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SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION

PROPOSED CLASSROOM BUILDING
MONTE VISTA ELEMENTARY SCHOOL
2620 ORANGE AVENUE
LA CRESCENTA-MONTROSE, CALIFORNIA

SALEM PROJECT NO. 3-220-0617
SEPTEMBER 18, 2020

PREPARED FOR:

MS. BARBARA HOWARD
GLENDALE UNIFIED SCHOOL DISTRICT
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September 18, 2020 Project No. 3-220-0617

Ms. Barbara Howard
Glendale Unified School District
349 West Magnolia Avenue
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SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION
PROPOSED CLASSROOM BUILDING
MONTE VISTA ELEMENTARY SCHOOL
2620 ORANGE AVENUE
LA CRESCENTA-MONTROSE, CA

Dear Ms. Howard:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation with Geologic Hazards Evaluation report for the Proposed Classroom Building to be located at the subject site.

The accompanying report presents our findings, conclusions, and recommendations regarding the geotechnical aspects of designing and constructing the project as presently proposed. In our opinion, the proposed project is feasible from a geotechnical viewpoint provided our recommendations are incorporated into the design and construction of the project.

We appreciate the opportunity to assist you with this project. Should you have questions regarding this report or need additional information, please contact the undersigned at (909) 980-6455.

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation with Geologic Hazards Evaluation for the Proposed Classroom Building to be located at Monte Vista Elementary School at 2620 Orange Avenue in La Crescenta-Montrose, California (see Figure 1, Vicinity Map).

The purpose of our geotechnical engineering investigation was to observe and sample the subsurface conditions encountered at the site, and provide conclusions and recommendations relative to the geotechnical aspects of constructing the project as presently proposed.

The scope of this investigation included a field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on August 31 2020, and September 2, 2020 and included the drilling of four (4) small-diameter soil borings to a maximum depth of 47 feet at the site. The locations of the soil borings are depicted on Figure 2, Site Plan. A detailed discussion of our field investigation, exploratory boring logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical properties for engineering analyses. Appendix B presents the laboratory test results in tabular and graphic format. The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions.

If project details vary significantly from those described herein, SALEM should be contacted to determine the necessity for review and possible revision of this report. Earthwork and Pavement Specifications are presented in Appendix D. If text of the report conflict with the specifications in Appendix D, the recommendations in the text of the report have precedence.

2. PROJECT DESCRIPTION

Based on the information provided to us, we understand that the site will include demolition of existing structures and construction of a two-story building with 10 standard classrooms distributed between the two levels. Maximum wall load is expected to be on the order of 5 kips per linear foot and column load is expected to be on the order of 80 kips. Floor slab soil bearing pressure is expected to be on the order of 150 psf.

A site grading plan was not available at the time of preparation of this report. As the existing project area is essentially level, we anticipate that cuts and fills during earthwork will be minimal and limited to providing a level pad and positive site drainage. In the event that changes occur in the nature or design of the project, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions of our report are modified. The site configuration and locations of proposed improvements are shown on the Site Plan, Figure 2.

3. SITE LOCATION AND DESCRIPTION

The property is identified as Monte Vista Elementary School which is located at 2620 Orange Avenue in the City of La Crescenta-Montrose, California (see Vicinity Plan, Figure 1). The proposed classroom building will be located in the center-east portion of the site. The classroom building will be replacing the existing modular buildings in the proposed location. The existing modular buildings will either be removed/demolished or transported to the lower playground area directly to the south/southwest.

At the time of our field exploration, the site consisted of asphalt pavement, a small tree, the existing buildings, and a retaining wall to the northeast. The lower playground area to the south/southwest is approximately 15 feet lower than that of the proposed location of the classroom building. The proposed building site area is relatively flat with no major changes in grade. The average elevation of the site of the proposed building is approximately 1,777 feet above mean sea level based on Google Earth imagery.

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-4) were drilled on September 2, 2020 in the areas shown on the Site Plan, Figure 2. The test borings were advanced with 6-inch hollow stem augers and 4-inch solid flight augers rotated by a truck-mounted CME 45C drill rig. Auger refusal was encountered at a depth of approximately 47 feet below existing grade due to very dense soil.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer and stratification lines were approximated on the basis of observations made at the time of drilling. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2488).

A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix "A." The logs of the test borings are presented in Appendix "A." The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The location of the test borings were determined by measuring from features shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants.

The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted.

Soil samples were obtained from the test borings at the depths shown on the logs of borings. The MCS samples were recovered and capped at both ends to preserve the samples at their natural moisture content;

SPT samples were recovered and placed in a sealed bag to preserve their natural moisture content. The borings were backfilled with soil cuttings after completion of the drilling.

5. LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory-testing program was formulated with emphasis on the evaluation of natural moisture, density, shear strength, consolidation potential, maximum density and optimum moisture determination, and gradation of the materials encountered. In addition, chemical tests were performed to evaluate the corrosivity of the soils to buried concrete and metal. Details of the laboratory test program and the results of laboratory test are summarized in Appendix "B." This information, along with the field observations, was used to prepare the final boring logs in Appendix "A."

6. SOIL AND GROUNDWATER CONDITIONS

6.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils within the depth of exploration consisted of medium dense to very dense silty sand with various amounts of gravel and gravelly sand with silt. The existing pavement consisted of 3 inches of asphaltic concrete (AC) underlain by 3 inches of aggregate base (AB).

Fill soils are expected to be present onsite between our test boring locations since the site was previously graded for the current development. Verification of the extent of fill should be determined during site grading. Field and laboratory tests suggest that the deeper native soils are moderately strong and slightly compressible. These soils extended to the termination depth of our borings.

The soils were classified in the field during the drilling and sampling operations. The stratification lines were approximated by the field engineer on the basis of observations made at the time of drilling. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted.

The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The locations of the test borings were determined by measuring from feature shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants.

6.2 Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was not encountered during this investigation. The historically highest groundwater is estimated to be at a depth of greater than 50 feet below ground surface according to the Seismic Hazard Zone Report 014, Pasadena 7.5-Minute Quadrangle, Plate 1.2 (Open-File Report 98-05), and other groundwater data. It should be recognized that water table elevations may fluctuate with time, being dependent upon seasonal precipitation, irrigation, land use, localized pumping, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

6.3 Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate and chloride levels and how they relate to cement reactivity with soil and/or water.

A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil sample was detected to be 100 mg/kg. ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class. ACI 318 requirements for site concrete based upon soluble sulfate are summarized in Table 6.3 below.

TABLE 6.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Water Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementations Materials Type
0.0010	Not Severe	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 20 mg/kg. This level of chloride concentration is considered to be mildly corrosive.

It is recommended that, at a minimum, applicable manufacturer's recommendations for corrosion protection of buried metal pipe be closely followed. Corrosion is dependent upon a complex variety of conditions, which are beyond the Geotechnical practice. Consequently, a qualified corrosion engineer should be consulted if the owner desires more specific recommendations.

7. GEOLOGIC AND SEISMIC HAZARDS EVALUATION

7.1 Geologic Setting

The subject site is located within the Verdugo basin situated between the Verdugo Mountains to the south/southeast and the San Gabriel Mountains to the north/northeast. Both mountain ranges are a part of the Transverse Range province of Southern California. The Verdugo Basin is filled with alluvial fan sediments shed from the San Gabriel and Verdugo Mountains. The younger alluvial fan deposits; which are very common within upper sediments in the basin; consist of unconsolidated fine to coarse grained sands and gravels with some interbedded clays. Deposits encountered on the subject site during exploratory drilling are discussed in detail in this report.

7.2 Faulting and Seismicity

The Peninsular Range has historically been a province of relatively high seismic activity. The nearest fault to the project site is the Sierra Madre Connected fault located approximately 0.6 miles from the site. There are no known active fault traces in the project vicinity. Based on mapping and historical seismicity, the seismicity of the Peninsular Range has been generally considered high by the scientific community. The project area is not within an Alquist-Priolo Earthquake Fault (Special Studies) Zone and will not require a special site investigation by an Engineering Geologist. Soils on site are classified as Site Class C in accordance with Chapter 16 of the California Building Code. The proposed structures are determined to be in Seismic Design Category D. To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 34.2301° North; site longitude is -118.2361° West. The ten closest active faults are summarized below in Table 7.2.

TABLE 7.2
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Sierra Madre Connected	0.6	7.3
Verdugo	3.8	6.9
Sierra Madre (San Fernando)	4.8	6.7
San Gabriel	6.6	7.3
Raymond	7.5	6.8
Hollywood	7.6	6.7
Santa Monica Connected alt 2	8.8	7.4
Elysian Park (Upper)	8.8	6.7
Northridge	9.8	6.9
Clamshell-Sawpit	14.0	6.7

The faults tabulated above and numerous other faults in the region are sources of potential ground motion. However, earthquakes that might occur on other faults throughout California are also potential generators of significant ground motion and could subject the site to intense ground shaking.

7.3 Geologic Hazards

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

7.3.1 Landslides

The site vicinity is gently sloping to the southeast at an approximate grade of 7 percent. There are no known landslides at the site, nor is the site in the path of any known or potential landslides (see Seismic Hazard Map, Figure 5). We do not consider the potential for a landslide to be a hazard to this project.

7.3.2 Flooding

Based on Lops Angeles FEMA Flood Map, the subject site is located in an area of minimal flood hazard (see Flood Zone Map, Figure 6). According to the California Department of Water Resources, Division of Safety of Dams (Dam Breach Inundation Map Web Publisher), the subject site is not located within a dam inundation zone.

7.3.3 Volcanic Activity

California includes six regions with a history of late Pleistocene and Holocene volcanic eruptions that are subject to hazards from future eruptions (Miller, 1989). Of these six regions, Amboy Crater Lavic Lake area is the closest. This area is located about 140 miles east/northeast of the site. Based on review of Plate 1, Miller 1989, the subject site is not located within any designated volcanic hazard zones.

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.

7.3.4 Expansive Soils

One of the potential geotechnical hazards evaluated at this site is the expansion potential of the near surface soils. Expansive soils experience shrink and swell due to moisture content fluctuations throughout the dry and wet season. If not addressed, the potential for shrinkage and heave would have an impact on foundations and lightly loaded slabs. The potential for damage to slabs-on-grade and foundations supported on expansive soils can be reduced by placing non-expansive fill below the slabs-on-grade.

Based on the laboratory tests performed the near surface soils, the expansion potential for the near surface soils is expected to be very low. Based on the very low expansive nature of the soils encountered, mitigation for heave due to expansive soils is not required.

7.3.5 Corrosion Protection

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 water-soluble chloride concentration detected in saturation extract from the soil samples was 17 mg/kg. This level of chloride concentration is considered to be slightly corrosive.

7.3.6 Sulfate Attack of Concrete

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate levels and how they relate to cement reactivity with soil and/or water. A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration. ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class.

The water-soluble sulfate concentration in the saturation extract from the soil sample was detected to be 100 mg/kg (0.0100 Percent by weight). Therefore, the potential for sulfate attack on concrete is considered negligible.

8. CONDITIONAL GEOLOGIC HAZARDS:

Conditional geologic hazards, as identified in section 31 of California Geological Survey Note 48, are discussed in the following subsections.

8.1 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

8.2 Hazardous Materials

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.

8.3 Radon Gas

Based on review of the EPA Map of Radon Zones, the site is located in an area identified as having indoor radon screening levels ranging from 2 to 4 pCi/L (https://www.epa.gov/radon/find-information-about-local-radon-zones-and-state-contact-information#radonmap). Given the site is expected to experience less than 4 pCi/L, the site is less than the recommended EPA's recommended action level for radon exposure. Provided the buildings are constructed with adequate ventilation, radon exposure is not considered a concern.

8.4 Naturally Occurring Asbestos

Asbestos commonly occurs in soil and ultramafic rocks such as serpentinite throughout California. Ultramafic rocks are scattered throughout much of the Sierra Nevada Mountain and the Coast Range regions. Based on review of the Open-File Report 2000-19, titled A General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos, prepared by the State of California Department of Conservation, Division of Mines and Geology, dated August, 2000, the closest ultramafic rock is identified in the Santa Barbara area about 80 miles northwest of the site. Based on the cited literature and our site observations, it is our opinion that the potential to encounter near surface naturally occurring asbestos containing rock at the site is negligible.

8.5 Hydrocollapse

Collapsible soils typically consist of loose, dry, low-density soils that, when wetted, will experience settlement/consolidation. Based on the results of testing performed on two (2) relatively undisturbed near surface soil samples, when wetted under a load of 1.6 kips per square foot these soils exhibited less than 1 percent collapse. Based on the results of the testing performed, provided the recommendations to support foundations on a uniform layer of engineered fill are followed, the potential for hydrocollapse of the near surface soils is considered slight. The results of the laboratory testing are included in Appendix B of this report.

8.6 Regional Subsidence

Based on our review of an online map published by California Water Science Center, the site is not located in an area of recorded subsidence (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html).

9. SEISMIC HAZARDS

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

9.1 Active Faulting and Surface Fault Rupture

Based on mapping and historical seismicity, the seismicity of the project area has been generally considered moderate to high by the scientific community. The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards nor within an Alquist-Priolo Earthquake Fault (Special Studies) Zone. Therefore, a site specific fault study investigation by an Engineering Geologist is not required. No active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. The nearest faults to the project site are associated with the Sierra Madre (Connected) Fault located approximately 0.6 mile northeast of the site. There are no known active fault traces in the immediate project vicinity. A map depicting the major active faults in the vicinity of the site is included on Figure 4. Considering the distance to the nearest known active fault, the potential for surface fault rupture at the site due to a known active fault is considered low.

9.2 Historic Seismic Activity

The general area of the site has experienced recurring seismic activity. Based on historical earthquake data obtained from the U.S. Geological Survey's earthquake database system, approximately 112 historical earthquakes with magnitude 4.5 or greater have been recorded from 1900 through September 18, 2020 within about 50 miles of the site. A map showing the location of the project site with relation to the approximate historical earthquake epicenter locations and magnitude category is presented in the Epicenter Location Map, Figure 7 at the end of this report.

The nearest earthquake event (estimated magnitude of 5.0) found during the search occurred on December 3, 1988, with a reported location of approximately 8 miles southeast of the site. The highest magnitude earthquake identified within a 50 mile search radius was the 6.7 magnitude Northridge earthquake, which occurred on January 17, 1994, approximately 17 miles west of the site (peak ground acceleration in the vicinity of the site of about 0.26g).

9.3 Design Seismic Ground Motion Parameters and Site Class

Seismic coefficients and spectral response acceleration values were developed based on the 2019 California Building Code (CBC). The CBC methodology for determining design ground motion values is based on the Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, which incorporate both probabilistic and deterministic seismic ground motion.

Based on the 2019 CBC, a Site Class C represents the on-site soil conditions with standard penetration resistance, N-values, greater than 50 blows per foot in the upper 100 feet below site grade. A table providing the recommended design acceleration parameters for the project site, based on a Site Class C designation, is included in section 10.2 of this report.

Based on Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.997g (based on both probabilistic and deterministic seismic ground motion).

9.4 Liquefaction and Seismic Settlement

Soil liquefaction is a state of soil particles suspension caused by a complete loss of strength when the effective stress drops to zero. Liquefaction normally occurs under saturated conditions in soils such as sand in which the strength is purely frictional. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile. However, liquefaction has occurred in soils other than clean sand.

The soils encountered within the depth of 47 feet on the project site consisted predominately of medium dense to very dense silty sand with various amounts of gravel and gravelly sand with silt. The historically highest groundwater is estimated to be at a depth of greater than 50 feet below ground surface according to the Seismic Hazard Zone Report 014, Pasadena 7.5-Minute Quadrangle, Plate 1.2 (Open-File Report 98-05) and other groundwater data.

Low to very low cohesion strength is associated with the sandy soil. A seismic hazard, which could cause damage to the proposed development during seismic shaking, is the post-liquefaction settlement of the liquefied sands.

In accordance with the State of California, Seismic Hazard Zone Map, Pasadena Quadrangle, dated March 25, 1999 the site is NOT located within the potential liquefaction zone. Based on the dense to very dense soil starting at least about 10 feet BSG, and lack of historic groundwater within the upper 50 feet BSG, the potential for liquefaction/seismic settlement to occur within the site is considered negligible.

9.5 Lateral Spreading

Lateral spreading is a phenomenon in which soils move laterally during seismic shaking and is often associated with liquefaction. The amount of movement depends on the soil strength, duration and intensity of seismic shaking, topography, and free face geometry. Due to the relatively flat site topography and low liquefaction potential, we judge the likelihood of lateral spreading to be low.

10. CONCLUSIONS AND RECOMMENDATIONS

10.1 General

10.1.1 Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed construction of improvements at the site as planned, provided the recommendations contained in this report are incorporated into the project design and construction. Conclusions and recommendations provided in this report are based on our review of available literature, analysis of data obtained from our field exploration and laboratory testing program, and our understanding of the proposed development at this time.

10.1.2 The primary geotechnical constraints identified in our investigation is the presence of near surface compressible material at the site. Recommendations to mitigate the effects of these soils are provided in this report.

10.1.3 Based on the soil/weathered bedrock conditions encountered, the site is designated as a Site Class C. Therefore, unless the structure is a seismically isolated structure or structure with seismic damping, a Site Specific Ground Motion Hazard Analysis would not be required. Based on our understanding of the proposed construction, the proposed building construction would not meet this requirement. Therefore, a Site Specific Ground Motion Hazard Analysis is not required for this project. In the event that a site specific ground motion analysis is required, SALEM should be contacted for these services.

