Appendix D

Geotechnical Engineering Investigation

October 31, 2019

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

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED RETAIL BUILDING
7028 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

SALEM PROJECT NO. 5-219-0790
OCTOBER 31, 2019

PREPARED FOR:

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October 31, 2019 Project No. 5-219-0790

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Subject: **GEOTECHNICAL ENGINEERING INVESTIGATION**
PROPOSED RETAIL BUILDING
7028 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

Dear Mr. Decker:

With your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation report for the proposed retail building to be located at the subject location. The proposed development will include construction of a retail building at the southwest edge of Santa Teresa Village Shopping Center parking lot in San Jose, California.

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

We appreciate the opportunity to assist you with this project. Should you have questions regarding this report or need additional information, please contact the undersigned at (559) 271-9700.

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RETAIL BUILDING
7028 SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation for the proposed retail building to be located at the south end of the existing Santa Teresa Village Shopping Center parking lot at 7028 Santa Teresa Boulevard in San Jose, California, as depicted on Figure 1, Vicinity Map.

SALEM Engineering Group, Inc. (SALEM) has completed this geotechnical engineering investigation with the purpose 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 recommendations presented herein are based on analysis of the data obtained during the investigation and our local 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.

2. SITE LOCATION AND DESCRIPTION

The subject site is located at the south end of the existing Santa Teresa Village Shopping Center parking lot at 7028 Santa Teresa Boulevard in San Jose, California (see Vicinity Map, Figure 1). The immediate project area is supports asphalt paved parking stalls and drive lanes along with planter boxes bordered by concrete curbs. The overall development was observed to be bounded to the northeast by existing asphalt paved parking and drive lanes, to the southwest by Santa Teresa Boulevard, and to the southeast and northwest by neighboring commercial developments. Mature trees in isolated landscape areas and overhead lighting were also noted throughout the development.

The vicinity surrounding the subject development includes primarily commercial developments.

The asphalt pavements that cover the project site were in good condition with very little sign distress. During the field exploration it was noted that there were two lifts of asphalt. It appears the parking area had been previously overlain.

The project site area is relatively flat with elevations of about 211 feet above mean sea level (AMSL), based on Google Earth Imagery.

3. PROJECT DESCRIPTION

An understanding of the project was provided by a preliminary site plan schematic provided by the client. We understand that development of the site includes construction of a retail building with a plan view area of about 7,000 square-feet and associated asphalt concrete paved parking and drives.

It is anticipated the proposed construction will comprise of CMU walls or wood framing supported on shallow spread foundations with concrete slabs on grade. Structural loads were not provided to us at the time this proposal was prepared. Based on our experience with similar projects maximum column and wall bearing loads of about 20 to 30 kips and 2 to 3 kips per foot, respectively, are anticipated. Floor slab soil bearing pressure is expected not to exceed 150 psf.

On-site asphalt concrete parking and drives, in addition to underground utilities and landscaping, will be associated with the development.

A site grading plan was not available at the time of preparation of this report.

Based on the relatively flat grade at the project site during our field exploration, it is anticipated that cuts and fills during earthwork will be on the order of 1 to 2 feet to providing a level area for the project area. 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 location and approximate locations of proposed improvements are shown on the Site Plan, Figure 2.

4. FIELD EXPLORATION

4.1. Drilling Test Borings

Our field exploration consisted of site surface reconnaissance and subsurface exploration. On October 17, 2019, a total of two (2) exploratory test borings (B-1 through B-2) were drilled to depths ranging from 20 to 50 feet below site grade (BSG). The test borings were drilled within or near the proposed building areas and parking areas at the approximate locations shown on Figure No. 2, Site Plan. The test borings were advanced with either 6-inch diameter hollow stem or solid-flight auger rotated by a truck-mounted CME-55 drill rig.

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

A Unified Soil Classification Chart and key to sampling is presented in Appendix A, including the test boring logs. Subsurface soil samples were obtained by driving a Modified California sampler (MCS) or a Standard Penetration Test (SPT) sampler. 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 site features determined from information 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.

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

Soil samples were obtained from the test borings at the depths shown on the test boring logs. 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 test borings were permitted through Contra Costa County. At the completion of drilling and sampling, the test borings were backfilled with cement grout backfill and capped with soil cuttings. Consequently, some settlement should be anticipated.

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, gradation, expansion index, plasticity index, resistance (R-value), and soil resistivity 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. FINDINGS AND RESULTS

6.1. Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils encountered predominantly consisted of sandy lean clays (CL) interbedded with sandy, silty clay (CL-ML) to depths of about 36 feet BSG, underlain by well graded sand with clay and gravel (SW-SC) to the maximum depth explored of 51.5 feet BSG.

A consolidation test resulted in about 8 percent consolidation under a load of 8 kips per square foot. When wetted under a load of 2 kips per square foot, these soils exhibited less than 1 percent collapse. A direct shear test resulted in an internal angle of friction of 33 degrees with a cohesion value of 335 pounds per square foot. The following table includes a summary of the four (4) Atterberg limits tests performed on samples collected during this investigation.

Boring	Depth	Plasticity Index	Liquid Limit
B-1	1.5	18	36
B-1	10	5	25
B-1	30	13	33

An expansion index test performed on a near surface soil sample resulted in an expansion index of 51. An R-value test resulted in an R-value of 15.

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.

6.2. Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was encountered during this investigation at a depth of 19 feet BSG. Available groundwater depth records with the Department of Water Resources (http://wdl.water.ca.gov/waterdatalibrary/) indicate, State Well No. 08S02E18L001M located approximately 1¼ mile northwest of the project site, reported a historical high groundwater depth of about 9 feet below ground surface in March 1941 and a most current groundwater depth of 20 feet below ground surface in September 2019.

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

6.3. Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate and chloride levels and how they relate to cement reactivity with soil and/or water. A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil sample was detected to be 260 mg/kg.

ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class. ACI 318 requirements for site concrete based upon soluble sulfate are summarized in Table 6.3 below.

TABLE 6.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Dissolved Sulfate (SO_4) in Soil % by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementitious Materials Type
0.026	Not Applicable	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 25 mg/kg. In addition, testing performed on a near surface soil resulted in a minimum resistivity value of 1,571 ohm-centimeters. Based on the results, these soils would be considered to have “highly corrosive” potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings).

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. Additional corrosion testing for minimum resistivity may need to be performed if required by the pipe manufacturer.

7. GEOLOGIC SETTING

The project area is located in the Santa Clara Valley within the Coast Ranges Geomorphic Province of California. The Coast Ranges generally consist of an alternating series of parallel mountains and valleys located adjacent to the Pacific Coast. The bedrock units that form the range have been disrupted by intense folding, faulting, and crushing that occurred when the range was formed by the processes of plate tectonics. During the Jurassic and Cretaceous Periods (about 150 to 80 million years ago), the Pacific Oceanic Plate, which was progressively moving towards the east, collided with the North American Continental Plate, which was moving toward the west.

Based on review of the Geologic map of the Santa Teresa Hills quadrangle, Santa Clara County, California[1], the site is located in an area mapped as (Qa) Quaternary era alluvial deposits including gravel, sand and clay of valley areas. The soils observed on site resemble the mapped alluvial deposits for this area.

8. GEOLOGIC HAZARDS

8.1. 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 moderate seismicity. 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. There are no known active fault traces in the immediate project vicinity.

The project area is not within an Alquist-Priolo 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 37.2264° North; site longitude is -121.7750° West. The ten closest active faults are summarized below in Table 8.1.

TABLE 8.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Monte Vista-Shannon	3.68	6.5
Calaveras;CN+CC	6.77	7.0
N. San Andreas;SAS	11.10	7.1
N. San Andreas;SAO+SAN+SAP	13.13	7.9
Zayante-Vergeles	14.32	7.0
Calaveras;CN	15.18	6.9
Hayward-Rodgers Creek;RC+HN+HS	16.23	7.3

[1] Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Santa Teresa Hills quadrangle, Santa Clara County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-158, scale 1:24,000

Greenville Connected	22.79	7.0
Calaveras;CS	23.51	5.8
Ortigalita	27.37	7.1

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.

8.2. 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.

8.3. Ground Shaking

Based on the 2019 CBC, a Site Class D (stiff soil) was selected for the site based on soil conditions with standard penetration resistance, N-values, averaging between 15 and 50 blows per foot. Table 9.6.1 includes design seismic coefficients and spectral response parameters, based on the 2019 California Building Code (CBC) for the project foundation design.

Based on Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.88g (based on both probabilistic and deterministic seismic ground motion).

8.4. 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.

The soils encountered predominantly consisted of sandy lean clays (CL) interbedded with sandy, silty clay (CL-ML) to depths of about 36 feet BSG, underlain by well graded sand with clay and gravel (SW-SC) to the maximum depth explored of 51.5 feet BSG. Based on available water well data, recent high groundwater depths were reported at depths of about 9 feet below ground surface in March 1941.

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 review of CGS Map Zones of Required Investigation Santa Teresa Hills, the site is located within an area mapped for liquefaction potential.

Based on Bray and Sancio (2006), soils with a water content to liquid limit ratio equal to or greater than 0.85 and soil plasticity index equal to or less to 12 are considered susceptible to liquefaction. Soils with plasticity indexes between 12 and 20, and water content to liquid limit ratio greater to or equal to 0.8 could be considered moderately susceptible to liquefaction. Based on the results of the samples tested and in-situ moisture contents, the sandy lean clay and sandy, silty, clay soils encountered below a depth of 9 feet BSG, within the upper 36 feet BSG are generally considered to not be susceptible to liquefaction/seismic settlement. However, it should be noted that sandy lean clay sample (81% fines, Wc/LL=0.81), at 30 feet BSG may be considered to have a "moderate" liquefaction potential. However, based on reduced moisture content around 35 feet BSG, the Wc/LL ratio of these materials is around 0.5, therefore, the thickness of these potentially "moderate" liquefiable soils were considered to be limited to between 30 and 34 feet BSG.

As such, the potential for soil liquefaction during a seismic event was evaluated using the Liquefy Pro computer program (version V.5) developed by CivilTech Corporation and utilizing data obtained from the test boring B-1 (projected to 51.5 feet BSG) conducted as part of this investigation. For the analysis, a maximum earthquake magnitude of 6.6 M_w, a peak horizontal ground surface acceleration of 0.88g (PGA_m), and groundwater depth of 9 feet below site grade. The maximum earthquake magnitude was derived from deaggregation of seismic sources obtained using the USGS 2008 Interactive Deaggregation website (http://geohazards.usgs.gov/deaggint/2008/).

Based on the results of the liquefaction/seismic settlement analysis performed, when considering sandy lean clay soils from 30 to 34 feet BSG to have "moderate" liquefaction potential, our analysis indicates total seismic settlement of about 2.25 inches was estimated due to a design level seismic event. However, it should be noted that the sandy lean clay samples screened for liquefaction at 30 feet BSG were borderline considered to have moderate liquefaction potential (Wc/LL=0.81). Based on the relatively high fines content (84 percent passing #200 sieve) these soils are not anticipated to be susceptible to liquefaction/seismic settlement. When considered non-liquefiable, total seismic settlements of about 0.75 inches was estimated. Therefore, based on the findings of this study, total and differential seismic settlements of about 2 inches and 1 inch in 40 feet, respectively, should be anticipated for design. The findings of the liquefaction/seismic settlement study are included at the end of this report.

8.5. 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, clayey nature of the near surface soils, and depth to potentially liquefiable soils (greater than 30 feet BSG), we judge the likelihood of lateral spreading to be low.

8.6. 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.

8.7. 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.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1. General Conclusions

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 soils encountered predominantly consisted of sandy lean clays (CL) interbedded with sandy, silty clay (CL-ML) to depths of about 36 feet BSG, underlain by well graded sand with clay and gravel (SW-SC) to the maximum depth explored of 51.5 feet BSG.

9.1.3 The near surface soils tested exhibited low compressibility characteristics and exhibited medium expansion potential. When compacted as engineered fill these soils are anticipated to have poor to fair pavement support characteristics.

9.1.4 Based on the results of the expansion index testing, this report includes recommendations to support slabs on grade on a uniform layer of imported non expansive engineered fill.

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

9.1.6 Provided the site is graded in accordance with the recommendations of this report and foundations constructed as described herein, we estimate that total settlement due to static loads utilizing conventional shallow foundations for the proposed building will be within 1-inch and corresponding differential settlement will be less than ½-inch in 40 feet.

9.1.7 The site is located within an area of mapped liquefaction potential. Based on the results of this investigation, total and differential seismic settlement of about 2 inches and 1 inch in 40 feet should be considered for design.

9.1.8 Based on the chemistry testing performed, the near surface soils have ‘negligible’ potential for sulfate attack on concrete and “highly corrosive” potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings).

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

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

9.2. Surface Drainage

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

9.2.2 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. Impervious surfaces within 10 feet of the building 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.

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

9.2.4. Due to the expansive nature of the near surface soils, bio-retention systems should not be planned within 25 feet of the proposed building.

9.3. Site Grading

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

9.3.2 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.

9.3.3 Site demolition activities shall include removal of all surface obstructions not intended to be incorporated into final site design. In addition, undocumented fill, underground buried structures, and/or utility lines (if any), existing foundation elements, etc., encountered during demolition and construction should be properly removed and the resulting excavations backfilled with Engineered Fill. After demolition activities, it is recommended that disturbed soils be removed and/or replaced with compacted engineered fill soils.

9.3.4 Excavations or depressions resulting from site clearing/demolition operations, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report.

9.3.5 Vegetation in existing planter areas consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. The upper 2 to 4 inches of the soils containing, vegetation, roots and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. The stripped vegetation will not be suitable for use as Engineered Fill or within 5 feet of building pads. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

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

9.3.7 To provide uniform support for the proposed building, it is recommended that over-excavation extend to the minimum depth of 12 inches below foundations, 30 inches below preconstruction site grades, or to the depth to remove undocumented fills (if any), whichever is greater. The resulting over-excavation shall be scarified to a depth of at least 8 inches, worked until uniform and free from large clods, moisture-conditioned to between 1 and 3 percent above optimum moisture, and compacted to a minimum of 90 percent of the maximum density. The horizontal limits of the over-excavation should extend throughout the building over-build zone, laterally to a minimum of 5 feet beyond the outer edges of the proposed footings.

9.3.8 Interior slabs on grade should be supported on a minimum of 6 inches of Class 2 aggregate base over 12 inches of imported non-expansive engineered fill, over the depth of engineered fill recommended below foundations.

9.3.9 Areas of exterior concrete slabs on grade located outside the building pad over-build zone, should be prepared by over-excavation to a minimum of 18 inches below existing grade or 12 inches below the bottom of slab on grade, whichever is greater. The zone of over-excavation should extend a minimum of 3 feet beyond these improvements. The bottom of excavation should be scarified 8 inches, moisture conditioned to between 1 and 3 percent above and compacted as engineered fill.

