Appendix B Geotechnical Engineering Investigation

Appendix

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

SALEM
engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED MIXED-USE BUILDING
8825 WASHINGTON BOULEVARD
PICO RIVERA, CALIFORNIA

SALEM PROJECT NO. 3-220-0499
JULY 31, 2020

PREPARED FOR:

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July 31, 2020 Project No. 3-220-0499

Mr. Jerome Mickelson
Executive Vice President
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**SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE BUILDING
8825 WASHINGTON BOULEVARD
PICO RIVERA, CALIFORNIA**

Dear Mr. Mickelson:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation report for the Proposed Mixed-Use Building to be located at the subject site.

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE BUILDING
8825 WASHINGTON BOULEVARD
PICO RIVERA, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation for the Proposed Mixed-Use Building to be located at 8825 Washington Boulevard in Pico Rivera, California (see Figure 1, Vicinity Map).

The purpose of our geotechnical engineering investigation was to observe and sample the subsurface conditions encountered at the site, and provide conclusions and recommendations relative to the geotechnical aspects of constructing the project as presently proposed. The scope of this investigation included a field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on July 7 and 8, 2020 and included the drilling of six (6) small-diameter soil borings to a maximum depth of 51½ feet at the site. Additionally, three (3) percolation tests were conducted at a depth of approximately 15 feet below ground surface. The locations of the soil borings and percolation tests are depicted on Figure 2, Site Plan. A detailed discussion of our field investigation, exploratory boring logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical properties for engineering analyses. Appendix B presents the laboratory test results in tabular and graphic format.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. If project details vary significantly from those described herein, SALEM should be contacted to determine the necessity for review and possible revision of this report.

Earthwork and Pavement Specifications are presented in Appendix C. If text of the report conflict with the specifications in Appendix C, the recommendations in the text of the report have precedence.

2. PROJECT DESCRIPTION

Based on the information provided to us, we understand the proposed development of the site will include demolition of a 34,445 square-foot restaurant building and construction of a mixed use, 6 story, 255-unit wrap-apartment with 7 levels of parking (1 level underground) and ±5,000 square feet of ground level retail spaces. The building also includes one level of underground public self-storage (±23,000 square feet) with a separate entrance. Maximum wall load is expected to be 30 kips per linear foot. Maximum column load is expected to be 500 kips. Floor slab soil bearing pressure is expected to be 150 psf.

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

3. SITE LOCATION AND DESCRIPTION

The project site encompasses approximately 2.6 acres and is located within the southwest portion of the Pico Rivera Marketplace which is situated at the northwest corner of Washington Boulevard and Rosemead Boulevard in the City of Pico Rivera, California (see Vicinity Map, Figure 1).

The site is currently occupied by a vacant restaurant building (8825 Washington Blvd) with paved parking and landscaping. The landscaping includes palm trees that are up to 50 feet tall.

At the time of our field exploration, a 4 to 5 foot deep trench with miscellaneous debris was present to the southwest of the vacant building. Excavated soils were stockpiled along the trench. Mounds of soil and debris were present directly west of the vacant building and at the westernmost portion of the site. The site is relatively flat with no major changes in grade and has an average elevation of approximately 165 feet above mean sea level based on Google Earth imagery.

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-6) were drilled on July 7 and 8, 2020 in the area shown on the Site Plan, Figure 2. The test borings were advanced with 6-inch diameter solid flight augers rotated by a truck-mounted CME 55 drill rig. The test borings were extended to a maximum depth of approximately 51½ feet below existing grade.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer and stratification lines were approximated on the basis of observations made at the time of drilling. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2487).

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

The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted. Soil samples were obtained from the test borings at the depths shown on the logs of borings. The MCS samples were recovered and capped at both ends to preserve the samples at their natural moisture content; SPT samples were recovered and placed in a sealed bag to preserve their natural moisture content. The borings were backfilled with soil cuttings after completion of the drilling.

5. LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory-testing program was formulated with emphasis on the evaluation of natural moisture, density, shear strength, consolidation potential, expansion, maximum density and optimum moisture determination, and gradation of the materials encountered.

In addition, chemical tests were performed to evaluate the corrosivity of the soils to buried concrete and metal. Details of the laboratory test program and the results of laboratory test are summarized in Appendix "B." This information, along with the field observations, was used to prepare the final boring logs in Appendix "A."

6. GEOLOGIC SETTING

The subject site is located in an area termed the central plain of the Los Angeles Basin between the Los Angeles River and San Gabriel River within the Peninsular Range of Southern California. This plain has been formed by deposition of alluvium within the flood plain of the Rio Hondo and San Gabriel River which flow generally southward from the hills and mountains to the north. Published reports indicate that the Quaternary Age alluvium is from 600 to 800 feet thick in the area, and is underlain by Tertiary Age marine sedimentary rocks several thousand feet in thickness. These deposits are generally fine to coarse grained, consisting primarily of mixtures of gravel, sand, and silt of valleys and floodplains. Tectonism of the region is dominated by the interaction of the East Pacific Plate and the North American Plate along a transform boundary. Deposits encountered on the subject site during exploratory drilling are discussed in detail in this report.

7. GEOLOGIC HAZARDS

7.1 Faulting and Seismicity

Based on the proximity of several dominant active faults and seismogenic structures, as well as the historic seismic record, the area of the subject site is considered subject to relatively high seismic activity. The seismic hazard most likely to impact the site is ground-shaking due to a large earthquake on one of the major active regional faults. Moderate to large earthquakes have affected the area of the subject site within historic time.

The project area is not within an Alquist-Priolo Earthquake Fault (Special Studies) Zone and will not require a special site investigation by an Engineering Geologist. Soils on site are classified as Site Class D in accordance with Chapter 16 of the California Building Code. The proposed structures are determined to be in Seismic Design Category D.

To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 33.9847° north; site longitude is 118.0984° west. The ten closest active faults are summarized below in Table 7.1.

TABLE 7.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Puente Hills (LA)	2.0	7.0
Elsinore; W+GI+T+J+CM	2.9	7.9
Puente Hills (Santa Fe Springs)	4.9	6.7
Elysian Park (Upper)	5.6	6.7
Puente Hills (Coyote Hills)	7.1	6.9
Raymond	9.4	6.8
Verdugo	10.6	6.9
Newport-Inglewood Connected alt 2	11.5	7.5
Newport-Inglewood Connected alt 1	11.8	7.5
Hollywood	12.0	6.7

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

7.2 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low.

7.3 Ground Shaking

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

Based on the 2019 CBC, a Site Class D represents the on-site soil conditions with standard penetration resistance, N-values, averaging between 15 and 50 blows per foot in the upper 100 feet below site grade. A table providing the recommended design acceleration parameters of the project site, based on a Site Class D designation, is included in Section 9.2.1 of this report.

Based on the Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.869g (based on both probabilistic and deterministic seismic ground motion).

7.4 Liquefaction

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.

Groundwater was not encountered during our investigation. Based on the State of California Seismic Hazard Zone Report 037, Whittier Quadrangle, Plate 1.2, Open-File Report 98-28, the historically highest groundwater is at a depth of approximately 15 feet below ground surface.

The soils encountered within the depth of 51½ feet on the project site consisted predominately of loose to very dense silty sand, well-graded sand, well-graded sand with silt, poorly graded sand, and poorly graded sand with silt; and soft to stiff silt, sandy silt, and sandy clay. Low to very low cohesion strength is associated with the sandy soil. A seismic hazard, which could cause damage to the proposed development during seismic shaking, is the post-liquefaction settlement of the liquefied sands.

Based on the State of California, Seismic Hazard Zone Map, Whittier Quadrangle, Dated March 25, 1999, the site is located within a liquefaction potential zone. The potential for soil liquefaction during a seismic event was evaluated using LiqIT computer program (version 4.7.5) developed by GeoLogismiki of Greece. For the analysis, a maximum earthquake magnitude of 7.9 M_w, a peak horizontal ground surface acceleration of 0.87g (with a 2% probability of exceedance in 50 years) and a groundwater depth of 15 feet were considered appropriate for the liquefaction analysis. The liquefaction analysis indicated that the site soils had a low potential for liquefaction under seismic conditions. The total liquefaction-induced settlements were calculated to be 2.34 to 3.87 inches. The liquefaction analysis report is included in Appendix A.

7.5 Lateral Spreading

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

7.6 Landslides

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.

7.7 Tsunamis and Seiches

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

8. SOIL AND GROUNDWATER CONDITIONS

8.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils within the depth of exploration consisted predominately of loose to very dense silty sand, well-graded sand, well-graded sand with silt, poorly graded sand, and poorly graded sand with silt; and soft to stiff silt, sandy silt, and sandy clay. The pavement within our test borings consisted of approximately 4 inches of asphalt concrete (AC) underlain by approximately 0 to 3 inches of aggregate base (AB). A layer of geofabric (Petromat) was encountered within the AC.

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

The soils were classified in the field during the drilling and sampling operations. The stratification lines were approximated by the field engineer on the basis of observations made at the time of drilling. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted. The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The locations of the test borings were determined by measuring from feature shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants.

8.2 Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was not encountered during this investigation. The historically highest groundwater is estimated to be at a depth of approximately 15 feet below existing grade based on the Seismic Hazard Zone Report 037, Whittier 7.5-Minute Quadrangle, Plate 1.2, Open-File Report 98-28.

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.

8.3 Soil Corrosion Screening

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

A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil sample was detected to be less than 50 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 8.3 below.

TABLE 8.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Water Soluble Sulfate (SO_4) in Soil, % by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Min. Concrete Compressive Strength	Cementations Materials Type
0.0050	Not Severe	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 24 mg/kg. This level of chloride concentration is considered to be mildly corrosive.

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.

8.4 Percolation Testing

Three (3) percolation tests (P-1 through P-3) were performed at the proposed infiltration system areas and were conducted in accordance with the criteria set in the Low Impact Development BMP Guideline of the County of Los Angeles, Department of Public Works. Results of the falling head tests are presented in the attachments to this report. The approximate locations of the percolation tests are shown on the attached Site Plan, Figure 2.

The 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 a certain time interval. The percolation rate data are presented in tabular format at the end of this Report. The difference in the percolation rates are reflected by the varied type of soil materials at the bottom of the test holes. The test results are shown on the table below.

TABLE 8.4
PERCOLATION TEST RESULTS

Test No.	Depth (feet)	Measured Percolation Rate (inch/hour)	Total Reduction Factor*	Design Infiltration Rate (inch/hour)**	Soil Type***
P-1	14.5	12.00	4	**3.00**	SAND (SP)
P-2	14.4	11.20	4	**2.80**	SAND w/Silt (SW/SM)
P-3	14.4	11.81	4	**2.95**	SAND (SP)

*** RF_t = 2, RF_v = 1, RF_s = 2, Total Reduction Factor, RF = RF_t x RF_v x RF_s = 4**
****Design Infiltration Rate = Measured Percolation Rate / RF**
***** At bottom of drilled holes**

RF_s = 1 to 3 which is based on the specified levels of pre-treatment and maintenance requirements. The value should be verified by the project Civil Engineer.

Please be advised that when performing percolation testing services in relatively small diameter borings, that the testing may not fully model the actual full scale long term performance of a given site. This is particularly true where percolation test data is to be used in the design of large infiltration system such as may be proposed for the site. The measured percolation rate includes dispersion of the water at the sidewalls of the boring as well as into the underlying soils. The soil infiltration or percolation rates are based on tests conducted with clear water. The infiltration/percolation rates may vary with time as a result of soil clogging from water impurities.

The soils may also become less permeable to impermeable if the soil is compacted. Thus, periodic maintenance consisting of clearing the bottom of the drainage system of clogged soils should be expected. The infiltration/percolation rate may become slower if the surrounding soil is wet or saturated due to prolonged rainfalls. Additional percolation tests should be conducted at bottom of the drainage system during construction to verify the infiltration/percolation rate. Groundwater, if closer to the bottom of the drainage system, will also reduce the infiltration/percolation rate. Infiltration systems shall be located, at minimum, a distance of 10 feet from any foundations and 10 feet from property lines. Infiltration in compacted fill is not allowed. Provided that the infiltration system is located at a minimum distance of 10 feet away from any foundations, the infiltration would not result in distress to the adjacent buildings.

The scope of our services did not include a groundwater study and was limited to the performance of percolation testing and soil profile description, and the submitted data only. Our services did not include those associated with septic system design. Neither did services include an Environmental Site Assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater, or atmosphere; or the presence of wetlands. Any statements, or absence of statements, in this report or on any boring logs regarding odors, unusual or suspicious items, or conditions observed, are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous and/or toxic assessment.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

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

9.1.2 The primary geotechnical constraints identified in our investigation is the presence of undocumented fill, liquefiable soils and compressible materials at the site. Recommendations to mitigate the effects of potentially compressible materials are provided in this report.

9.1.3 The scope of this investigation did not include subsurface exploration within the existing building during field exploration. As such, subsurface soil conditions and materials present below the existing site structures are unknown and may be different than those noted within this report. The presence of potentially unacceptable fill materials, undocumented fill, and/or loose soil material that may be present below existing site features shall be taken into consideration. Our firm shall be present at the time of demolition activities to verify soil conditions are consistent with those identified as part of this investigation.

9.1.4 Fill soils are anticipated to be present onsite between our test boring locations since the site was previously graded for the current development. Undocumented fill materials are not suitable to support any future structures and should be excavated and replaced with Engineered Fill. The extent and consistency of the fills should be verified during site construction. Prior to fill placement, SALEM should inspect the bottom of the excavation to verify the bottom condition.

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

9.1.6 Geogrid is a commonly and economically used method to reduce structural damage due to liquefaction. This method has been accepted by cities and counties throughout California, and implemented into design and construction of many retail buildings. However, this method may not be accepted by some local jurisdictions. We have no control for the acceptance of this method for this project. To use geogrid method, it is recommended that the proposed building be designed and the structural drawings be prepared **after** this report is approved by the City of Pico Rivera.

9.1.7 Recommendations for the geogrid system (option 1) are provided in Section 9.6 of this report. As an alternative to the use of geogrid, the proposed building may be supported by a structural slab system. A structural slab system will help reduce structural damage caused by liquefaction. Recommendations for a structural slab system (option 2) are provided in Section 9.7 of this report.

9.1.8 In lieu of the geogrid reinforcement method or the structural slab system, the buildings may be supported on deep foundations or by utilizing stone columns. Recommendations for a deep foundation system or the stone column method may be provided to the client by Salem Engineering Group, Inc. upon request.

9.1.9 SALEM shall review the project grading and foundation 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. If SALEM is not provided plans and specifications for review, we cannot assume any responsibility for the future performance of the project.

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

9.1.11 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.

9.2 Seismic Design Criteria

9.2.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2019 CBC, our recommended parameters are shown below. These parameters are based on Probabilistic Ground Motion of 2% Probability of Exceedance in 50 years. The Site Class was determined based on the results of our field exploration.

TABLE 9.2.1
SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	ASCE 7-16 or 2019 CBC Reference
Site Coordinates (Datum = NAD 83)		33.9847 Lat -118.0984 Lon	
Site Class	--	D	ASCE 7-16 Table 20.3-1
Soil Profile Name	--	Stiff Soil	ASCE 7-16 Table 20.3-1
Risk Category	--	II	CBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.1	ASCE 7-16 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.869g	ASCE 7-16 Equation 11.8-1
Seismic Design Category	SDC	D	ASCE 7-16 Section 11.6
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	1.833 g	CBC Figure 1613.2.1(1)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.656 g	CBC Figure 1613.2.1(2)
Site Class Modified Site Coefficient	F_a	1	CBC Table 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	*1.7	CBC Table 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	1.833 g	CBC Equation 16-36
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	*1.115 g	CBC Equation 16-37
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.222 g	CBC Equation 16-38
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	*0.743 g	CBC Equation 16-39
Short Term Transition Period (S_{D1}/S_{DS}), seconds	T_S	0.608	ASCE 7-16, Section 11.4.6
Long Term Transition Period (seconds)	T_L	8	ASCE 7-16, Figure 22-14

*** Determined per ASCE Table 11.4-2 for use in calculating T_S only**

9.2.2 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 S_1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific motion analysis may not be required based on Exceptions listed in ASCE 11.4.8. The Structural Engineer should verify whether Exception No. 2 of ASCE 7-16, Section 11.4.8, is valid for the site. In the event that a site specific ground motion analysis is required, SALEM should be contacted for these services.

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

9.3 Soil and Excavation Characteristics

9.3.1 Based on the soil conditions encountered in our soil borings, the onsite soils can be excavated with moderate effort using conventional excavation equipment.

9.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements. Temporary excavations are further discussed in a later Section of this report.

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

9.4 Materials for Fill

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

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

9.4.3 Import soil shall be well-graded, slightly cohesive silty fine sand or sandy silt, with relatively impervious characteristics when compacted. A clean sand or very sandy soil is not acceptable for this purpose. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 9.4.3.

TABLE 9.4.3
IMPORT FILL REQUIREMENTS

Minimum Percent Passing No. 200 Sieve	15
Maximum Percent Passing No. 200 Sieve	50
Minimum Percent Passing No. 4 Sieve	70
Maximum Particle Size	3"
Maximum Plasticity Index	12
Maximum CBC Expansion Index	20

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

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

9.5 Grading

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

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

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

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

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

9.5.6 Recommendations for grading of the proposed building area are provided in Sections 9.6 and 9.7 (Options 1 or 2 for liquefaction mitigation) of this report.

9.5.7 Prior to placement of fill soils, the upper 10 to 12 inches of native subgrade soils should be scarified, moisture-conditioned to **<u>no less</u>** than the optimum moisture content and re-compacted to a minimum of 95% of the maximum dry density based on ASTM D1557-07 Test Method.

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

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

9.5.10 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.

9.5.11 Within pavement areas, it is recommended that scarification, moisture conditioning and re-compaction be performed to at least 12 inches below existing grade or finish grade, whichever is deeper. In addition, the upper 12 inches of final pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned to no less than the optimum moisture content and compacted to at least 95% relative compaction.

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

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

9.5.14 We do not anticipate groundwater or seepage to adversely affect construction if conducted during the drier months of the year (typically summer and fall). However, groundwater and soil moisture conditions could be significantly different during the wet season (typically winter and spring) as surface soil becomes wet; perched groundwater conditions may develop. Grading during this time period will likely encounter wet materials resulting in possible excavation and fill placement difficulties. Project site winterization consisting of placement of aggregate base and protecting exposed soils during construction should be performed. If the construction schedule requires grading operations during the wet season, we can provide additional recommendations as conditions warrant.

9.5.15 Wet soils will become non conducive to site grading as the upper soils yield under the weight of the construction equipment. Therefore, mitigation measures should be performed for stabilization. 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, slurry or crushed rock may be utilized for stabilization provided this method is approved by the owner for the cost purpose. If the use of slurry or crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of 2-sack slurry or ¾-inch to 1-inch crushed rocks. The thickness of the slurry or rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of crushed rock material will provide a stable platform.

It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock. A layer of geofabric is recommended to be placed on top of the compacted crushed rock to minimize migration of soil particles into the voids of the crushed rock, resulting in soil movement. Although it is not required, the use of geogrid (e.g. Tensar TX7) 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.

9.6 Option 1 - Shallow Foundations and Slabs with Geogrid

9.6.1 The site is suitable for use of conventional shallow foundations consisting of continuous strip footings in combination with isolated spread footings bearing on geogrid reinforced Engineered Fill.

