GEOTECHNICAL • ENVIRONMENTAL • GEOLOGY • MATERIALS TESTING & INSPECTION • FORENSIC • LABORATORY

SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION WITH GEOLOGIC – SEISMIC HAZARDS EVALUATION

**GEOTECHNICAL ENGINEERING INVESTIGATION
WITH GEOLOGIC-SEISMIC HAZARDS EVALUATION
MODERNIZATION OF ATHLETIC TRACK AND LIGHTING,
FUTURE BUILDING AND BLEACHERS,
AND UNDERGROUND STORMWATER DETENTION SYSTEM
DESMOND MIDDLE SCHOOL
26490 MARTIN STREET
MADERA, CALIFORNIA**

**SALEM PROJECT NO. 1-224-1068A
FEBRUARY 12, 2025**

PREPARED FOR:

**MR. GEORGE CUMMINGS
MADERA UNIFIED SCHOOL DISTRICT
1902 HOWARD ROAD
MADERA, CALIFORNIA 93637**

PREPARED BY:

**SALEM ENGINEERING GROUP, INC.
4729 W. JACQUELYN AVENUE
FRESNO, CA 93722
P: (559) 271-9700
F: (559) 275-0827**

4729 W. Jacquelyn Avenue
Fresno, CA 93722
Phone (559) 271-9700
Fax (559) 275-0827

February 12, 2025 Project No. 1-224-1068A

Mr. George Cummings
Madera Unified School District
1902 Howard Road
Madera, California 93637

**Subject: GEOTECHNICAL ENGINEERING INVESTIGATION
WITH GEOLOGIC-SEISMIC HAZARDS EVALUATION
MODERNIZATION OF ATHLETIC TRACK AND LIGHTING,
FUTURE BUILDINGS AND BLEACHERS,
AND UNDERGROUND STORM WATER DETENTION SYSTEM
DESMOND MIDDLE SCHOOL
26490 MARTIN STREET
MADERA, CALIFORNIA 93638**

Dear Mr. Cummings:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation with Geologic-Seismic Hazards Evaluation for the Modernization of the Athletic Track and Lighting, Future Buildings and Bleachers, and Underground Storm Water Detention System at Desmond Middle School, 26490 Martin Street, Madera, California.

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 (559) 271-9700.

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Ken Clark
Senior Engineering Geologist
CEG 1864

Dean B. Ledgerwood II, PE, PG, CEG
Geotechnical Manager
PE 94395 / PG 8725 / CEG 2613

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4729 W. Jacquelyn Avenue
Fresno, CA 93722
Phone (559) 271-9700
Fax (559) 275-0827

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1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation and Geologic-Seismic Hazards Evaluation for the Modernization of the Athletic Track and Lighting, Future Buildings and Bleachers, and Underground Storm Water Detention System at Desmond Middle School, 26490 Martin Street, Madera, California. The school site is located in the north portion of the City of Madera (see Figure No. 1, Vicinity Map).

The purpose of our geotechnical engineering investigation was to conduct site observations, observe and sample the subsurface conditions encountered at the project site, and to provide conclusions and recommendations relative to the geotechnical aspects of designing and constructing the project as presently proposed. Additionally, our scope included preparation of a Geologic Seismic Hazard Evaluation in accordance with California Geological Survey (CGS) Note 48. The recommendations presented herein are based on analysis of the data obtained and reviewed 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 C. If the text of this report conflicts with the specifications in Appendix C, the recommendations in the text of this report have precedence.

2. PROJECT AND SITE DESCRIPTION

Our understanding of the project is based on your request for proposal (email dated October 14, 2024), including site plans prepared by Darden Architects. The project will include construction of athletic field lights, new track surface with asphaltic concrete underlay, Contech type underground drainage system, 10 row (750 seat) stadium bleachers, 5 row (250 seat) stadium bleachers, snack bar/toilet building (less than 5,000 square feet), and partial infill of an existing basin to extend the track. Maximum building wall load and column loads are expected to be on the order of 1 to 3 kips per linear foot and about 30 kips, respectively. Maximum allowable total and differential settlement is expected to be 1 inch and ½ inch, respectively. Appurtenant construction is expected to include new utilities, flatwork, chain link fencing/gates, and asphaltic concrete and Portland cement concrete pavements. Basements are not anticipated.

The proposed improvements are to be located in the south portion of the school campus, near the existing athletic track, storm water basin, and baseball diamond. At the time of our field investigation, the existing athletic facilities includes a dirt track with grass surfaced soccer/football field. The areas proposed for the building and bleachers were grass surfaced/playfield or existing bleacher areas. The unlined drainage basin was located about 15 feet east of the east end of the existing track. The majority of the basin sides and bottom were covered with grasses/weeds and brush, with some scattered small trees. The locations of the proposed improvement are shown on the Site Plan, Figure No. 2, attached to this report.

The general area of the existing athletic facilities is relatively flat, with an approximate elevation of 280 feet above mean sea level (AMSL). We anticipate that cuts and fills during earthwork will be on the order of 1 to 2 feet to provide level building/bleacher pads and positive site drainage.

3. SITE HISTORY AND PREVIOUS REPORTS

Our review of several on-line historic satellite images, dated September 1998 to October 2023, indicates that the areas proposed for the athletic track, lighting, future buildings and bleachers, and underground storm water detention system were a vacant, undeveloped field in September 1998. Grading for the school site was underway in July 2004, with some campus buildings and some grading indicated in December 2005. The project site appears fully constructed and in March 2006, and has remained relatively unchanged to the time of our field investigation.

No documents pertaining to previous geologic or geotechnical studies were provided to SALEM for review at the time of this investigation. If previous geologic or geotechnical studies reports become available, SALEM should be provided these documents for review.

4. FIELD EXPLORATION

4.1. Site Surface Reconnaissance and Subsurface Exploration

Our field exploration consisted of site surface reconnaissance and subsurface exploration. Test borings B-1 through B-11, HA-1, P-1, and P-2 were drilled on November 14th, 19th and December 23, 2024, to depths ranging from 5 to 50 feet below site grade (BSG). Thirteen (13) of the borings were drilled to depths of 5 to 51½ feet BSG using 6-5/8 inch diameter hollow-stem auger rotated by truck-mounted CME-55 and CME-75 drill rigs. The remaining boring (HA-1) was drilled to a depth of about 3½ feet BSG using hand auger equipment. The approximate locations of the exploratory borings are shown on the Site Plan, Figure No. 2. A detailed discussion of our field investigation and exploratory boring logs are presented in Appendix A.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2487). The test boring logs are presented in Appendix A of this report and include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbols. The locations of the test borings were determined by measuring from the existing site features shown on the Site Plan. Hence, accuracy can be implied only to the degree that this method warrants.

Penetration resistance blow counts were obtained by dropping a 140-pound automated trip hammer through a 30-inch free fall to drive the sampler to a maximum penetration of 18 inches. The number of blows required to drive the last 12 inches, or less if very dense or hard, was recorded as Penetration Resistance (blows/foot) on the logs of borings.

Soil samples were obtained from the test borings at the depths shown on the logs of borings. The Modified California Sampler (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. At the completion of drilling and sampling, the test borings were backfilled with drill cuttings.

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.

4.2. Percolation Testing and Results

Percolation testing was conducted in the five (5) boring holes (percolation test holes) listed in the table below. The percolation test holes were drilled in the area of the proposed underground storm water detention system, at the approximate locations shown the attached Site Plan, Figure No. 2.

The borings, designated B-7 through B-11, were drilled to depths ranging from 10 to 15 feet. Soil samples were collected and selected samples were subjected to sieve analyses and Atterberg Limits testing.

After drilling the boring, a perforated PVC pipe was installed in each test hole and pea gravel was placed in the annulus to prevent caving of the holes. The dimensions of the test holes are provided on the percolation test logs included in Appendix A of this report, after the test boring logs.

Field percolation tests were conducted in the five (5) test holes on December 26, 2024. The percolation test holes were pre-saturated before percolation testing commenced. Percolation rates were measured by filling the test holes with clean water and measuring the water drops at certain time intervals. The percolation test data are presented on the percolation test logs. The difference in the percolation rates are reflected by the varied types of soil materials encountered at the bottom portions of the test holes. The table below provides the soil type at the bottom and wetted sidewall portions of the test hole, as well as an estimate of the vertical infiltration rate for consideration in storm water infiltration design.

TABLE 4.2
ESTIMATED UNFACTORED INFILTRATION RATES
BASED ON PERCOLATION TESTING RESULTS

Test No./ Boring No.	Test Hole Depth (feet below ground surface)	Unfactored Infiltration Rate (inch/hour)**	Soil Type
P-3/B-7	8.7	0.09	Silty Sand (fine grained)
P-4/B-8	13.7	0.19	Silty Sand (fine to coarse grained, with trace clay)
P-5/B-9	11.5	0.05	Poorly Graded Sand with Silt
P-6/B-10	10.2	0.02	Sandy Lean Clay (weakly cemented and interbedded with fine silty sand)
P-7/B-11	15.0	Negligible	Lean Clay with Sand (very weakly cemented)

**** Unfactored infiltration rate calculated as inches of water entering the soil exposed in the sidewalls and bottom of test hole. Appropriate factors of safety should be applied in design.**

Based on our review of the boring logs, it is our opinion that neither the sandy nor the clayey soils are laterally continuous. Lateral migration of subflow is expected in the sandy soils and the “negligible” test result P7/B-11) is not anticipated to prevent the downward migration of the water through the soil column below the proposed detention system.

It should be noted that the field percolation tests do not take into account the long term effects of silt accumulation, sediment, suspended soils, etc. in the discharge water that can result in clogging of the pore spaces in the soil, thus reducing the soil infiltration rate over time (appropriate pre-treatment of water is recommended). Percolation testing is a relatively small scale test. Variations in soil type and soil density/cementation across the infiltration area of the system can influence the infiltration rate. A minimum factor of safety equal to or greater than 1.5 should be used for design, and the sidewall area of the underground detention system should not be considered as infiltration area for the design. The system design engineer should determine whether a higher factor of safety is appropriate for incorporation into the storm water infiltration system design, considering the information in this report and the severity of ramifications resulting from overflow of the system.

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, expansion index, Atterberg limits, gradation, and R-value of the materials encountered. The results of laboratory tesing are included on the boring logs attached to the end of this report.

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, were used to prepare the final boring logs in Appendix A.

6. SOIL AND GROUNDWATER CONDITIONS

6.1 Subsurface Conditions

The subsurface soil conditions encountered generally appear typical of those found in the geologic region of the site. The near surface soil conditions encountered in the area of the track improvements, within the upper 5 feet BSG, were predominantly silty sand, clayey sand, sandy silt, with subordinate low plastic lean clay. Much of these soils were noted to be very dense or hard, with weak to moderate cementation (hardpad). Below these upper soils, the soils predominantly comprised very stiff to hard silt and lean clay layers, and medium dense to very dense silty sand and clayey sand layers extending to the maximum depth explored of 51½ feet BSG. Also, medium dense and dense poorly graded sands with silt were encountered in two (2) borings between 8½ and 15 feet BSG. Detailed descriptions of the soils encountered are provided on the boring logs, attached at the end of this report.

Consolidation testing was conducted on two (2) soil samples collected from the proposed building area. The locations, depths, soil types and results of testing are included in Table 6.1, below. The results of testing performed on relatively undisturbed near surface soil samples indicate that the near surface soils exhibited moderate compressibility and low collapse potential.

TABLE 6.1 - RESULTS OF CONSOLIDATION TESTING

Boring/Depth of Sample	Soil Type	Total Consolidation %	Collapse/Swell Upon Wetting % At 2 kips normal
B-3 / 1.5-3’	Silty SAND	5.6	0.2% Collapse
B-4 /3.5-5’	Silty SAND	5.4	0.2% Collapse

The results of an expansion index test conducted on a near surface silty sand/sandy lean clay sample collected from depths of about 0 to 3 feet BSG indicated a low expansion index of 28.

The results of an R-value test conducted on a near surface soil sample indicates an R- value of 43.

Soil conditions described in the previous paragraphs are generalized. Therefore, the reader should consult exploratory boring logs in Appendix A for soil type, color, moisture, consistency, and USCS classification of the materials encountered at specific locations and elevations. Laboratory test result plates are included in Appendix B of this report.

6.2 Groundwater

During our field exploration, the borings were checked for the presence of groundwater. Groundwater was not encountered in the borings to the maximum depth explored of 50 feet BSG. Based on review of the seasonal groundwater contour maps for available yearly data from 2014 to 2023, provided on the Department of Water Resources (DWR) SGMA Portal: https://sgma.water.ca.gov/webgis/?appid=SGMADataViewer#gwlevels), the depth below ground surface to the unconfined groundwater aquifer in the immediate vicinity of the site ranged from over 200 feet to about 30 feet BSG during the years 2014 to 2023. The regional groundwater aquifer is not anticipated to impact the project.

The Department of Water Resources (DWR) Water Data Library website (http://www.water.ca.gov/) was reviewed for historic groundwater level data in the area of the site. Four (4) wells with historic groundwater table data are shown within about 1 mile of the site.

The results of about 6 measurements were provided for State Well Number 11S17E12E001M located about 2,200 feet west of the project site, for the period of December 1959 to February 1962. The highest and lowest groundwater levels reported corresponded to a depth of about 60 feet BSG in December 1959 and about 72 feet BSG in February 1962.

The results of about 20 measurements were provided for Site Code 369923N1200825W003 located about 4,200 feet northwest of the project site, for the period of September 2019 to October 2024. The highest and lowest groundwater levels reported corresponded to a depth of about 273 feet BSG in March 2021 and about 297 feet BSG in October 2024. Two (2) of the measurements, indicating groundwater at ground surface level, were considered unreliable.

The results of about 4 measurements were provided for State Well Number 11S1806P001M located about 5,200 feet northeast of the project site, for the period of March 1960 to December 1961. The highest and lowest groundwater levels reported corresponded to a depth of about 66 feet BSG in March 1960 and about 70 feet BSG in December 1960.

The results of about 37 measurements were provided for State Well Number 11S17E02Q001M located about 5,400 feet northwest of the project site, for the period of November 1944 to February 1965. The highest and lowest groundwater levels reported corresponded to a depth of about 56 feet BSG in November 1944 and about 92 feet BSG in February 1965. Data for this well indicates relatively consistent decline of groundwater levels after December 1946, with all measurements after 1950 indicating groundwater depths of greater than 57 feet.

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 2019 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. Near surface soil samples were obtained and tested for the evaluation of the potential for concrete deterioration and steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentrations detected in the saturation extract from the soil samples were 217 and 103 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

Sample Location/Depth	Water Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementations Materials Type
B-1/0-3’	0.0217	S0	N/A	2,500 psi	No Restriction
B-3/0-3’	0.0103	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentrations detected in the saturation extracts from the soil samples were 26 and 65 mg/kg. In addition, testing performed on the same soil samples as listed in the table above resulted in minimum resistivity values of 1,849 and 1,653 ohm-centimeter. Based on the results, the soils tested would be considered to have a “moderately corrosive” corrosion potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings). It is recommended that, at a minimum, applicable manufacturer’s recommendations for corrosion protection of buried metal pipe be closely followed.

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, applicable manufacturer's recommendations for corrosion protection of buried metal pipe be closely followed.

7. GEOLOGIC AND SEISMIC HAZARD EVALUATIONS

7.1 Geologic Setting

The project site is in San Joaquin Valley, which is a topographic and structural basin bound on the east by the Sierra Nevada geomorphic province and on the west by the Coast Ranges geomorphic province. The Coast Ranges are broken by numerous faults, the San Andreas Fault being the most notable feature. The Coast Ranges evolved as a result of folding, faulting and accretion of diverse geologic terrains and contain folded and faulted, chiefly Mesozoic and Cenozoic age sedimentary and metamorphic rocks. These rocks underlie the west portions of the Valley at depth and non-conformably overlie the basement complex.

The Sierra Nevada, an uplifted fault block dipping gently southwestward, is composed of mainly igneous and metamorphic rocks of pre-Tertiary age that comprise the basement complex beneath the Valley.

The San Joaquin (Great Valley Geomorphic Province) is an alluvial plain about 50 miles wide and 400 miles long in the central part of California (California Geologic Survey (CGS) Note 36). The Great Valley is an elongated trough in which sediments have been deposited almost continuously for the last approximately 160 million years (Jurassic), with sediments reaching depths of about 30,000 feet at its southern end. Surficial soils covering the majority of the valley floor comprise recent alluvium and basin deposits. Much of the eastern portions of the Valley have been uplifted exposing older alluvium (Pleistocene, non-marine) deposits derived from the adjacent Sierra Nevada. The shallow sediments in the Tulare-Selma area include both recent alluvium fan and Pleistocene non-marine deposits.

Based on review of the Geologic Map of California, Santa Cruz Sheet[1], the subject site is located in an area mapped as underlain by Pleistocene non-marine sedimentary deposits (Qc) described as: *"Older Alluvium, older fan deposits in the Great Valley having characteristic mature soil profile."*

A Regional Geologic Map is included as Figure No. 3 at the end of this report. Based on the relatively flat nature of the project area and uniform geologic conditions, site specific geologic cross sections are not determined necessary.

7.2 Geologic Hazards Evaluation

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

7.2.1 Flooding

Based on FEMA Flood Insurance Rate Map No. 06039C1155E, effective September 26, 2008, the subject site area is labeled as Zone X: *"Areas determined to be outside the 0.2% annual chance floodplain."* The flood hazard map is provided as Figure No. 6, attached to this report.

[1] Compilation by Charles W. Jennings and Rudolph G. Strand, 1958, Geologic Map of California, Santa Cruz Sheet, California Division of Mines and Geology, scale 1:250,000

The U.S. Army Corps of Engineers (USACE) website, National Inventory of Dams (NAD), indicates that the site would be impacted by flooding due to at least one breach scenario at Hidden Dam (Lake Hensley, located about 13 miles northeast of the site. The NID website risk assessment, dated October 17, 2017, states: *"Dams do not eliminate all risk of flooding. USACE works to address all types of flood risk associated with the dam. Dams have limited capacity to store water. Water may be released through the dam to manage water levels up or downstream or to relieve pressure on the dam to maintain its structural integrity. Severe weather events that bring inconsistent or larger amounts of water into the system can also lead to dam releases or in some cases overwhelm and lead to issues occurring at a dam … USACE manages dam-related flood risks by continually monitoring the condition and health of the dam, prioritizing activities that will most impact the risks, and engaging upstream and downstream emergency managers and members of the public to raise awareness of the dam and support actions to prepare and be ready to respond in the case of a dam-related emergency. USACE works closely with local emergency managers to share what is known about the dam and support the development of local emergency and evacuation plans. USACE regularly updates the emergency action plan for the dam. Regular maintenance and repairs are performed as needed to keep the dam functioning properly. More detail related to this specific dam will be added at a future time."*

No other dams are known to have the potential to cause flooding at the site due to breaching. Considering the information above, the potential for dam breach to cause flooding at the site is considered low.

7.2.2 Landslides

The site vicinity is flat. There are no known landslides at the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a hazard to this project.

7.2.3 Volcanic Activity

The subject site is not located within any designated volcanic hazard zones. 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, the Mono Lake-Long Valley Area is the closest to the site. The pyroclastic flow hazard zone (locally unprecedented) for this source is located as close as about 48 miles northeast of the site. Areas receiving 2 and 8 inches of compacted ash are estimated to be as close as 20 and 48 miles northeast of the site, respectively.

Based on the distance to the nearest volcanic hazard zones, the potential for volcanic hazards to impact the site during the design life of the facility is considered very low.

8. OTHER GEOLOGIC HAZARDS

8.1 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 soil types encountered and results of the laboratory tests performed, the near surface soils are considered to have a low expansion potential. Thus, the potential for damage to the proposed improvements caused by heave of expansive soils is considered low.

8.2 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).

Testing performed on a near surface soil resulted in a minimum resistivity values of 1,849 and 1,653 ohm-centimeter. Based on the results, these soils would be considered to have a "moderately corrosive" corrosion potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings).

8.3 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 2019 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. As indicated in Section 6.3 of this report, the exposure class of S0 was determined for two (2) soil samples obtained from the project site. Thus, the potential for concrete deterioration due to sulfate in soils is considered low.

9. CONDITIONAL GEOLOGIC HAZARDS:

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

9.1 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 very unlikely.

9.2 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.

9.3 Radon Gas

Based on review of the California Geologic Survey Indoor Radon Test Results[2] for the area of the site zip code (93638), two (2) of the twenty-two (22) test results indicated an indoor radon concentration of greater than or equal to the U.S. EPA action level for radon in air of 4 picocuries per liter. Considering the test results, and that the building is expected to be adequately ventilated with no basement, the potential for indoor radon exposure is not considered a concern for this project.

2 https://www.cdph.ca.gov/Programs/CEH/DRSEM/CDPH%20Document%20Library/EMB/Radon/Radon%20Test%20Results.pdf

9.4 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, ultramafic rock is identified about 33 miles northeast of the site. Based on review of the Open-File Report 2011-1188, Reported Historic Asbestos Mines, Historic Asbestos Prospects, and Other Natural Occurrences of Asbestos in California, prepared by the California Geological Survey and U.S. Geological Survey, dated August, 2011, the nearest reported occurrence of asbestos (prospect) is about 32 miles to the northeast. 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 or soil at the site is very low.

9.5 Hydro-collapse

Collapsible soils typically consist of loose, dry, low-density soils that, when wetted, will experience settlement/consolidation. Based on the soils encountered in the test borings and the results of testing performed on relatively undisturbed near surface soil samples, the near surface site soils exhibit moderate compressibility and low collapse potential. This report includes recommendations to reduce the potential for damage to buildings resulting from hydro-collapse by over-excavation and support foundations and floor slabs on engineered fill.

9.6 Regional Subsidence

Based on our review of the USGS article titled *"Land Subsidence in the San Joaquin Valley"*, dated October 17, 2018, the site is not located in an area of recorded subsidence due to groundwater pumping. Therefore, regional subsidence is not considered a concern for this project.

10. SEISMIC HAZARDS

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

10.1 Active Faulting and Surface Fault Rupture

Numerous active and potentially active faults are located in the site region and contribute to design seismic ground motion estimates. An "active fault" is defined, for the purpose of this evaluation, as a fault that has had surface displacement within the Holocene age (about the last 11,700 years). Based on the distance to active faults in the region, as well as the historic seismic record, the area of the subject site is considered to be subject to low to moderate seismicity.

The project area is not located within an Earthquake Fault Rupture Hazard Zone and a fault rupture hazard investigation is not required.

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*, supplemented with the Fault Activity Map of California-web application (California Geological Survey). The ten (10) active seismic faults closest to the site are summarized below in Table 10.1.

TABLE 10.1
SUMMARY OF REGIONAL ACTIVE SEISMIC FAULTS

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Great Valley 11	39.7	6.6
Great Valley 9	40.0	6.8
Great Valley 13 (Coalinga)	46.7	7.1
Ortigalita	48.4	7.1
Great Valley 8	51.7	6.8
Great Valley 14 (Kettleman Hills)	59.3	7.2
Quien Sabe	65.7	6.6
Great Valley 7	68.5	6.9
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO	73.5	8.2
Hartley Springs	73.9	6.8

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.

The site is not located within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards (Special Studies Zone). The nearest active fault segments to the project site are Segments 11 and 9 of the Great Valley fault about 40 miles to the southwest. However, these blind thrust fault segments do not exhibit surface rupture. The nearest active seismic fault with the potential for surface rupture is the Ortigalita fault, located about 48 miles west of the site. A map depicting the major active faults in the vicinity of the site is included on Figure No. 4 at the end of this report. 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 very low.

10.2 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 281 historical earthquakes with magnitude 4.5 or greater have been recorded from January 1, 1900 through February 4, 2025, within about 100 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 on Figure No. 5 at the end of this report.

The nearest earthquake event (estimated magnitude of 4.6) found during the search occurred on August 3, 1975, approximately 2 miles south-southwest of Three Rocks, California. The highest magnitude earthquake identified within a 100 mile search radius was the 6.7 magnitude Coalinga Earthquake, located near Coalinga, California, which occurred on July 21, 1952, approximately 40 miles southwest of the site.

10.3 Design Seismic Ground Motion Parameters and Site Class

Seismic coefficients and spectral response acceleration values were developed based on the 2022 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. A site specific ground motion hazard analysis was not included in this investigation. Based on our understanding of the proposed project the project Structural Engineer will utilize code exceptions listed in ASCE 7-16 section 11.4.8 for design of the planned foundations. Therefore, Site Specific Ground Motion Hazard Analysis is not required.

Based on the 2022 CBC, a Site Class D represents the on-site soil conditions with a weighted average, standard penetration resistance, N-value, averaging between 15 and 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 D (stiff soil) designation, is included in Section 11.6.1 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 (PGAm) was determined to be 0.338g.

10.4 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. A seismic hazard, which could potentially cause damage to the proposed development during seismic shaking, is the post-liquefaction settlement of the liquefied sands.

The area of the site has not been mapped by the State of California Seismic Hazard Zonation Program and the site is not located in a locally designated liquefaction hazard zone.

Liquefaction and seismic settlement were evaluated using LiquefyPro computer program (version 5.9c) developed by Civiltech. A maximum earthquake magnitude of 5.5 M_w (based on deaggregation of the 2 percent probability in 50 year seismic event using the USGS Unified Hazard Tool, Dynamic Conterminous U.S. 2014 v4.2.0), and a design peak horizontal ground surface acceleration of 0.338 g (PGA_M) were used in the analysis. Soil data provided on the log for boring B-3 were used in the analysis.

Groundwater was not encountered in the borings drilled for our field investigation, to a maximum depth of 50 feet BSG. Based on the historic groundwater level data referenced in Section 6.2, an historic high groundwater depth of 50 feet was used in the analysis.

Based on our analysis, liquefaction would is not predicted to occur. Total dry seismic induced settlement is expected to be about 0.1 inch total and the differential seismic settlement is estimated to be negligible. The analysis result summary and graph are included after the boring logs in Appendix A of this report.

10.5 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. Considering the results of the liquefaction analysis and the relatively flat nature of the site, we judge the likelihood of lateral spreading to be negligible.

11. CONCLUSIONS AND RECOMMENDATIONS

11.1 General

11.1.1 Based upon the data collected during this investigation, 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.

11.1.2 The near surface soil conditions encountered in the area of the track improvements, within the upper 5 feet BSG, were predominantly silty sand, clayey sand, sandy silt, with subordinate low plastic lean clay. Much of these soils were noted to be very dense or hard, with weak to moderate cementation (hardpad). Below these upper soils, the soils predominantly comprised very stiff to hard silt and lean clay layers, and medium dense to very dense silty sand and clayey sand layers extending to the maximum depth explored of 51½ feet BSG. Also, medium dense and dense poorly graded sands with silt were encountered in two (2) borings between 8½ and 15 feet BSG.

11.1.3 The results of testing performed on relatively undisturbed near surface soil samples indicate that the near surface soils in the proposed building area exhibited moderate compressibility and low collapse potential. Considering the over-excavation recommendations under Section 11.3 of this report, the potential for damage due to hydro-collapse of soils is considered very low.

11.1.4 Based on the soil types encountered and results of the laboratory tests performed, the near surface soils are considered to have a low expansion potential. Thus, the potential for damage due to heave of expansive soils is considered low.

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

11.1.6 Provided the site is graded in accordance with the recommendations of this report, we would estimate a total settlement due to static loads utilizing conventional shallow foundations of about 1-inc, with a corresponding differential static settlement of ½ inch in 40 feet. Total seismic settlement of 0.1 inch and negligible differential seismic settlement are also estimated.

11.1.7 Laboratory tests indicate the near surface soils have a sulfate exposure Class S0 (refer to Table 6.3 for requirements). Based on the testing performed, the near surface soils have “moderately corrosive” degree of corrosivity to buried metal objects.

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

11.1.9 SALEM should be retained to review the project plans as they develop further, provide engineering consultation as-needed, and perform geotechnical observation and testing services during construction.

11.2 Surface Drainage

11.2.1 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.

11.2.2 The exposed ground immediately adjacent to foundations shall be sloped away from the building at a slope of not less than 5 percent for a minimum distance of 10 feet. Impervious surfaces within 10 feet of building foundations shall be sloped a minimum of 1 percent away from the building and drainage gradients maintained to carry all surface water to collection facilities and off site. These grades should be maintained for the life of the project. Ponding of water should not be allowed adjacent to the structures. Over-irrigation within landscaped areas adjacent to the structures should not be performed.

11.2.3 Roof drains should be installed with appropriate downspout extensions out-falling on splash blocks so as to direct water a minimum of 10 feet away from the structures or be connected to the storm drain system for the development. Grading and drainage design should prevent ponding of surface water within 15 feet of the track and/or building.

11.2.4 Storm water infiltration should not be designed to occur within 20 feet from the building. Unlined bioswales or storm water basins shall be a located a minimum of 20 feet from proposed building foundations. If required to install bioswales or basins within 20 feet of the building, the bioswales/basins should be lined with an impermeable liner.

11.3 Site Grading

11.3.1 A representative of our firm should be present during all site clearing and grading operations to test and/or 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.

11.3.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor (including demolition and grading contractors), civil engineer and geotechnical engineer in attendance.

11.3.3 Site preparation should begin with stripping of vegetation and demolition/removal of existing surface/subsurface structures in areas of the proposed new improvements, hardscape and aggregate base (if present), underground utilities (as required), disturbed soil, trees, and existing uncertified/undocumented fill (if any). 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. The stripped vegetation will not be suitable for use as Engineered Fill anywhere on the project. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

Excavations or depressions resulting from site clearing and demolition operations, tree removal, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report. <u>It is expected demolition of the existing improvements may disturb the upper subgrade soils.</u> Any disturbed subgrade, undocumented fill materials or loose unsuitable materials encountered during grading should be removed and replaced as engineered fill.