10.1.4 The scope of this investigation did not include subsurface exploration within the existing building areas or areas not accessible to the drill rig. As such, subsurface soil conditions and materials present below the existing structures are unknown and may be different than those noted within this report. The likely presence of unacceptable fill materials, undocumented fill, and/or loose soil material that may be resent below existing structures, in addition to the areas identified as part of this investigation, shall be taken into consideration. Our firm should be consulted at the time of demolition activities if soil conditions not consistent with those identified as part of this investigation are encountered so that we can provide additional recommendations as needed.

10.1.5 Site demolition activities shall include removal of all surface obstructions not intended to be incorporated into final site design. In addition, underground buried structures and/or utility lines encountered during demolition and construction should be properly removed and the resulting excavations backfilled with Engineered Fill. It is suspected that possible demolition activities of the existing structures may disturb the upper soils. After demolition activities, it is recommended that disturbed soils be removed and/or recompacted.

10.1.6 The scope of our services for the investigation did not include a slope stability evaluation of the site. For the proposed buildings adjacent to the descending slopes, a setback equals to one-third (1/3) of the slope height but needs not exceed 40 feet should be provided between the footing bottom and the slope face. If the slope is steeper than 1:1 (horizontal to vertical), the required setback should be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

10.1.7 The horizontal distance between the outer edges of the footing bottom and the adjacent slope face should be at least 7 feet.

10.1.8 The near-surface onsite soils are moisture-sensitive and are moderately compressible under saturated conditions. Excessive post-construction settlement may be experienced by the proposed structures if foundation soils become near saturated. The compressible or weak soils should be removed and recompacted according to the recommendations in the Grading section of this report (Section 10.5).

10.1.9 Based on the subsurface conditions at the site and the anticipated structural loading, we anticipate that the proposed building may be supported using conventional shallow foundations provided that the recommendations presented herein are incorporated in the design and construction of the project.

10.1.10 Provided the site is graded in accordance with the recommendations of this report and foundations constructed as described herein, we estimate that total settlement due to static and seismic loads utilizing conventional shallow foundations for the proposed building will be within 1 inch and the corresponding differential settlement will be less than ½ inch.

10.1.11 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

10.1.12 SALEM shall review the project grading and foundation plans prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required. If SALEM is not provided plans and specifications for review, we cannot assume any responsibility for the future performance of the project.

10.1.13 SALEM shall be present at the site during site demolition and preparation to observe site clearing/demolition, preparation of exposed surfaces after clearing, and placement, treatment and compaction of fill material.

10.1.14 SALEM's observations should be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. Moisture content of footings and slab subgrade should be tested immediately prior to concrete placement. SALEM should observe foundation excavations prior to placement of reinforcing steel or concrete to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report.

10.2 Site Specific Seismic Design Criteria

10.2.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2019 CBC, our recommended parameters are shown below. These parameters were determined using California's Office of Statewide Health Planning and Development (OSHPD) Seismic Design Map Tool Website (https://seismicmaps.org/) in accordance with the 2019 CBC.

TABLE 10.2.1
SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2016 ASCE 7-16 or 2019 CBC Reference
Site Coordinates (Datum = NAD 83)		34.2301 Lat -118.2361 Lon	
Site Class	--	C	ASCE 7 Table 20.3-1
Soil Profile Name	--	Very Dense Soil	ASCE 7 Table 20.3-1
Risk Category	--	II	CBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.200	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.997g	ASCE 7 Equation 11.8-1
Seismic Design Category	SDC	D	CBC Table 1613.2.5
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	1.952g	CBC Figure 1613.2.1(1-8)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.730g	CBC Figure 1613.2.1(1-8)
Site Class Modified Site Coefficient	F_a	1.200	CBC Table 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	1.400	CBC Table 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	2.342 g	CBC Equation 16-36
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	1.022 g	CBC Equation 16-37
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.561 g	CBC Equation 16-38
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	0.681 g	CBC Equation 16-39

10.2.2 Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, seismically isolated structures and structures with damping systems on site with S_1 greater than or equal to 0.6 may require Site Specific Ground Motion Hazard Analysis (SSGMHA). Based on our understanding of the proposed construction, SSGMHA would not be required. In the event that a site specific ground motion analysis is required, SALEM should be contacted for these services.

10.2.3 Conformance to the criteria in the above table for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

10.3 Soil and Excavation Characteristics

10.3.1 Based on the soil conditions encountered in our soil borings, the onsite soils can be excavated with moderate to greater effort using conventional or heavy-duty earthmoving equipment.

10.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

10.3.3 The near surface soils identified as part of our investigation are, generally slightly moist to moist due to the absorption characteristics of the soil. Earthwork operations may encounter very moist unstable soils which may require removal to a stable bottom. Exposed native soils exposed as part of site grading operations shall not be allowed to dry out and should be kept continuously moist prior to placement of subsequent fill.

10.4 Materials for Fill

10.4.1 Excavated soils generated from cut operations at the site are suitable for use as general Engineered Fill in structural areas, provided they do not have an Expansion Index greater than 20 (EI≤20) and do not contain deleterious matter, organic material, or rock material larger than 3 inches in maximum dimension.

10.4.2 The preferred materials specified for Engineered Fill are suitable for most applications with the exception of exposure to erosion. Project site winterization and protection of exposed soils during the construction phase should be the sole responsibility of the Contractor, since they have complete control of the project site.

10.4.3 Import soil shall be well-graded, slightly cohesive silty fine sand or sandy silt, with relatively impervious characteristics when compacted. A clean sand or very sandy soil is not acceptable for this purpose. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 10.4.3.

TABLE 10.4.3
IMPORT FILL REQUIREMENTS

Minimum Percent Passing No. 200 Sieve	15
Maximum Percent Passing No. 200 Sieve	50
Minimum Percent Passing No. 4 Sieve	80
Maximum Particle Size	3"
Maximum Plasticity Index	10
Maximum CBC Expansion Index	15

10.4.4 Prior to importing fill, the Contractor shall submit test data that demonstrates that the proposed import complies with the recommended criteria for both geotechnical and environmental compliance. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the School District) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, analytical data specific to the source of the import material in accordance with the Department of Toxic Substances Control, "Informational Advisory, Clean Imported Fill Material," dated October 2001. The list of constituents to be tested for the fill source shall be submitted to FUSD for review and approval prior to the Contractor testing the fill. Contractors should provide a minimum of 14 working days after sample collection to complete the DTSC and geotechnical testing.

10.4.5 Environmental characteristics and corrosion potential of import soil materials should also be considered.

10.4.6 Proposed import materials should be sampled, tested, and approved by SALEM prior to its transportation to the site.

10.4.7 Aggregate base material should meet the requirements of a Caltrans Class 2 Aggregate Base. The aggregate base material should conform to the requirements of Section 26 of the Standard Specifications for Class 2 material, ¾-inch or 1½-inches maximum size. The aggregate base material should be compacted to a minimum relative compaction of 95 percent based ASTM D1557. The aggregate base material should be spread in layers not exceeding 6 inches and each layer of aggregate material course should be tested and approved by the Soils Engineer prior to the placement of successive layers.

10.5 Grading

10.5.1 A representative of our firm should be present during all site clearing and grading operations to test and observe earthwork construction. This testing and observation is an integral part of our service as acceptance of earthwork construction is dependent upon compaction of the material and the stability of the material. The Geotechnical Engineer may reject any material that does not meet compaction and stability requirements. Further recommendations of this report are predicated upon the assumption that earthwork construction will conform to recommendations set forth in this section as well as other portions of this report.

10.5.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance.

10.5.3 Site demolition activities shall include removal of all surface obstructions not intended to be incorporated into final site design. In addition, underground buried structures and/or utility lines encountered during demolition and construction should be properly removed and the resulting excavations backfilled with Engineered Fill. After demolition activities, it is recommended that disturbed soils be removed and/or recompacted.

10.5.4 Site preparation should begin with removal of existing surface/subsurface structures, underground utilities (as required), any existing uncertified fill, and debris. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report.

10.5.5 Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. The upper 2 to 4 inches of the soils containing, vegetation, roots and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. In addition, existing concrete and asphalt materials shall be removed from areas of proposed improvements and stockpiled separately from excavated soil material. The stripped vegetation, asphalt and concrete materials will not be suitable for use as Engineered Fill or within 5 feet of building pads or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

10.5.6 Structural building pad areas should be considered as areas extending a minimum of 5 feet horizontally beyond the outside dimensions of building, including footings and non-cantilevered overhangs carrying structural loads.

10.5.7 Any undocumented fill materials encountered during grading should be removed and replaced with engineered fill. The actual depth of the overexcavation and recompaction should be determined by our field representative during construction.

10.5.8 To minimize post-construction soil movement and provide uniform support for the proposed building, overexcavation and recompaction within the proposed building areas should be performed to a minimum depth of **three (3) feet** below existing grade or **one (1) foot** below proposed footing bottom, whichever is deeper. The overexcavation and recompaction should also extend laterally to a minimum of 5 feet beyond the outer edges of the proposed footings.

10.5.9 Overexcavation and recompaction for building pads should also extend laterally to a minimum of 5 feet beyond the outer edges of proposed footings.

10.5.10 Prior to placement of fill soils, the upper 8 to 10 inches of native subgrade soils should be scarified, moisture-conditioned to near optimum moisture content and recompacted to a minimum of 95% of the maximum dry density based on ASTM D1557 Test Method latest edition.

10.5.11 All Engineered Fill (including scarified ground surfaces and backfill) should be placed in thin lifts which will allow for adequate bonding and compaction (typically 6 to 8 inches in loose thickness).

10.5.12 Engineered Fill soils should be placed, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction.

10.5.13 Within pavement areas, it is recommended that scarification, moisture conditioning and recompaction be performed to at least 12 inches below existing grade or finish grade, whichever is deeper. In addition, the upper 12 inches of final pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned to slightly above the optimum moisture content and compacted to at least 95% relative compaction.

10.5.14 An integral part of satisfactory fill placement is the stability of the placed lift of soil. If placed materials exhibit excessive instability as determined by a SALEM field representative, the lift will be considered unacceptable and shall be remedied prior to placement of additional fill material. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

10.5.15 Final pavement subgrade should be finished to a smooth, unyielding surface. We further recommend proof-rolling the subgrade with a loaded water truck (or similar equipment with high contact pressure) to verify the stability of the subgrade prior to placing aggregate base.

10.5.16 The most effective site preparation alternatives will depend on site conditions prior to grading. We should evaluate site conditions and provide supplemental recommendations immediately prior to grading, if necessary.

10.5.17 We do not anticipate groundwater or seepage to adversely affect construction if conducted during the drier months of the year (typically summer and fall). However, groundwater and soil moisture conditions could be significantly different during the wet season (typically winter and spring) as surface soil becomes wet; perched groundwater conditions may develop. Grading during this time period will likely encounter wet materials resulting in possible excavation and fill placement difficulties.

Project site winterization consisting of placement of aggregate base and protecting exposed soils during construction should be performed. If the construction schedule requires grading operations during the wet season, we can provide additional recommendations as conditions warrant.

10.5.18 Typical remedial measures include: discing and aerating the soil during dry weather; mixing the soil with dryer materials; removing and replacing the soil with an approved fill material or placement of crushed rocks or aggregate base material; or mixing the soil with an approved lime or cement product.

The most common remedial measure of stabilizing the bottom of the excavation due to wet soil condition is to reduce the moisture of the soil to near the optimum moisture content by having the subgrade soils scarified and aerated or mixed with drier soils prior to compacting. However, the drying process may require an extended period of time and delay the construction operation. To expedite the stabilizing process, crushed rock may be utilized for stabilization provided this method is approved by the owner for the cost purpose.

If the use of crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of ¾-inch to 1-inch crushed rocks. The thickness of the rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of crushed rock material will provide a stable platform. It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock. A layer of geofabric is recommended to be placed on top of the compacted crushed rock to minimize migration of soil particles into the voids of the crushed rock, resulting in soil movement. Although it is not required, the use of geogrid (e.g. Tensar TX 140) below the crushed rock will enhance stability and reduce the required thickness of crushed rock necessary for stabilization.

Our firm should be consulted prior to implementing remedial measures to provide appropriate recommendations.

10.6 Shallow Foundations

10.6.1 The site is suitable for use of conventional shallow foundations consisting of continuous footings and isolated pad footings bearing in properly compacted Engineered Fill.

10.6.2 The bearing wall footings considered for the structure should be continuous with a minimum width of 15 inches and extend to a minimum depth of 18 inches below the lowest adjacent grade. Isolated column footings should have a minimum width of 24 inches and extend a minimum depth of 18 inches below the lowest adjacent grade.

10.6.3 The bottom of footing excavations should be maintained free of loose and disturbed soil. Footing concrete should be placed into a neat excavation.

10.6.4 The horizontal distance between the outer edges of the footing bottom and the adjacent slope face should be at least 7 feet.

10.6.5 Footings proportioned as recommended above may be designed for the maximum allowable soil bearing pressures shown in the table below.

Loading Condition	Allowable Bearing
Dead Load Only	2,000 psf
Dead-Plus-Live Load	2,500 psf
Total Load, Including Wind or Seismic Loads	3,325 psf

10.6.6 For design purposes, total settlement due to static and seismic loadings on the order of 1 inch may be assumed for shallow footings. Differential settlement due to static and seismic loadings, along a 20-foot exterior wall footing or between adjoining column footings, should be ½ inch, producing an angular distortion of 0.002. Most of the settlement is expected to occur during construction as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated. The footing excavations should not be allowed to dry out any time prior to pouring concrete.

10.6.7 Resistance to lateral footing displacement can be computed using an allowable coefficient of friction factor of 0.45 acting between the base of foundations and the supporting subgrade.

10.6.8 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 420 pounds per cubic foot acting against the appropriate vertical native footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combination that includes wind or earthquake loads.

10.6.9 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

10.6.10 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Prior to placing rebar reinforcement, foundation excavations should be evaluated by a representative of SALEM for appropriate support characteristics and moisture content. Moisture conditioning may be required for the materials exposed at footing bottom, particularly if foundation excavations are left open for an extended period.

10.7 Concrete Slabs-on-Grade

10.7.1 Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that non-structural slabs-on-grade be at least 4 inches thick and underlain by six (6) inches of compacted granular aggregate subbase material compacted to at least 95% relative compaction.

10.7.2 Granular aggregate subbase material shall conform to ASTM D-2940, Latest Edition (Table 1, bases) with at least 95 percent passing a 1½-inch sieve and not more than 8% passing a No. 200 sieve or its approved equivalent to prevent capillary moisture rise. Crushed Miscellaneous Base (CMB) or any base materials containing asphalt should not be used as subbase material within the building pad area.

10.7.3 We recommend reinforcing slabs, at a minimum, with No. 3 reinforcing bars placed 18 inches on center, each way.

10.7.4 Slabs subject to structural loading may be designed utilizing a modulus of subgrade reaction K of 180 pounds per square inch per inch. The K value was approximated based on inter-relationship of soil classification and bearing values (Portland Cement Association, Rocky Mountain Northwest).

10.7.5 The spacing of crack control joints should be designed by the project structural engineer. In order to regulate cracking of the slabs, we recommend that construction joints or control joints be provided at a maximum spacing of 15 feet in each direction for 5-inch thick slabs and 12 feet for 4-inch thick slabs.

10.7.6 Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. The exterior floors should be poured separately in order to act independently of the walls and foundation system.

10.7.7 It is recommended that the utility trenches within the structure be compacted, as specified in our report, to minimize the transmission of moisture through the utility trench backfill. Special attention to the immediate drainage and irrigation around the structures is recommended.

10.7.8 Moisture within the structure may be derived from water vapors, which were transformed from the moisture within the soils. This moisture vapor penetration can affect floor coverings and produce mold and mildew in the structure. To minimize moisture vapor intrusion, it is recommended that a vapor retarder be installed in accordance with manufacturer's recommendations and/or ASTM guidelines, whichever is more stringent. In addition, ventilation of the structure is recommended to reduce the accumulation of interior moisture.

10.7.9 In areas where it is desired to reduce floor dampness where moisture-sensitive coverings, coatings, underlayments, adhesives, moisture sensitive goods, humidity controlled environments, or climate cooled environments are anticipated, construction should have a suitable waterproof vapor retarder (a minimum of 15 mils thick, is recommended, polyethylene vapor retarder sheeting, Raven Industries "VaporBlock 15, Stego Industries 15 mil "StegoWrap" or W.R. Meadows Sealtight 15 mil "Perminator") incorporated into the floor slab design. The water vapor retarder should be a decay resistant material complying with ASTM E96 or ASTM E1249 not exceeding 0.01 perms, ASTM E154 and ASTM E1745 Class A. The vapor retarder should, maintain the recommended presence after conditioning tests per ASTM E1745. The vapor barrier should be placed between the concrete slab and the compacted granular aggregate subbase material. The water vapor retarder (vapor barrier) should be installed in accordance with ASTM Specification E 1643-18.

10.7.10 The concrete may be placed directly on vapor retarder. The vapor retarder should be inspected prior to concrete placement. Cut or punctured retarder should be repaired using vapor retarder material lapped 6 inches beyond damaged areas and taped.

10.7.11 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive.