9.6.2 Subsurface soils within the site are prone to liquefaction under high ground shaking acceleration during an earthquake. Our preliminary calculations indicated that the building areas, and at least 5 feet beyond, should be over-excavated to a depth of 4 feet below proposed footing bottom and the resulting excavation should be backfilled with a layered system of Engineered Fill and geogrid reinforcing fabric.

Any undocumented and uncompacted fills encountered during grading should be removed and replaced with engineered fill. The depth of the over-excavation should be measured from existing ground or rough pad grade, whichever is greater.

A preliminary design procedure is provided below. Global seismic induced settlement of the site is still anticipated when liquefaction occurs. Prior to placing the geogrid, the bottom of the subgrade should be scarified to a depth of 10 to 12 inches, moisture conditioned to no less than the optimum moisture content, and re-compacted to a minimum of 95% relative compaction based on ASTM D1557.

The first layer of geogrid reinforcement will be placed directly on the prepared subgrade at a depth of 4 feet below proposed footing bottom. The geogrid material should be overlapped a minimum of 3 feet in all directions. The interlock between the geogrid and Engineered Fill will provide load transfer. No vehicles may traverse the geogrid prior to placement of the Engineered Fill cover. The next layer of geogrid should be placed on top of the compacted Engineered Fill. This and subsequent layers need only be overlapped a minimum of 1 foot on all sides.

The fill soils excavated from the area may be moisture conditioned and re-compacted between geogrid layers as reinforced fill. The reinforced fill should be moisture conditioned to near optimum moisture content and re-compacted to a minimum of 95% of the maximum dry density based on ASTM D 1557 Test Method.

A total of <u>four (4)</u> geogrid layers, including the layer at the base of the excavation should be installed at vertical increments of 1 foot. The geogrid layers should extend to a minimum of 5 feet beyond the exterior footing perimeter of the structure. The geogrid reinforcement fabric should consist of <u>Tensar® TX7 Geogrid</u>. Any additional unstable soils within building areas should be excavated and backfilled with Engineered Fill.

It is recommended that the entire site be excavated at once, and soils be stockpiled on adjacent or nearby properties. The geogrid and excavated soil may then be placed and re-compacted as recommended herein.

Alternatively, the contractor may elect to excavate the site in two stages, where excavated soil can be stockpiled over one-half of the site while the other half is mitigated. However, if the contractor elects the option of two stages over the preferred option of using one stage, a minimum of 5 feet of geogrid from the first half should overlap the second half.

Furthermore, the overlapping geogrid should be protected from damages, which may be caused by operating equipment. It is further recommended that flexible utility connections be used for the project.

9.6.3 It is recommended that continuous bearing wall footings to be utilized for the building have a minimum width of 15 inches, and a minimum embedment depth of 24 inches below lowest adjacent pad grade (18 inches for below ground structures). Isolated column footings should have a minimum width of 24 inches, and a minimum embedment depth of 24 inches below lowest adjacent pad grade (18 inches for below ground structures).

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

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

Loading Condition	Allowable Bearing
Dead Load Only	2,500 psf
Dead-Plus-Live Load	3,000 psf
Total Load, Including Wind or Seismic Loads	4,000 psf

9.6.6 For design purposes, total static settlement not exceeding ½ inch may be assumed for shallow foundations. Differential static settlement should not exceed ¼ inch over 30 feet. Most of the settlement is expected to occur during construction as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated. The footing excavations should not be allowed to dry out any time prior to pouring concrete.

9.6.7 The total settlement due to seismic loads is expected to be on the order of 2.34 to 3.87 inches. With the geogrid reinforcement, the seismic induced differential settlement is expected to be reduced to approximately ½ inch over 30 feet.

9.6.8 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 native subgrade.

9.6.9 Lateral resistance for footings can alternatively be developed using an equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical native footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combination that includes wind or earthquake loads.

9.6.10 Minimum reinforcement for continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

9.6.11 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.

9.6.12 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.

9.6.13 Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that non-structural slabs-on-grade be at least 4 inches thick and underlain by six (6) inches of compacted clean granular aggregate subbase material compacted to at least 95% relative compaction. Crushed Miscellaneous Base (CMB) should not be used within the building pad area.

9.6.14 Granular aggregate subbase material shall conform to ASTM D-2940, Latest Edition (Table 1, bases) with at least 95 percent passing a 1½-inch sieve and not more than 8% passing a No. 200 sieve or its approved equivalents to prevent capillary moisture rise.

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

9.6.16 Slabs subject to structural loading may be designed utilizing a modulus of subgrade reaction K of 140 pounds per square inch per inch. The K value was approximated based on inter-relationship of soil classification and bearing values (Portland Cement Association, Rocky Mountain Northwest).

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

9.6.18 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.

9.6.19 It is recommended that the utility trenches within the structures 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.

9.6.20 Moisture within the structures 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 structures. 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 structures is recommended to reduce the accumulation of interior moisture.

9.6.21 In areas where it is desired to reduce floor dampness where moisture-sensitive coverings are anticipated, construction should have a suitable waterproof vapor retarder (a minimum of 15 mils thick polyethylene vapor retarder sheeting, Raven Industries "VaporBlock 15, Stego Industries 15 mil "StegoWrap" or W.R. Meadows Sealtight 15 mil "Perminator") incorporated into the floor slab design. The water vapor retarder should be decay resistant material complying with ASTM E96 not exceeding 0.04 perms, ASTM E154 and ASTM E1745 Class A. 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 1643-94.

9.6.22 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.

9.6.23 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.

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

9.7 Option 2 – Structural Slabs

9.7.1 As an alternative to the geogrid method, the building may be supported on a reinforced structural slab foundation system (e.g. mat foundation, modified mat foundation, post-tensioned slab or stiffened footings with rigid grade beams) to resist damage due to seismic-induced differential settlement.

9.7.2 The foundation can be designed utilizing allowable bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. This value may be increased by 1/3 for short duration loads such as wind or seismic. The thickness and reinforcement of the structural slab should be determined by the Structural Engineer.

9.7.3 The structural slab should have a minimum depth of 12 inches below the lowest adjacent exterior grade. <u>The structural slab should be supported by at least 3 feet of Engineered Fill except in areas where slab subgrade is deeper than 10 feet below existing grade.</u>

9.7.4 <u>Any undocumented and uncompacted fills encountered during grading should be removed and replaced with engineered fill.</u>

9.7.5 Slab subgrade and Engineer Fill should be moisture conditioned to near optimum moisture content and re-compacted to a minimum of 95% of the maximum dry density based on ASTM D 1557 Test Method.

9.7.6 The total settlement due to foundation loads (static) is not expected to exceed 1 inch. Differential settlement due to static loads should be less than ½ inch over 30 feet. Most of the settlement is expected to occur during construction as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated.

9.7.7 The seismic-induced total settlements are expected to be on the order of 2.34 to 3.87 inches. The seismic-induced differential settlement is estimated to be one half of the total settlements. It is further recommended that flexible utility connectors be used for this project.

9.7.8 Resistance to lateral footing displacement can be computed using an allowable friction factor of 0.35 acting between the base of foundations and the supporting subgrade. Lateral resistance for footings can alternatively be developed using an equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical slab faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance.

9.8 Lateral Earth Pressures and Frictional Resistance

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

Lateral Pressure Conditions	Equivalent Fluid Pressure, pcf
Active Pressure, Drained	40
At-Rest Pressure, Drained	60
Passive Pressure	350
Traffic Surcharge (resultant at mid-highest)*****	$0.64Q/(m^2+1)$
Related Parameters	
Allowable Coefficient of Friction	0.40
In-Place Soil Density (lbs/ft^3)	120

***Q = line load from traffic, m=x/h where x= horizontal distance between wall and traffic, h=wall height.**

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

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

9.8.4 A safety factor consistent with the design conditions should be included in their usage.

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

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

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

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

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

9.9 Retaining Walls

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

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

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

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

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

9.10 Temporary Excavations

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

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

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

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

RECOMMENDED EXCAVATION SLOPES

Depth of Excavation (ft)	Slope (Horizontal : Vertical)
0-5	1:1
5-10	2:1
10-15	4:1

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

9.10.6 Braced shorings should be designed for a maximum pressure distribution of 26H, (where H is the depth of the excavation in feet). The foregoing does not include excess hydrostatic pressure or surcharge loading. Fifty percent of any surcharge load, such as construction equipment weight, should be added to the lateral load given herein. Equipment traffic should concurrently be limited to an area at least 3 feet from the shoring face or edge of the slope.

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

9.11 Temporary Soldier Pile Shoring System

9.11.1 The maximum cut required for construction of the underground parking garage and basement is estimated to be approximately 15 to 20 feet below exiting grade. Temporary excavations can be achieved using a soldier-pile and lagging system.

9.11.2 For design of a Soldier Pile Shoring System, it's recommended a uniform distribution of 26H be used for temporary shoring design of a restrained system (tieback or rakers). Passive pressure for temporary shorings soldier beam may be doubled (i.e. effective pile width = 2 times the pile diameter) for isolated pile condition. Pile size should be determined by the structural engineer.

9.11.3 Additional horizontal pressures from surface load should be considered. Surcharge load can be computed as $0.64Q/(m^2+1)$ where Q = line load from traffic, structures, or heavy materials, m=x/h where x= horizontal distance between shoring wall and line load, h=wall height. The resultant load is located at mid height of the wall.

9.11.4 The minimum depth of pile embedment should be 12 feet below bottom of footing or 1.4H for cantilever beams. The minimum depth of embedment should be 8 feet below bottom of footing or 0.5H for braced beams.

9.11.5 For tieback anchors, soil active wedge angle should be 35 degrees from the wall. Tieback anchor insertion angle should be a minimum of 15° from horizontal. The most common angle is between 15 to 30°. An angle steeper than 30° may be used if the ends of the anchors need to be deep to avoid encroachment of existing structures. The minimum bond length for tieback should equal to [Tendon Force / (Circumference x Shear Strength of Grout)] or 10 feet. An ultimate bond strength of post-grout (high pressure) of 1,500 psf may be used for the tieback design. The minimum overburden for the anchors should be 15 feet. Pressure grouting will be required if fissures are encountered. All anchors should be tested to 150% of the design load. At least 10% of the anchors should be tested to 200% of the design load for 30 minutes and some quantity, usually 4 anchors, should be tested for 200% of the design load for 24 hours.

9.11.6 The maximum design pressure for lagging is 65L psf, where L is the lagging clear span. Due to the property line constraints, the drainage system to be installed behind the basement wall may consist of a 6-inch wide and 24-inch high gravel pocket (equivalent to one cubic foot per foot) wrapped with filter fabric or a drainage board (e.g. Miradrain or Terradrain).

9.11.7 All void space behind lagging and shoring should be completely filled with sand/cement slurry. Care should be exercised when excavating into the on-site soils since potential of caving or sloughing of these materials is moderately high. Shoring of excavation is the responsibility of contractor.

9.12 Underground Utilities

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

9.12.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to approximately 6 to 12 inches above the crown of the pipe. Pipe bedding and backfill material should conform to the requirements of the governing utility agency.

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

9.12.4 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.

9.13 Surface Drainage

9.13.1 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.

9.13.2 The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than 5 percent for a minimum distance of 10 feet.

9.13.3 Impervious surfaces within 10 feet of the buildings foundation shall be sloped a minimum of 2 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 structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed.

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

9.14 Pavement Design

9.14.1 Based on site soil conditions, an R-value of 25 was used for preliminary flexible asphaltic concrete pavement design. The R-value may be verified during grading of the pavement areas.

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

TABLE 9.14.2
ASPHALT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Asphaltic Concrete	Crushed Aggregate Base*	Compacted Subgrade*
5.0 (Parking & Vehicle Drive Areas)	4.0"	4.0"	12.0"
6.0 (Heavy Truck Areas)	4.0"	7.0"	12.0"

95% compaction based on ASTM D1557 Test Method

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

TABLE 9.14.3
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

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

**** Minimum Compressive Strength of 4,000 psi; Minimum Reinforcement of #4 bars at 15" O.C., Each Way***
***** 95% compaction based on ASTM D1557 Test Method***

10. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

10.1 Plan and Specification Review

10.1.1 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.

10.2 Construction Observation and Testing Services

10.2.1 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.

10.2.2 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.

10.2.3 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.

11. LIMITATIONS AND CHANGED CONDITIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the test borings drilled at the approximate locations shown on the Site Plan, Figure 2. The report does not reflect variations which may occur between borings. The nature and extent of such variations may not become evident until construction is initiated. If variations then appear, a re-evaluation of the recommendations of this report will be necessary after performing on-site observations during the excavation period and noting the characteristics of such variations. The findings and recommendations presented in this report are valid as of the present and for the proposed construction.

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

SALEM does not practice in the field of corrosion engineering. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, that manufacturer's recommendations for corrosion protection be closely followed. Further, a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of concrete slabs and foundations in direct contact with native soil.

The importation of soil and or aggregate materials to the site should be screened to determine the potential for corrosion to concrete and buried metal piping. The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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Geotechnical Staff Engineer

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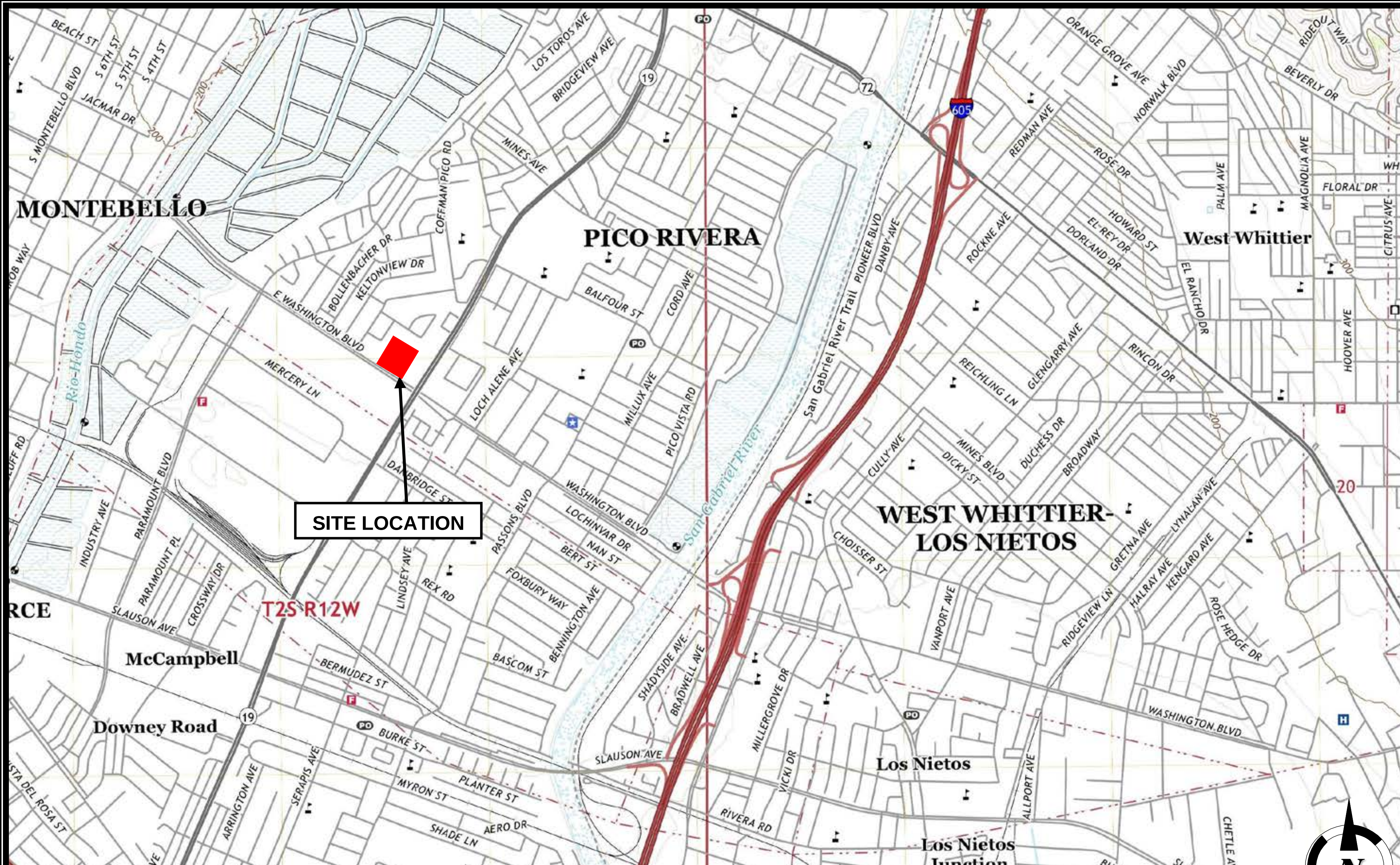


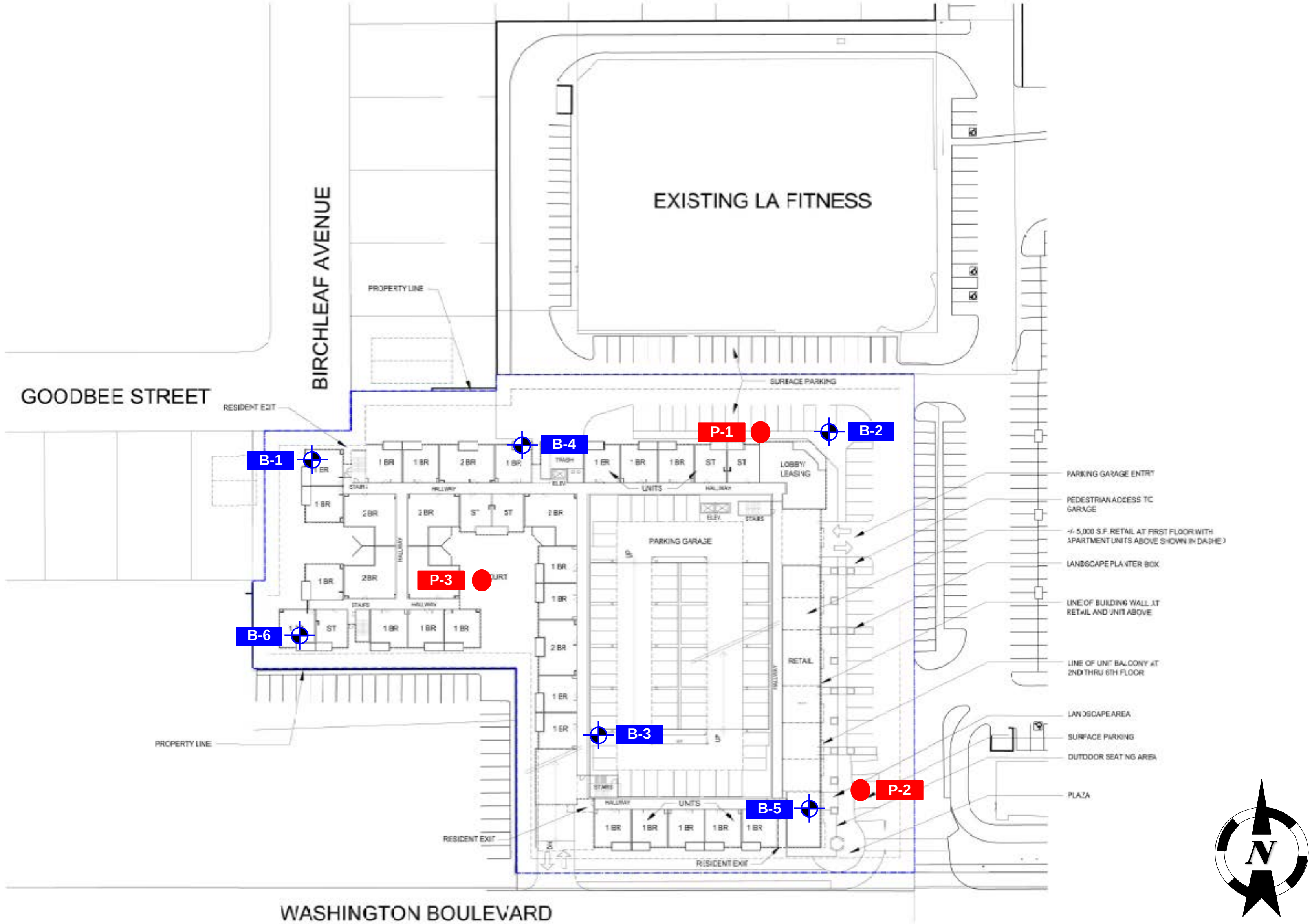
Source Image: https://ngmdb.usgs.gov/topoview

VICINITY MAP

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Mixed-Use Building
8825 Washington Boulevard
Pico Rivera, California

SCALE: NOT TO SCALE	DATE: 07/2020
DRAWN BY: JC	APPROVED BY: CJ
PROJECT NO. 3-220-0499	FIGURE NO. 1

SALEM engineering group, inc.