Exterior concrete slabs on grade should be supported on a minimum of 4 inches of Class 2 aggregate base compacted to 95 percent relative compaction over 8 inches of imported non expansive engineered fill over subgrade soils prepared as recommended above.

9.3.10 Areas of proposed asphaltic concrete and Portland cement concrete pavements should be prepared by over-excavation to a minimum of 12 inches below preconstruction site grade or 12 inches below the bottom of proposed pavement section. The zone of over-excavation should extend to a minimum of 3 feet beyond these improvements. The bottom of excavation should be scarified 8 inches, moisture conditioned to between 1 and 3 percent above and compacted as engineered fill. The upper 12 inches below pavements should be compacted to 95 percent relative compaction

9.3.11 Areas to receive engineered fill outside the building pad over-build zone, should be prepared by scarification of the upper 12 inches below existing grade or 12 inches below the recommended base section, whichever is greater. These soils should be moisture conditioned to between 1 to 4 percent above optimum and compacted as engineered fill.

9.3.12 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.

9.3.13 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.

9.3.14 We do not anticipate groundwater or seepage to adversely affect shallow excavations (less than 5 feet BSG) if conducted during the drier months of the year (typically summer and fall). Excavations greater than about 5 feet BSG may encounter free-water, thus, dewatering and stabilization should be anticipated. 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.

9.3.15 Due to the elevated moisture contents of the near surface materials encountered, aeration/stabilization of the upper soils may be required. Typical remedial measures include: discing and aerating the soil during dry weather; mixing the soil with dryer materials; removing and replacing the soil with an approved fill material or placement of crushed rocks or aggregate base material; or mixing the soil with an approved lime or cement product.

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

If the use of crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of ¾-inch to 1-inch crushed rocks. The thickness of the rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of crushed rock material will provide a stable platform. It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock. All open graded crushed rock/gravel should be fully encapsulated with a geotextile fabric (such as Mirafi 140N) to minimize migration of soil particles into the voids of the crushed rock. Although it is not required, the use of geogrid (e.g. Tensar BX 1100, BX 1200 or TX 160) below the crushed rock will enhance stability and reduce the required thickness of crushed rock necessary for stabilization.

In addition, chemical drying of the bottom of the excavation and engineered fill soils could be considered. For bidding purposes, the Contractor may assume 5 percent high calcium quicklime for chemical stabilization/drying of on-site soils. The actual application rate will need to be adjusted based on conditions encountered during grading.

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

9.4. Soil and Excavation Characteristics

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

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

9.4.3 Although not encountered, due to the current site development, undocumented fill material may be encountered throughout the site. This report includes recommendations that all abandoned subsurface structures, and undocumented fill material, be fully removed and/or compacted as engineered fill.

9.4.4 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.

9.5. Materials for Fill

9.5.1 The on-site soils are considered suitable for use as general Engineered Fill in structural areas, at depths greater than 18 inches below interior concrete slabs on grade, and 12 inches below exterior slabs on grade, provided they do not contain deleterious matter, organic material, or material larger than 3 inches in maximum dimension. The upper 18 and 12 inches of soils below interior and exterior slabs on grade, respectively, should include a combination of aggregate base over imported non expansive fills as specified in section 9.3 of this report.

9.5.2 Imported Engineered Fill soil, should be well-graded, low-to-non-expansive slightly cohesive silty sand or sandy silt. A clean sand or very sandy soil is not acceptable for this purpose. A sandy soil will allow the surface water to drain into the expansive clayey soils below, which may result in unacceptable swelling. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 9.5.2.

TABLE 9.5.2
IMPORT FILL REQUIREMENTS

Percent Passing 3-inch Sieve	100
Percent Passing No.4 Sieve	75-100
Percent Passing No 200 Sieve	15-40
Maximum Plasticity Index	15
Organic Content, Percent by Weight	Less than 3%
Maximum Expansion Index (ASTM D4829)	20

Prior to importing the Contractor should demonstrate to the Owner that the proposed import meets the requirements for import fill specified in this report. In addition, the material should be verified by the Contractor that the soils do not contain any environmental contaminates as regulated by local, state, or federal agencies, as applicable

9.5.3 All Engineered Fill (including scarified ground surfaces and backfill) should be placed in lifts no thicker than will allow for adequate bonding and compaction (typically 6 to 8 inches in loose thickness).

9.5.4 On-Site soils used as engineered fill soils should moisture conditioned to between 1 and 3 percent above optimum moisture content and compacted to at least 90 percent relative compaction (ASTM D1557).

9.5.5 Import Engineered Fill, if selected, should be placed, moisture conditioned to slightly above optimum moisture content, and compacted to at least 92 percent relative compaction (ASTM D1557).

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

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

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

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

9.6. Seismic Design Criteria

9.6.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2019 CBC, our recommended parameters are shown below. These parameters were determined using Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps by location website (https://seismicmaps.org/), in accordance with the 2019 CBC. The Site Class was determined based on the soils encountered during our field exploration.

TABLE 9.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	ASCE 7-16 or 2019 CBC Reference
Site Coordinates (Datum = NAD 83)		37.2264 Lat -121.7750 Lon	
Site Class	--	E	ASCE 7 Table 20.3-1
Soil Profile Name	--	Soft Clay Soils	ASCE 7 Table 20.3-1
Risk Category	--	II	CBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.1	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.88	ASCE 7 Equation 11.8-1

Seismic Item	Symbol	Value	ASCE 7-16 or 2019 CBC Reference
Seismic Design Category	SDC	E	CBC Table 1613.2.5
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	1.941 g	CBC Figure 1613.2.1(1-8)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.691 g	CBC Figure 1613.2.1(1-8)
Site Class Modified Site Coefficient	F_a	1.2*	CBC Table 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	2.0**	CBC Table 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	2.33*	CBC Equation 16-36
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	1.38**	CBC Equation 16-37
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.55*	CBC Equation 16-38
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	0.92**	CBC Equation 16-39
Ts (S_{D1}/S_{DS})		0.594**	

Notes:

***** **Calculated based on Exception 1 of ASCE 7-16, 11.4.8**

****** **Calculated values based ASCE 7-16 and ASCE7-16 Supplement for calculating values of Ts only.**

Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, Structures on Site Class E, using Fa of 1.2 (Exception 1, ASCE11.4.8) and Exception 3 -the fundamental period of the building (T) is less than or equal to 0.2, provided that T is less than Ts (Ts=S_{D1}/S_{Ds}), and the equivalent static force procedure is used for design.

The Structural Engineer should verify whether the building design meets exceptions under 11.4.8 of ASCE 7-16. In the event a site specific ground motion analysis is required, SALEM should be contacted for these services.

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

9.7. Shallow Foundations

9.7.1 The site is suitable for use of conventional shallow foundations consisting of continuous footings and isolated pad footings supported on engineered fill prepared in accordance with Section 9.3 of this report. Shallow foundations supported on engineered fill as recommended in this report may be designed based on total and differential static settlement of 1 inch and ½ inch in 40 feet, respectively. Total and differential seismic settlement of about 2 inches and 1 inch in 40 feet should be considered for design.

9.7.2 The bearing wall footings considered for the structure should be continuous with a minimum width of 18 inches and extend to a minimum depth of 18 inches below the lowest adjacent grade or 18 inches below the bottom of slab on grade, whichever is greater. Isolated column footings should have a minimum width of 18 inches and extend a minimum depth of 18 inches below the lowest adjacent grade.

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

9.7.4 Foundations supported on engineered fill as recommended in this report may be designed based on an allowable bearing capacity of 2,000 pounds per square foot. This value may be increased by one-third for wind and seismic loading.

9.7.5 Resistance to lateral footing displacement can be computed using an allowable coefficient of friction factor of 0.40 acting between the base of foundations and the supporting native subgrade.

9.7.6 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 300 pounds per cubic foot acting against the appropriate vertical native footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combination in Section 1605.3.2 of the 2019 CBC that includes wind or earthquake loads.

9.7.7 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.

9.7.8 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.

9.8. Interior Concrete Slabs-on-Grade

9.8.1 Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that non-structural slabs-on-grade be at least 4 inches thick and underlain by six (6) inches of class 2 aggregate base over 12 inches of imported non-expansive engineered fill over engineered fill extending below foundations.

9.8.2 The structural engineer should determine the minimum reinforcing required for interior slabs on grade. We recommend reinforcing slabs, at a minimum, with No. 3 reinforcing bars placed 18 inches on center, each way. If the owner is willing to accept additional risk for slab cracking, alternatives such as wire mesh or fiber reinforcement may be considered.

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

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

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

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

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

9.8.8 The concrete may be placed directly on vapor retarder. The vapor retarder should be inspected prior to concrete placement. Cut or punctured retarder should be repaired using vapor retarder material lapped 6 inches beyond damaged areas and taped. Extend vapor retarder over footings and seal to foundation wall or slab at an elevation consistent with the top of the slab or terminate at impediments such as water stops or dowels. Seal around penetrations such as utilities or columns in order to create a monolithic membrane between the surface of the slab and moisture sources below the slab as well as at the slab perimeter.

9.8.9 Avoid use of stakes driven through the vapor retarder.

9.8.10 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

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

9.9. Exterior Slabs on Grade

9.9.1 The following recommendations are intended for lightly loaded exterior slabs on grade not subject to vehicular traffic. Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that non-structural slabs-on-grade be at least 4 inches thick and underlain by four (4) inches of class 2 aggregate base over 8 inches of imported non expansive engineered fill over subgrade soils prepared in accordance with section 9.3 of this report. Due to the expansive potential of the near surface soils, slabs may be subject to ½ inch of heave over 40 feet.

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

9.9.3 Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement.

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

9.10. Lateral Earth Pressures and Frictional Resistance

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

Lateral Pressure Conditions	Soil Equivalent Fluid Pressure
Active Pressure, Drained, pcf	40
At-Rest Pressure, Drained, pcf	62
Allowable Passive Pressure, psf	300
Allowable Coefficient of Friction	0.40
Minimum Wet Unit Weight (lbs/ft^3) [γ_{min}]	100
Maximum Wet Unit Weight (lbs/ft^3) [γ_{max}]	135

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

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

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

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

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

Dynamic Seismic Lateral Loading Equation
Dynamic Seismic Lateral Load = $\frac{3}{8}\gamma K_h H^2$
Where: γ = Maximum In-Place Soil Density (Section 9.10.1 above)
K_h = Horizontal Acceleration = $\frac{2}{3}PGA_M$ (Section 9.6.1 above)
H = Wall Height

9.11. Temporary Excavations

9.11.1. We anticipate that the majority of the dense site soils will be classified as Cal-OSHA "Type B" soil when encountered in excavations during site development and construction. If the subgrade becomes unstable due to excessive moisture, the excavations should conform to Cal-OSHA "Type C" soil. 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.

9.11.2. 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.

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

9.11.4. 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	1½:1
10-15	2:1

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

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

9.11.7. 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.

9.12. 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 92 percent relative compaction at or above optimum moisture content. The upper 12 inches of trench backfill within asphalt or concrete paved areas shall be moisture conditioned to at or above optimum moisture content and compacted to at least 95 percent relative compaction.

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

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. Pavement Design

9.13.1. R-Value testing was performed on a sample obtained from the site at the location shown on the attached site plan (Boring B-1). The sample was tested in accordance with the State of California Materials Manual Test Designation 301. R-Value testing on a near surface sample resulted in an R-value of 15.

9.13.2 During grading subgrade samples should be tested to verify the recommendations included in this report remain valid. The pavement design recommendations provided herein are based on the State of California Department of Transportation (CALTRANS) design manual. Based on the results of the R-value testing performed and the requirements of Caltrans Highway Design Manual, an R-value of 15 was selected for design.

9.13.3 The asphaltic concrete (flexible pavement) is based on a 20-year pavement life utilizing traffic indexes of ranging from 4.0 to 7.0. The Civil Engineer should select the appropriate pavement section based on the anticipated traffic loading. The following table shows the recommended pavement sections for various traffic indices.

TABLE 9.13.3
ASPHALT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Asphaltic Concrete, (inches)	Class 2 Aggregate Base, (inches)*	Compacted Subgrade, (inches)*
4.0	3.0	4.0	12.0
5.0	3.0	8.0	12.0
6.0	3.0	11.5	12.0
7.0	4.0	13.0	12.0

95% compaction based on ASTM D1557 Test Method

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

TABLE 9.13.4
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Portland Cement Concrete, (inches)*	Class 2 Aggregate Base, (inches)**	Compacted Subgrade. (inches)**
4.0	5.0	6.0	12.0
5.0	5.5	6.0	12.0
6.0	6.0	6.0	12.0
7.0	6.5	6.0	12.0

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

9.13.5 Asphalt concrete should conform to Section 39 of Caltrans' latest Standard Specifications for ½ inch Hot Mix Asphalt (HMA) Type A or B.

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

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

9.13.8 An integral part of satisfactory fill placement is the stability of the placed lift of soil. Prior to placement of aggregate base, the subgrade soils should be proof-rolled by a loaded water truck (or equivalent) to verify no deflections of greater than ½ inch occur. If placed materials exhibit excessive instability as determined by a SALEM field representative, the lift will be considered unacceptable and shall be remedied prior to placement of additional fill material. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.13.9 A representative of our firm should be present during all site clearing and grading operations to test and observe earthwork construction. This testing and observation is an integral part of our service as acceptance of earthwork construction is dependent upon compaction of the material and the stability of the material.

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 1. The report does not reflect variations which may occur between borings. The nature and extent of such variations may not become evident until construction is initiated.

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

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

If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (559) 271-9700.

