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

SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED RETAIL DEVELOPMENT
NWC 6TH STREET & MAIN STREET
CORONA, CALIFORNIA

SALEM PROJECT NO. 3-222-1216
DECEMBER 20, 2022

PREPARED FOR:

MS. ELIZABETH RESENDIZ
NORTHGATE GONZALES REAL ESTATE, LLC
1201 N. MAGNOLIA AVENUE
ANAHEIM, CA 92801

PREPARED BY:

SALEM ENGINEERING GROUP, INC.
8711 MONROE COURT, SUITE A
RANCHO CUCAMONGA, CA 91730
P: (909) 980-6455
F: (909) 980-6435
www.salem.net

SALEM engineering group, inc.

8711 Monroe Court, Suite A
Rancho Cucamonga, CA 91730
Phone (909) 980-6455
Fax (909) 980-6435

December 20, 2022

Project No. 3-222-1216

Ms. Elizabeth Resendiz
Northgate Gonzales Real Estate, LLC
1201 N. Magnolia Avenue
Anaheim, CA 92801

SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RETAIL DEVELOPMENT
NWC 6TH STREET & MAIN STREET
CORONA, CALIFORNIA

Dear Ms. Resendiz:

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

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Ibrahim Foud Ibrahim, PE
Senior Managing Engineer
RCE 86724

Clarence Jiang, GE
Senior Geotechnical Engineer
RGE 2477

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

8711 Monroe Court, Suite A
Rancho Cucamonga, CA 91730
Phone (909) 980-6455
Fax (909) 980-6435

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RETAIL DEVELOPMENT
NWC 6TH STREET & MAIN STREET
CORONA, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation for the site of the Proposed Retail Development to be located at the northwest corner of the intersection of 6th Street and Main Street in the city of Corona, California (see Figure 1, Vicinity Map).

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

The scope of this investigation included a field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on December 1, 2022, and included the drilling of eight (8) small-diameter soil borings to a maximum depth of 21½ feet at the site. Additionally, two (2) percolation tests were performed at depths of approximately 2½ and 4 feet below ground surface to determine the infiltration rates. The locations of the soil borings and percolation tests are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation, exploratory boring logs, and percolation tests are presented in Appendix A.

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

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

2. PROJECT DESCRIPTION

Based on the Site Plan provided to us, we understand that the proposed development of the site will include demolition of two existing commercial buildings and construction of a 35,000 square-foot market building and an 8,000 square-foot restaurant/shops building, and remodel of an existing 11,273 square-foot building. Maximum wall load is expected to be on the order of 3 kips per linear foot. Maximum column load is expected to be on the order of 50 kips. Floor slab soil bearing pressure is expected to be on the order of 150 psf.

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

3. SITE LOCATION AND DESCRIPTION

The subject site is near rectangular in shape and is located at the northwest corner of the intersection of 6th Street and Main Street in the city of Corona, California (see Vicinity Map, Figure 1). The site extends to 4th Street to the north, with 5th street dividing the northern and southern halves of the site.

At the time of our field exploration, the site was predominately developed with 3 commercial buildings and a drive-thru kiosk with associated asphalt concrete pavement and landscaping. The northern portion of the site was mostly vacant with miscellaneous grass, and former slabs and paved areas. The site is gently sloping to the north with elevations ranging from 669 to 647 feet above mean sea level based on Google Earth imagery.

Based on available historical imagery, the northern portion of the site was previously occupied by single-family residences and a commercial/industrial building. Those buildings were demolished starting from around 2005 through 2013.

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-8) were drilled on December 1, 2022, at the approximate locations shown on the Site Plan, Figure 2. The test borings were advanced with 6⅝-inch hollow stem augers rotated by a truck-mounted CME 75 drill rig. The test borings were extended to a maximum depth of 21½ feet below existing grade. Drilling depth was limited at borings B-1 through B-4 due to auger refusal on gravel and cobbles.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer and stratification lines were approximated on the basis of observations made at the time of drilling. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2488). A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix "A." The logs of the test borings are presented in Appendix "A." The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol.