10.7.12 The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

10.7.13 Proper finishing and curing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

10.8 Lateral Earth Pressures and Frictional Resistance

10.8.1 Active, at-rest and passive unit lateral earth pressures against footings and walls are summarized in the table below:

Lateral Pressures – Level Backfill and Drained Conditions	Equivalent Fluid Pressure, pcf
Active Pressure	34
At-Rest Pressure	53
Passive Pressure	420
Related Parameters	
Allowable Coefficient of Friction	0.45
In-Place Soil Density (lbs/ft^3)	120

10.8.2 Active pressure applies to walls, which are free to rotate. At-rest pressure applies to walls, which are restrained against rotation. The preceding lateral earth pressures assume sufficient drainage behind retaining walls to prevent the build-up of hydrostatic pressure.

10.8.3 The top one-foot of adjacent subgrade should be deleted from the passive pressure computation.

10.8.4 The foregoing values of lateral earth pressures represent allowable soil values and a safety factor consistent with the design conditions should be included in their usage.

10.8.5 For stability against lateral sliding, which is resisted solely by the passive pressure, we recommend a minimum safety factor of 1.5.

10.8.6 For stability against lateral sliding, which is resisted by the combined passive and frictional resistance, a minimum safety factor of 2.0 is recommended.

10.8.7 For lateral stability against seismic loading conditions, we recommend a minimum safety factor of 1.1.

10.8.8 For dynamic seismic lateral loading the following equation shall be used:

Dynamic Seismic Lateral Loading Equation
Dynamic Seismic Lateral Load = $\frac{3}{8}\gamma K_h H^2$
Where: γ = In-Place Soil Density
K_h = Horizontal Acceleration = $\frac{2}{3}PGA_M$
H = Wall Height

10.9 Retaining Walls

10.9.1 Retaining and/or below grade walls should be drained with either perforated pipe encased in free-draining gravel or a prefabricated drainage system. The gravel zone should have a minimum width of 12 inches wide and should extend upward to within 12 inches of the top of the wall. The upper 12 inches of backfill should consist of native soils, concrete, asphaltic-concrete or other suitable backfill to minimize surface drainage into the wall drain system. The gravel should conform to Class II permeable materials graded in accordance with the current CalTrans Standard Specifications.

10.9.2 Prefabricated drainage systems, such as Miradrain®, Enkadrain®, or an equivalent substitute, are acceptable alternatives in lieu of gravel provided they are installed in accordance with the manufacturer's recommendations. If a prefabricated drainage system is proposed, our firm should review the system for final acceptance prior to installation.

10.9.3 Drainage pipes should be placed with perforations down and should discharge in a non-erosive manner away from foundations and other improvements. The top of the perforated pipe should be placed at or below the bottom of the adjacent floor slab or pavements. The pipe should be placed in the center line of the drainage blanket and should have a minimum diameter of 4 inches. Slots should be no wider than 1/8-inch in diameter, while perforations should be no more than ¼-inch in diameter.

10.9.4 If retaining walls are less than 5 feet in height, the perforated pipe may be omitted in lieu of weep holes on 4 feet maximum spacing. The weep holes should consist of 2-inch minimum diameter holes (concrete walls) or unmortared head joints (masonry walls) and placed no higher than 18 inches above the lowest adjacent grade. Two 8-inch square overlapping patches of geotextile fabric (conforming to the CalTrans Standard Specifications for "edge drains") should be affixed to the rear wall opening of each weep hole to retard soil piping.

10.9.5 During grading and backfilling operations adjacent to any walls, heavy equipment should not be allowed to operate within a lateral distance of 5 feet from the wall, or within a lateral distance equal to the wall height, whichever is greater, to avoid developing excessive lateral pressures. Within this zone, only hand operated equipment ("whackers," vibratory plates, or pneumatic compactors) should be used to compact the backfill soils.

10.10 Temporary Excavations

10.10.1 We anticipate that the majority of the sandy site soils will be classified as Cal-OSHA "Type C" soil when encountered in excavations during site development and construction. Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and make appropriate recommendations where necessary.

10.10.2 It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements. All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load.

10.10.3 Temporary excavations and slope faces should be protected from rainfall and erosion. Surface runoff should be directed away from excavations and slopes.

10.10.4 Open, unbraced excavations in undisturbed soils should be made according to the slopes presented in the following table:

RECOMMENDED EXCAVATION SLOPES

Depth of Excavation (ft)	Slope (Horizontal : Vertical)
0-5	1½:1
5-10	2½:1

10.10.5 If, due to space limitation, excavations near property lines or existing structures are performed in a vertical position, slot cuts, braced shorings or shields may be used for supporting vertical excavations. Therefore, in order to comply with the local and state safety regulations, a properly designed and installed shoring system would be required to accomplish planned excavations and installation. A Specialty Shoring Contractor should be responsible for the design and installation of such a shoring system during construction.

10.10.6 Braced shorings should be designed for a maximum pressure distribution of 30H, (where H is the depth of the excavation in feet). The foregoing does not include excess hydrostatic pressure or surcharge loading. Fifty percent of any surcharge load, such as construction equipment weight, should be added to the lateral load given herein. Equipment traffic should concurrently be limited to an area at least 3 feet from the shoring face or edge of the slope.

10.10.7 The excavation and shoring recommendations provided herein are based on soil characteristics derived from the borings within the area. Variations in soil conditions will likely be encountered during the excavations. SALEM Engineering Group, Inc. should be afforded the opportunity to provide field review to evaluate the actual conditions and account for field condition variations not otherwise anticipated in the preparation of this recommendation. Slope height, slope inclination, or excavation depth should in no case exceed those specified in local, state, or federal safety regulation, (e.g. OSHA) standards for excavations, 29 CFR part 1926, or Assessor's regulations.

10.11 Underground Utilities

10.11.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than 3 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches and compacted to at least 95% relative compaction at or above optimum moisture content.

10.11.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to approximately 12 inches above the crown of the pipe. Pipe bedding, haunches and initial fill extending to 1 foot above the pipe should consist of a clean well graded sand with 100 percent passing the #4 sieve, a maximum of 15 percent passing the #200 sieve, and a minimum sand equivalent of 20.

10.11.3 It is suggested that underground utilities crossing beneath new or existing structures be plugged at entry and exit locations to the buildings or structures to prevent water migration. Trench plugs can consist of on-site clay soils, if available, or sand cement slurry. The trench plugs should extend 2 feet beyond each side of individual perimeter foundations.

10.11.4 The contractor is responsible for removing all water-sensitive soils from the trench regardless of the backfill location and compaction requirements. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

10.12 Surface Drainage

10.12.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

10.12.2 Site drainage should be collected and transferred away from improvements in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers are not permitted onto unprotected soils within five feet of the buildings perimeters. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within 5 feet of the buildings perimeter footings should be kept to a minimum to just support vegetative life.

10.12.3 Roof drains should be installed with appropriate downspout extensions out-falling on splash blocks so as to direct water a minimum of 5 feet away from the structures or be connected to the storm drain system for the development.

10.13 Pavement Design

10.13.1 Based on site soil conditions and laboratory test results, an R-value of 50 was used for the preliminary flexible asphaltic concrete pavement design. The R-value may be verified during grading of the pavement areas.

10.13.2 If the design pavement is less than the existing pavement section, the new pavement section should match the existing pavement section. Based on the borings, the existing pavement section was approximately 3 inches of AC underlain by 3 inches of AB.

10.13.3 The pavement design recommendations provided herein are based on the State of California Department of Transportation (CALTRANS) design manual. The following table shows the recommended pavement sections for various traffic indices.

TABLE 10.13.3
ASPHALT CONCRETE PAVEMENT

Traffic Index	Asphaltic Concrete	Clean Class II Aggregate Base*	Compacted Subgrade*
5.0 (Parking and Vehicle Drive Areas)	3.0"	4.0"	12.0"
6.0 (Heavy Truck Areas)	3.0"	4.0"	12.0"

95% compaction based on ASTM D1557-07 Test Method

10.13.4 The following recommendations are for light-duty and heavy-duty Portland Cement Concrete pavement sections.

TABLE 10.13.4
PORTLAND CEMENT CONCRETE PAVEMENT

Traffic Index	Portland Cement Concrete*	Clean Class II Aggregate Base**	Compacted Subgrade**
5.0 (Light Duty)	5.0"	4.0"	12.0"
6.0 (Heavy Duty)	6.0"	4.0"	12.0"

**** Minimum Compressive Strength of 4,000 psi; Minimum Reinforcement of #4 Bars at 18" O.C. Each Way***
***** 95% compaction based on ASTM D1557-07 Test Method***

11. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

11.1 Plan and Specification Review

11.1.1 SALEM should review the project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

11.2 Construction Observation and Testing Services

11.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

11.2.2 SALEM should be present at the site during site preparation to observe site clearing, preparation of exposed surfaces after clearing, and placement, treatment and compaction of fill material.

11.2.3 SALEM's observations should be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. Moisture content of footings and slab subgrade should be tested immediately prior to concrete placement. SALEM should observe foundation excavations prior to placement of reinforcing steel or concrete to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report.

12. LIMITATIONS AND CHANGED CONDITIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the test borings drilled at the approximate locations shown on the Site Plan, Figure 2. The report does not reflect variations which may occur between borings. The nature and extent of such variations may not become evident until construction is initiated.

If variations then appear, a re-evaluation of the recommendations of this report will be necessary after performing on-site observations during the excavation period and noting the characteristics of such variations. The findings and recommendations presented in this report are valid as of the present and for the proposed construction.

If site conditions change due to natural processes or human intervention on the property or adjacent to the site, or changes occur in the nature or design of the project, or if there is a substantial time lapse between the submission of this report and the start of the work at the site, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed by SALEM and the conclusions of our report are modified or verified in writing. The validity of the recommendations contained in this report is also dependent upon an adequate testing and observations program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction. SALEM has prepared this report for the exclusive use of the owner and project design consultants.

SALEM does not practice in the field of corrosion engineering. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, that manufacturer's recommendations for corrosion protection be closely followed. Further, a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of concrete slabs and foundations in direct contact with native soil. The importation of soil and or aggregate materials to the site should be screened to determine the potential for corrosion to concrete and buried metal piping. The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (909) 980-6455.

Respectfully Submitted,

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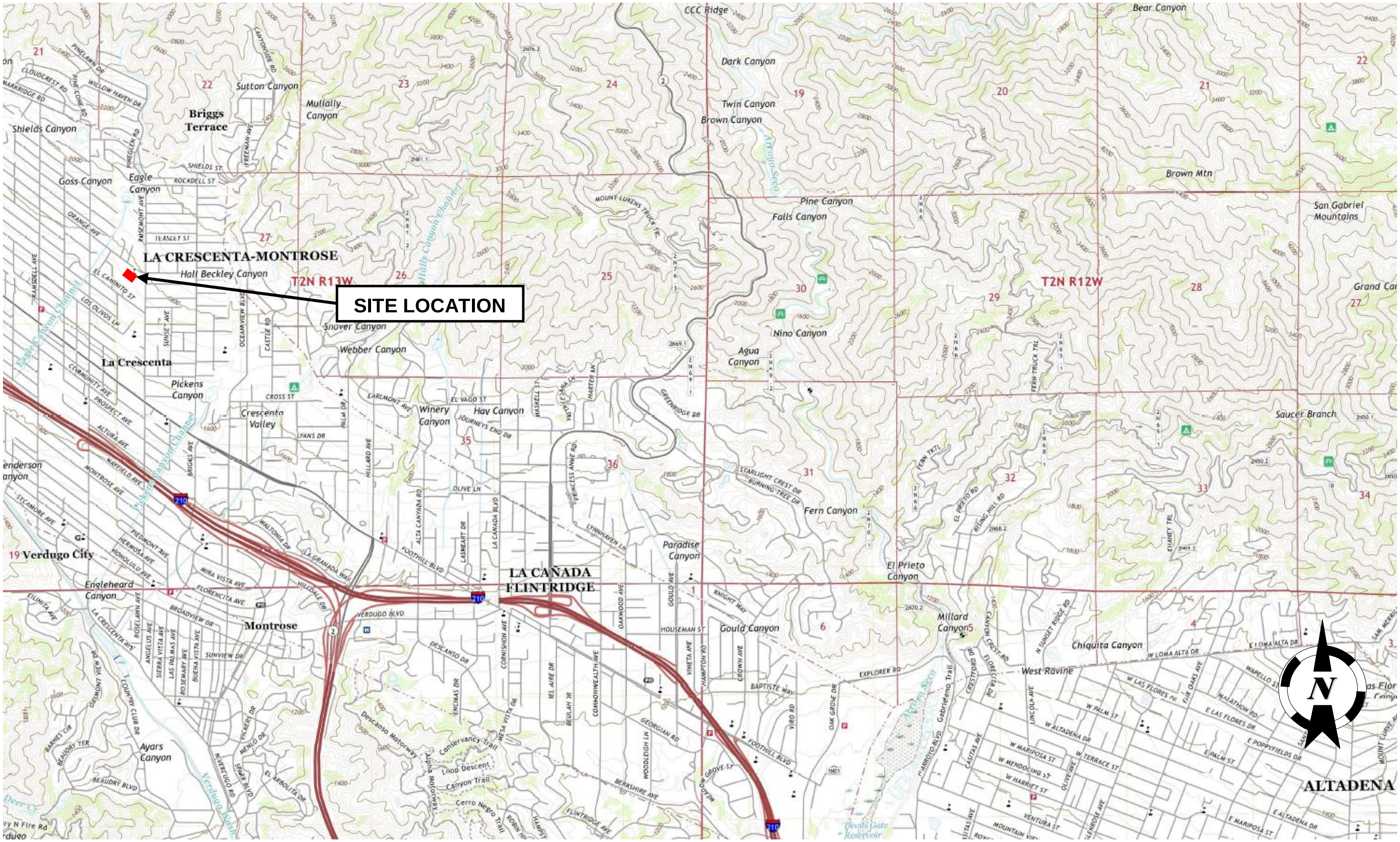
Exp 12/31/21
STATE OF CALIFORNIA



Source Image: U.S. Geologic Survey, Pasadena, California, https://ngmdb.usgs.gov/topoview (2018)

VICINITY MAP	SCALE: NOT TO SCALE	DATE: 09/2020	SALEM engineering group, inc.
GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION Proposed Classroom Building Monte Vista Elementary School – 2620 Orange Avenue La Crescenta-Montrose, California	DRAWN BY: KV	APPROVED BY: CJ	
	PROJECT NO. 3-220-0617	FIGURE NO. 1	

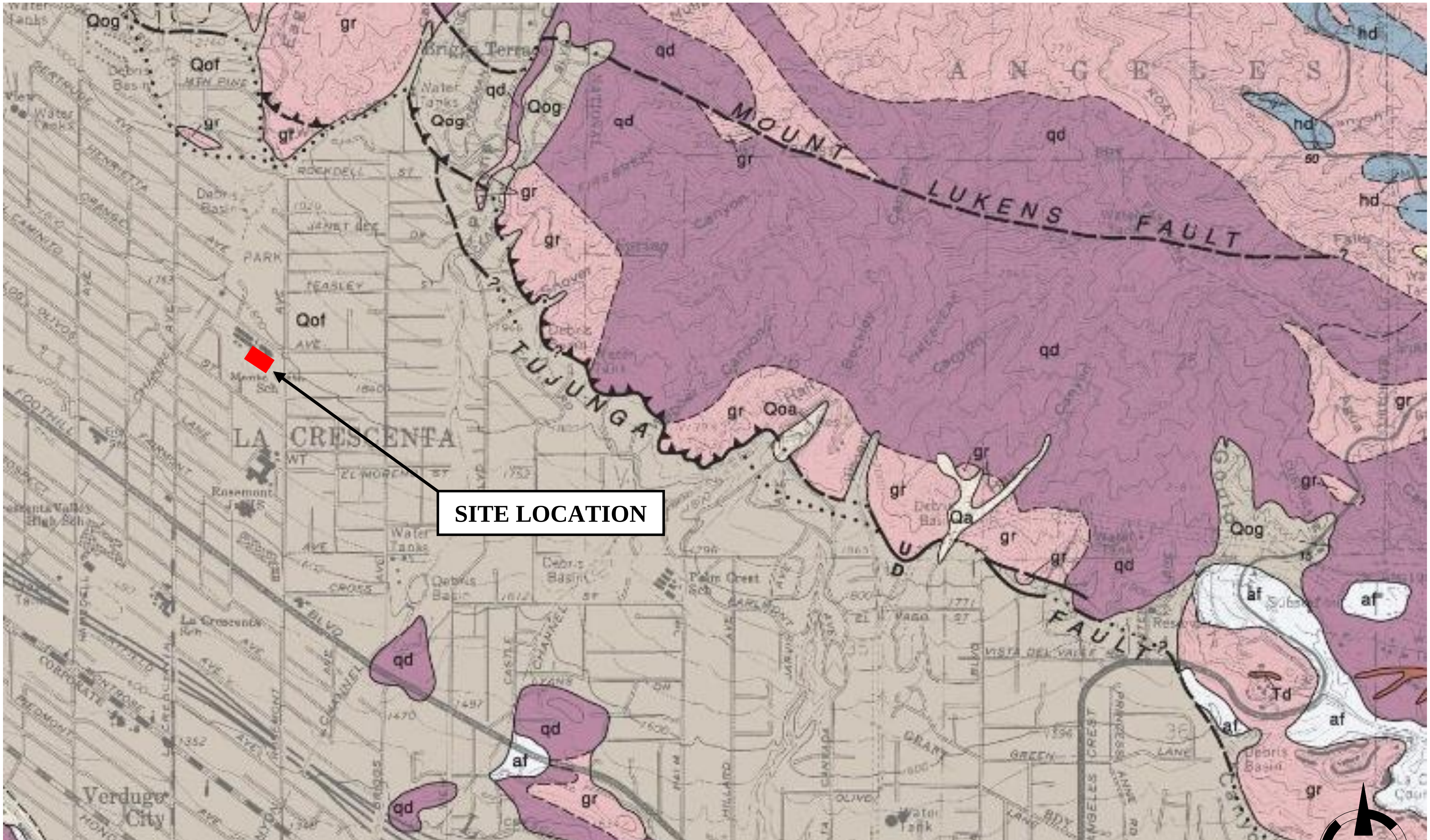
SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION
Proposed Classroom Building
Monte Vista Elementary School – 2620 Orange Avenue
La Crescenta-Montrose, California