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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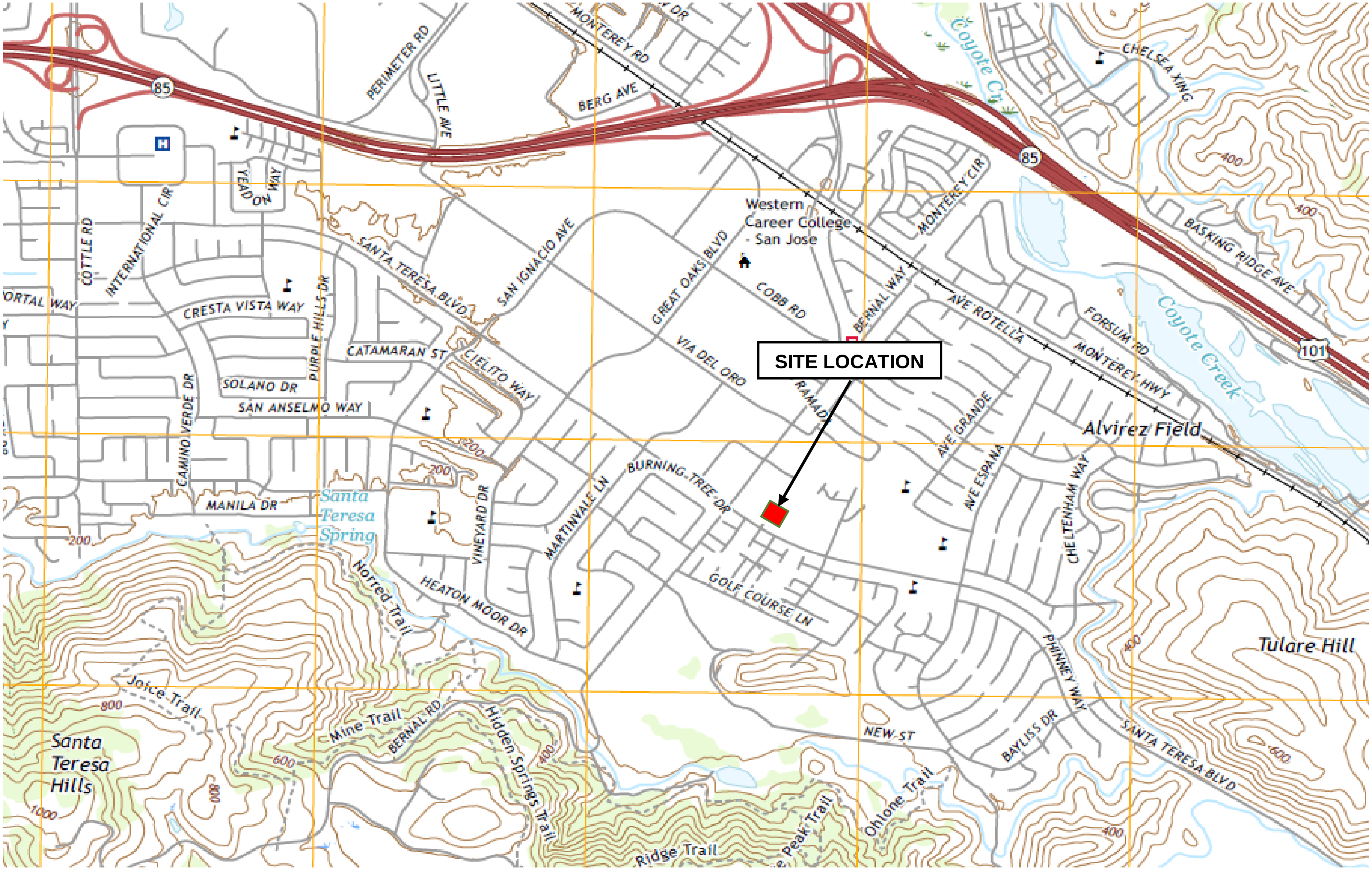
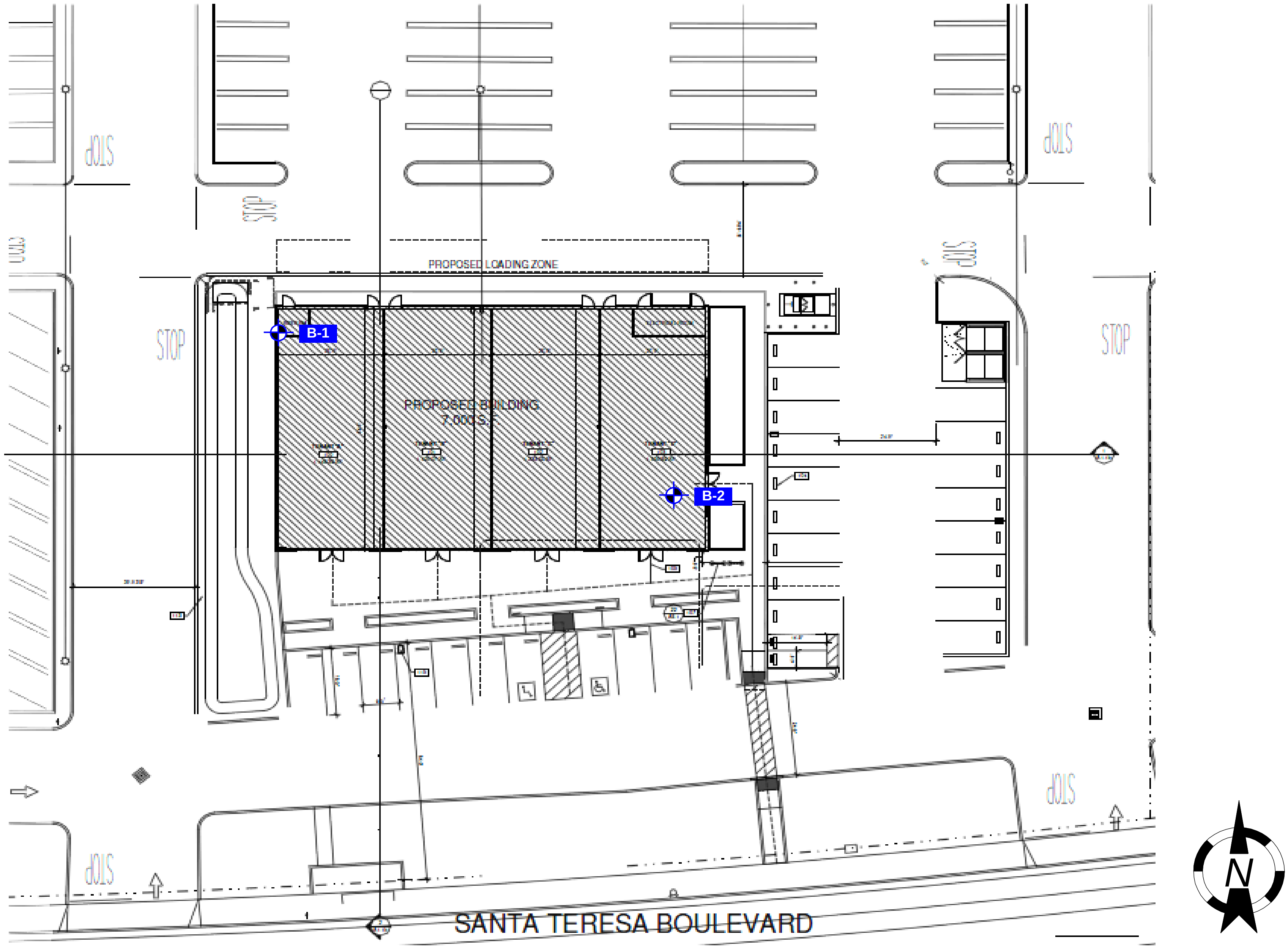


Source Image: U.S. Geological Survey, Santa Teresa Hills, Calif. 7.5' Quadrangle, 2018 (2016ed.)

VICINITY MAP **GEOTECHNICAL ENGINEERING INVESTIGATION** **Proposed Retail Building** **7028 Santa Teresa Boulevard** **San Jose, California**	SCALE: NOT TO SCALE	DATE: 10/2019	SALEM engineering group, inc.
	DRAWN BY: VT	APPROVED BY: JH	
	PROJECT NO. 5-219-0790	FIGURE NO. 1	



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Retail Store
7028 Santa Teresa Boulevard
San Jose, CA

SCALE:	DATE:
NOT TO SCALE	10/2019
DRAWN BY: VT	APPROVED BY: JH
PROJECT NO. 5-219-0790	FIGURE NO. 2

LEGEND:

B-1 Soil Boring Location

All Locations Approximate

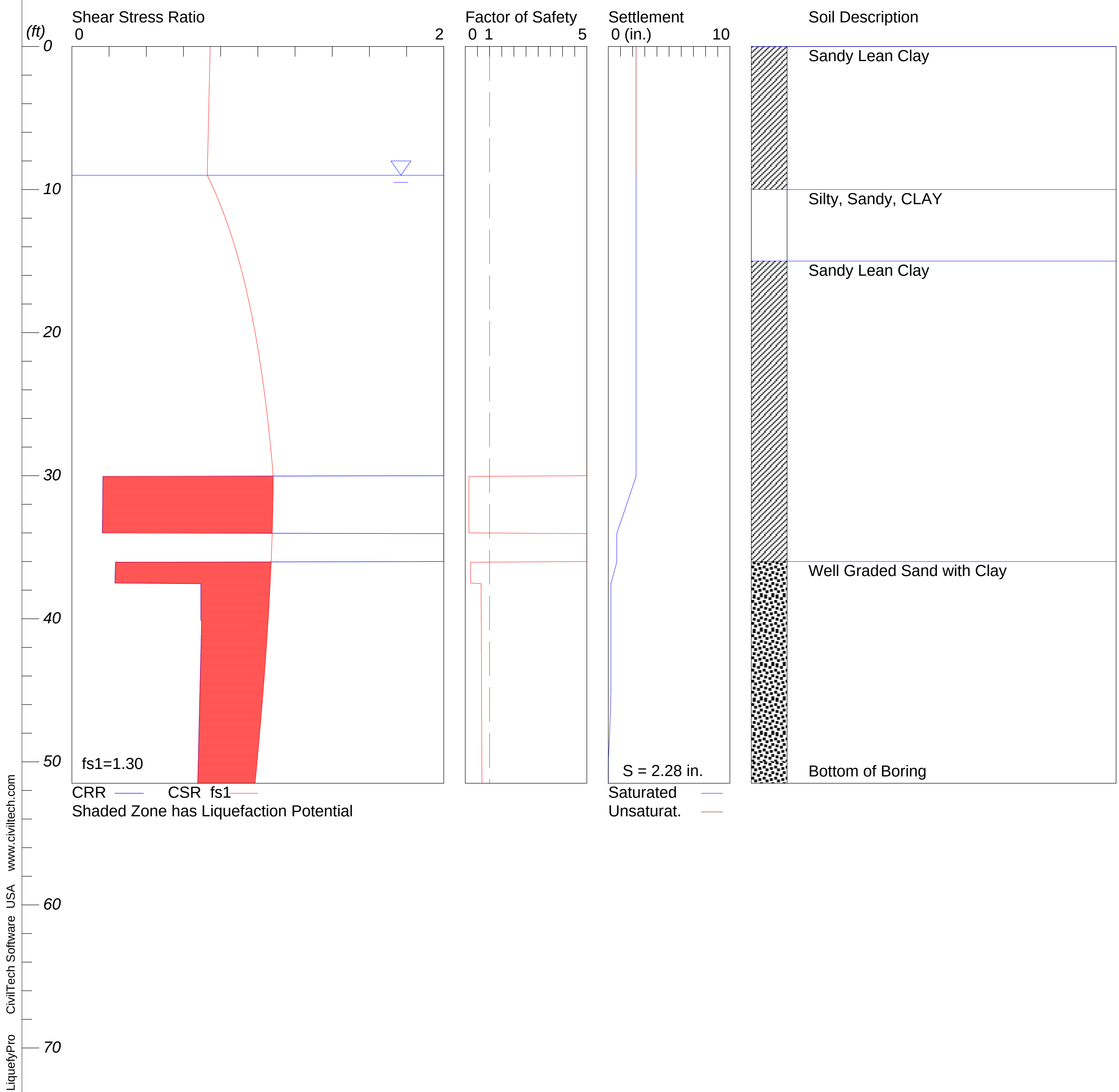
SALEM engineering group, inc.

LIQUEFACTION ANALYSIS

Retail Developement - San Jose, CA

SCENARIO 1, Soils 30-34 Feet Considered Moderately Susceptibel to Liquefaction

Hole No.=B-1 Water Depth=9 ft ***Magnitude=6.6***
Acceleration=0.88g



SCENARIO 1, Soils 30-34 Feet Considered Moderately Susceptibel to Liquefaction

****

LIQUEFACTION ANALYSIS SUMMARY
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****

Title: Retail Developement - San Jose, CA
Subtitle: 5-219-0790

Hole No.=B-1
Depth of Hole= 51.50 ft
Water Table during Earthquake= 9.00 ft
Water Table during In-Situ Testing= 15.00 ft
Max. Acceleration= 0.88 g
Earthquake Magnitude= 6.60

Input Data:

Surface Elev.=
Hole No.=B-1
Depth of Hole=51.50 ft
Water Table during Earthquake= 9.00 ft
Water Table during In-Situ Testing= 15.00 ft
Max. Acceleration=0.88 g
Earthquake Magnitude=6.60
No-Liquefiable Soils: Based on Analysis

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

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	9.00	132.00	NoLiq
1.50	9.00	132.00	NoLiq
3.00	14.00	131.00	NoLiq
5.00	4.00	131.00	NoLiq
6.50	5.00	118.00	NoLiq
10.00	7.00	118.00	NoLiq
11.50	8.00	118.00	NoLiq
15.00	5.00	118.00	NoLiq
20.00	5.00	118.00	NoLiq
25.00	5.00	118.00	NoLiq
30.00	4.00	118.00	84.00
34.00	4.00	118.00	NoLiq
36.00	12.00	118.00	10.00
37.50	32.00	118.00	10.00
40.00	33.00	118.00	10.00
45.00	28.00	118.00	11.00
48.50	28.00	118.00	11.00
50.00	36.00	118.00	11.00

Output Results:

Settlement of Saturated Sands=2.28 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=2.28 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.74	5.00	2.28	0.00	2.28
1.00	2.00	0.74	5.00	2.28	0.00	2.28
2.00	2.00	0.74	5.00	2.28	0.00	2.28
3.00	2.00	0.74	5.00	2.28	0.00	2.28
4.00	2.00	0.74	5.00	2.28	0.00	2.28
5.00	2.00	0.73	5.00	2.28	0.00	2.28
6.00	2.00	0.73	5.00	2.28	0.00	2.28
7.00	2.00	0.73	5.00	2.28	0.00	2.28
8.00	2.00	0.73	5.00	2.28	0.00	2.28
9.00	2.00	0.73	5.00	2.28	0.00	2.28
10.00	2.00	0.76	5.00	2.28	0.00	2.28
11.00	2.00	0.80	5.00	2.28	0.00	2.28
12.00	2.00	0.83	5.00	2.28	0.00	2.28
13.00	2.00	0.85	5.00	2.28	0.00	2.28
14.00	2.00	0.88	5.00	2.28	0.00	2.28
15.00	2.00	0.90	5.00	2.28	0.00	2.28
16.00	2.00	0.92	5.00	2.28	0.00	2.28
17.00	2.00	0.94	5.00	2.28	0.00	2.28
18.00	2.00	0.95	5.00	2.28	0.00	2.28
19.00	2.00	0.97	5.00	2.28	0.00	2.28
20.00	2.00	0.99	5.00	2.28	0.00	2.28
21.00	2.00	1.00	5.00	2.28	0.00	2.28
22.00	2.00	1.01	5.00	2.28	0.00	2.28
23.00	2.00	1.02	5.00	2.28	0.00	2.28
24.00	2.00	1.03	5.00	2.28	0.00	2.28
25.00	2.00	1.04	5.00	2.28	0.00	2.28
26.00	2.00	1.05	5.00	2.28	0.00	2.28
27.00	2.00	1.06	5.00	2.28	0.00	2.28
28.00	2.00	1.07	5.00	2.28	0.00	2.28
29.00	2.00	1.08	5.00	2.28	0.00	2.28
30.00	2.00	1.08	5.00	2.28	0.00	2.28
31.00	0.17	1.08	0.15*	1.91	0.00	1.91
32.00	0.17	1.08	0.15*	1.51	0.00	1.51
33.00	0.16	1.08	0.15*	1.11	0.00	1.11
34.00	0.16	1.08	0.15*	0.71	0.00	0.71
35.00	2.00	1.07	5.00	0.69	0.00	0.69
36.00	2.00	1.07	5.00	0.69	0.00	0.69
37.00	0.23	1.07	0.22*	0.39	0.00	0.39
38.00	0.69	1.06	0.65*	0.21	0.00	0.21
39.00	0.69	1.06	0.65*	0.21	0.00	0.21
40.00	0.69	1.06	0.66*	0.21	0.00	0.21
41.00	0.70	1.05	0.66*	0.21	0.00	0.21
42.00	0.69	1.05	0.66*	0.21	0.00	0.21
43.00	0.69	1.04	0.67*	0.21	0.00	0.21
44.00	0.69	1.03	0.67*	0.21	0.00	0.21
45.00	0.69	1.03	0.67*	0.21	0.00	0.21
46.00	0.69	1.02	0.67*	0.19	0.00	0.19
47.00	0.68	1.02	0.67*	0.15	0.00	0.15
48.00	0.68	1.01	0.68*	0.11	0.00	0.11
49.00	0.68	1.00	0.68*	0.06	0.00	0.06
50.00	0.68	1.00	0.68*	0.00	0.00	0.00
51.00	0.68	0.99	0.69*	0.00	0.00	0.00

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

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

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

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

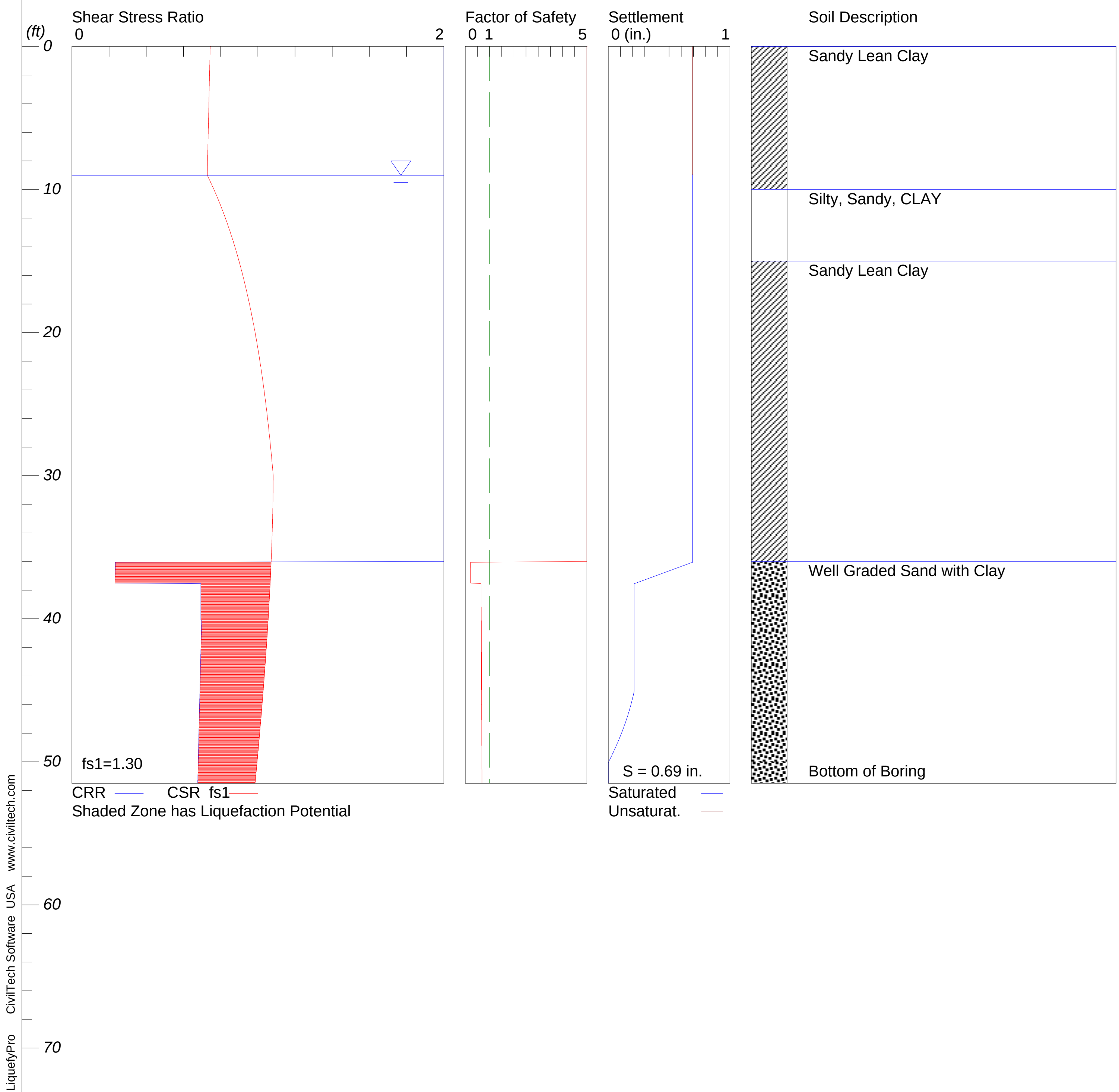
LIQUEFACTION ANALYSIS

Retail Developement - San Jose, CA

SCENARIO 2, Soils 30-34 Feet Considered Non Liquefiable

Hole No.=B-1 Water Depth=9 ft

Magnitude=6.6
Acceleration=0.88g



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LIQUEFACTION ANALYSIS SUMMARY

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Title: Retail Development - San Jose, CA
Subtitle: 5-219-0790

SCENARIO 2, Soils 30-34 Feet Considered Non Liquefiable

Hole No.=B-1
Depth of Hole= 51.50 ft
Water Table during Earthquake= 9.00 ft
Water Table during In-Situ Testing= 15.00 ft
Max. Acceleration= 0.88 g
Earthquake Magnitude= 6.60

Input Data:

Hole No.=B-1
Depth of Hole=51.50 ft
Water Table during Earthquake= 9.00 ft
Water Table during In-Situ Testing= 15.00 ft
Max. Acceleration=0.88 g
Earthquake Magnitude=6.60
No-Liquefiable Soils: Based on Analysis

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

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	9.00	132.00	NoLiq
1.50	9.00	132.00	NoLiq
3.00	14.00	131.00	NoLiq
5.00	4.00	131.00	NoLiq
6.50	5.00	118.00	NoLiq
10.00	7.00	118.00	NoLiq
11.50	8.00	118.00	NoLiq
15.00	5.00	118.00	NoLiq
20.00	5.00	118.00	NoLiq
25.00	5.00	118.00	NoLiq
30.00	4.00	118.00	NoLiq
34.00	4.00	118.00	NoLiq
36.00	12.00	118.00	10.00
37.50	32.00	118.00	10.00
40.00	33.00	118.00	10.00
45.00	28.00	118.00	11.00
48.50	28.00	118.00	11.00
50.00	36.00	118.00	11.00

Output Results:

Settlement of Saturated Sands=0.69 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=0.69 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.74	5.00	0.69	0.00	0.69
1.00	2.00	0.74	5.00	0.69	0.00	0.69
2.00	2.00	0.74	5.00	0.69	0.00	0.69
3.00	2.00	0.74	5.00	0.69	0.00	0.69
4.00	2.00	0.74	5.00	0.69	0.00	0.69
5.00	2.00	0.73	5.00	0.69	0.00	0.69
6.00	2.00	0.73	5.00	0.69	0.00	0.69
7.00	2.00	0.73	5.00	0.69	0.00	0.69
8.00	2.00	0.73	5.00	0.69	0.00	0.69
9.00	2.00	0.73	5.00	0.69	0.00	0.69
10.00	2.00	0.76	5.00	0.69	0.00	0.69
11.00	2.00	0.80	5.00	0.69	0.00	0.69
12.00	2.00	0.83	5.00	0.69	0.00	0.69
13.00	2.00	0.85	5.00	0.69	0.00	0.69
14.00	2.00	0.88	5.00	0.69	0.00	0.69
15.00	2.00	0.90	5.00	0.69	0.00	0.69
16.00	2.00	0.92	5.00	0.69	0.00	0.69
17.00	2.00	0.94	5.00	0.69	0.00	0.69
18.00	2.00	0.95	5.00	0.69	0.00	0.69
19.00	2.00	0.97	5.00	0.69	0.00	0.69
20.00	2.00	0.99	5.00	0.69	0.00	0.69
21.00	2.00	1.00	5.00	0.69	0.00	0.69
22.00	2.00	1.01	5.00	0.69	0.00	0.69
23.00	2.00	1.02	5.00	0.69	0.00	0.69
24.00	2.00	1.03	5.00	0.69	0.00	0.69
25.00	2.00	1.04	5.00	0.69	0.00	0.69
26.00	2.00	1.05	5.00	0.69	0.00	0.69
27.00	2.00	1.06	5.00	0.69	0.00	0.69
28.00	2.00	1.07	5.00	0.69	0.00	0.69
29.00	2.00	1.08	5.00	0.69	0.00	0.69
30.00	2.00	1.08	5.00	0.69	0.00	0.69
31.00	2.00	1.08	5.00	0.69	0.00	0.69
32.00	2.00	1.08	5.00	0.69	0.00	0.69
33.00	2.00	1.08	5.00	0.69	0.00	0.69
34.00	2.00	1.08	5.00	0.69	0.00	0.69
35.00	2.00	1.07	5.00	0.69	0.00	0.69
36.00	2.00	1.07	5.00	0.69	0.00	0.69
37.00	0.23	1.07	0.22*	0.39	0.00	0.39
38.00	0.69	1.06	0.65*	0.21	0.00	0.21
39.00	0.69	1.06	0.65*	0.21	0.00	0.21
40.00	0.69	1.06	0.66*	0.21	0.00	0.21
41.00	0.70	1.05	0.66*	0.21	0.00	0.21
42.00	0.69	1.05	0.66*	0.21	0.00	0.21
43.00	0.69	1.04	0.67*	0.21	0.00	0.21
44.00	0.69	1.03	0.67*	0.21	0.00	0.21
45.00	0.69	1.03	0.67*	0.21	0.00	0.21
46.00	0.69	1.02	0.67*	0.19	0.00	0.19
47.00	0.68	1.02	0.67*	0.15	0.00	0.15
48.00	0.68	1.01	0.68*	0.11	0.00	0.11
49.00	0.68	1.00	0.68*	0.06	0.00	0.06
50.00	0.68	1.00	0.68*	0.00	0.00	0.00
51.00	0.68	0.99	0.69*	0.00	0.00	0.00

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

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

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

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

APPENDIX

A

APPENDIX A

FIELD EXPLORATION

Fieldwork for our investigation was conducted on October 17, 2019 and included a site visit, subsurface exploration, and soil sampling. The locations of the exploratory borings are shown on the Site Plan, Figure 1. 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.

The test borings were advanced with 6 inch hollow-stem or solid flight augers rotated by a truck-mounted CME-55 drill rig. Visual classification of the materials encountered in the test borings was generally made in accordance with the Unified Soil Classification System (ASTM D2487).

Penetration resistance blow counts were obtained by dropping a 140-pound automated trip hammer through a 30-inch free fall to drive the sampler to a maximum penetration of 18 inches. The number of blows required to drive the last 12 inches, or less if very dense or hard, is recorded as Penetration Resistance (blows/foot) on the logs of borings. Soil samples were obtained from the test borings at the depths shown on the logs of borings. The MCS samples were recovered and capped at both ends to preserve the samples at their natural moisture content; SPT samples were recovered and placed in a sealed bag to preserve their natural moisture content. The test borings were permitted (Contra Costa County Environmental Health Division, Permit No. 0025942, Permit dated May 23, 2019), and at the completion of drilling and sampling, were inspected per county personnel and backfilled with neat cement grout per permit requirements.

SALEM engineering group, inc.

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

Project Number: 5-219-0790

Date: 10/17/2019

Client: Bergman KPRS

Project: Proposed Retail Building

Location: 7028 Santa Teresa Boulevard, San Jose, CA.

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

Drill Type: CME 45C **Elevation:** N/A

Auger Type: 6in. Hollow Stem Auger **Initial Depth to Groundwater:** Not Encountered

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

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 = 2.25 inches				
		AB	Aggregate Base = 0.73 inches				
	2/6 4/6 5/6	CL	Sandy Lean CLAY; stiff, dark brown, moist, low plasticity.	9	16.7	--	EI=51 -200=87 PI=18 LL=36 Wc/LL=0.9
	5/6 9/6 12/6		Grades as above; very stiff.	21	16.2	113.1	
5	1/6 2/6 2/6		Grades as above; soft.	4	13.0	--	
	3/6 4/6 4/6		Grades as above; firm.	8	15.9	102.4	
10	3/6 3/6 4/6	CL-ML	Silty, Sandy, Clay; firm, brown, moist, low plasticity.	7	17.5	--	-200=66% PI=5 LL=25 Wc/LL=0.7
	2/6 4/6 4/6	CL	Sandy Lean CLAY; firm, brown, moist, low plasticity.	8	23.4	--	
15	2/6 2/6 3/6		Grades as above.	5	29.2	--	
20	2/6 2/6 3/6		Grades as above.	5	26.6	--	-200=81%
25	2/6 2/6 3/6		Grades as above.	5	26.7	--	

Notes:

Figure Number A-1

SALEM engineering group, inc.

Project Number: 5-219-0790

Date: 10/17/2019

Test Boring: B-1

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
30	2/6 2/6 2/6		Grades as above; soft.	4	27.1	--	-200=84% PI=13 LL=33 Wc/LL= 0.81
35	3/6 5/6 7/6		Grades as above.	12	17.4	--	
		SW-SC	Well Graded Sand with Clay and Gravel; medium dense, brown, wet, fine to coarse grained.				
	7/6 15/6 17/6		Grades as above; dense.	32	10.8	--	
40	6/6 16/6 17/6		Grades as above.	33	9.1	--	-200=10%
45	7/6 12/6 16/6		Grades as above; medium dense.	28	8.4	--	
	6/6 12/6 16/6		Grades as above.	28	11.9	--	
50	10/6 17/6 19/6		Grades as above; dense.	36	11.2	--	-200=11%
			End of boring at 51.5ft. BSG				
55							
60							

Notes:

Figure Number A-1

SALEM engineering group, inc.

