldspar (orthoclase) as described by Allen, 1954; rock hard but brittle and closely fractured*

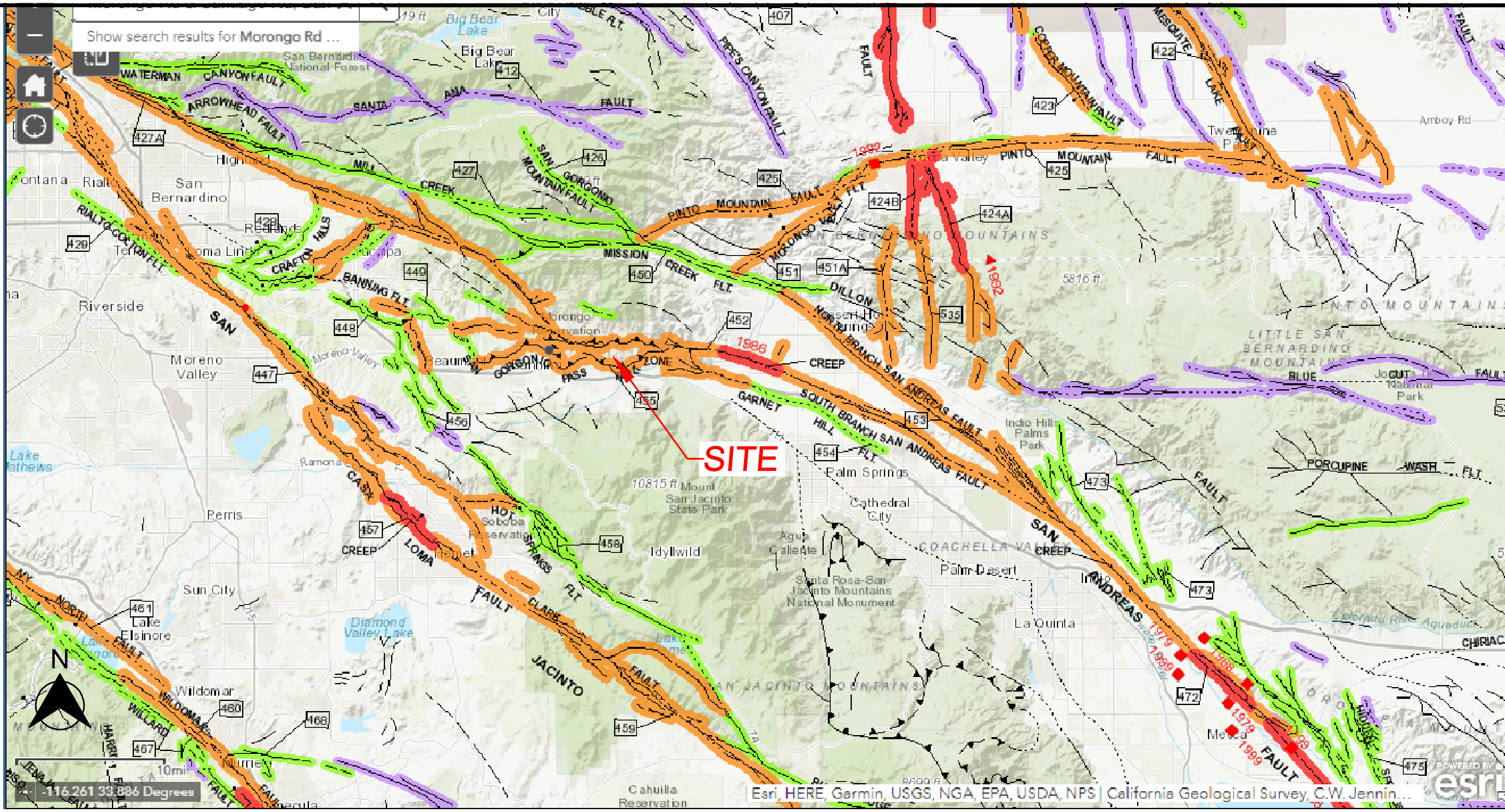
SOURCE: GEOLOGIC MAP OF THE CABAZON QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA BY THOMAS W. DIBBLEE, JR., 2004

N

0 600

APPROXIMATE SCALE IN FEET

REGIONAL GEOLOGIC MAP PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/28/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H17401.01	DRAWING NO. 3	



LEGEND

Fault along which historic (last 200 years) displacement has occurred

Holocene fault displacement (during past 11,700 years) without historic record

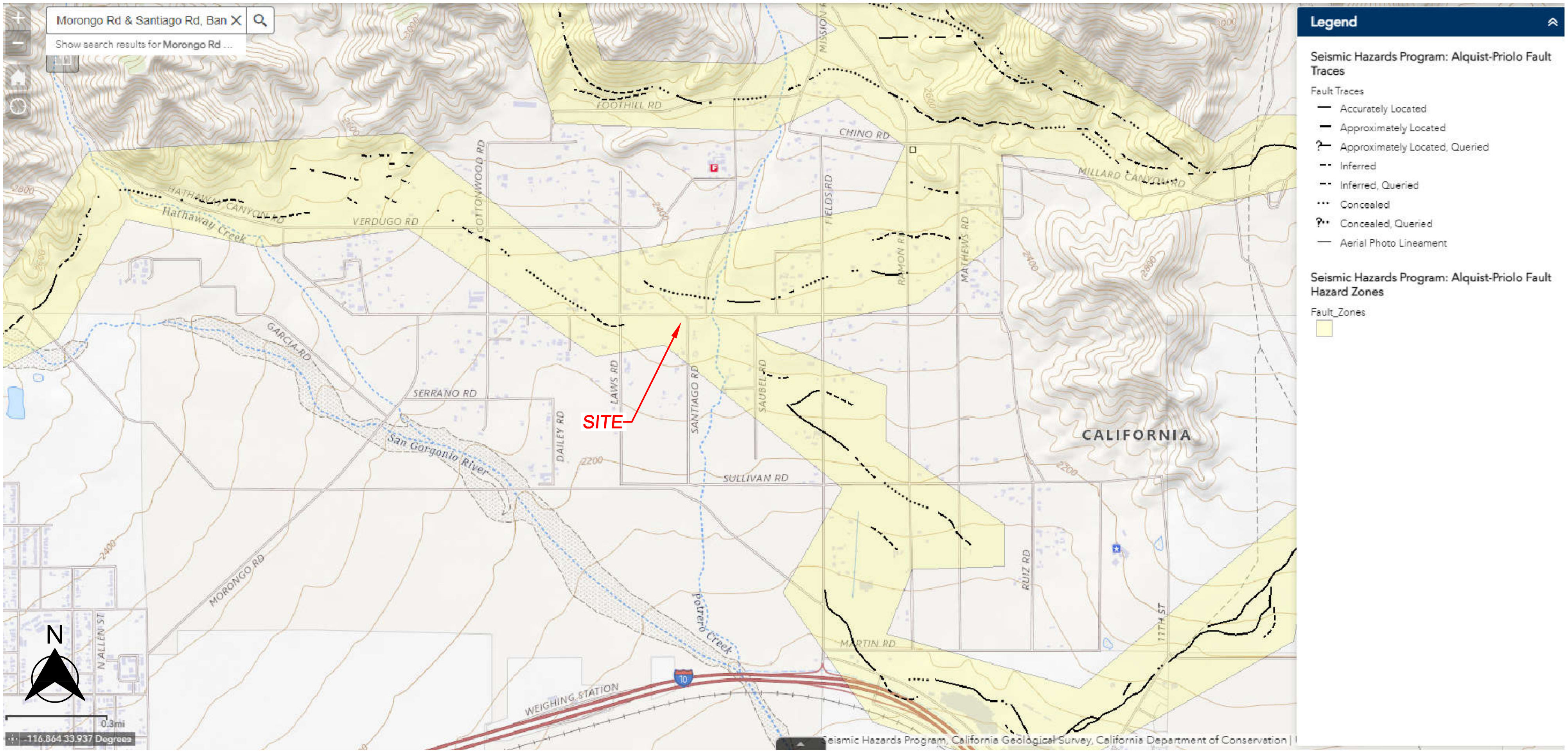
Late Quaternary fault displacement (during past 700,000 years)

Quaternary fault (age undifferentiated)

Reference -http://www.conservation.ca.gov/cgs/information/publications/Pages/quaternaryfaults_ver2.aspx)

MAP OF ACTIVE AND POTENTIALLY ACTIVE FAULTS RELATIVE TO SITE PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/27/2023
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. H17401.01	DRAWING NO. 4

MOORE TWINING ASSOCIATES, INC.



Reference: Treinman - State of California Department of Conservation, Division of Mines and Geology Fault Evaluation Report FER-235 titled The San Gorgonio Pass, Banning and Related Faults, 1994

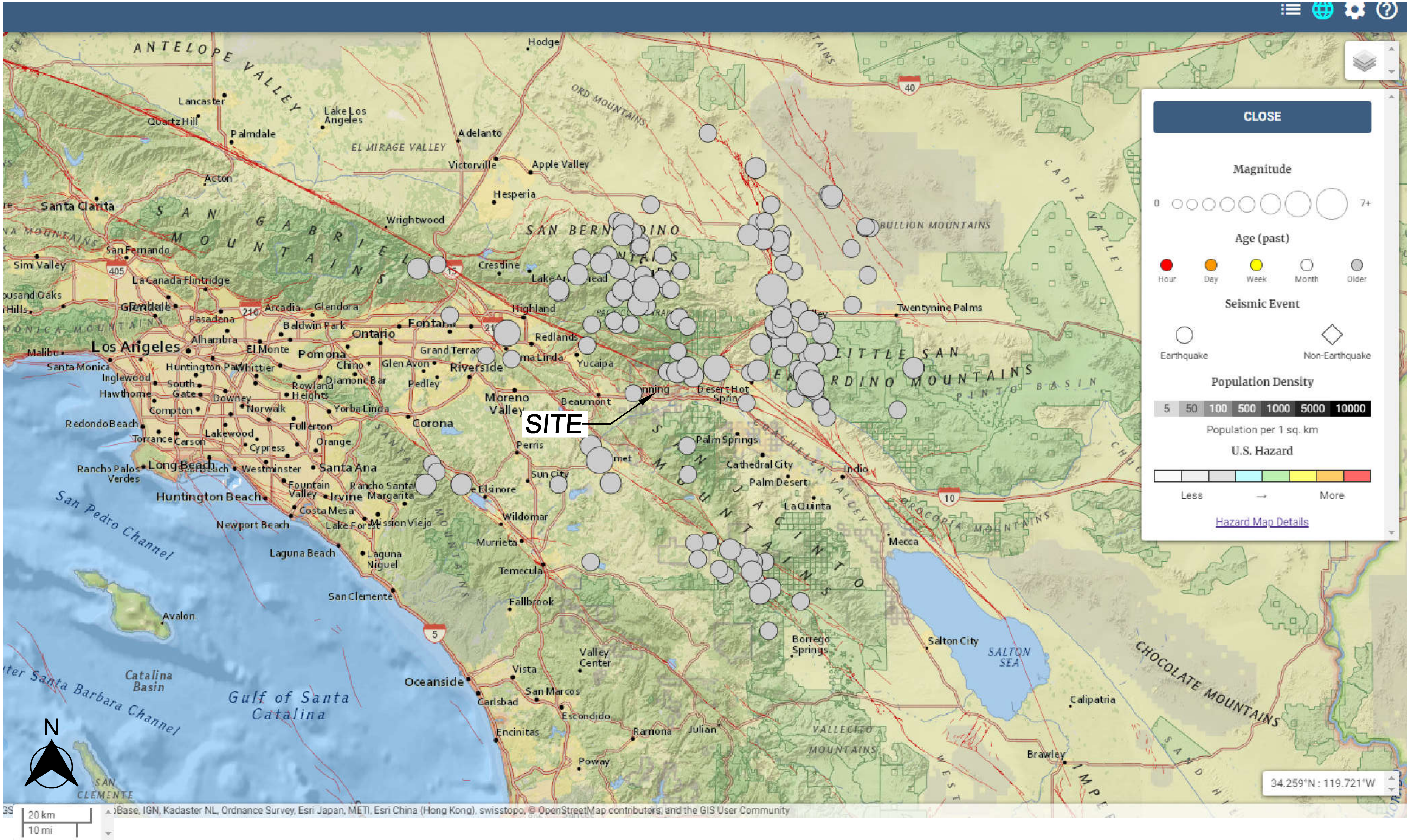
STATE OF CALIFORNIA
EARTHQUAKE FAULT ZONES
Delineated in compliance with
Chapter 7.5, Division 2 of the California Public Resources Code
(Alquist-Priolo Earthquake Fault Zoning Act)