SITE PLAN	SCALE: NOT TO SCALE	DATE: 07/2020	LEGEND:
GEOTECHNICAL ENGINEERING INVESTIGATION Proposed Mixed-Use Building 8825 Washington Boulevard Pico Rivera, California	DRAWN BY: KV	APPROVED BY: CJ	B-1 Soil Boring Locations P-1 Percolation Locations
	PROJECT NO. 3-220-0499	FIGURE NO. 2	All Locations Approximate

SALEM engineering group, inc.

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation (drilling) was conducted on July 7 and 8, 2020 and included a site visit, subsurface exploration, and soil sampling. The locations of the exploratory borings are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

In general, our borings were performed using a truck-mounted CME 55 drill rig equipped with 6-inch solid flight augers. Sampling in the borings was accomplished using a hydraulic 140-pound hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT "N" values; corrections have not been applied. Upon completion, the borings were backfilled with drill cuttings.

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

SALEM engineering group, inc.

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

Project Number: 3-220-0499

Date: 07/07/2020

Client: Optimus Properties, Inc.

Project: Proposed Mixed-Use Building

Location: 8825 Washington Boulevard, Pico Rivera, California

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

Drill Type: CME 55 **Elevation:** 165'

Auger Type: 6 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
165 — 0		AC	Asphalt Concrete w/Petromat = 4" *No Aggregate Base				
	3/6 3/6 3/6	SM	Silty SAND Loose; moist; light brown; fine grain sand.	6	3.0	103.8	
160 — 5	2/6 3/6 4/6	ML	Sandy SILT Firm; very moist; dark brown; fine grain sand; trace clay.	7	22.4	90.3	
155 — 10	6/6 9/6 13/6	SP	Poorly graded SAND Medium dense; damp; light brown; fine to medium grain sand.	22	1.3	99.2	Cu=2.50 Cc=1.23
150 — 15	5/6 8/6 11/6		Grades as above; slightly moist; olive brown.	19	3.5	-	
145 — 20	8/6 14/6 15/6		Grades as above.	29	3.3	-	Cu=3.00 Cc=1.33
140 — 25	8/6 15/6 16/6	SW	Well-graded SAND Dense; moist; light brown; fine to coarse grain sand; trace gravel.	31	1.9	-	Cu=6.00 Cc=1.31

Notes:

Figure Number A-1

SALEM engineering group, inc.

Project Number: 3-220-0499
Date: 07/07/2020
Test Boring: B-1

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
135 / 30	7/6 15/6 20/6	SM	Silty SAND Dense; moist; light brown; fine to coarse grain sand.	35	3.5	-	
130 / 35	3/6 7/6 7/6	CL	Sandy CLAY Stiff; moist; dark gray; fine grain sand.	14	30.2	-	
125 / 40	9/6 16/6 21/6	SM	Silty SAND Dense; moist; dark brown; fine to medium grain sand.	37	8.8	-	
120 / 45	18/6 23/6 27/6		Grades as above; very dense; slightly moist; brown.	50	2.2	-	
115 / 50	6/6 28/6 50/1		Grades as above.	78/7"	4.9	-	
			End of boring at 51.5 feet BSG.				
110 / 55							
105 / 60							

Notes:

Figure Number A-1

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

SALEM engineering group, inc.

Project Number: 3-220-0499

Date: 07/07/2020

Client: Optimus Properties, Inc.

Project: Proposed Mixed-Use Building

Location: 8825 Washington Boulevard, Pico Rivera, California

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

Drill Type: CME 55 **Elevation:** 164'

Auger Type: 6 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
164 / 0		AC	Asphalt Concrete w/Petromat = 4"				
		AB	Aggregate Base = 3 in.				
	2/6 3/6 3/6	ML	Sandy SILT Firm; moist; dark gray; fine grain sand; trace clay.	6	16.8	88.3	
160 / 5	2/6 4/6 6/6		Grades as above; with clay.	10	27.9	85.7	
155 / 10	4/6 7/6 8/6	SP	Poorly graded SAND Medium dense; slightly moist; light brown; fine to medium grain sand.	15	1.8	-	
150 / 15	6/6 10/6 10/6		Grades as above.	20	1.8	-	
145 / 20	13/6 23/6 20/6		Grades as above; dense; damp; olive.	43	1.0	-	
140 / 25	11/6 14/6 23/6		Grades as above; slightly moist.	37	2.3	-	
			End of boring at 26.5 feet BSG.				

Notes:

Figure Number A-2

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

SALEM engineering group, inc.

Project Number: 3-220-0499

Date: 07/07/2020

Client: Optimus Properties, Inc.

Project: Proposed Mixed-Use Building

Location: 8825 Washington Boulevard, Pico Rivera, California

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

Drill Type: CME 55 **Elevation:** 163'

Auger Type: 6 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete w/Petromat = 4"				
		AB	Aggregate Base = 3 in.				
160	3/6 4/6 8/6	SM	Silty SAND Loose; moist; olive; fine to coarse grain sand.	12	6.2	100.2	
5	3/6 5/6 6/6		Grades as above; with clay.	11	13.5	89.7	
155							
10	4/6 7/6 7/6	SP	Poorly graded SAND Medium dense; slightly moist; light brown; fine to medium grain sand.	14	1.3	-	
150							
15	4/6 6/6 10/6		Grades as above.	16	1.3	-	
145							
20	5/6 13/6 22/6		Grades as above.	35	1.3	109.7	Cu=2.86 Cc=1.40
140							
25	8/6 13/6 15/6		Grades as above.	28	1.5	-	
			End of boring at 26.5 feet BSG.				
135							

Notes:

Figure Number A-3

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

Project Number: 3-220-0499

Date: 07/07/2020

Client: Optimus Properties, Inc.

Project: Proposed Mixed-Use Building

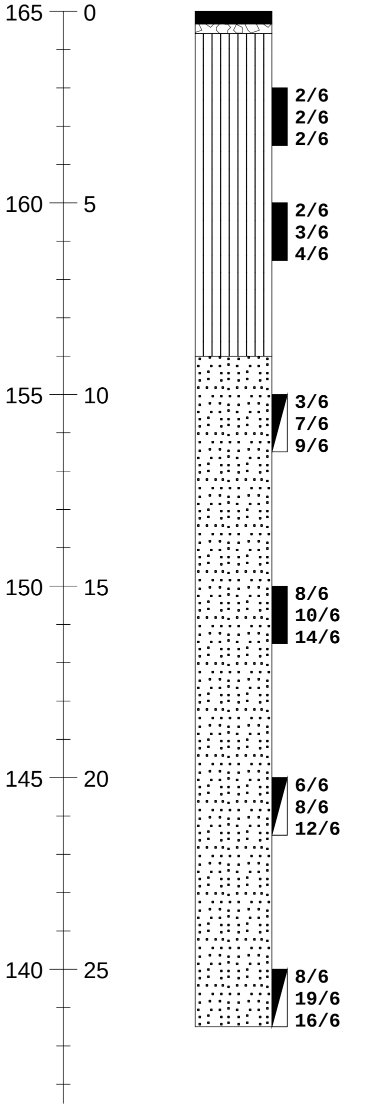
Location: 8825 Washington Boulevard, Pico Rivera, California

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

Drill Type: CME 55 **Elevation:** 165'

Auger Type: 6 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
165 / 0		AC	Asphalt Concrete w/Petromat = 4"				
		AB	Aggregate Base = 3 in.				
	2/6 2/6 2/6	ML	Sandy SILT Soft; moist; dark brown; fine grain sand; with clay.	4	25.5	79.8	
160 / 5	2/6 3/6 4/6		Grades as above; firm; trace clay.	7	17.6	93.4	
155 / 10	3/6 7/6 9/6	SP	Poorly graded SAND Medium dense; slightly moist; light brown; fine to medium grain sand.	16	1.7	-	
150 / 15	8/6 10/6 14/6		Grades as above.	24	2.0	94.6	Cu=2.33 Cc=0.76
145 / 20	6/6 8/6 12/6		Grades as above.	20	1.9	-	
140 / 25	8/6 19/6 16/6		Grades as above; dense.	35	2.1	-	
			End of boring at 26.5 feet BSG.				

Notes:

Figure Number A-4

Test Boring: B-5 **Page** 1 **Of:** 2

SALEM engineering group, inc.

Project Number: 3-220-0499

Date: 07/07/2020

Client: Optimus Properties, Inc.

Project: Proposed Mixed-Use Building

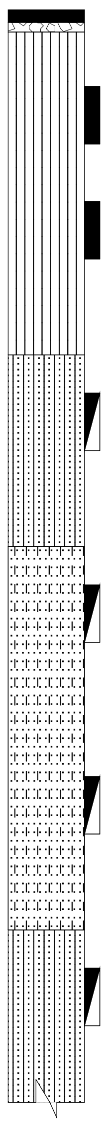
Location: 8825 Washington Boulevard, Pico Rivera, California

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

Drill Type: CME 55 **Elevation:** 162'

Auger Type: 6 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete w/Petromat = 4"				
		AB	Aggregate Base = 3 in.				
160	2/6 3/6 4/6	ML	SILT with Sand Firm; very moist; dark brown; fine grain sand.	7	21.1	79.6	
5 / 155	2/6 3/6 3/6	ML	SILT Firm; wet; dark gray; fine grain sand.	6	41.2	77.5	
10 / 150	3/6 6/6 8/6	SM	Silty SAND Medium dense; moist; light brown; fine to coarse grain sand.	14	3.7	-	
15 / 145	4/6 8/6 10/6	SW-SM	Well-graded SAND with Silt Medium dense; slightly moist; brown; fine to coarse grain sand.	18	2.0	-	
20 / 140	6/6 9/6 9/6		Grades as above.	18	3.8	-	
25 / 135	7/6 11/6 9/6	SM	Silty SAND Medium dense; slightly moist; light brown; fine to medium grain sand.	20	3.1	-	

Notes:

Figure Number A-5

SALEM engineering group, inc.

Project Number: 3-220-0499

Date: 07/07/2020

Test Boring: B-5

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
30 / 130	6/6 13/6 18/6		Grades as above; dense.	31	3.9	-	
35 / 125	4/6 5/6 7/6		Grades as above; medium dense; moist; brown.	12	6.0	-	
40 / 120	8/6 14/6 20/6		Grades as above; dense.	34	9.9	-	
45 / 115	14/6 23/6 23/6		Grades as above; light brown.	46	3.5	-	
50 / 110	27/6 50/1 -		Grades as above; very dense.	50/1"	4.0	-	
			End of boring at 51.5 feet BSG.				
55 / 105							
60 / 100							

Notes:

Figure Number A-5

SALEM engineering group, inc.

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

Project Number: 3-220-0499

Date: 07/07/2020

Client: Optimus Properties, Inc.

Project: Proposed Mixed-Use Building

Location: 8825 Washington Boulevard, Pico Rivera, California

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

Drill Type: CME 55 **Elevation:** 165'

Auger Type: 6 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
165 — 0		AC	Asphalt Concrete w/Petromat = 4"				
		AB	Aggregate Base = 3 in.				
	2/6 1/6 1/6	SM	Silty SAND Very loose; moist; light brown; fine grain sand.	2	8.0	89.8	
160 — 5	3/6 3/6 4/6		Grades as above; loose; brown.	7	12.4	90.7	
155 — 10	3/6 5/6 5/6	SP-SM	Poorly graded SAND with Silt Loose; slightly moist; light brown; fine to medium grain sand.	10	1.7	-	
150 — 15	11/6 18/6 23/6		Grades as above; dense.	41	1.5	99.1	Cu=3.00 Cc=0.93
145 — 20	8/6 17/6 22/6		Grades as above; damp; with gravel.	39	1.0	-	
140 — 25	12/6 14/6 13/6		Grades as above; medium dense; slightly moist.	27	2.2	-	
			End of boring at 26.5 feet BSG.				

Notes:

Figure Number A-6

KEY TO SYMBOLS

Symbol Description

Strata symbols

Asphaltic Concrete

Silty sand

Silt

Poorly graded sand

Well graded sand

Lean Clay

Aggregate Base

Well graded sand with silt

Poorly graded sand with silt

Misc. Symbols

Boring continues

Soil Samplers

California sampler

Symbol Description

Standard penetration test

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: Proposed Mixed-Use Building
8825 Washington Boulevard
Pico Rivera, California

Job No.: 3-220-0499
Date Drilled: 7/7/2020
Soil Classification: SAND (SP)

Vol. in 1" Wtr Col. (in^3): 28.3

Hole Dia.: 6 in.
Pipe Dia.: 3 in.

Test Hole No.: P-1
Tested by: EGR
Drilled Hole Depth: 14.5 ft.

Presoaking Date: 7/7/2020
Test Date: 7/8/2020

Pipe stickup: 0.5 ft

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Volume of Water Discharged (in^3)	Test Area (sidewalls & bottom) (in^2)		Measured Perc Rate (in/hr)
8:00	8:10	15.0	Y	0:10	14.00	14.98	11.76	10	332.51	143.6		13.89
8:10	8:11	15.0	N	0:01	14.98	drained		1				
water reamined in the hole after 10 minutes but drained before 30 minutes, the time interval between readings is 10 minutes												
8:12	8:22	15.0	Y	0:10	13.00	14.85	22.20	10	627.69	271.4		13.88
8:23	8:33	15.0	Y	0:10	13.00	14.80	21.60	10	610.73	277.1		13.22
8:34	8:44	15.0	Y	0:10	13.00	14.75	21.00	10	593.76	282.7		12.60
8:45	8:55	15.0	Y	0:10	13.00	14.75	21.00	10	593.76	282.7		12.60
8:56	9:06	15.0	Y	0:10	13.00	14.70	20.40	10	576.80	288.4		12.00
9:07	9:17	15.0	Y	0:10	13.00	14.70	20.40	10	576.80	288.4		12.00
9:18	9:28	15.0	Y	0:10	13.00	14.70	20.40	10	576.80	288.4		12.00
9:29	9:39	15.0	Y	0:10	13.00	14.70	20.40	10	576.80	288.4		12.00
Recommended for Design:								**Percolation Rate***	**576.80**			**12.00**

* Average of last 3 readings

Percolation Test Worksheet

Project: Proposed Mixed-Use Building
8825 Washington Boulevard
Pico Rivera, California

Job No.: 3-220-0499
Date Drilled: 7/7/2020
Soil Classification: SAND with Silt (SW-SM)

Vol. in 1" Wtr Col. (in^3): 28.3
Hole Dia.: 6 in.
Pipe Dia.: 3 in.

Test Hole No.: P-2
Tested by: EGR
Drilled Hole Depth: 14.4 ft.

Presoaking Date: 7/7/2020
Test Date: 7/8/2020

Pipe stickup: 0.3 ft

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill-Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Volume of Water Discharged (in^3)	Test Area (sidewalls & bottom) (in^2)		Measured Perc Rate (in/hr)
9:45	9:55	14.7	Y	0:10	13.70	14.65	11.40	10	322.33	147.0		13.15
9:55	9:58	14.7	N	0:03	14.65	drained		3				
water reamined in the hole after 10 minutes but drained before 30 minutes, the time interval between readings is 10 minutes												
10:00	10:10	14.7	Y	0:10	13.00	14.50	18.00	10	508.94	243.2		12.56
10:11	10:21	14.7	Y	0:10	13.00	14.45	17.40	10	491.97	248.8		11.86
10:22	10:32	14.7	Y	0:10	13.00	14.45	17.40	10	491.97	248.8		11.86
10:33	10:43	14.7	Y	0:10	13.00	14.40	16.80	10	475.01	254.5		11.20
10:44	10:54	14.7	Y	0:10	13.00	14.40	16.80	10	475.01	254.5		11.20
10:55	11:05	14.7	Y	0:10	13.00	14.40	16.80	10	475.01	254.5		11.20
11:06	11:16	14.7	Y	0:10	13.00	14.40	16.80	10	475.01	254.5		11.20
11:17	11:27	14.7	Y	0:10	13.00	14.40	16.80	10	475.01	254.5		11.20
Recommended for Design:								**Percolation Rate***	**480.66**			**11.20**

* Average of last 3 readings

Percolation Test Worksheet

Project: Proposed Mixed-Use Building
8825 Washington Boulevard
Pico Rivera, California

Job No.: 3-220-0499
Date Drilled: 7/7/2020
Soil Classification: SAND (SP)

Vol. in 1" Wtr Col. (in^3): 28.3

Hole Dia.: 6 in.
Pipe Dia.: 3 in.

Test Hole No.: P-2
Tested by: EGR
Drilled Hole Depth: 14.4 ft.