11.3.4 Site demolition activities shall include removal of all surface and subsurface obstructions not intended to be incorporated into final site design. In addition, undocumented fill, underground buried structures, and/or utility lines encountered during demolition and construction should be properly removed and the resulting excavations backfilled with Engineered Fill. SALEM should be retained to observe site demolition activities involving removal of subsurface structures, trees, etc. and to document/test the placement of engineered fill placed to restore the excavations.

11.3.5 If existing trees are to be removed, their root systems should be thoroughly cleared of root balls as well as isolated roots greater than ¼-inch in diameter. The root system removal may disturb a significant quantity of soil. Following tree removal, all loose and disturbed soil should be removed from the tree wells. Any areas or pockets of soft or loose soils, void spaces made by burrowing animals, undocumented fill, or other disturbed soil (i.e. soil disturbed by root removal) that are encountered, should be excavated to expose approved firm native material. Care should be taken during site grading to mitigate (e.g. excavate and compact as engineered fill) all soil disturbed by demolition and tree removal activities. SALEM should be retained to document removal of tree roots and to document/test the placement of engineered fill placed to restore the excavations.

11.3.6 Where fill is to be placed on existing slopes (such as the existing basin slope) an inclination of 6H to 1V or steeper, such as at the existing basin, fill slope grading should commence with constructing a minimum 6-foot wide keyway below the toe of the new fill slope. Excavation of the keyway should be to a minimum depth of 3 feet below preconstruction site grade and extend from the toe of the slope at least 6 feet in the upslope direction. The bottom of the keyway should slope down at about 2 percent in the upslope direction. The bottom of the keyway should be scarified to a depth of 8 inches and compacted prior to placement of fill. Prior to backfilling the keyway and construction of the new slope, the contractor should survey to document the elevations and aerial extent of the bottom of keyway and provide the survey to the project engineer. The engineered fill placed on existing slopes with an inclination of 6H to 1V or steeper should be placed on a near horizontal surface and benched horizontally into the existing slope.

Benching should include cutting horizontally at least 3 feet beyond the pre-grading slope profile. Individual bench heights should be a minimum of 18 inches.

11.3.7 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.

11.3.8 The structural building pad areas and over-build zones should be considered as areas extending throughout the entire building area and a minimum of 5 feet horizontally beyond the outside dimensions of buildings, including footings and non-cantilevered overhangs carrying structural loads, and to 3 feet beyond the edges of new exterior slabs adjacent to the building, whichever is further.

11.3.9 To provide uniform support for the proposed building over-excavation should be conducted to minimum depths of one (1) foot below existing grade, to the bottom of proposed footings, or to the depth required to remove undocumented fills (if encountered), whichever is deeper. The over-excavation should be uniform throughout the building pad and extend laterally to a minimum of 5 feet beyond the outer edges of the exterior of the building and proposed footings. The resulting bottom-of-excavation shall be scarified to a depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to slightly above optimum moisture, and compacted to a minimum of 92 percent of the maximum density, prior to placement of engineered fill.

If the engineered fill soils placed 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.

11.3.10 Interior slabs on grade should be supported on engineered fill described in Section 11.3.9 and a minimum of 8 inches of non-recycled Class 2 aggregate base compacted to 95 percent relative compaction, over the depth of engineered fill recommended below foundations (Section 11.3.9).

11.3.11 In areas of proposed lightly loaded shallow spread foundations or mat foundations outside the building pad, for retaining walls, screen walls, or bleacher), it is recommended that over-excavation be extended to at least one (1) foot below preconstruction site grade, to the bottom of foundations, or to the depth required to remove any loose undocumented fill soils (if any encountered), whichever is greater. Upon approval by the geotechnical engineer, the resulting bottom of excavation shall be scarified to a minimum depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to slightly above optimum moisture, and compacted to at least 92 percent of the maximum density. The horizontal limits of the over-excavation should extend throughout the footing area and over-build zone, laterally to a minimum of 3 feet beyond the outer edges of the proposed footings.

11.3.12 Areas of exterior concrete slabs on grade (hardscape, sidewalks, etc.) located outside the building pad over-build zone (see Section 11.3.8 for over-build zone) should be prepared by scarification, moisture conditioning, and compaction of the upper 12 inches below existing grade as engineered fill, or scarification, moisture conditioning, and compaction of the upper 12 inches below the bottom of the recommended aggregate base section, whichever is deeper. These soils should be moisture conditioned to one (1) to four (4) percent above optimum and compacted as engineered fill. The zone of subgrade preparation should extend a minimum of 2 feet horizontally beyond the edges of the slab. It is recommended that exterior slabs on grade be supported on a minimum of 4 inches of Class 2 aggregate base compacted to 95 percent relative compaction, over compacted subgrade soils. This recommendation is made to provide a smooth firm surface to pour slab concrete and to reduce the potential of slab cracking that could result from indentations of native subgrade soils. As an alternative, if the School District is willing to accept additional risk for distress to exterior slabs, slabs on grade located outside the building overbuild zone (Section 11.3.8) may be supported directly over subgrade soils compacted as recommended above.

11.3.13 Areas proposed for asphaltic concrete under the track surface, or asphaltic concrete or Portland cement concrete pavements should be prepared by scarification, moisture conditioning, and compaction of the upper 12 inches below existing grade as engineered fill, or scarification, moisture conditioning, and compaction of the upper 12 inches below the bottom of the recommended aggregate base section, whichever is deeper. These soils should be moisture conditioned to one (1) to four (4) percent above optimum and compacted as engineered fill. The zone of subgrade preparation should extend a minimum of 2 feet beyond the edges of the track/pavements. Prior to placement of aggregate base, the subgrade soils should be proof-rolled by a loaded water truck (or equivalent) to verify no deflections of greater than ½ inch occur. 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.

11.3.14 Areas to receive only engineered fill outside the basin slope and improvement areas (described above) should be prepared by scarification of the upper 12 inches below existing grade after stripping. These soils should be moisture conditioned to slightly above optimum and compacted as engineered fill.

11.3.15 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.

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

11.3.17 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. 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.

11.3.18 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. All open graded crushed rock/gravel should be fully encapsulated with a geotextile fabric (such as Mirafi 140N) to minimize migration of soil particles into the voids of the crushed rock. Although it is not required, the use of geogrid (e.g. Tensar BX 1100, BX 1200 or TX 160) 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.

11.4 Soil and Excavation Characteristics

11.4.1 Based on the soil conditions encountered in our borings, the onsite soils can be excavated with low to moderate difficulty using conventional excavation equipment. It should be noted that hardpan was encountered in our borings and additional effort will be required to excavate this material and to reduce hardpan fragment dimensions to 3 inches or less, and blend to achieve a well graded soil mixture to be used as engineered fill.

11.4.2 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. Temporary excavations are further discussed in a later Section of this report.

11.4.3 The near surface soils identified as part of our investigation were, generally, damp due to the absorption characteristics of the soil. Earthwork operations conducted during wet inclement periods of the year 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.

11.5 Engineered Fill Materials

11.5.1 On-site soils are considered suitable for use as general Engineered Fill, provided they do not contain deleterious matter, organic material, or material larger than 3 inches in maximum dimension. Hardpan fragments will need to be reduced as discussed above.

11.5.2 Imported Non-Expansive Engineered Fill soil (if required), should be well-graded, very low-to-non-expansive slightly cohesive silty sand or sandy silt. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 11.5.2.

TABLE 11.5.2
IMPORT FILL REQUIREMENTS

Percent Passing 3-inch Sieve	100
Percent Passing No.4 Sieve	75-100
Percent Passing No 200 Sieve	15-40
Maximum Plasticity Index	10
Maximum Organic Content	3% by Weight
Maximum Expansion Index (ASTM D4829)	10

Proposed import materials should be sampled, tested for geotechnical properties, and approved by SALEM prior to its transportation to the site. Prior to importing fill, the Contractor shall have the source sampled and 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 (DTSC), "Informational Advisory, Clean Imported Fill Material," dated October 2001. The list of constituents to be tested for the fill source and a map of proposed sample locations shall be submitted to the project owner for review prior to the Contractor sampling testing the fill. Contractors should provide a minimum of 14 working days after sample collection to complete the DTSC and geotechnical testing.

11.5.3 All Engineered Fill (including scarified ground surfaces and backfill) should be placed in lifts no thicker than will allow for adequate bonding and compaction (maximum 8 inches in loose thickness).

11.5.4 On-Site derived engineered Fill soils used as engineered fill soils should be moisture conditioned to one (1) to four (4) percent above optimum moisture content, and compacted to at least 92 percent relative compaction (ASTM D1557). Soils placed or compacted within 12 inches below the aggregate base section for the track or pavements should be moisture content, and compacted to at least 95 percent relative compaction (ASTM D1557).

11.5.5 Imported soils used as engineered fill soils should be moisture conditioned to slightly above optimum moisture content, and compacted to at least 92 percent relative compaction (ASTM D1557). Soils placed or compacted within 12 inches below the aggregate base section for the track or pavements should be moisture content, and compacted to at least 95 percent relative compaction (ASTM D1557).

11.5.6 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.

11.5.7 Proposed import materials should be sampled, tested for geotechnical properties, and approved by SALEM prior to its transportation to the site.

11.5.8 Environmental characteristics (Section 11.5.2) and corrosion potential of import soil materials should also be considered.

11.5.9 Aggregate base material should meet the requirements of a Caltrans Class 2 Aggregate Base. Aggregate base placed within the limits of proposed building pads should be non-recycled. 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.

11.6 Seismic Design Criteria

11.6.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2022 CBC, our recommended parameters are shown below. These parameters were determined using California's Office of Statewide Health Planning and Development (OSHPD) (https://seismicmaps.org/) in accordance with the 2022 CBC. The Site Class was determined based on the soils encountered during our field exploration. Based on our understanding of the project, the Structural Engineer will utilize code exceptions summarized under ASCE 7-16, Section 11.4.8. Therefore, a site specific ground motion hazard analysis is not required.

TABLE 11.6.1
2022 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	ASCE 7-16 or 2022 CBC Reference
Site Coordinates	--	Lat: 36.9898 Long: -120.0653	---
Site Class	--	D	ASCE 7-16 Table 20.3
Soil Profile Name	--	Stiff Soil	ASCE 7-16 Table 20.3
Risk Category	--	II	CBC Table 1604.5
Site Amplification Factor at PGA	F_{PGA}	1.35	ASCE 7-16 Table 11.8-1

Seismic Item	Symbol	Value	ASCE 7-16 or 2022 CBC Reference
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.338g	ASCE 7-16 Equation 11.8-1
Seismic Design Category	SDC	D	ASCE 7-16 Table 11.6-1 & 2
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	0.58 g	CBC Figure 1613.2.1(1)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.23 g	CBC Figure 1613.2.1(3)
Site Class Modified Site Coefficient	F_a	1.336	CBC Table 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	**2.140***	CBC Table 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	0.775 g	CBC Equation 16-20
MCE Spectral Response Acceleration (1.0 sec. period) $1.5*S_{M1} = 1.5 (F_v S_1)$	$1.5 * S_{M1}$	**0.738 g***	CBC Equation 16-21 / ASCE 7-16 Supplement 3
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	0.517 g	CBC Equation 16-22
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	**0.492 g***	CBC Equation 16-23
Short Period Transition Period (S_{D1}/S_{DS}), Seconds	T_S	0.952	ASCE 7-16, Section 11.4.6
Long Period Transition period (seconds)	T_L	12	ASCE 7-16, Figures 22-14 through 22-17

***Note: * Values Fv, SM1, and SD1 determined per ASCE Table 11.4.2 for use in calculating TS only. These values should not be used in structural design. Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, Structures on Site Class D, with S1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. The value reported for SM1 includes a 50% increase in accordance with exceptions listed in ASCE 7-16 - Supplement 3. In the event a site specific ground motion analysis is required, SALEM should be contacted for these services.**

11.6.2 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.

11.7 Shallow Foundations

11.7.1 The site is suitable for use of conventional shallow foundations consisting of continuous footings and isolated pad footings supported on engineered fill soils prepared in accordance with the recommendations under Section 11.3 of this report. Maximum wall load and column loads are anticipated to be on the order of to be on the order of 1 to 3 kips per linear foot and up to about 30 kips, respectively. In the event that the design structural loads exceed these values, SALEM should be contacted to provide alternate recommendations. Shallow foundations supported on engineered fill as recommended in this report may be designed based on total and differential static settlements of 1 inch and ½ inch in 40 feet, respectively. Total seismic settlement of 0.1 inch and negligible differential seismic settlement are also estimated.

11.7.2 The bearing wall footings for the subject building should be continuous with a minimum width of 12 inches, and extend to a minimum depth of 12 inches below the lowest adjacent grade.

11.7.3 Lightly loaded foundations for screen walls, retaining walls, etc., should have a minimum width of 12 inches and minimum depth of 12 inches below adjacent grade.

11.7.4 Footing concrete should be placed into neat excavation. The footing bottoms shall be maintained free of loose and disturbed soil.

11.7.5 Foundations for the building, supported on engineered fill as recommended in this report, may be designed based on an allowable bearing capacity of 2,500 pounds per square foot (dead plus live load). This value may be increased by 1/3 for wind and seismic loading.

11.7.6 Shallow conventional foundations for the non-habitable structures outside building pad, supported on the minimum thickness of engineered fill recommended in this report for those structures, may be designed based on an allowable bearing capacity of 2,000 pounds per square foot (dead plus live load). This value may be increased by 1/3 for wind and seismic loading. Shallow foundations for the non-habitable structures outside building pad, supported on engineered fill as recommended in this report, may be designed based on total and differential static settlements of 1 inch and ½ inch in 40 feet, respectively.

11.7.7 Resistance to lateral footing displacement can be computed using an allowable coefficient of friction factor of 0.35 acting between the base of foundations and the supporting subgrade.

11.7.8 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 275 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 in Section 1605.3.2 of the 2022 CBC that includes wind or earthquake loads.

11.7.9 Reinforced slabs/mat foundations, if used for bleachers, may be designed utilizing a modulus of subgrade reaction (K-value) of 140 pounds per square inch per inch. This value is based on a one-foot square plate with a maximum load of 1 kip. The design engineer should apply a modulus of subgrade reaction value which incorporates the size of the bearing pressure area in design of the mat foundation slab.

11.7.10 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.

11.7.11 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.

11.8 Pile Foundations for Lighting Poles, Shade Structures, Signs, and Playground Equipment

11.8.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the cast-in-drilled-hole (CIDH) pile foundations for support of lighting poles, shade structures, signs and playground equipment based on total static settlement of 1 inch and differential static settlement of ½ inch between foundations. Total seismic settlement of 0.1 inch and negligible differential seismic settlement are also estimated. We recommend that the pile footings for the stadium lighting have a minimum diameter of 18 inches and extend a minimum depth of 6 feet below the lowest adjacent grade.

11.8.2 Soil descriptions are provided in Section 6.1 of this report and include granular soils (silty sands, etc.) with relatively low cohesion and low stand-up capacity. Borings B-1 and B-2 encountered poorly graded sand with silt at depths between 8½ and 15 feet BSG. The poorly graded sand with silt is expected to have a very low stand up capacity (very susceptible to sloughing).

11.8.3 Piles should be placed no closer than three pile diameters (center to center). For alternate spacing, the capacity of the piles in groups should be reduced using appropriate group reduction formulas.

11.8.4 CIDH piles extending to a depth of at least 6 feet below the lowest adjacent grade may be designed using a downward allowable side friction of 120 pounds per square foot. CIDH piles extending to a depth of at least 10 feet below the lowest adjacent grade may be designed using a downward allowable side friction of 185 pounds per square foot. The side friction for the upper 1 foot of subgrade soils should be neglected in design. This value is for dead-plus-live loads. An increase of one-third may be applied for wind or earthquake loads. End bearing resistance should be neglected.

11.8.5 Lateral load resistance may be estimated using the CBC non-constrained procedure (CBC Section 1806.8.2.1). Passive lateral resistance should be neglected to a depth of 1 foot below the lowest ground surface at the pile, or to a depth providing a horizontal setback to a sloping ground surface (slope face) of at least 5 feet, whichever is deeper. The passive resistance of the CIDH pile foundations below the neglect depth (piles spaced at a minimum of three (3) pile diameters) may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot (psf/ft), to a maximum of 2,500 pounds per square foot. These values may be increased by one-third for wind and seismic loading. For example, where a passive pressure of 250 pounds per cubic foot per foot is recommended, a passive pressure of 500 pounds per cubic foot per foot could be applied across the pile diameter.

11.8.6 The uplift resistance of the pile foundations may be determined based on a tension load capacity applied as skin friction of 110 pounds per square foot below a depth of 1 foot below the lowest grade directly adjacent to the pile. The weight of the pile may also be used in combination with the skin friction to resist uplift.

11.8.7 The sandy soils encountered have a moderate to high potential for caving during shaft drilling operations (i.e. not stand vertical), as noted in Section 11.8.2. The Contractor should evaluate these conditions and consider use of temporary casing or other methods. Temporary casing used for support of drilled pile excavations during construction should be slowly removed from the shaft excavation during placement of concrete while ensuring the casing is not raised above the level of the concrete during shaft construction. The bottom of the casing should be lifted slowly as the concrete is deposited and kept at least two feet below the top of the concrete to avoid sloughing soils from mixing with the concrete.

11.8.8 Casing (where used) should be able to withstand the external pressures of the caving soils. The outside diameter of the casing should not be less than the design diameter of the CIDH pile.

11.8.9 Drilled holes for pile foundations should be drilled within 2 degrees of vertical. The rebar cage should be suspended within 2 degrees of vertical in the center of the excavation. Minimum concrete cover, as specified by the project design engineer, should be maintained throughout the length of the excavation. These conditions should be verified and documented by the CTL during construction.

11.8.10 Loose materials should be removed from the bottom of the drilled shaft excavations prior to placement of reinforcing steel and concrete by use of a clean-out bucket or other acceptable methods to effectively remove loose materials.

11.8.11 SALEM should inspect the drilling of the shafts to verify that the materials encountered are consistent with those evaluated during our geotechnical engineering investigation. **This inspection should be conducted during drilling and prior to placement of reinforcing steel and concrete.**

11.9 Interior Concrete Slabs-on-Grade

The following recommendations are intended for the interior slabs on grade.

11.9.1 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 eight (8) inches of non-recycled Caltrans Class 2 aggregate base compacted to 95 percent relative compaction, over engineered fill extending below foundations (see Sections 11.3.9 and 11.3.10).

11.9.2 At a minimum, it is recommended that welded wire or fiber mesh reinforcement be used in interior slabs. The type of reinforcement should be selected by the structural engineer.

11.9.3 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 full depth construction joints or control joints be provided at a maximum spacing of 15 feet in each direction for 5-inch thick slabs.

11.9.4 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.

11.9.5 It is recommended that the utility trenches within the structure area 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.

11.9.6 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.

11.9.7 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 incorporated into the floor slab design (a minimum of 15 mil thick, is recommended, polyethylene vapor retarder sheeting, Raven Industries "VaporBlock 15, Stego Industries 15 mil "StegoWrap" or W.R. Meadows Sealtight 15 mil "Perminator"). 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 permeance **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.

11.9.8 The concrete maybe 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. Extend vapor retarder over footings and seal to foundation wall or slab at an elevation consistent with the top of the slab or terminate at impediments such as water stops or dowels. Seal around penetrations such as utilities or columns in order to create a monolithic membrane between the surface of the slab and moisture sources below the slab as well as at the slab perimeter.

11.9.9 Avoid use of stakes driven through the vapor retarder.

11.9.10 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. 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.

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

11.10 Exterior Concrete Slabs on Grade

11.10.1 The following recommendations are intended for lightly loaded exterior slabs on grade not subject to vehicular traffic (i.e. hardscape, sidewalks, etc.). 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 four (4) inches of Caltrans Class 2 aggregate base over subgrade soils prepared in accordance with the recommendations in Section 11.3.12 of this report. As an alternative, if the School District is willing to accept additional risk for distress to exterior slabs, slabs on grade located outside the building pad may be supported directly over compacted subgrade soils as recommended in Section 11.3.12 of this report. In the event that the District elects to allow placement of exterior slabs directly on prepared native subgrade soils, the contractor should ensure/document that the subgrade soils upon which to pour the exterior concrete slabs are prepared as required by this report and the upper surface is smooth and firm.

11.10.2 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 full depth 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.

11.10.3 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.

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

11.11 Lateral Earth Pressures and Frictional Resistance

11.11.1 Retaining walls retaining greater than 5 feet of backfill are not anticipated. SALEM's geotechnical engineering department should be contacted if retaining walls retaining greater than 5 feet of soil are planned, and supplemental recommendations may be warranted. Lateral earth pressures and coefficient of friction for retaining wall design are provided below based on drained conditions and use of onsite silty sandy soils or select imported backfill behind the wall (see under Section 11.5 for import fill recommendations). All retaining walls should be drained (see under Section 11.12). Retaining walls should NOT be designed for active pressure unless the shell is expected to rotate at least 0.0005 radians at the top. The at-rest soil pressure is applicable to retaining structures that are fully fixed against both rotation and translation. Retaining wall reinforcement should be designed by a structural engineer to accommodate any expected surcharge loads (such as adjacent foundations), if any.

11.11.2 If on-site soils are to be used as retaining wall backfill, the soils should be stockpiled and evaluated by the geotechnical engineer. On-site soils used for retaining wall backfill should be approved by the geotechnical engineer prior to use as backfill. When approved on-site soils, or import soils meeting the recommendations of Section 11.5 are used as wall backfill, the following allowable active, at-rest, and passive pressures may be used.

Lateral Pressure Conditions	Soil Equivalent Fluid Pressure
Active Pressure, Drained, pcf	43
At-Rest Pressure, Drained, pcf	65
Allowable Passive Pressure, pcf	275
Allowable Coefficient of Friction	0.35
Minimum Wet Unit Weight (lbs/ft^3)	100
Maximum Wet Unit Weight (lbs/ft^3)	130

11.11.3 Active pressure applies to walls, which are free to rotate (see Section 11.11.1). 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. The top one-foot of adjacent subgrade should be deleted from the passive pressure computation.

11.11.4 The allowable parameters include a safety factor of 1.5 and can be used in design for direct comparison of resisting loads against lateral driving loads.

11.11.5 If combined passive and frictional resistance is used in design, a 50 percent reduction in frictional resistance is recommended.

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

11.11.7 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 = Maximum In-Place Soil Density (Section 11.11.2 above)
K_h = Horizontal Acceleration = $\frac{2}{3}PGA_M$ (Section 11.6.1 above)
H = Wall Height

11.12 Retaining Walls

11.12.1 Retaining walls retaining greater than 5 feet of backfill are not anticipated for this project. SALEM's geotechnical engineering department should be contacted if retaining walls retaining greater than 5 feet of soil are planned, and supplemental recommendations may be warranted. Retaining walls should be backfilled with engineered fill soils (see Section 11.11.2). 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 2 permeable materials graded in accordance with the current Caltrans Standard Specifications.

11.12.2 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.

11.12.3 Drainage pipes should be placed with perforations down and should discharge in a non-erosive manner away from foundations and other improvements.

11.12.4 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.

11.12.5 Retaining walls retaining greater than 5 feet of backfill are not anticipated for this project. For retaining walls retaining less than 5 feet of soil, 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.

11.12.6 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.

11.13 Design and Construction of Pavements for Track and Vehicles

11.13.1 New pavement subgrade soils should be prepared as recommended in Section 11.3.13 of this report. Considering the soil types and shallow hardpan encountered, soil water may become trapped on relatively impermeable soils acting as barriers to the downward migration of water through the soil. Storm water and over-irrigation impacting the grass areas near the track could migrate below the track and affect subgrade performance. <u>The outer and inner edges of the track should have a deep mow-strip/curb extending to a depth of at least 24 inches below the top of the track, or 24 inches below the lowest finished ground level adjacent to the curb.</u>

11.13.2 The pavement design recommendations provided herein are based on the State of California Department of Transportation (CALTRANS) design manual and the results of the R-value testing performed. An R-value of 43 was utilized for design of project pavements.

11.13.3 Table 11.13.3 presents minimum sections recommended for flexible asphaltic concrete pavement design and a minimum constructible aggregate base section thickness of 4 inches, and a minimum asphaltic concrete section of 2.5 inches. The pavement design recommendations are provided based on a 20-year pavement life.

TABLE 11.13.3
ASPHALT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Asphaltic Concrete, (inches)	Class 2 Aggregate Base, (inches)*	Compacted Subgrade, (inches)**
4.0	2.5	4.0	12.0
4.5	2.5	4.0	12.0
5.0	2.5	5.5	12.0
6.0	3.0	6.5	12.0
7.0	4.0	7.0	12.0
8.0	4.5	8.5	12.0

** Minimum recommended constructible AC and AB sections for flexible asphaltic concrete.*
*** 95% minimum compaction of AC and AB based on ASTM D1557 Test Method.*

11.13.4 The following recommendations are for Portland Cement Concrete pavement sections.

TABLE 11.13.4
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Portland Cement Concrete, (inches)*	Class II Aggregate Base, (inches)**	Compacted Subgrade. (inches)**
4.0	5.5	4.0	12.0
5.0	5.5	4.0	12.0
6.0	6.0	4.0	12.0
7.0	6.0	4.0	12.0
8.0	6.5	4.0	12.0

** Minimum Compressive Strength of 4,000 psi*
*** 95% compaction based on ASTM D1557 Test Method*

11.13.5 Asphalt concrete should conform to Section 39 of Caltrans' latest Standard Specifications for ½ inch Hot Mix Asphalt (HMA) Type A or B. Asphaltic concrete pavements should be placed and compacted in accordance with Caltrans Standard Specifications.

11.13.6 Excavations, depressions, or soft and pliant areas extending below planned finished subgrade levels should be cleaned to firm, undisturbed soil and backfilled with Engineered Fill. Any buried structures encountered during construction should be properly removed and backfilled.

11.13.7 Buried structures encountered during construction should be properly removed/rerouted and the resulting excavations backfilled. It is suspected that demolition activities of the existing pavement will disturb the upper soils. After demolition activities, it is recommended that disturbed soils within pavement areas be removed and/or compacted as engineered fill under the observation and testing of SALEM.

11.13.8 An integral part of satisfactory fill placement is the stability of the placed lift of soil. The subgrade soils should be proof-rolled by a loaded water truck (or equivalent) to verify no deflections of greater than ½ inch occur, prior to placement of aggregate base or pavements (AC or PCC). 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.

11.13.9 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.

11.14 Temporary Excavations

11.14.1 We anticipate that the majority of the 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.

11.14.2 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.

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

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

TABLE 11.14.4
MAXIMUM RECOMMENDED EXCAVATION SLOPES

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

11.14.5 If, due to space limitation, excavations near existing structures are performed in a vertical position, braced shoring 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.

11.14.6 Braced shoring should be designed for a maximum uniform pressure distribution of 30H, (where H is the depth of the excavation in feet). The foregoing does not include 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.

11.14.7 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.

11.15 Underground Utilities

11.15.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as final backfill (above 12 inches above the pipe) 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 92 percent relative compaction at or above optimum moisture content. The upper 12 inches of trench backfill within asphalt or concrete paved areas shall be moisture conditioned to at or above optimum moisture content and compacted to at least 95 percent relative compaction.

11.15.2 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 imported, 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.

11.15.3 It is suggested that underground utilities crossing beneath proposed or existing structures/foundations (or under the track) be plugged at entry and exit locations to the building/structure/track to prevent water migration. For utilities crossing under proposed structures/foundations/track, trench plugs should consist of controlled low strength material (CLSM) as described below. The trench plugs should extend 2 feet beyond each side of individual perimeter foundations. The CLSM should have a compressive strength of 100 to 150 psi and be vibrated in place. The CLSM should fill the utility trench, extend to at least 2 feet beyond each edge of the existing foundation, and should extend up to the bottom of the foundation. A CLSM mix design should be provided by the contractor at least 1 week prior to the scheduled CLSM pour.

The contractor shall also schedule testing and inspection for CLSM, with the testing and inspection of CLSM consistent with that required for the shallow foundations.

11.15.4 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.

12. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

12.1 Plan and Specification Review

12.1.1 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.

12.2 Construction Observation and Testing Services

12.2.1 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.

12.2.2 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.

12.2.3 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.

13. 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 No. 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 (559) 271-9700.

SALEM ENGINEERING GROUP, INC.

CERTIFIED ENGINEERING GEOLOGIST
KENNETH J. CLARK
No. 1864
exp 5-31-25
STATE OF CALIFORNIA

Ken Clark, CEG 1864
Senior Engineering Geologist

CERTIFIED ENGINEERING GEOLOGIST
Dean B. Ledgerwood II
No. 2613
Exp 12/31/25
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
DEAN B. LEDGERWOOD II
C 94395
EXP. 03/31/27
CIVIL
STATE OF CALIFORNIA

Dean B. Ledgerwood II, PE, PG, CEG
Geotechnical Manager
PE 94395 / PG 8725 / CEG 2613

R. Sammy Salem, MS, PE, GE
Principal Managing Engineer
RCE 52762 / RGE 2549

REGISTERED PROFESSIONAL ENGINEER
R. SAMMY SALEM
GE 2549
EXP. 12-31-2026
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

California Department of Public Health Indoor Radon Test Results, https://www.cdph.ca.gov/Programs/CEH/DRSEM/CDPH%20Document%20Library/EMB/Radon/Radon%20Test%20Results.pdf

California Division of Mines and Geology staff, Compilation by Charles W. Jennings and Rudolph G. Strand, 1958, Geologic Map of California, Santa Cruz Sheet, California Division of Mines and Geology, scale 1:250,000.