CABAZON QUADRANGLE

REVISED OFFICIAL MAP

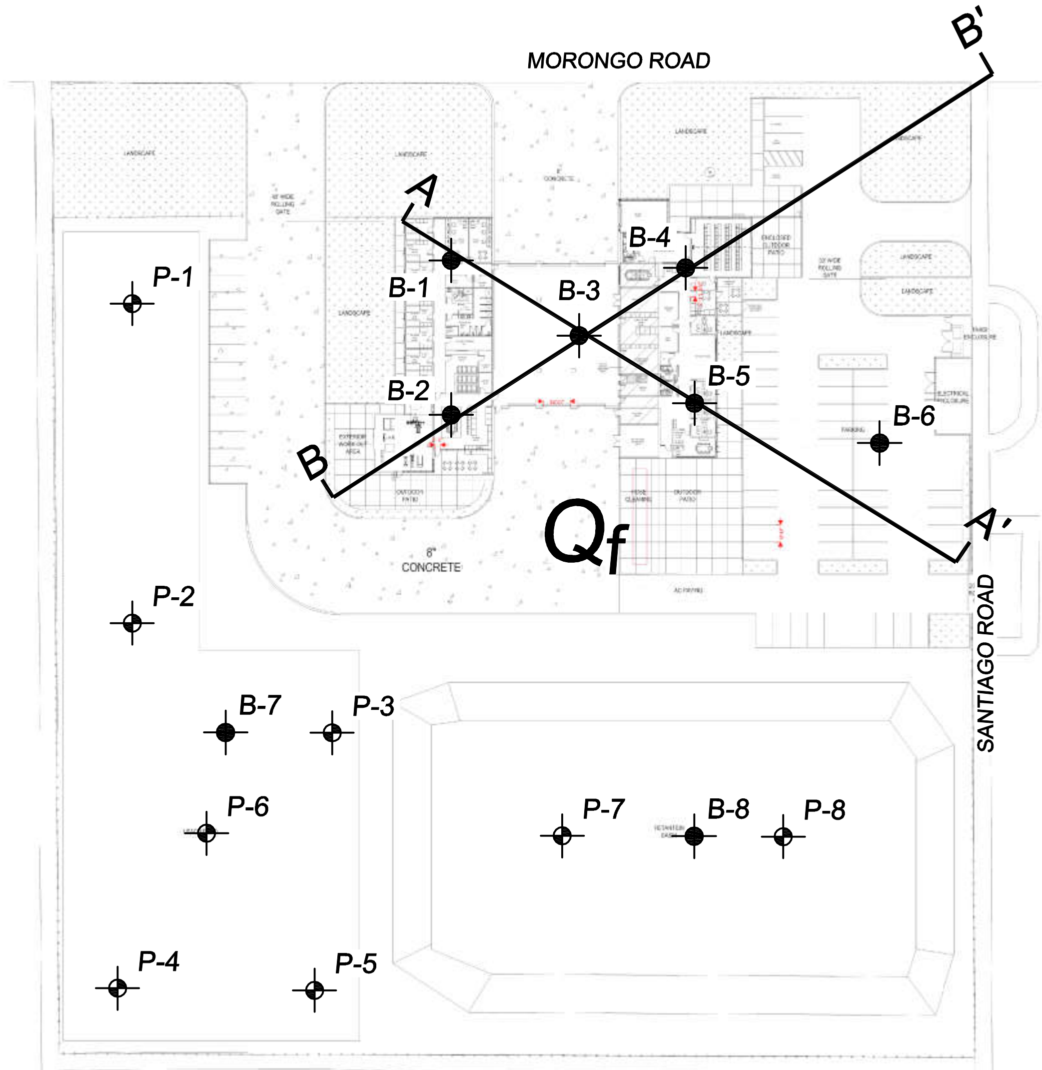
Effective: June 1, 1995

MAP OF SAN GOCONIO PASS FAULT HAZARD ZONE AND MAPPED TRACES PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/27/2023	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H17401.01	DRAWING NO. 5	



Reference - United States Geological Survey, Circular Area Earthquake
https://earthquake.usgs.gov/earthquakes/search/

HISTORICAL EARTHQUAKE EPICENTER MAP PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/27/2023	MOORE TWINING ASSOCIATES, INC. Since 1898
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H17401.01	DRAWING NO. 5	



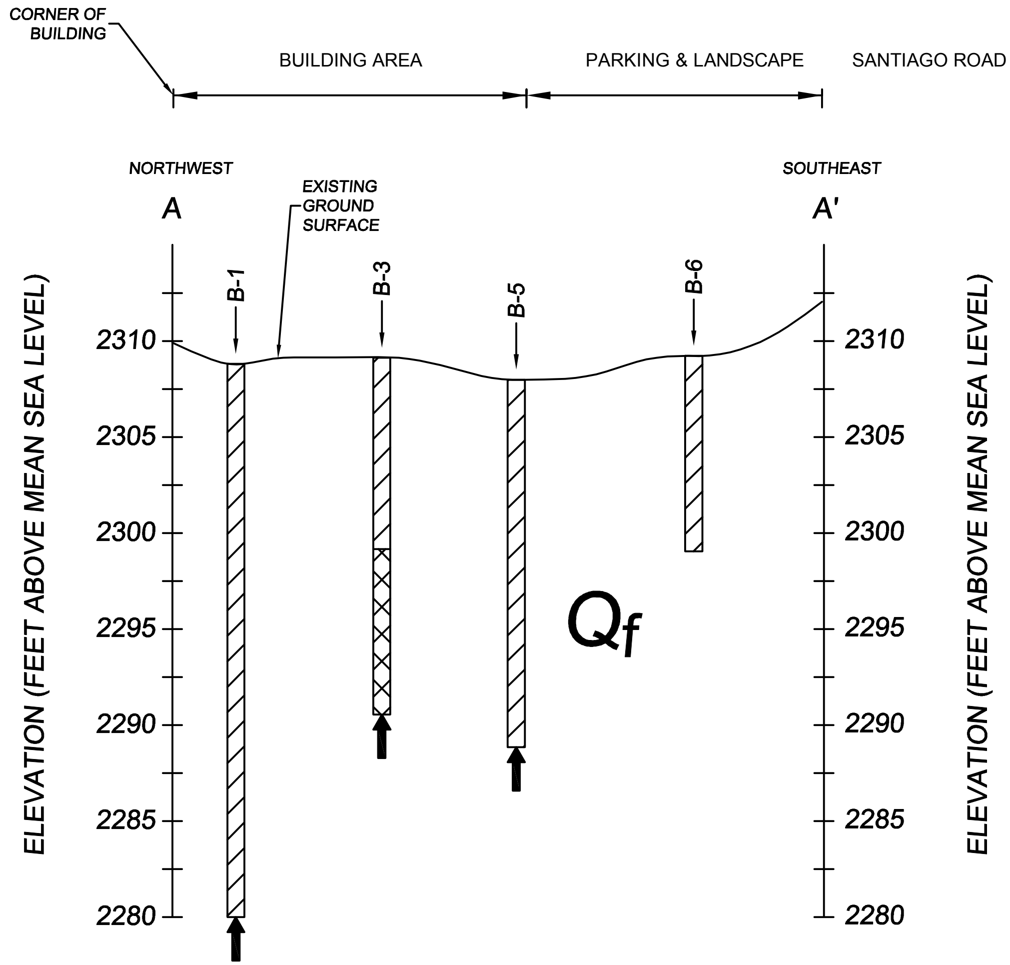
Qf - ALLUVIAL FAN OF SAN GORGONIO PASS, SAND AND GRAVEL OF PLUTONIC AND GNEISSIC DETRITUS DERIVED FROM RISING SAN BERNARDINO MOUNTAINS TO NORTH; SLIGHTLY DISSECTED BY STREAM CHANNEL; INCLUDES SMALL ALLUVIAL FANS AT BASE OF AND DERIVED FROM SAN JACINTO MOUNTAINS IN SOUTH AREA

APPROXIMATE TEST BORING LOCATION

APPROXIMATE PERCOLATION TEST LOCATION

A A' CROSS SECTION

SITE GEOLOGIC MAP PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 02/09/23	MOORE TWINING ASSOCIATES, INC. Since 1898
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H17401.01	DRAWING NO. 7	

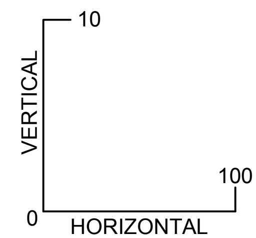


GEOLOGIC UNITS

Q_f: ALLUVIAL FAN OF SAN GORGONIO PASS, SAND AND GRAVEL OF PLUTONIC AND GNEISSIC DETRITUS DERIVED FROM RISING SAN BERNARDINO MOUNTAINS TO NORTH; SLIGHTLY DISSECTED BY STREAM CHANNEL; INCLUDES SMALL ALLUVIAL FANS AT BASE OF AND DERIVED FROM SAN JACINTO MOUNTAINS IN SOUTH AREA

SOIL CLASSIFICATION

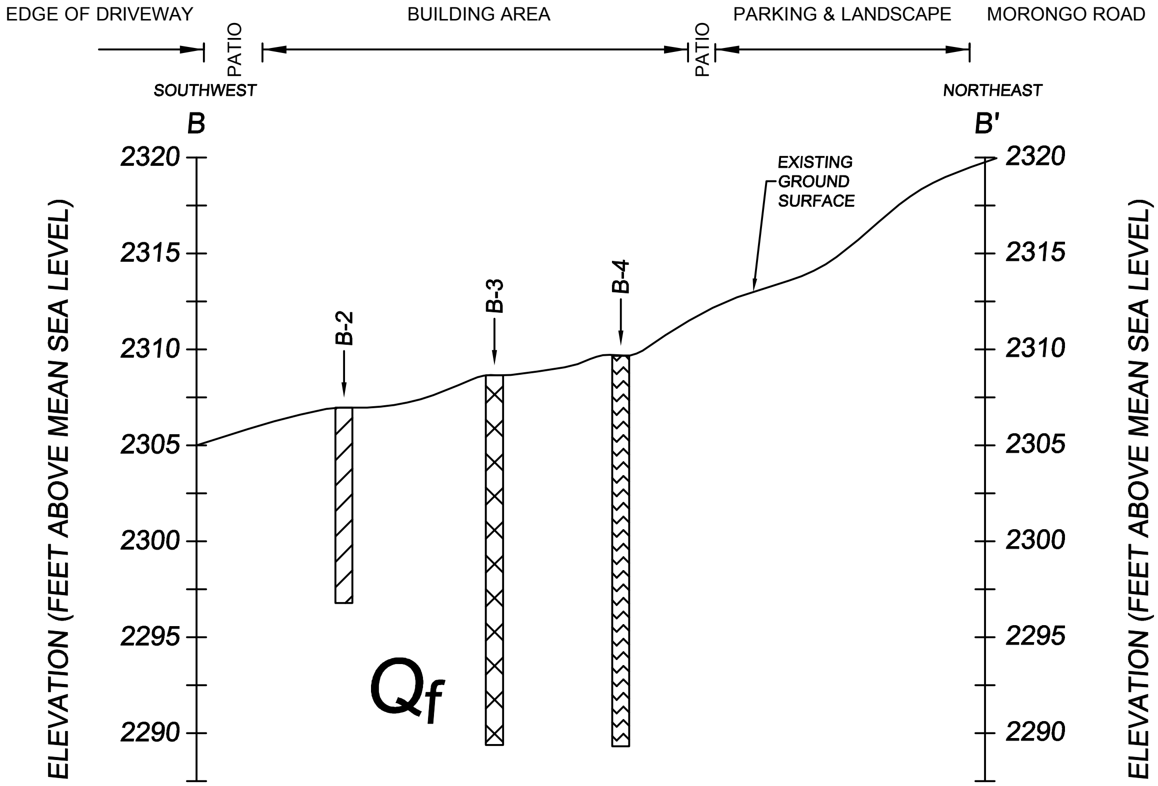
- GP-GM
- GW-GM



APPROXIMATE SCALE IN FEET

GEOLOGIC SOIL PROFILE CROSS SECTION A-A' PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/28/2023
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. H17401.01	DRAWING NO. 8

MOORE TWINING ASSOCIATES, INC.

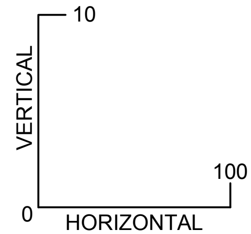


GEOLOGIC UNITS

Q_f: ALLUVIAL FAN OF SAN GORGONIO PASS, SAND AND GRAVEL OF PLUTONIC AND GNEISSIC DETRITUS DERIVED FROM RISING SAN BERNARDINO MOUNTAINS TO NORTH; SLIGHTLY DISSECTED BY STREAM CHANNEL; INCLUDES SMALL ALLUVIAL FANS AT BASE OF AND DERIVED FROM SAN JACINTO MOUNTAINS IN SOUTH AREA

SOIL CLASSIFICATION

- GP-GM
- GM
- GW-GM



APPROXIMATE SCALE IN FEET

GEOLOGIC SOIL PROFILE CROSS SECTION B-B' PROPOSED FIRE STATION SOUTHWEST OF THE INTERSECTION OF MORONGO ROAD AND SANTIAGO ROAD RIVERSIDE COUNTY, CALIFORNIA	FILE NO. 17401-01-02	DATE DRAWN: 04/28/2023
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. H17401.01	DRAWING NO. 9

MOORE TWINING ASSOCIATES, INC.