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

Project Number: 5-219-0790

Date: 10/17/2019

Client: Bergman KPRS

Project: Proposed Retail Building

Location: 7028 Santa Teresa Boulevard, San Jose, CA.

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

Drill Type: CME 45C **Elevation:** N/A

Auger Type: 6in. Solid Flight Auger **Initial Depth to Groundwater:** 19ft.

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

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 = 2.5 inches				
		AB	Aggregate Base = 2.5 inches				
	5/6 9/6 16/6	CL	Sandy Lean CLAY; very stiff, dark brown, moist, medium plasticity.	25	15.9	115.6	
	4/6 6/6 7/6		Grades as above; stiff, brown, low to meidum plasticity.	13	13.1	--	
5	8/6 8/6 11/6		Grades as above; low plasticity.	19	15.1	112.7	
	3/6 3/6 4/6		Grades as above; firm.	7	14.2	--	
10	4/6 7/6 9/6		Grades as above; stiff.	16	14.3	11.9	
15	2/6 3/6 3/6		Grades as above; firm, very moist.	6	29.9	--	
20	2/6 3/6 3/6		Grades as above; wet.	6	26.0	--	
			End of boring at 21.5ft. BSG				
25							

Notes:

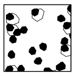
Figure Number A-2

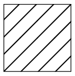
KEY TO SYMBOLS

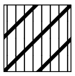
Symbol Description

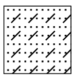
Strata symbols

 **Asphaltic Concrete**

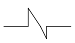
 **Aggregate Base**

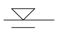
 **Lean Clay**

 **Silty low plasticity clay**

 **Well graded sand with clay**

Misc. Symbols

 **Boring continues**

 **Water table during drilling**

Soil Samplers

 **Standard penetration test**

 **California sampler**

Notes:

Consistency Classification

Blows Per Foot (Uncorrected)

Granular Soils

	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

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

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

APPENDIX

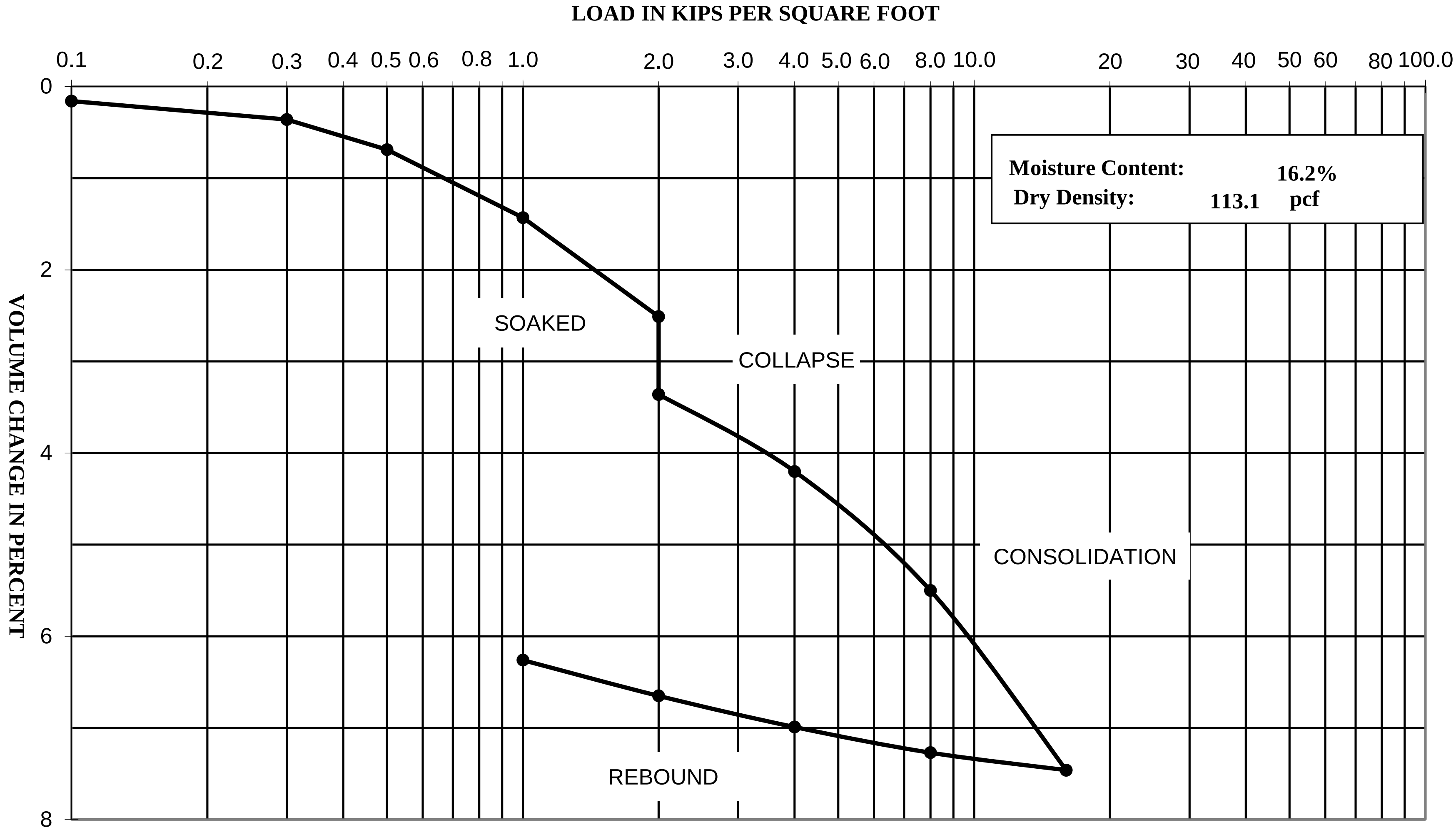
B

APPENDIX B
LABORATORY TESTING

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

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435



Project Name: Retail Building - San Jose, CA

Project Number: 5-219-0790

Boring: B-1 @ 3'

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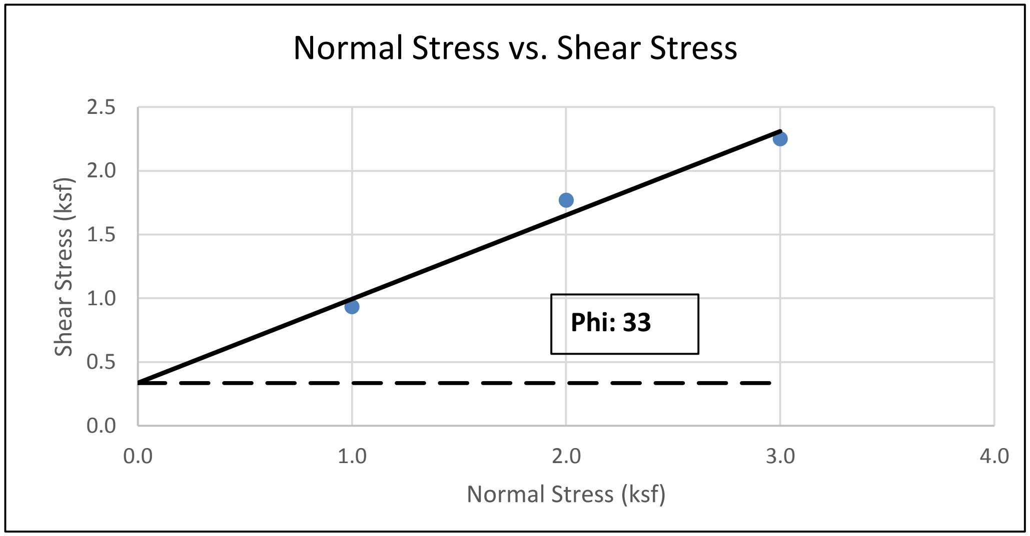
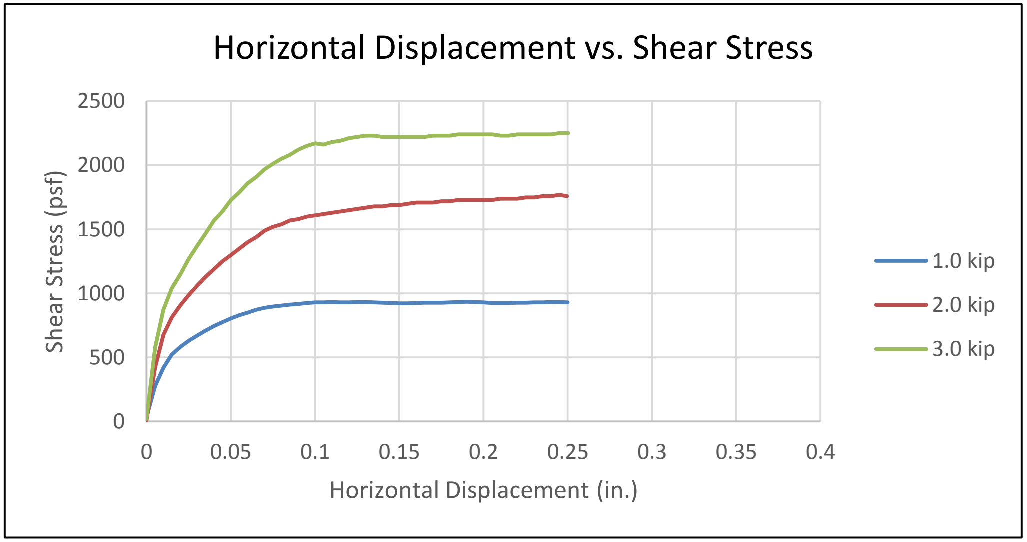
Direct Shear Test (ASTM D3080)

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Client: Bergman KPRS
Boring: B-2 @ 1.5'
Soil Type: Sandy Lean CLAY (CL)
Sample Type: Undisturbed Ring
Tested By: NL
Reviewed By: JH
Date of Test: 10/22/19
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	2.0 kip	3.0 kip
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0025	0.0025	0.0025
Peak Shear Stress (ksf)	0.93	1.77	2.25
Residual Shear Stress (ksf)	0.00	0.00	0.00

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.936	0.925	0.918
Post-Shear Sample Height (in.)	0.929	0.916	0.908
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	15.9		
Dry Density (pcf)	115.6	113.8	115.1
Saturation %	92.5	87.9	91.1
Void Ratio	0.47	0.49	0.48
Consolidated Void Ratio	0.37	0.38	0.35
Final (post-shear) Values			
Final Moisture Content (%)	20.1	19.9	20.2
Dry Density (pcf)	120.2	119.5	121.1
Saturation %	132.2	131.0	141.1
Void Ratio	0.41	0.41	0.39



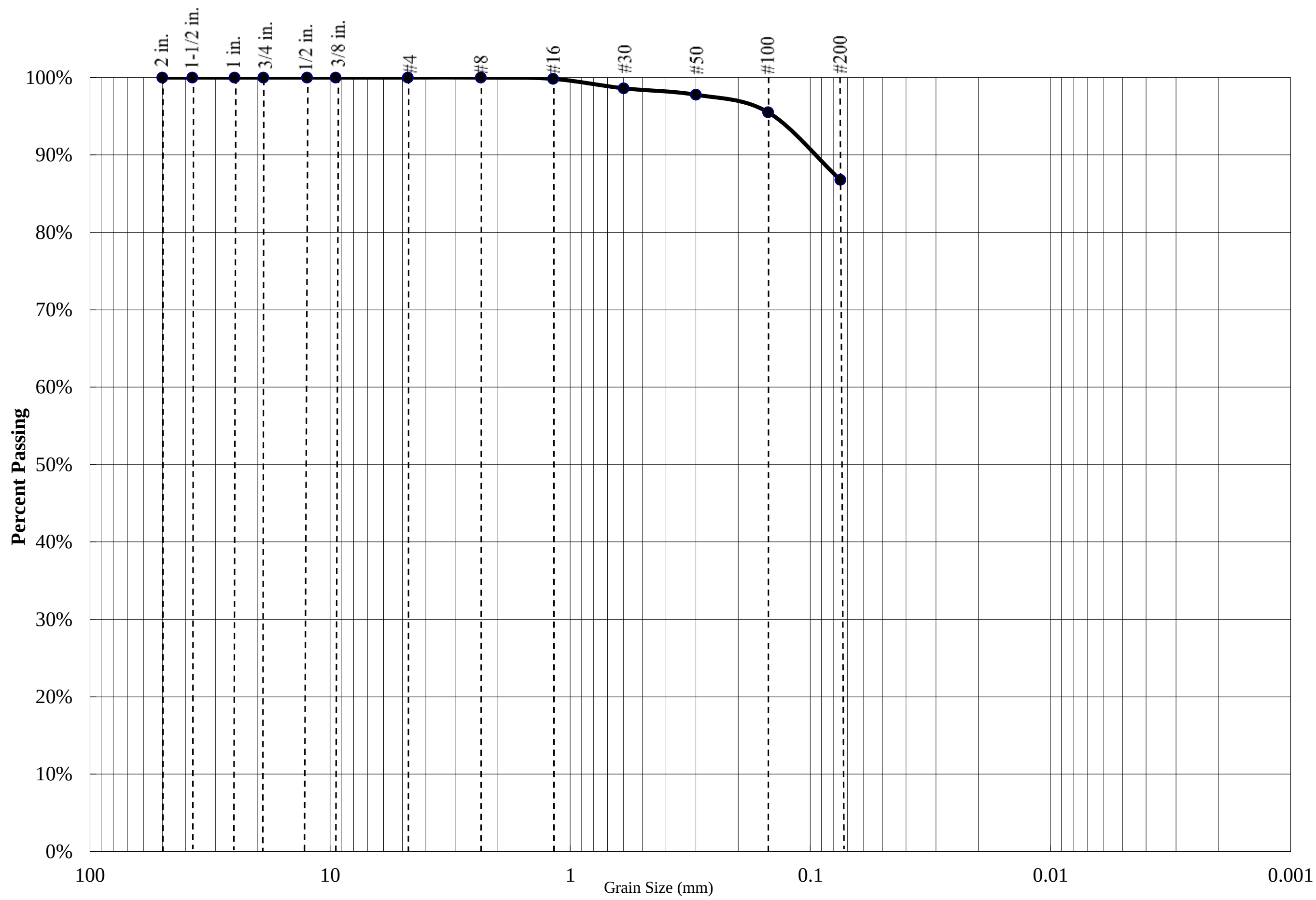


Peak Shear Strength Values		Residual Shear Strength Values	
Slope	0.66	**Slope**	0.00
Friction Angle	33	**Friction Angle**	0
Cohesion (psf)	335	**Cohesion (psf)**	0