The location of the test borings were determined by measuring from features shown on the Site Plan provided to us. Hence, accuracy can be implied only to the degree that this method warrants. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted.

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

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

5. LABORATORY TESTING

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

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

6. GEOLOGIC SETTING

The subject site is located within the Inland Valley, within the Peninsular Ranges Geomorphic Province of California. The Inland Valley is situated between the San Bernardino Mountains to the northeast, the San Gabriel Mountains to the north, the Chino Hills to the southwest, and to the southeast by the hilly uplands that separate it from the San Jacinto Basin. These mountain ranges are part of the Transverse Ranges Geomorphic Province of California.

The Inland Valley is dominated by northwest-trending faults and adjacent anticlinal uplifts. The intervening deep synclinal troughs are filled with poorly consolidated Upper Pleistocene and unconsolidated Holocene sediments. Tectonism of the region is dominated by the interaction of the East Pacific Plate and the North American Plate along a transform boundary. The Inland Valley has been filled with a variable thickness of relatively young, heterogeneous alluvial deposits. Deposits encountered on the subject site during exploratory drilling are discussed in detail in this report.

7. GEOLOGIC HAZARDS

7.1 Faulting and Seismicity

Based on the proximity of several dominant active faults and seismogenic structures, as well as the historic seismic record, the area of the subject site is considered subject to relatively high 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 project vicinity. The project area is not within an Alquist-Priolo Earthquake Fault (Special Studies) Zone and will not require a special site investigation by an Engineering Geologist. Soils on site are classified as Site Class D – Default in accordance with Chapter 16 of the California Building Code.

The proposed structures are determined to be in **Seismic Design Category E**. To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 33.8774° North; site longitude is 117.5679° West. The ten closest active faults are summarized in Table 7.1.

TABLE 7.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Max. Earthquake Magnitude, M_w
Chino; alt 2	2.3	6.8
Chino; alt 1	2.4	6.7
Elsinore; W+GI+T+J+CM	3.6	7.9
Elsinore; GI+T+J+CM	3.6	7.7
San Jose	17.1	6.7
Puente Hills (Coyote Hills)	17.3	6.9
Elsinore; T+J+CM	17.7	7.6
San Joaquin Hills	18.6	7.1
Cucamonga	19.4	6.7
Sierra Madre Connected	19.6	7.3

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

7.2 Surface Fault Rupture

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

7.3 Ground Shaking

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

Based on the 2019 CBC, a Site Class D – Default represents the on-site soil conditions. A table providing the recommended design acceleration parameters for the project site, based on the Site Class D – Default designation, is included in Section 9.2.1 of this report.

Based on the Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 1.026g (based on both probabilistic and deterministic seismic ground motion).

7.4 Liquefaction

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

The soils encountered within the depth of 21½ feet on the project site consisted predominately of loose to very dense silty sand with various amounts of gravel, gravelly silty sand and sandy gravel; and firm to hard sandy silt with various amounts of gravel. The historically highest groundwater is estimated to be at a depth of greater than 50 feet below ground surface according to the regional groundwater data. In according with the Riverside County Office of Information Technology GIS, the site is located within a low liquefaction potential zone. Based on the depth to groundwater, the liquefaction potential of the site is considered to be low and mitigation measures are not warranted.

7.5 Lateral Spreading

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

7.6 Landslides

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

7.7 Tsunamis and Seiches

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

8. SOIL AND GROUNDWATER CONDITIONS

8.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils within the depth of exploration consisted of loose to very dense silty sand with various amounts of gravel, gravelly silty sand and sandy gravel; and firm to hard sandy silt with various amounts of gravel.

Fill soils are expected to be present onsite between our test boring locations since the site was graded for the previous and current developments. The consistency of the fill materials should be verified during site grading. Prior to fill placement, Salem Engineering Group, Inc. should inspect the bottom of the excavation to verify no additional excavation will be required.

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

8.2 Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was not encountered during this investigation. The historically highest groundwater within the site vicinity is estimated to be at a depth greater than 50 feet below ground surface according to regional groundwater well data.

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

8.3 Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate and chloride levels and how they relate to cement reactivity with soil and/or water. A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil sample was detected to be less than 50 mg/kg. ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class.