APPENDIX B

LOGS OF BORINGS

This appendix contains the final logs of borings. These logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-1

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2309.5 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 7/6 10/6	GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; medium dense, moist, fine sand to coarse gravel, brown	+4=47.3% Sand=42.4% -200=10.3%	17	4.3
2305	13/6 17/6 15/6		dense. light brown		32	
5	50/4		Very dense, some cobble		>50	
2300 / 10	50/5 29/6 35/6 38/6		slight increase in fine gravel	DD=130.2 pcf +4=51.5% Sand=38% -200=10.5%	>50 >50	2.1
2295 / 15	49/6 31/6 21/6				52	2.0
2290 / 20	21/6 25/6 40/6		slight increase in fines content		65	
2285 / 25	49/6 49/6 50/4		no cobble		>50	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2309.5 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
	50/5				>50	
2280 / 30			Auger and Sample refusal at 29 feet			
2275 / 35						
2270 / 40						
2265 / 45						
2260 / 50						
2255 / 55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: B-2

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

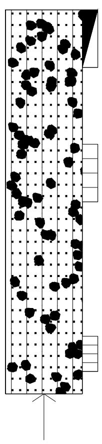
Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2306.7 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0 (2306.7)	6/6 5/6 12/6	GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; medium dense, moist, fine sand to coarse gravel, brown	+4=47.3% Sand=42.4% -200=10.3%	17	7.9
2305						
	11/6 14/6 12/6				26	3.7
5						
2300						
	25/6 50/5		Very dense, light brown, with some cobble	DD=122.6 pcf	>50	2.1
10			Auger refusal at 10 feet due to cobble			
2295						
15						
2290						
20						
2285						
25						
2280						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

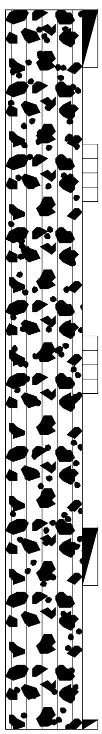
Logged By: JF

Date: March 7, 2023

Elevation: 2309.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 10/6 13/6	GW-GM	WELL GRADED GRAVEL WITH SILT AND SAND; medium dense, moist, fine sand to coarse gravel, brown	+4=58% Sand=35.7% -200=6.3% LL=NV PI=NP	23	5.8
2305 / 5	22/6 13/6 15/6		Increase in sand content	Disturbed sample	28	2.5
2300 / 10	11/6 28/6 32/6		Very dense	DD=126.3 pcf	60	3.3
2295 / 15	18/6 30/6 32/6				62	2.3
2290 / 20	50/3		With some cobble		>50	
			Bottom of Boring at 18.7 feet			
2285 / 25						



Notes:

Figure Number

Test Boring: B-4

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 6, 2023

Elevation: 2310.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
2310 / 0	5/6 5/6 7/6	GM	SILTY GRAVEL WITH SAND; medium dense, moist, fine sand to coarse gravel, brown	EI=0	12	4.1
	50/5		Very dense, with some cobble		>50	
2305 / 5	28/6 24/6 38/6			DD=127.4 pcf +4=44.2% Sand=42 -200=13.8%	62	2.7
2300 / 10	29/6 34/6 26/6				60	2.7
2295 / 15	32/6 40/6 50/5		increase in gravel content		>50	1.4
	28/6 33/6 37/6				70	
2290 / 20			Bottom of Boring at 20 feet			
2285 / 25						

Notes:

Figure Number

Test Boring: B-5

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

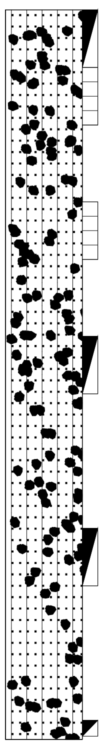
Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2307.5 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	8/6 5/6 7/6	GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; medium dense, fine sand to coarse gravel, brown		12	4.0
2305	13/6 30/6 38/6		very dense	DD=126.7 pcf	68	2.2
5	14/6 30/6 22/6			DD=131.7 pcf	42	4.6
2300 / 10	35/6 21/6 35/6		Very dense, decrease in moisture content		56	2.8
2295 / 15	16/5 22/6 42/6				64	
2290	50/5				>50	
19			Bottom of Boring at 19 feet			
20, 2285, 25, 2280						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2310.2

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
2310 / 0	6/6 12/6 14/6	GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; medium dense, moist, fine sand to coarse gravel, brown	+4=52% Sand=38.4% -200=9.6% LL=NV PL=NP	26	
	24/6 25/6 23/6		dense		48	
2305 / 5						
	20/6 16/6 26/6		increase in gravel content		42	
2300 / 10			Bottom of Boring B-6 at 10 feet			
2295 / 15						
2290 / 20						
2285 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-7

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2297.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; moist, fine sand to coarse gravel, brown			
2295						
	41/6 50/3		very dense, with some cobble		>50	
5						
2290						
	27/6 25/6 19/6				44	
10						
2285						
	16/6 23/6 22/6				45	
15			Bottom of Boring B-7 at 15 feet			
2280						
20						
2275						
25						
2270						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-8

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2302.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		GM	SILTY GRAVEL WITH SAND; moist, fine sand to coarse gravel, brown			
2300						
	2/6 9/6 10/6		Medium dense		19	
5						
2295						
	14/6 21/6 19/6		dense, increase in gravel content		38	
10						
2290						
	37/6 39/6 36/6				75	
15						
2285						
	21/6 25/6 22/6				47	
20			Bottom of Boring at 20 feet			
2280						
25						
2275						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-1

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2304.2 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; moist, fine sand to coarse gravel, brown			
2300 / 5						
2295 / 10	7/6 17/6 14/6		dense		31	
			Bottom of Boring at 12 feet			
2290 / 15						
2285 / 20						
2280 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-2

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2301.0 Feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0 / 2300		GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; moist, fine sand to coarse gravel, brown			
5 / 2295	4/6 15/6 21/6		Dense		36	
			Bottom of Boring at 7 feet			
10 / 2290						
15 / 2285						
20 / 2280						
25 / 2275						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-3

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2300.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
2300 — 0		GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; moist, fine sand to coarse gravel, brown			
2295 — 5						
	24/6 40/6 22/6		very dense		62	
			Bottom of Boring at 9 feet			
2290 — 10						
2285 — 15						
2280 — 20						
2275 — 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-4

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2294.0 feet

Depth to Groundwater
First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; moist, fine sand to coarse gravel, brown			
2290	41/6 49/6 49/6		Very dense		>50	
5			Bottom of Boring at 5 feet			
2285 / 10						
2280 / 15						
2275 / 20						
2270 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-5

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2293.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		GP-GM	POORLY GRADED GRAVEL WITH SILT AND SAND; moist, fine sand to coarse gravel, brown Medium dense			
2290	21/6 10/6 11/6				21	
			Bottom of Boring at 4 feet			
5						
2285						
10						
2280						
15						
2275						
20						
2270						
25						
2265						

Notes:

Figure Number

Test Boring: P-6

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2295.0 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
2295 / 0		GM	SILTY GRAVEL WITH SAND; moist, fine sand to coarse gravel, brown			
	19/6 6/6 8/6				14	
2290 / 5			Bottom of Boring at 5 feet			
2285 / 10						
2280 / 15						
2275 / 20						
2270 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-7

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Logged By: JF

Drilled By: JC

Date: March 7, 2023

Drill Type: CME-75

Elevation: 2300.0 feet

Auger Type: 8" Hollow Stem Augers

Hammer Type: 140# Auto-Trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
2300 — 0		GM	SILTY GRAVEL WITH SAND; moist, fine sand to coarse gravel, brown			
2295 — 5						
2290 — 10						
	20/6 19/6 15/6		very dense		34	
2285 — 15			Bottom of Boring at 15 feet			
2280 — 20						
2275 — 25						

Notes:

Figure Number

Test Boring: P-8

Project: Proposed Morongo Reservation Fire Station

Project Number: H17401.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 8" Hollow Stem Augers

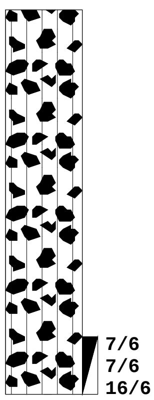
Hammer Type: 140# Auto-Trip

Logged By: JF

Date: March 7, 2023

Elevation: 2304.0 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		GM	SILTY GRAVEL WITH SAND; moist, fine sand to coarse gravel, brown			
2300 / 5						
2295	7/6 7/6 16/6		Medium dense		23	
10			Bottom of Boring at 10 feet			
2290 / 15						
2285 / 20						
2280 / 25						

Notes:

Figure Number

KEY TO SYMBOLS

Symbol	Description	Symbol	Description
Strata symbols		**Soil Samplers**	
	Poorly graded gravel with silt		Standard penetration test
	Well graded gravel with silt		California Modified split barrel ring sampler
	Silty gravel		

Misc. Symbols

Symbol	Description
	Drill rejection
	Boring continues

Notes:

1. Exploratory borings were drilled on 3/6 and 3/7/2023 using a CME 75 drill rig equipped with 6-5/8" and 8" outside diameter hollow stem augers.

2. Groundwater was not encountered in any of the borings.

3. Boring locations were measured or paced from existing features. Boring elelvations were interpolated to the nearest 0.2 feet based on contours Presented on the topographic map provided.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value should not be interpreted as an SPT equivalent N-value.

6. Results of tests conducted on samples recovered are reported on the logs.

DD	= Natural dry density (pcf)	LL	= Liquid Limit (%)
+4	= Percent retained on the No. 4 sieve(%)	PI	= Plasticity Index (%)
-200	= Percent passing the No. 200 sieve (%)	EI	= Expansion Index
Sand	= Percent passing the No. 4 sieve and retained on No. 200 sieve (%)	Gravel	= Percent passing 3-inch & retained on No. 4 sieves(%)
pH	= Soil pH	SR	= Soil resistivity (ohms-cm)
SS	= Soluble sulfates (%)	Cl	= Soluble chlorides (%)
ø	= Internal Angle of Friction (degrees)	c	= Cohesion (psf)
pcf	= Pounds per cubic foot	psf	= Pounds per square foot
O.D.	= Outside diameter	AMSL	= Above mean sea level
N/A	= Not applicable	N/E	= Not encountered

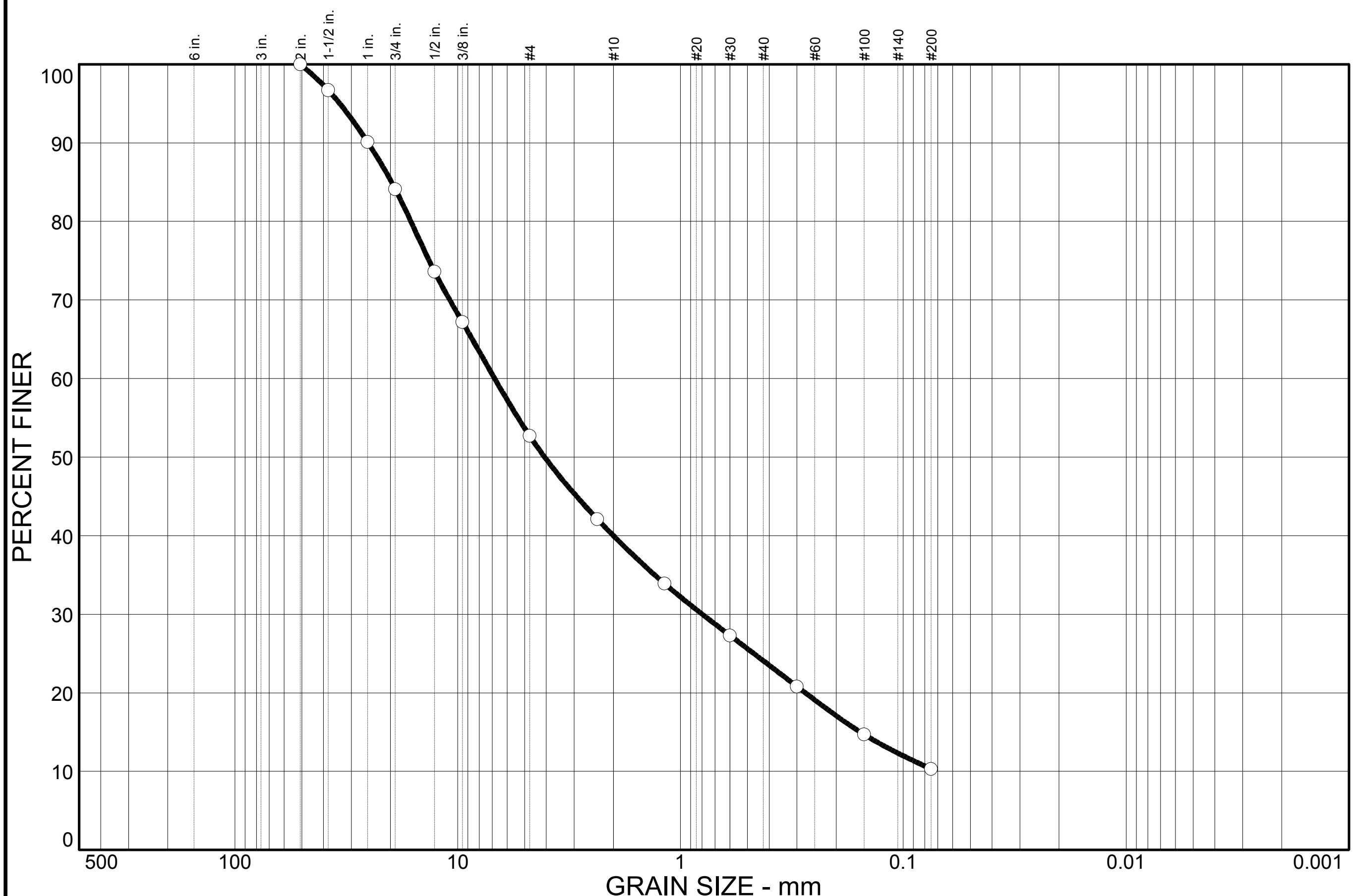
APPENDIX C

RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Moisture Content (ASTM D2216)	Moisture contents representative of field conditions at the time the sample was taken.
Dry Density (ASTM D2937)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., clay, silt, sand, and gravel.
Atterberg Limits (ASTM D4318)	Determines the moisture content where the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.
R-Value (ASTM D2844)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Sulfate Content (Cal Test 417)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (Cal Test 422)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM G187)	The potential of the soil to corrode metal.