SALEM engineering group, inc.
4729 W. Jacquelyn Ave.
Fresno, CA 93722

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	13%	87%

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	99.8%
#30	98.6%
#50	97.8%
#100	95.5%
#200	86.8%

Atterberg Limits					
PL=	18	**LL=**	36	**PI=**	18

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

USCS CLASSIFICATION
Sandy Lean CLAY (CL)

Project Name: Retail Building - San Jose, CA

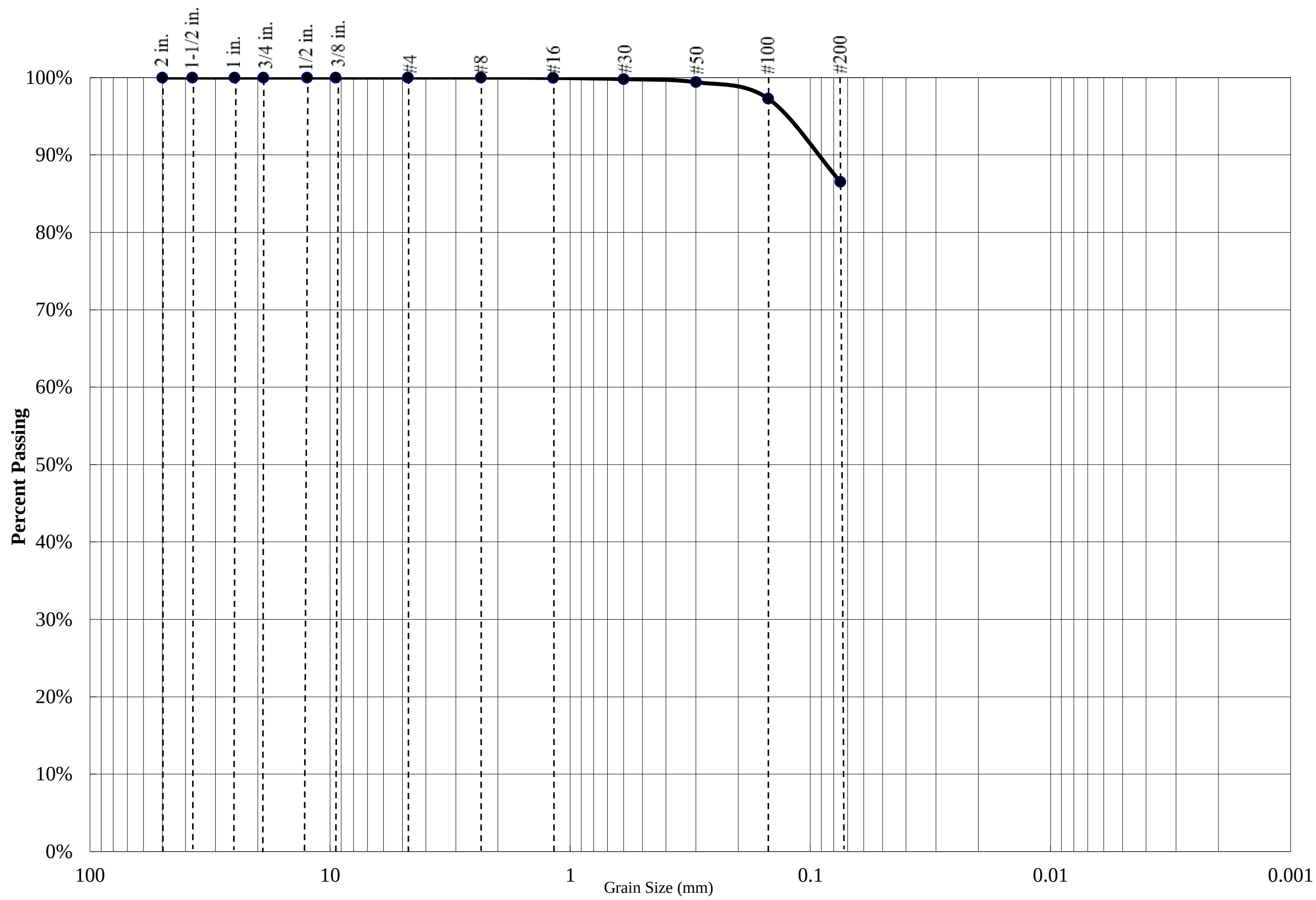
Project Number: 5-219-0790

Boring: B-1 @ 1.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	13%	87%

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	100.0%
#30	99.8%
#50	99.4%
#100	97.3%
#200	86.5%

Atterberg Limits					
PL=	N/A	**LL=**	N/A	**PI=**	N/A

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

USCS CLASSIFICATION
Sandy Lean CLAY (CL)

Project Name: Retail Building - San Jose, CA

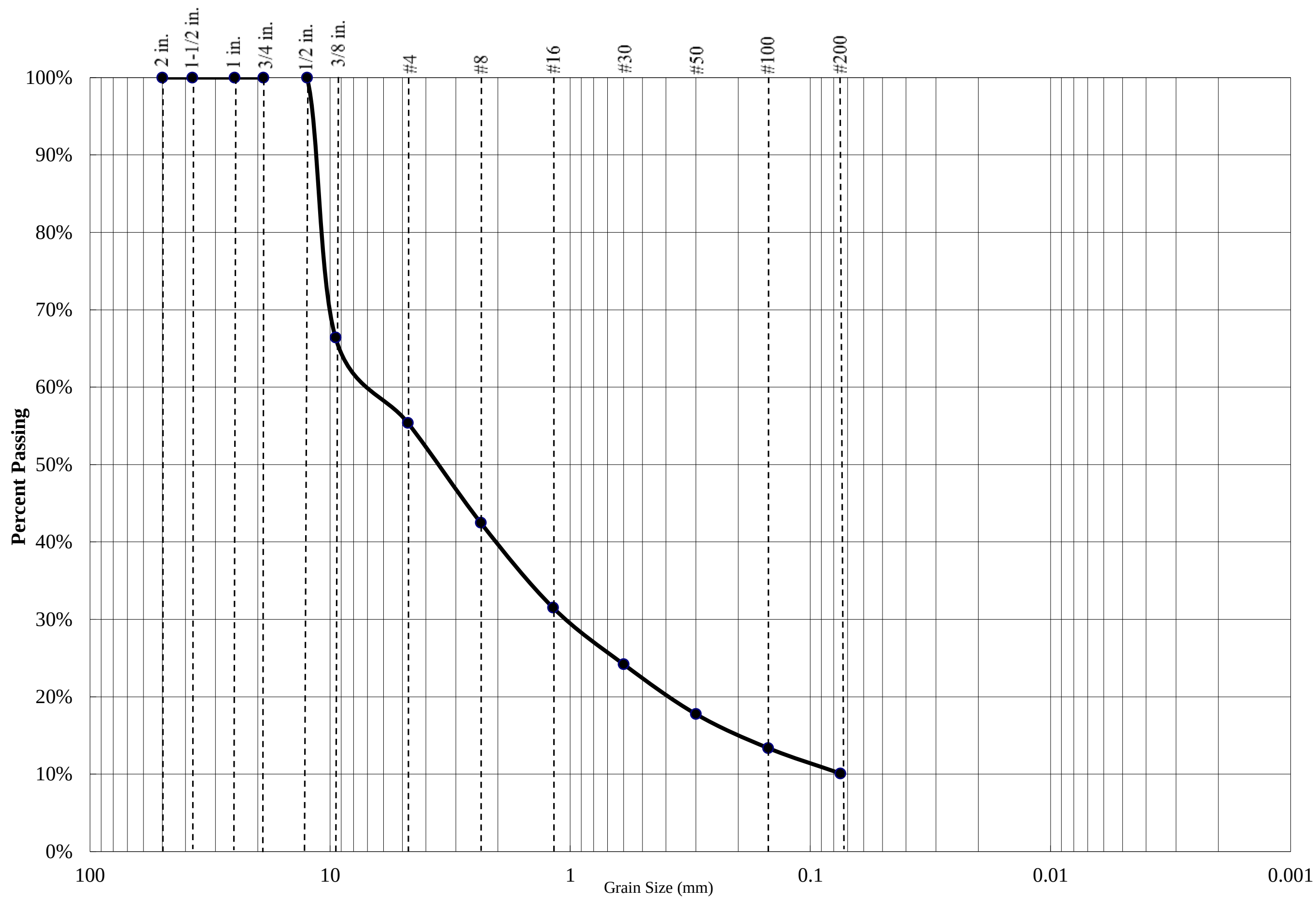
Project Number: 5-219-0790

Boring: B-1 @ 3.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
45%	45%	10%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	66.4%
#4	55.4%
#8	42.5%
#16	31.5%
#30	24.2%
#50	17.8%
#100	13.4%
#200	10.1%

Atterberg Limits					
PL=	N/A	**LL=**	N/A	**PI=**	N/A

Coefficients					
D_{85}=	13	**D_{60}=**	7	**D_{50}=**	3.6
D_{30}=	1.1	**D_{15}=**	0.2	**D_{10}=**	0.075
C_u=	93.33	**C_c=**	2.30		

USCS CLASSIFICATION
Well Graded SAND with Clay and Gravel (SW-SC)

Project Name: Retail Building - San Jose, CA

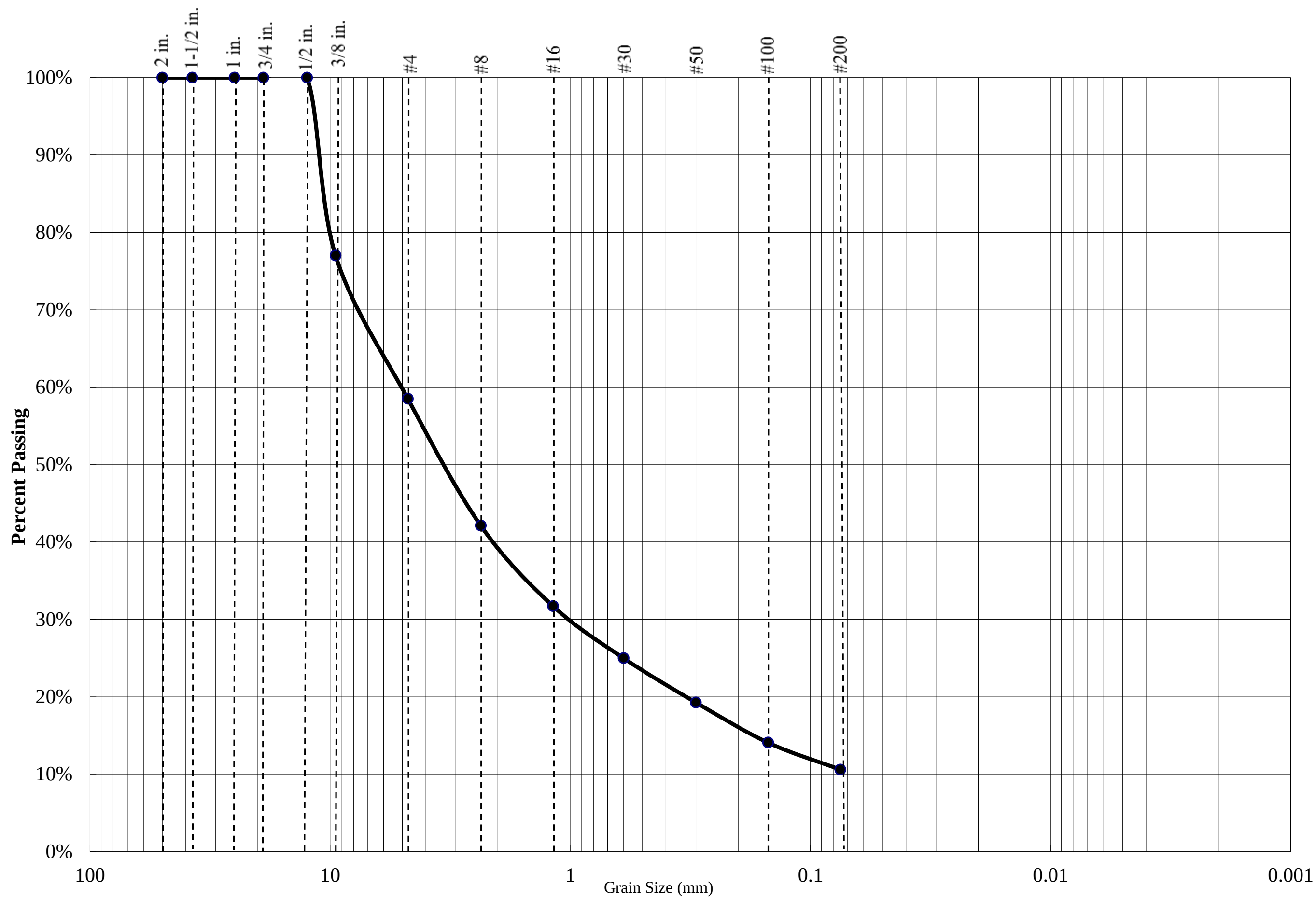
Project Number: 5-219-0790

Boring: B-1 @ 40'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
41%	48%	11%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	77.0%
#4	58.5%
#8	42.1%
#16	31.7%
#30	25.0%
#50	19.3%
#100	14.1%
#200	10.6%

Atterberg Limits					
PL=	N/A	**LL=**	N/A	**PI=**	N/A

Coefficients					
D_{85}=	12	**D_{60}=**	5	**D_{50}=**	3.5
D_{30}=	1	**D_{15}=**	0.18	**D_{10}=**	0.075
C_u=	66.67	**C_c=**	2.67		

USCS CLASSIFICATION
Well Graded SAND with Clay and Gravel (SW-SC)