California Geological Survey, Open-File Report 2000-19, August 2000, A General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos

California Geological Survey and U.S. Geological Survey, Open-File Report 2011-1188, Reported Historic Asbestos Mines, Historic Asbestos Prospects, and Other Natural Occurrences of Asbestos in California, dated August, 2011, Van Gosen, B. S. (USGS) and Clinkenbeard, J. P. (CGS).

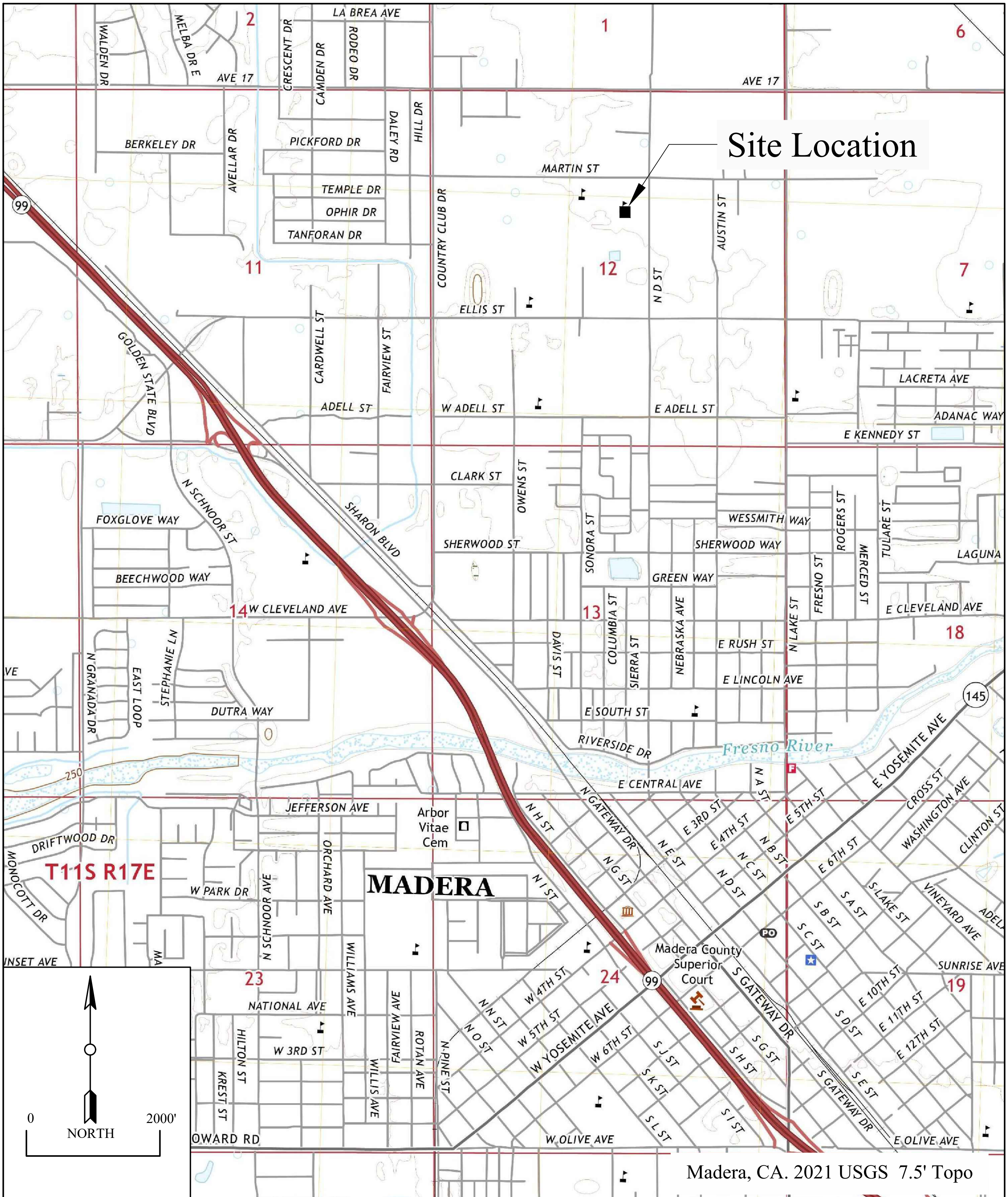
Federal Emergency Management Agency, Flood Insurance Rate Maps, Community Panel No. 06039C155E, effective September 26, 2008.

Miller, C. D., 1989, Potential Hazards from Future Volcanic Eruptions in California, U. S. Geologic Survey Bulletin 1847.

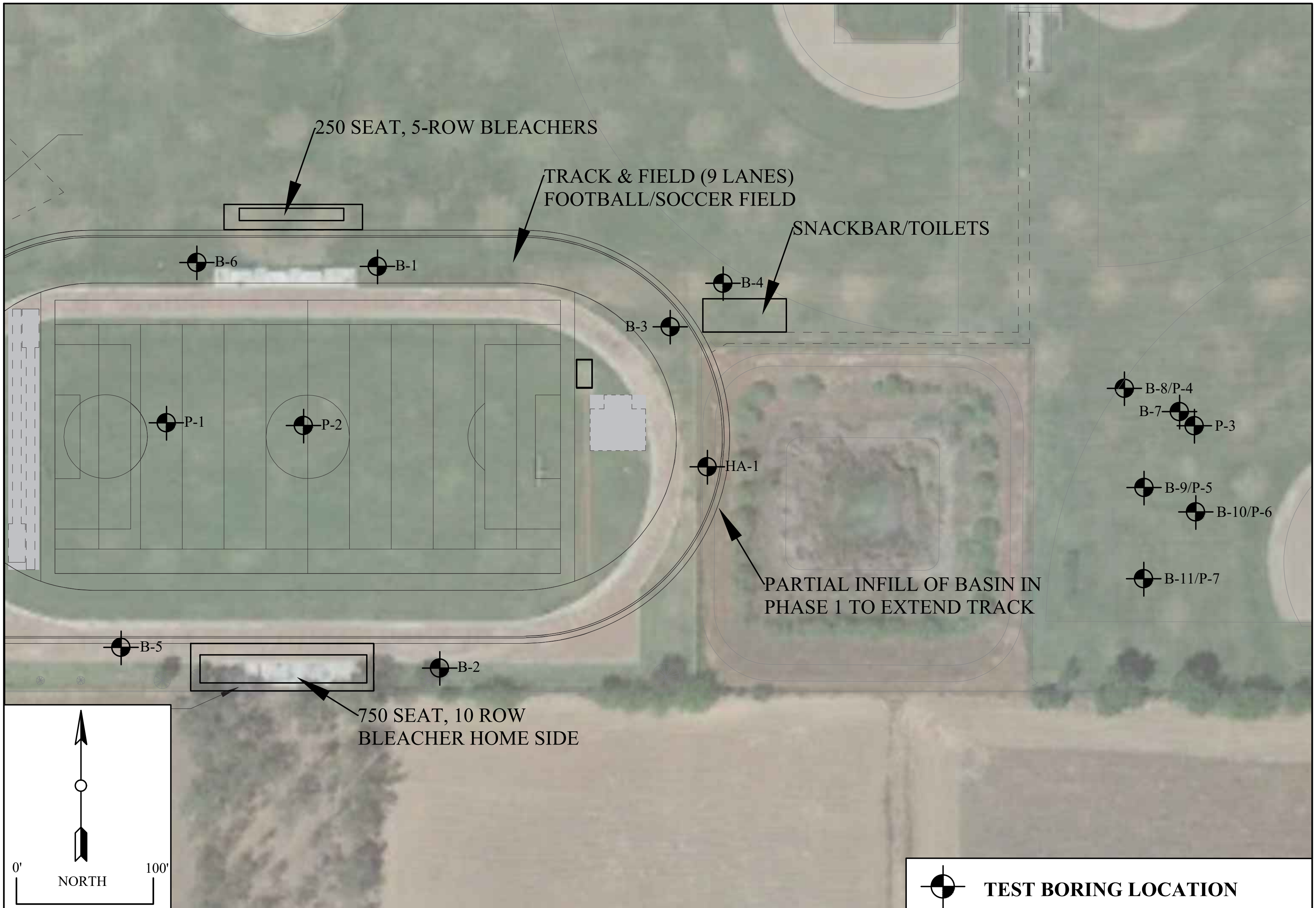
OSHPD California's Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps (https://seismicmaps.org/)

United States Geological Survey, Circular Area Earthquake Search" (http:// earthquake.usgs.gov/earthquakes/eqarchives/epic/epic_circ.php)

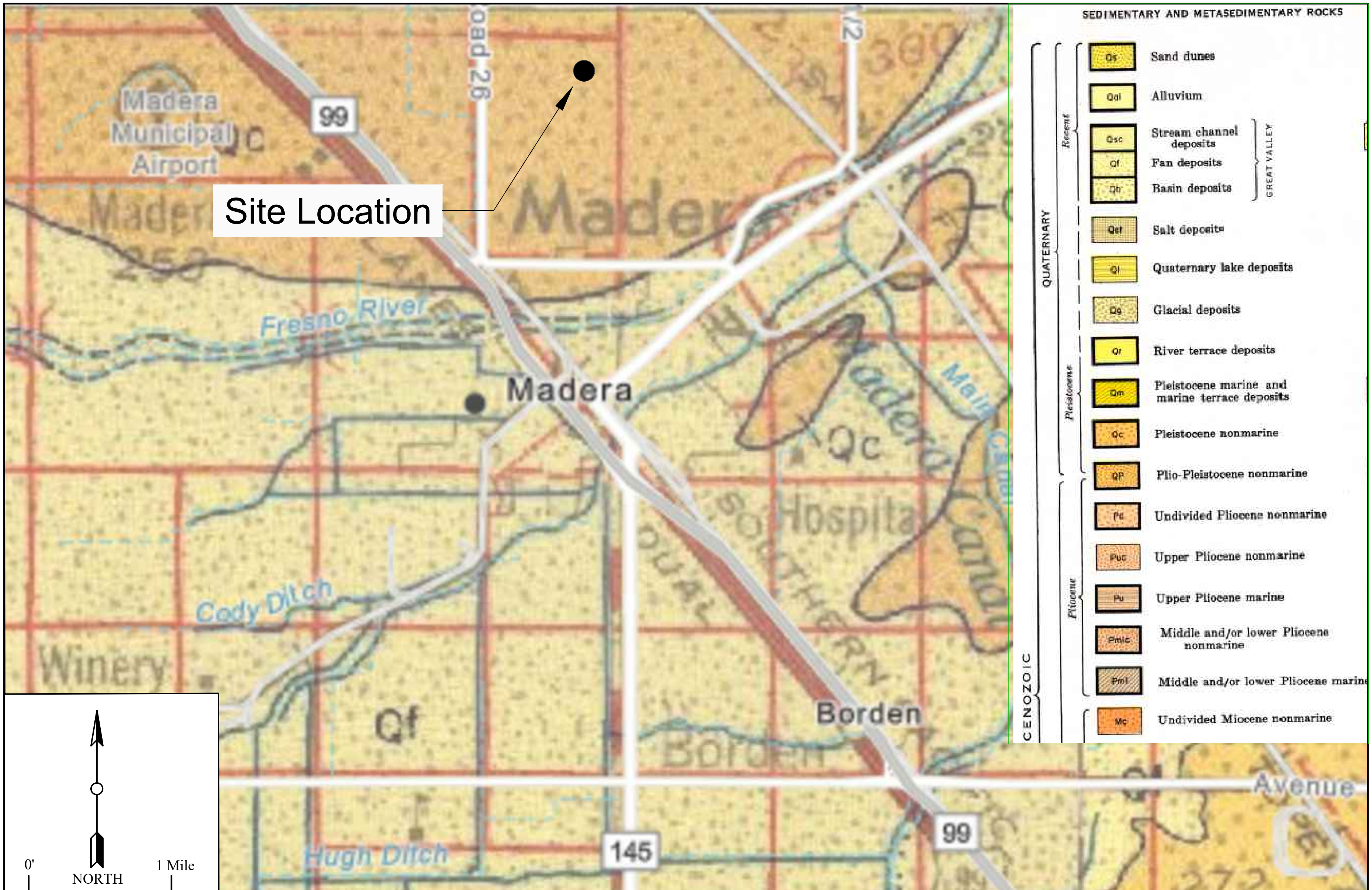
USGS Land Subsidence in the San Joaquin Valley, dated October 17, 2018, Justin Brandt https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html



VICINITY MAP	SCALE: 1" = 2000'	DATE: Feb. 2025	SALEM engineering group, inc.
GEOTECHNICAL ENGINEERING INVESTIGATION MODERNIZATION OF ATHLETIC FACILITIES DESMOND MIDDLE SCHOOL 26490 MARTIN STREET MADERA, CALIFORNIA	DRAWN BY: VT	APPROVED BY: KC	
	PROJECT NO. 1-224-1068 A	FIGURE NO. 1	

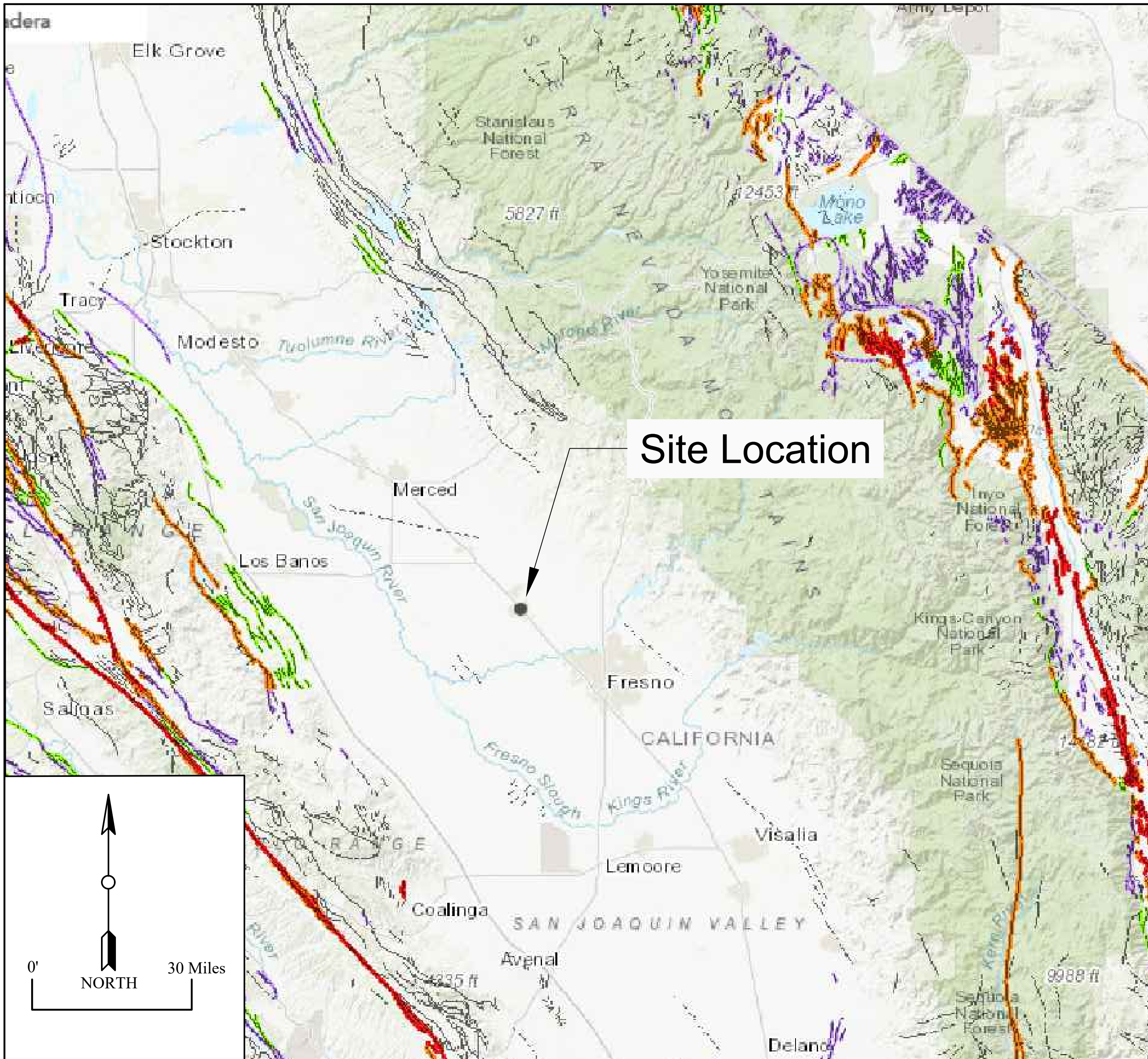


SITE PLAN	SCALE: 1" = 100'	DATE: Dec. 2024	SALEM engineering group, inc.
MODERNIZATIONS OF ATHLETIC FACILITIES DESMOND MIDDLE SCHOOL 26490 MARTIN STREET MADERA, CALIFORNIA	DRAWN BY: VT	APPROVED BY: KC	
	PROJECT NO. 1-224-1068	FIGURE NO. 2	



GEOLOGIC MAP OF CALIFORNIA OLAF P. JENKINS EDITION SANTA CRUZ SHEET - COMPILATION BY CHARLES W. JENNINGS, & RUDOLPH G. STRAND 1958

REGIONAL GEOLOGIC MAP	SCALE: 1"= 1 Mile	DATE: Feb. 2025	SALEM engineering group, inc.
MODERNIZATIONS OF ATHLETIC FACILITIES DESMOND MIDDLE SCHOOL 26490 MARTIN STREET MADERA, CALIFORNIA	DRAWN BY: VT	APPROVED BY: DL	
	PROJECT NO. 1-224-1068 A	FIGURE NO. 3	



SYMBOL EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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

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.

Date bracketed by triangles indicates local fault break.

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.

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

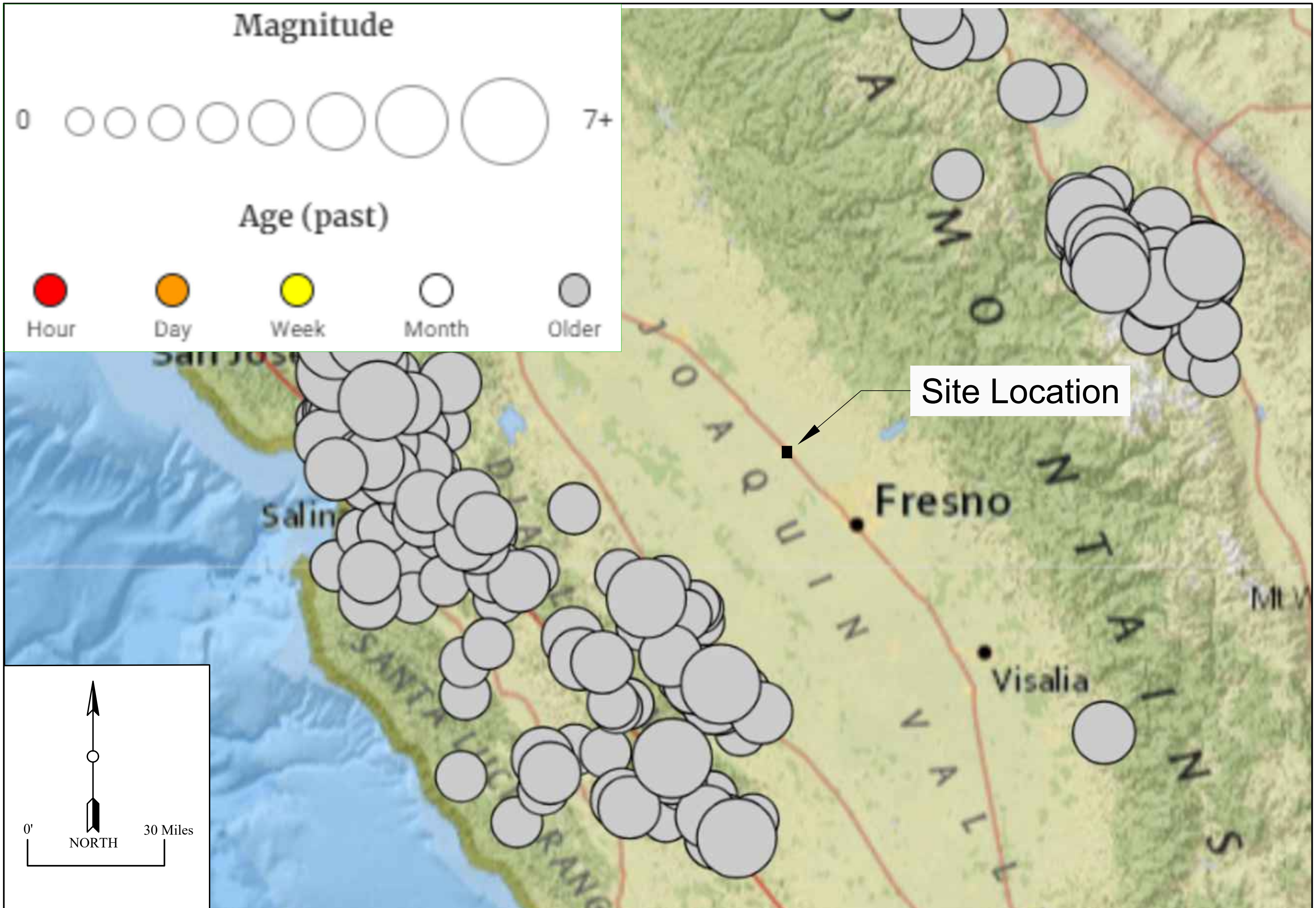
Bar and ball on downthrown side (relative or apparent).

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

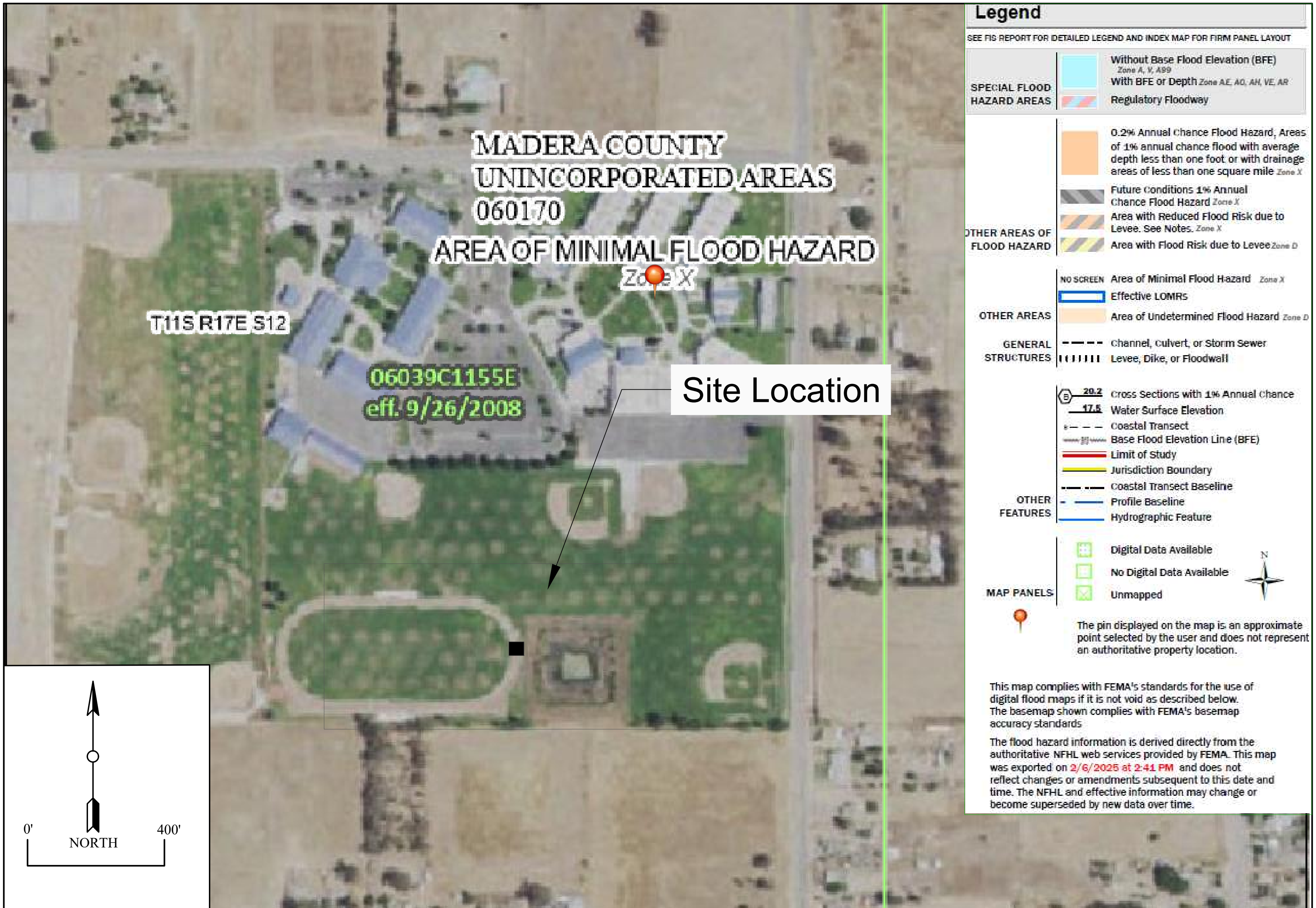
Arrow on fault indicates direction of dip.

FAULT LOCATION MAP	SCALE: 1"= 30 Miles	DATE: Feb. 2025
MODERNIZATIONS OF ATHLETIC FACILITIES DESMOND MIDDLE SCHOOL 26490 MARTIN STREET MADERA, CALIFORNIA	DRAWN BY: VT	APPROVED BY: DL
	PROJECT NO. 1-224-1068 A	FIGURE NO. 4

SALEM
engineering group, inc.