Presoaking Date: 7/7/2020
Test Date: 7/8/2020

Pipe stickup: 0.6 ft

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill-Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Volume of Water Discharged (in^3)	Test Area (sidewalls & bottom) (in^2)		Measured Perc Rate (in/hr)
11:35	11:45	15.0	Y	0:10	14.00	14.95	11.40	10	322.33	147.0		13.15
11:45	11:47	15.0	N	0:02	14.95	drained		2				
water reamined in the hole after 10 minutes but drained before 30 minutes, the time interval between readings is 10 minutes												
11:48	11:58	15.0	Y	0:10	13.00	14.80	21.60	10	610.73	277.1		13.22
11:59	12:09	15.0	Y	0:10	13.00	14.75	21.00	10	593.76	282.7		12.60
12:10	12:20	15.0	Y	0:10	13.00	14.70	20.40	10	576.80	288.4		12.00
12:21	12:31	15.0	Y	0:10	13.00	14.70	20.40	10	576.80	288.4		12.00
12:32	12:42	15.0	Y	0:10	13.00	14.65	19.80	10	559.83	294.1		11.42
12:43	12:53	15.0	Y	0:10	13.00	14.65	19.80	10	559.83	294.1		11.42
12:54	13:04	15.0	Y	0:10	13.00	14.65	19.80	10	559.83	294.1		11.42
13:05	13:15	15.0	Y	0:10	13.00	14.65	19.80	10	559.83	294.1		11.42
Recommended for Design:							**Percolation Rate***		**571.14**			**11.81**

* Average of last 3 readings

Salem Engineering Group, Inc.
8711 Monroe Court, Suite A
Rancho Cucamonga, CA 91730
(909) 980-6455

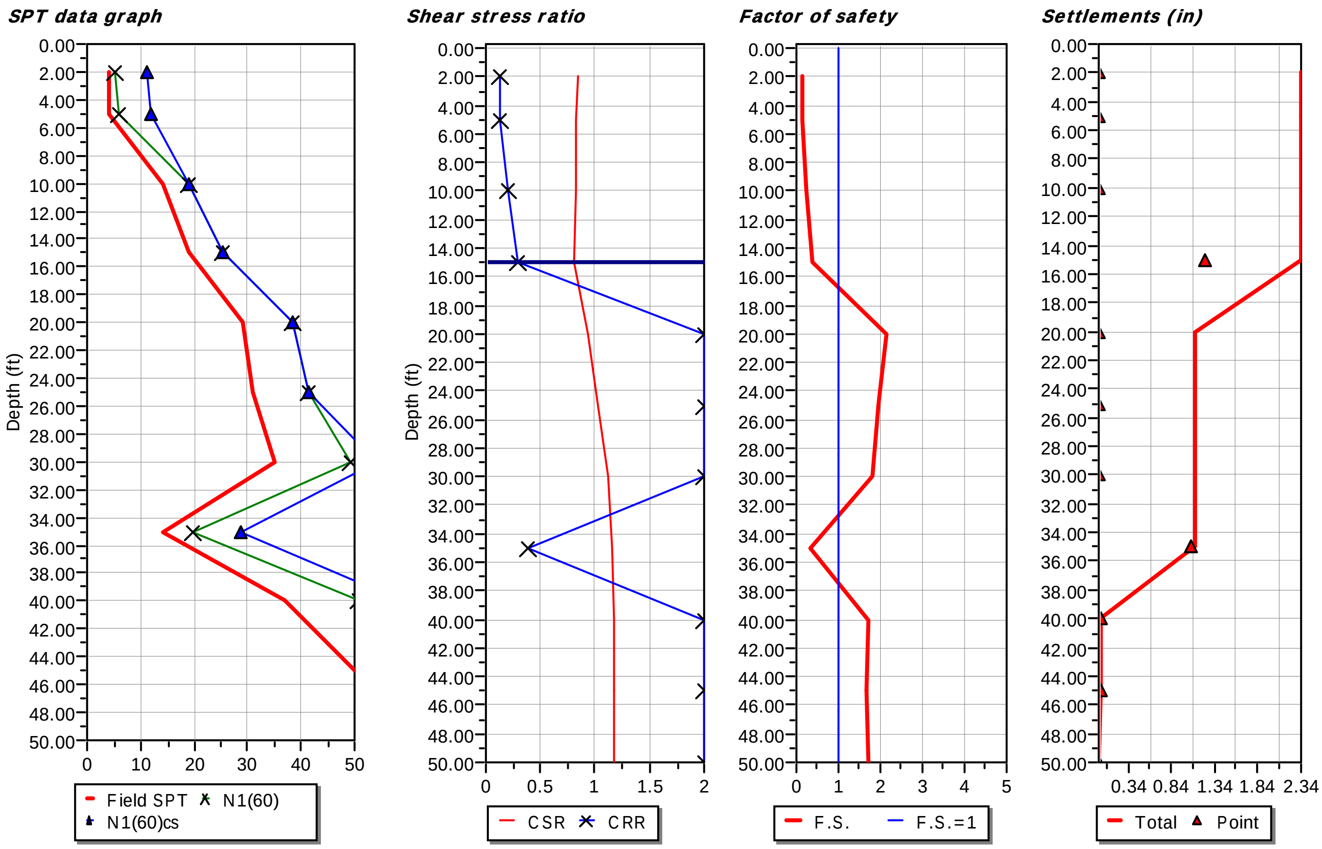
LIQUEFACTION ANALYSIS REPORT

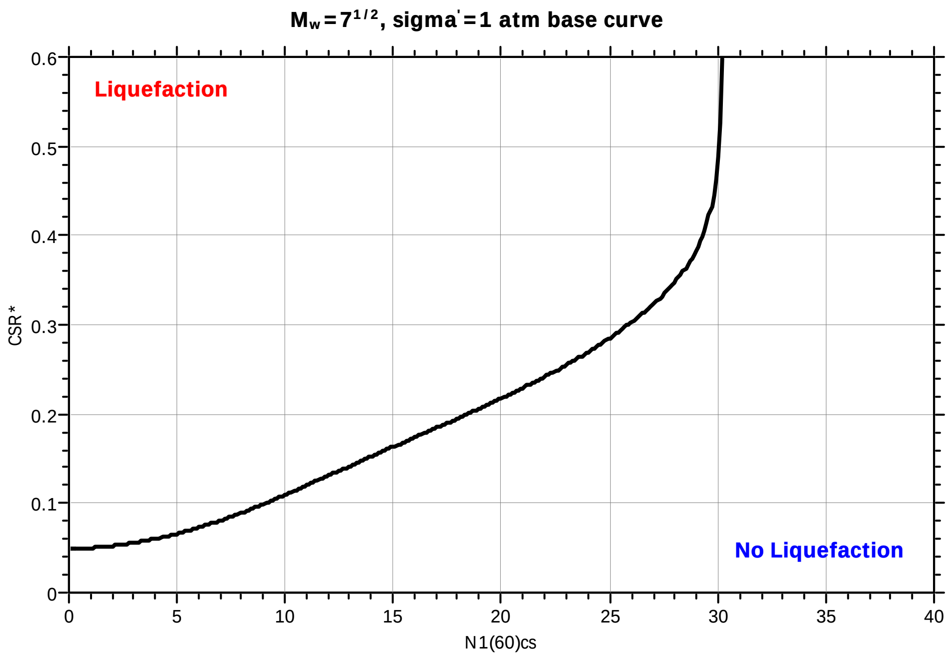
Project title : 3-220-0499

Project subtitle : B-1

Input parameters and analysis data

In-situ data type:	Standard Penetration Test	Depth to water table:	15.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.90
Analysis method:	NCEER 1998	Peak ground accelaration:	0.87 g
Fines correction method:	Idriss & Seed	User defined F.S.:	1.30





:: Field input data ::

Point ID	Depth (ft)	Field N_{SPT} (blows/feet)	Unit weight (pcf)	Fines content (%)
1	2.00	4.00	110.00	40.00
2	5.00	4.00	110.00	65.00
3	10.00	14.00	110.00	2.00
4	15.00	19.00	110.00	4.00
5	20.00	29.00	110.00	4.00
6	25.00	31.00	110.00	4.00
7	30.00	35.00	110.00	15.00
8	35.00	14.00	110.00	70.00
9	40.00	37.00	110.00	20.00
10	45.00	50.00	110.00	15.00
11	50.00	78.00	110.00	13.00

Depth : Depth from free surface, at which SPT was performed (ft)
Field SPT : SPT blows measured at field (blows/feet)
Unit weight : Bulk unit weight of soil at test depth (pcf)
Fines content : Percentage of fines in soil (%)

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*
1	2.00	0.11	0.00	0.11	1.00	0.56	0.88	0.64	1.00	0.64
2	5.00	0.28	0.00	0.28	0.99	0.56	0.88	0.64	1.00	0.64
3	10.00	0.55	0.00	0.55	0.98	0.55	0.88	0.63	1.00	0.63
4	15.00	0.83	0.00	0.83	0.97	0.55	0.88	0.62	1.00	0.62
5	20.00	1.10	0.16	0.94	0.95	0.63	0.88	0.72	1.00	0.72
6	25.00	1.38	0.31	1.06	0.94	0.69	0.88	0.79	1.00	0.79
7	30.00	1.65	0.47	1.18	0.93	0.73	0.88	0.84	0.98	0.86
8	35.00	1.93	0.62	1.30	0.89	0.75	0.88	0.85	0.96	0.89
9	40.00	2.20	0.78	1.42	0.85	0.74	0.88	0.85	0.94	0.90
10	45.00	2.48	0.94	1.54	0.81	0.74	0.88	0.84	0.93	0.91
11	50.00	2.75	1.09	1.66	0.77	0.72	0.88	0.82	0.91	0.90

Depth : Depth from free surface, at which SPT was performed (ft)
Sigma : Total overburden pressure at test point, during earthquake (tsf)
u : Water pressure at test point, during earthquake (tsf)
Sigma' : Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$ CSR adjusted for M=7.5
K_{sigma} Effective overburden stress factor
CSR^* CSR fully adjusted

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
1	4.00	1.70	0.86	1.00	0.75	1.20	5.27	6.05	11.32	0.12
2	4.00	1.70	0.90	1.00	0.80	1.20	5.89	6.18	12.07	0.13
3	14.00	1.38	0.97	1.00	0.85	1.20	19.12	0.00	19.12	0.21
4	19.00	1.13	1.04	1.00	0.95	1.20	25.37	0.00	25.37	0.29
5	29.00	1.05	1.11	1.00	0.95	1.20	38.61	0.00	38.61	2.00
6	31.00	0.99	1.18	1.00	0.95	1.20	41.32	0.00	41.32	2.00
7	35.00	0.94	1.25	1.00	1.00	1.20	49.31	4.87	54.18	2.00
8	14.00	0.90	1.32	1.00	1.00	1.20	19.84	8.97	28.81	0.38
9	37.00	0.86	1.33	1.00	1.00	1.20	50.78	7.65	58.42	2.00
10	50.00	0.82	1.33	1.00	1.00	1.20	65.91	5.67	71.58	2.00
11	78.00	0.79	1.33	1.00	1.00	1.20	99.06	5.54	104.61	2.00

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$

C_n : Overburden corretion factor
C_e : Energy correction factor
C_b : Borehole diameter correction factor
C_r : Rod length correction factor
C_s : Liner correction factor
$N_{1(60)}$: Corrected N_{SPT}
DeltaN : Addition to corrected N_{SPT} value due to the presence of fines
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines
$CRR_{7.5)}$: Cyclic resistance ratio for M=7.5

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
1	11.32	9.44	0.15	3.63	0.00
2	12.07	10.06	0.16	3.49	0.00
3	19.12	15.94	0.25	2.59	0.00
4	25.37	21.14	0.36	2.05	1.23
5	38.61	32.18	2.14	0.00	0.00
6	41.32	34.44	1.95	0.00	0.00
7	54.18	45.15	1.79	0.01	0.01
8	28.81	24.01	0.32	1.79	1.07
9	58.42	48.69	1.70	0.02	0.01
10	71.58	59.65	1.69	0.02	0.01
11	104.61	87.17	1.70	0.02	0.01

Total settlement : 2.34

$N_{1,(60)}$: Stress normalized and corrected SPT blow count
N_1: Japanese equivalent corrected value
FS_L: Calculated factor of safety
e_v: Post-liquefaction volumentric strain (%)
Settle.: Calculated settlement (in)

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L
1	0.85	9.70	5.04
2	0.84	9.24	7.11
3	0.75	8.48	9.65
4	0.64	7.71	7.53
5	0.00	6.95	0.00
6	0.00	6.19	0.00
7	0.00	5.43	0.00
8	0.68	4.67	4.80
9	0.00	3.90	0.00
10	0.00	3.14	0.00
11	0.00	2.38	0.00

Overall potential I_L : 34.12

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

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LIQUEFACTION ANALYSIS REPORT

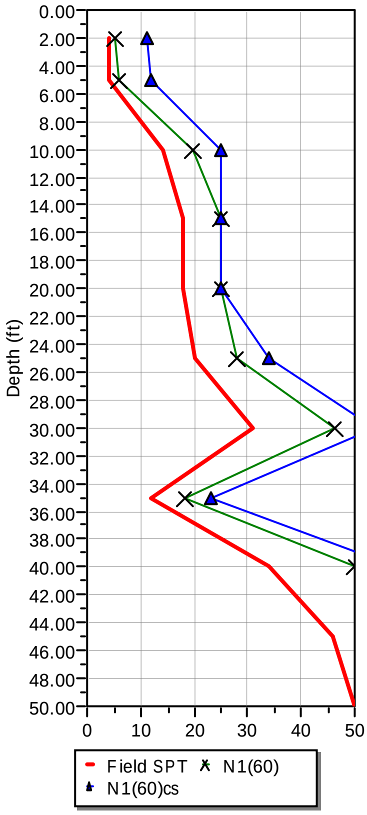
Project title : 3-220-0499

Project subtitle : B-5

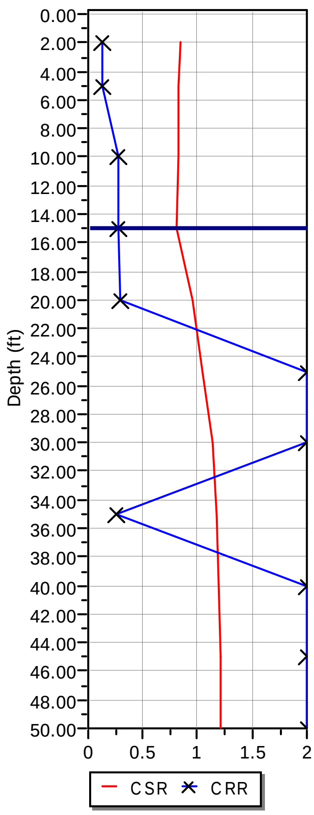
Input parameters and analysis data

In-situ data type:	Standard Penetration Test	Depth to water table:	15.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.90
Analysis method:	NCEER 1998	Peak ground accelaration:	0.87 g
Fines correction method:	Idriss & Seed	User defined F.S.:	1.30

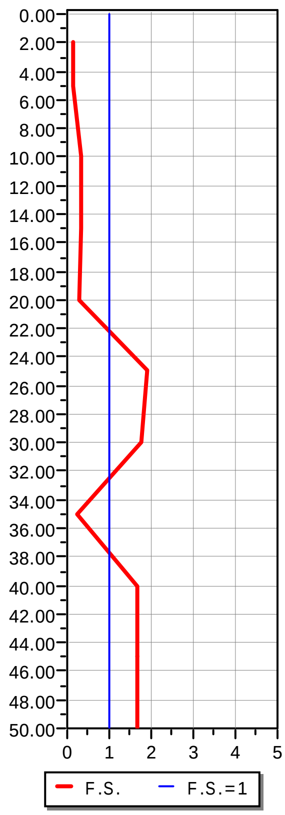
SPT data graph



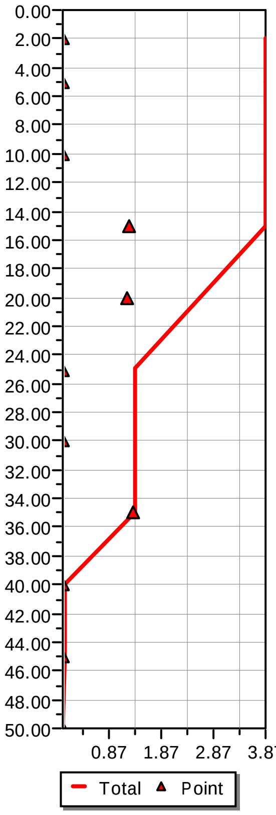
Shear stress ratio



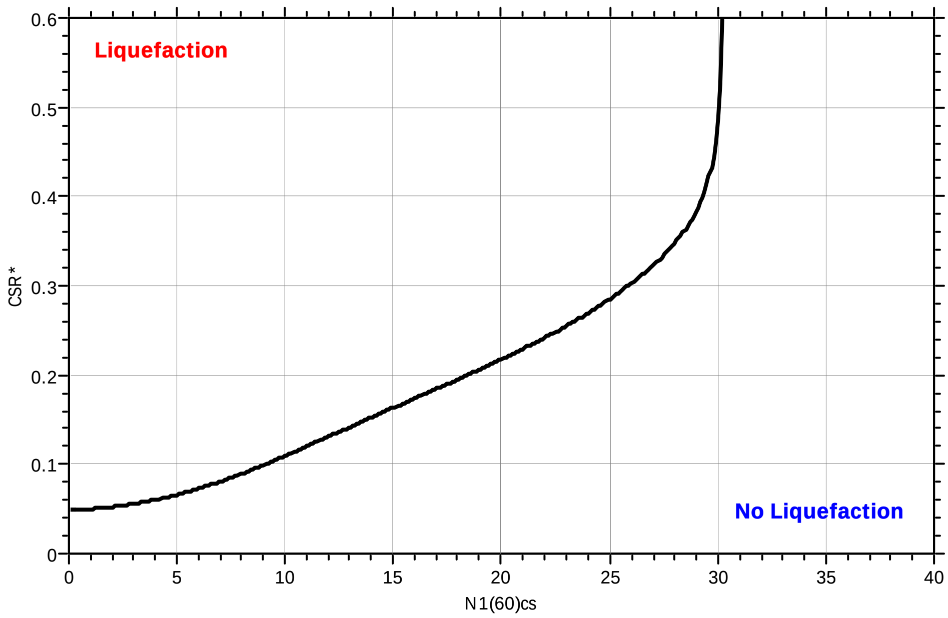
Factor of safety



Settlements (in)



$M_w = 7^{1/2}$, sigma' = 1 atm base curve



:: Field input data ::

Point ID	Depth (ft)	Field N_{SPT} (blows/feet)	Unit weight (pcf)	Fines content (%)
1	2.00	4.00	100.00	78.00
2	5.00	4.00	110.00	70.00
3	10.00	14.00	100.00	20.00
4	15.00	18.00	100.00	5.00
5	20.00	18.00	100.00	5.00
6	25.00	20.00	100.00	20.00
7	30.00	31.00	100.00	20.00
8	35.00	12.00	100.00	20.00
9	40.00	34.00	100.00	20.00
10	45.00	46.00	100.00	20.00
11	50.00	50.00	100.00	20.00

Depth : Depth from free surface, at which SPT was performed (ft)
Field SPT : SPT blows measured at field (blows/feet)
Unit weight : Bulk unit weight of soil at test depth (pcf)
Fines content : Percentage of fines in soil (%)

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*
1	2.00	0.10	0.00	0.10	1.00	0.56	0.88	0.64	1.00	0.64
2	5.00	0.27	0.00	0.27	0.99	0.56	0.88	0.64	1.00	0.64
3	10.00	0.52	0.00	0.52	0.98	0.55	0.88	0.63	1.00	0.63
4	15.00	0.77	0.00	0.77	0.97	0.55	0.88	0.62	1.00	0.62
5	20.00	1.02	0.16	0.86	0.95	0.64	0.88	0.73	1.00	0.73
6	25.00	1.27	0.31	0.95	0.94	0.71	0.88	0.81	1.00	0.81
7	30.00	1.52	0.47	1.05	0.93	0.76	0.88	0.87	1.00	0.87
8	35.00	1.77	0.62	1.14	0.89	0.78	0.88	0.89	0.98	0.91
9	40.00	2.02	0.78	1.23	0.85	0.78	0.88	0.90	0.97	0.93
10	45.00	2.27	0.94	1.33	0.81	0.78	0.88	0.89	0.95	0.94
11	50.00	2.52	1.09	1.42	0.77	0.77	0.88	0.88	0.94	0.93