SCALE:	DATE:
NOT TO SCALE	09/2020
DRAWN BY: KV	APPROVED BY: CJ
PROJECT NO. 3-220-0617	FIGURE NO. 2

LEGEND:
B-1 Soil Boring Locations
All Locations Approximate

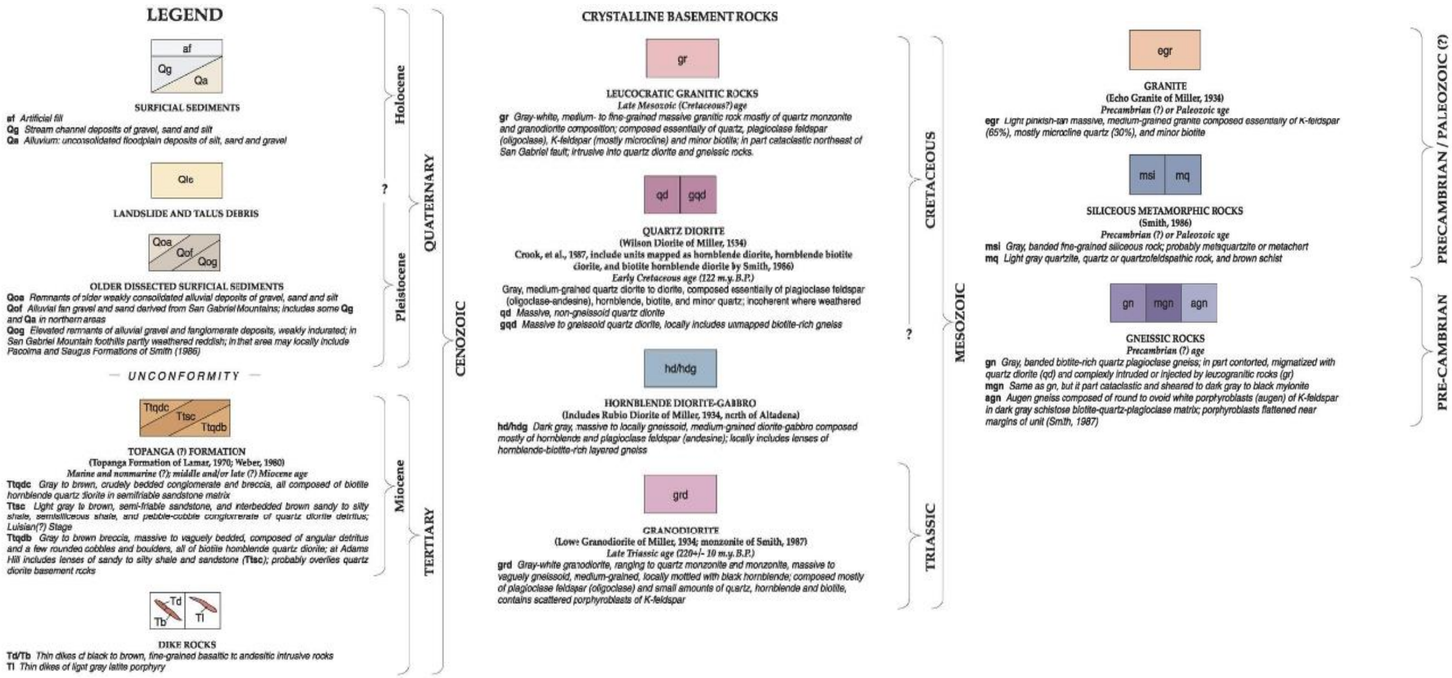
SALEM engineering group, inc.



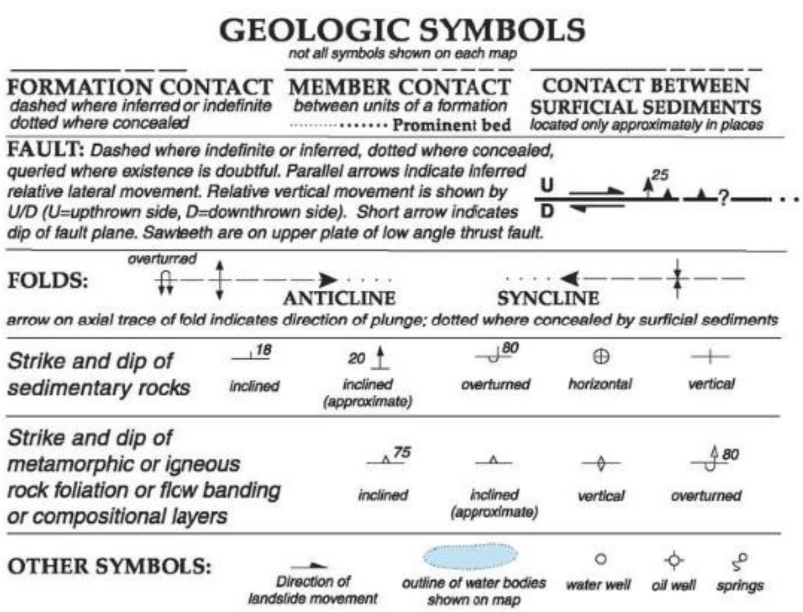
Dibblee, Jr, Thomas W., (1989), Minch, John A., (2010), Geologic map of the Pasadena 7.5' quadrangle, Los Angeles County, California: U.S. Geological Survey, scale 1:24,000.

Regional Geologic Map **GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION** **Proposed Classroom Building** **Monte Vista Elementary School – 2620 Orange Avenue** **La Crescenta-Montrose, California**	SCALE: NOT TO SCALE	DATE: 09/2020	SALEM engineering group, inc.
	DRAWN BY: JC	APPROVED BY: CJ	
	PROJECT NO. 3-220-0617	FIGURE NO. 3A	

Geologic Unit Explination



Symbol Explination



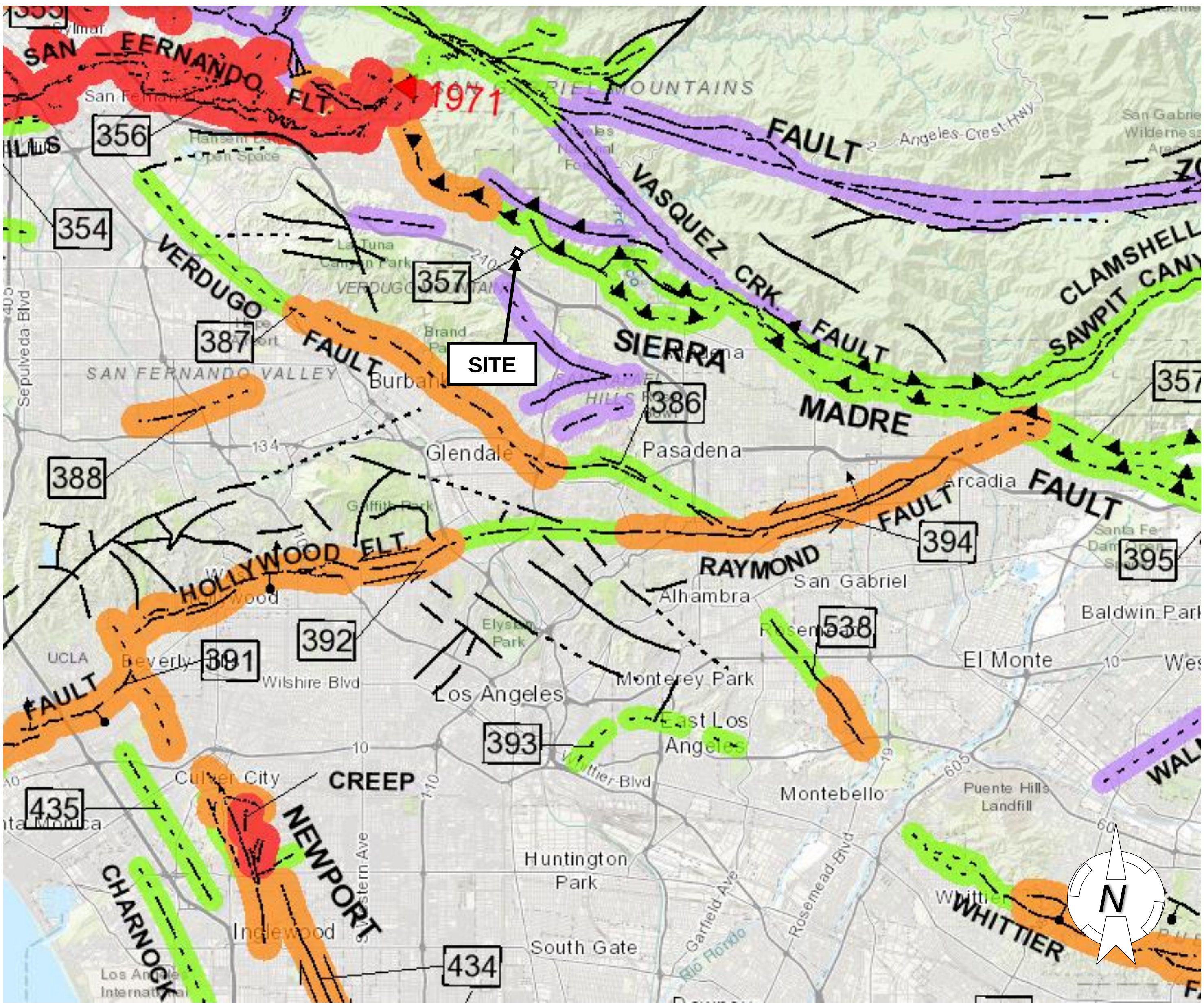
Dibblee, Jr, Thomas W., (1989), Minch, John A., (2010), Geologic map of the Pasadena 7.5' quadrangle, Los Angeles County, California: U.S. Geological Survey, scale 1:24,000.

Regional Geologic Map

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION
Proposed Classroom Building
Monte Vista Elementary School – 2620 Orange Avenue
La Crescenta-Montrose, California

SCALE:	DATE:
NOT TO SCALE	09/2020
DRAWN BY:	APPROVED BY:
JC	CJ
PROJECT NO.	FIGURE NO.
3-220-0617	3B

SALEM engineering group, inc.



FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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

1906 ▶ ◀ 1906

1838 ▷ ◁ 1838

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

▶ 1951 ◀

Date bracketed by triangles indicates local fault break.

1992

No triangle by date indicates an intermediate point along faultbreak.

CREEP

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

1969

1968 ■ ■ 1968

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

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

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

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate).

OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

Regional Fault Map

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION
Proposed Classroom Building
Monte Vista Elementary School – 2620 Orange Avenue
La Crescenta-Montrose, California

SCALE: NOT TO SCALE	DATE: 09/2020
DRAWN BY: JC	APPROVED BY: CJ
PROJECT NO. 3-220-0617	FIGURE NO. 4

SALEM engineering group, inc.

MAP EXPLANATION

Zones of Required Investigation:

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Overlapping Liquefaction and Earthquake-Induced Landslides
Areas that lie within zones of required investigation for both liquefaction and earthquake-induced landslides. (See above for explanation of each zone.)

IMPORTANT - PLEASE NOTE

1) This map may not show all areas that have the potential for liquefaction, landsliding, strong earthquake ground shaking or other earthquake and geologic hazards. Also, a single earthquake capable of causing liquefaction or triggering landslide failure will not uniformly affect the entire area zoned.

2) Liquefaction zones may also contain areas susceptible to the effects of earthquake-induced landslides. This situation typically exists at or near the toe of existing landslides, downslope from rockfall or debris flow source areas, or adjacent to steep stream banks.

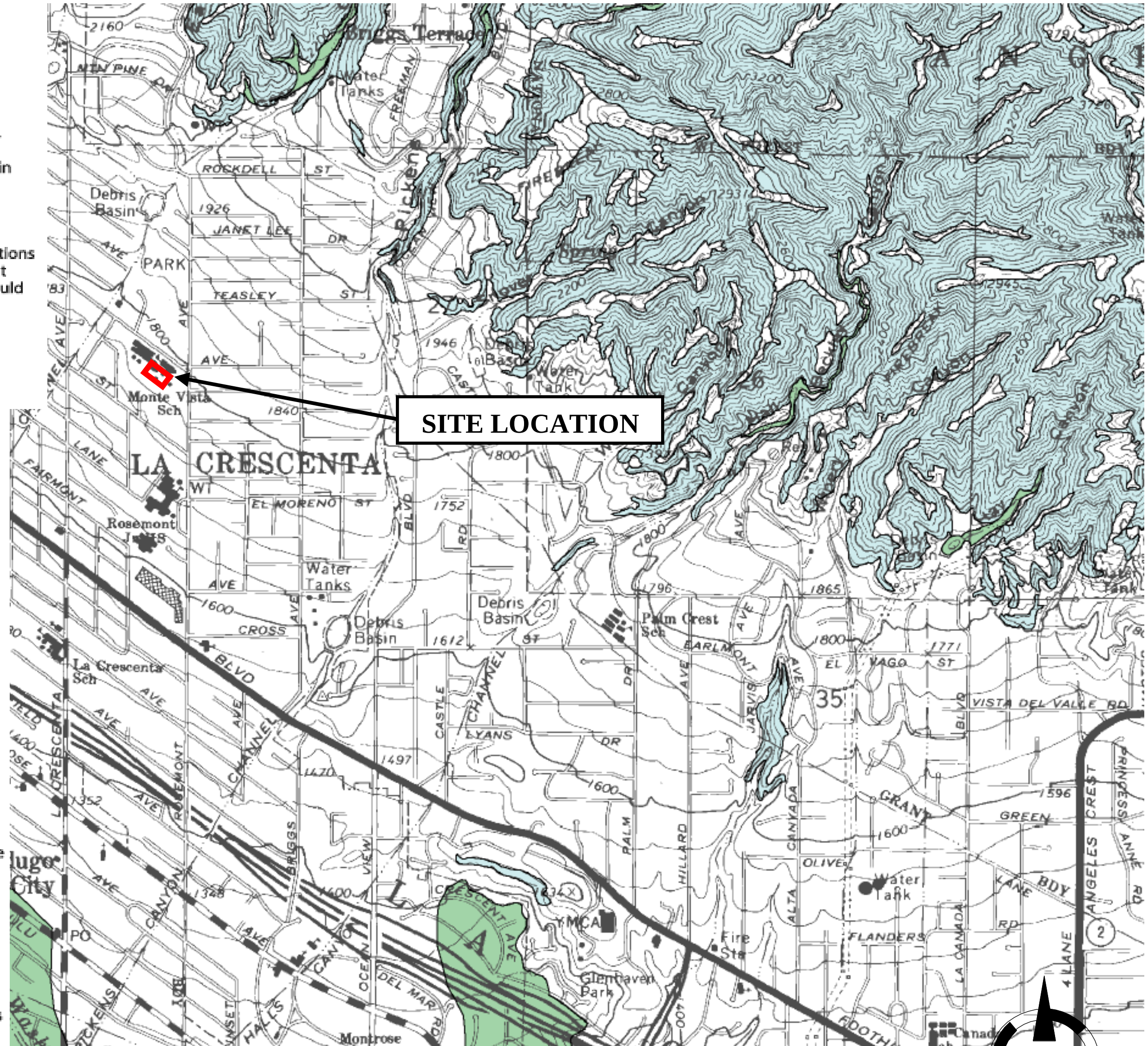
3) This map does not show Alquist-Priolo earthquake fault zones, if any, that may exist in this area. Please refer to the latest official map of earthquake fault zones for disclosures and other actions that are required by the Alquist-Priolo Earthquake Fault Zoning Act. For more information on this subject and an index to available maps, see DMG Special Publication 42.

4) Landslide zones on this map were determined, in part, by adapting methods first developed by the U.S. Geological Survey (USGS). A new generation of landslide hazard maps being prepared by the USGS (Jibson and Harp, in preparation) uses an experimental approach designed to explore new methods to assess earthquake-induced landslide hazards. Although aspects of this new methodology may be incorporated in future seismic hazard zone maps, the experimental USGS maps should not be used as substitutes for these official earthquake-induced landslide zone maps.

5) U.S. Geological Survey base map standards provide that 90 percent of cultural features be located within 40 feet (horizontal accuracy) at the scale of this map. The identification and location of liquefaction and earthquake-induced landslide zones are based on available data. However, the quality of data used is varied. The zone boundaries depicted have been drawn as accurately as possible at this scale.