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	15.9	31.4	12.7	15.9	13.8	10.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2 in.	100.0		
1-1/2 in.	96.7		
1 in.	90.1		
3/4 in.	84.1		
1/2 in.	73.6		
3/8 in.	67.2		
#4	52.7		
#8	42.1		
#16	33.9		
#30	27.3		
#50	20.8		
#100	14.7		
#200	10.3		

* (no specification provided)

Material Description

Poorly graded gravel with silt and sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 19.8 D_{60}= 6.83 D_{50}= 4.07
D_{30}= 0.799 D_{15}= 0.156 D_{10}=
C_u= C_c=

Classification

USCS= GP-GM AASHTO=

Remarks

Sample No.: Composite B-1 & B-2 **Source of Sample:** **Date:** 3/6/23
Location: **Elev./Depth:** 0-5'

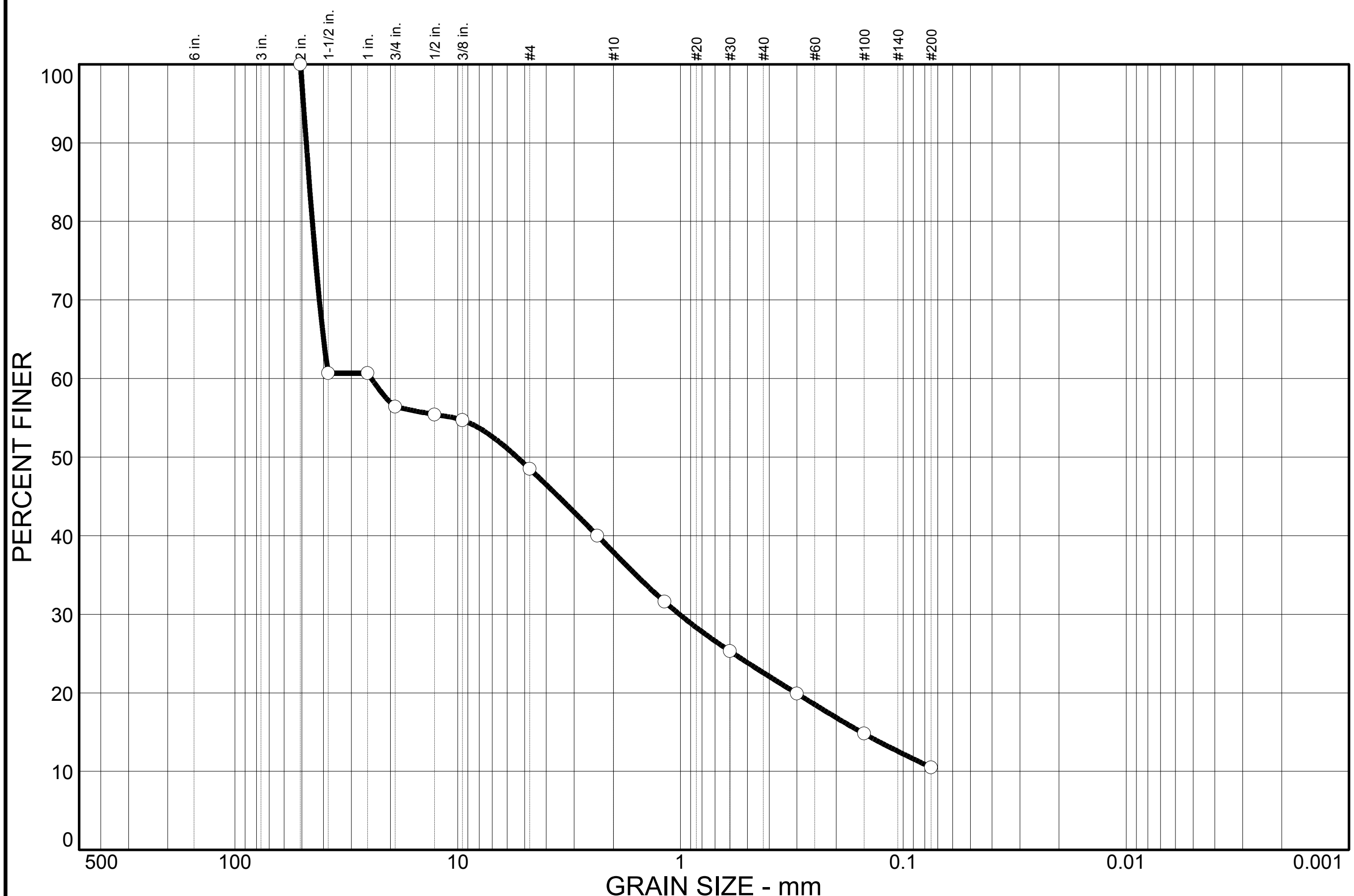
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Morongo Reservation Fire Station

Project No: **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	43.6	7.9	10.6	15.4	12.0	10.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2 in.	100.0		
1-1/2 in.	60.7		
1 in.	60.7		
3/4 in.	56.4		
1/2 in.	55.4		
3/8 in.	54.7		
#4	48.5		
#8	40.0		
#16	31.6		
#30	25.3		
#50	19.9		
#100	14.8		
#200	10.5		

* (no specification provided)

Material Description

Poorly graded gravel with silt and sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 46.4 D_{60}= 24.4 D_{50}= 5.42
D_{30}= 1.01 D_{15}= 0.154 D_{10}=
C_u= C_c=

Classification

USCS= GP-GM AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 3/6/23
Location: **Elev./Depth:** 8.5-10'

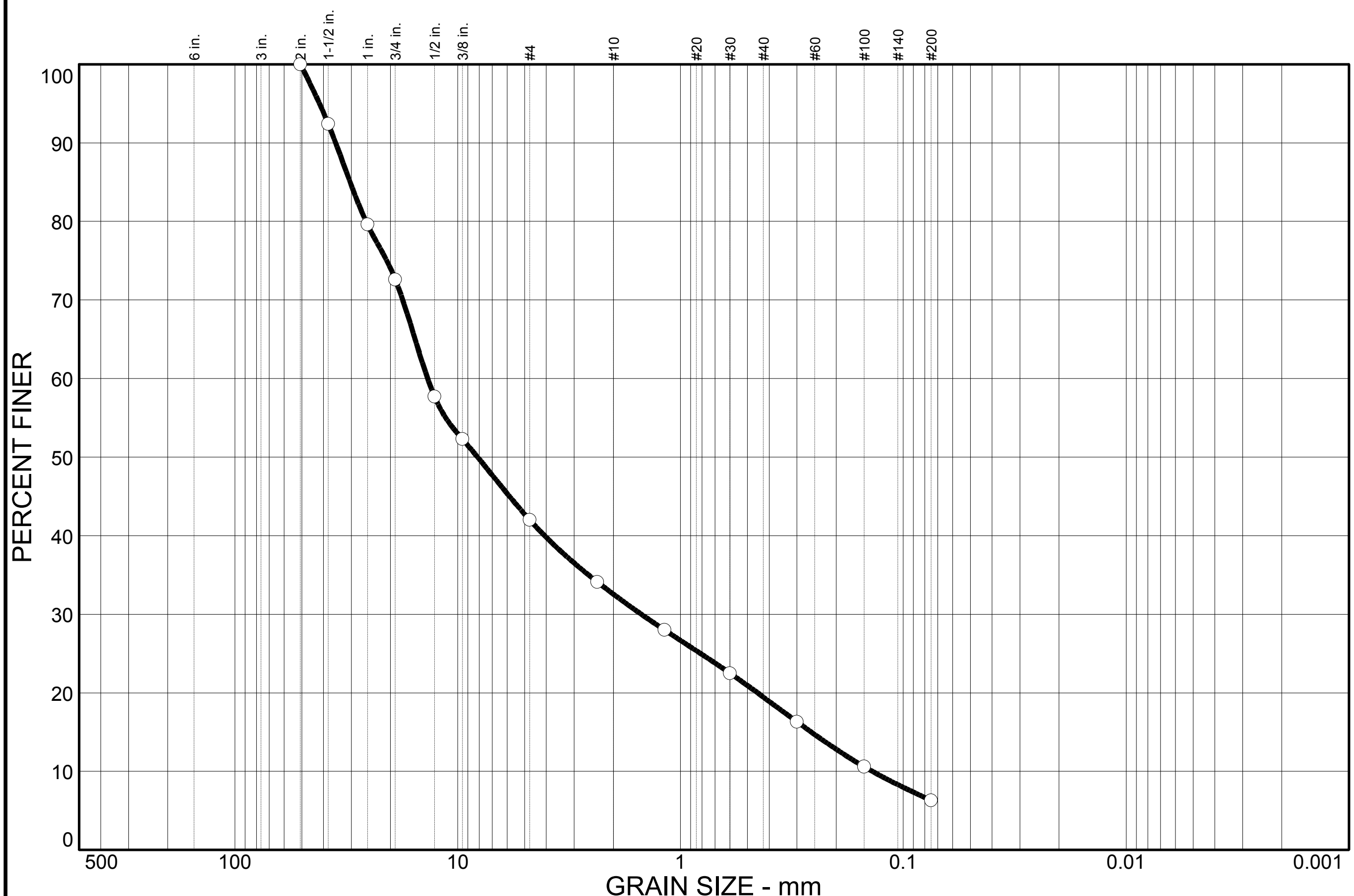
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Morongo Reservation Fire Station

Project No: **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	27.4	30.6	9.5	13.1	13.1	6.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2 in.	100.0		
1-1/2 in.	92.4		
1 in.	79.6		
3/4 in.	72.6		
1/2 in.	57.7		
3/8 in.	52.3		
#4	42.0		
#8	34.1		
#16	28.0		
#30	22.5		
#50	16.3		
#100	10.6		
#200	6.3		

* (no specification provided)

Material Description

Well-graded gravel with silt and sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 30.4 D_{60}= 13.6 D_{50}= 8.14
D_{30}= 1.50 D_{15}= 0.259 D_{10}= 0.138
C_u= 99.09 C_c= 1.20

Classification

USCS= GW-GM AASHTO=

Remarks

Sample No.: B-3 **Source of Sample:** **Date:** 3/6/23
Location: **Elev./Depth:** 0-5'

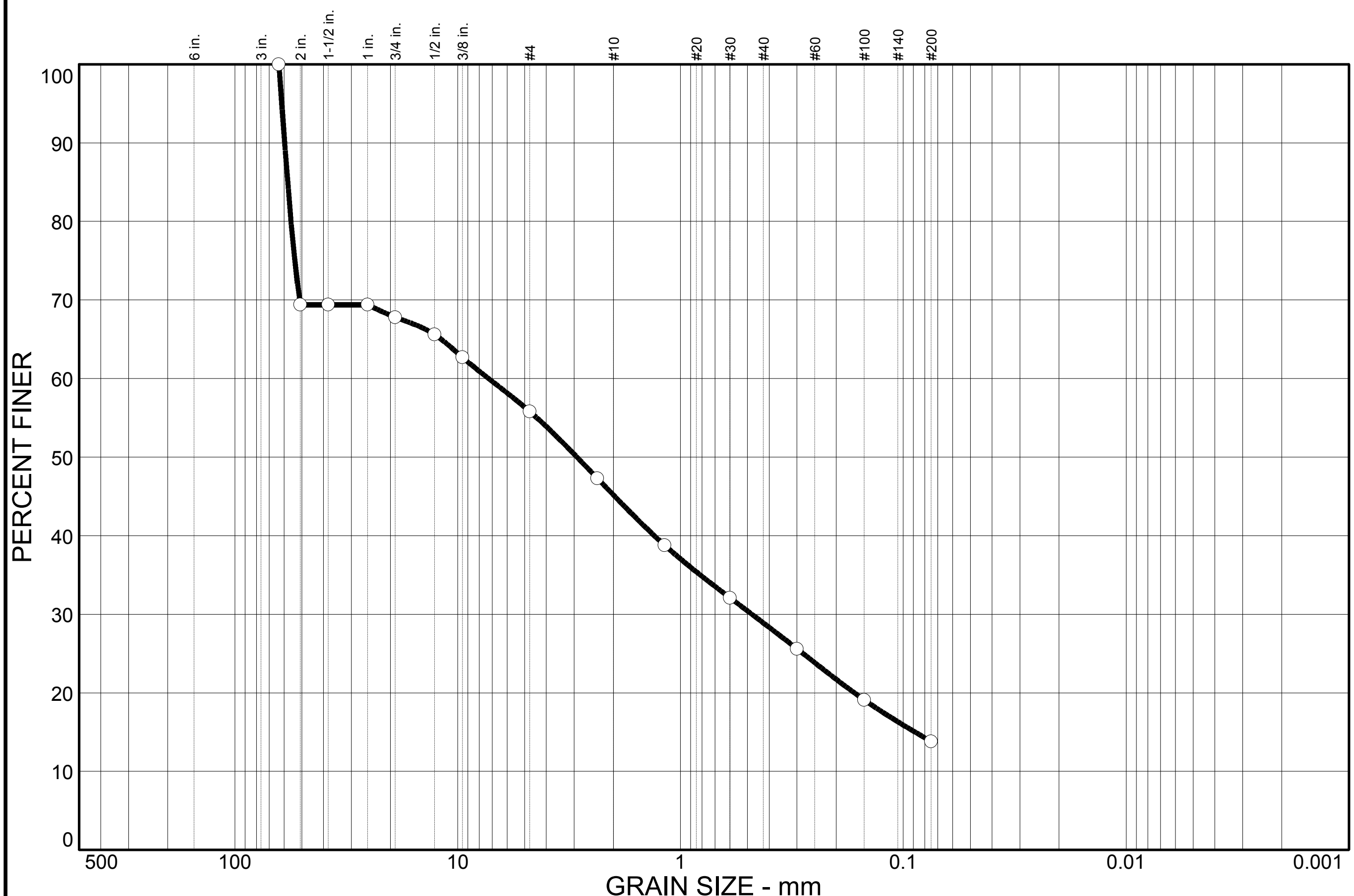
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Morongo Reservation Fire Station

Project No: **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	32.2	12.0	10.6	16.3	15.1	13.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2-1/2 in.	100.0		
2 in.	69.4		
1-1/2 in.	69.4		
1 in.	69.4		
3/4 in.	67.8		
1/2 in.	65.6		
3/8 in.	62.7		
#4	55.8		
#8	47.3		
#16	38.8		
#30	32.1		
#50	25.6		
#100	19.1		
#200	13.8		

* (no specification provided)

Material Description

Silty gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 57.9 D_{60}= 7.27 D_{50}= 2.91
D_{30}= 0.479 D_{15}= 0.0886 D_{10}=
C_u= C_c=

Classification

USCS= GM AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 3/6/23
Location: **Elev./Depth:** 5-6.5'