Depth : Depth from free surface, at which SPT was performed (ft)
Sigma : Total overburden pressure at test point, during earthquake (tsf)
u : Water pressure at test point, during earthquake (tsf)
Sigma' : Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$ CSR adjusted for M=7.5
K_{sigma} Effective overburden stress factor
CSR^* CSR fully adjusted

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
1	4.00	1.70	0.86	1.00	0.75	1.20	5.27	6.05	11.32	0.12
2	4.00	1.70	0.90	1.00	0.80	1.20	5.89	6.18	12.07	0.13
3	14.00	1.42	0.97	1.00	0.85	1.20	19.76	5.18	24.95	0.28
4	18.00	1.17	1.04	1.00	0.95	1.20	24.96	0.00	24.96	0.28
5	18.00	1.10	1.11	1.00	0.95	1.20	25.12	0.00	25.12	0.29
6	20.00	1.05	1.18	1.00	0.95	1.20	28.16	5.85	34.01	2.00
7	31.00	1.00	1.25	1.00	1.00	1.20	46.41	7.30	53.71	2.00
8	12.00	0.96	1.32	1.00	1.00	1.20	18.16	5.06	23.22	0.26
9	34.00	0.92	1.33	1.00	1.00	1.20	50.03	7.59	57.62	2.00
10	46.00	0.89	1.33	1.00	1.00	1.20	65.26	8.80	74.05	2.00
11	50.00	0.86	1.33	1.00	1.00	1.20	68.55	9.06	77.61	2.00

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$

C_n : Overburden corretion factor
C_e : Energy correction factor
C_b : Borehole diameter correction factor
C_r : Rod length correction factor
C_s : Liner correction factor
$N_{1(60)}$: Corrected N_{SPT}
DeltaN : Addition to corrected N_{SPT} value due to the presence of fines
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines
$CRR_{7.5)}$: Cyclic resistance ratio for M=7.5

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
1	11.32	9.44	0.15	3.63	0.00
2	12.07	10.06	0.16	3.49	0.00
3	24.95	20.79	0.35	2.08	0.00
4	24.96	20.80	0.35	2.08	1.25
5	25.12	20.94	0.30	2.07	1.24
6	34.01	28.34	1.90	0.01	0.00
7	53.71	44.76	1.77	0.01	0.01
8	23.22	19.35	0.22	2.22	1.33
9	57.62	48.02	1.66	0.02	0.01
10	74.05	61.71	1.65	0.02	0.01
11	77.61	64.67	1.65	0.02	0.01

Total settlement : 3.87

$N_{1,(60)}$: Stress normalized and corrected SPT blow count
N_1: Japanese equivalent corrected value
FS_L: Calculated factor of safety
e_v: Post-liquefaction volumentric strain (%)
Settle.: Calculated settlement (in)

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L
1	0.85	9.70	5.04
2	0.84	9.24	7.11
3	0.65	8.48	8.44
4	0.65	7.71	7.63
5	0.70	6.95	7.38
6	0.00	6.19	0.00
7	0.00	5.43	0.00
8	0.78	4.67	5.55
9	0.00	3.90	0.00
10	0.00	3.14	0.00
11	0.00	2.38	0.00

Overall potential I_L : 41.15

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

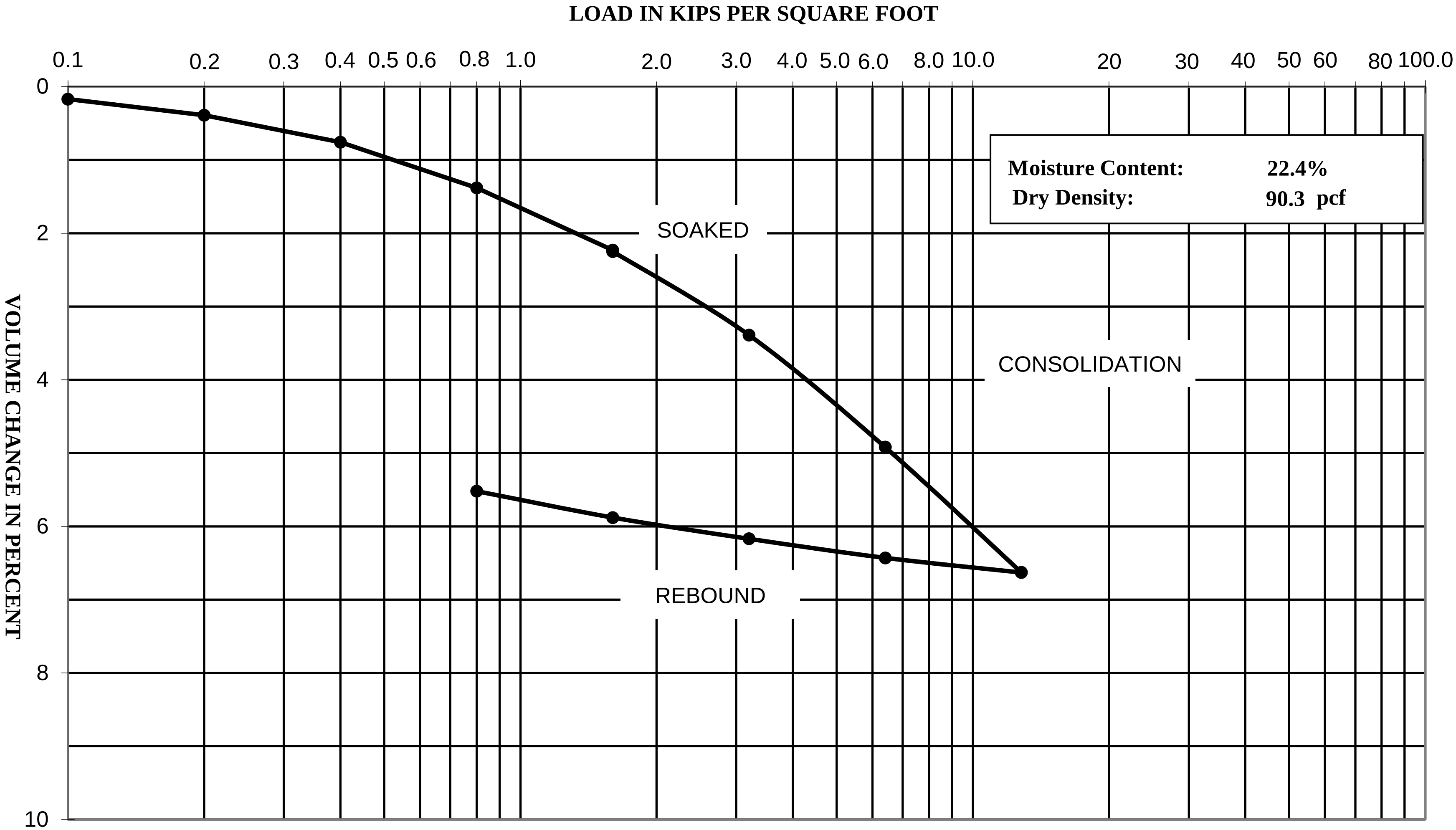
APPENDIX

B

APPENDIX B
LABORATORY TESTING

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

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

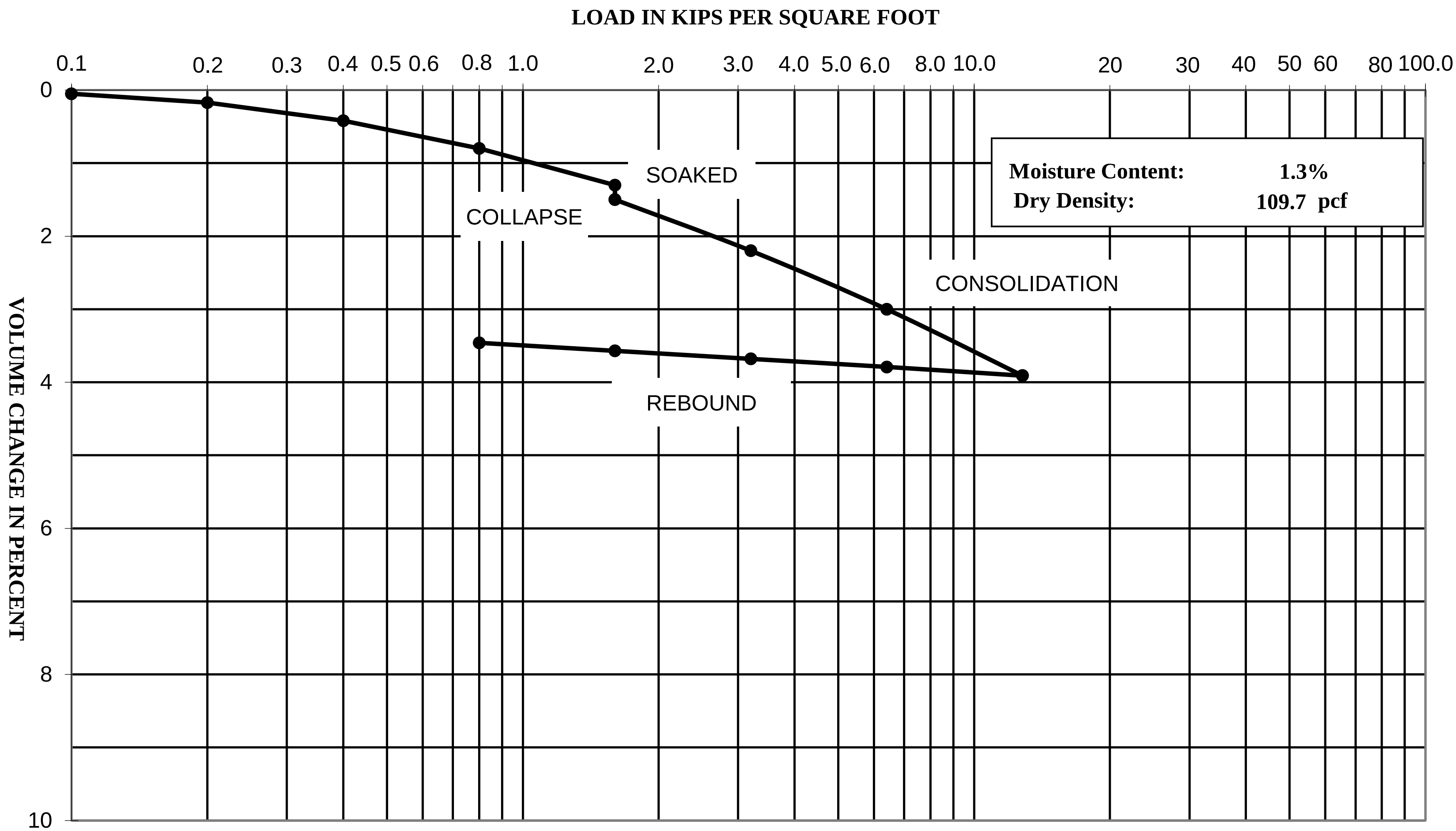
Project Number: 3-220-0499

Boring: B-1 @ 5'

SALEM engineering group, inc.

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435



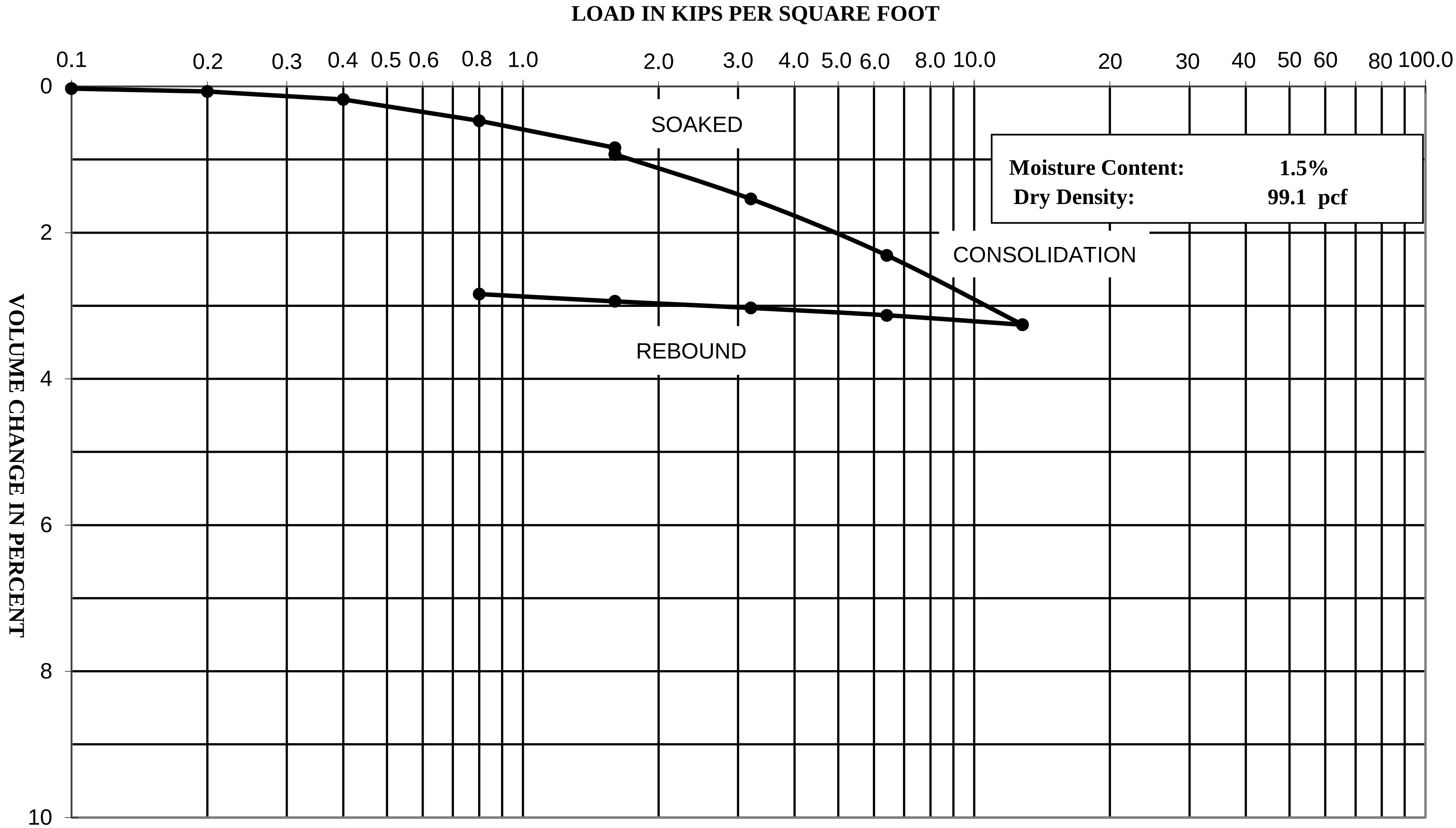
Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

Project Number: 3-220-0499

Boring: B-3 @ 20'

SALEM engineering group, inc.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

Project Number: 3-220-0499

Boring: B-6 @ 15'

SALEM engineering group, inc.

Direct Shear Test (ASTM D3080)

Project Name:	Proposed Mixed-Use Building - Pico Rivera, CA
Project Number:	3-220-0499
Client:	Optimus Properties, Inc.
Sample Location:	B-1 @ 10'
Sample Type:	Undisturbed Ring
Soil Classification:	Poorly graded SAND (SP)
Tested By:	M. Noorzay
Reviewed By:	CJ
Date:	7/20/2020
Equipment Used:	Geomatic Direct Shear Machine

	Sample 1	Sample 2	Sample 3
Normal Stress (ksf)	1.000	2.000	3.000
Shear Rate (in/min)	0.004		
Peak Shear Stress (ksf)	0.865	1.560	2.316
Residual Shear Stress (ksf)	0.000	0.000	0.000

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in.)	1	1	1
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	1.3		
Final Moisture Content (%)	32.4	24.5	23.9
Dry Density (pcf)	91.1	94.2	94.4

Peak Shear Strength Values	
Slope	0.73
Friction Angle	36.0
Cohesion (psf)	129

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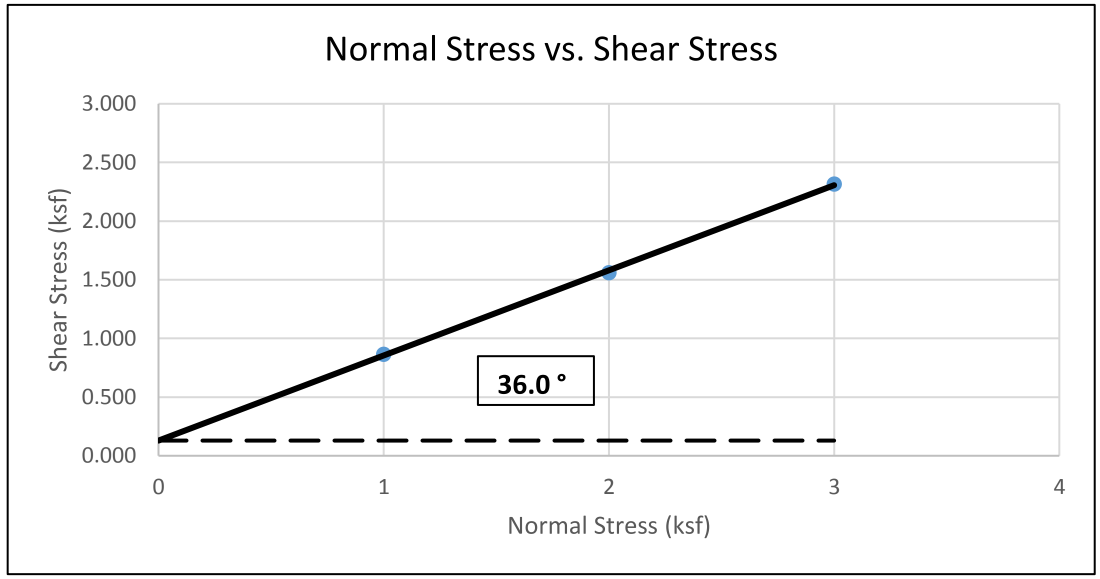
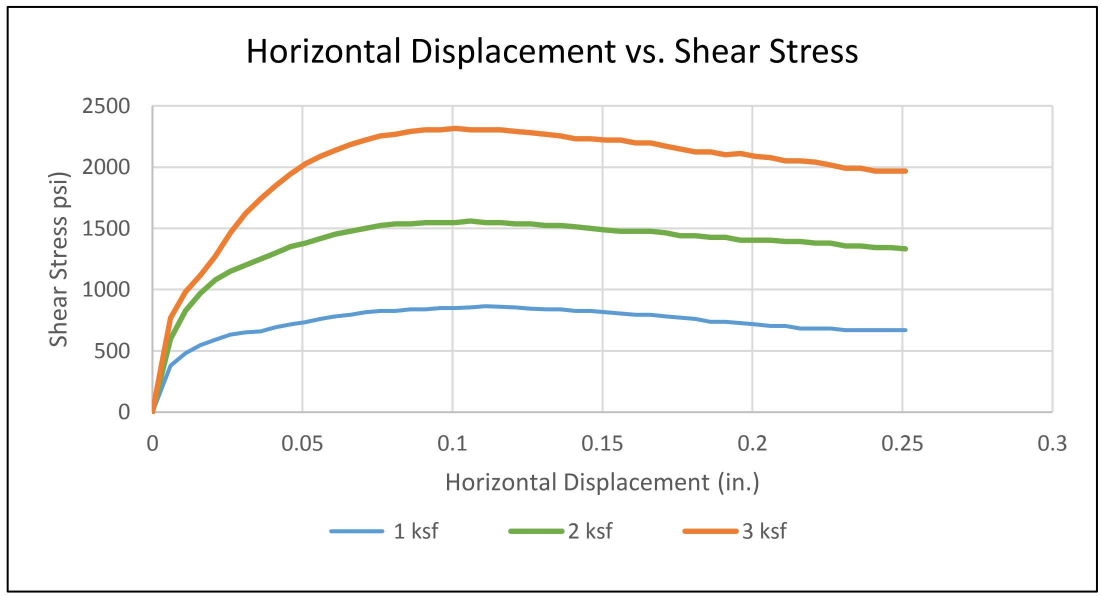
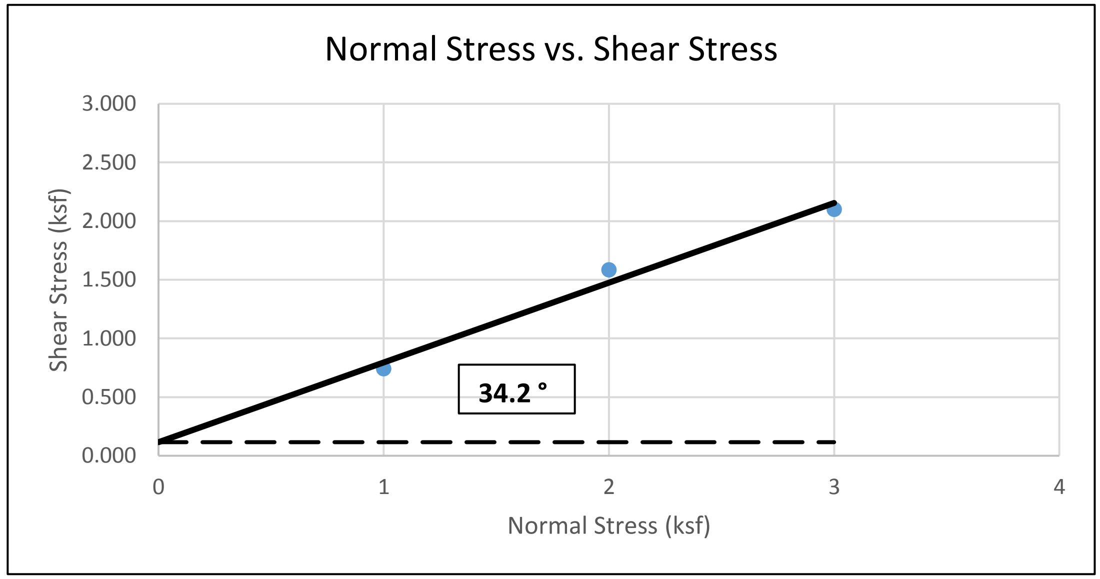




SALEM engineering group, inc.