6) Information on this map is not sufficient to serve as a substitute for the geologic and geotechnical site investigations required under Chapters 7.5 and 7.8 of Division 2 of the Public Resources Code.

7) DISCLAIMER: The State of California and the Department of Conservation make no representations or warranties regarding the accuracy of the data from which these maps were derived. Neither the State nor the Department shall be liable under any circumstances for any direct, indirect, special, incidental or consequential damages with respect to any claim by any user or any third party on account of or arising from the use of this map.



State of California Seismic Hazard Zones, Pasadena Quadrangle Official Map, Released March 25, 1999

Seismic Hazard Map	SCALE: NOT TO SCALE	DATE: 09/2020	SALEM engineering group, inc.
GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION	DRAWN BY: JC	APPROVED BY: CJ	
Proposed Classroom Building **Monte Vista Elementary School – 2620 Orange Avenue** **La Crescenta-Montrose, California**	PROJECT NO. 3-220-0617	FIGURE NO. 5	

Legend

SPECIAL FLOOD HAZARD AREAS		Without Base Flood Elevation (BFE) Zone A, V, A99
		With BFE or Depth Zone AE, AO, AH, VE, AR
		Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile Zone X
		Future Conditions 1% Annual Chance Flood Hazard Zone X
		Area with Reduced Flood Risk due to Levee. See Notes. Zone X
		Area with Flood Risk due to Levee Zone D
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard Zone X
		Effective LOMRs
		Area of Undetermined Flood Hazard Zone D
GENERAL STRUCTURES		Channel, Culvert, or Storm Sewer
		Levee, Dike, or Floodwall
OTHER FEATURES	20.2 / 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
		Coastal Transect
	513	Base Flood Elevation Line (BFE)
		Limit of Study
		Jurisdiction Boundary
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
MAP PANELS		Digital Data Available
		No Digital Data Available
		Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 9/8/2020 at 4:17 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

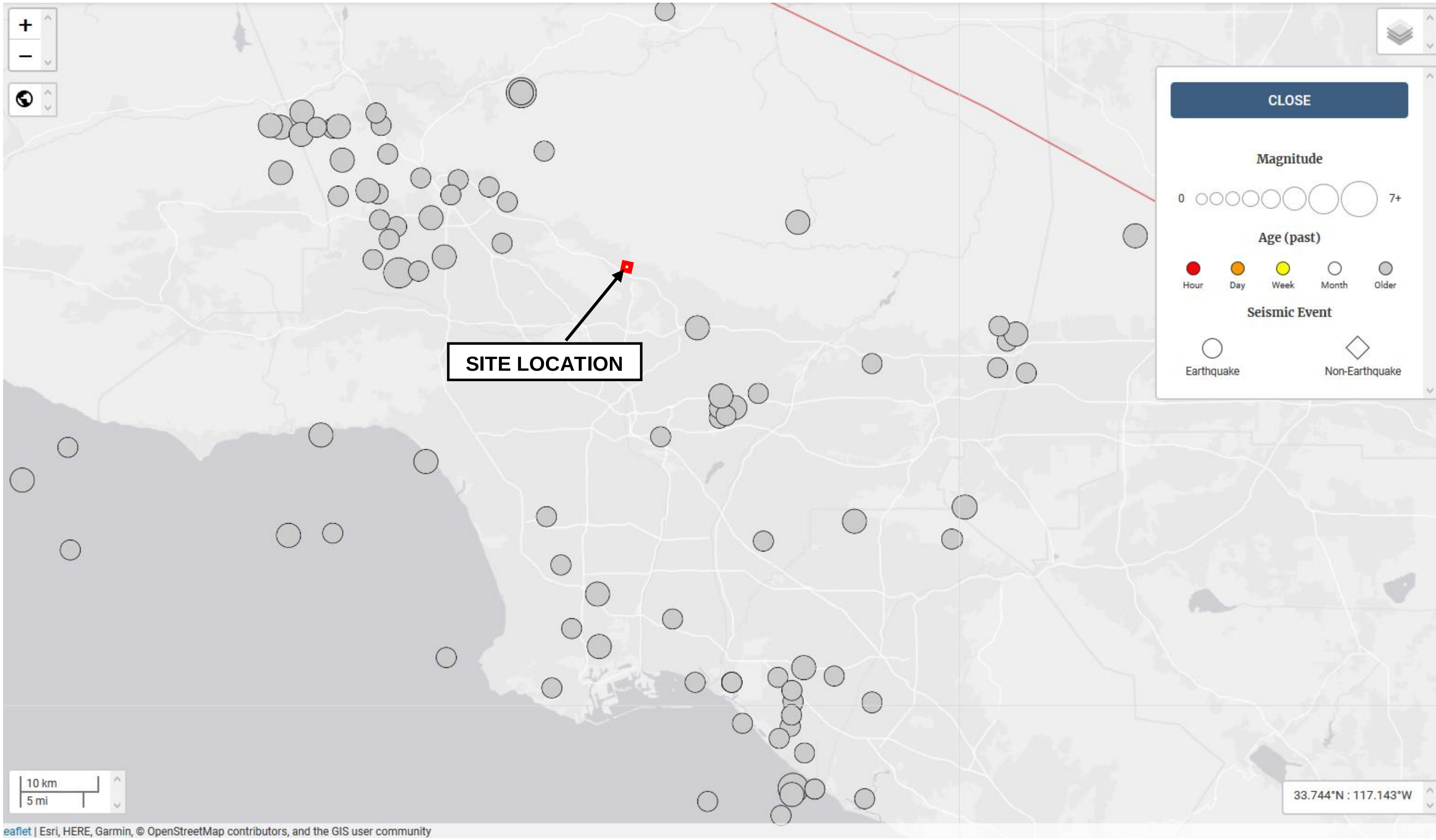
This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Flood Zone Map

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION
Proposed Classroom Building
Monte Vista Elementary School – 2620 Orange Avenue
La Crescenta-Montrose, California

SCALE:	DATE:
AS SHOWN	09/2020
DRAWN BY: JC	APPROVED BY: CJ
PROJECT NO. 3-220-0617	FIGURE NO. 6

SALEM engineering group, inc.



Earthquake Epicenter Map

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC HAZARDS EVALUATION
Proposed Classroom Building
Monte Vista Elementary School – 2620 Orange Avenue
La Crescenta-Montrose, California

SCALE:	DATE:
NOT TO SCALE	09/2020
DRAWN BY: II	APPROVED BY: CJ
PROJECT NO. 3-220-0617	FIGURE NO. 7

SALEM engineering group, inc.

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation was conducted on September 2, 2020 and included a site visit, subsurface exploration, soil sampling. The locations of the exploratory borings are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

In general, our borings were performed using a truck-mounted CME 45C drill rig equipped with 4-inch solid flight augers and 6-inch hollow stem auger. Sampling in the borings was accomplished using a hydraulic 140-pound hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT "N" values; corrections have not been applied. Upon completion the borings were backfilled with drill cuttings.

Subsurface conditions encountered in the exploratory borings were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The logs depict soil and geologic conditions encountered and depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

SALEM engineering group, inc.

Test Boring: B-1 **Page** 1 **Of:** 2

Project Number: 3-220-0617

Date: 08/31/2020

Client: Glendale Unified School District

Project: Proposed Classroom Building - Monte Vista Elementary School

Location: 2620 Orange Avenue, La Crescenta-Montrose, California

Drilled By: SALEM **Logged By:** EGR

Drill Type: CME 45C **Elevation:** 1776'

Auger Type: 6 in. Hollow Stem Auger **Initial Depth to Groundwater:** N/A

Hammer Type: Automatic Trip - 140 lb/30 in **Final Depth to Groundwater:** N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0 / 1776		AC	Asphalt Concrete = 3 in.				
		AB	Aggregate Base = 3 in.				
1775	10/6 6/6 8/6	SM	Silty SAND Medium dense; moist; brown; fine to coarse grain sand; with fine gravel.	14	5.8	-	
	23/6 30/6 33/6	SP-SM	Gravelly SAND with Silt Dense; slighlty moist; fine to coarse grain sand; fine to coarse gravel.	63	2.9	134.6	Cu=53.33 Cc=0.83
5 / 1770							
	17/6 60/2 -	SM	Silty SAND Very dense; slightly moist; fine to coarse grain sand.	60/2"	3.2	125.9	
10 / 1765							
15 / 1760	22/6 29/6 31/6		Grades as above; with fine to coarse gravel.	60	5.1	-	
20 / 1755	20/6 23/6 31/6		Grades as above.	54	7.4	-	
25 / 1750	18/6 31/9 20/6		Grades as above.	51	5.5	-	

Notes:

Figure Number A-1

SALEM
engineering group, inc.

Project Number: 3-220-0617
Date: 08/31/2020
Test Boring: B-1

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
30 / 1745	15/6 11/6 13/6		Grades as above; moist; trace gravel.	24	7.7	-	
35 / 1740	11/6 18/6 20/6		Grades as above.	38	11.7	-	
40 / 1735	8/6 28/6 28/6		Grades as above.	56	7.9	-	
45 / 1730	18/6 26/6 37/6		Grades as above.	63	6.2	-	
			Auger refusal at 47 feet BSG.				
50 / 1725							
55 / 1720							
60 / 1715							

Notes:

Figure Number A-1

SALEM engineering group, inc.

Test Boring: B-2 **Page** 1 **Of:** 1

Project Number: 3-220-0617

Date: 08/31/2020

Client: Glendale Unified School District

Project: Proposed Classroom Building - Monte Vista Elementary School

Location: 2620 Orange Avenue, La Crescenta-Montrose, California

Drilled By: SALEM **Logged By:** EGR

Drill Type: CME 45C **Elevation:** 1763'

Auger Type: 6 in. Hollow Stem Auger **Initial Depth to Groundwater:** N/A

Hammer Type: Automatic Trip - 140 lb/30 in **Final Depth to Groundwater:** N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 3 in.				
		AB	Aggregate Base = 3 in.				
1760	10/6 29/6 36/6	SP-SM	Gravelly SAND with Silt Dense; moist; brown; fine to coarse grain sand; fine to coarse gravel.	65	2.6	127.2	
5	16/6 21/6 60/3	SM	Silty SAND Very dense; slightly moist; fine to coarse grain sand; with gravel.	81/9"	5.9	124.3	
1755			Auger refusal at 7 feet BSG.				
10							
1750							
15							
1745							
20							
1740							
25							
1735							

Notes:

Figure Number A-2

SALEM engineering group, inc.

Test Boring: B-3 **Page** 1 **Of:** 1

Project Number: 3-220-0617

Date: 08/31/2020

Client: Glendale Unified School District

Project: Proposed Classroom Building - Monte Vista Elementary School

Location: 2620 Orange Avenue, La Crescenta-Montrose, California

Drilled By: SALEM **Logged By:** EGR

Drill Type: CME 45C **Elevation:** 1775'

Auger Type: 4 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

Hammer Type: Automatic Trip - 140 lb/30 in **Final Depth to Groundwater:** N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
1775 — 0		AC	Asphalt Concrete = 3 in.				
		AB	Aggregate Base = 3 in.				
	13/6 13/6 16/6	SP-SM	Gravelly SAND with Silt Medium dense; slightly moist; brown; fine to coarse grain sand; fine gravel.	29	2.7	116.6	Cu=85.71 Cc=0.48
1770 — 5	13/6 15/6 16/6		Grades as above.	31	1.9	118.4	
1765 — 10	16/6 21/6 30/6	SM	Silty SAND Very dense; slightly moist; brown; fine to coarse grain sand; with gravel.	51	3.3	-	
1760 — 15	21/6 27/6 30/6		Grades as above.	57	4.6	-	
1755 — 20	20/6 21/6 27/6		Grades as above; dense; moist.	48	10.4	-	
			End of boring at 21.5 feet BSG.				
1750 — 25							

Notes:

Figure Number A-3

SALEM engineering group, inc.

Test Boring: B-4 **Page** 1 **Of:** 1

Project Number: 3-220-0617

Date: 08/31/2020

Client: Glendale Unified School District

Project: Proposed Classroom Building - Monte Vista Elementary School

Location: 2620 Orange Avenue, La Crescenta-Montrose, California

Drilled By: SALEM **Logged By:** EGR

Drill Type: CME 45C **Elevation:** 1761'

Auger Type: 4 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

Hammer Type: Automatic Trip - 140 lb/30 in **Final Depth to Groundwater:** N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 3 in.				
1760		AB	Aggregate Base = 3 in.				
	38/6 60/3 -	SP-SM	Gravelly SAND with Silt Very dense; slightly moist; brown; fine to coarse grain sand; fine to coarse gravel.	60/3"	1.2	125.9	
5			Auger refusal at 4 feet BSG.				
1755							
10							
1750							
15							
1745							
20							
1740							
25							
1735							

Notes:

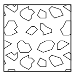
Figure Number A-4

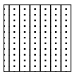
KEY TO SYMBOLS

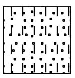
Symbol Description

Strata symbols

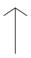
 **Asphaltic Concrete**

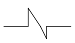
 **Aggregate Base**

 **Silty sand**

 **Poorly graded sand with silt**

Misc. Symbols

 **Drill rejection**

 **Boring continues**

Soil Samplers

 **Standard penetration test**

 **California sampler**

Notes:

Granular Soils
Blows Per Foot (Uncorrected)

	MCS	SPT
Very loose	<5	<4
Loose	5-15	4-10
Medium dense	16-40	11-30
Dense	41-65	31-50
Very dense	>65	>50

Cohesive Soils
Blows Per Foot (Uncorrected)

	MCS	SPT
Very soft	<3	<2
Soft	3-5	2-4
Firm	6-10	5-8
Stiff	11-20	9-15
Very Stiff	21-40	16-30
Hard	>40	>30

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

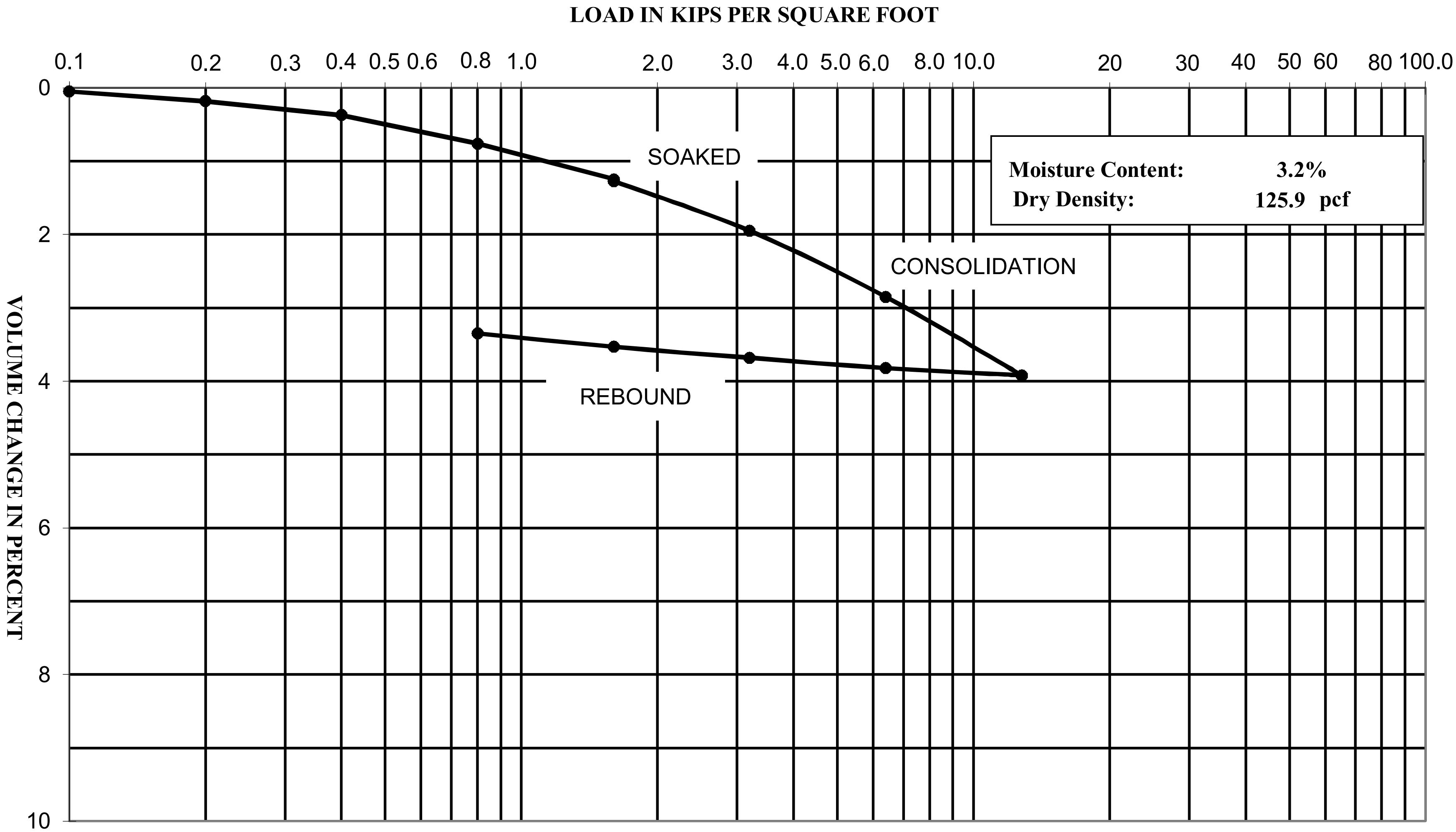
APPENDIX

B

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM), Caltrans, or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, corrosivity, consolidation, shear strength, maximum density and optimum moisture content, and grain size distribution. The results of the laboratory tests are summarized in the following figures.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