ACI 318 requirements for site concrete based upon soluble sulfate are summarized in Table 8.3 below.

TABLE 8.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

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

The water-soluble chloride concentration detected in saturation extract from the soil samples was 50 mg/kg. This level of chloride concentration is considered to be mildly corrosive. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, applicable manufacturer's recommendations for corrosion protection of buried metal pipe be closely followed.

8.4 Percolation Testing

Two percolation tests (P-1 and P-2) were performed within assumed infiltration areas and were conducted in accordance with the guidelines established by the County of Riverside. Results of the falling head tests are presented in the attachments to this report.

The approximate locations of the percolation tests are shown on the attached Site Plan, Figure 2. The boreholes were advanced to the depths shown on the percolation test worksheets. The holes were pre-saturated before percolation testing commenced. Percolation rates were measured by filling the test holes with clean water and measuring the water drops at a certain time interval. The difference in the percolation rates are reflected by the varied type of soil materials at the bottom of the test holes. The test results are shown on the table below.

TABLE 8.4
PERCOLATION TEST RESULTS

Test No.	Depth (Feet)	Tested Infiltration Rate[1] (inch/hour)	Design Infiltration Rate[2] (inch/hour)	Soil Type[3]
P-1	3	**0.66**	0.22	Silty SAND (SM)
P-2	4	**0.42**	0.14	Silty SAND (SM)

[1] Tested infiltration Rate = (ΔH 60 r) / (Δt(r + 2H_{avg}))
[2] FS=3 according to the Riverside County – Low Impact Development BMP Design Handbook
[3] At bottom of drilled holes

The soil infiltration rate is based on test conducted with clear water. The infiltration rate may vary with time as a result of soil clogging from water impurities. The infiltration rate will deteriorate over time due to the soil conditions and an appropriate factor of safety (FS) may be applied. SALEM recommends a minimum factor of safety of 3 be used in design. The soils may also become less permeable to impermeable if the soil is compacted. Thus, periodic maintenance consisting of clearing the bottom of the drainage system of clogged soils should be expected. The infiltration rate may become slower if the surrounding soil is wet or saturated due to prolonged rainfalls. Additional infiltration tests should be conducted at bottom of the drainage system during construction to verify the infiltration rate. Groundwater, if closer to the bottom of the drainage system, will also reduce the infiltration rate.

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

The geotechnical engineering information presented herein is based upon professional interpretation utilizing standard engineering practices. The work conducted through the course of this investigation, including the preparation of this report, has been performed in accordance with the generally accepted standards of geotechnical engineering practice, which existed in the geographic area at the time the report was written. No other warranty, express or implied, is made. Please be advised that when performing percolation testing services in relatively small diameter borings, that the testing may not fully model the actual full scale long term performance of a given site. This is particularly true where percolation test data is to be used in the design of large infiltration system such as may be proposed for the site. The measured percolation rate includes dispersion of the water at the sidewalls of the boring as well as into the underlying soils. Subsurface conditions, including percolation rates, can change over time as fine-grained soils migrate. It is not warranted that such information and interpretation cannot be superseded by future geotechnical engineering developments. We emphasize that this report is valid for the project outlined above and should not be used for any other sites.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

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

9.1.2 The primary geotechnical constraints identified in our investigation is the presence of potentially compressible (collapsible) materials at the site. Recommendations to mitigate the effects of these soils are provided in this report.

9.1.3 No significant fill soils were encountered in our test borings. Fill soils are anticipated to be present onsite between our test boring locations since the site was graded for the former and current developments. Undocumented fill materials are not suitable to support any future structures and should be excavated and replaced with Engineered Fill. Prior to fill placement, Salem Engineering Group, Inc. should inspect the bottom of the excavation to verify no additional excavation will be required. Verification of the extent of fill should be determined during site grading.

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

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

9.1.6 The near-surface onsite soils are moisture-sensitive and are anticipated to be moderately compressible (collapsible) under saturated conditions. Proposed structures may experience excessive post-construction settlement, when the foundation soil become near saturated. The compressible or weak soils should be removed and re-compacted according to the recommendations in the Grading section of this report (Section 9.5).