EARTHQUAKE EPICENTER LOCATION MAP	SCALE: 1"= 30 Miles	DATE: Feb. 2025	SALEM engineering group, inc.
MODERNIZATIONS OF ATHLETIC FACILITIES DESMOND MIDDLE SCHOOL 26490 MARTIN STREET MADERA, CALIFORNIA	DRAWN BY: VT	APPROVED BY: DL	
	PROJECT NO. 1-224-1068 A	FIGURE NO. 5	



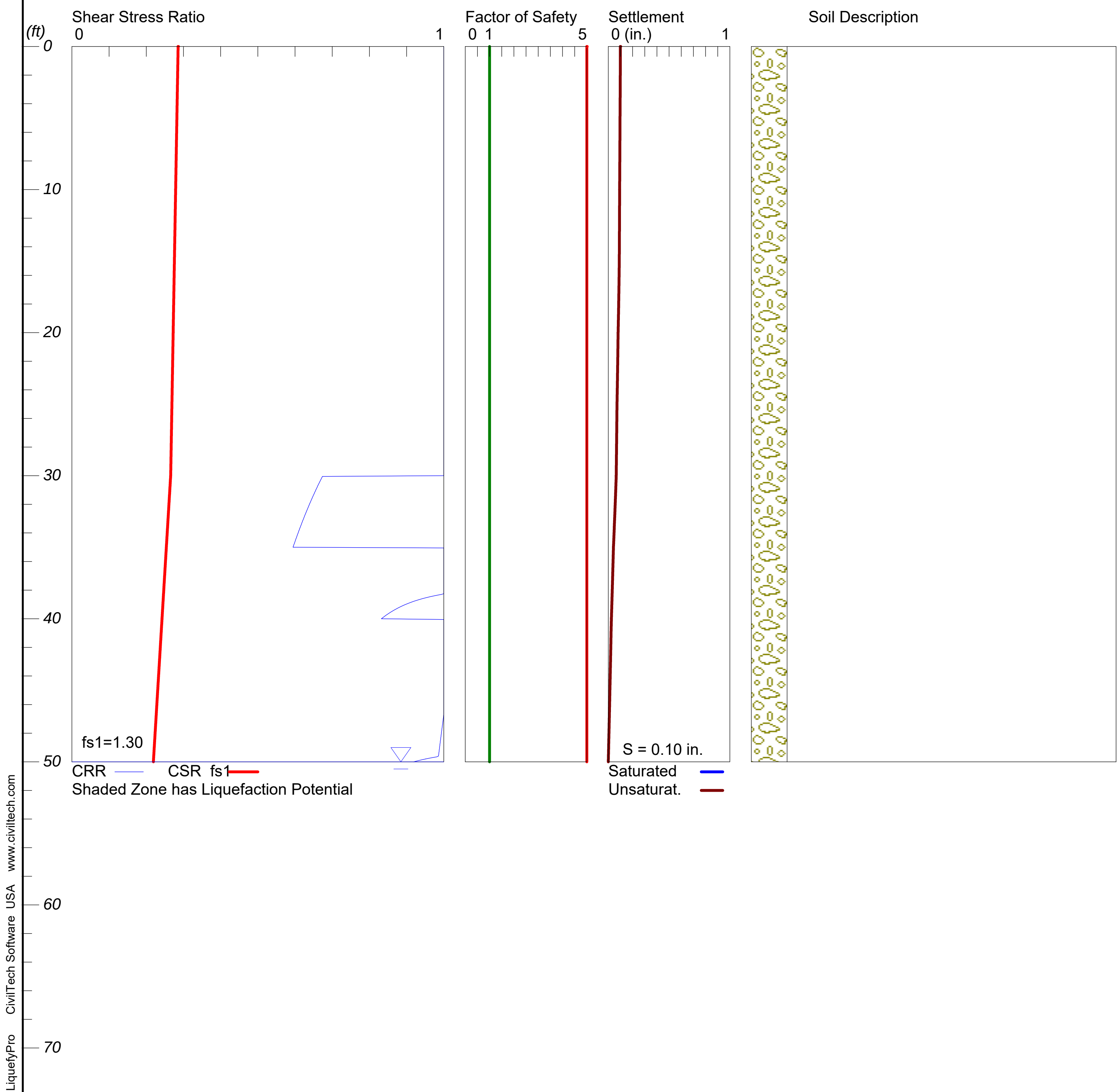
FLOOD ZONE MAP	SCALE: 1"= 400'	DATE: Feb. 2025	SALEM engineering group, inc.
MODERNIZATIONS OF ATHLETIC FACILITIES DESMOND MIDDLE SCHOOL 26490 MARTIN STREET MADERA, CALIFORNIA	DRAWN BY: VT	APPROVED BY: DL	
	PROJECT NO. 1-224-1068 A	FIGURE NO. 6	

LIQUEFACTION ANALYSIS

Desmond Middle School

Hole No.=B-3 Water Depth=50 ft Surface Elev.=280

Magnitude=5.5
Acceleration=0.338g



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LIQUEFACTION ANALYSIS SUMMARY

****
****

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Input File Name: UNTITLED
Title: Desmond Middle School
Subtitle:

Surface Elev.=280
Hole No.=B-3
Depth of Hole= 50.00 ft
Water Table during Earthquake= 50.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration= 0.34 g
Earthquake Magnitude= 5.50

Input Data:
Surface Elev.=280
Hole No.=B-3
Depth of Hole=50.00 ft
Water Table during Earthquake= 50.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration=0.34 g
Earthquake Magnitude=5.50
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio,   Ce = 1.35
7. Borehole Diameter,   Cb= 1
8. Sampling Method,   Cs= 1.2
9. User request factor of safety (apply to CSR) ,   User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: No
* Recommended Options

In-Situ Test Data:
Depth   SPT   gamma   Fines

ft		pcf	%
0.00	29.00	120.00	48.00
1.50	14.00	120.00	25.00
5.00	20.00	120.00	65.00
10.00	50.00	120.00	40.00
15.00	17.00	120.00	25.00
20.00	18.00	120.00	55.00
25.00	25.00	120.00	55.00
30.00	16.00	120.00	25.00
35.00	19.00	120.00	55.00
40.00	24.00	120.00	25.00
50.00	22.00	120.00	25.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.10 in.
Total Settlement of Saturated and Unsaturated Sands=0.10 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	1.11	0.29	5.00	0.00	0.10	0.10
1.00	1.11	0.28	5.00	0.00	0.10	0.10
2.00	1.11	0.28	5.00	0.00	0.10	0.10
3.00	1.11	0.28	5.00	0.00	0.10	0.10
4.00	1.11	0.28	5.00	0.00	0.10	0.10
5.00	1.11	0.28	5.00	0.00	0.10	0.10
6.00	1.11	0.28	5.00	0.00	0.10	0.10
7.00	1.11	0.28	5.00	0.00	0.10	0.10
8.00	1.11	0.28	5.00	0.00	0.09	0.09
9.00	1.11	0.28	5.00	0.00	0.09	0.09
10.00	1.11	0.28	5.00	0.00	0.09	0.09
11.00	1.11	0.28	5.00	0.00	0.09	0.09
12.00	1.11	0.28	5.00	0.00	0.09	0.09
13.00	1.11	0.28	5.00	0.00	0.09	0.09
14.00	1.11	0.28	5.00	0.00	0.09	0.09
15.00	1.11	0.28	5.00	0.00	0.09	0.09
16.00	1.11	0.27	5.00	0.00	0.09	0.09
17.00	1.11	0.27	5.00	0.00	0.09	0.09
18.00	1.11	0.27	5.00	0.00	0.09	0.09
19.00	1.11	0.27	5.00	0.00	0.08	0.08
20.00	1.11	0.27	5.00	0.00	0.08	0.08
21.00	1.11	0.27	5.00	0.00	0.08	0.08
22.00	1.11	0.27	5.00	0.00	0.08	0.08
23.00	1.11	0.27	5.00	0.00	0.08	0.08
24.00	1.11	0.27	5.00	0.00	0.07	0.07
25.00	1.11	0.27	5.00	0.00	0.07	0.07

26.00	1.11	0.27	5.00	0.00	0.07	0.07
27.00	1.11	0.27	5.00	0.00	0.07	0.07
28.00	1.11	0.27	5.00	0.00	0.07	0.07
29.00	1.10	0.27	5.00	0.00	0.07	0.07
30.00	1.09	0.27	5.00	0.00	0.07	0.07
31.00	0.66	0.26	5.00	0.00	0.06	0.06
32.00	0.64	0.26	5.00	0.00	0.06	0.06
33.00	0.62	0.26	5.00	0.00	0.05	0.05
34.00	0.61	0.26	5.00	0.00	0.05	0.05
35.00	0.59	0.25	5.00	0.00	0.04	0.04
36.00	1.06	0.25	5.00	0.00	0.04	0.04
37.00	1.05	0.25	5.00	0.00	0.04	0.04
38.00	1.05	0.25	5.00	0.00	0.03	0.03
39.00	0.90	0.24	5.00	0.00	0.03	0.03
40.00	0.83	0.24	5.00	0.00	0.03	0.03
41.00	1.03	0.24	5.00	0.00	0.02	0.02
42.00	1.02	0.24	5.00	0.00	0.02	0.02
43.00	1.02	0.24	5.00	0.00	0.02	0.02
44.00	1.01	0.23	5.00	0.00	0.02	0.02
45.00	1.01	0.23	5.00	0.00	0.01	0.01
46.00	1.00	0.23	5.00	0.00	0.01	0.01
47.00	1.00	0.23	5.00	0.00	0.01	0.01
48.00	0.99	0.22	5.00	0.00	0.01	0.01
49.00	0.99	0.22	5.00	0.00	0.00	0.00
50.00	0.92	0.22	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

_

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. Test borings B-1 through B-11, HA-1, P-1, and P-2 were drilled on November 14th, 19th and December 23, 2024, to depths ranging from 5 to 50 feet below site grade (BSG). Thirteen (13) of the borings were drilled to depths of 5 to 51½ feet BSG using 6-5/8 inch diameter hollow-stem auger rotated by truck-mounted CME-55 and CME-75 drill rigs. The remaining boring (HA-1) was drilled to a depth of about 3½ feet BSG using hand auger equipment. The approximate locations of the exploratory borings are shown on the Site Plan, Figure No. 2.

Soil samples were obtained from the test borings at the depths shown on the logs of borings. Soil sampling 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.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2487). 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 estimated 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. 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 this appendix should be consulted. Where applicable, the field logs were revised based on subsequent laboratory testing.

The Modified California Sampler (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 test boring logs are presented in this appendix include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbols. The locations of the test borings were determined by using existing reference points. Therefore, actual boring locations may deviate slightly. Upon completion, the borings were backfilled with drill cuttings.

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

SALEM engineering group, inc.

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 5/8 in. Hollow Stem **Initial Depth to Groundwater:** N/E

Hammer Type: 140lbs./30in. Automatic trip **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
280 / 0		ML	Sandy SILT; light brown, damp to moist, mostly fines, very dense.				At 0-3': SAND=42% -#200=57% +#4=1%
	14/6 27/6 42/6	CL	Lean CLAY; reddish brown, damp, low plasticity, hard, trace fine sand.	>50	12.7	118.2	
	5/6 10/6 15/6		Grades as above; very stiff.	25	13.9		PI=8 LL=24
275 / 5							
270 / 10	11/6 20/6 27/6	SP-SM	Poorly Graded SAND with Silt; reddish brown, moist, dense, fine to medium grained.	47	11.3	121.8	
265 / 15	8/6 16/6 20/6	CL	Sandy Lean CLAY; brown, dry to damp, hard, low to medium plasticity.	36	13.7		SAND=49% -#200=51%
260 / 20	6/6 8/6 11/6	SM	Silty SAND; brown, damp, mostly fines, medium dense.	19	13.5		
			End of boring at 20 feet BSG.				
255 / 25							

Notes: *Grass field surface outside track.*

Figure Number A-1

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

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

SALEM engineering group, inc.

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 5/8 in. Hollow Stem **Initial Depth to Groundwater:** N/E

Hammer Type: 140lbs./30in. Automatic trip **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
280 / 0	3/6 50/4	SM	Silty SAND; reddish brown, very dense, damp, fine grained, with interbedded cemented soils (hardpan).	>50	13.4	70.2	
275 / 5	13/6 19/6 31/6		Grades as above; light brown, very dense, damp, fine sand.	50	5.3	118.5	
	15/6 15/6 16/6	CL	Lean CLAY; light brown, very stiff, damp, low to medium plasticity.	31	13.6		
270 / 10	8/6 10/6 16/6	SP-SM	Poorly Graded SAND with Silt; brown, moist, fine sand, medium dense.	26	5.7	93.6	
265 / 15	5/6 11/6 17/6	ML	SILT with sand; light brown, very stiff, damp, non-plastic, interbedded with lean clay (low to medium plasticity).	28	11.3	111.8	
260 / 20	5/6 5/6 6/6		Silt; light brown, very stiff, damp, non- plastic.	11	12.9		
			End of boring at 21.5 feet BSG.				
255 / 25							

Notes: *Grass field surface outside track.*

Figure Number A-2

SALEM engineering group, inc.

Test Boring: B-3 **Page** 1 **Of:** 2

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 5/8 in. Hollow Stem **Initial Depth to Groundwater:** N/E

Hammer Type: 140lbs./30in. Automatic trip **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
280 / 0	3/6 13/6 16/6	CL	Clayey SAND; medium dense, reddish brown, moist, weakly cemented.	29	13.2		SAND = 48% -#200 = 48% +#4 = 4% R = 43 EI = 28 ø = 33° c' = 743 psf
	9/6 10/6 10/6	SM	Silty SAND; brown, medium dense, damp, weakly cemented, fine grained, trace of clay.	20	11.7	114.5	
275 / 5	4/6 8/6 12/6	CL	Sandy Lean CLAY; very stiff, orangish brown, wet, fine to coarse sand.	20	10.2		PI=11 LL=29
270 / 10	14/6 29/6 40/6	SC	Clayey SAND; very dense, reddish brown, damp to moist.	>50	17.7	113.7	
265 / 15	6/6 9/6 8/6	SM	Silty SAND; light brown, damp, medium dense.	17	12.1		
260 / 20	4/6 6/6 12/6	ML	SILT with sand; brown, very stiff, damp to moist, low to medium plasticty.	18	21.1		
255 / 25	4/6 10/6 15/6		Grades as above; light brown, very stiff, moist, low to medium plasticity, trace clay.	25			

Notes:

Figure Number A-3

SALEM engineering group, inc.

Project Number: 1-224-1068

Date: November 14, 2024

Test Boring: B-3

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
250 — 30	4/6 6/6 10/6	SM	Silty SAND; light brown, moist, medium dense, some fines.	16			
245 — 35	5/6 6/6 13/6	ML	SILT with sand; light brown, moist, non-plastic, very stiff.	19			
240 — 40	8/6 10/6 14/6	SM	Silty SAND; light brown, moist, mostly fines, medium dense.	24			
235 — 45	6/6 11/6 13/6	SM	Grades as above.	24			
230 — 50	3/6 9/6 13/6		Grades as above.	22			
			End of boring at 51.5 feet BSG.				
225 — 55							
220 — 60							

Notes:

Figure Number A-3

SALEM engineering group, inc.

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

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 in. Solid Stem **Initial Depth to Groundwater:** N/E

Hammer Type: 140lbs./30in. Automatic trip **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
280 / 0		SM	Silty SAND; medium dense, orangish brown, dry to damp, trace clay.	29	8.8		
	14/6 12/6 17/6						
	11/6 19/6 32/6		Grades as above; dark brown, moist, very dense, fine to medium grained.	>50	13.3	117.4	ø = 47° c' = -63 psf
275 / 5							
	50/3		Grades as above; orangish brown, moist, very dense, medium grained, cemented (hardpan).	>50			
270 / 10			End of boring at 10 feet BSG.				
265 / 15							
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-4

SALEM engineering group, inc.

Test Boring: B-5 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 in. Solid Stem **Initial Depth to Groundwater:** N/E

Hammer Type: 140lbs./30in. Automatic trip **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
280 / 0	21/6 23/6 37/6	SM	Silty SAND; brown, dense, dry to damp, cemented (hardpan).	>50	8.6		
275 / 5	30/6 34/6 22/6		Grades as above; brown, damp, fine grained, dense.	>50	10.4	108.5	
270 / 10	5/6 7/6 16/6		Grades as above; reddish brown, dry to damp, medium dense, fine to medium grained, with stringers of hardpan.	23	4.7		
			End of boring at 10 feet BSG.				
265 / 15							
260 / 20							
255 / 25							

Notes: *Dirt track surface.*

Figure Number A-5

Test Boring: B-6 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 in. Solid Stem **Initial Depth to Groundwater:** N/E

Hammer Type: 140lbs./30in. Automatic trip **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
280 / 0	11/6 29/6 40/6	CL-ML	Silty Lean CLAY; hard, light brown, moist, low plasticity, trace fine sand.	>50	9.4	120.8	
	12/6 21/6 50/5	SM	Silty SAND; very dense brown, damp to moist, fine to medium grained.	>50	8.6	128.1	
275 / 5			End of boring at 5 feet BSG.				
270 / 10							
265 / 15							
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-6

Test Boring: HA-1 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: November 19, 2024

Client: Madera Unified School District

SALEM engineering group, inc.

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: Hand Auger **Elevation:** 280 feet AMSL.

Auger Type: 6 in. **Initial Depth to Groundwater:** N/E

Hammer Type: N/A **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
280 / 0		SM	Silty SAND; brown, damp to moist, fine to medium grained. Cemented/hardpan at 0.5 feet. Grades as above. Grades as above; some cementation. Grades as above; cemntation/hard pan.				
			Hand Auger refusal at 3.5 feet BSG.				
275 / 5							
270 / 10							
265 / 15							
260 / 20							
255 / 25							

Notes:

Figure Number A-7

SALEM engineering group, inc.

Test Boring: P-1 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 5/8 in. Hollow Stem **Initial Depth to Groundwater:** N/E

Hammer Type: N/A **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
280 / 0	11/6 24/6 35/6	ML	Silt; very hard, light brown, damp, cemented (hardpan.	>50			
275 / 5	5/6 10/6 26/6		Sandy SILT; very stiff, slightly moist, brown.	36			
			End of boring at 6 feet BSG.				
270 / 10							
265 / 15							
260 / 20							
255 / 25							

Notes:

Figure Number A-8

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

Project Number: 1-224-1068

Date: November 14, 2024

Client: Madera Unified School District

SALEM engineering group, inc.

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R.

Drill Type: CME-75 **Elevation:** 280 feet AMSL.

Auger Type: 6 5/8 in. Hollow Stem **Initial Depth to Groundwater:** N/E

Hammer Type: N/A **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
280 / 0	8/6 18/6 14/6	ML	SILT; hard, light brown, damp, weakly cemented hardpan.	32			
275 / 5	34/6 50/4		Sandy SILT; hard, brown, moist, weakly cemented, fine sand.	>50			
			End of boring at 6 feet BSG.				
270 / 10							
265 / 15							
260 / 20							
255 / 25							

Notes:

Figure Number A-9

SALEM engineering group, inc.

Test Boring: B-7/P-3 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: 12/23/2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** RS

Drill Type: CME 55 **Elevation:** 280ft. AMSL

Auger Type: 6-5/8in. Hollow Stem Auger **Initial Depth to Groundwater:** N/E

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
280 / 0	3/6 4/6 7/6	SM	Silty SAND; medium dense, reddish brown, damp, fine grained.	11	4.6		
275 / 5	6/6 9/6 8/6		Moist, fine to medium gravel, slightly cemented, trace clay.	17	11.6		
	5/6 4/6 4/6		Grades as above; loose, brown, moist, fine grained.	8	12.1		SAND=63% -#200=37%
270 / 10	4/6 5/6 16/6			21	13.6		
	11/6 14/6 17/6	CL	Lean CLAY; hard, light brown, medium plasticity, trace sand.	31	26.0		PI=16 LL=40
265 / 15			End of boring at 15 feet BSG.				
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-10

Test Boring: B-8/P-4 **Page** 1 **Of:** 1

SALEM engineering group, inc.

Project Number: 1-224-1068

Date: 12/23/2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** RS

Drill Type: CME 55 **Elevation:** 280ft. AMSL

Auger Type: 6-5/8in. Hollow Stem Auger **Initial Depth to Groundwater:** N/E

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
280 / 0		SM	Silty SAND; reddish brown, damp, fine grained.				
	11/6 15/6 20/6	SC	Clayey SAND; dense, brown, moist, fine to medium grained.	35	11.7		
	14/6 17/6 17/6	SM	Silty SAND; reddish brown, damp, fine to coarse grained.	34	9.5		
275 / 5							
	3/6 3/6 6/6	ML	Sandy SILT; stiff, brown, moist, fine grained sand.	9	20.7		SAND=44% -#200=56%
270 / 10							
	5/6 8/6 10/6	SM	Silty SAND; medium dense, light brown, moist, fine to coarse grained, with trace of clay.	18	12.6		SAND=83% -#200=17%
			End of boring at 13.5 feet BSG				
265 / 15							
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-11

SALEM engineering group, inc.

Test Boring: B-9/P-5 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: 12/23/2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** RS

Drill Type: CME 55 **Elevation:** 280ft. AMSL

Auger Type: 6-5/8in. Hollow Stem Auger **Initial Depth to Groundwater:** N/E

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
280 / 0		SM	Silty SAND; reddish brown, damp, fine grained.				
	1/6 2/6 2/6	SC	Clayey SAND; loose, reddish brown, moist, fine to medium grained. Hardpan at 4 ft.	4	12.5		
275 / 5	12/6 14/6 20/6	ML	SILT; hard, light brown, moist, non-plastic.	34	19.6		
	4/6 6/6 10/6		Grades as above.	16	15.6		
270 / 10	8/6 9/6 10/6	SP-SM	Poorly Graded SAND with Silt; medium dense, brown, moist, fine to coarse grained.	19	10.2		SAND=89% -#200=11% PI = non-plastic
			End of boring at 11.5 feet BSG				
265 / 15							
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-12

SALEM engineering group, inc.

Test Boring: B-10/P-6 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: 12/23/2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** RS

Drill Type: CME 55 **Elevation:** 280ft. AMSL

Auger Type: 6-5/8in. Hollow Stem Auger **Initial Depth to Groundwater:** N/E

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
280 / 0	1/6 2/6 4/6	CL	Sandy Lean CLAY; mdium stiff, reddish brown, moist, trace clay.	6	18.3		
275 / 5	11/6 16/6 23/6	CL-ML	Silty CLAY; hard, brown, damp, weakly cemented.	39	18.2		
	4/6 6/6 9/6	CL	Sandy Lean CLAY (predominant); very stiff, fine grained sand, low to medium plasticity, weakly cemented, interbedded with fine silty sand (subordinate).	17	28.3		SAND=31% -#200=69% LL-31 PI=10
270 / 10			End of boring at 10 feet BSG.				
265 / 15							
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-13

SALEM engineering group, inc.

Test Boring: B-11/P-7 **Page** 1 **Of:** 1

Project Number: 1-224-1068

Date: 12/23/2024

Client: Madera Unified School District

Project: Athletic Facility Modernization, Desmond Middle School

Location: 26490 Martin Street, Madera, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** RS

Drill Type: CME 55 **Elevation:** 280ft. AMSL

Auger Type: 6-5/8in. Hollow Stem Auger **Initial Depth to Groundwater:** N/E

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
280 / 0		SM	Silty SAND; reddish brown, damp, fine grained.				
	2/6 2/6 3/6	CL	Lean CLAY; medium stiff, moist, medium plasticity, with trace sand.	5	11.7		
275 / 5	19/6 29/6 27/6	SM	Silty SAND; very dense, reddish brown, damp, fine to coarse grained, weakly cemented.	56	10.4		SAND=67% -#200=33%
270 / 10	3/6 7/6 7/6	ML	SILT with sand; stiff, light brown, moist, trace clay.	14	24.1		SAND=27% -#200=73%
265 / 15	5/6 8/6 9/6	CL	Lean CLAY with sand; very stiff, moist, medium plasticity, very weakly cemented.	17	26.0		SAND=24% -#200=74% PI=14 LL = 38
			End of boring at 15 feet BSG.				
260 / 20							
255 / 25							

Notes: *Grass at surface.*

Figure Number A-14

KEY TO SYMBOLS

Symbol Description

Strata symbols

- Silt
- Lean Clay
- Poorly graded sand with silt
- Silty Sand
- Clayey Sand
- Silty low plasticity clay

Misc. Symbols

- Boring continues
- Drill rejection

Soil Samplers

- California sampler
- Standard penetration test
- Bulk/Grab sample

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

Percolation Test Worksheet

Project:	Percolation Testing Desmond Middle School	Job No.:	1-224-1068-A
		Date Drilled:	12/23/2024
		Soil Classification:	Silty SAND (fine sand)
Test Hole No.:	P-3		
Tested By:	CR	Presoaking Date:	12/23/2024
Drilled Hole Depth:	8.7 Feet	Test Date:	12/26/2024

Length of Pipe	104	in.
Pipe stickup:	0.2	ft
Hole Dia.:	6.625	in.
Pipe Dia.:	3	in.
Gravel Below Pipe:	2.0	in.
Gravel pack porosity:	0.40	
Gravel Correc Factor:	0.52	

Time Start (hr:min:sec)	Time Finish (hr:min:sec)	Refill- Yes or No	Elapsed Time (hrs:min:sec)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Uncorrected Percolation Rate (min/in)	Gravel Pack Corrected Unfactored Percolation Rate (min/in)	Estimated Unfactored Infiltration Rate (inches/hr)
9:36:00	10:06:00	Y	0:30:00	7.34	7.41	0.84	30.00	35.71	68.28	0.08
10:06:00	10:36:00	N	0:30:00	7.41	7.52	1.32	30.00	22.73	43.45	0.13
10:36:00	11:06:00	N	0:30:00	7.52	7.60	0.96	30.00	31.25	59.75	0.10
11:06:00	11:36:00	N	0:30:00	7.60	7.68	0.96	30.00	31.25	59.75	0.10
11:36:00	12:06:00	N	0:30:00	7.68	7.75	0.84	30.00	35.71	68.28	0.10
12:06:00	12:36:00	N	0:30:00	7.75	7.82	0.84	30.00	35.71	68.28	0.10
12:38:00	13:08:00	Y	0:30:00	7.28	7.36	0.96	30.00	31.25	59.75	0.08
13:08:00	13:38:00	N	0:30:00	7.36	7.44	0.96	30.00	31.25	59.75	0.09
13:38:00	14:08:00	N	0:30:00	7.44	7.52	0.96	30.00	31.25	59.75	0.09
14:08:00	14:38:00	N	0:30:00	7.52	7.59	0.84	30.00	35.71	68.28	0.09
14:38:00	15:08:00	N	0:30:00	7.59	7.66	0.84	30.00	35.71	68.28	0.09
15:08:00	15:38:00	N	0:30:00	7.66	7.73	0.84	30.00	35.71	68.28	0.10

Estimated Unfactored Infiltration Rate (in/hr)	0.09
Corrected Unfactored Percolation Rate (min/in)	68.0

SALEM engineering group, inc.

Percolation Test Worksheet

Project:	**Percolation Testing Desmond Middle School**	**Job No.:**	**1-224-1068-A**	Length of Pipe	165.75 in.
		Date Drilled:	**12/23/2024**	Pipe stickup:	0.6 ft
		Soil Classification:	**Silty Sand, fine to coarse grained, with trace clay**	Hole Dia.:	6.625 in.
Test Hole No.:	**B-8/P-4**			Pipe Dia.:	3 in.
Tested By:	**CR**			Gravel Below Pipe:	5.3 in.
Drilled Hole Depth:	**13.7 Feet**	**Presoaking Date:**	**12/23/2024**	Gravel pack porosity:	0.40
		Test Date:	**12/26/2024**	Gravel Correc Factor:	0.52

Time Start (hr:min:sec)	Time Finish (hr:min:sec)	Refill-Yes or No	Elapsed Time (hrs:min:sec)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Uncorrected Percolation Rate (min/in)	Gravel Pack Corrected Unfactored Percolation Rate (min/in)	Estimated Unfactored Infiltration Rate (inches/hr)
9:40:00	10:10:00	Y	0:30:00	12.69	12.94	3.00	30.00	10.00	19.12	0.28
10:10:00	10:40:00	N	0:30:00	12.94	13.11	2.04	30.00	14.71	28.12	0.22
10:40:00	11:10:00	N	0:30:00	13.11	13.21	1.20	30.00	25.00	47.80	0.14
11:12:00	11:42:00	Y	0:30:00	12.62	12.87	3.00	30.00	10.00	19.12	0.26
11:42:00	12:17:00	N	0:35:00	12.87	13.06	2.28	35.00	15.35	29.35	0.20
12:17:00	12:46:00	N	0:29:00	13.06	13.17	1.32	29.00	21.97	42.00	0.15
12:47:30	13:17:30	Y	0:30:00	12.71	12.85	1.68	30.00	17.86	34.14	0.15
13:17:30	13:47:00	N	0:29:30	12.85	13.01	1.92	29.50	15.36	29.38	0.19
13:47:00	14:16:00	N	0:29:00	13.01	13.12	1.32	29.00	21.97	42.00	0.15
14:16:00	14:46:00	N	0:30:00	13.12	13.22	1.20	30.00	25.00	47.80	0.14
14:47:00	15:17:00	Y	0:30:00	12.60	12.82	2.64	30.00	11.36	21.73	0.23
15:17:00	15:37:00	N	0:20:00	12.82	12.93	1.32	20.00	15.15	28.97	0.19

Estimated Unfactored Infiltration Rate (in/hr)	0.19
Corrected Unfactored Percolation Rate (min/in)	29.0

Percolation Test Worksheet

Project:	**Percolation Testing Desmond Middle School**	**Job No.:**	**1-224-1068-A**
		Date Drilled:	**12/23/2024**
		Soil Classification:	**Poorly Graded Sand with Silt**
Test Hole No.:	**B-9/P-5**		
Tested By:	**CR**	**Presoaking Date:**	**12/23/2024**
Drilled Hole Depth:	**11.5** Feet	**Test Date:**	**12/26/2024**

Length of Pipe	140	in.
Pipe stickup:	0.4	ft
Hole Dia.:	6.625	in.
Pipe Dia.:	3	in.
Gravel Below Pipe:	2.5	in.
Gravel pack porosity:	0.40	
Gravel Correc Factor:	0.52	

Time Start (hr:min:sec)	Time Finish (hr:min:sec)	Refill-Yes or No	Elapsed Time (hrs:min:sec)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Uncorrected Percolation Rate (min/in)	Gravel Pack Corrected Unfactored Percolation Rate (min/in)	Estimated Unfactored Infiltration Rate (inches/hr)
9:43:00	10:13:00	Y	0:30:00	10.21	10.22	0.12	30.00	250.00	477.98	0.01
10:13:00	10:43:00	N	0:30:00	10.22	10.28	0.72	30.00	41.67	79.66	0.06
10:43:00	11:14:00	N	0:31:00	10.28	10.32	0.48	31.00	64.58	123.48	0.04
11:14:00	11:44:00	N	0:30:00	10.32	10.37	0.60	30.00	50.00	95.60	0.05
11:44:00	12:18:00	N	0:34:00	10.37	10.41	0.48	34.00	70.83	135.43	0.04
12:18:00	12:48:00	N	0:30:00	10.41	10.45	0.48	30.00	62.50	119.50	0.04
12:48:00	13:18:00	N	0:30:00	10.45	10.49	0.48	30.00	62.50	119.50	0.04
13:18:00	13:48:00	N	0:30:00	10.49	10.53	0.48	30.00	62.50	119.50	0.05
13:48:00	14:18:00	N	0:30:00	10.53	10.57	0.48	30.00	62.50	119.50	0.05
14:18:00	14:48:00	N	0:30:00	10.57	10.61	0.48	30.00	62.50	119.50	0.05

Estimated Unfactored Infiltration Rate (in/hr)	0.05
Corrected Unfactored Percolation Rate (min/in)	119.5

SALEM engineering group, inc.