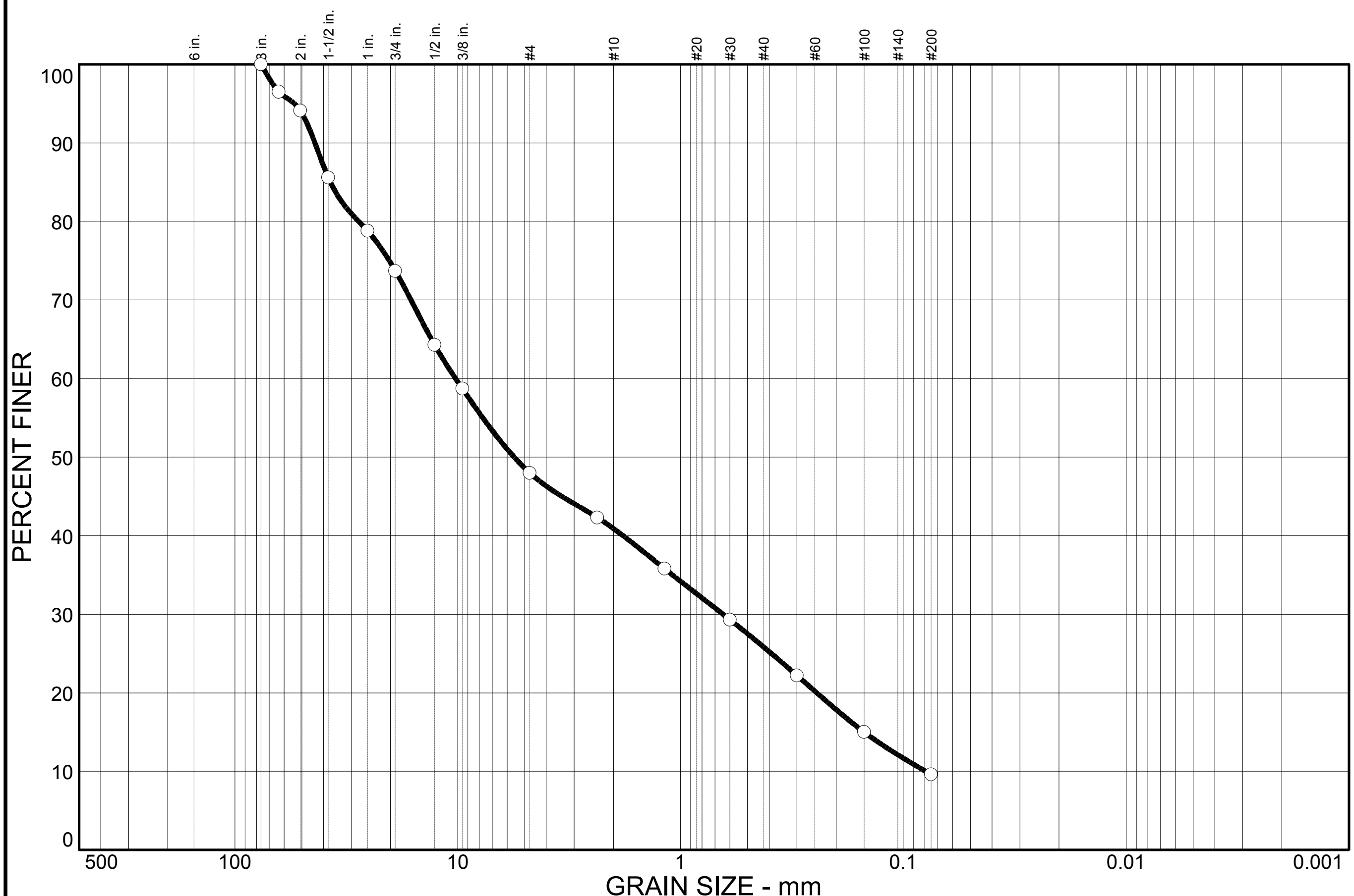
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Morongo Reservation Fire Station

Project No: **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	26.3	25.7	7.1	15.0	16.3	9.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3 in.	100.0		
2-1/2 in.	96.5		
2 in.	94.1		
1-1/2 in.	85.6		
1 in.	78.8		
3/4 in.	73.7		
1/2 in.	64.3		
3/8 in.	58.7		
#4	48.0		
#8	42.3		
#16	35.8		
#30	29.3		
#50	22.2		
#100	15.0		
#200	9.6		

* (no specification provided)

Material Description

Poorly graded gravel with silt and sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 37.3 D_{60}= 10.2 D_{50}= 5.58
D_{30}= 0.645 D_{15}= 0.150 D_{10}= 0.0793
C_u= 128.91 C_c= 0.51

Classification

USCS= GP-GM AASHTO=

Remarks

Sample No.: B-6 **Source of Sample:** **Date:** 3/6/23
Location: **Elev./Depth:** 0-5'

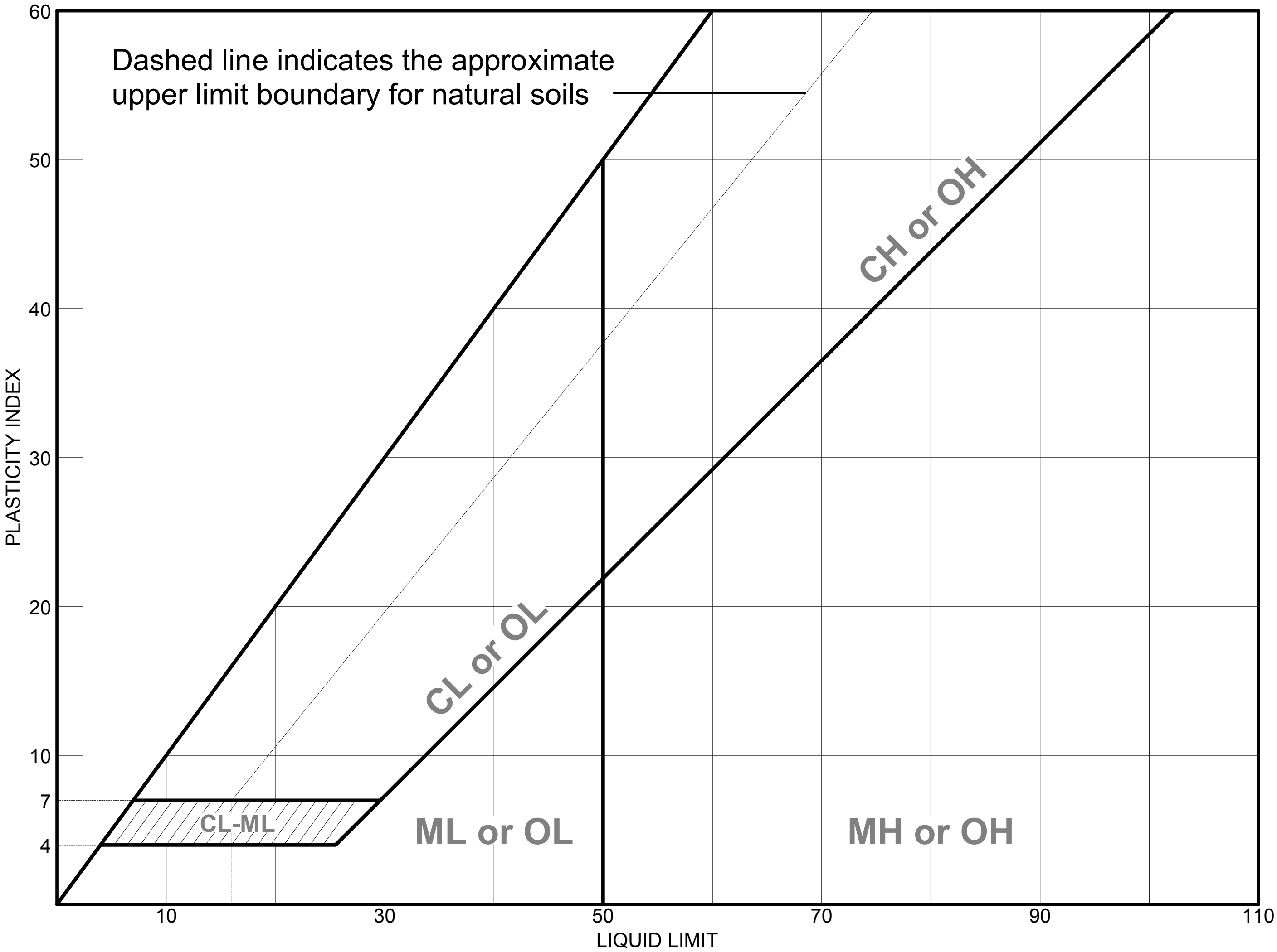
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Morongo Reservation Fire Station

Project No: **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Well-graded gravel with silt and sand	NV	NP	NP	19.5	6.3	GW-GM

Project No. **Client:**

Project: Morongo Reservation Fire Station

● **Source:** **Sample No.:** B-3 **Elev./Depth:** 0-5'

Remarks:

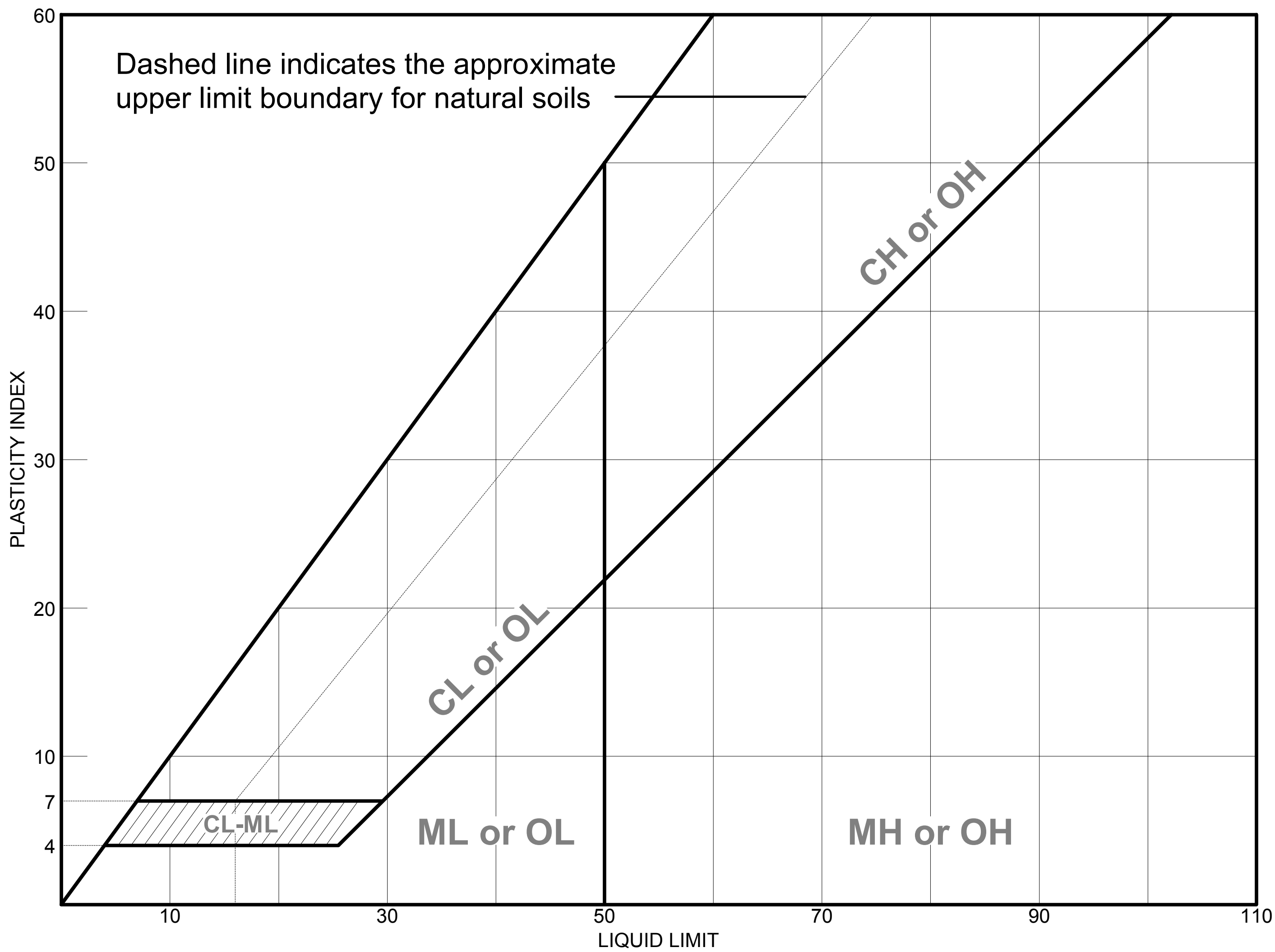
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Poorly graded gravel with silt and sand	NV	NP	NP	25.9	9.6	GP-GM

Project No. **Client:**

Project: Morongo Reservation Fire Station

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 0-5'

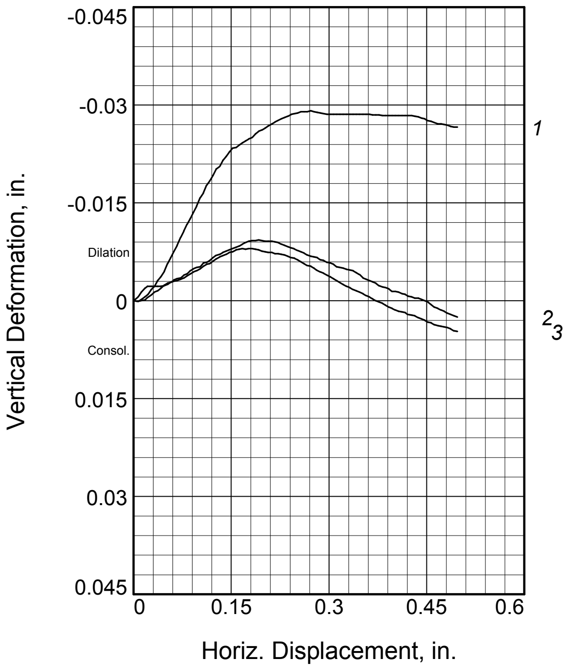
Remarks:

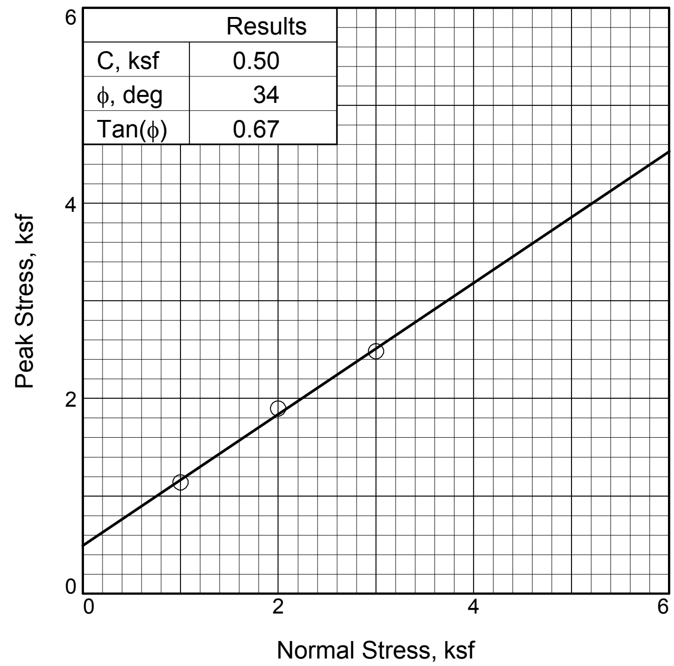
●

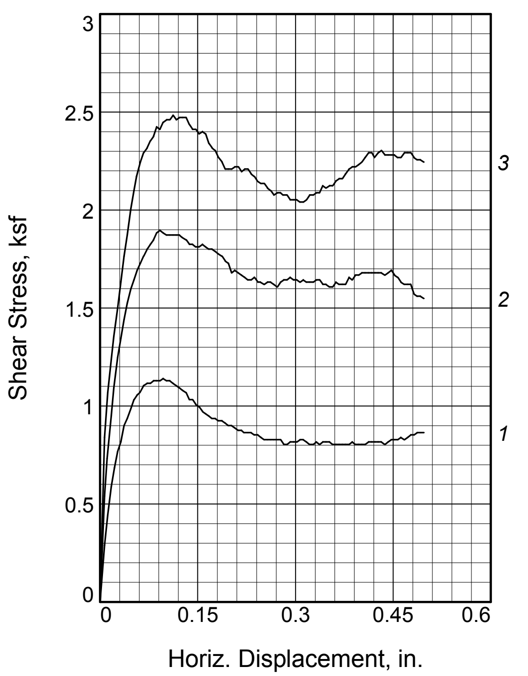
Moore Twining Associates, Inc.

Fresno, CA

Figure







Sample No.		1	2	3
Initial	Water Content, %	7.7	7.5	7.5
	Dry Density, pcf	121.0	120.8	121.2
	Saturation, %	55.5	53.8	54.3
	Void Ratio	0.3673	0.3692	0.3645
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	13.2	13.5	13.2
	Dry Density, pcf	121.8	121.7	122.2
	Saturation, %	97.5	99.5	98.4
	Void Ratio	0.3583	0.3589	0.3543
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.99	0.99
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		1.14	1.90	2.48
Displacement, in.		0.10	0.09	0.11
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		4.0	3.8	4.6

Sample Type:

Description: Poorly graded gravel with silt and sand

Specific Gravity= 2.65

Remarks: Remolded shear to approx. 120.0 pcf & 7.5% M.C.

Figure ________

Client:

Project: Morongo Reservation Fire Station

Sample Number: Composite B-1 & B-2 **Depth:** 0-5'

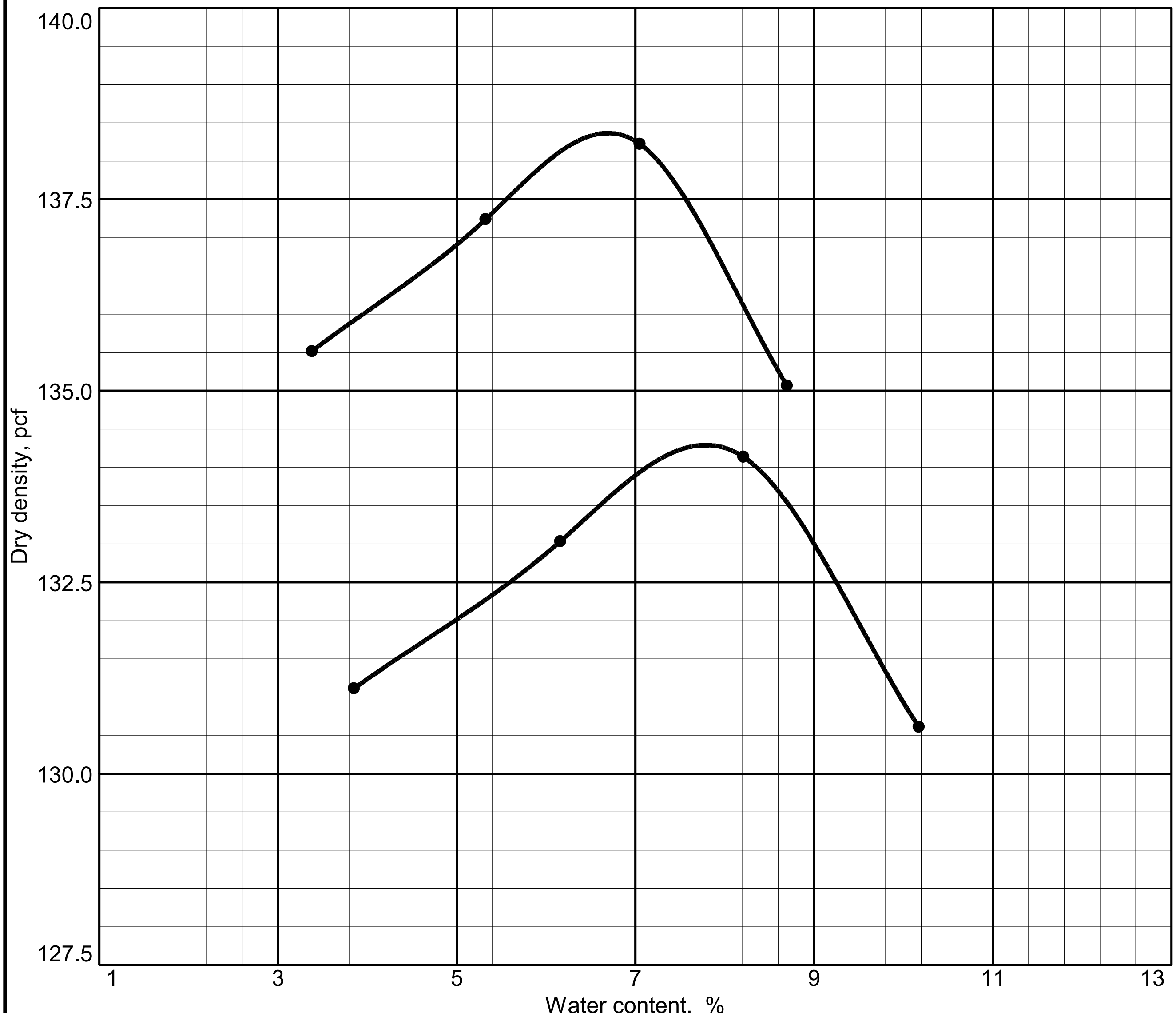
Proj. No.: C236A3.01 **Date Sampled:** 3/6/23

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Fresno, CA

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Procedure C Modified
Oversize correction applied to each point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
0-5'	GP-GM						15.9	

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 138.4 pcf	134.3 pcf	Poorly graded gravel with silt and sand
Optimum moisture = 6.7 %	7.8 %	

Project No. C236A3.01 **Client:**
Project: Morongo Reservation Fire Station

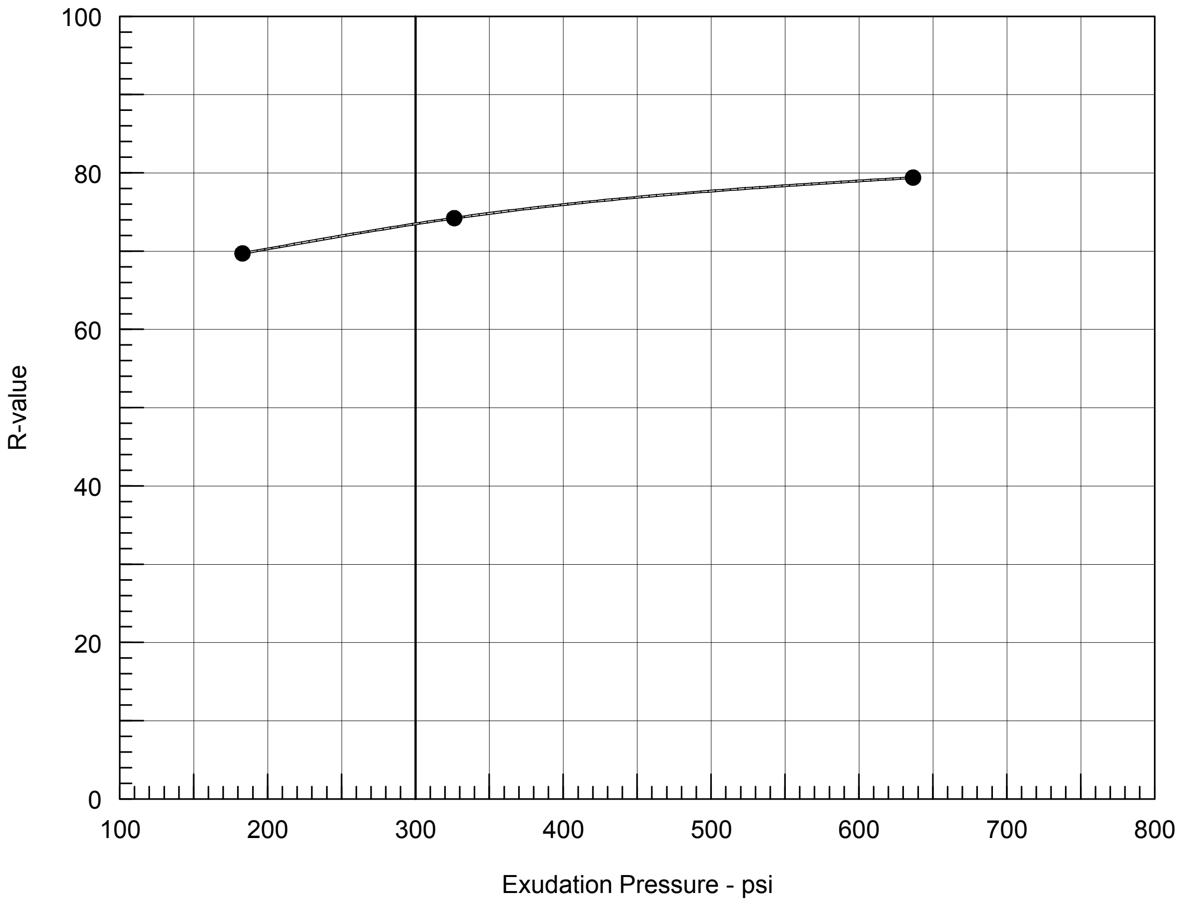
● **Source:** **Sample No.:** Composite **Elev./Depth:** 0-5'

Moore Twining Associates, Inc.
Fresno, CA

Remarks:

Figure

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	132.5	6.5	0.00	18	2.46	637	79	79
2	350	131.3	7.9	0.00	22	2.48	326	74	74
3	350	130.1	8.8	0.00	26	2.50	183	70	70

Test Results	Material Description
R-value at 300 psi exudation pressure = 73	Well-graded gravel with silt and sand

Project No.: C236A3.01

Project: Morongo Reservation Fire Station

Sample Number: B-3 **Depth:** 0-5'

Date: 4/10/2023

Tested by: MS

Checked by: MS

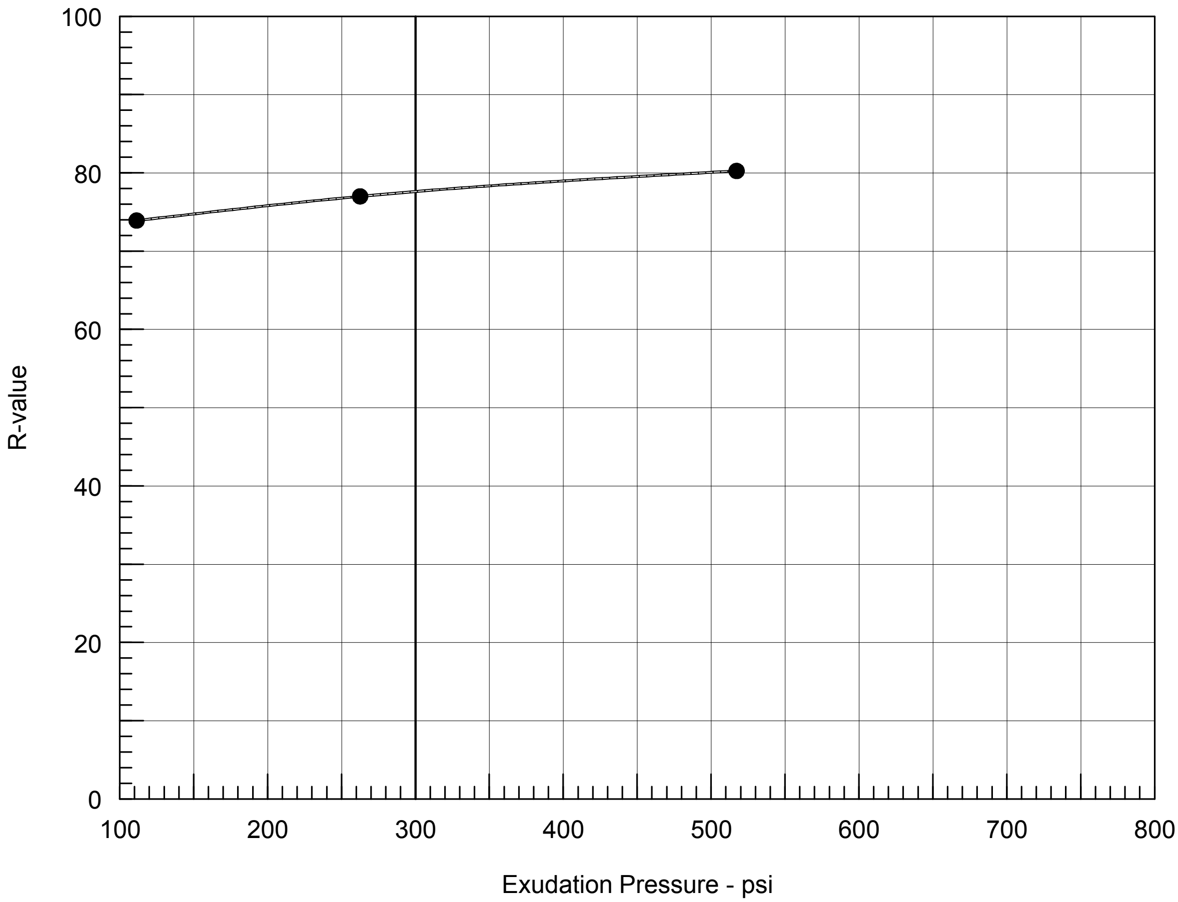
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	132.9	9.1	0.00	26	2.48	111	74	74
2	350	133.5	8.2	0.00	23	2.46	263	77	77
3	350	135.9	7.3	0.00	19	2.44	517	81	80