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

9.1.8 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 buildings will be within 1 inch and corresponding differential settlement will be less than ½ inch over a horizontal distance of 20 feet.

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

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

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

9.2 Seismic Design Criteria

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

TABLE 9.2.1
SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2016 ASCE 7 or 2019 CBC Reference
Site Coordinates (Datum = NAD 83)		33.8774 Lat -117.5679 Lon	
Site Class	--	D	ASCE 7 Table 20.3-1
Soil Profile Name	--	Default	ASCE 7 Table 20.3-1
Risk Category	--	II	Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.2	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	1.026g	ASCE 7 Equation 11.8-1
Seismic Design Category	SDC	**E**	CBC Table 1613.2.5
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	2.037 g	CBC Figure 1613.2.1(1-8)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.772 g	CBC Figure 1613.2.1(1-8)
Site Class Modified Site Coefficient	F_a	1.2	CBC Figure 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	1.7*	CBC Figure 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	2.444 g	CBC Equation 16-36
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	1.312* g	CBC Equation 16-37
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.629 g	CBC Equation 16-38
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	0.875* g	CBC Equation 16-39
Short Term Transition Period (S_{D1}/S_{DS}), Seconds	T_S	0.537	ASCE 7-16, Section 11.4.6
Long Period Transition Period (seconds)	T_L	8	ASCE 7-16, Figure 22-14

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

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

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

9.3 Soil and Excavation Characteristics

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

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

9.3.3 The upper soils are moisture-sensitive and moderately collapsible under saturated conditions. These soils, in their present condition, possess moderate risk to construction in terms of possible post-construction movement of the foundations and floor systems if no mitigation measures are employed. Accordingly, measures are considered necessary to reduce anticipated collapse potential. Mitigation measures will not eliminate post-construction soil movement, but will reduce the soil movement. Success of the mitigation measures will depend on the thoroughness of the contractor in dealing with the soil conditions.

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

9.4 Materials for Fill

9.4.1 Excavated soils generated from cut operations at the site are suitable for use as general Engineered Fill in structural areas provided they do not contain deleterious matter, organic material, or <u>rocks larger than 3 inches in maximum dimension</u>.

9.4.2 Rocks greater than 3 inches but less than 8 inches in size may be placed below a minimum depth of 2 feet of finish grade as engineered fill provided they comprise less than 20 percent of the fill,. The oversized rocks should be placed in such a manner as to assure the filling of all voids around the rocks and with sufficient well graded soils to avoid any rock-to-rock contact. Rocks over 8 inches in size should not be used as Engineered Fill. Any areas containing insufficient fines or with rock nesting conditions should be reworked with ample water and additional fines to the satisfaction of the geotechnical consultant.

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

9.4.4 Import soil shall be well-graded, slightly cohesive silty fine sand or sandy silt, with relatively impervious characteristics when compacted. A clean sand or very sandy soil is not acceptable for this purpose. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized in Table 9.4.4 on next page.

TABLE 9.4.4
IMPORT FILL REQUIREMENTS

Minimum Percent Passing No. 200 Sieve	15
Maximum Percent Passing No. 200 Sieve	50
Minimum Percent Passing No. 4 Sieve	70
Maximum Particle Size	3"
Maximum Plasticity Index	10
Maximum CBC Expansion Index	15

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

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

9.5 Grading

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

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

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

9.5.4 Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. The upper 2 to 4 inches of soil 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 or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

9.5.5 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 ½ inch in diameter. Tree roots removed in parking areas may be limited to the upper 2 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.

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

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

9.5.8 Rocks greater than 3 inches but less than 8 inches in size may be placed below a minimum depth of 2 feet of finish grade as engineered fill provided they comprise less than 20 percent of the fill,. The oversized rocks should be placed in such a manner as to assure the filling of all voids around the rocks and with sufficient well graded soils to avoid any rock-to-rock contact. Rocks over 8 inches in size should not be used as Engineered Fill. Any areas containing insufficient fines or with rock nesting conditions should be reworked with ample water and additional fines to the satisfaction of the geotechnical consultant.