Percolation Test Worksheet

Project:	Percolation Testing Desmond Middle School	Job No.:	1-224-1068-A	Length of Pipe	123 in.
		Date Drilled:	12/23/2024	Pipe stickup:	0.3 ft
		Soil Classification:	Sandy Lean CLAY, weakly cemented, interbedded w/fine silty sand	Hole Dia.:	6.625 in.
Test Hole No.:	B-10/P-6			Pipe Dia.:	3 in.
Tested By:	CR	Presoaking Date:	12/23/2024	Gravel Below Pipe:	3.0 in.
Drilled Hole Depth:	10.2 Feet	Test Date:	12/26/2024	Gravel pack porosity:	0.40
				Gravel Correc Factor:	0.52

Time Start (hr:min:sec)	Time Finish (hr:min:sec)	Refill-Yes or No	Elapsed Time (hrs:min:sec)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Uncorrected Percolation Rate (min/in)	Gravel Pack Corrected Unfactored Percolation Rate (min/in)	Estimated Unfactored Infiltration Rate (inches/hr)
9:46:00	10:16:00	Y	0:30:00	9.06	9.06	0.00	30.00	Negligible	Negligible	0.000
10:16:00	11:16:00	N	1:00:00	9.06	9.12	0.72	60.00	83.33	159.33	0.034
11:16:00	12:19:00	N	1:03:00	9.12	9.17	0.60	63.00	105.00	200.75	0.028
12:19:00	13:19:00	N	1:00:00	9.17	9.21	0.48	60.00	125.00	238.99	0.024
13:19:00	14:19:00	N	1:00:00	9.21	9.24	0.36	60.00	166.67	318.65	0.018
14:19:00	15:19:00	N	1:00:00	9.24	9.27	0.36	60.00	166.67	318.65	0.019

Estimated Unfactored Infiltration Rate (in/hr)	0.02
Corrected Unfactored Percolation Rate (min/in)	318.0

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Percolation Test Worksheet

Project:	**Percolation Testing Desmond Middle School**	**Job No.:**	**1-224-1068-A**	Length of Pipe	180 in.
		Date Drilled:	**12/23/2024**	Pipe stickup:	0.2 ft
		Soil Classification:	**Lean CLAY with Sand very weakly cemented**	Hole Dia.:	6.625 in.
Test Hole No.:	**B-11/P-7**			Pipe Dia.:	3 in.
Tested By:	**CR**	**Presoaking Date:**	**12/23/2024**	Gravel Below Pipe:	2.5 in.
Drilled Hole Depth:	**15.0** Feet	**Test Date:**	**12/26/2024**	Gravel pack porosity:	0.40
				Gravel Correc Factor:	0.52

Time Start (hr:min:sec)	Time Finish (hr:min:sec)	Refill-Yes or No	Elapsed Time (hrs:min:sec)	Initial Water Level[#] (ft)	Final Water Level[#] (ft)	Δ Water Level (in.)	Δ Min.	Uncorrected Percolation Rate (min/in)	Gravel Pack Corrected Unfactored Percolation Rate (min/in)	Estimated Unfactored Infiltration Rate (inches/hr)
9:50:00	10:20:00	Y	0:30:00	13.55	13.55	0.00	30.00	Negligible	Negligible	0.0000
10:20:00	11:20:00	N	1:00:00	13.55	13.55	0.00	60.00	Negligible	Negligible	0.0000
11:20:00	12:20:00	N	1:00:00	13.55	13.56	0.12	60.00	500.00	955.96	0.0048
12:20:00	13:20:00	N	1:00:00	13.56	13.57	0.12	60.00	500.00	955.96	0.0049
13:20:00	14:20:00	N	1:00:00	13.57	13.58	0.12	60.00	500.00	955.96	0.0049
14:20:00	15:20:00	N	1:00:00	13.58	13.59	0.12	60.00	500.00	955.96	0.0049

Estimated Unfactored Infiltration Rate (in/hr)	Negligible
Corrected Unfactored Percolation Rate (min/in)	Negligible

SALEM engineering group, inc.

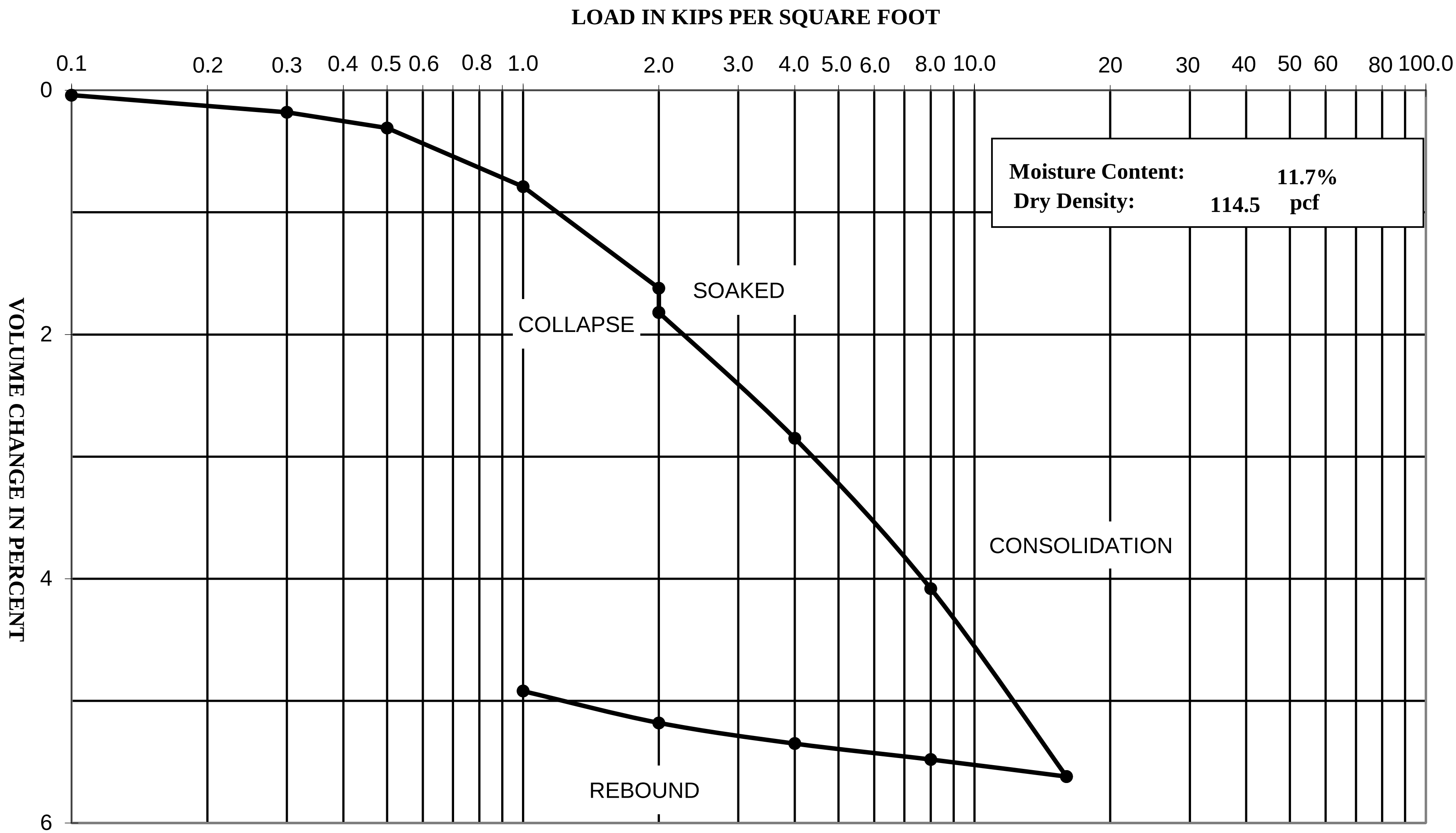
APPENDIX

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, grain size distribution, Atterberg Limits, consolidation, shear strength, expansion index, R-value, corrosivity, and soil resistivity. The results of the laboratory tests are summarized in the following figures.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Desmond Middle School - Madera, CA

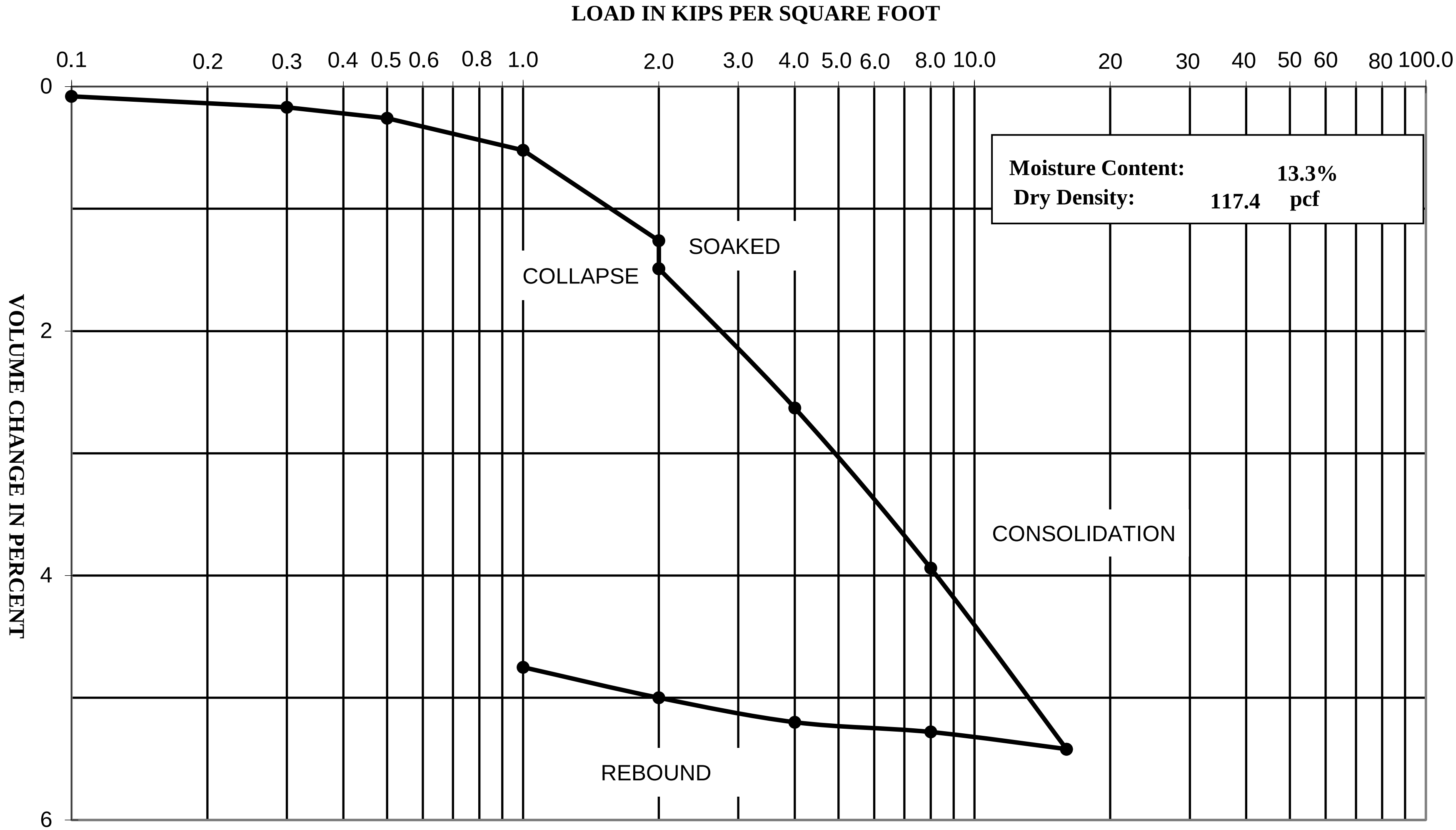
Project Number: 1-224-1068-A

Boring: B-3 @ 1.5'

SALEM engineering group, inc.

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435



Project Name: Desmond Middle School - Madera, CA

Project Number: 1-224-1068-A

Boring: B-4 @ 3.5'

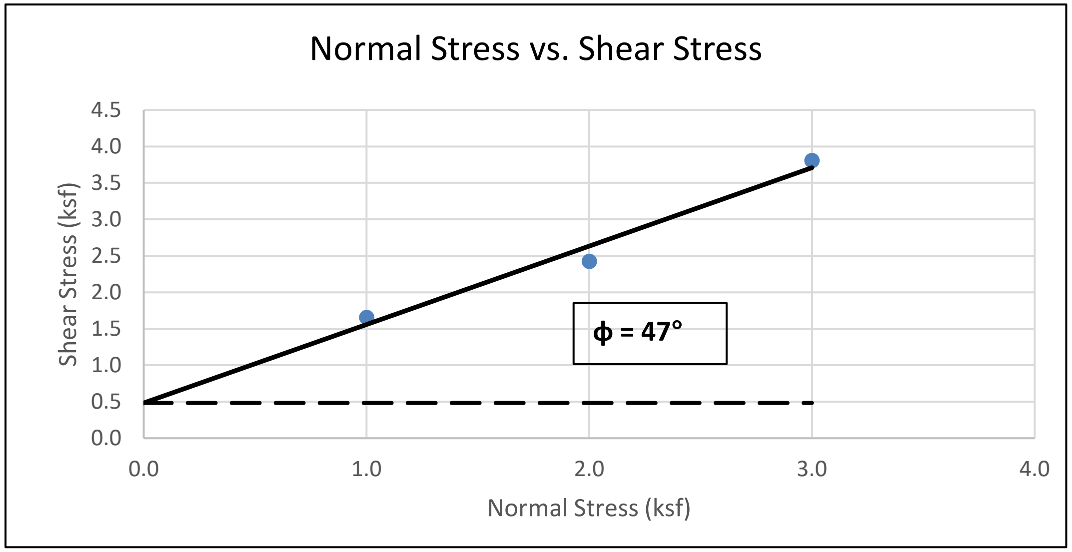
SALEM engineering group, inc.

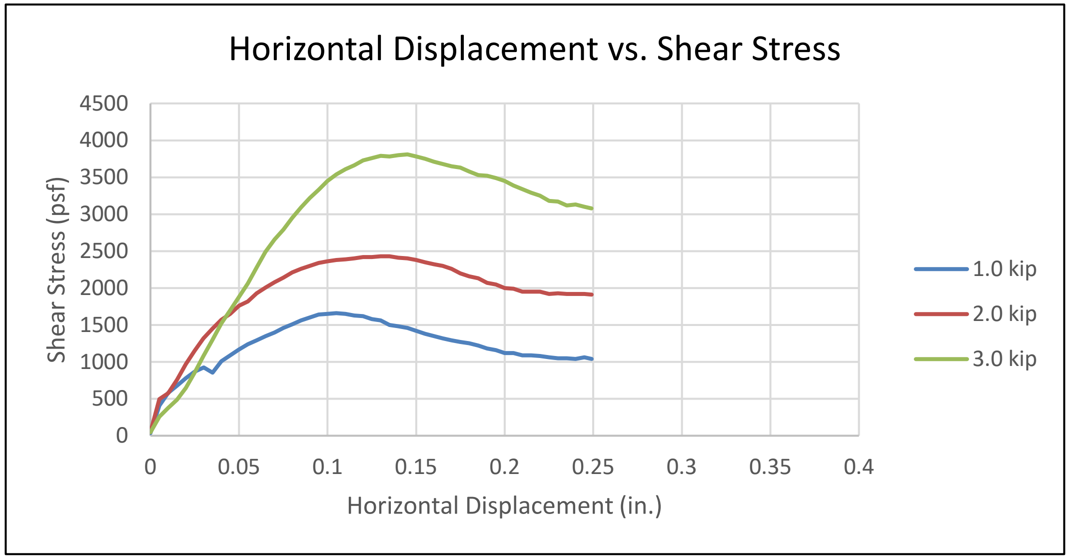
Direct Shear Test (ASTM D3080)

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Client:
Boring: B-1 @ 8.5'
Soil Type: Poorly Graded SAND wit
Sample Type: Undisturbed Ring
Tested By: MC
Reviewed By:
Date of Test: 11/26/24
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	1.66	2.43	3.81

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.975	0.949	0.961
Post-Shear Sample Height (in.)	1.003	0.968	0.974
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	11.3		
Dry Density (pcf)	119.0	119.9	114.9
Saturation %	74.0	75.9	65.9
Void Ratio	0.41	0.40	0.46
Consolidated Void Ratio	0.38	0.33	0.40
Final (post-shear) Values			
Final Moisture Content (%)	18.6	18.3	19.7
Dry Density (pcf)	115.3	119.1	113.1
Saturation %	98.8	111.5	100.1
Void Ratio	0.51	0.44	0.53





Peak Shear Strength Values	
Slope	1.08
Friction Angle	47
Cohesion (psf)	483

SALEM engineering group, inc.
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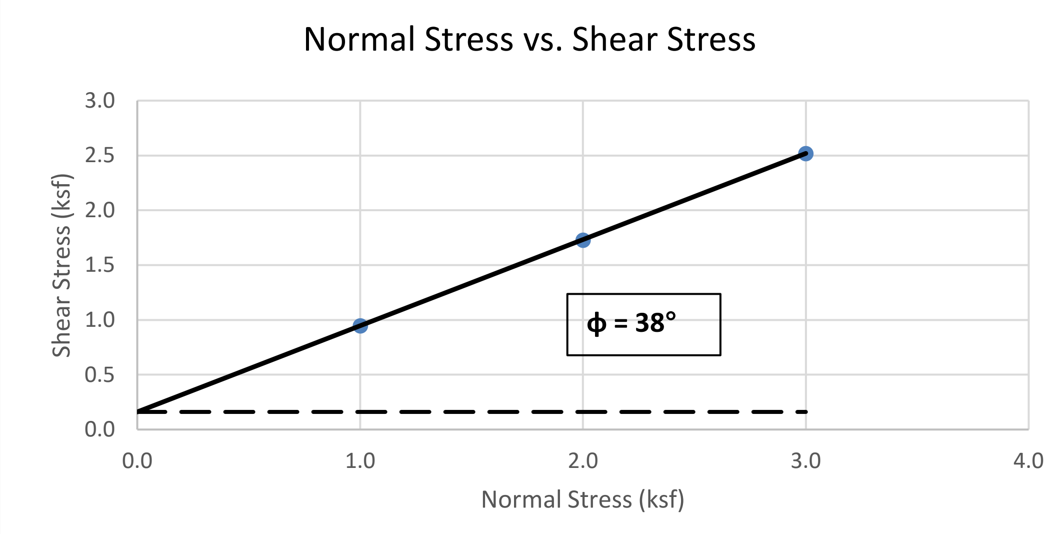
Fresno, CA 93722

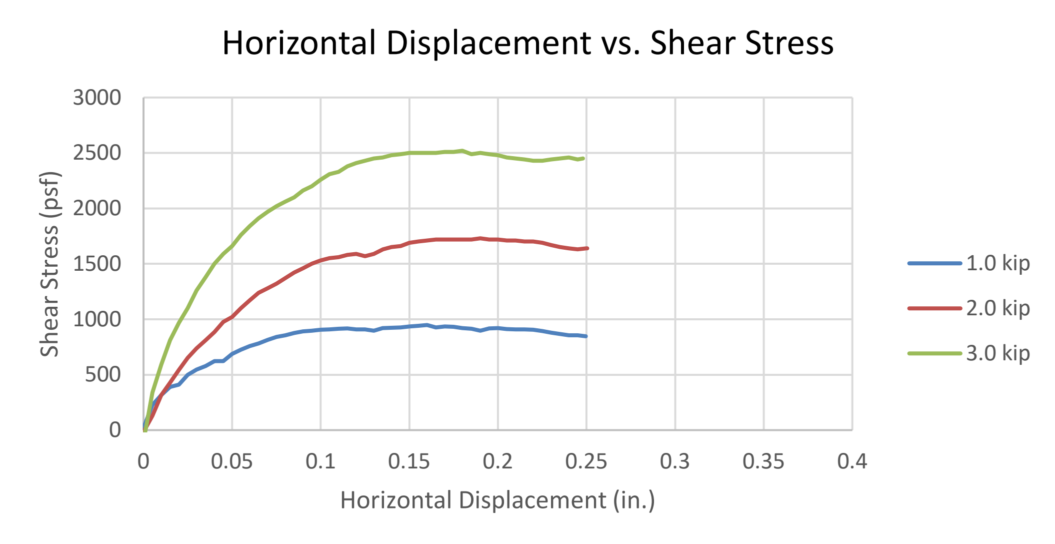
Direct Shear Test (ASTM D3080)

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Client:
Boring: B-2 @ 15'
Soil Type: SILT with Sand (ML)
Sample Type: Undisturbed Ring
Tested By: MC / NL
Reviewed By:
Date of Test: 12/2/24
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	0.95	1.73	2.52

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.903	0.870	0.809
Post-Shear Sample Height (in.)	0.892	0.851	0.788
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	11.3		
Dry Density (pcf)	111.7	114.8	113.2
Saturation %	60.5	65.6	62.9
Void Ratio	0.50	0.46	0.48
Consolidated Void Ratio	0.36	0.27	0.20
Final (post-shear) Values			
Final Moisture Content (%)	24.6	24.4	21.5
Dry Density (pcf)	112.6	119.2	129.6
Saturation %	132.1	167.7	209.5
Void Ratio	0.50	0.39	0.28





Peak Shear Strength Values	
Slope	0.79
Friction Angle	38
Cohesion (psf)	161

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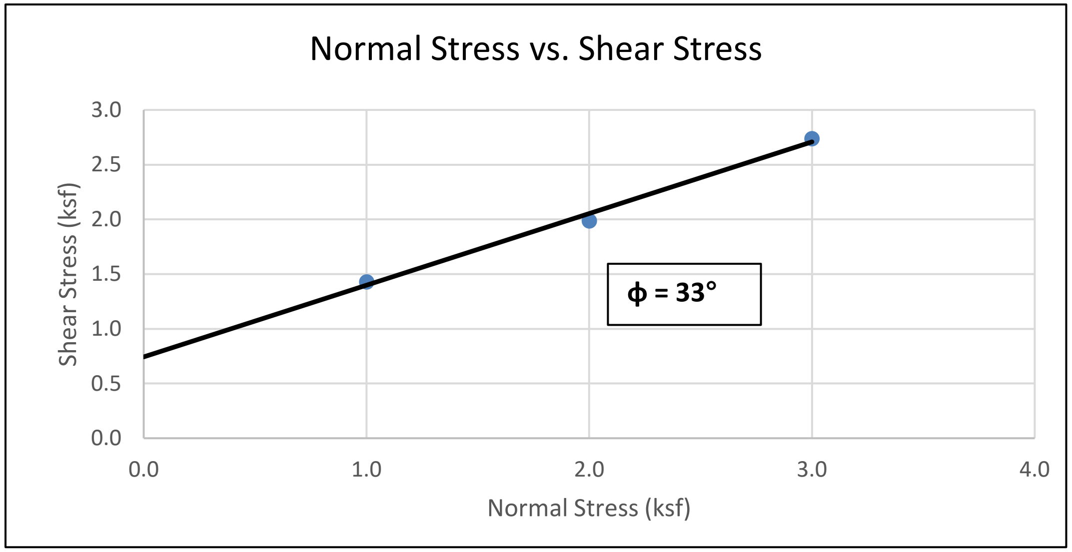
Fresno, CA 93722

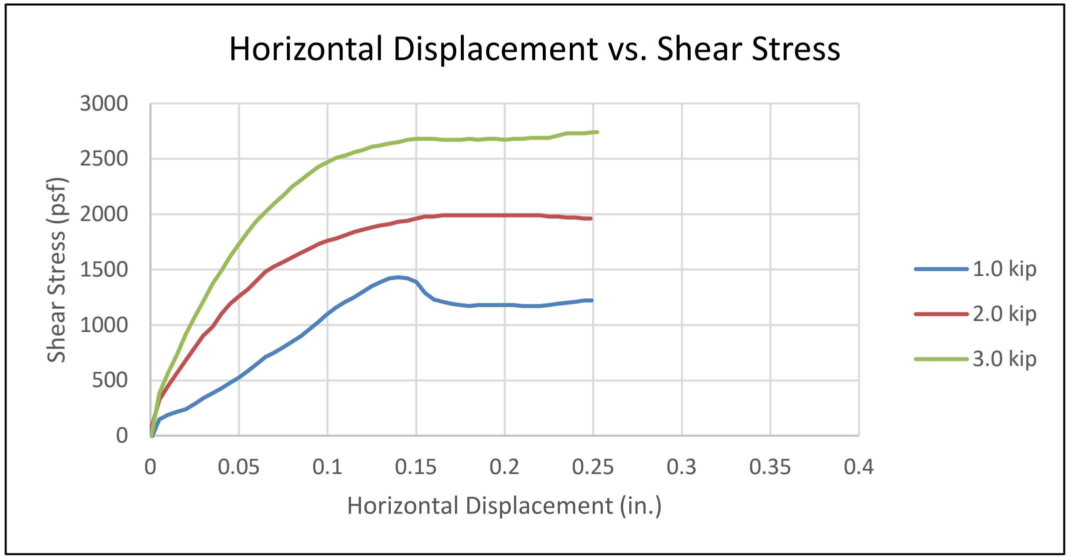
Direct Shear Test (ASTM D3080)

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Client:
Boring: B-3 @ 1.5'
Silty SAND (SM)
Sample Type: Undisturbed Ring
Tested By: NL / MC
Reviewed By:
Date of Test: 12/2/24 & 12/3/24
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	1.43	1.99	2.74

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.967	0.922	0.884
Post-Shear Sample Height (in.)	0.958	0.902	0.863
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	11.7		
Dry Density (pcf)	113.6	114.9	118.9
Saturation %	65.5	68.0	75.9
Void Ratio	0.48	0.46	0.41
Consolidated Void Ratio	0.43	0.35	0.25
Final (post-shear) Values			
Final Moisture Content (%)	22.5	22.3	20.0
Dry Density (pcf)	106.8	113.9	121.7
Saturation %	109.2	135.4	173.0
Void Ratio	0.55	0.44	0.31





Peak Shear Strength Values	
Slope	0.66
Friction Angle	33
Cohesion (psf)	743

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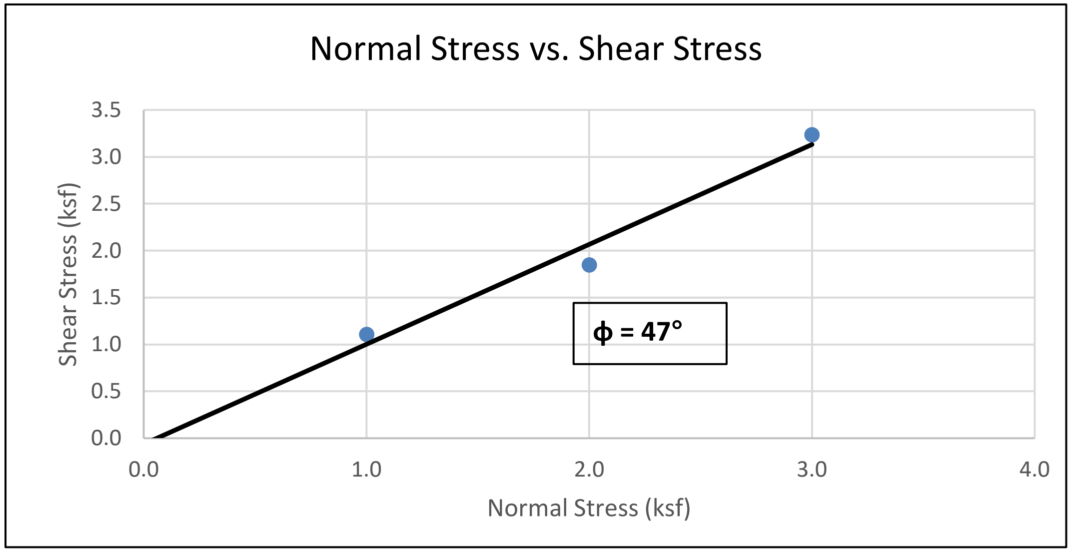
Fresno, CA 93722

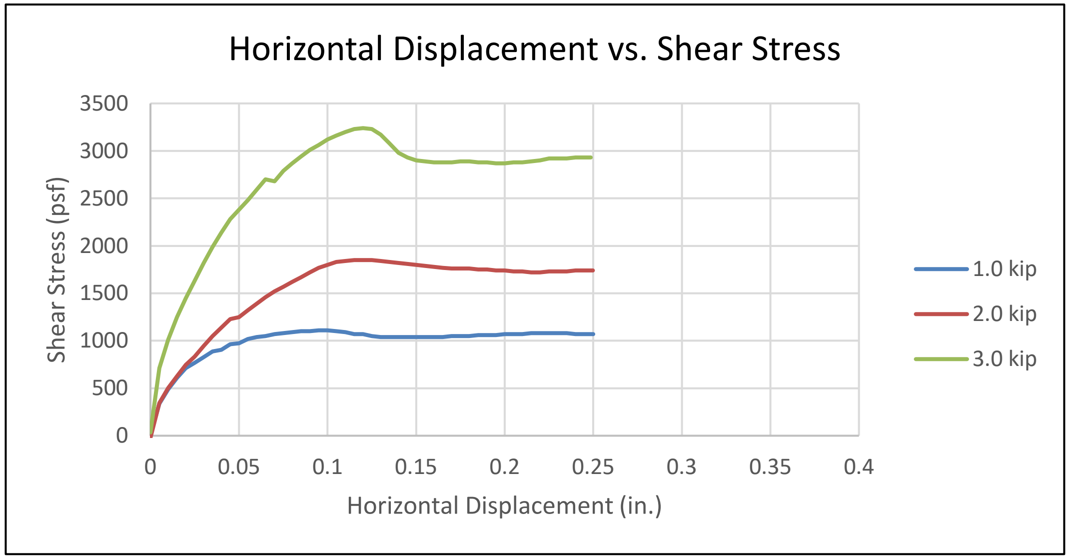
Direct Shear Test (ASTM D3080)

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Client:
Boring: B-4 @ 3.5'
Soil Type: Silty SAND (SM)
Sample Type: Undisturbed Ring
Tested By: MC / NL
Reviewed By:
Date of Test: 12/3/24 & 12/4/24
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	1.11	1.85	3.24