Test Results	Material Description
R-value at 300 psi exudation pressure = 78	Poorly graded gravel with silt and sand

Project No.: C236A3.01

Project: Morongo Reservation Fire Station

Sample Number: B-6 **Depth:** 0-5'

Date: 4/10/2023

Tested by: MS

Checked by: MS

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

MOORE TWINING ASSOCIATES, INC.
Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Morongo Reservation Fire Station	REPORT DATE:	4/3/2023
		TEST DATE:	3/29/2023
MTA PROJECT NO.:	H17401.01		
SAMPLE I.D.:	B-4 @ 0-3.5'		
SAMPLED BY:	JF		
SAMPLE DATE:	3/6/2023	TESTED BY:	BP

MATERIALS DESCRIPTION: Silty sand

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9976
Pan + Dry Soil Wt., gm	233.6	Dry Soil Wt., lbs	0.8821
Pan Wt., gm	0.0		
Initial % Moisture Content	7.0	Final % Moisture Content	13.1

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.9440	Ring + Sample Wt., lbs	0.9976
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9440	Remolded Wt., lbs	0.9976
Remolded Wet Density, pcf	129.8	Remolded Wet Density, pcf	137.2
Remolded Dry Density, pcf	121.3	Remolded Dry Density, pcf	121.3

Initial Volume	Final Volume
0.00727222	0.007269

Expansion Data:

Initial Gage Reading, in:	0.3517
Final Gage Reading, in:	0.3513
Expansion, in:	-0.0004
Expansion Index	0

Comments: **Very Low Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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FX: 559.268.7126
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Fresno, CA 93721

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

April 18, 2023

Work Order #: **JC31005**

Read Andersen
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Morongo Reservation Fire Station

Enclosed are the analytical results for samples received by our laboratory on **03/31/23** . For your reference, these analyses have been assigned laboratory work order number **JC31005**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Lauren Cox
Client Services Representative

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Morongo Reservation Fire Station **Project Number:** [none] **Project Manager:** Read Andersen	**Reported:** 04/18/2023

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B1 & B2 @ 0'-5' (Modified)		JC31005-01	Soil	03/06/23 00:00	03/31/23 08:45

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Morongo Reservation Fire Station **Project Number:** [none] **Project Manager:** Read Andersen	**Reported:** 04/18/2023

B1 & B2 @ 0'-5' (Modified)

JC31005-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Flag
Inorganics								
Chloride	ND	0.0040	% by Weight	[CALC]	04/14/23	04/14/23	[CALC]	
Chloride	ND	40	mg/kg	B3D1312	04/13/23	04/14/23	Cal Test 422	
pH	**7.9**	0.10	pH Units	B3D1312	04/13/23	04/14/23	Cal Test 643	**HT2**
Sulfate as SO4	**0.0082**	0.0040	% by Weight	[CALC]	04/14/23	04/14/23	[CALC]	
Sulfate as SO4	**82**	40	mg/kg	B3D1312	04/13/23	04/14/23	Cal Test 417	

Notes and Definitions

HT2	This sample was analyzed past the EPA recommended holding time for this parameter due to late delivery of the sample to the laboratory.
PREP	Modified preparation by pulverizing sample to pass #40 sieve and soaked for a minimum of 12 hours using a minimum dilution ratio of 1:10
ND	Analyte NOT DETECTED at or above the reporting limit
mg/kg	milligrams per kilogram (parts per million concentration units)

MOORE TWINING ASSOCIATES, INC.

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

CHAIN OF CUSTODY / ANALYSIS REQUEST

2527 FRESNO STREET • FRESNO, CA 93721 • PHONE (559) 268-7021 • FAX: (559) 268-0740

WORK ORDER #: JC 31005
PAGE 01 OF 01

REPORT TO:
ATTENTION: Read A.
COMPANY NAME: MTA GeoTechnical Division
ADDRESS:
PHONE:
EMAIL / FAX:

☐ INVOICE TO: ☐ REPORT COPY TO:
ATTENTION:
COMPANY NAME:
ADDRESS:
PHONE:
EMAIL / FAX:

REPORTING:
☒ STANDARD FORMAT ☐ PDF
☐ EDT (SWRCB) ☐ EXCEL
☐ GEOTRACKER/COELT (LUFT)
GLOBAL ID:
☐ COUNTY ENVIRONMENTAL HEALTH:
☐ STATE WATER RESOURCES CONTROL BOARD:
☐ OTHER:

SAMPLE INFORMATION
SAMPLED BY (PRINT): Jordi F.
SIGNATURE:

☐ PUBLIC SYSTEM ☒ ROUTINE
☐ PRIVATE WELL ☐ REPEAT
☐ OTHER ☐ REPLACEMENT

TURN AROUND TIME
☒ STANDARD
☐ RUSH, DUE ON:

SAMPLE TYPES
SOLID:
BS - BIOSOLID
CR - CERAMIC
SL - SOIL/SOLID
LIQUID:
DW - DRINKING WATER
GW - GROUND WATER
OL - OIL
SF - SURFACE WATER
ST - STORM WATER
WW - WASTEWATER

PROJECT INFORMATION
CONTRACT / P.O. NO.: MTP 0731-22
PROJECT: Morongo Reservation Fire Station
PROJECT NUMBER:
PROJECT MANAGER: Read A.

NOTES ON RECEIVED CONDITION:
☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLES(S) DAMAGED
☐ ON ICE ☒ AMBIENT TEMP. ☐ INCORRECT PRESERVATION

ANALYSIS REQUESTED

LAB USE	CLIENT SAMPLE ID	DATE	TIME	TYPE	PH	Chloride	Sulfate								STATION CODE
	B1 & B2 @ 0'- 5' (Modified	03-06-2023	NA	SL	x	x	x								

COMMENTS / ADDITIONAL INSTRUCTIONS:

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
Bounsone Phoxikhamboy		03-31-2023	08:45 AM	W McCullogh	MTA
Bounsone P.					

FL-SC-0005-04 A

Moore Twining Associates

Sample Integrity Page 2 of 2 WO# SC31005 MTA Bottles: Yes or No

COC Info			
Was temperature within range? Chemistry ≤6°C Micro <10°C Temp______°C	Yes No N/A	Did all bottle labels agree with COC?	Yes No N/A
If samples were taken today, is there evidence that chilling has begun? Recvd______C°	Yes No N/A	Was a sufficient amount of sample received?	Yes No N/A
Did all bottles arrive unbroken and intact?	Yes No N/A	Were correct containers and preservatives received for the tests requested?	Yes No N/A
Do samples have a hold time <72 hours?	Yes No N/A		
Were there bubbles in VOA vials? (Volatiles Only)	Yes No N/A	Was PM notified of discrepancies? PM: By/Time:	Yes No N/A

Bottles Received

125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)															
Bacti $Na_2S_2O_3$	1														
None (P)															
None (AG)															
None (CG) 500ml															
Cr6 Buffer (P) Borate Carbonate Buffer															
Dissolved Oxygen 300ml (P)															
HNO_3 (P)															
HCl (AG)															
H_2SO_4 (P)															
H_3PO_4 (AG)															
NaOH (P)															
NaOH + ZnAc (P)															
$Na_2S_2O_3$ (AG)															
$Na_2S_2O_3$ (CG)															
$Na_2S_2O_3$ 250ml (Brown P) 549															
Thio/K Citrate															
NH_4Cl (AG) 552															
Other:															
Client Own															
Low Level Hg/Metals Double Bag															
Plastic Bag	1														
Glass Jar: 125 / 250 / 500															
Soil Tube: Brass / Steel / Plastic															
5g Encore															
1G Cubitaner															

Comments

Filter or Split	Container	Preservative	Date/Time/Initials
S P F			
S P F			
S P F			
S P F			
S P F			

Labeled by M @______ Labels checked by: HB @ 0935

FL-SC-0003-07

MOORE TWINING ASSOCIATES, INC.

Project Name:	Morongo Reservation Fire Station	Report Date:	4/3/2023
		Sample Date:	3/6/2023
Project Number:	H17401.01		
		Sampled By:	JF
Subject:	Minimum Resistivity, ASTM G187	Tested By:	BP
Material Description:	Poorly graded gravel with silt and sand	Test Date:	3/31/2023
Location:	Composite B-1 & B-2 @ 0-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
100 mls	33,000
125 mls	25,000
150 mls	10,300
175 mls	9,600
200 mls	9,100
225 mls	9,500

Remarks: Min. Resistivity is 9,100 Ohm-cm

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2527 Fresno Street
Fresno, CA 93721

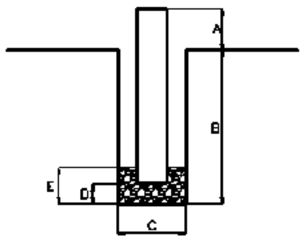
APPENDIX D

RESULTS OF FIELD PERCOLATION TESTS

This appendix contains the individual results of the six Percolation Tests conducted for this investigation.

PERCOLATION TEST
No. P-1

Project:	Proposed Morongo Fire Station	**Project No.**	H17401.01
Location:	SWC of Morongo and Santiago Roads	**Test Date:**	3/9/2023



A. Top of Pipe Above Ground	22	Inches
B. Depth of Hole	148	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2	Inches
E. Total Gravel Layer Depth	48	Inches
F. Pipe Length	168	Inches
G. Pipe Diameter	2	Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

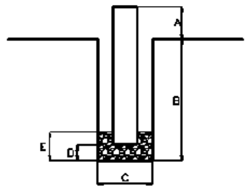
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	8:50:00	12.75				
Presoak		9:20:00	12.81	30.0	0.72	106.6	0.1
2		9:20:00	12.81				
Presoak		9:50:00	12.86	30.0	0.6	128.0	0.1
3		9:50:00	12.86				
	3/9/2023	8:01:00	14.05	Depth Measured at start. (dry)			
4		8:01:00	13.45				
Pre-Trial	3/9/2023	8:31:00	13.47	30.0	0.24	319.9	0.0
5		8:38:00	13.45				
Pre-Trial	3/9/2023	9:08:00	13.49	30.0	0.48	160.0	0.1
6		9:08:00	13.45				
	3/9/2023	9:38:00	13.5	30.0	0.6	128.0	0.1
7		9:38:00	13.45				
	3/9/2023	10:08:00	13.46	30.0	0.12	639.9	0.0
8		10:08:00	13.45				
	3/9/2023	10:38:00	13.47	30.0	0.24	319.9	0.0
9		10:38:00	13.45				
	3/9/2023	11:08:00	13.47	30.0	0.24	319.9	0.0
10		11:08:00	13.45				
	3/9/2023	11:38:00	13.47	30.0	0.24	319.9	0.0
11		11:38:00	13.45				
	3/9/2023	12:08:00	13.48	30.0	0.36	213.3	0.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

Project:	Proposed Morongo Fire Station	**Project No.**	H17401.01
Location:	SWC of Morongo and Santiago Roads	**Test Date:**	3/9/2023



A. Top of Pipe Above Ground	48	Inches
B. Depth of Hole	86	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2	Inches
E. Total Gravel Layer Depth	46	Inches
F. Pipe Length	132	Inches
G. Pipe Diameter	2	Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

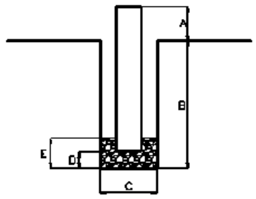
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	9:45:00	9.95				
Presoak		10:15:00	10.25	30.0	3.6	21.3	0.4
2		10:15:00	9.98				
Presoak	3/8/2023	10:45:00	10.2	30.0	2.64	29.1	0.3
3		10:45:00	9.93				
Presoak	3/8/2023	11:15:00	10.18	30.0	3	25.6	0.3
4		11:15:00	9.95				
Presoak	3/8/2023	11:45:00	10.18	30.0	2.76	27.8	0.3
5	3/8/2023	11:45:00	9.96				
	3/9/2023	8:45:00		Depth Measured at start. (dry)			
6		8:45:00	10.5				
Pre-Trial	3/9/2023	9:15:00	10.53	30.0	0.36	213.3	0.1
7		9:15:00	10.5				
Pre-Trial	3/9/2023	9:45:00	10.56	30.0	0.72	106.6	0.1
8		9:45:00	10.5				
	3/9/2023	10:15:00	10.56	30.0	0.72	106.6	0.1
9		10:15:00	10.5				
	3/9/2023	10:45:00	10.56	30.0	0.72	106.6	0.1
10		10:45:00	10.5				
	3/9/2023	11:15:00	10.58	30.0	0.96	80.0	0.2
11		11:15:00	10.5				
	3/9/2023	11:45:00	10.59	30.0	1.08	71.1	0.2
12		11:45:00	10.49				
	3/9/2023	12:15:00	10.57	30.0	0.96	80.0	0.2
13		12:15:00	10.49				
	3/9/2023	12:45:00	10.57	30.0	0.96	80.0	0.2
14		12:45:00	10.49				
	3/9/2023	1:15:00	10.56	30.0	0.84	91.4	0.1
15		15:22:00	9				
2ft. Check	3/9/2023	15:57:00	9.59	35.0	7.08	12.7	0.4
16		15:57:00	9.59				
2ft. Check	3/9/2023	16:27:00	9.85	30.0	3.12	24.6	0.3