Direct Shear Test (ASTM D3080)

Project Name:	Proposed Mixed-Use Building - Pico Rivera, CA
Project Number:	3-220-0499
Client:	Optimus Properties, Inc.
Sample Location:	B-4 @ 15'
Sample Type:	Undisturbed Ring
Soil Classification:	Poorly graded SAND (SP)
Tested By:	M. Noorzay
Reviewed By:	CJ
Date:	7/23/2020
Equipment Used:	Geomatic Direct Shear Machine



	Sample 1	Sample 2	Sample 3
Normal Stress (ksf)	1.000	2.000	3.000
Shear Rate (in/min)	0.004		
Peak Shear Stress (ksf)	0.741	1.584	2.100
Residual Shear Stress (ksf)	0.000	0.000	0.000

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in.)	1	1	1
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	2.0		
Final Moisture Content (%)	26.0	24.8	25.0
Dry Density (pcf)	93.3	93.2	94.1

Peak Shear Strength Values	
Slope	0.68
Friction Angle	34.2
Cohesion (psf)	116

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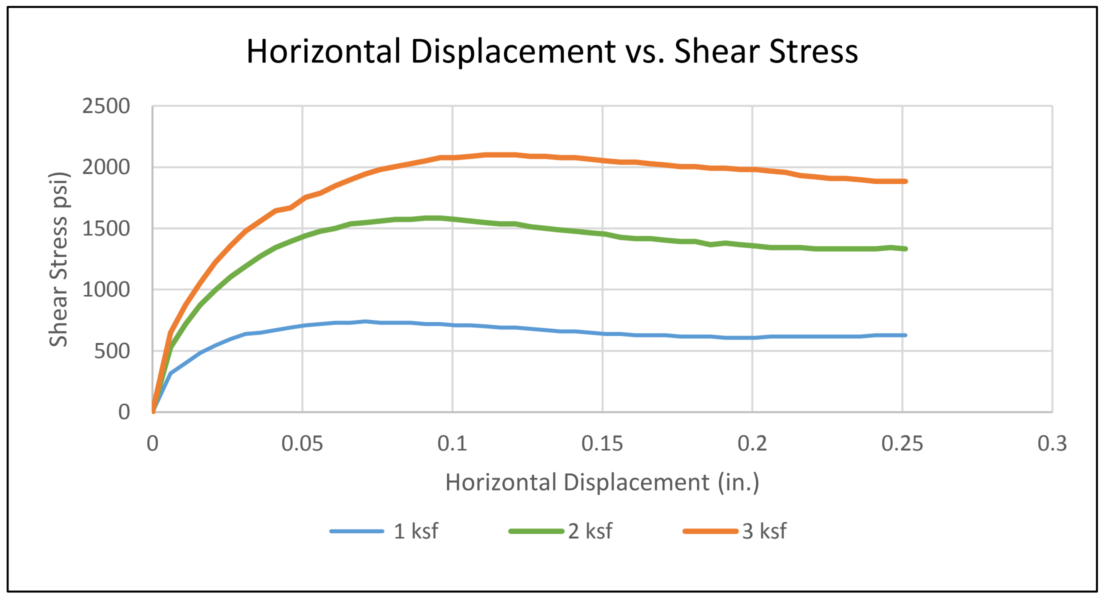

SALEM engineering group, inc.

Direct Shear Test (ASTM D3080)

Project Name:	Proposed Mixed-Use Building - Pico Rivera, CA
Project Number:	3-220-0499
Client:	Optimus Properties, Inc.
Sample Location:	B-5 @ 2'
Sample Type:	Undisturbed Ring
Soil Classification:	SILT with Sand (ML)
Tested By:	M. Noorzay
Reviewed By:	CJ
Date:	7/21/2020
Equipment Used:	Geomatic Direct Shear Machine

	Sample 1	Sample 2	Sample 3
Normal Stress (ksf)	1.000	2.000	3.000
Shear Rate (in/min)	0.004		
Peak Shear Stress (ksf)	0.774	1.332	1.920
Residual Shear Stress (ksf)	0.000	0.000	0.000

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in.)	1	1	1
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	20.3		
Final Moisture Content (%)	36.1	35.7	36.5
Dry Density (pcf)	80.3	80.5	79.3

Peak Shear Strength Values	
Slope	0.57
Friction Angle	29.8
Cohesion (psf)	196

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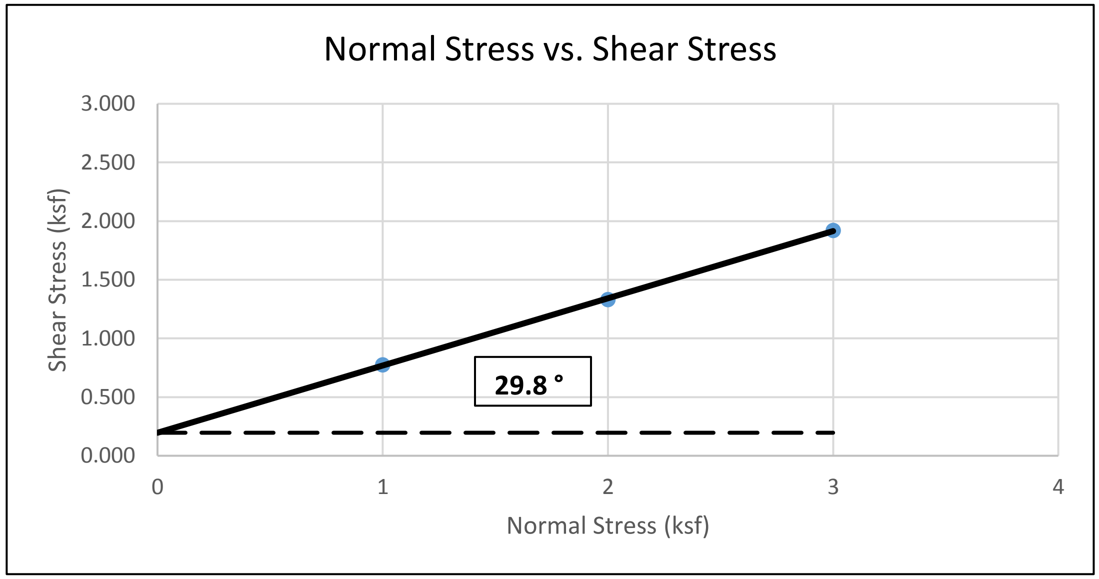
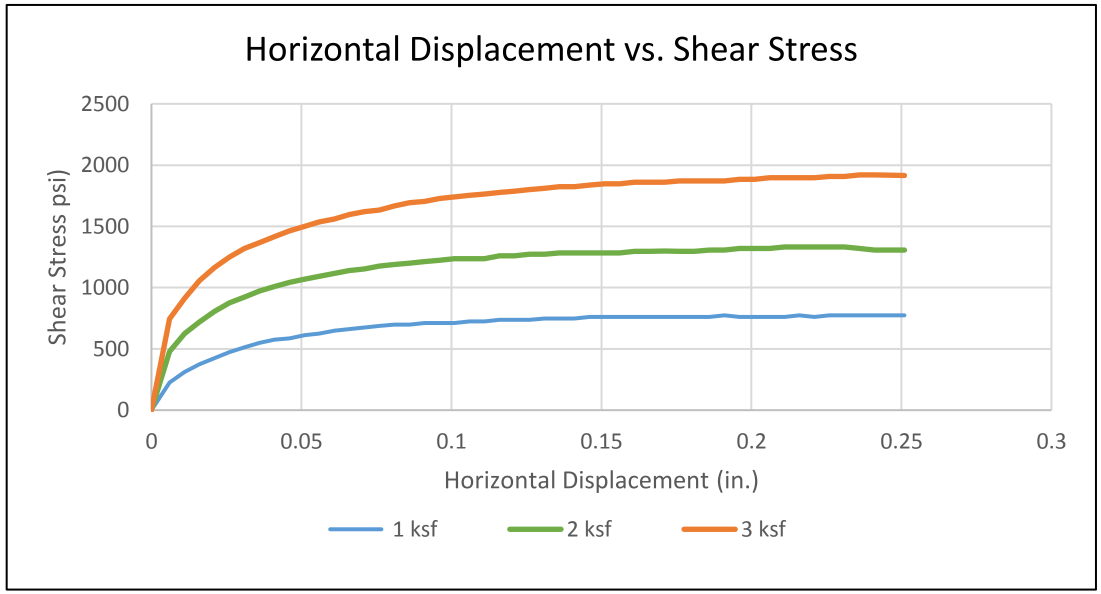
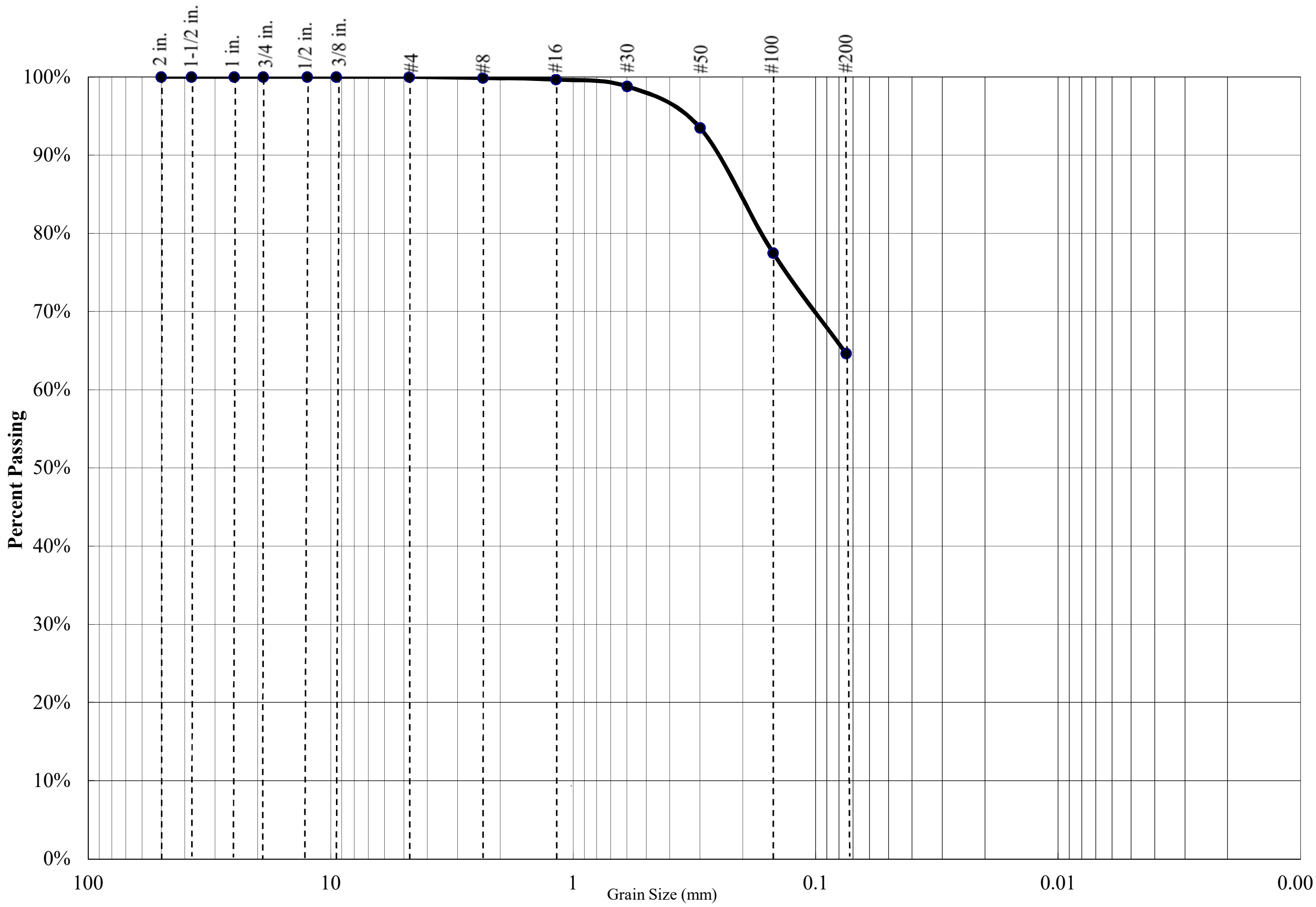




PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	35%	65%

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	99.7%
#30	98.8%
#50	93.5%
#100	77.5%
#200	64.6%

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: Proposed Mixed-Use Building - Pico Rivera, CA

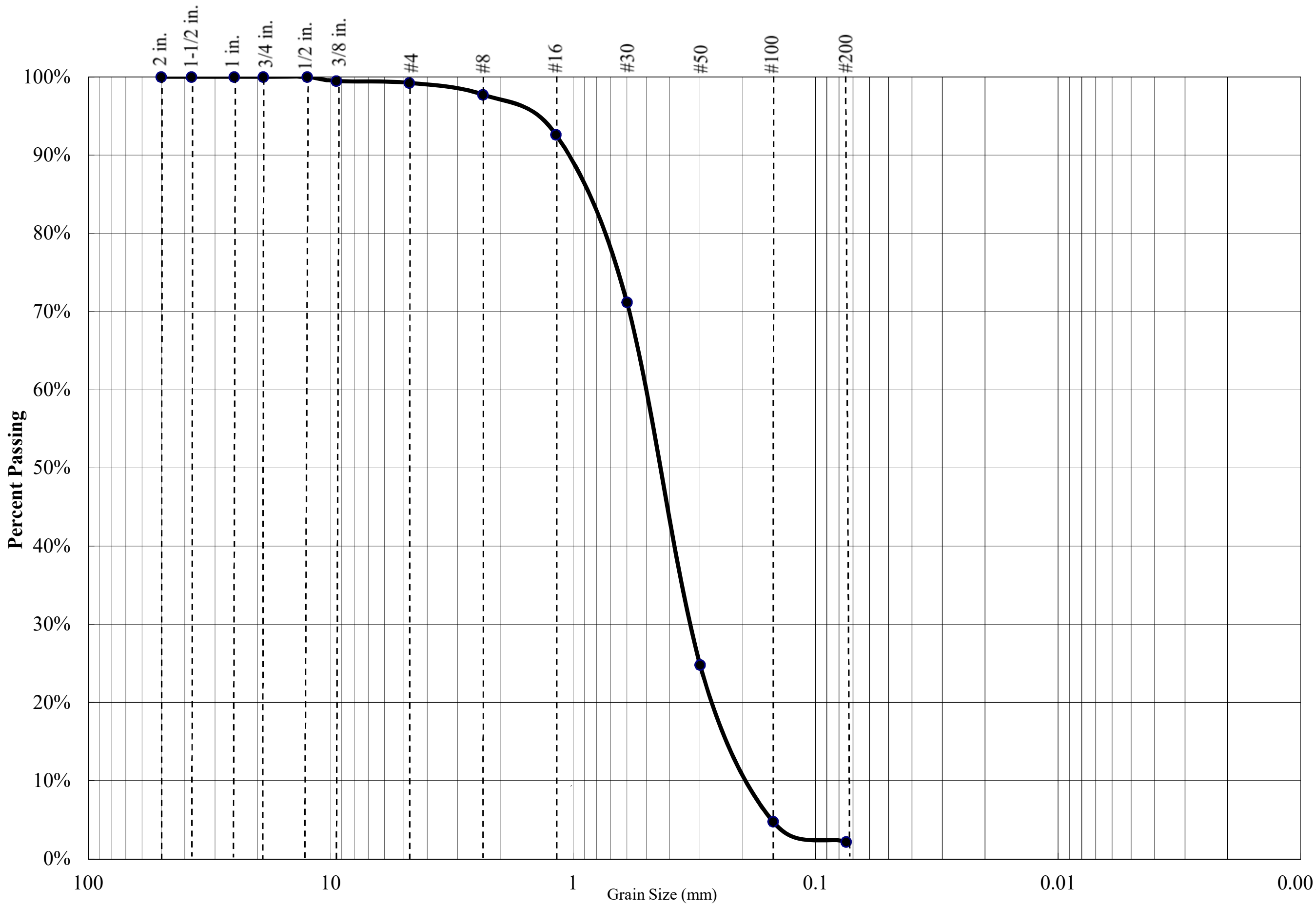
Project Number: 3-220-0499

Boring: B-1 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	97%	2%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	99.5%
#4	99.2%
#8	97.7%
#16	92.6%
#30	71.2%
#50	24.8%
#100	4.8%
#200	2.2%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	0.5	D_{50}=	
D_{30}=	0.35	D_{15}=		D_{10}=	0.2
C_u=	2.50	C_c=	1.23		