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.960	0.956	0.919
Post-Shear Sample Height (in.)	0.960	0.951	0.912
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	13.3		
Dry Density (pcf)	119.2	116.6	115.7
Saturation %	87.2	81.0	79.0
Void Ratio	0.41	0.44	0.45
Consolidated Void Ratio	0.35	0.38	0.33
Final (post-shear) Values			
Final Moisture Content (%)	20.8	20.8	18.8
Dry Density (pcf)	117.9	114.6	121.9
Saturation %	126.3	121.2	129.9
Void Ratio	0.44	0.46	0.39





Peak Shear Strength Values	
Slope	1.07
Friction Angle	47
Cohesion (psf)	-63

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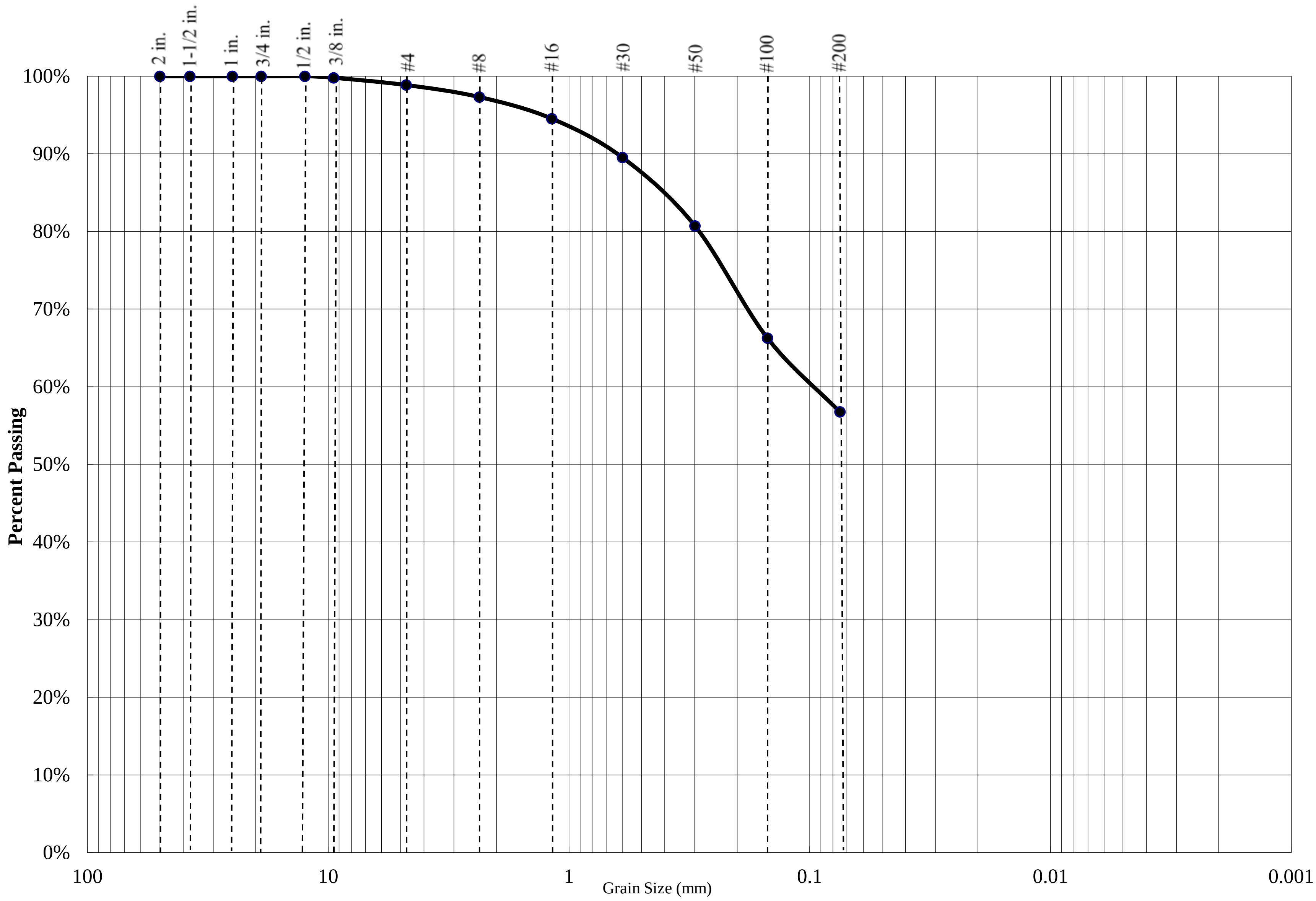
Fresno, CA 93722

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	42%	57%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	99.8%
#4	98.9%
#8	97.3%
#16	94.5%
#30	89.5%
#50	80.7%
#100	66.3%
#200	56.8%

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
Sandy SILT/Lean CLAY

Project Name: Desmond Middle School - Madera, CA

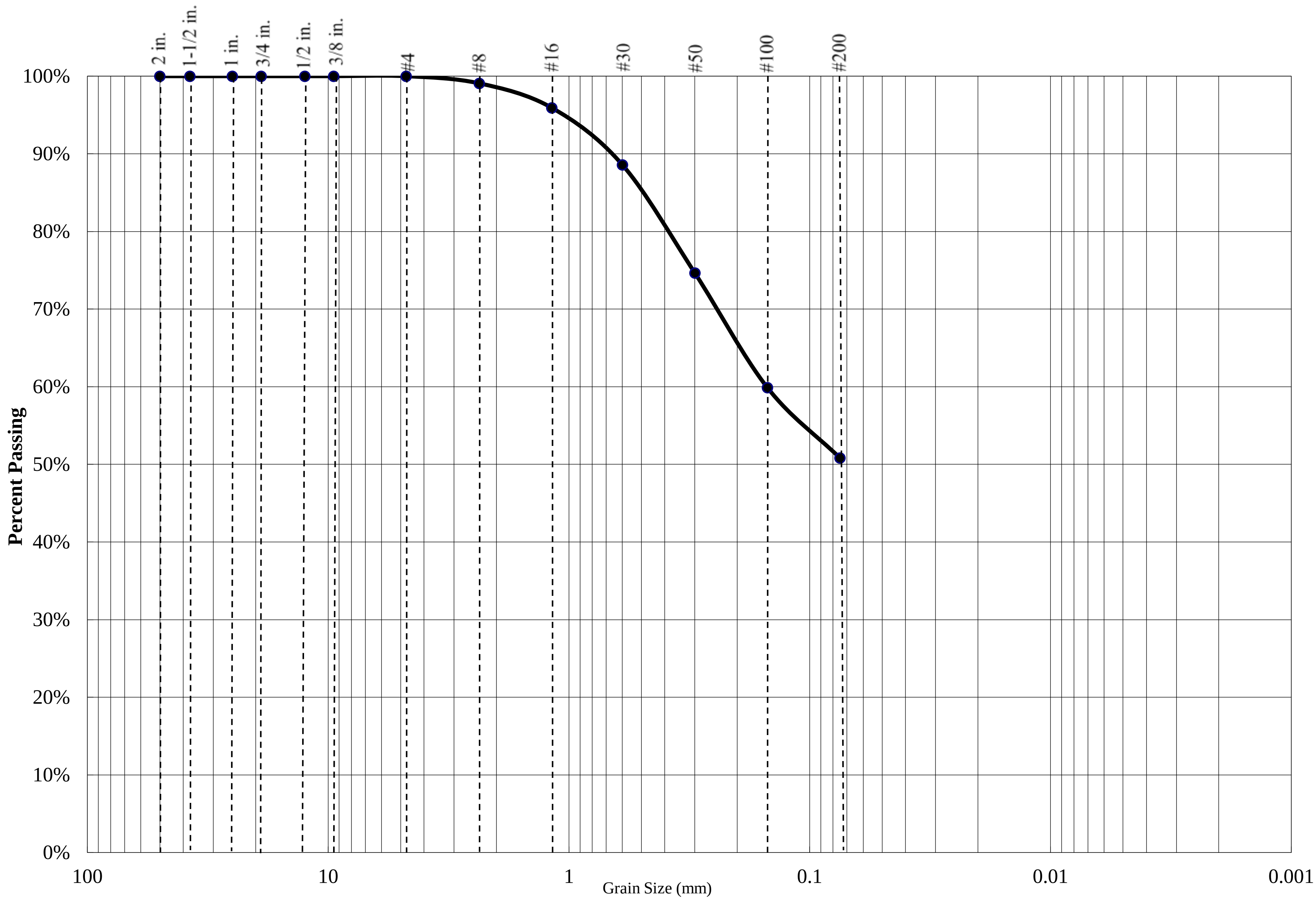
Project Number: 1-224-1068-A

Boring: B-1 @ 0 - 3'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	49%	51%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	99.1%
#16	95.9%
#30	88.6%
#50	74.6%
#100	59.9%
#200	50.8%

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
Sandy Lean CLAY (CL)

Project Name: Desmond Middle School - Madera, CA

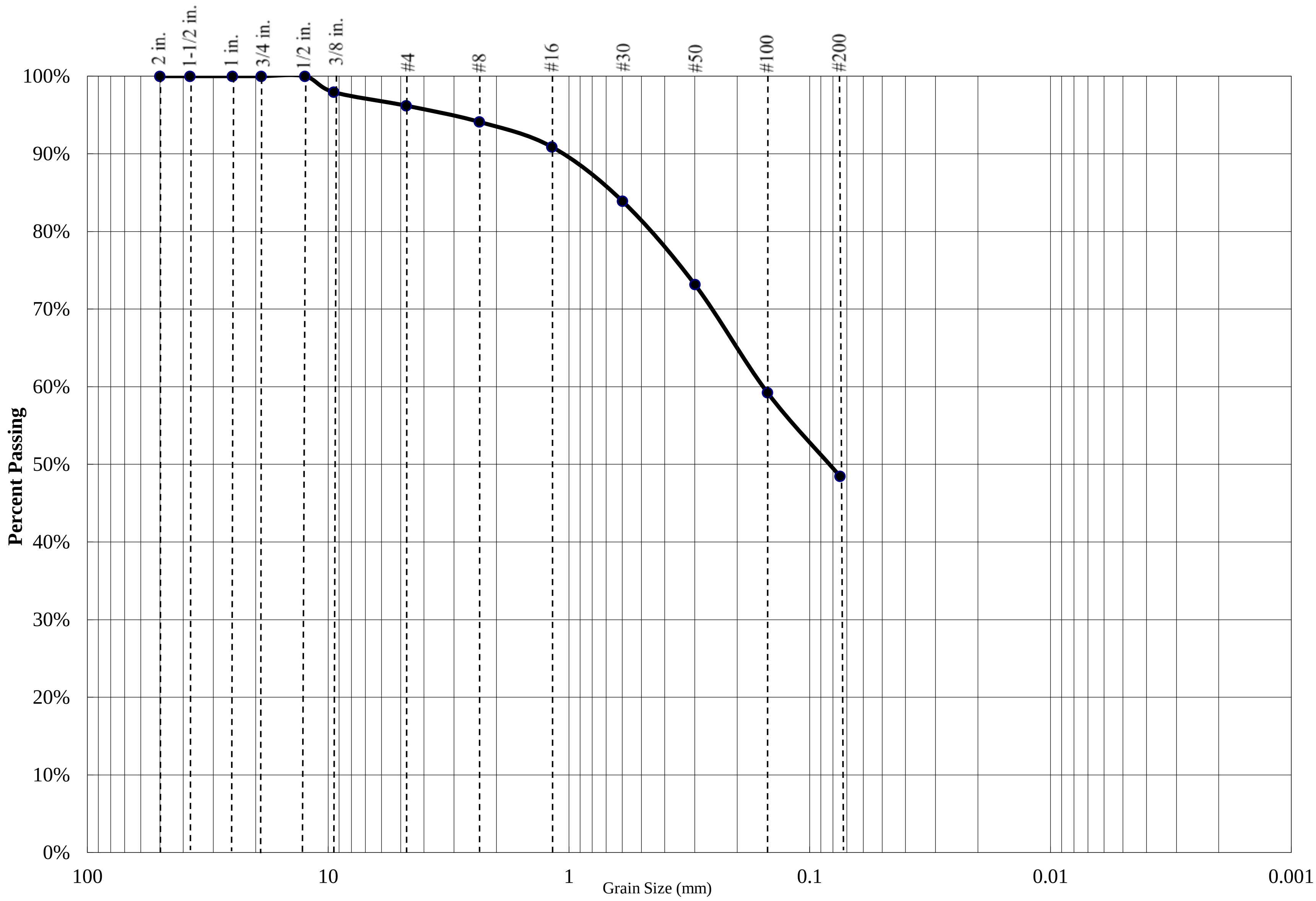
Project Number: 1-224-1068-A

Boring: B-1 @ 13.5'

SALEM
engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
4%	48%	48%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	97.9%
#4	96.2%
#8	94.1%
#16	90.9%
#30	83.9%
#50	73.2%
#100	59.3%
#200	48.5%

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 /Sandy Lean CLAY

Project Name: Desmond Middle School - Madera, CA

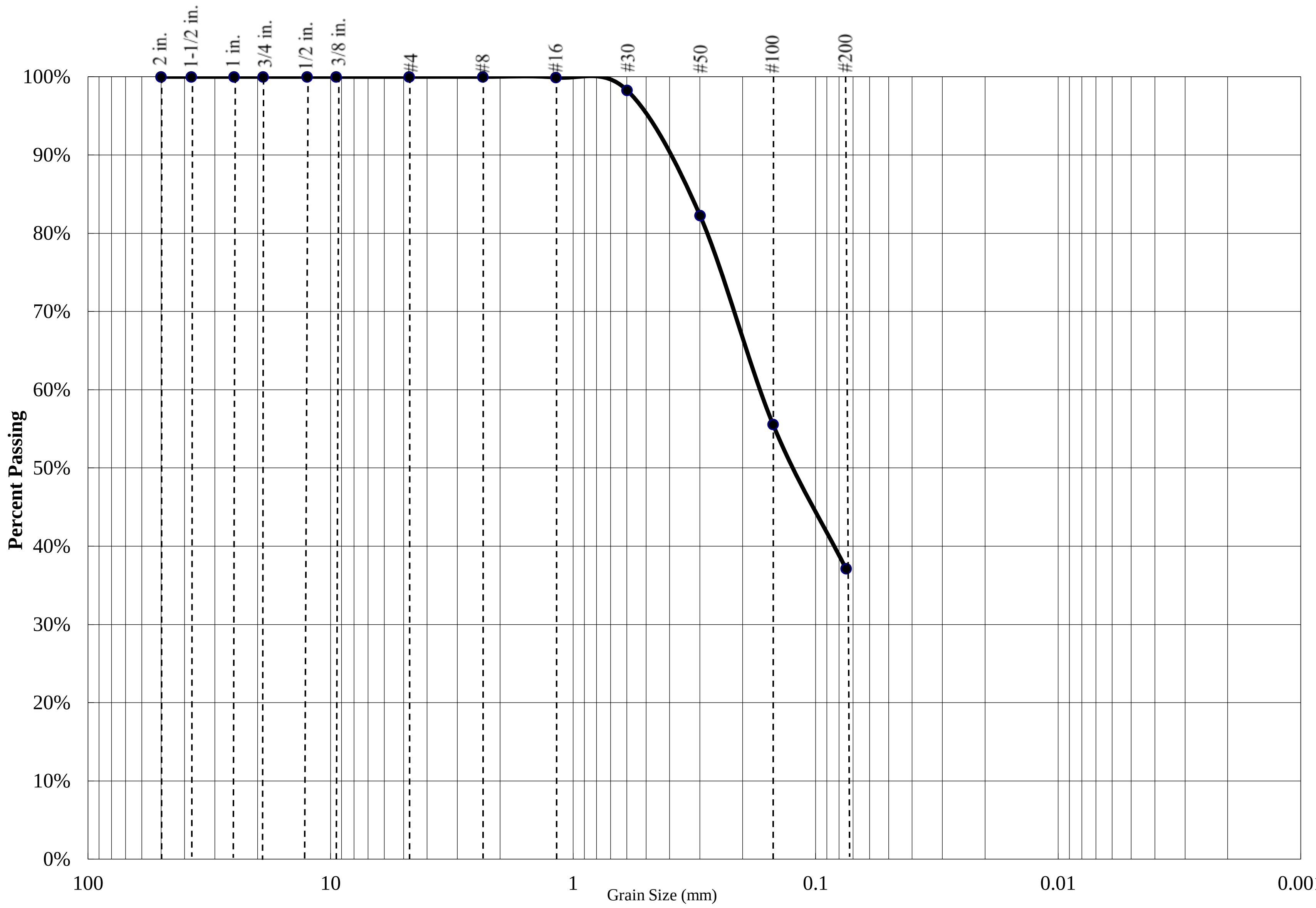
Project Number: 1-224-1068-A

Boring: B-3 @ 0 - 3'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	63%	37%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	100.0%
#16	99.9%
#30	98.3%
#50	82.3%
#100	55.6%
#200	37.1%

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: Modernizations for Desmond Middle School

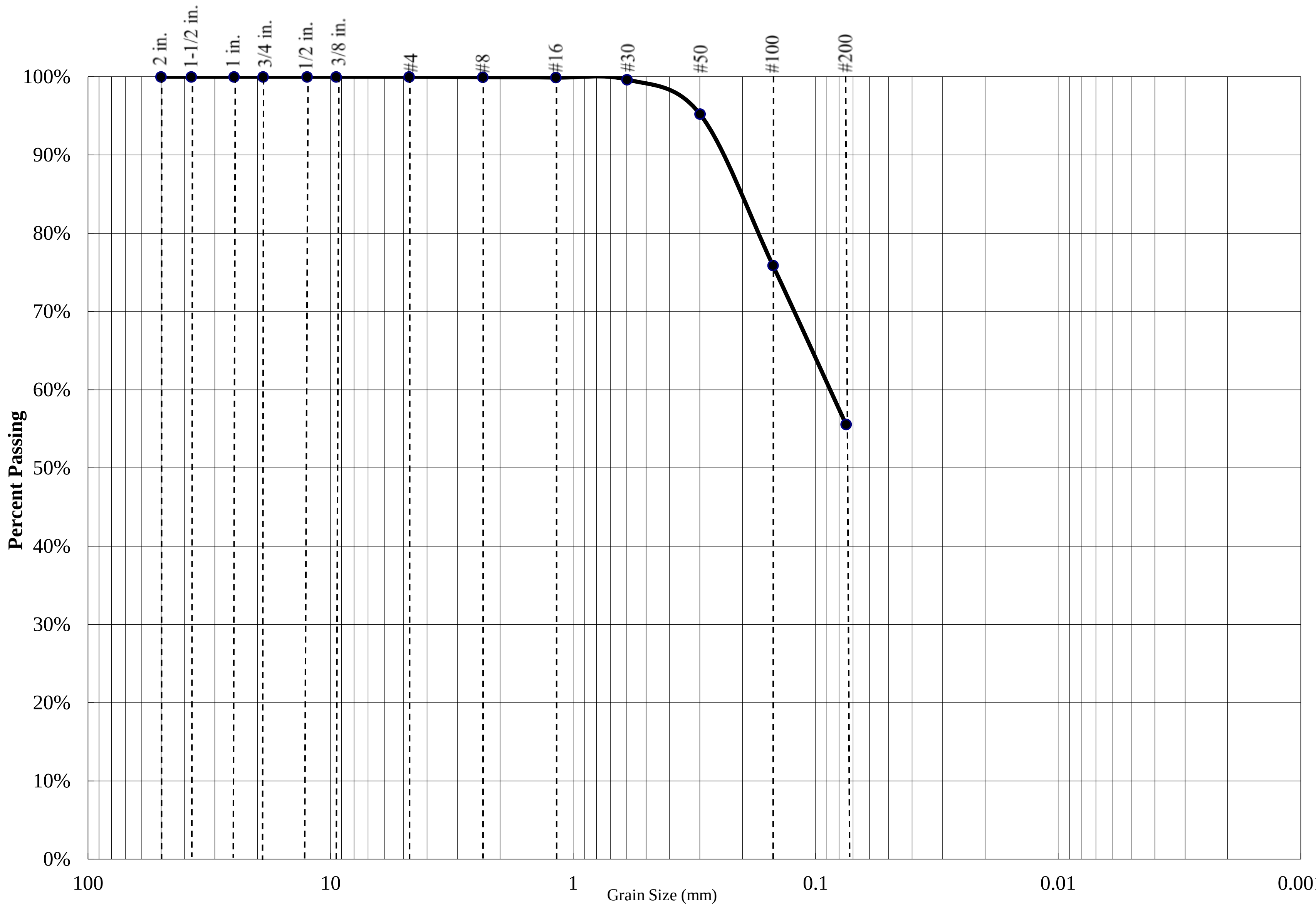
Project Number: 1-224-1068A

Boring: B-7 @ 7'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	44%	56%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	99.9%
#16	99.9%
#30	99.6%
#50	95.2%
#100	75.9%
#200	55.6%

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
Sandy Silt (ML)

Project Name: Modernizations for Desmond Middle School

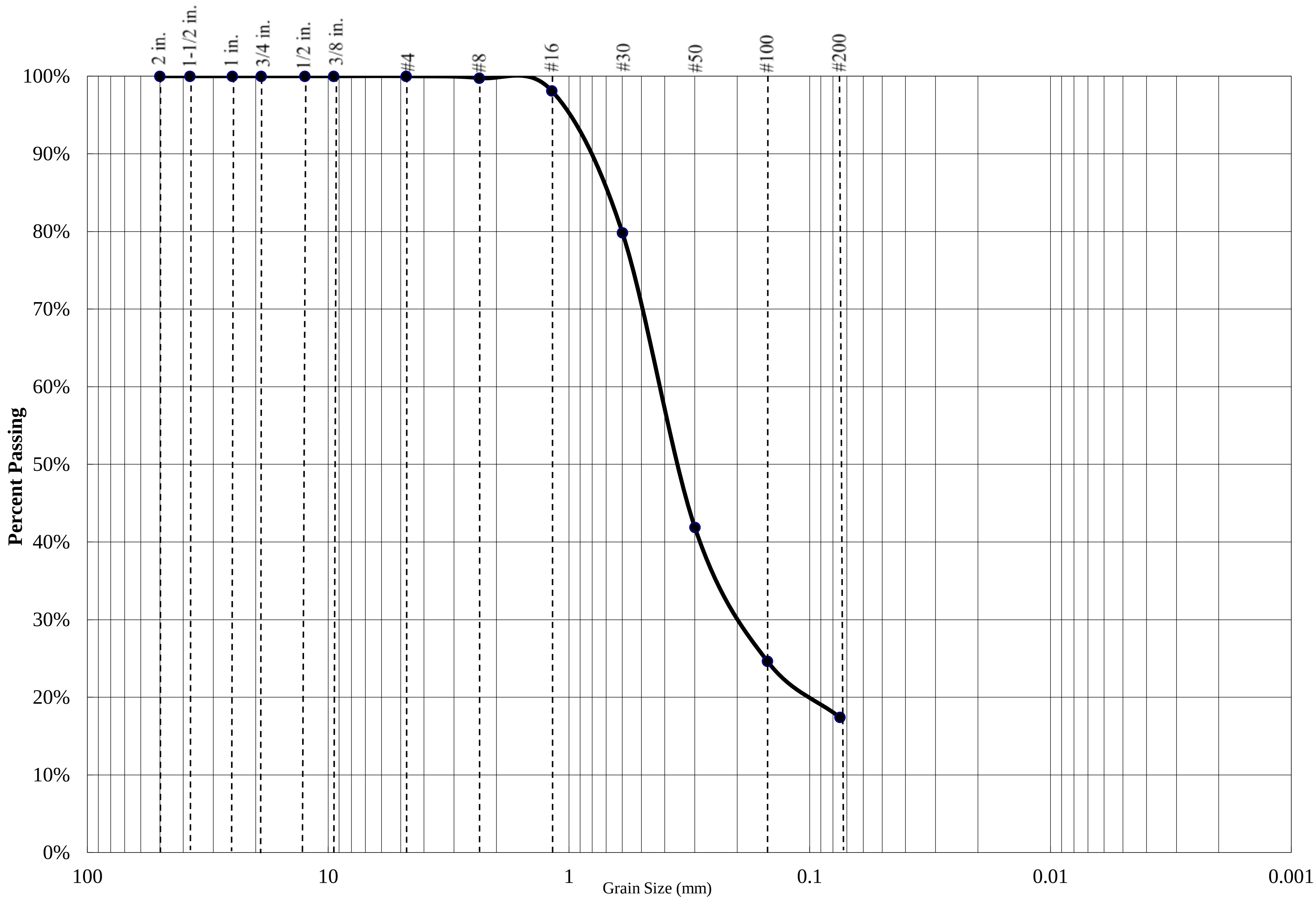
Project Number: 1-224-1068A

Boring: B-8 @ 8.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	83%	17%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	99.8%
#16	98.1%
#30	79.8%
#50	41.9%
#100	24.7%
#200	17.4%

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: Modernizations for Desmond Middle School

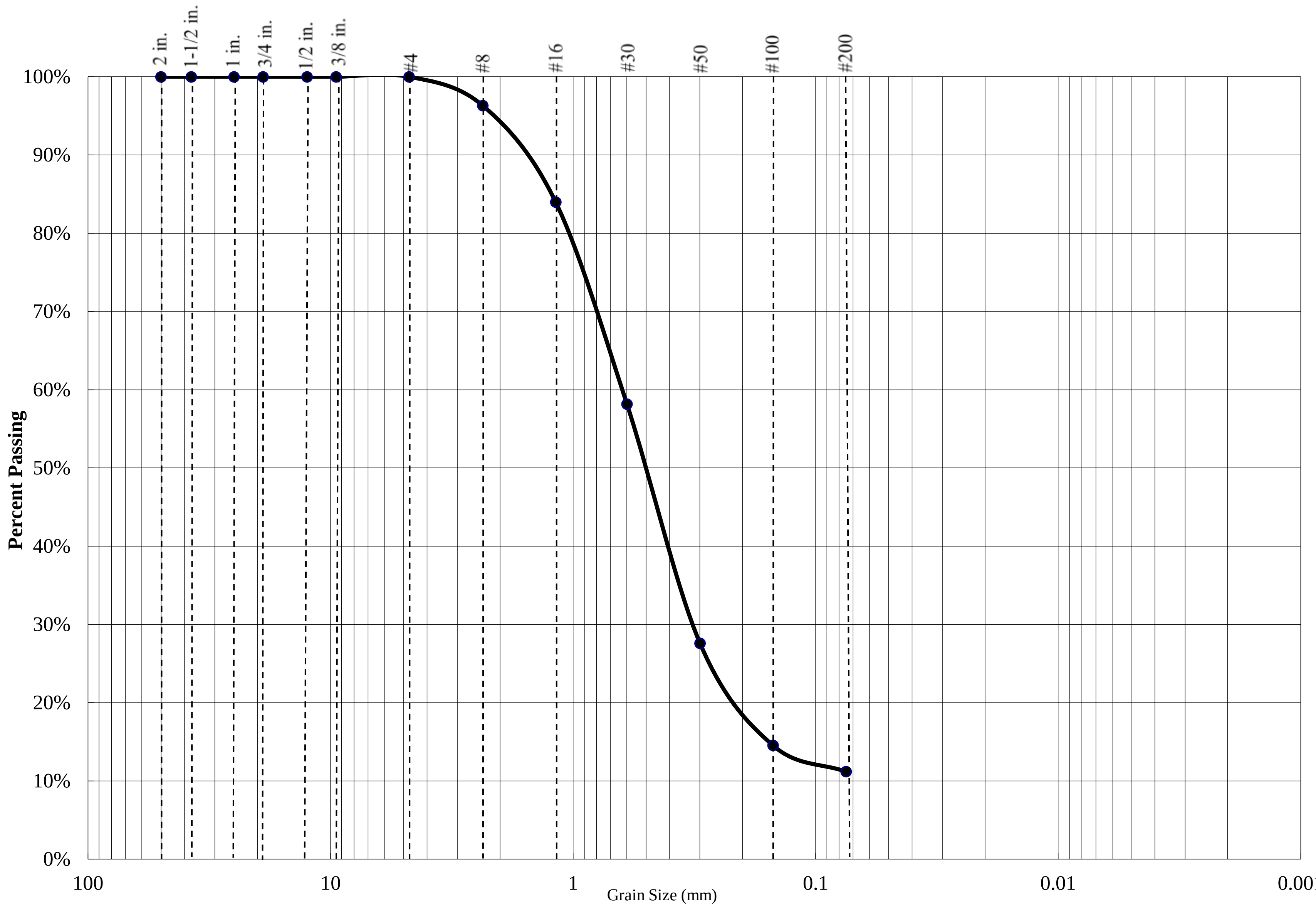
Project Number: 1-224-1068A

Boring: B-8 @ 12'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	89%	11%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	96.3%
#16	84.0%
#30	58.2%
#50	27.6%
#100	14.5%
#200	11.2%

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
Poorly Graded SAND with Silt