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

9.5.10 Within pavement areas, it is recommended that overexcavation and recompaction be performed to a minimum depth of **one (1) foot** below existing grade or finished grade, whichever is deeper. The subgrade should be uniformly moisture-conditioned to near optimum moisture content and compacted to at least 95% relative compaction.

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

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

9.5.13 Engineered Fill soils should be moisture conditioned to near optimum moisture content and compacted to at least 95% of the maximum dry density based on ASTM D1557-07 Test Method.

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

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

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

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

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

9.5.18 Wet soils may become non conducive to site grading as the upper soils yield under the weight of the construction equipment. Therefore, mitigation measures should be performed for stabilization.

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

The most common remedial measure of stabilizing the bottom of the excavation due to wet soil condition is to reduce the moisture of the soil to near the optimum moisture content by having the subgrade soils scarified and aerated or mixed with drier soils prior to compacting. However, the drying process may require an extended period of time and delay the construction operation.

To expedite the stabilizing process, slurry or crushed rock may be utilized for stabilization provided this method is approved by the owner for the cost purpose. If the use of slurry or crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of 2-sack slurry or ¾-inch to 1-inch crushed rocks. The thickness of the slurry or rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of slurry or crushed rock material will provide a stable platform.

It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock. A layer of geofabric is recommended to be placed on top of the compacted crushed rock to minimize migration of soil particles into the voids of the crushed rock, resulting in soil movement. Although it is not required, the use of geogrid (e.g. Tensar NX750) below the crushed rock will enhance stability and reduce the required thickness of crushed rock necessary for stabilization.

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

9.6 Shallow Foundations

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

9.6.2 The bearing wall footings considered for the structure should be continuous with a minimum width of 15 inches and extend to a minimum depth of 18 inches below the lowest adjacent soil grade. Isolated column footings should have a minimum width of 24 inches and extend a minimum depth of 18 inches below the lowest adjacent soil grade. Footing depth should be measured at the time of footing trench excavation not to include any future material (e.g. base, concrete, asphalt, etc.) over the subgrade.

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

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

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

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

9.6.6 Resistance to lateral footing displacement can be computed using an allowable coefficient of friction factor of 0.43 acting between the base of foundations and the supporting native subgrade.

9.6.7 Lateral resistance for footings can alternatively be developed using an equivalent fluid passive pressure of 320 pounds per cubic foot acting against the appropriate vertical native footing faces. An increase of one-third is permitted when using the alternate load combination that includes wind or earthquake loads. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance.

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

9.6.9 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.

9.7 Concrete Slabs-on-Grade

9.7.1 The following recommendations are intended for lightly loaded interior 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 six (6) inches of clean crushed aggregate base (CAB) over the depth of engineered fill recommended in section 9.5 of this report. The CAB should meet the Greenbook requirements and be compacted to at least 95% relative compaction.

9.7.2 Crushed Miscellaneous or Recycled Base (CMB) containing recycled materials should not be used as granular aggregate subbase within the building areas.

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

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

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

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

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

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

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

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

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

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

9.8 Lateral Earth Pressures and Frictional Resistance

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

Lateral Pressure Level Backfill and Drained Conditions	Equivalent Fluid Pressure, pcf
Active Pressure	35
At-Rest Pressure	55
Passive Pressure	320
Related Parameters	
Allowable Coefficient of Friction	0.43
In-Place Soil Density (lbs/ft^3)	120

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

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

9.8.4 A safety factor consistent with the design conditions should be included when using the values in the above table.

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

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

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

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

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

9.9 Retaining Walls

9.9.1 Retaining and/or below grade walls should be drained with either perforated pipe encased in free-draining gravel or a prefabricated drainage system. The gravel zone should have a minimum width of 12 inches wide and should extend upward to within 12 inches of the top of the wall. The upper 12 inches of backfill should consist of native soils, concrete, asphaltic-concrete or other suitable backfill to minimize surface drainage into the wall drain system. The gravel should be completely wrapped in nonwoven polypropylene geotextiles (filter fabric) to minimize migration of soil particles into the voids of the crushed rock.