USCS CLASSIFICATION
Poorly graded SAND (SP)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

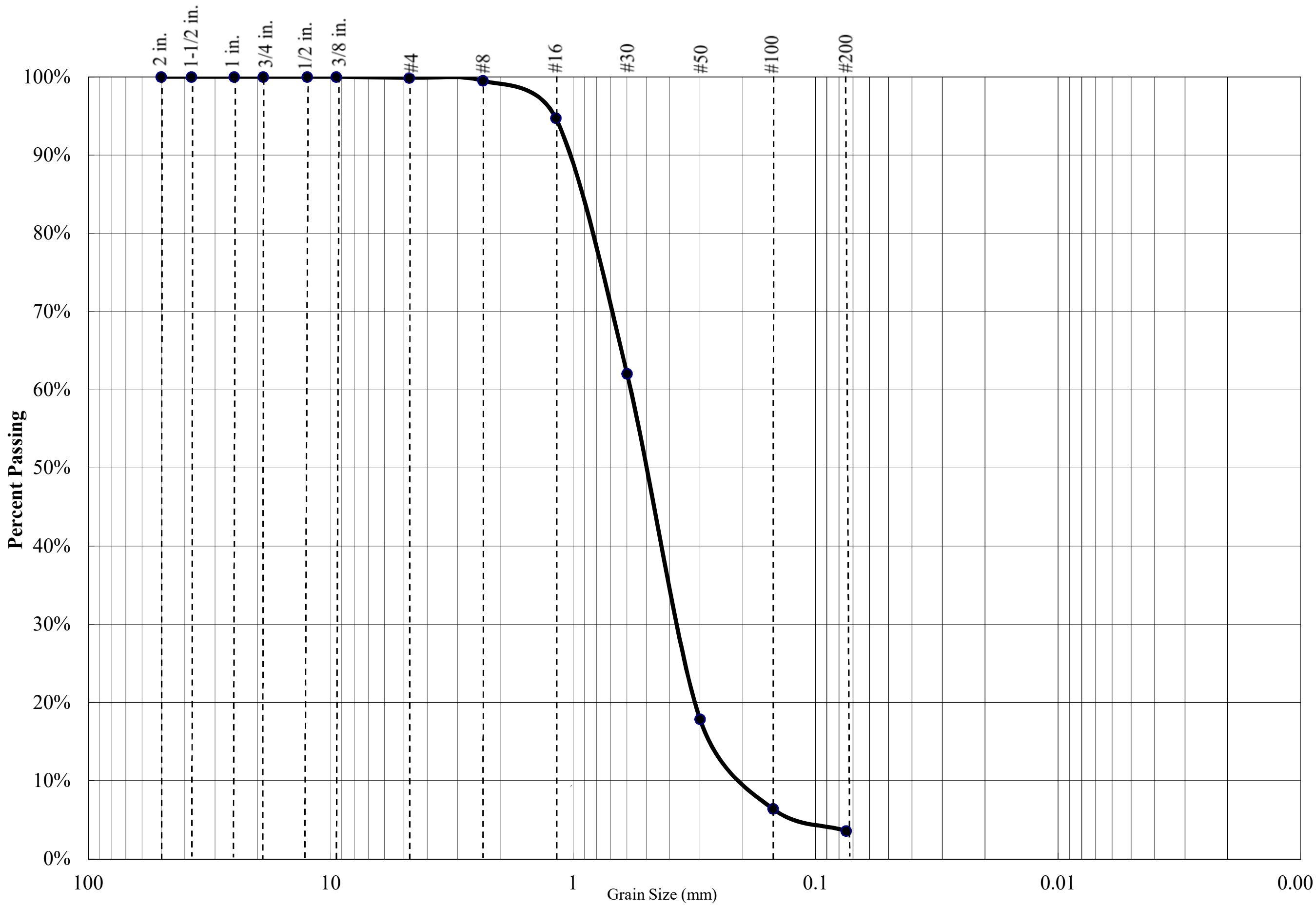
Project Number: 3-220-0499

Boring: B-1 @ 10'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	96%	4%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.9%
#8	99.5%
#16	94.7%
#30	62.1%
#50	17.9%
#100	6.4%
#200	3.6%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	0.6	D_{50}=	
D_{30}=	0.4	D_{15}=		D_{10}=	0.2
C_u=	3.00	C_c=	1.33		

USCS CLASSIFICATION
Poorly graded SAND (SP)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

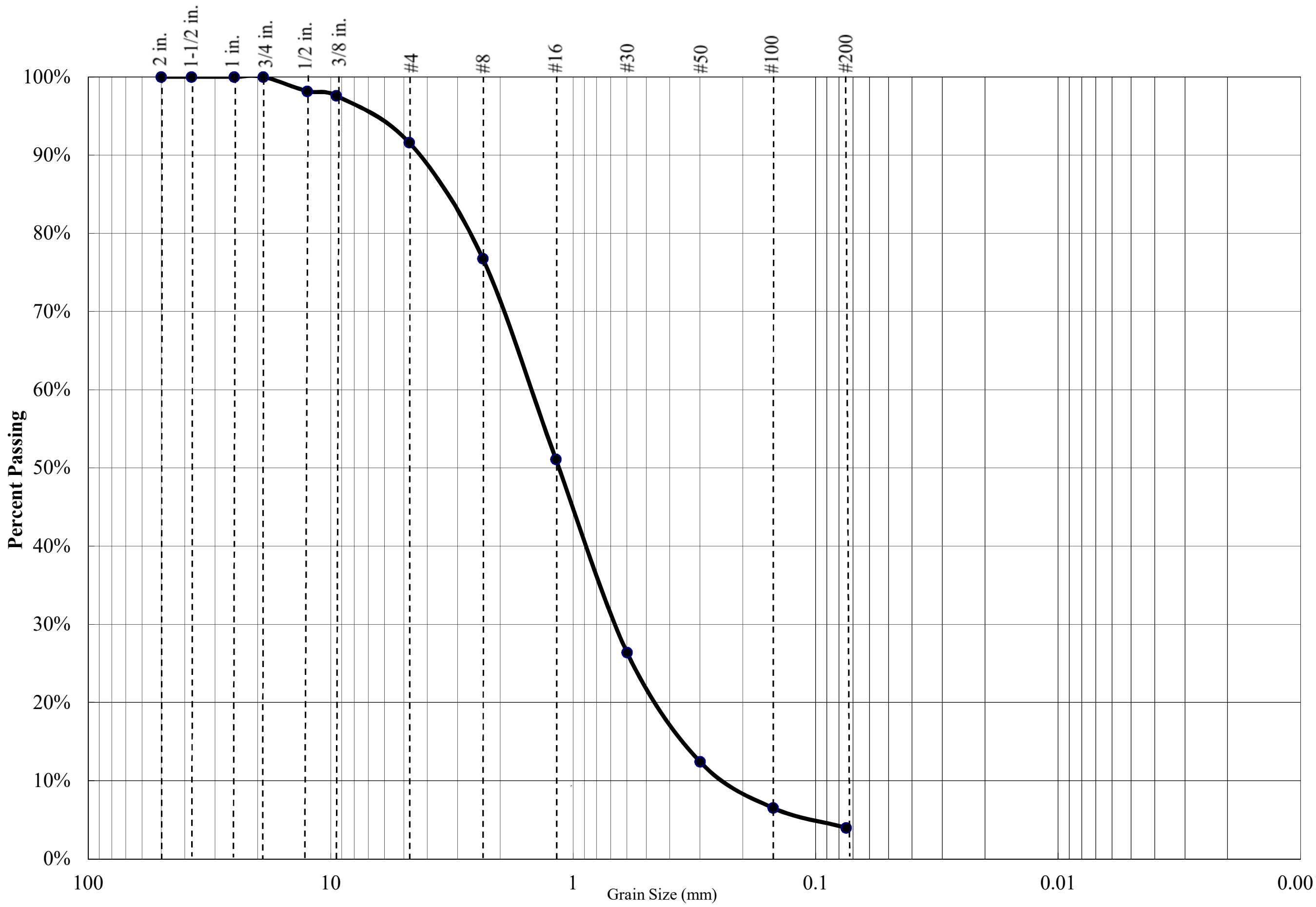
Project Number: 3-220-0499

Boring: B-1 @ 20'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
8%	88%	4%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	98.2%
3/8 inch	97.6%
#4	91.6%
#8	76.8%
#16	51.1%
#30	26.4%
#50	12.4%
#100	6.5%
#200	4.0%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	1.5	D_{50}=	
D_{30}=	0.7	D_{15}=		D_{10}=	0.25
C_u=	6.00	C_c=	1.31		

USCS CLASSIFICATION
Well-graded SAND (SW)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

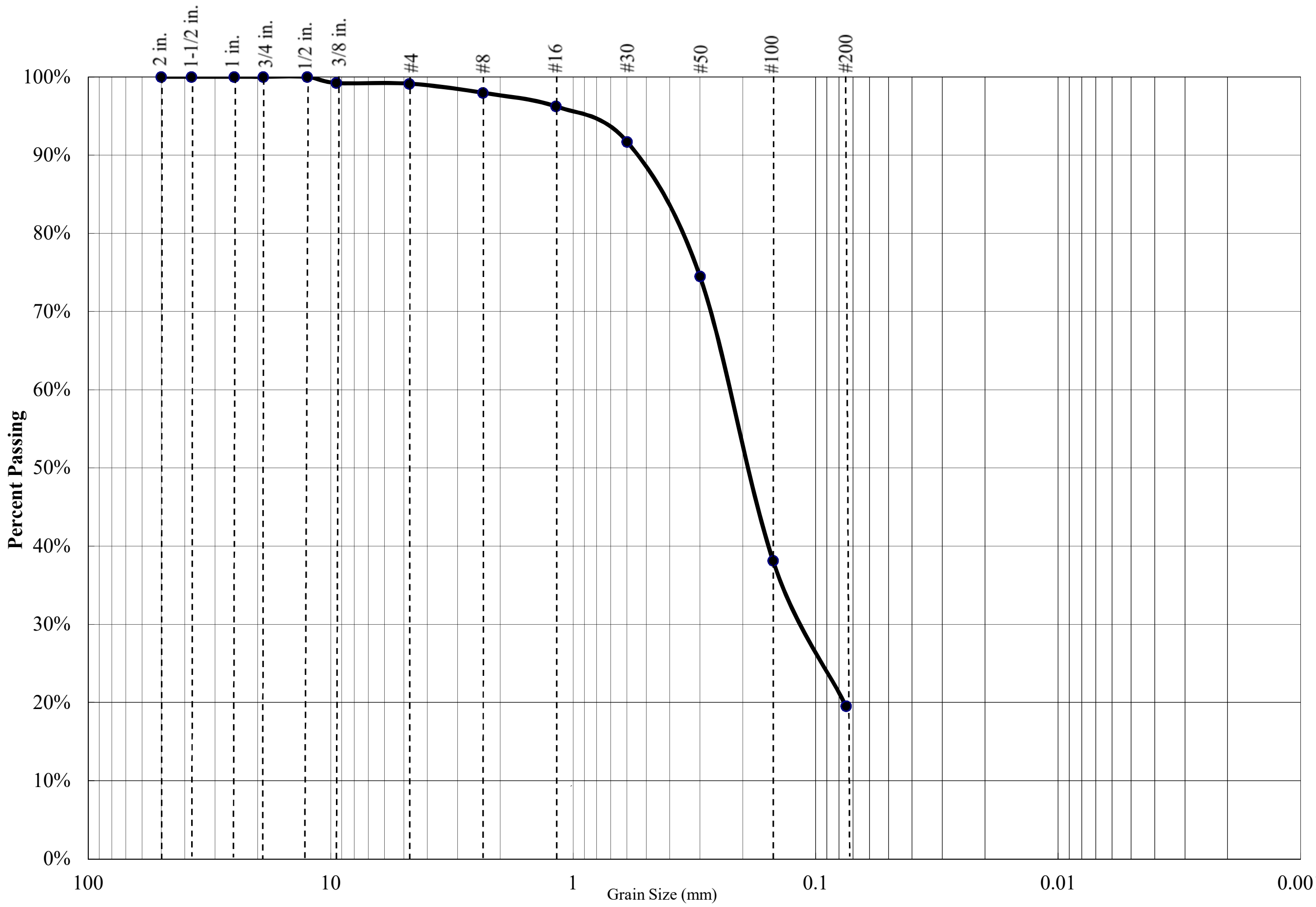
Project Number: 3-220-0499

Boring: B-1 @ 25'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	80%	20%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	99.2%
#4	99.1%
#8	98.0%
#16	96.2%
#30	91.7%
#50	74.5%
#100	38.2%
#200	19.5%

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 (SM)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

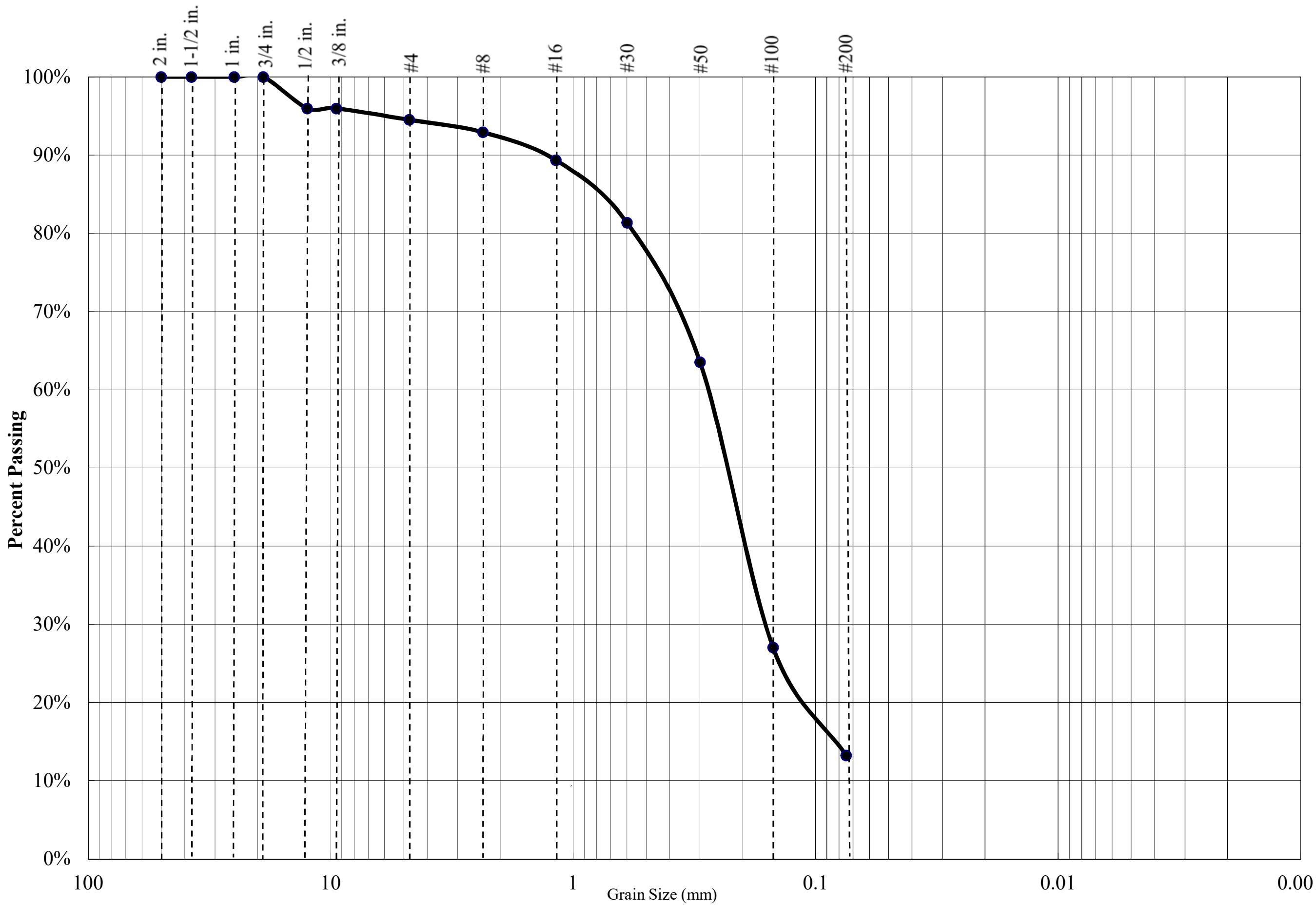
Project Number: 3-220-0499

Boring: B-1 @ 40'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
5%	81%	13%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	96.0%
3/8 inch	96.0%
#4	94.5%
#8	92.9%
#16	89.4%
#30	81.3%
#50	63.5%
#100	27.0%
#200	13.2%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

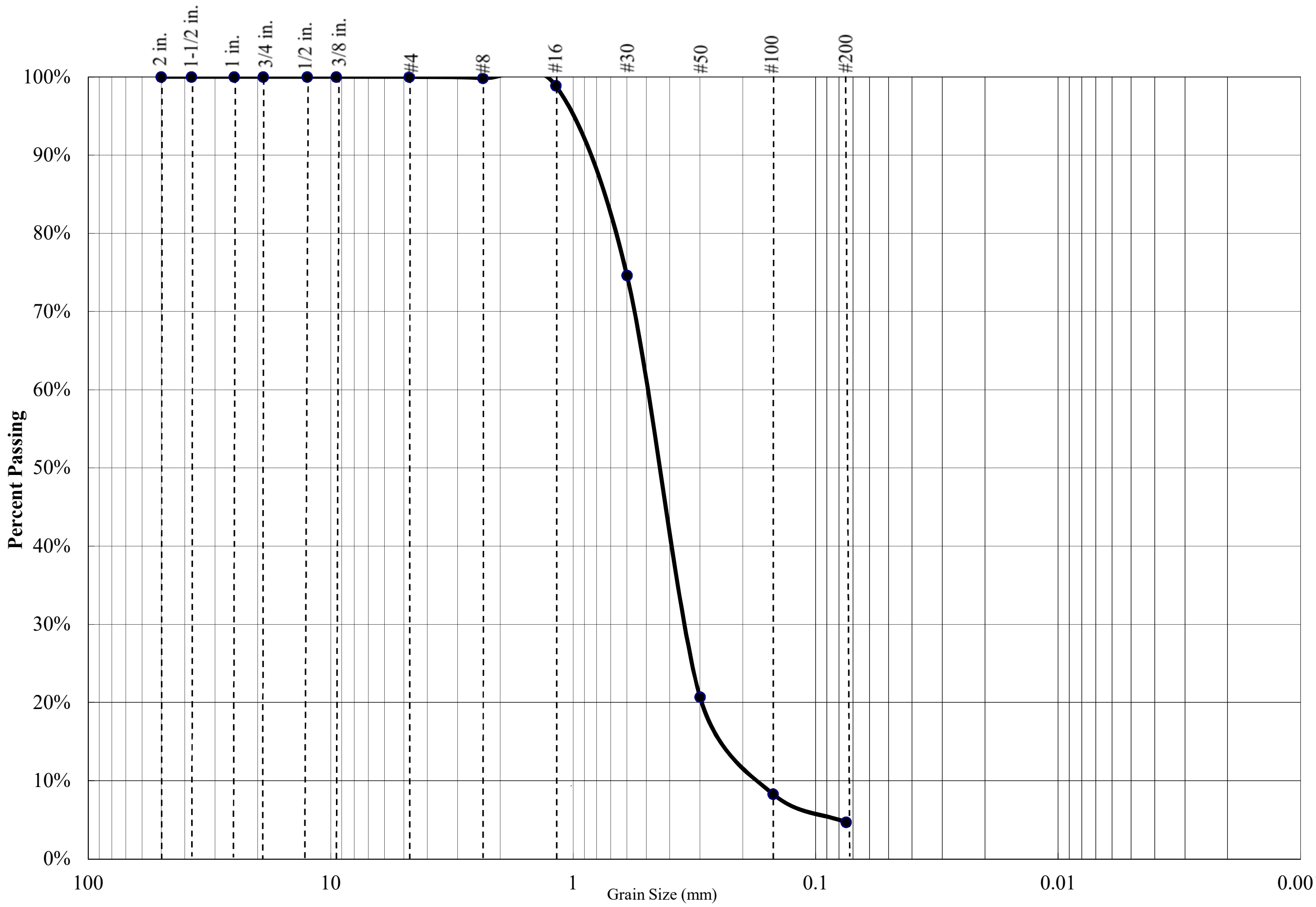
Project Number: 3-220-0499

Boring: B-1 @ 50'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	95%	5%

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	98.9%
#30	74.6%
#50	20.7%
#100	8.3%
#200	4.7%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	0.5	D_{50}=	
D_{30}=	0.35	D_{15}=		D_{10}=	0.175
C_u=	2.86	C_c=	1.40		