Project Name: Modernizations for Desmond Middle School

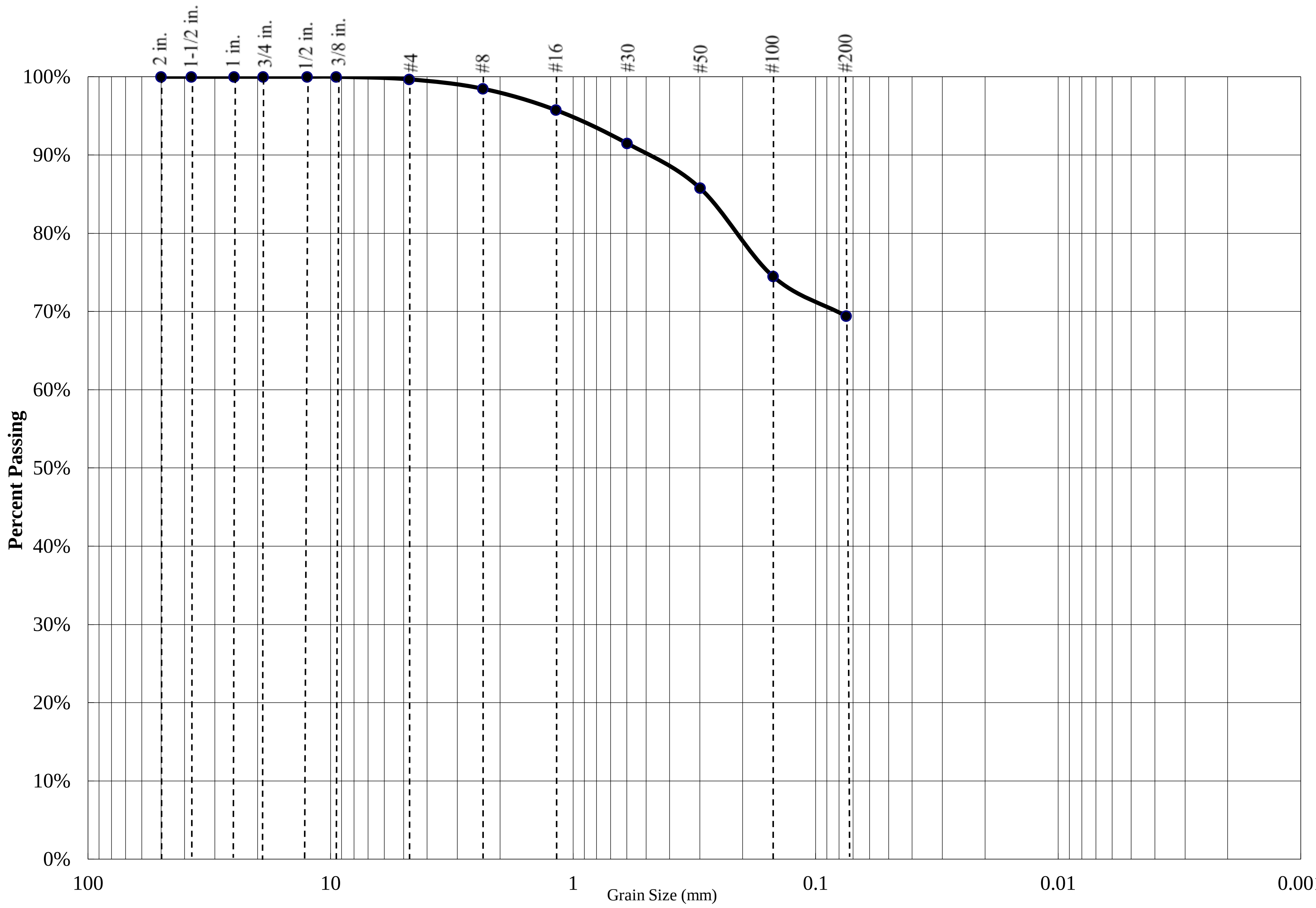
Project Number: 1-224-1068A

Boring: B-9 @ 10'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	31%	69%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.7%
#8	98.5%
#16	95.7%
#30	91.5%
#50	85.8%
#100	74.5%
#200	69.4%

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
Sandy Lean CLAY (CL)

Project Name: Modernizations for Desmond Middle School

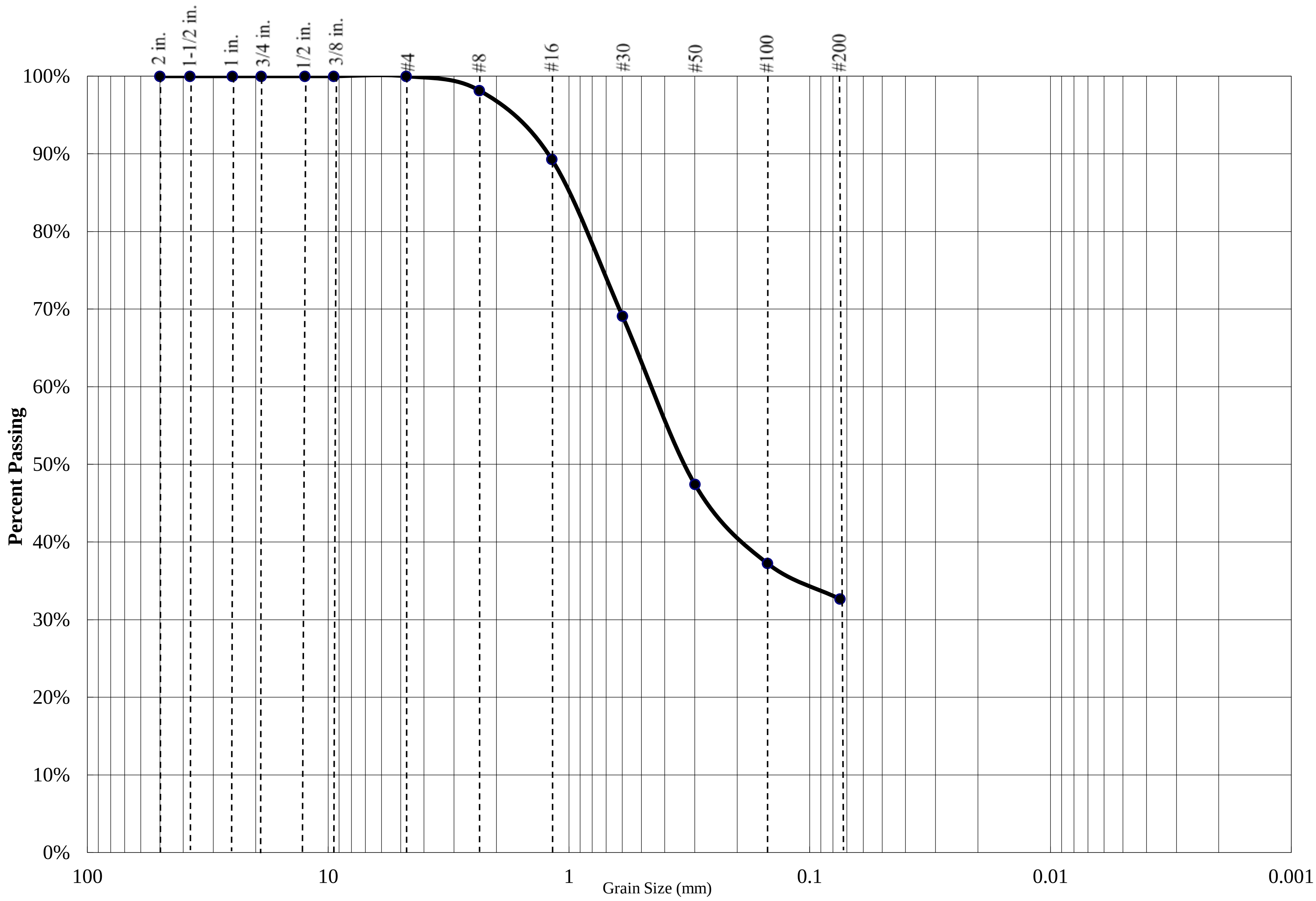
Project Number: 1-224-1068A

Boring: B-10 @ 8.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	67%	33%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	98.2%
#16	89.3%
#30	69.1%
#50	47.4%
#100	37.3%
#200	32.7%

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: Modernizations for Desmond Middle School

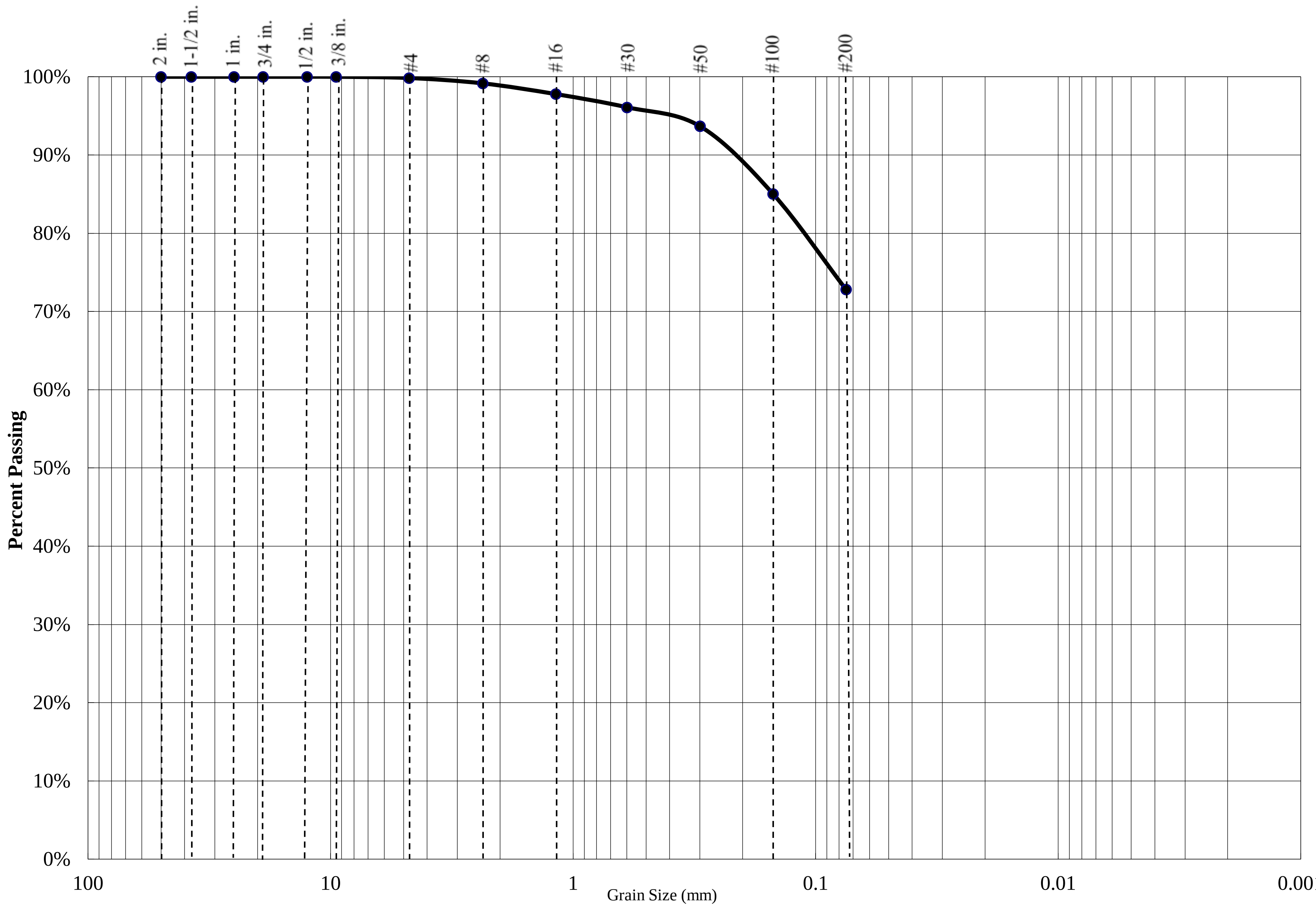
Project Number: 1-224-1068A

Boring: B-11 @ 3.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	27%	73%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.8%
#8	99.2%
#16	97.8%
#30	96.1%
#50	93.7%
#100	85.0%
#200	72.8%

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
SILT with Sand (ML)

Project Name: Modernizations for Desmond Middle School

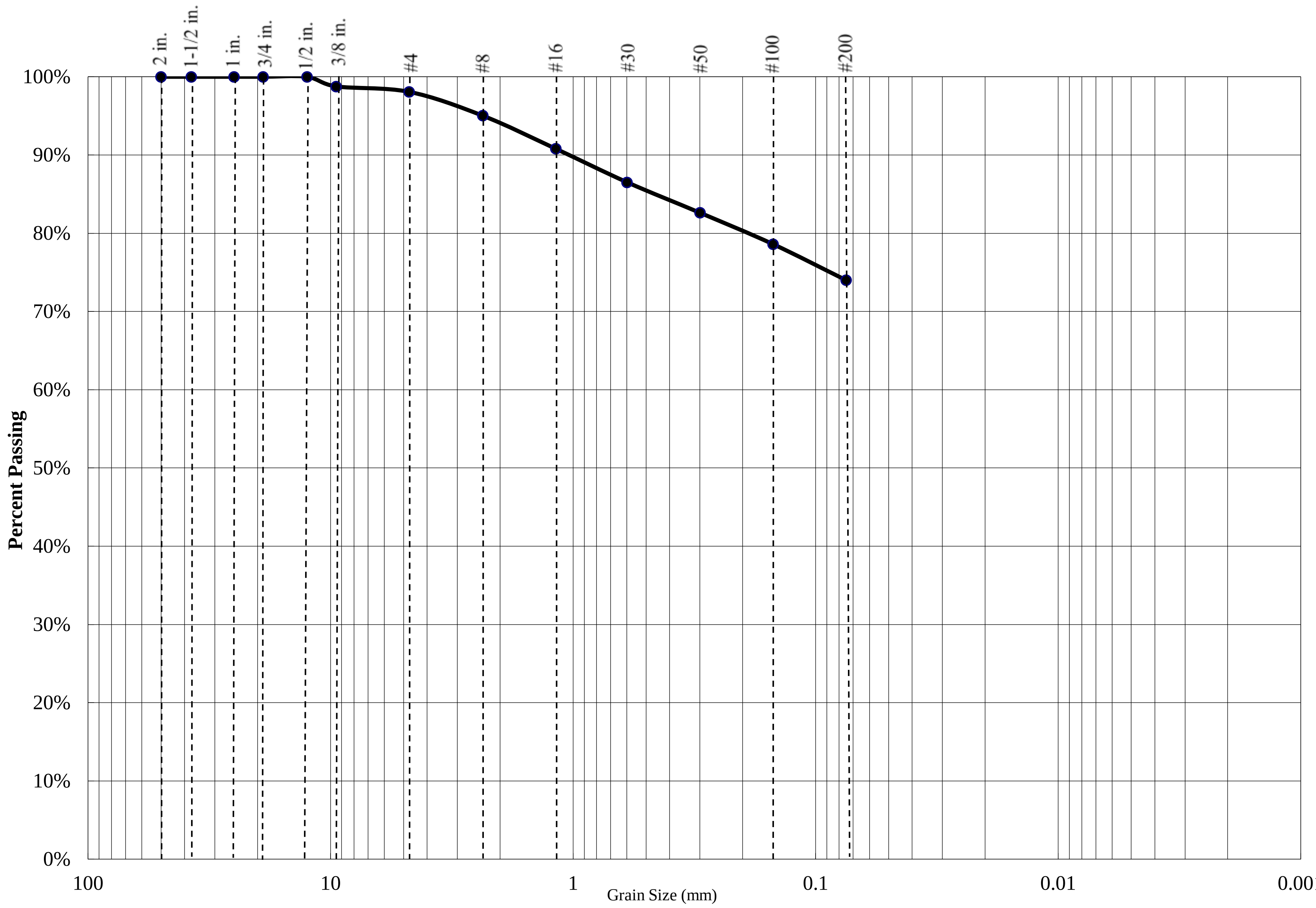
Project Number: 1-224-1068A

Boring: B-11 @ 8.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
2%	24%	74%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	98.8%
#4	98.1%
#8	95.0%
#16	90.8%
#30	86.5%
#50	82.6%
#100	78.6%
#200	74.0%

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
Lean CLAY with Sand (CL)

Project Name: Modernizations for Desmond Middle School

Project Number: 1-224-1068A

Boring: B-11 @ 13.5

SALEM engineering group, inc.

CHEMICAL ANALYSIS

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

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Date Sampled: 11/14/24 Date Tested: 12/3/24
Sampled By: CR/PC Tested By: MC
Soil Description: Sandy SILT/Lean CLAY

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 0 - 3'	220 mg/kg	26 mg/kg	7.5
1b.	B-1 @ 0 - 3'	210 mg/kg	25 mg/kg	7.5
1c.	B-1 @ 0 - 3'	220 mg/kg	26 mg/kg	7.5
Average:		**217 mg/kg**	**26 mg/kg**	**7.5**

SALEM engineering group, inc.

CHEMICAL ANALYSIS

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

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Date Sampled: 11/14/24 Date Tested: 12/3/24
Sampled By: CR/PC Tested By: MC
Soil Description: Silty SAND /Sandy Lean CLAY

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-3 @ 0 - 3'	100 mg/kg	64 mg/kg	7.2
1b.	B-3 @ 0 - 3'	110 mg/kg	65 mg/kg	7.2
1c.	B-3 @ 0 - 3'	100 mg/kg	66 mg/kg	7.2
Average:		**103 mg/kg**	**65 mg/kg**	**7.2**

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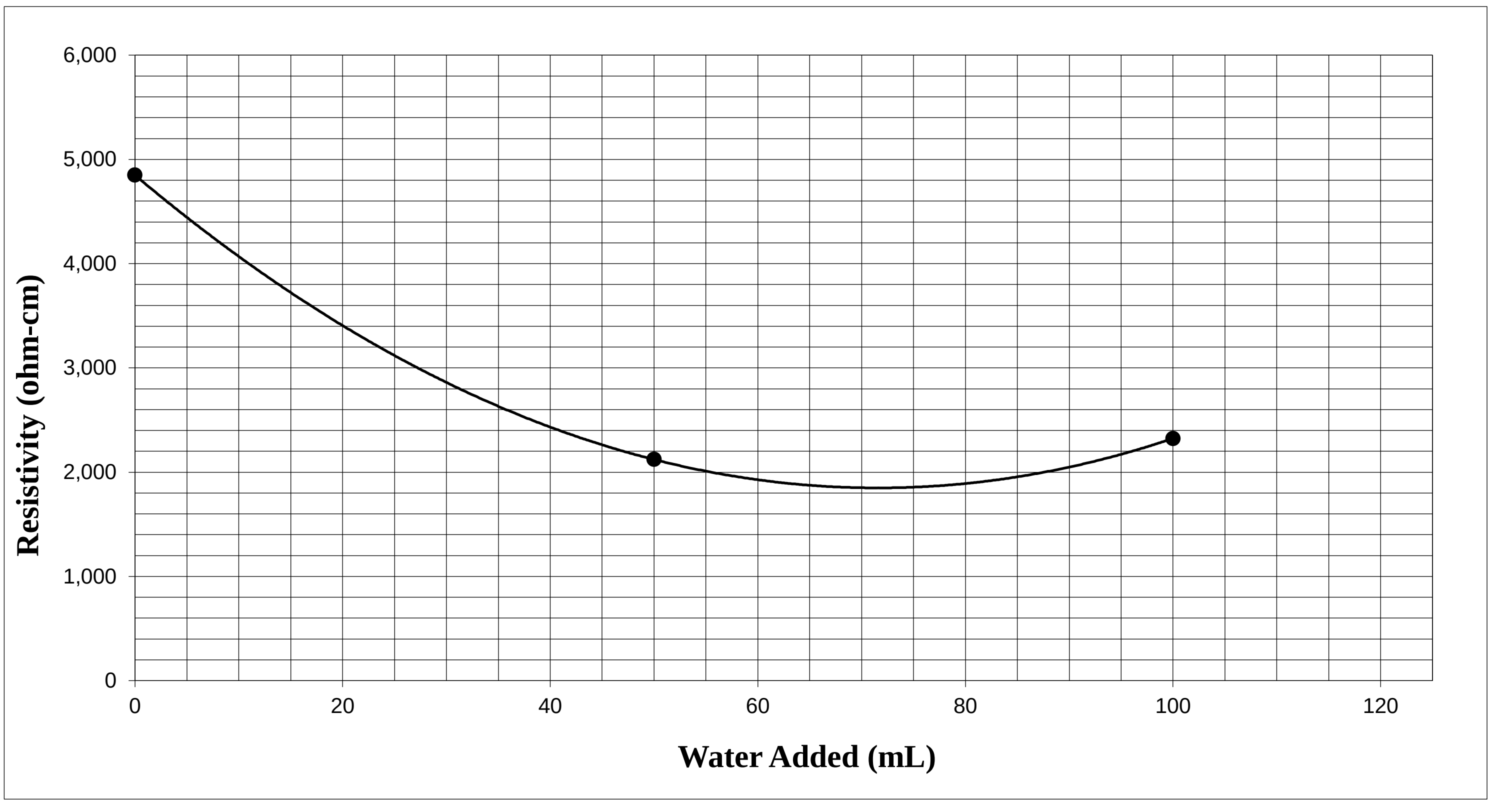
SOIL RESISTIVITY
CTM 643

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A — Date Sampled: 11/14/24
Sample Location: B-1 @ 0 - 3' — Sampled By: CR/PC
Soil Description: Sandy SILT/Lean CLAY — Date Tested: 12/2/24 — Tested By: DD

Chloride Content: 26 mg/Kg — Initial Sample Weight: 700 gms
Sulfate Content: 217 mg/Kg — Test Box Constant: 1.010 cm
Soil pH: 7.5

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm-cm)
1	0	4.8	1,000	4,800	4,848
2	50	2.1	1,000	2,100	2,121
3	100	2.3	1,000	2,300	2,323



Minimum Resistivity:	**1,849** ohm-cm

SALEM engineering group, inc.

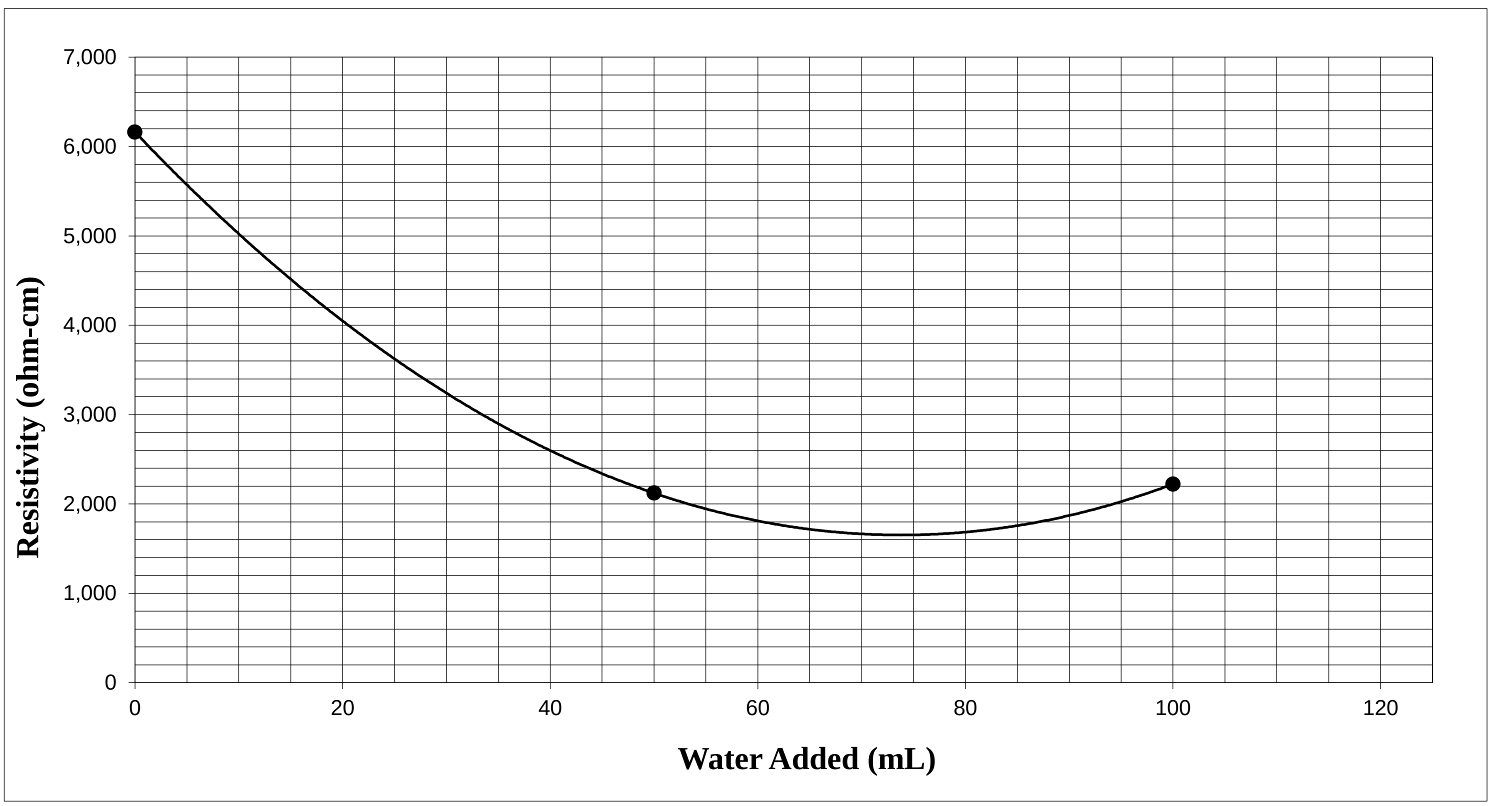
SOIL RESISTIVITY
CTM 643

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A   Date Sampled: 11/14/24
Sample Location: B-3 @ 0 - 3'   Sampled By: CR/PC
Soil Description: Silty SAND /Sandy Lean CLAY   Date Tested: 12/2/24   Tested By: DD

Chloride Content: 65 mg/Kg   Initial Sample Weight: 700 gms
Sulfate Content: 103 mg/Kg   Test Box Constant: 1.010 cm
Soil pH: 7.2

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm-cm)
1	0	6.1	1,000	6,100	6,161
2	50	2.1	1,000	2,100	2,121
3	100	2.2	1,000	2,200	2,222



Minimum Resistivity:	**1,653** ohm-cm

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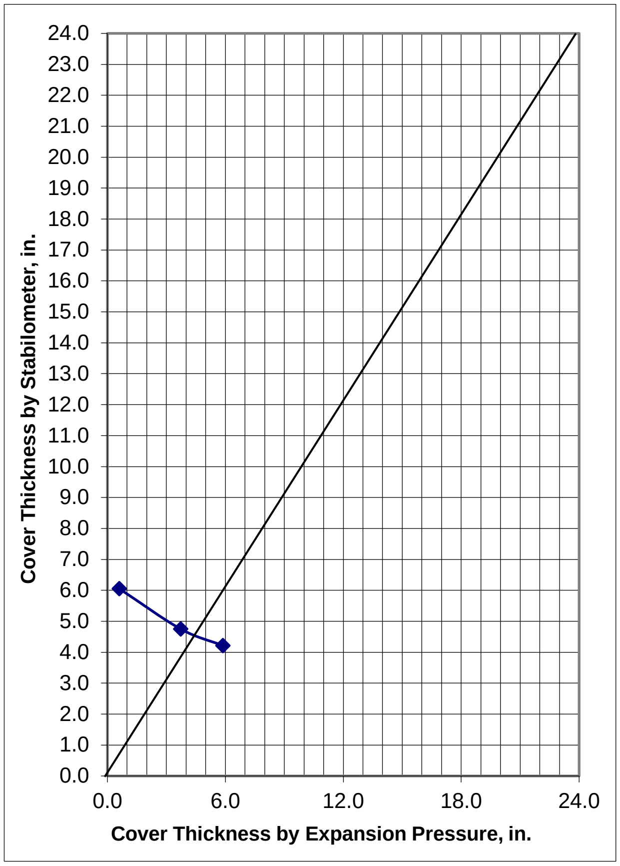
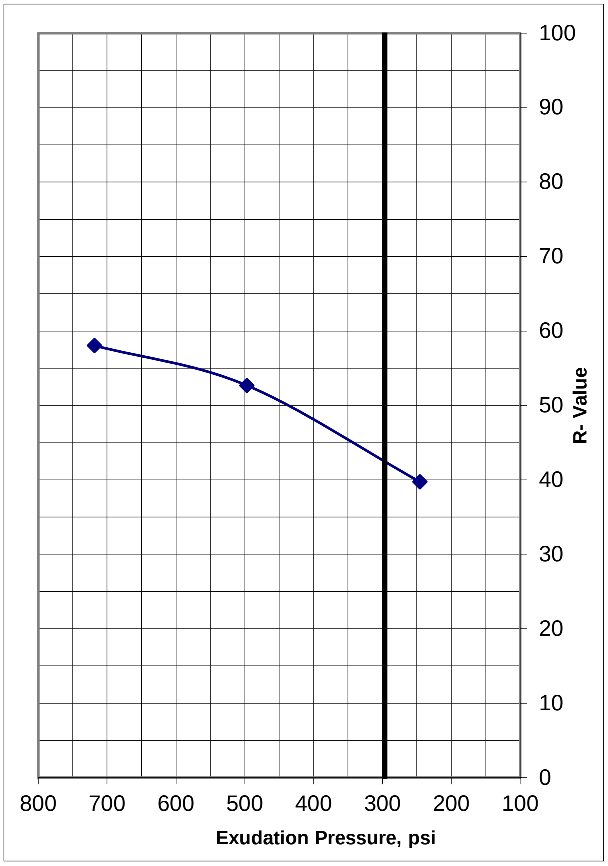
Resistance R-Value and Expansion Pressure of Compacted Soils

ASTM D2844

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Date Sampled: 11/14/24
Date Tested: 12/2/24
Sampled By: CR/PC
Tested By: JTA
Sample Location: B-3 @ 0 - 3'
Soil Description: Silty SAND /Sandy Lean CLAY