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

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

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

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

9.10 Temporary Excavations

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

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

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

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

RECOMMENDED EXCAVATION SLOPES

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

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

9.10.6 Braced shorings should be designed for a maximum pressure distribution of 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.

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

9.11 Underground Utilities

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

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

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

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

9.12 Surface Drainage

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

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

9.12.3 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.

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

9.13 Pavement Design

9.13.1 Based on site soil conditions, an R-value of 30 was assumed for the preliminary flexible asphaltic concrete pavement design. The R-value may be verified during grading of the pavement areas.

9.13.2 The asphaltic concrete (flexible pavement is based on a 20-year pavement life for traffic indexes of 5.0 and 6.0. If higher traffic loading is anticipated, SALEM should be contacted to provide revised pavement thickness recommendations.

TABLE 9.13.2
ASPHALT CONCRETE PAVEMENT THICKNESSES

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

95% compaction based on ASTM D1557-07 Test Method

9.13.3 The following recommendations are for light-duty and medium-duty Portland Cement Concrete pavement sections.

TABLE 9.13.3
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

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

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

10. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

10.1 Plan and Specification Review

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

10.2 Construction Observation and Testing Services

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

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

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

11. LIMITATIONS AND CHANGED CONDITIONS

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

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

If site conditions change due to natural processes or human intervention on the property or adjacent to the site, or changes occur in the nature or design of the project, or if there is a substantial time lapse between the submission of this report and the start of the work at the site, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed by SALEM and the conclusions of our report are modified or verified in writing.

The validity of the recommendations contained in this report is also dependent upon an adequate testing and observations program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction. SALEM has prepared this report for the exclusive use of the owner and project design consultants.

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

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

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

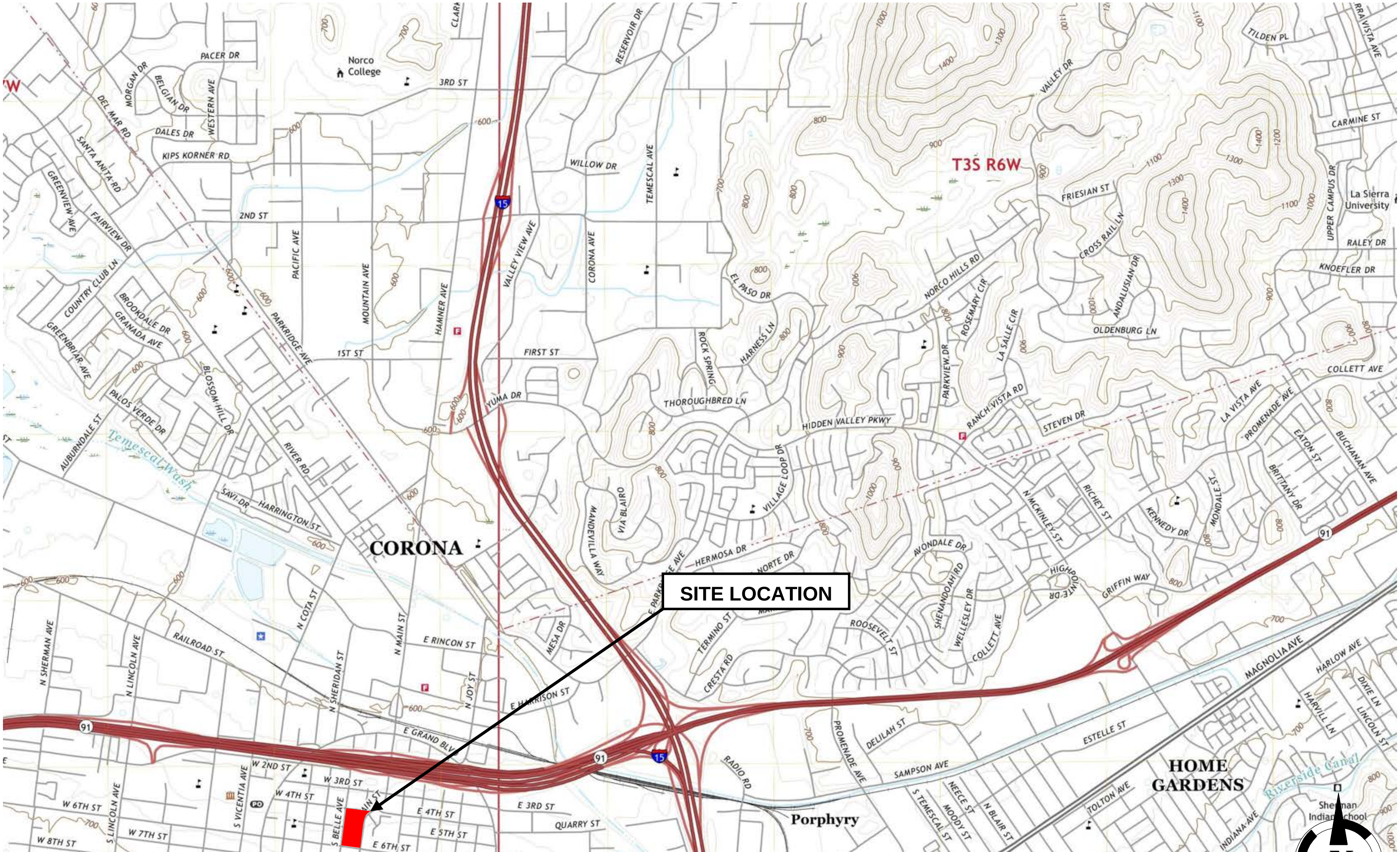
Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Jared Christiansen, MS, EIT
Geotechnical Staff Engineer