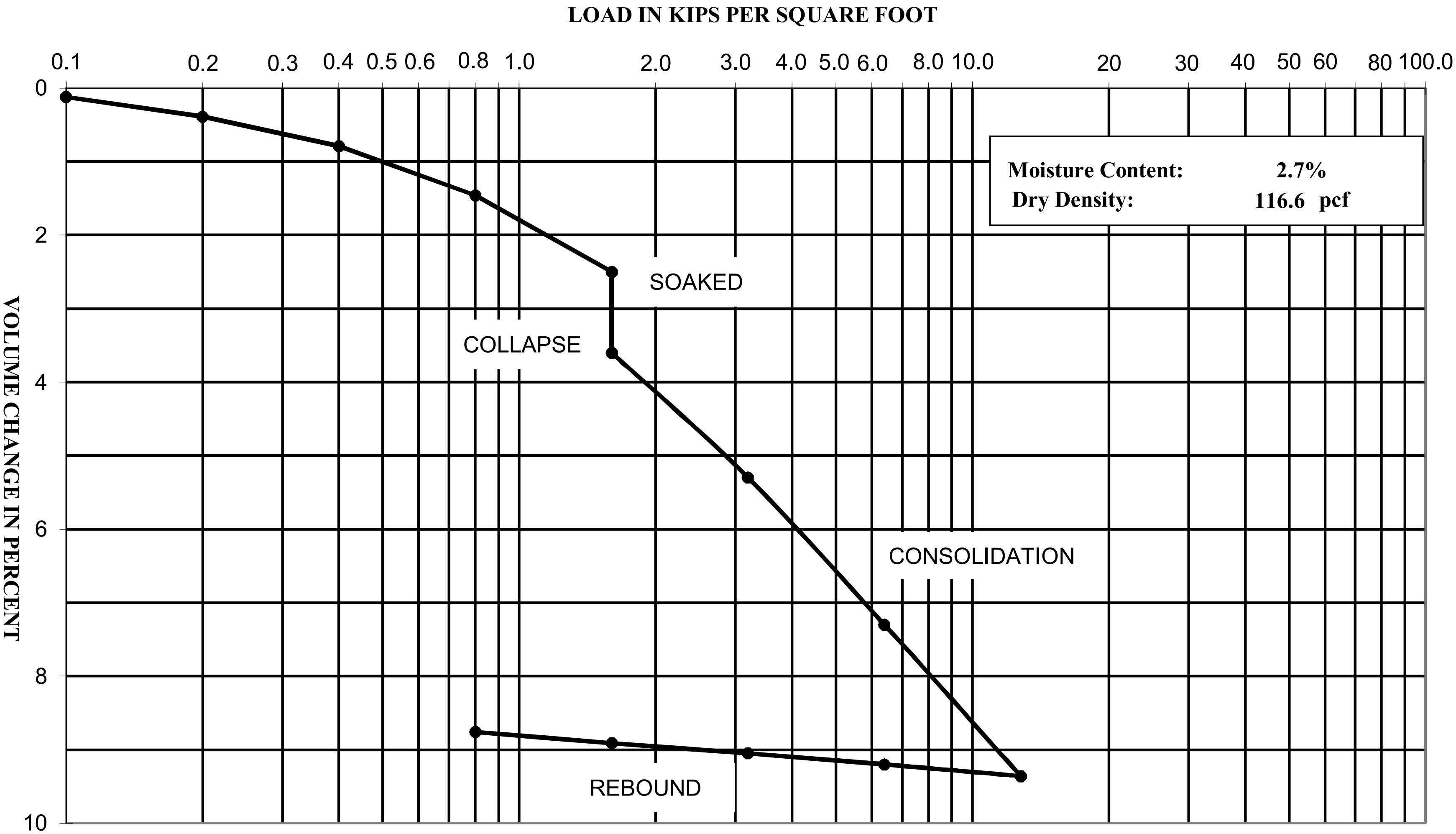
Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

Project Number: 3-220-0617

Boring: B-1 @ 8.5'

SALEM engineering group, inc.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

Project Number: 3-220-0617

Boring: B-3 @ 1.5'

SALEM engineering group, inc.

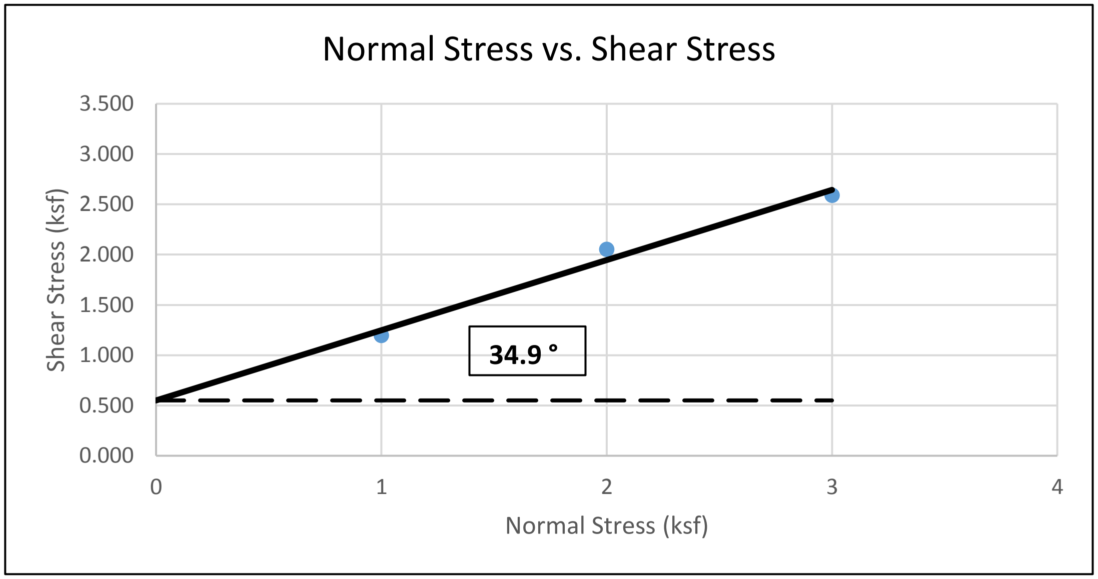
Direct Shear Test (ASTM D3080)

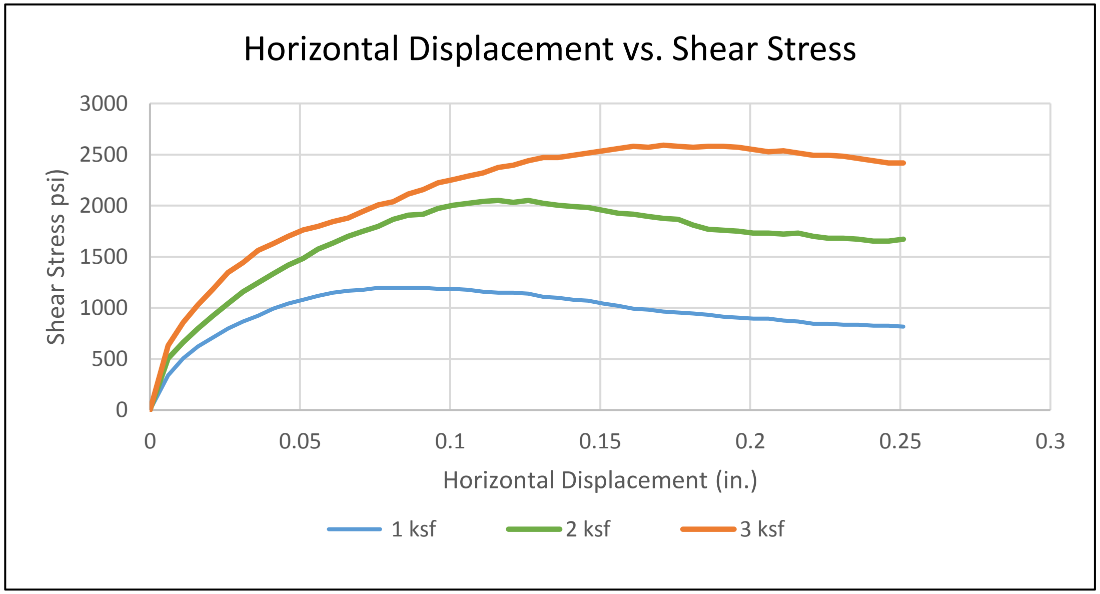
Project Name:	Proposed Classroom Building-MVES - La Crescenta, CA
Project Number:	3-220-0617
Client:	Glendale Unified School District
Sample Location:	B-1 @ 3.5'
Sample Type:	Undisturbed Ring
Soil Classification:	Gravelly SAND w/Silt (SP-SM)
Tested By:	M. Noorzay
Reviewed By:	CJ
Date:	9/10/2020
Equipment Used:	Geomatic Direct Shear Machine

	Sample 1	Sample 2	Sample 3
Normal Stress (ksf)	1.000	2.000	3.000
Shear Rate (in/min)	0.002		
Peak Shear Stress (ksf)	1.196	2.051	2.591
Residual Shear Stress (ksf)	0.000	0.000	0.000

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in.)	1	1	1
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	2.8		
Final Moisture Content (%)	14.5	14.8	15.1
Dry Density (pcf)	121.0	118.6	112.3

Peak Shear Strength Values	
Slope	0.70
Friction Angle	34.9
Cohesion (psf)	550





SALEM
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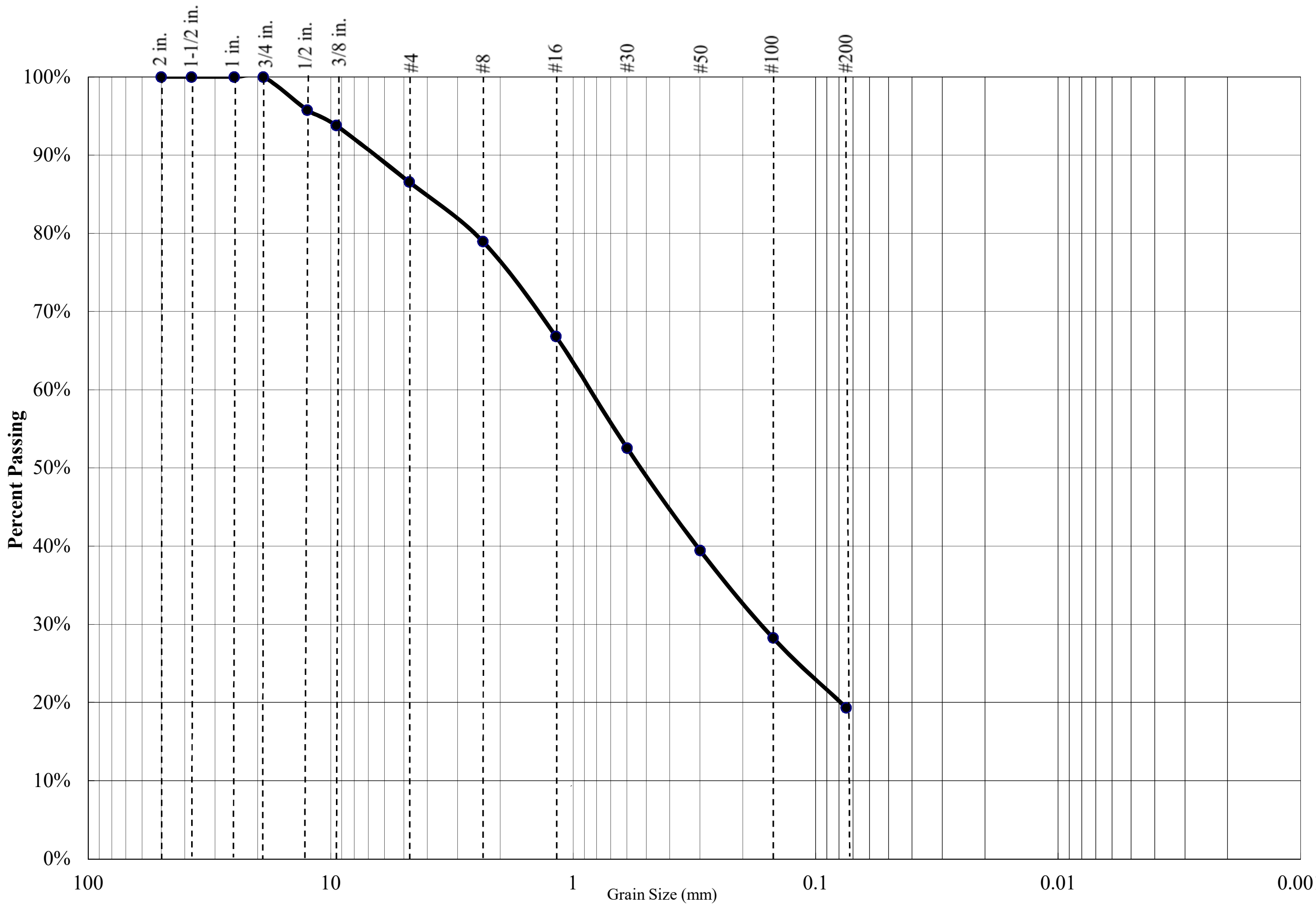
PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
13%	67%	19%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	95.8%
3/8 inch	93.8%
#4	86.6%
#8	79.0%
#16	66.8%
#30	52.6%
#50	39.5%
#100	28.3%
#200	19.3%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=		D_{50}=	
D_{30}=		D_{15}=		D_{10}=	
C_u=	N/A	C_c=	N/A		

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

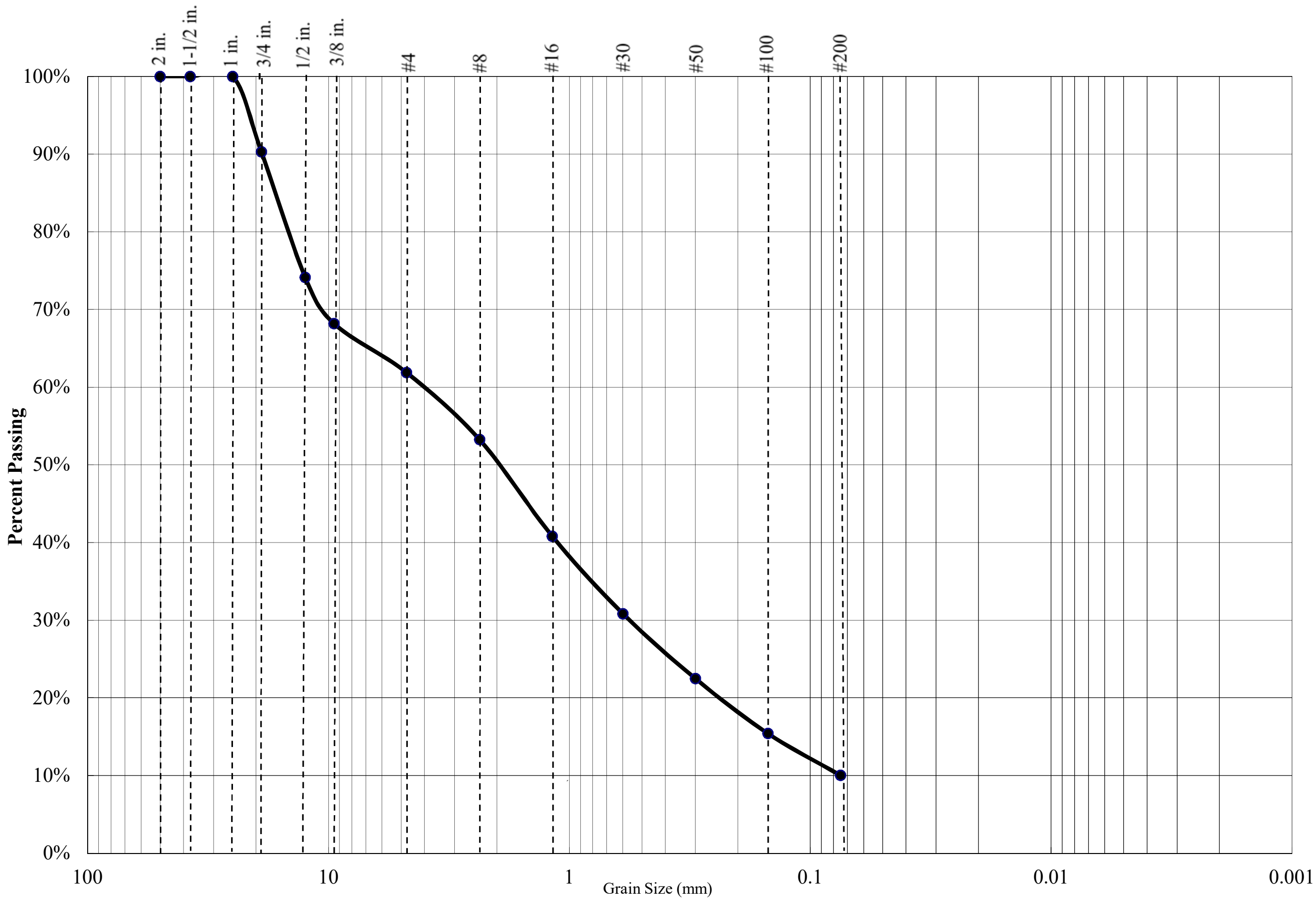
Project Number: 3-220-0617

Boring: B-1 @ 1.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
38%	52%	10%