Specimen	1	2	3
Exudation Pressure, psi	718.3	496.9	245.4
Moisture at Test, %	11.2	11.7	13.2
Dry Density, pcf	124.2	123.6	122.9
Expansion Pressure, psf	637	403	65
Thickness by Stabilometer, in.	4.2	4.8	6.1
Thickness by Expansion Pressure, in.	5.9	3.7	0.6
R-Value by Stabilometer	58	53	40
R-Value by Expansion Pressure	55		
R-Value at 300 psi Exudation Pressure	42.5		

Controlling R-Value	**43**

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EXPANSION INDEX TEST
ASTM D4829

Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Date Sampled: 11/14/24 Date Tested: 12/2/24
Sampled By: CR/PC Tested By: DD
Sample Location: B-3 @ 0 - 3'
Soil Silty SAND /Sandy Lean CLAY

Trial #	1	2	3
Weight of Soil & Mold, g.	596.7		
Weight of Mold, g.	187.8		
Weight of Soil, g.	408.9		
Wet Density, pcf	123.3		
Weight of Moisture Sample (Wet), g.	810.0		
Weight of Moisture Sample (Dry), g.	742.3		
Moisture Content, %	9.1		
Dry Density, pcf	113.0		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	50.2		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.0167	0.0238	0.0273	--	0.028

Expansion Index $_{measured}$ = 28
Expansion Index $_{50}$ = 28.1

Expansion Index = 28

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

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Atterberg Limits Determination
ASTM D4318

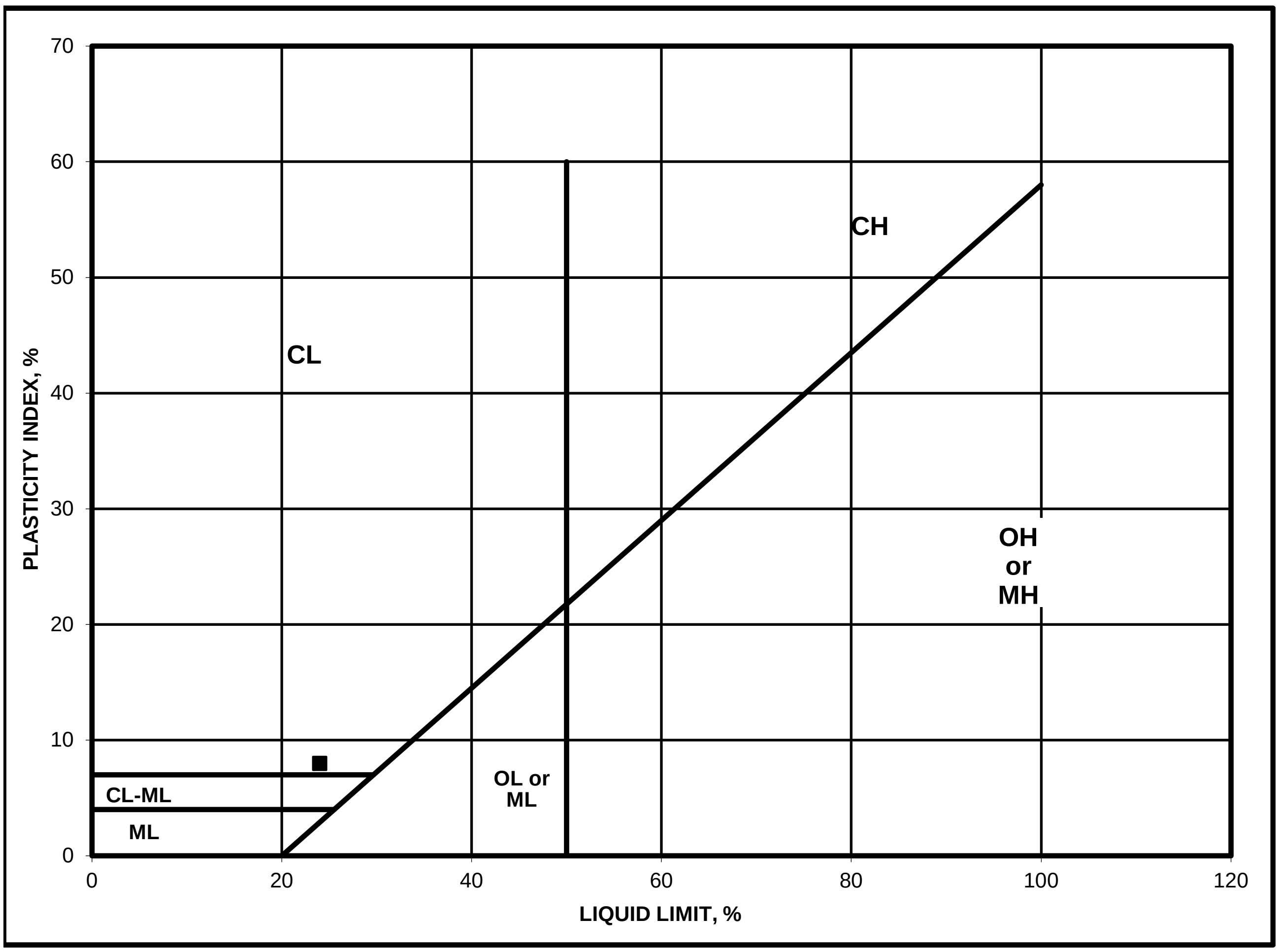
Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Date Sampled: 11/14/24 Date Tested: 12/2/24
Sampled By: CR/PC Tested By: MC
Sample Location: B-1 @ 3.5'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare	27.94	27.92	27.42	26.82	28.09	32.55
Weight of Dry Soil & Tare	26.96	26.89	26.44	24.71	25.71	30.26
Weight of Water	0.98	1.03	0.98	2.11	2.38	2.29
Weight of Tare	20.80	20.45	20.17	15.59	15.62	20.61
Weight of Dry Soil	6.16	6.44	6.27	9.12	10.09	9.65
Water Content	15.9	16.0	15.6	23.1	23.6	23.7
Number of Blows				32	27	20

Plastic Limit : 16 **Liquid Limit : 24**

Plasticity Index : 8

Unified Soil Classification : CL



Atterberg Limits Determination
ASTM D4318

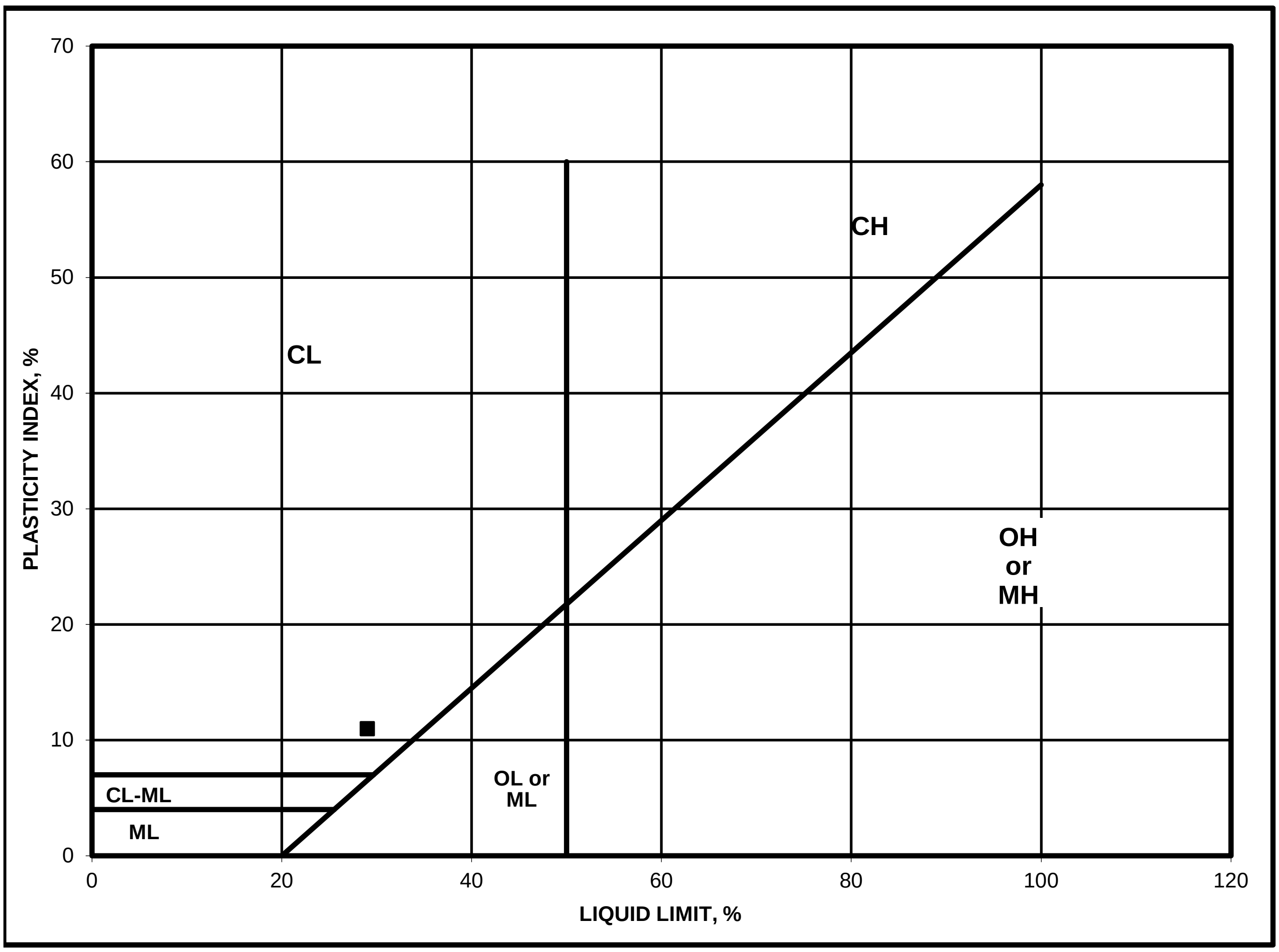
Project Name: Desmond Middle School - Madera, CA
Project Number: 1-224-1068-A
Date Sampled: 11/14/24 Date Tested: 12/2/24
Sampled By: CR/PC Tested By: MC
Sample Location: B-3 @ 5'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare	29.08	29.76	29.51	31.19	31.09	32.42
Weight of Dry Soil & Tare	27.98	28.60	28.39	28.91	28.88	29.91
Weight of Water	1.10	1.16	1.12	2.28	2.21	2.51
Weight of Tare	21.69	22.11	22.14	20.81	21.14	21.19
Weight of Dry Soil	6.29	6.49	6.25	8.10	7.74	8.72
Water Content	17.5	17.9	17.9	28.1	28.6	28.8
Number of Blows				31	26	21

Plastic Limit : 18 **Liquid Limit : 29**

Plasticity Index : **11**

Unified Soil Classification : **CL**



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Atterberg Limits Determination
ASTM D4318

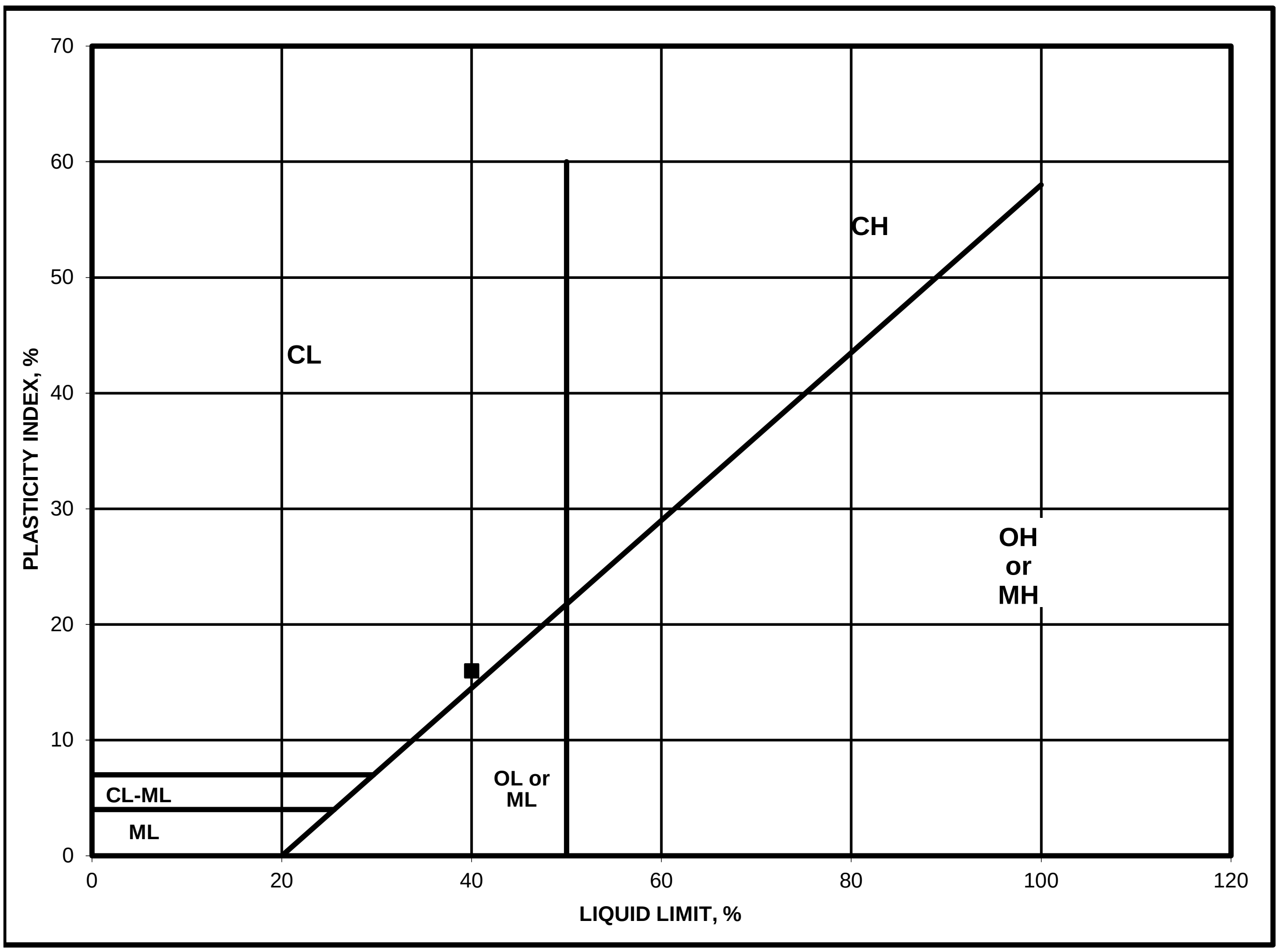
Project Name: Modernizations for Desmond Middle School
Project Number: 1-224-1068A
Date Sampled: 12/23/24 Date Tested: 1/2/24
Sampled By: R. Shaw Tested By: MC
Sample Location: B-7 @ 13.5'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare	22.27	27.26	27.50	30.66	32.19	32.39
Weight of Dry Soil & Tare	21.00	26.01	26.21	27.88	29.29	29.19
Weight of Water	1.27	1.25	1.29	2.78	2.90	3.20
Weight of Tare	15.66	20.76	20.82	20.74	21.92	21.30
Weight of Dry Soil	5.34	5.25	5.39	7.14	7.37	7.89
Water Content	23.8	23.8	23.9	38.9	39.3	40.6
Number of Blows				32	28	23

Plastic Limit : 24 **Liquid Limit : 40**

Plasticity Index : **16**

Unified Soil Classification : **CL**



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Atterberg Limits Determination
ASTM D4318

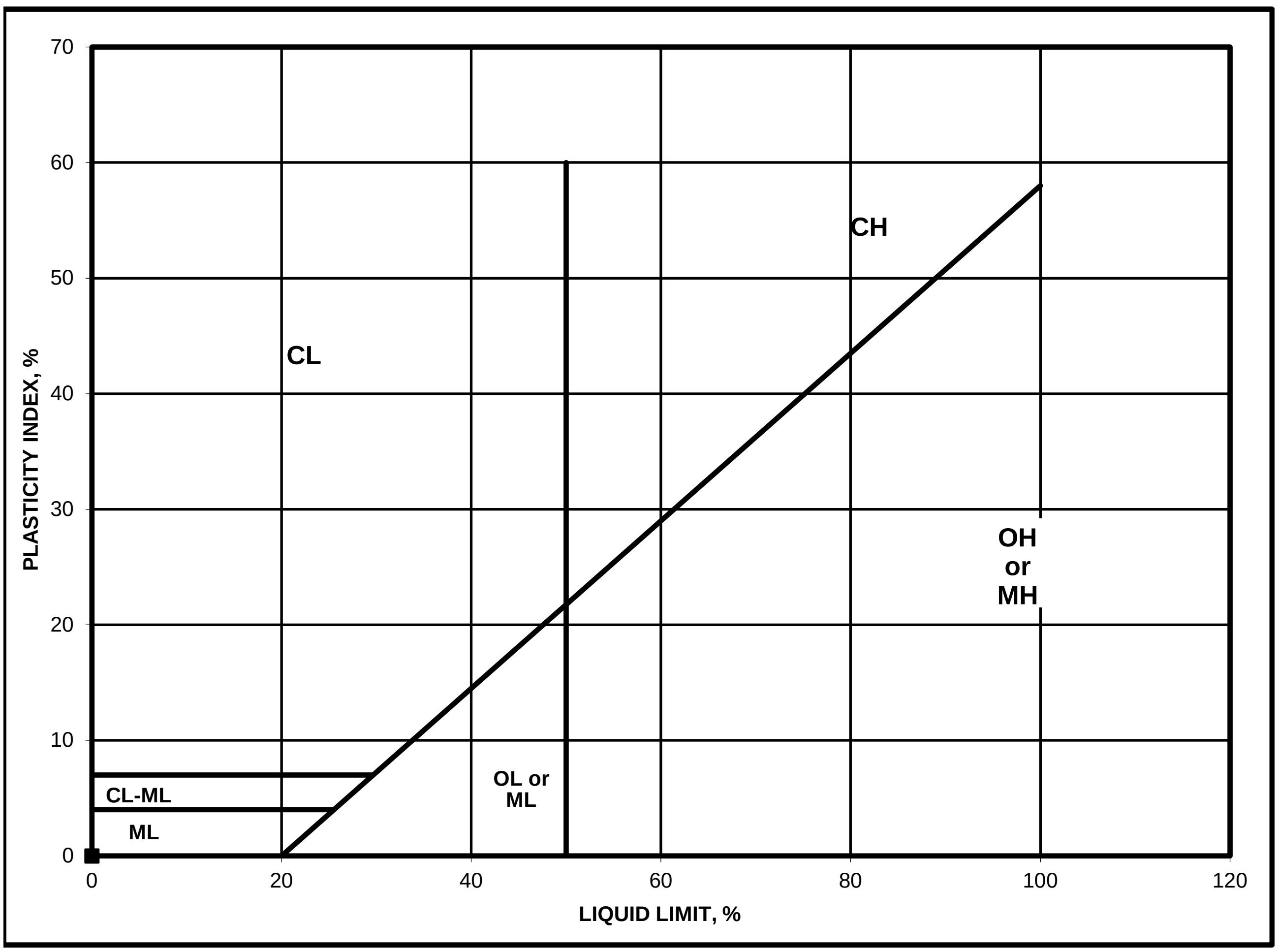
Project Name: Modernizations for Desmond Middle School
Project Number: 1-224-1068A
Date Sampled: 12/23/24 Date Tested: 1/2/24
Sampled By: R. Shaw Tested By: MC
Sample Location: B-9 @ 10'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare						
Weight of Dry Soil & Tare						
Weight of Water						
Weight of Tare	Does Not Roll			Slides on Cup		
Weight of Dry Soil						
Water Content						
Number of Blows						

Plastic Limit : **Liquid Limit :**

Plasticity Index **:**

Unified Soil Classification **:**



Atterberg Limits Determination
ASTM D4318

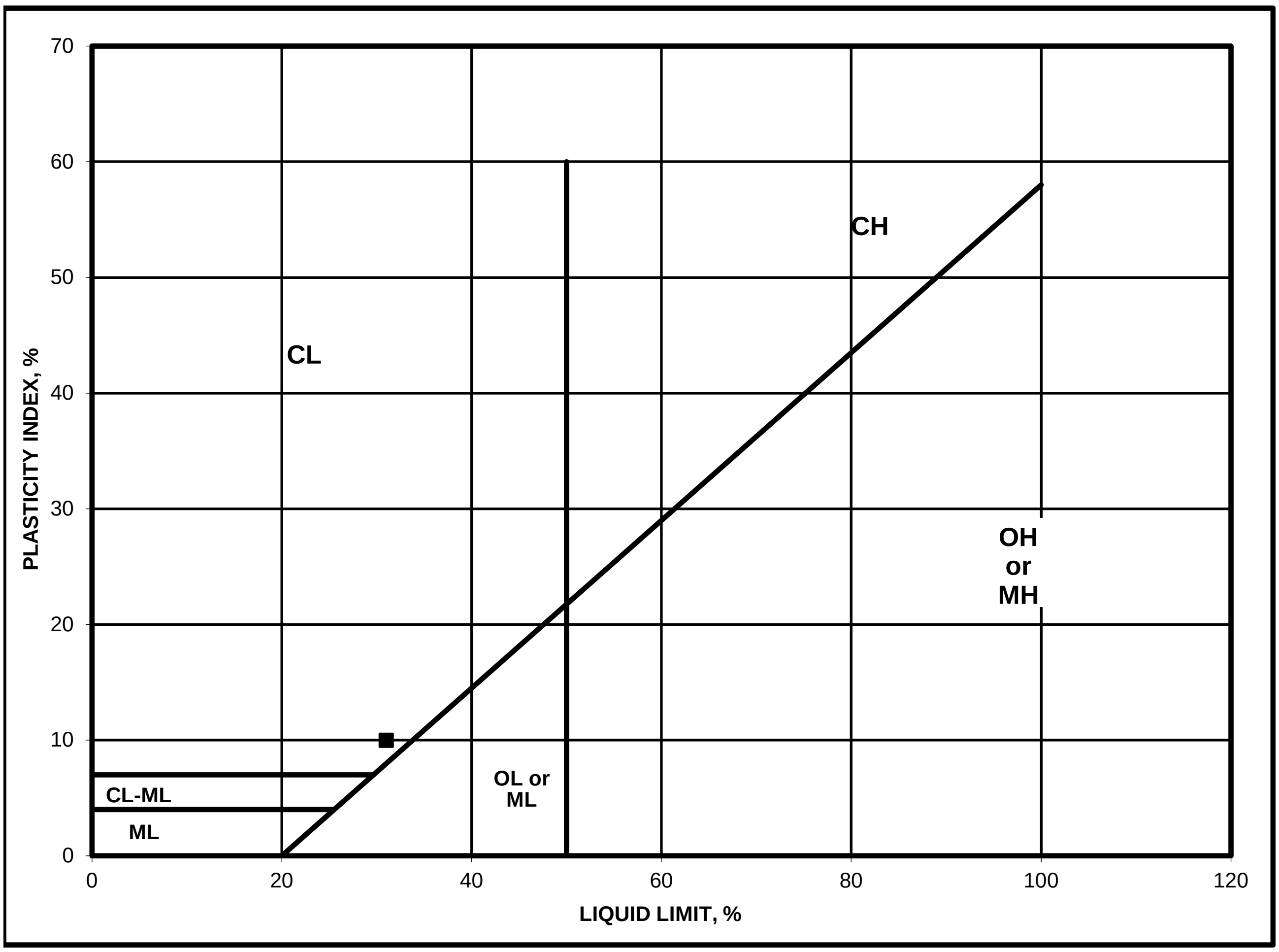
Project Name: Modernizations for Desmond Middle School
Project Number: 1-224-1068A
Date Sampled: 12/23/24 Date Tested: 1/2/24
Sampled By: R. Shaw Tested By: MC
Sample Location: B-10 @ 8.5'

Run Number	Plastic Limit 1	Plastic Limit 2	Plastic Limit 3	Liquid Limit 1	Liquid Limit 2	Liquid Limit 3
Weight of Wet Soil & Tare	27.25	26.75	26.73	28.31	27.07	30.89
Weight of Dry Soil & Tare	26.11	25.64	25.54	25.36	24.35	28.35
Weight of Water	1.14	1.11	1.19	2.95	2.72	2.54
Weight of Tare	20.77	20.42	20.09	15.58	15.62	20.53
Weight of Dry Soil	5.34	5.22	5.45	9.78	8.73	7.82
Water Content	21.3	21.3	21.8	30.2	31.2	32.5
Number of Blows				29	23	18

Plastic Limit : 21 **Liquid Limit : 31**

Plasticity Index : 10

Unified Soil Classification : CL



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Atterberg Limits Determination
ASTM D4318

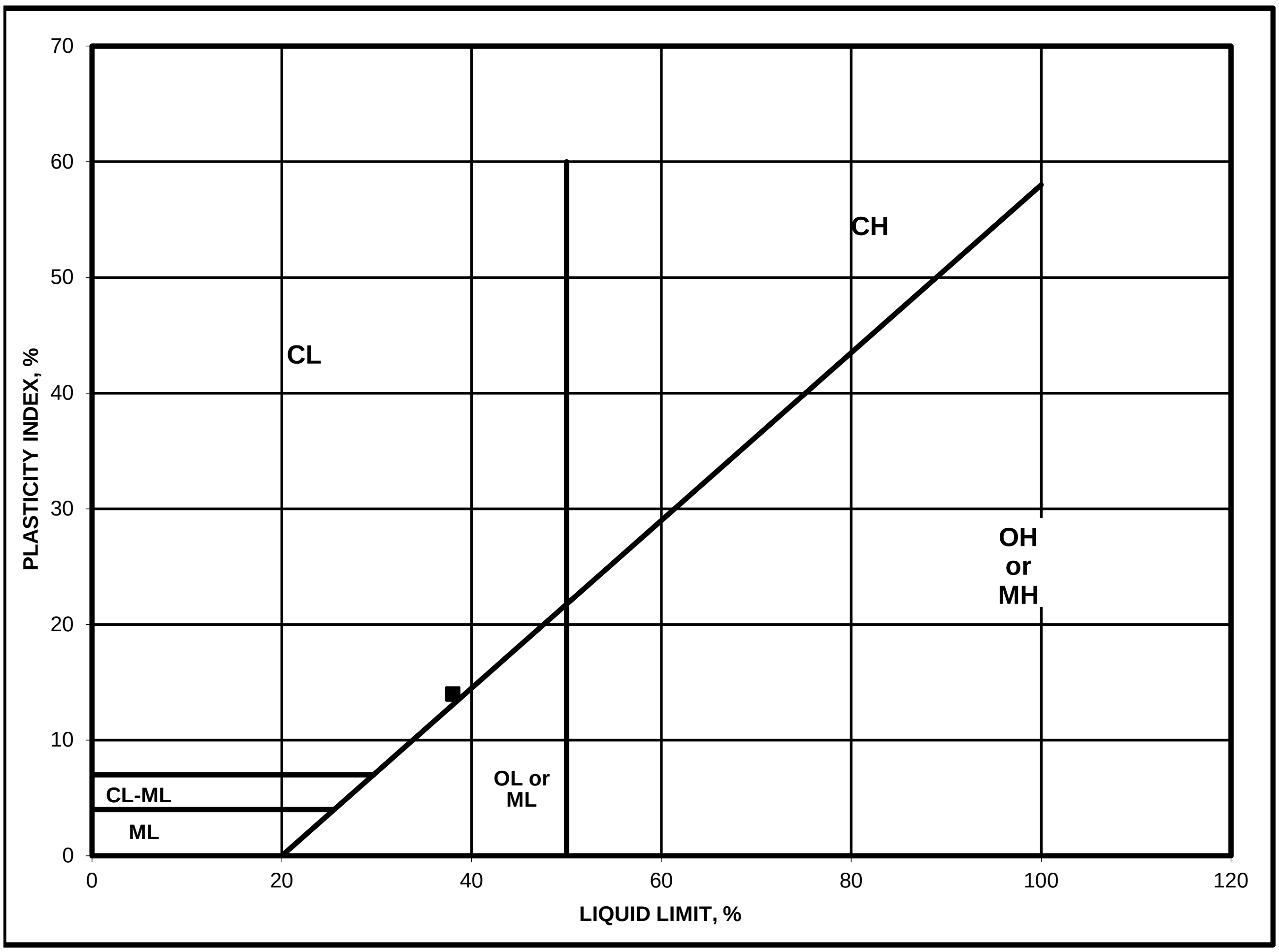
Project Name: Modernizations for Desmond Middle School
Project Number: 1-224-1068A
Date Sampled: 12/23/24 Date Tested: 1/2/24
Sampled By: R. Shaw Tested By: MC
Sample Location: B-11 @ 13.5'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare	21.67	27.15	22.00	27.08	27.52	32.38
Weight of Dry Soil & Tare	20.48	25.85	20.79	23.97	24.19	29.13
Weight of Water	1.19	1.30	1.21	3.11	3.33	3.25
Weight of Tare	15.63	20.49	15.78	15.68	15.52	20.88
Weight of Dry Soil	4.85	5.36	5.01	8.29	8.67	8.25
Water Content	24.5	24.3	24.2	37.5	38.4	39.4
Number of Blows				27	22	17

Plastic Limit : 24 **Liquid Limit : 38**

Plasticity Index : 14

Unified Soil Classification : CL



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APPENDIX

C

APPENDIX C
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 that 95 percent of relative compaction (90 percent for cohesive soils) 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 percent relative compaction (90 percent for cohesive soils).

Loose soil areas and/or areas of disturbed soil shall be moisture-conditioned as necessary and recompacted to 95 percent relative compaction (90 percent for cohesive soils). 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 percent 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 equipment.