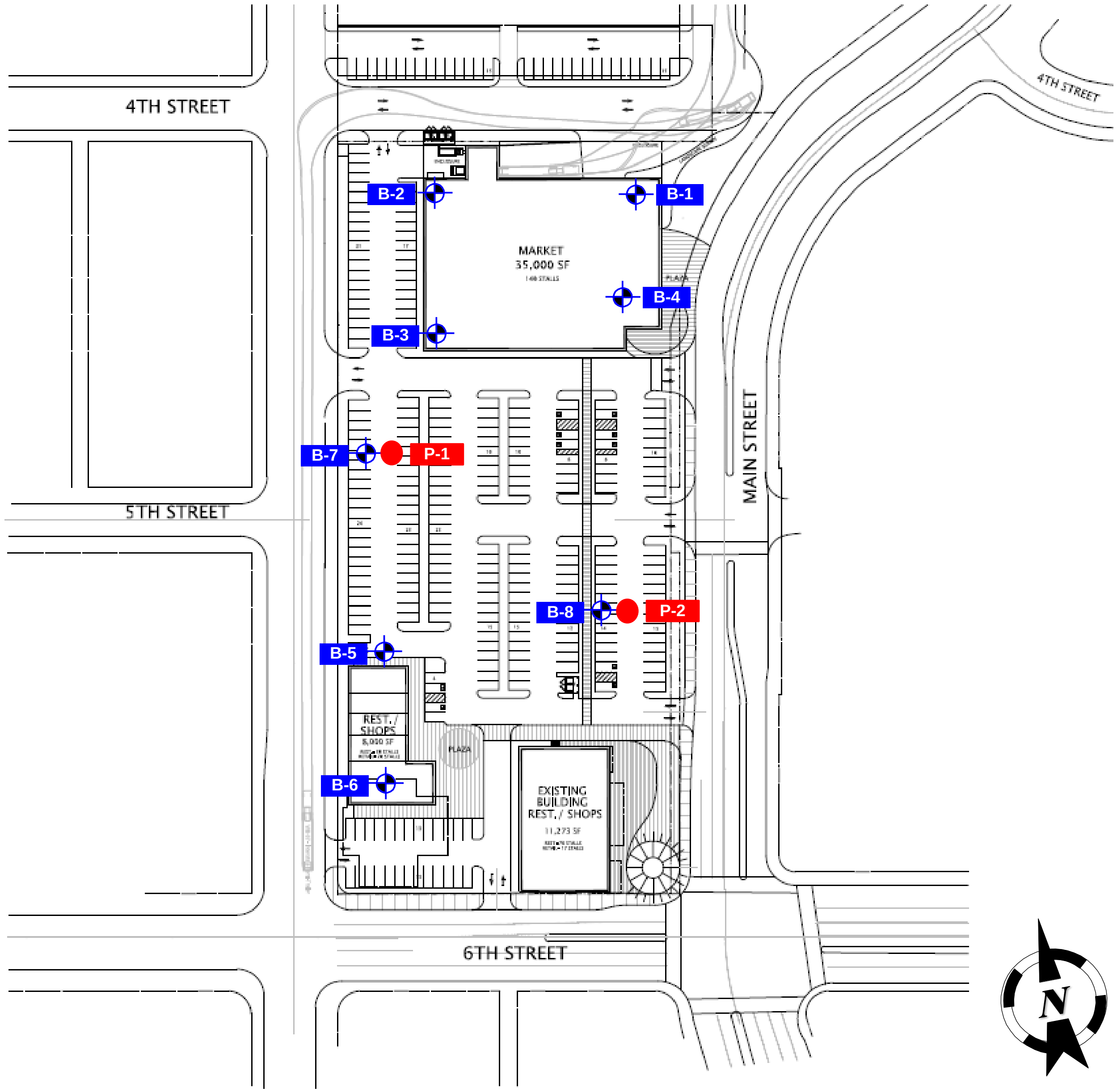
Ibrahim Foud Ibrahim, PE
Senior Managing Engineer
RCE 86724