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-3

Project: Proposed Morongo Fire Station **Project No.** H17401.01
Location: SWC of Morongo and Santiago Roads **Test Date:** 3/9/2023



A. Top of Pipe Above Ground 48 Inches
B. Depth of Hole 111 Inches
C. Diameter of Hole 8 Inches
D. Depth of Gravel Below Pipe 2 Inches
E. Total Gravel Layer Depth 48 Inches
F. Pipe Length 157 Inches
G. Pipe Diameter 2 Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

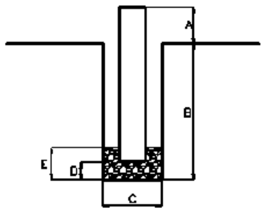
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	10:40:00	11.95				
Presoak		11:10:00	12.87	30.0	11.04	7.0	1.4
2		11:10:00	11.8				
Presoak	3/8/2023	11:40:00	12.97	30.0	14.04	5.5	1.7
3		11:40:00	11.85				
Presoak	3/8/2023	12:10:00	12.8	30.0	11.4	6.7	1.3
4	3/8/2023	12:10:00	11.6				
	3/9/2023	8:55:00		Depth Measured at start. (dry)			
5		8:55:00	12.6				
Pre-Trial	3/9/2023	9:05:00	12.83	10.0	2.76	9.3	1.5
6		9:07:00	12.6				
Pre-Trial	3/9/2023	9:32:00	13.05	25.0	5.4	11.8	1.4
7		9:32:00	12.59				
	3/9/2023	9:56:00	12.99	24.0	4.8	12.8	1.2
8		9:56:00	12.59				
	3/9/2023	10:26:00	13.04	30.0	5.4	14.2	1.2
9		10:26:00	12.6				
	3/9/2023	10:56:00	13.06	30.0	5.52	13.9	1.2
10		10:56:00	12.6				
	3/9/2023	11:26:00	13.07	30.0	5.64	13.6	1.3
11		11:26:00	12.59				
	3/9/2023	11:56:00	13.08	30.0	5.88	13.1	1.3
12		11:56:00	12.59				
	3/9/2023	12:26:00	13.07	30.0	5.76	13.3	1.3
13		12:26:00	12.49				
	3/9/2023	12:56:00	13.06	30.0	6.84	11.2	1.4
14		12:56:00	12.5				
	3/9/2023	13:26:00	13.05	30.0	6.6	11.6	1.3
15		15:59:00	11.85				
	3/9/2023	16:20:00	12.47	21.0	7.44	7.2	1.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-4

Project:	Proposed Morongo Fire Station	**Project No.**	H17401.01
Location:	SWC of Morongo and Santiago Roads	**Test Date:**	3/9/2023



A. Top of Pipe Above Ground	42 Inches
B. Depth of Hole	57 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	48 Inches
F. Pipe Length	97 Inches
G. Pipe Diameter	2 Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

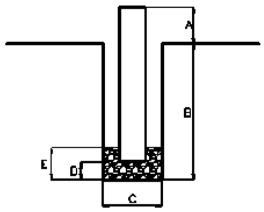
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	15:00:00	7.18				
Presoak		15:30:00	7.2	30.0	0.24	319.9	0.0
2		15:30:00	7.2				
Presoak	3/8/2023	16:00:00	7.21	30.0	0.12	639.9	0.0
3	3/8/2023	16:00:00	7.21				
	3/9/2923	8:00:00	8.12	Depth Measured at start. (dry)			
4		8:00:00	7.5				
Pre-Trial	3/9/2923	8:30:00	7.57	30.0	0.84	91.4	0.1
5		8:30:00	7.5				
Pre-Trial	3/9/2923	9:00:00	7.58	30.0	0.96	80.0	0.1
6		9:00:00	7.5				
	3/9/2923	9:30:00	7.58	30.0	0.96	80.0	0.1
7		9:30:00	7.51				
	3/9/2923	10:00:00	7.57	30.0	0.72	106.6	0.1
8		10:00:00	7.49				
	3/9/2923	10:30:00	7.56	30.0	0.84	91.4	0.1
9		10:30:00	7.5				
	3/9/2923	11:00:00	7.57	30.0	0.84	91.4	0.1
10		11:00:00	7.5				
	3/9/2923	11:30:00	7.57	30.0	0.84	91.4	0.1
11		11:30:00	7.49				
	3/9/2923	12:00:00	7.56	30.0	0.84	91.4	0.1
12		12:00:00	7.5				
	3/9/2923	12:30:00	7.55	30.0	0.6	128.0	0.1
13		12:30:00	7.49				
	3/9/2923	13:00:00	7.55	30.0	0.72	106.6	0.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-5

Project: Proposed Morongo Fire Station **Project No.** H17401.01
Location: SWC of Morongo and Santiago Roads **Test Date:** 3/9/2023



A. Top of Pipe Above Ground	25	Inches
B. Depth of Hole	49	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2	Inches
E. Total Gravel Layer Depth	46	Inches
F. Pipe Length	72	Inches
G. Pipe Diameter	2	Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

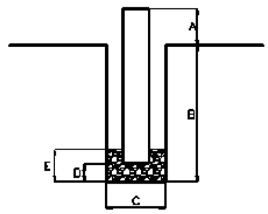
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	14:10:00	5				
Presoak		14:40:00	5.92	30.0	11.04	7.0	1.6
2		14:40:00	4.95				
Presoak	3/8/2023	15:10:00	5.8	30.0	10.2	7.5	1.4
3	3/8/2023	15:10:00	4.95				
	3/9/2023	11:05:00	6.1	Depth Measured at start. (dry)			
4		11:05:00	5.58				
Pre-trial	3/9/2023	11:35:00	5.85	30.0	3.24	23.7	0.7
5		11:35:00	5.5				
Pre-trial	3/9/2023	12:05:00	5.78	30.0	3.36	22.9	0.6
6		12:05:00	5.51				
	3/9/2023	12:35:00	5.78	30.0	3.24	23.7	0.6
7		12:35:00	5.6				
	3/9/2023	13:05:00	5.82	30.0	2.64	29.1	0.5
8		13:05:00	5.6				
	3/9/2023	13:35:00	5.81	30.0	2.52	30.5	0.5
9		13:35:00	5.6				
	3/9/2023	14:05:00	5.83	30.0	2.76	27.8	0.6
10		14:05:00	5.6				
	3/9/2023	14:35:00	5.81	30.0	2.52	30.5	0.5
11		14:35:00	5.57				
	3/9/2023	15:05:00	5.83	30.0	3.12	24.6	0.6
12		15:05:00	5.45				
	3/9/2023	15:35:00	5.75	30.0	3.6	21.3	0.6
13		15:53:00	5.48				
	3/9/2023	16:23:00	5.73	30.0	3	25.6	0.5

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-6

Project:	Proposed Morongo Fire Station	**Project No.**	H17401.01
Location:	SWC of Morongo and Santiago Roads	**Test Date:**	3/9/2023



A. Top of Pipe Above Ground	40 Inches
B. Depth of Hole	60 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	4 Inches
E. Total Gravel Layer Depth	48 Inches
F. Pipe Length	96 Inches
G. Pipe Diameter	2 Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

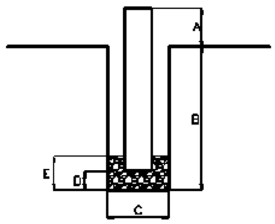
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	11:20:00	6.85				
Presoak		11:50:00	7.4	30.0	6.6	11.6	0.6
2		11:50:00	6.8				
Presoak	3/8/2023	12:20:00	7.3	30.0	6	12.8	0.5
3	3/8/2023	12:20:00	6.8	Depth Measured at start. (dry)			
	3/9/2023	11:10:00	8				
4		11:10:00	7.5				
Pre-trial	3/9/2023	11:40:00	7.62	30.0	1.44	53.3	0.2
5		11:40:00	7.5				
Pre-trial	3/9/2023	12:10:00	7.63	30.0	1.56	49.2	0.2
6		12:10:00	7.5				
	3/9/2023	12:40:00	7.64	30.0	1.68	45.7	0.2
7		12:40:00	7.5				
	3/9/2023	13:10:00	7.62	30.0	1.44	53.3	0.2
8		13:10:00	7.5				
	3/9/2023	13:40:00	7.63	30.0	1.56	49.2	0.2
9		13:40:00	7.5				
	3/9/2023	14:10:00	7.61	30.0	1.32	58.2	0.2
10		14:10:00	7.4				
	3/9/2023	14:40:00	7.59	30.0	2.28	33.7	0.3
11		14:40:00	7.39				
	3/9/2023	15:10:00	7.63	30.0	2.88	26.7	0.4
12		15:10:00	7.46				
	3/9/2023	15:40:00	7.59	30.0	1.56	49.2	0.2
13		16:01:00	7.49				
	3/9/2023	16:31:00	7.57	30.0	0.96	80.0	0.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-7

Project: Proposed Morongo Fire Station — **Project No.** H17401.01
Location: SWC of Morongo and Santiago Roads — **Test Date:** 3/9/2023



A. Top of Pipe Above Ground 26 Inches
B. Depth of Hole 182 Inches
C. Diameter of Hole 8 Inches
D. Depth of Gravel Below Pipe 2 Inches
E. Total Gravel Layer Depth 46 Inches
F. Pipe Length 206 Inches
G. Pipe Diameter 2 Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

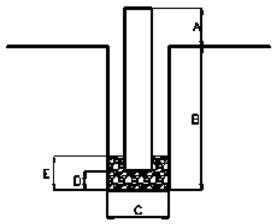
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	13:15:00	14.98				
Presoak		13:45:00	17.2	30.0	26.64	2.9	2.4
2		13:45:00	14.9				
Presoak	3/8/2023	14:15:00	17.2	30.0	27.6	2.8	2.4
3	3/8/2023	14:15:00	14.93				
	3/9/2923	9:15:00	17.2	Depth Measured at start. (dry)			
4		9:15:00	15.2				
Pre-trial	3/9/2923	9:40:00	16.67	25.0	17.64	3.6	1.7
5		9:40:00	15.2				
Pre-trial	3/9/2923	10:05:00	16.31	25.0	13.32	4.8	1.2
6		10:05:00	15.2				
	3/9/2923	10:15:00	16	10.0	9.6	2.7	1.9
7		10:15:00	15.2				
	3/9/2923	10:25:00	15.82	10.0	7.44	3.4	1.4
8		10:25:00	15.2				
	3/9/2923	10:35:00	15.92	10.0	8.64	3.0	1.7
9		10:35:00	15.2				
	3/9/2923	10:45:00	15.8	10.0	7.2	3.6	1.4
10		10:45:00	15.2				
	3/9/2923	10:55:00	15.81	10.0	7.32	3.5	1.4
11		10:55:00	15.21				
	3/9/2923	11:05:00	15.82	10.0	7.32	3.5	1.4
12		11:05:00	15.19				
	3/9/2923	11:15:00	15.81	10.0	7.44	3.4	1.5

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-8

Project: Proposed Morongo Fire Station
Location: SWC of Morongo and Santiago Roads
Project No. H17401.01
Test Date: 3/9/2023



A. Top of Pipe Above Ground — 20 Inches
B. Depth of Hole — 122 Inches
C. Diameter of Hole — 8 Inches
D. Depth of Gravel Below Pipe — 2 Inches
E. Total Gravel Layer Depth — 45 Inches
F. Pipe Length — 140 Inches
G. Pipe Diameter — 2 Inches

Pre-soak conducted on 3/8 /2023- see shaded area below

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/8/2023	13:50:00	9.2				
Presoak		14:20:00	11.6	30.0	28.8	2.7	2.3
2		14:20:00	9.15				
Presoak	3/8/2023	14:50:00	11.6	30.0	29.4	2.6	2.3
3	3/8/2023	14:50:00	9.18				
	3/9/2023	9:20:00	11.6	Depth Measured at start. (dry)			
4		9:20:00	9.6				
Pre-trial	3/9/2023	9:45:00	11.31	25.0	20.52	3.1	2.0
5		9:45:00	9.6				
Pre-trial	3/9/2023	10:10:00	10.95	25.0	16.2	3.9	1.4
6		10:10:00	9.6				
	3/9/2023	10:20:00	10.6	10.0	12	2.1	2.4
7		10:20:00	9.6				
	3/9/2023	10:30:00	10.4	10.0	9.6	2.7	1.8
8		10:30:00	9.59				
	3/9/2023	10:40:00	10.53	10.0	11.28	2.3	2.3
9		10:40:00	9.6				
	3/9/2023	10:50:00	10.45	10.0	10.2	2.5	2.0
10		10:50:00	9.6				
	3/9/2023	11:00:00	10.47	10.0	10.44	2.5	2.1
11		11:00:00	9.61				
	3/9/2023	11:10:00	10.46	10.0	10.2	2.5	2.0
12		11:10:00	9.6				
	3/9/2023	11:20:00	10.46	10.0	10.32	2.5	2.0

* Depth to water measured from top of pipe