Project Name: Retail Building - San Jose, CA

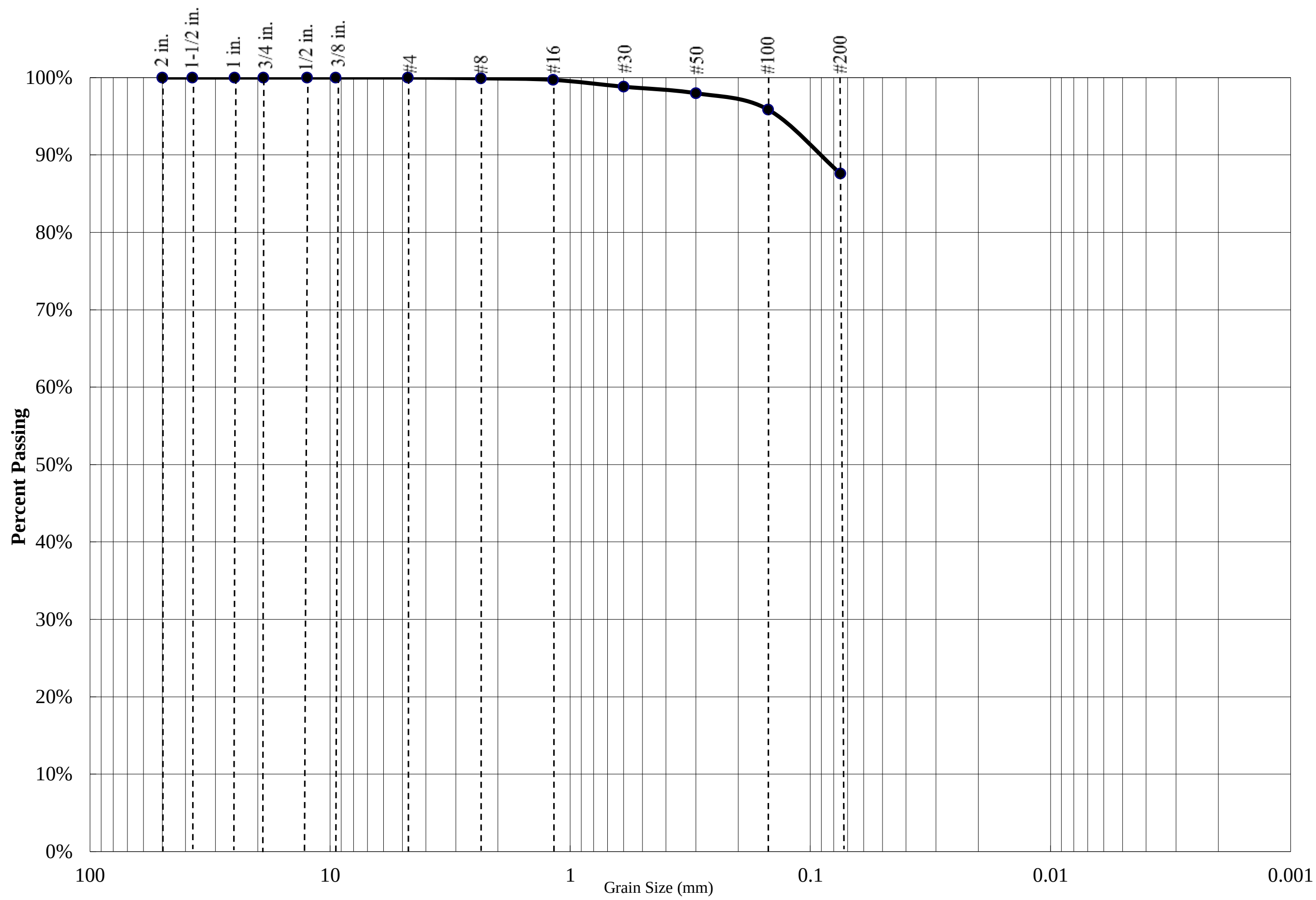
Project Number: 5-219-0790

Boring: B-1 @ 48.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	12%	88%

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	98.0%
#100	95.9%
#200	87.6%

Atterberg Limits					
PL=	N/A	**LL=**	N/A	**PI=**	N/A

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

USCS CLASSIFICATION
Sandy Lean CLAY (CL)

Project Name: Retail Building - San Jose, CA

Project Number: 5-219-0790

Boring: B-2 @ 1.5'

SALEM engineering group, inc.

Material Finer than 75-µm (No.200)

ASTM D1140

Method Used (A or B):	A	
Time Soaked:	N/A	
Max Particle Size:	<3/8"	
Determination of Dry Mass:	Directly	
Wash 200:		
Original Oven Dry + Tare:	151	g
Tare:	14.3	g
Starting Weight:	136.7	g
Oven dry + Tare :	60.7	g
Tare:	14.3	g
Soil Retained:	46.4	g
Wash 200:	66.1	% passing
% Passing	**66.1**	

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Sample Location: B-1 @ 10'

SALEM
engineering group, inc.

Material Finer than 75-µm (No.200)

ASTM D1140

Method Used (A or B):	A	
Time Soaked:	N/A	
Max Particle Size:	<3/8"	
Determination of Dry Mass:	Directly	
Wash 200:		
Original Oven Dry + Tare:	200.5	g
Tare:	14.2	g
Starting Weight:	186.3	g
Oven dry + Tare :	49.2	g
Tare:	14.2	g
Soil Retained:	35	g
Wash 200:	81.2	% passing
% Passing	**81.2**	

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Sample Location: B-1 @ 20'

SALEM engineering group, inc.

Material Finer than 75-µm (No.200)

ASTM D1140

Method Used (A or B):	A	
Time Soaked:	N/A	
Max Particle Size:	<3/8"	
Determination of Dry Mass:	Directly	
Wash 200:		
Original Oven Dry + Tare:	199.7	g
Tare:	14.4	g
Starting Weight:	185.3	g
Oven dry + Tare :	44.9	g
Tare:	14.4	g
Soil Retained:	30.5	g
Wash 200:	83.5	% passing
% Passing	**83.5**	

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Sample Location: B-1 @ 30'

SALEM engineering group, inc.

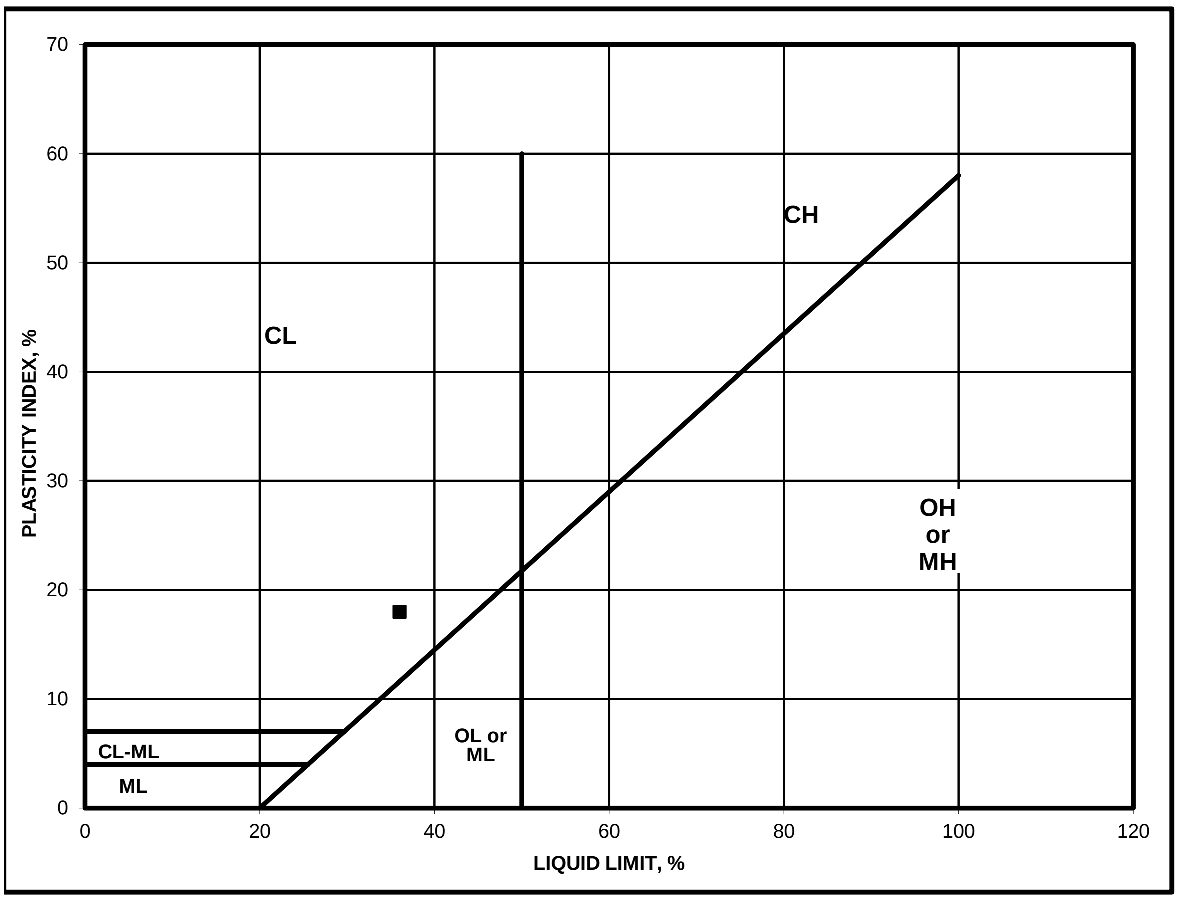
Atterberg Limits Determination
ASTM D4318

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Date Sampled: 10/17/19 Date Tested: 11110/22/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 1.5'

Run Number	Plastic Limit			Liquid Limit		
	1	2	3	1	2	3
Weight of Wet Soil & Tare	29.03	28.18	28.43	31.34	26.78	29.62
Weight of Dry Soil & Tare	27.79	27.06	27.30	28.76	25.28	27.38
Weight of Water	1.24	1.12	1.13	2.58	1.50	2.24
Weight of Tare	20.80	20.79	20.94	20.72	20.88	21.06
Weight of Dry Soil	6.99	6.27	6.36	8.04	4.40	6.32
Water Content	17.7	17.9	17.8	32.1	34.1	35.4
Number of Blows				35	30	25

Plastic Limit : 18 **Liquid Limit : 36**

Plasticity Index **: 18**
Unified Soil Classification **: CL**



SALEM
engineering group, inc.

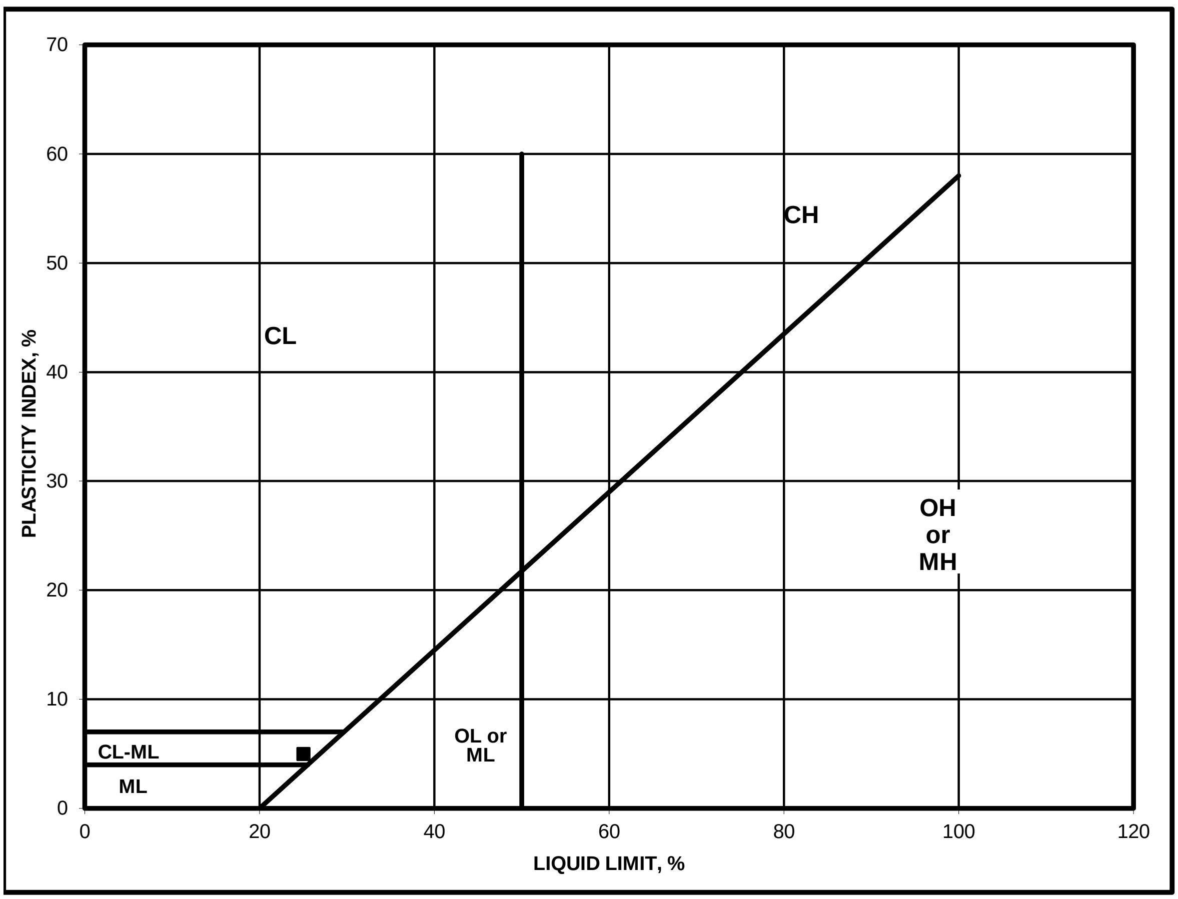
Atterberg Limits Determination
ASTM D4318

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Date Sampled: 10/17/19 Date Tested: 10/22/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 10'

Run Number	Plastic Limit 1	Plastic Limit 2	Plastic Limit 3	Liquid Limit 1	Liquid Limit 2	Liquid Limit 3
Weight of Wet Soil & Tare	28.26	29.59	27.98	36.06	31.57	33.36
Weight of Dry Soil & Tare	27.04	28.16	26.83	33.30	29.44	30.86
Weight of Water	1.22	1.43	1.15	2.76	2.13	2.50
Weight of Tare	20.91	21.07	21.12	21.04	20.90	21.04
Weight of Dry Soil	6.13	7.09	5.71	12.26	8.54	9.82
Water Content	19.9	20.2	20.1	22.5	24.9	25.5
Number of Blows				35	28	20

Plastic Limit : 20 **Liquid Limit : 25**

Plasticity Index : 5
Unified Soil Classification : CL/ML



SALEM engineering group, inc.