USCS CLASSIFICATION
Poorly graded SAND (SP)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

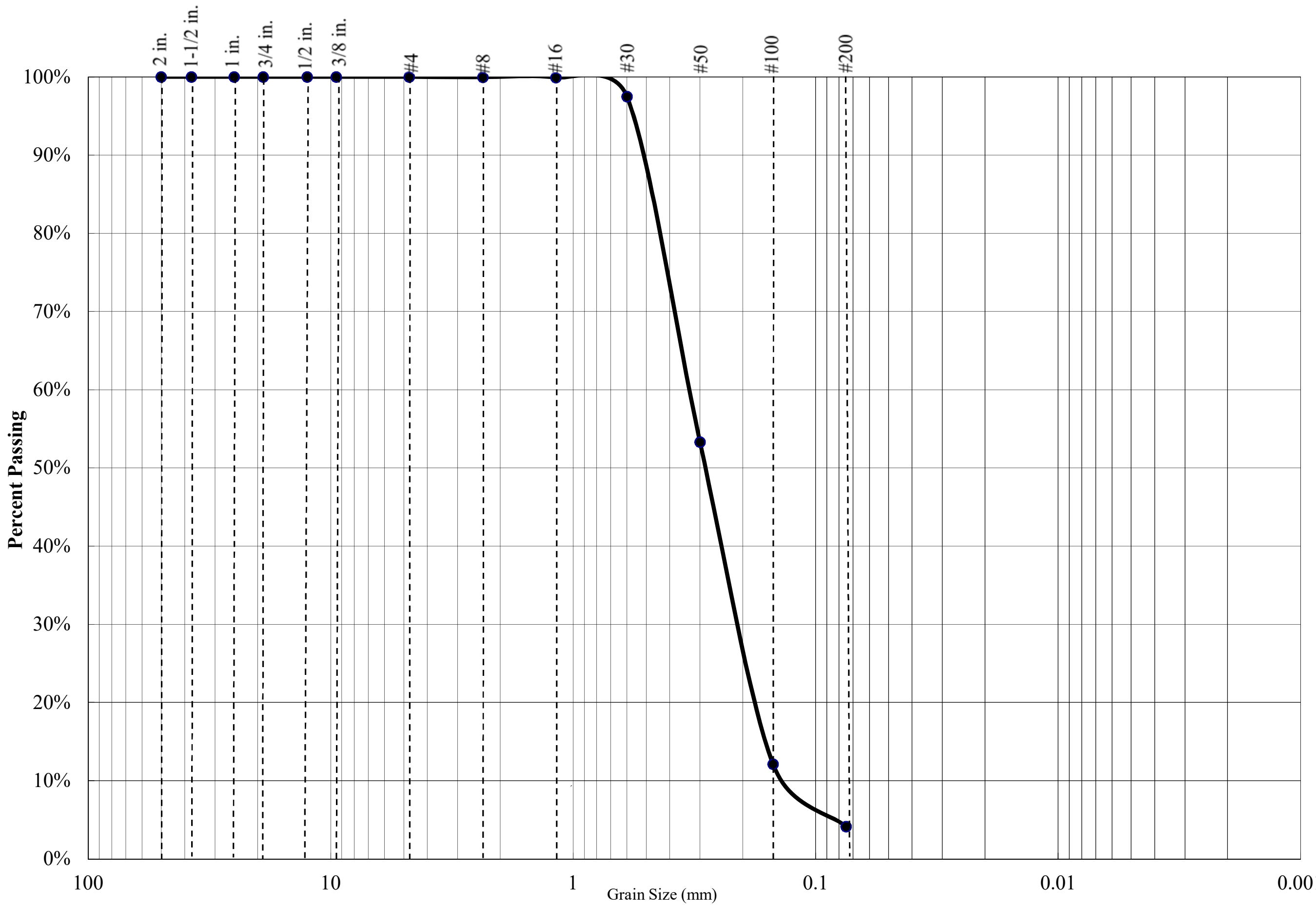
Project Number: 3-220-0499

Boring: B-3 @ 20'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	96%	4%

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	97.5%
#50	53.3%
#100	12.1%
#200	4.2%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	0.35	D_{50}=	
D_{30}=	0.2	D_{15}=		D_{10}=	0.15
C_u=	2.33	C_c=	0.76		

USCS CLASSIFICATION
Poorly graded SAND (SP)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

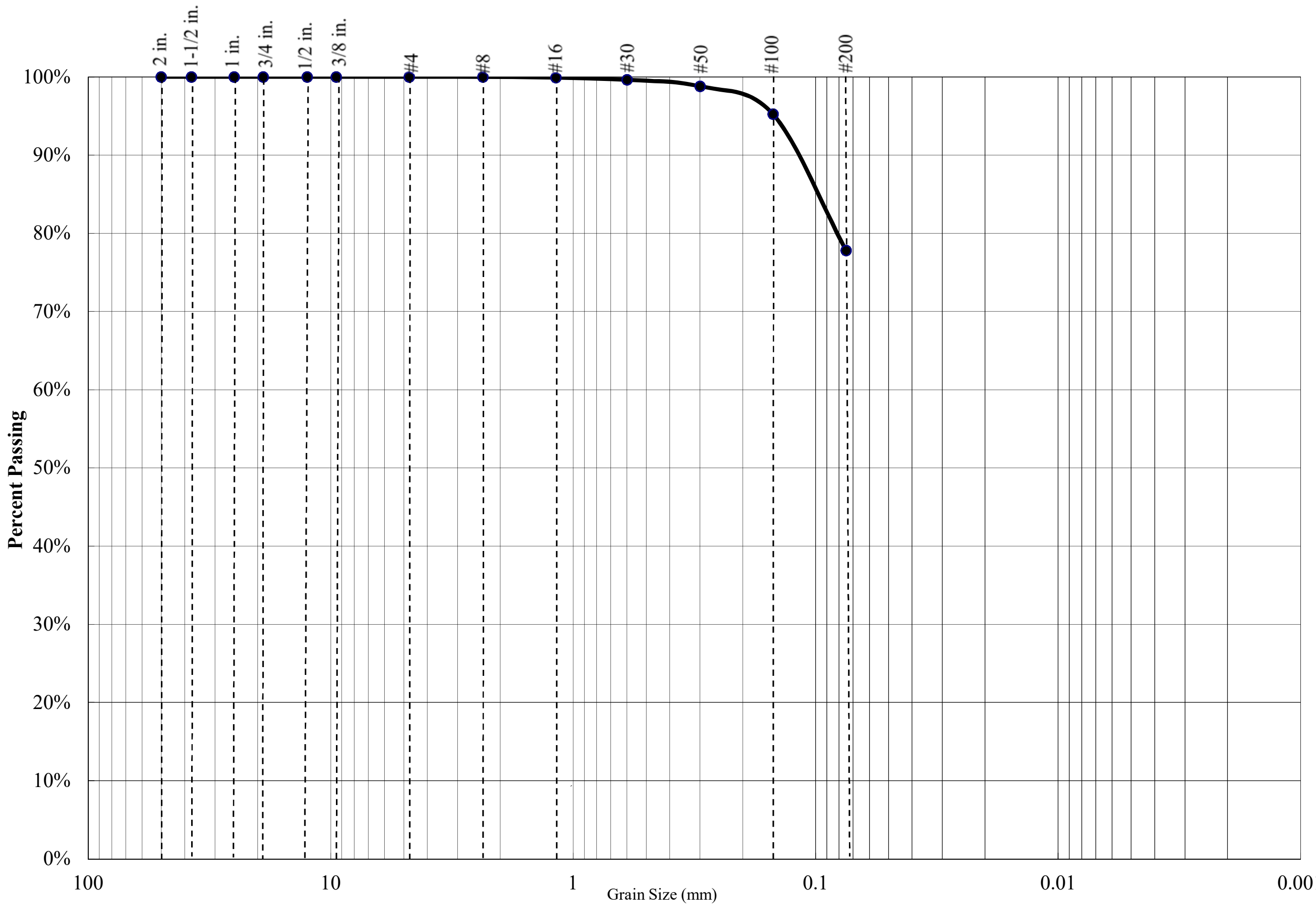
Project Number: 3-220-0499

Boring: B-4 @ 15'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	22%	78%

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	99.6%
#50	98.8%
#100	95.2%
#200	77.8%

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: Proposed Mixed-Use Building - Pico Rivera, CA

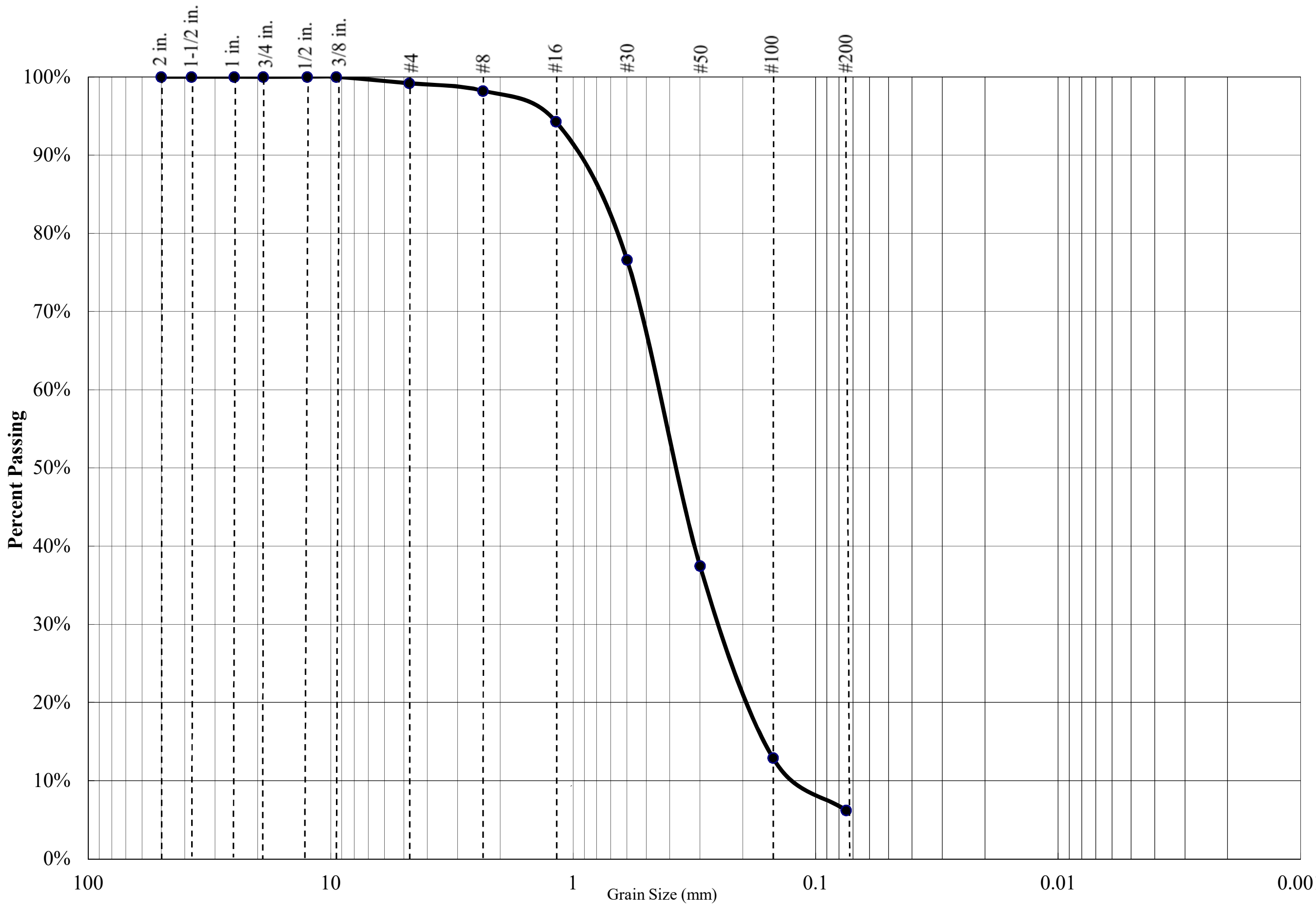
Project Number: 3-220-0499

Boring: B-5 @ 2'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	93%	6%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.2%
#8	98.2%
#16	94.3%
#30	76.6%
#50	37.5%
#100	12.9%
#200	6.2%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	0.45	D_{50}=	
D_{30}=	0.25	D_{15}=		D_{10}=	0.15
C_u=	3.00	C_c=	0.93		

USCS CLASSIFICATION
Poorly graded SAND with Silt (SP-SM)

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA

Project Number: 3-220-0499

Boring: B-6 @ 15'

SALEM engineering group, inc.

CHEMICAL ANALYSIS

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

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA
Project Number: 3-220-0499
Date Sampled: 7/7/2020 - 7/8/2020 Date Tested: 7/22/2020
Sampled By: EGR Tested By: MN
Soil Description: Light Brown Silty SAND (SM)

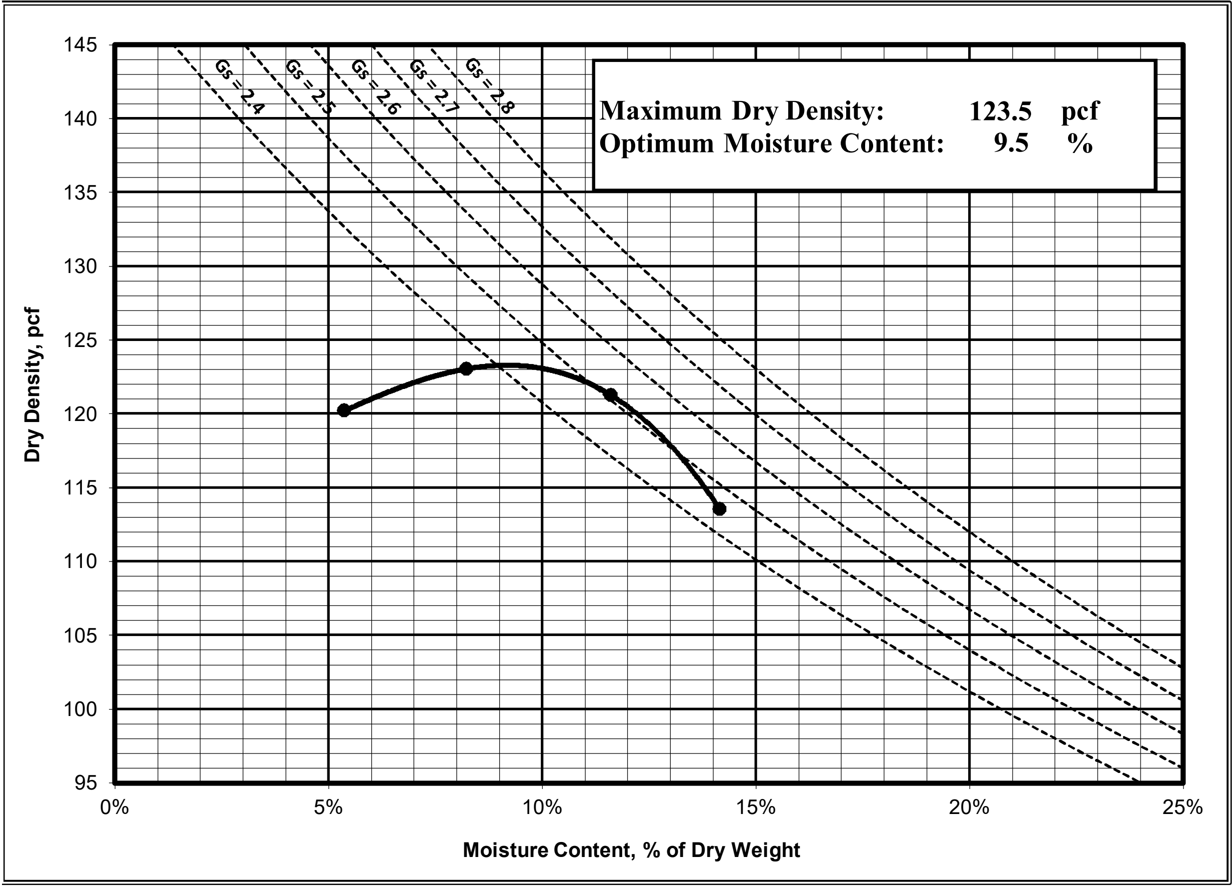
Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 1'-4'	<50 mg/kg	24 mg/kg	8.0
1b.	B-1 @ 1'-4'	<50 mg/kg	24 mg/kg	8.0
1c.	B-1 @ 1'-4'	<50 mg/kg	24 mg/kg	8.0
Average:		**<50 mg/kg**	**24 mg/kg**	**8.0**

SALEM engineering group, inc.

Laboratory Compaction Curve
ASTM D1557

Project Name: Proposed Mixed-Use Building - Pico Rivera, CA
Project Number: 3-220-0499
Date Sampled: 7/7/2020 - 7/8/2020 Date Tested: 7/23/2020
Sampled By: EGR Tested By: MN
Sample Location: B-1 @ 1'-4'
Soil Description: Light Brown Silty SAND (SM)
Test Method: Method A

	1	2	3	4
Weight of Moist Specimen & Mold, (g)	3928.7	4026.9	4059.9	3973.2
Weight of Compaction Mold, (g)	2013.7	2013.7	2013.7	2013.7
Weight of Moist Specimen, (g)	1915.0	2013.2	2046.2	1959.5
Volume of Mold, (ft^3)	0.0333	0.0333	0.0333	0.0333
Wet Density, (pcf)	126.7	133.1	135.3	129.6
Weight of Wet (Moisture) Sample, (g)	100.0	100.0	100.0	100.0
Weight of Dry (Moisture) Sample, (g)	94.9	92.4	89.6	87.6
Moisture Content, (%)	5.4%	8.2%	11.6%	14.2%
Dry Density, (pcf)	120.2	123.0	121.3	113.5



SALEM engineering group, inc.

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 fine grained 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 fine grained 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 fine grained 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% (90% for fine grained cohesive soil) based upon ASTM D1557. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

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

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

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