Sieve Size	Percent Passing
3/4 inch	90.3%
1/2 inch	74.1%
3/8 inch	68.2%
#4	61.9%
#8	53.2%
#16	40.8%
#30	30.8%
#50	22.5%
#100	15.4%
#200	10.0%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D85=		D60=	4.0	D50=	
D30=	0.5	D15=		D10=	0.075
C_u=	53.33	C_c=	0.83		

USCS CLASSIFICATION
Gravelly SAND with Silt (SP-SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

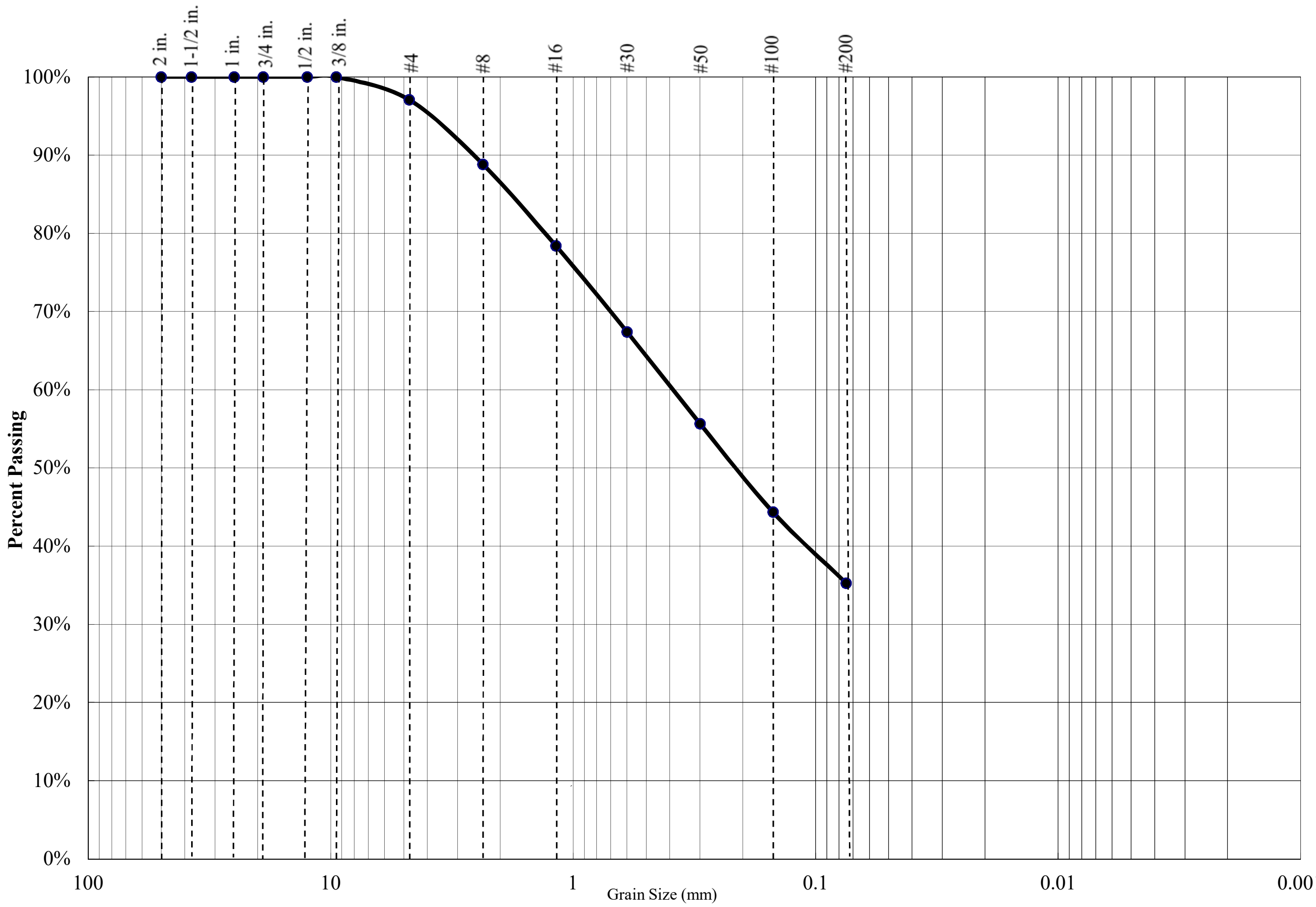
Project Number: 3-220-0617

Boring: B-1 @ 3.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
3%	62%	35%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	97.1%
#8	88.8%
#16	78.4%
#30	67.4%
#50	55.6%
#100	44.4%
#200	35.3%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D85=		D60=		D50=	
D30=		D15=		D10=	
C_u=	N/A	C_c=	N/A		

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

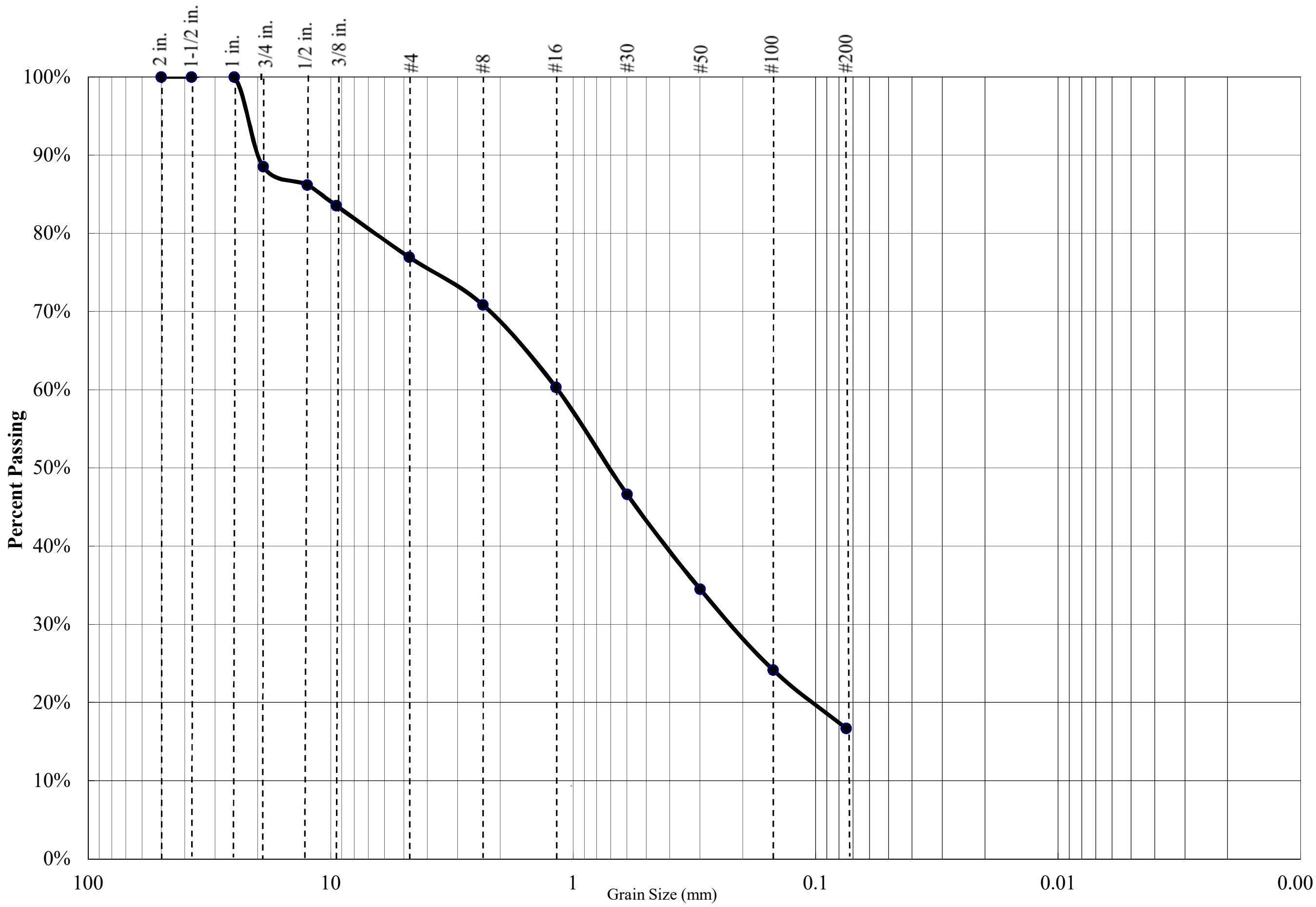
Project Number: 3-220-0617

Boring: B-1 @ 8.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
23%	60%	17%

Sieve Size	Percent Passing
3/4 inch	88.5%
1/2 inch	86.2%
3/8 inch	83.5%
#4	77.0%
#8	70.9%
#16	60.3%
#30	46.6%
#50	34.5%
#100	24.2%
#200	16.7%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=		D_{50}=	
D_{30}=		D_{15}=		D_{10}=	
C_u=	N/A	C_c=	N/A		

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

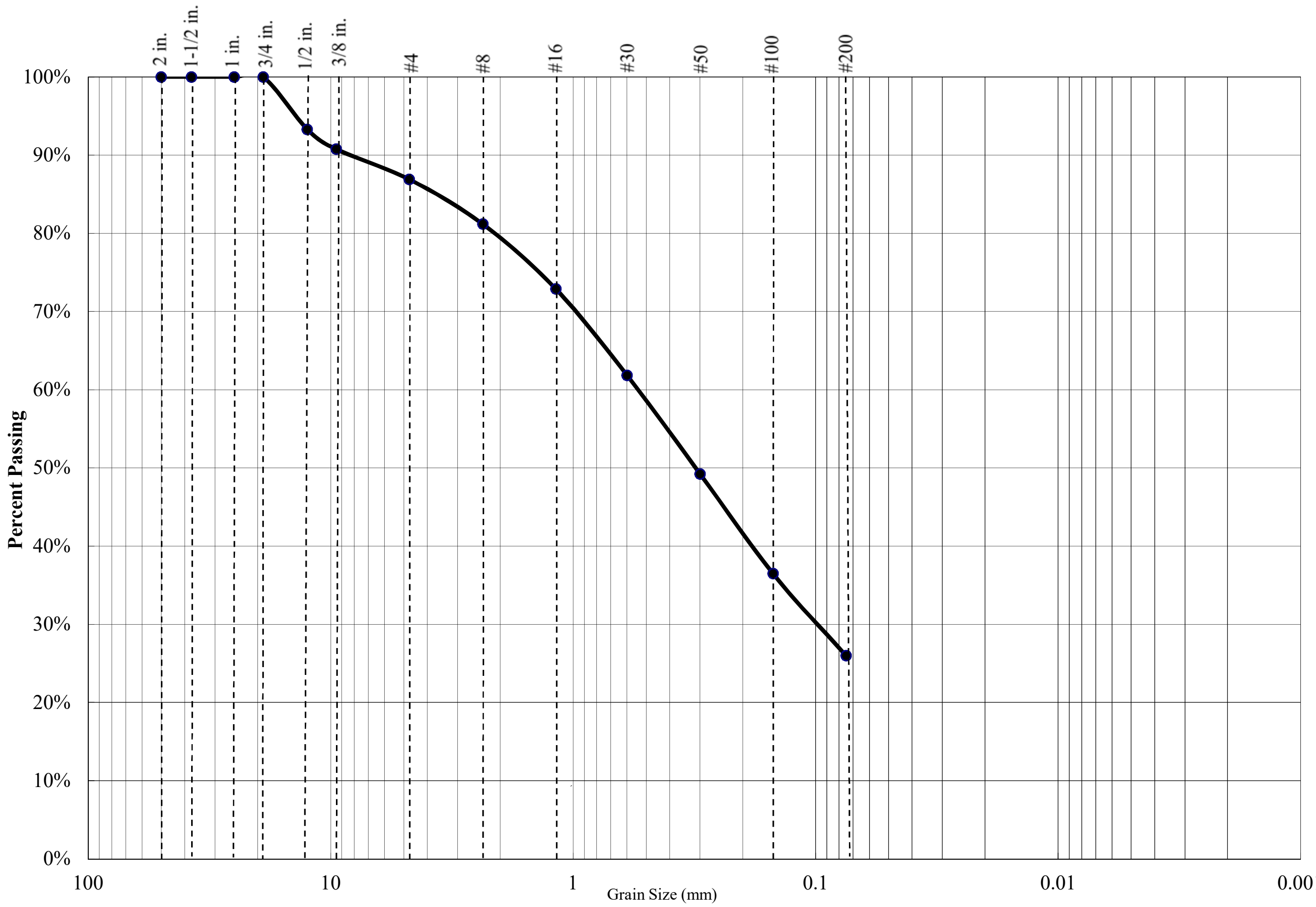
Project Number: 3-220-0617

Boring: B-1 @ 15'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
13%	61%	26%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	93.3%
3/8 inch	90.8%
#4	86.9%
#8	81.2%
#16	72.9%
#30	61.8%
#50	49.2%
#100	36.5%
#200	26.0%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=		D_{50}=	
D_{30}=		D_{15}=		D_{10}=	
C_u=	N/A	C_c=	N/A		

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

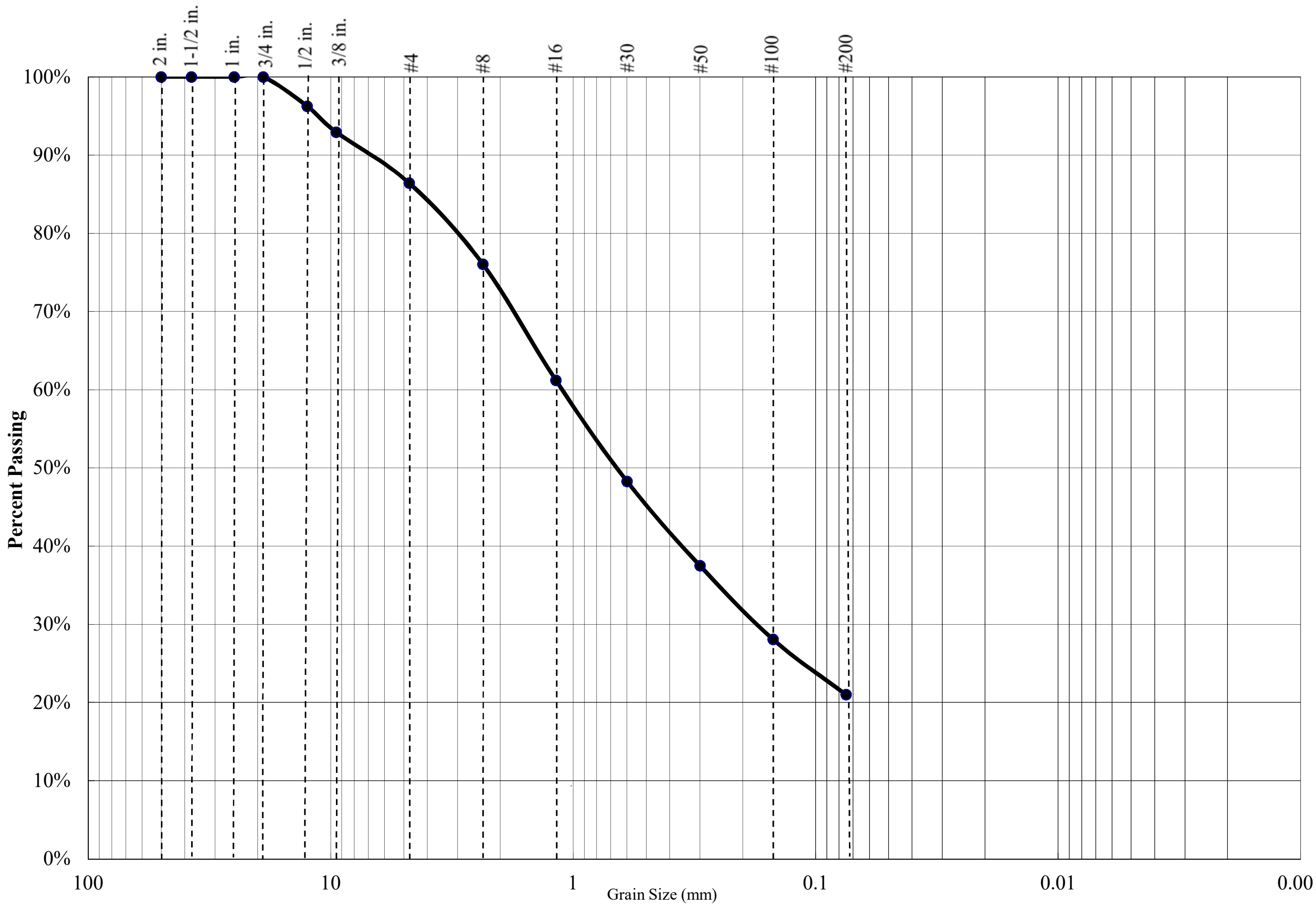
Project Number: 3-220-0617

Boring: B-1 @ 30'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
14%	66%	21%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	96.3%
3/8 inch	92.9%
#4	86.4%
#8	76.1%
#16	61.2%
#30	48.3%
#50	37.5%
#100	28.1%
#200	21.0%