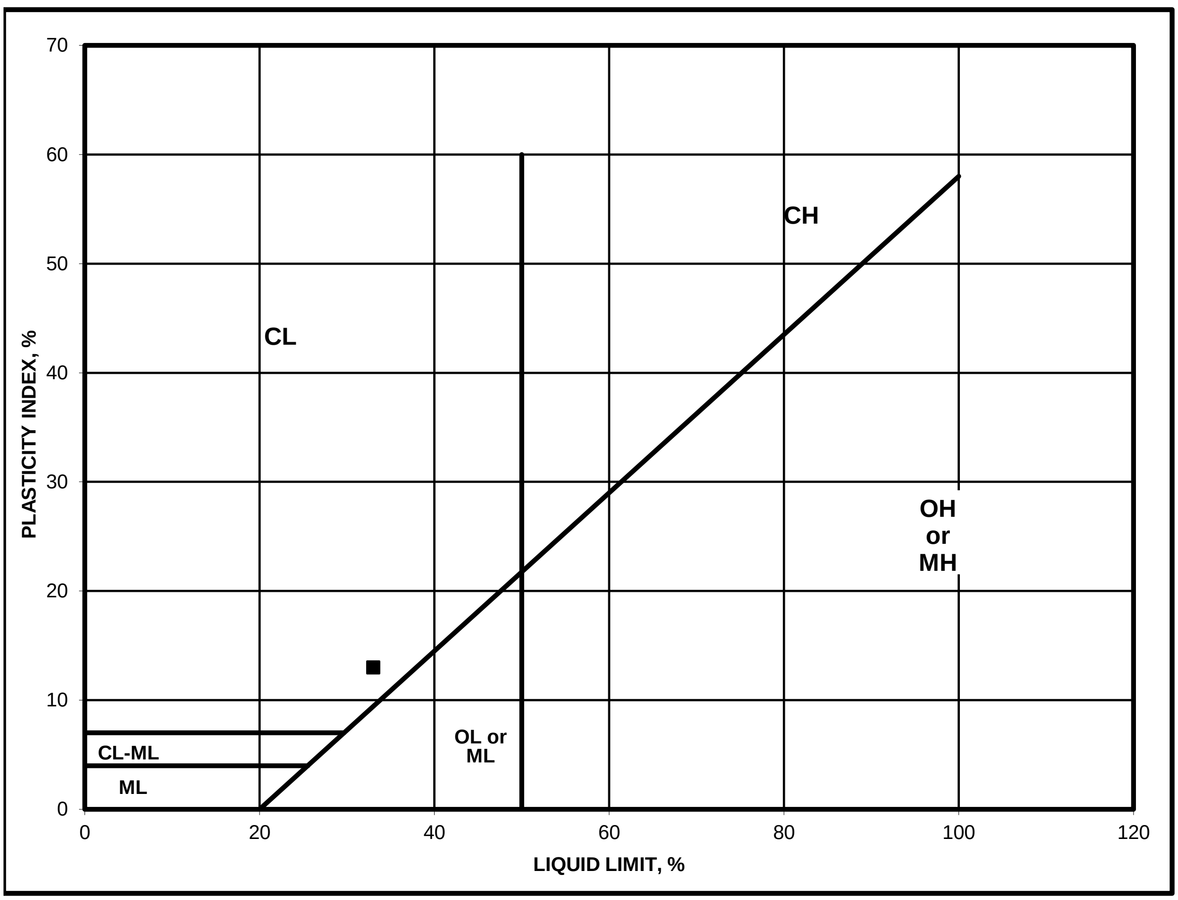
Atterberg Limits Determination
ASTM D4318

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Date Sampled: 10/17/19 Date Tested: 10/22/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 30'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare	28.42	28.55	28.95	31.91	30.35	30.82
Weight of Dry Soil & Tare	27.23	27.30	27.60	29.23	28.06	28.28
Weight of Water	1.19	1.25	1.35	2.68	2.29	2.54
Weight of Tare	21.09	21.11	21.13	20.92	21.20	20.89
Weight of Dry Soil	6.14	6.19	6.47	8.31	6.86	7.39
Water Content	19.4	20.2	20.9	32.3	33.4	34.4
Number of Blows				28	21	15

Plastic Limit : 20 **Liquid Limit : 33**

Plasticity Index **:** **13**
Unified Soil Classification **:** **CL**



SALEM engineering group, inc.

EXPANSION INDEX TEST
ASTM D4829

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Date Sampled: 10/17/19 Date Tested: 10/23/19
Sampled By: SEG Tested By: NW
Sample Location: B-1 @ 1' - 4'
Soil Description: Sandy Lean CLAY (CL)

Trial #	1	2	3
Weight of Soil & Mold, g.	579.0		
Weight of Mold, g.	188.2		
Weight of Soil, g.	390.8		
Wet Density, pcf	117.9		
Weight of Moisture Sample (Wet), g.	815.0		
Weight of Moisture Sample (Dry), g.	733.3		
Moisture Content, %	11.1		
Dry Density, pcf	106.0		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	51.1		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.0343	0.0446	0.0493	--	0.0501

Expansion Index $_{measured}$ = 50.1
Expansion Index $_{50}$ = 50.8

Expansion Index = 51

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

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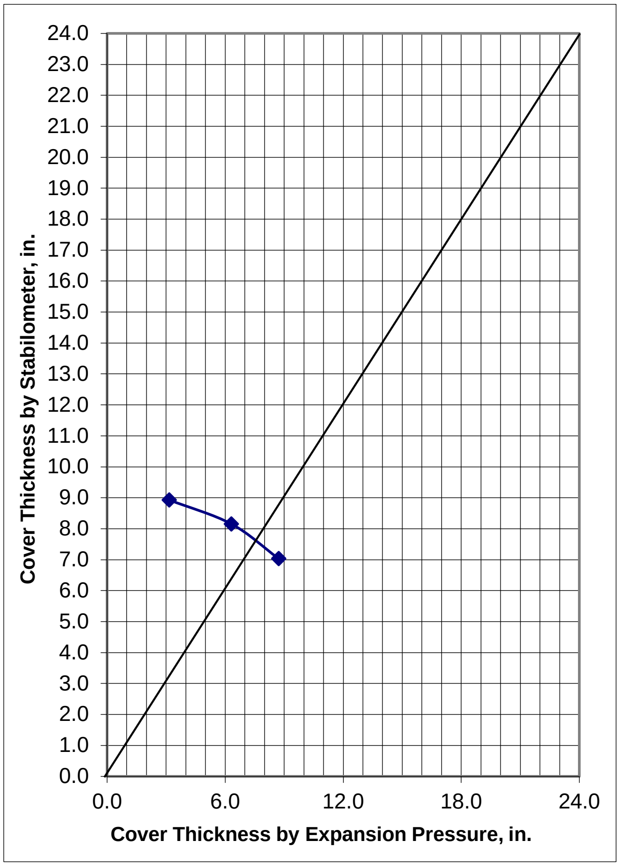
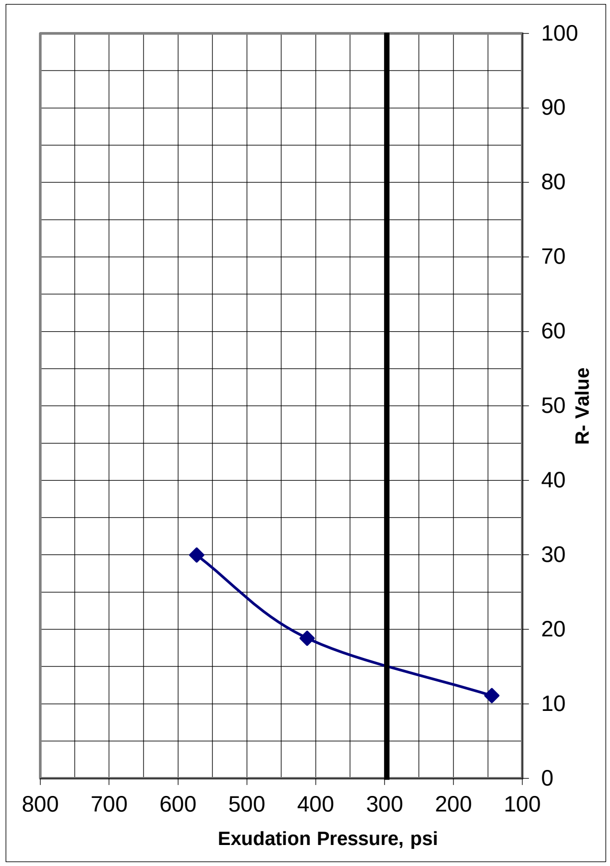
Resistance R-Value and Expansion Pressure of Compacted Soils

ASTM D2844

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Date Sampled: 10/17/19
Date Tested: 10/24/19
Sampled By: SEG
Tested By: NW
Sample Location: B-1 @ 1' - 4'
Soil Description: Sandy Lean CLAY (CL)





Specimen	1	2	3
Exudation Pressure, psi	573	412.4	144.2
Moisture at Test, %	11.5	12.1	13.1
Dry Density, pcf	124.3	121.7	119.1
Expansion Pressure, psf	944	684	342
Thickness by Stabilometer, in.	7.0	8.2	8.9
Thickness by Expansion Pressure, in.	8.7	6.3	3.2
R-Value by Stabilometer	30	19	11
R-Value by Expansion Pressure	32		
R-Value at 300 psi Exudation Pressure	15		

Controlling R-Value	**15**

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CHEMICAL ANALYSIS

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

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Date Sampled: 10/17/19 Date Tested: 10/24/19
Sampled By: SEG Tested By: HA
Soil Description: Sandy Lean CLAY (CL)

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 1' - 4'	260 mg/kg	26 mg/kg	7.6
1b.	B-1 @ 1' - 4'	260 mg/kg	25 mg/kg	7.6
1c.	B-1 @ 1' - 4'	260 mg/kg	25 mg/kg	7.6
Average:		**260 mg/kg**	**25 mg/kg**	**7.6**

SALEM engineering group, inc.

SOIL RESISTIVITY
CTM 643

Project Name: Retail Building - San Jose, CA
Project Number: 5-219-0790
Sample Location: B-1 @ 1' - 4'
Soil Description: Sandy Lean CLAY (CL)

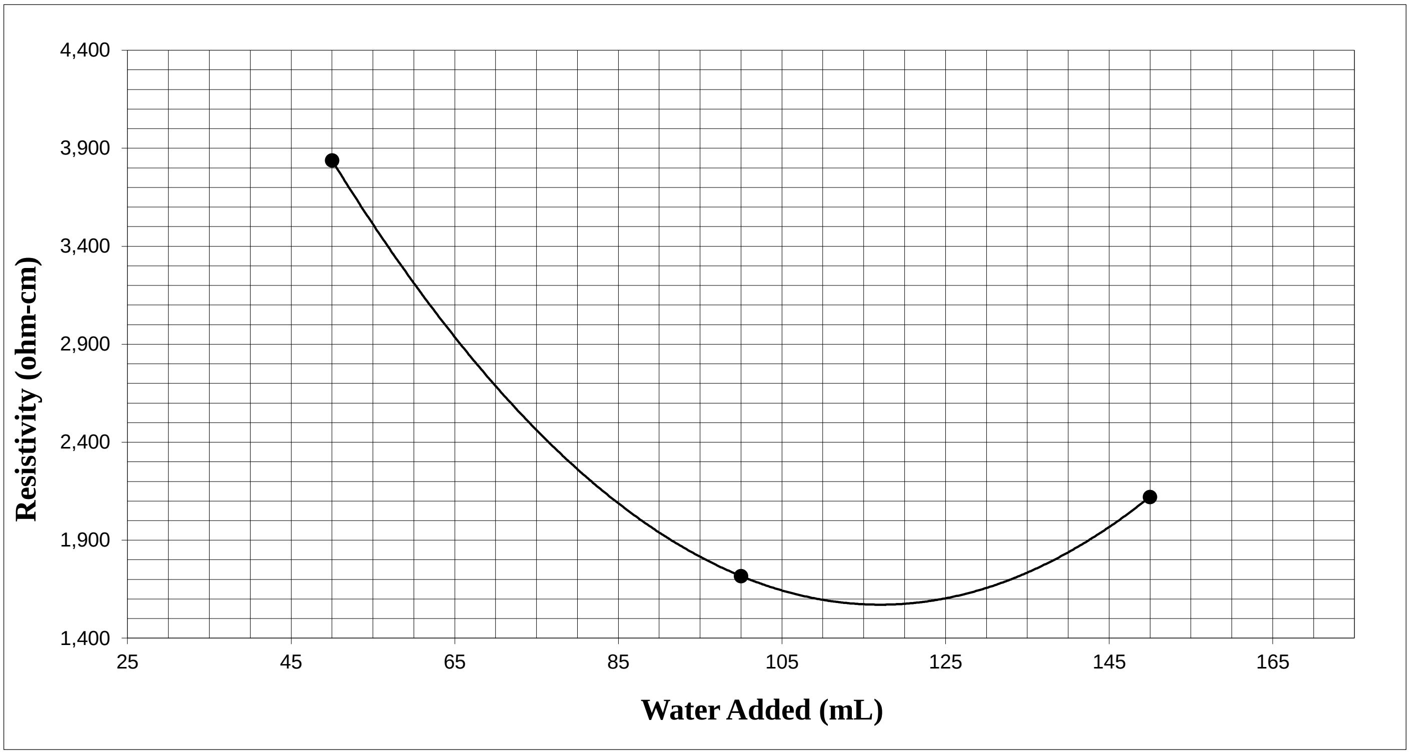
Date Sampled: 10/17/19
Sampled By: SEG
Date Tested: 10/23/19
Tested By: HA

Chloride Content: 25 mg/Kg
Sulfate Content: 260 mg/Kg
Soil pH: 7.6

Initial Sample Weight: 700 gms
Test Box Constant: 1.010 cm

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm-cm)
1	50	3.8	1,000	3,800	3,838
2	100	1.7	1,000	1,700	1,717
3	150	2.1	1,000	2,100	2,121



Minimum Resistivity:	**1,571** ohm-cm

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APPENDIX

C

APPENDIX C
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

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

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

2.0 PERFORMANCE: The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of SALEM Engineering Group, Incorporated, hereinafter referred to as the Soils Engineer and/or Testing Agency. Attainment of design grades, when achieved, shall be certified by the project Civil Engineer. Both the Soils Engineer and the Civil Engineer are the Owner's representatives. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary adjustments until all work is deemed satisfactory as determined by both the Soils Engineer and the Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Soils Engineer, Civil Engineer, or project Architect.

No earthwork shall be performed without the physical presence or approval of the Soils Engineer. The Contractor shall notify the Soils Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork.

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

3.0 TECHNICAL REQUIREMENTS: All compacted materials shall be densified to no less that 92 percent of relative compaction (based on ASTM D1557 Test Method (latest edition), 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 compacted to 92 percent relative compaction.

Loose soil areas and/or areas of disturbed soil shall be moisture-conditioned as necessary and compacted to 92 percent relative compaction. All ruts, hummocks, or other uneven surface features shall be removed by surface grading prior to placement of any fill materials. All areas which are to receive fill materials shall be approved by the Soils Engineer prior to the placement of any fill material.

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

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

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

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

12.0 DEFINITIONS - The term "pavement" shall include asphaltic concrete surfacing, untreated aggregate base, and aggregate subbase. The term "subgrade" is that portion of the area on which surfacing, base, or subbase is to be placed.

The term "Standard Specifications": hereinafter referred to, is the most recent edition of the Standard Specifications of the State of California, Department of Transportation. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as determined by ASTM D1557 Test Method (latest edition).

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

14.0 AGGREGATE BASE - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class 2 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 ASTM D1557. 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 2 Subbase material. The aggregate subbase material shall be compacted to a minimum relative compaction of 95 percent based on ASTM D1557, 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.