Atterberg Limits		
PL=	LL=	PI=

Coefficients				
D_{85}=		D_{60}=		D_{50}=
D_{30}=		D_{15}=		D_{10}=
C_u=	N/A	C_c=	N/A	

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

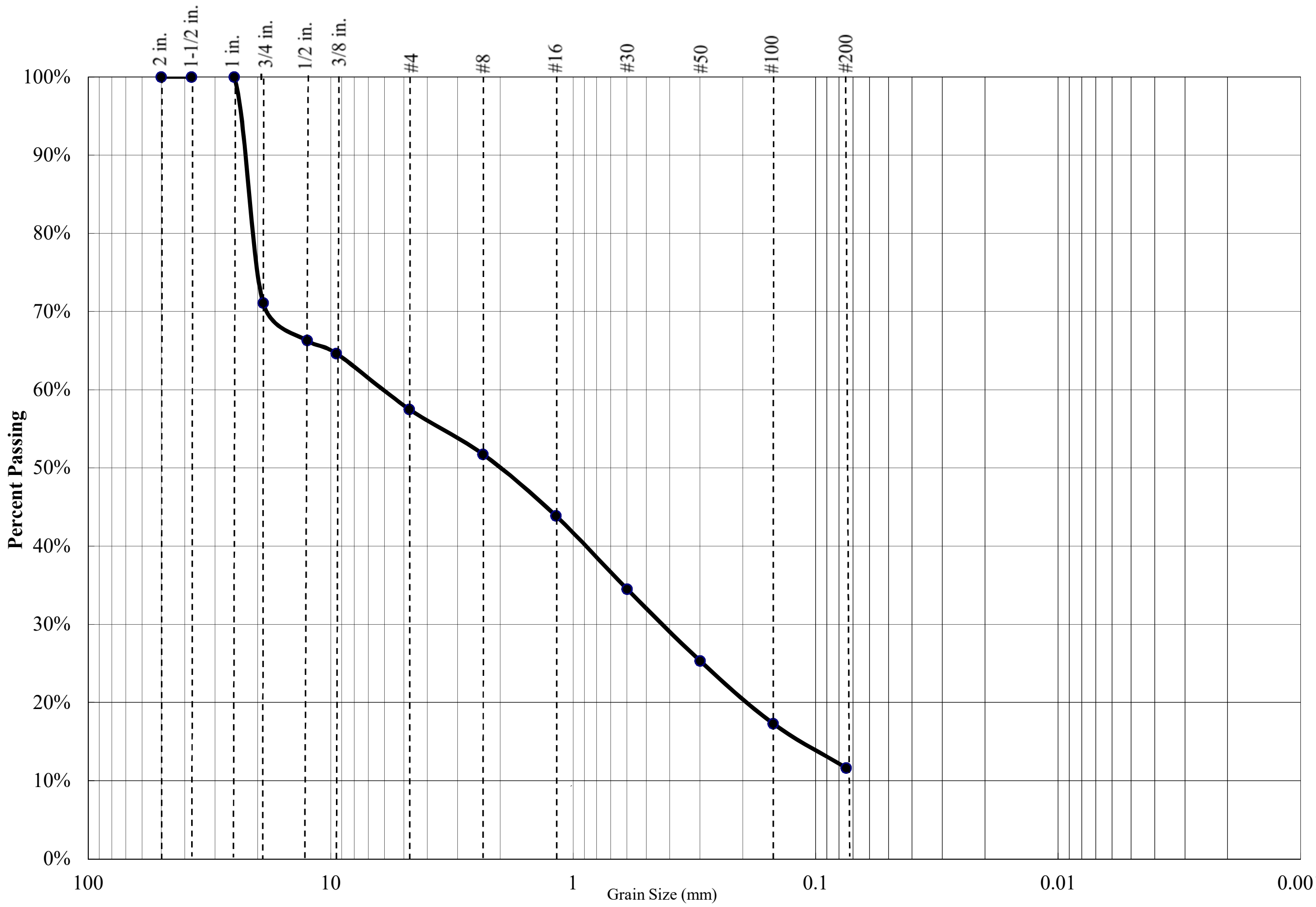
Project Number: 3-220-0617

Boring: B-1 @ 40'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
42%	46%	12%

Sieve Size	Percent Passing
3/4 inch	71.1%
1/2 inch	66.3%
3/8 inch	64.7%
#4	57.5%
#8	51.8%
#16	43.9%
#30	34.5%
#50	25.3%
#100	17.3%
#200	11.7%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	6.0	D_{50}=	
D_{30}=	0.45	D_{15}=		D_{10}=	0.07
C_u=	85.71	C_c=	0.48		

USCS CLASSIFICATION
Gravelly SAND with Silt (SP-SM)

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA

Project Number: 3-220-0617

Boring: B-3 @ 1.5'

SALEM engineering group, inc.

CHEMICAL ANALYSIS

SO_4 - Modified CTM 417 & Cl - Modified CTM 417/422

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA
Project Number: 3-220-0617
Date Sampled: 9/2/2020 Date Tested: 9/9/2020
Sampled By: EGR Tested By: M. Noorzay
Soil Description: Brown Silty SAND (SM) w/Fine to Coarse Gravel

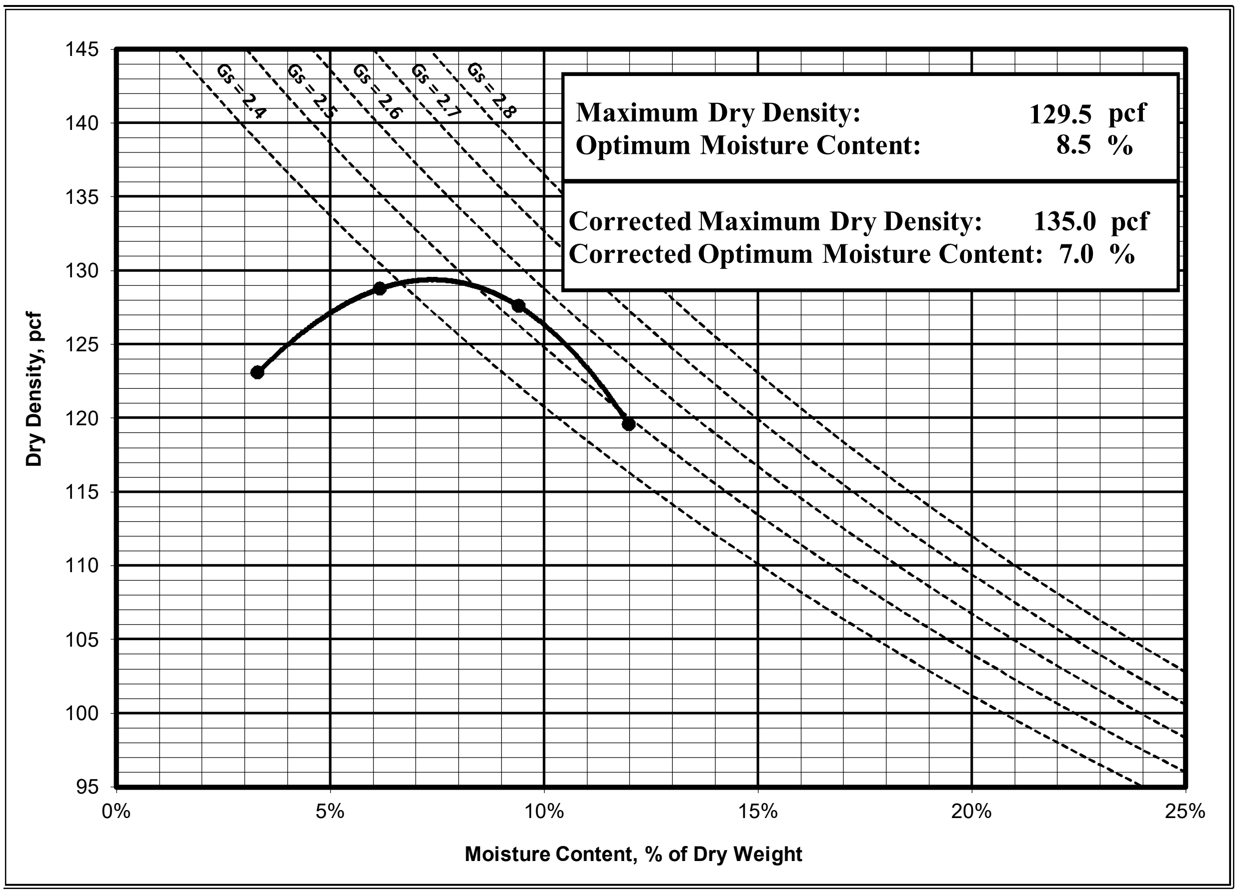
Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 1'-4'	100 mg/kg	20 mg/kg	7.0
1b.	B-1 @ 1'-4'	100 mg/kg	20 mg/kg	7.0
1c.	B-1 @ 1'-4'	100 mg/kg	20 mg/kg	7.0
Average:		**100 mg/kg**	**20 mg/kg**	**7.0**

SALEM engineering group, inc.

Laboratory Compaction Curve
ASTM D1557

Project Name: Proposed Classroom Building - MVES, La Crescenta-Montrose, CA
Project Number: 3-220-0617
Date Sampled: 9/2/2020 Date Tested: 9/10/2020
Sampled By: EGR Tested By: M. Noorzay
Sample Location: B-1 @ 1'-4'
Soil Description: Brown Silty SAND (SM) w/Fine to Coarse Gravel
Test Method: Method A

	1	2	3	4
Weight of Moist Specimen & Mold, (g)	3936.4	4080.3	4124.0	4037.8
Weight of Compaction Mold, (g)	2013.7	2013.7	2013.7	2013.7
Weight of Moist Specimen, (g)	1922.7	2066.6	2110.3	2024.1
Volume of Mold, (ft^3)	0.0333	0.0333	0.0333	0.0333
Wet Density, (pcf)	127.2	136.7	139.6	133.9
Weight of Wet (Moisture) Sample, (g)	100.0	100.0	100.0	100.0
Weight of Dry (Moisture) Sample, (g)	96.8	94.2	91.4	89.3
Moisture Content, (%)	3.3%	6.2%	9.4%	12.0%
Dry Density, (pcf)	123.1	128.8	127.6	119.5



SALEM engineering group, inc.

APPENDIX

C

APPENDIX D
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

When the text of the report conflicts with the general specifications in this appendix, the recommendations in the report have precedence.

1.0 SCOPE OF WORK: These specifications and applicable plans pertain to and include all earthwork associated with the site rough grading, including, but not limited to, the furnishing of all labor, tools and equipment necessary for site clearing and grubbing, stripping, preparation of foundation materials for receiving fill, excavation, processing, placement and compaction of fill and backfill materials to the lines and grades shown on the project grading plans and disposal of excess materials.

2.0 PERFORMANCE: The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of SALEM Engineering Group, Incorporated, hereinafter referred to as the Soils Engineer and/or Testing Agency. Attainment of design grades, when achieved, shall be certified by the project Civil Engineer. Both the Soils Engineer and the Civil Engineer are the Owner's representatives. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary adjustments until all work is deemed satisfactory as determined by both the Soils Engineer and the Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Soils Engineer, Civil Engineer, or project Architect. No earthwork shall be performed without the physical presence or approval of the Soils Engineer. The Contractor shall notify the Soils Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork.

The Contractor shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the Contractor shall defend, indemnify and hold the Owner and the Engineers harmless from any and all liability, real or alleged, in connection with the performance of work on this project, except for liability arising from the sole negligence of the Owner or the Engineers.

3.0 TECHNICAL REQUIREMENTS: All compacted materials shall be densified to no less than 95% (90% for fine grained cohesive soils) of relative compaction based on ASTM D1557 Test Method (latest edition), UBC or CAL-216, or as specified in the technical portion of the Soil Engineer's report. The location and frequency of field density tests shall be determined by the Soils Engineer. The results of these tests and compliance with these specifications shall be the basis upon which satisfactory completion of work will be judged by the Soils Engineer.

4.0 SOILS AND FOUNDATION CONDITIONS: The Contractor is presumed to have visited the site and to have familiarized himself with existing site conditions and the contents of the data presented in the Geotechnical Engineering Report. The Contractor shall make his own interpretation of the data contained in the Geotechnical Engineering Report and the Contractor shall not be relieved of liability for any loss sustained as a result of any variance between conditions indicated by or deduced from said report and the actual conditions encountered during the progress of the work.

5.0 DUST CONTROL: The work includes dust control as required for the alleviation or prevention of any dust nuisance on or about the site or the borrow area, or off-site if caused by the Contractor's operation either during the performance of the earthwork or resulting from the conditions in which the Contractor leaves the site. The Contractor shall assume all liability, including court costs of codefendants, for all claims related to dust or wind-blown materials attributable to his work. Site preparation shall consist of site clearing and grubbing and preparation of foundation materials for receiving fill.

6.0 CLEARING AND GRUBBING: The Contractor shall accept the site in this present condition and shall demolish and/or remove from the area of designated project earthwork all structures, both surface and subsurface, trees, brush, roots, debris, organic matter and all other matter determined by the Soils Engineer to be deleterious. Such materials shall become the property of the Contractor and shall be removed from the site.

Tree root systems in proposed improvement areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots greater than 1 inch in diameter. Tree roots removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill of tree root excavations is not permitted until all exposed surfaces have been inspected and the Soils Engineer is present for the proper control of backfill placement and compaction. Burning in areas which are to receive fill materials shall not be permitted.

7.0 SUBGRADE PREPARATION: Surfaces to receive Engineered Fill and/or building or slab loads shall be prepared as outlined above, scarified to a minimum of 12 inches, moisture-conditioned as necessary, and recompacted to 95% (90% for fine grained cohesive soils) relative compaction.

Loose soil areas and/or areas of disturbed soil shall be moisture-conditioned as necessary and recompacted to 95% (90% for fine grained cohesive soils) relative compaction. All ruts, hummocks, or other uneven surface features shall be removed by surface grading prior to placement of any fill materials. All areas which are to receive fill materials shall be approved by the Soils Engineer prior to the placement of any fill material.

8.0 EXCAVATION: All excavation shall be accomplished to the tolerance normally defined by the Civil Engineer as shown on the project grading plans. All over-excavation below the grades specified shall be backfilled at the Contractor's expense and shall be compacted in accordance with the applicable technical requirements.

9.0 FILL AND BACKFILL MATERIAL: No material shall be moved or compacted without the presence or approval of the Soils Engineer. Material from the required site excavation may be utilized for construction site fills, provided prior approval is given by the Soils Engineer. All materials utilized for constructing site fills shall be free from vegetation or other deleterious matter as determined by the Soils Engineer.

10.0 PLACEMENT, SPREADING AND COMPACTION: The placement and spreading of approved fill materials and the processing and compaction of approved fill and native materials shall be the responsibility of the Contractor. Compaction of fill materials by flooding, ponding, or jetting shall not be permitted unless specifically approved by local code, as well as the Soils Engineer. Both cut and fill shall be surface-compacted to the satisfaction of the Soils Engineer prior to final acceptance.

11.0 SEASONAL LIMITS: No fill material shall be placed, spread, or rolled while it is frozen or thawing, or during unfavorable wet weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until the Soils Engineer indicates that the moisture content and density of previously placed fill is as specified.

12.0 DEFINITIONS - The term "pavement" shall include asphaltic concrete surfacing, untreated aggregate base, and aggregate subbase. The term "subgrade" is that portion of the area on which surfacing, base, or subbase is to be placed. The term “Standard Specifications”: hereinafter referred to, is the most recent edition of the Standard Specifications of the State of California, Department of Transportation. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as determined by ASTM D1557 Test Method (latest edition) or California Test Method 216 (CAL-216), as applicable.

13.0 PREPARATION OF THE SUBGRADE - The Contractor shall prepare the surface of the various subgrades receiving subsequent pavement courses to the lines, grades, and dimensions given on the plans. The upper 12 inches of the soil subgrade beneath the pavement section shall be compacted to a minimum relative compaction of 95% (90% for fine grained cohesive soils) based upon ASTM D1557. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

14.0 AGGREGATE BASE - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class II material, ¾-inch or 1½-inches maximum size. The aggregate base material shall be compacted to a minimum relative compaction of 95 percent based upon CAL-216. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

15.0 AGGREGATE SUBBASE - The aggregate subbase shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate subbase material shall conform to the requirements of Section 25 of the Standard Specifications for Class II Subbase material. The aggregate subbase material shall be compacted to a minimum relative compaction of 95 percent based upon CAL-216, and it shall be spread and compacted in accordance with the Standard Specifications. Each layer of aggregate subbase shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

16.0 ASPHALTIC CONCRETE SURFACING - Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at a central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades, and dimensions shown on the plans. The viscosity grade of the asphalt shall be PG 64-10, unless otherwise stipulated or local conditions warrant more stringent grade. The mineral aggregate shall be Type A or B, ½ inch maximum size, medium grading, and shall conform to the requirements set forth in Section 39 of the Standard Specifications. The drying, proportioning, and mixing of the materials shall conform to Section 39. The prime coat, spreading and compacting equipment, and spreading and compacting the mixture shall conform to the applicable chapters of Section 39, with the exception that no surface course shall be placed when the atmospheric temperature is below 50 degrees F. The surfacing shall be rolled with a combination steel-wheel and pneumatic rollers, as described in the Standard Specifications. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.