Clarence Jiang, GE
Senior Geotechnical Engineer
RGE 2477



Source Image: U.S. Geological Survey, Corona North, California, https://ngmdb.usgs.gov/topoview (2022)

VICINITY MAP	SCALE:	DATE:	
GEOTECHNICAL ENGINEERING INVESTIGATION	NOT TO SCALE	12/2022	SALEM engineering group, inc.
Proposed Retail Development	DRAWN BY: JC	APPROVED BY: CJ	
NWC of 6th Street and Main Street	PROJECT NO. 3-222-1216	FIGURE NO. 1	
Corona, California			



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Retail Development
NWC of 6th Street and Main Street
Corona, California

SCALE:	DATE:
NOT TO SCALE	12/2022
DRAWN BY:	APPROVED BY:
JC	CJ
PROJECT NO.	FIGURE NO.
3-222-1216	2

LEGEND:

B-1 Soil Boring Locations
P-1 Percolation Locations

All Locations Approximate

SALEM engineering group, inc.

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation (drilling) was conducted on December 1, 2022, and included a site visit, subsurface exploration, and soil sampling. The locations of the exploratory borings and are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

In general, our borings were performed using a truck-mounted CME 75 drill rig equipped with 6⅝-inch diameter hollow-stem augers. Sampling in the borings was accomplished using a hydraulic 140-pound hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT "N" values; corrections have not been applied. Upon completion, the borings were backfilled with soil cuttings, and patched with cold asphalt (within paved areas).

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

SALEM engineering group, inc.

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

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 649'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 4.25 in.				
		SM	*No Aggregate Base				
	3/6 5/6 8/6		Gravelly, Silty SAND Loose; moist; brown; fine to coarse grain sand; fine to coarse gravel.	13	4.0	102.2	
645 / 5	50/1 - -		Grades as above; less gravel.	50/1"	6.0	-	Disturbed sample.
			Refusal at 5.5 feet BSG due to gravel/cobbles.				
640 / 10							
635 / 15							
630 / 20							
625 / 25							

Notes:

Figure Number A-1

SALEM engineering group, inc.

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

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 647'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0 (647)		SM	Silty SAND Loose; slightly moist; brown; fine to coarse grain sand; with gravel.				
645	4/6 6/6 7/6			13	10.2	105.7	
5	4/6 5/6 5/6	ML	Sandy SILT Firm; moist; brown; fine to medium grain sand; trace gravel.	10	10.7	113.1	
640							
10	50/4 - -		Grades as above; hard.	50/4"	12.6	-	
			Refusal at 10.5 feet BSG due to gravel/cobbles.				
635							
15							
630							
20							
625							
25							
620							

Notes:

Figure Number A-2

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

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

SALEM engineering group, inc.

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

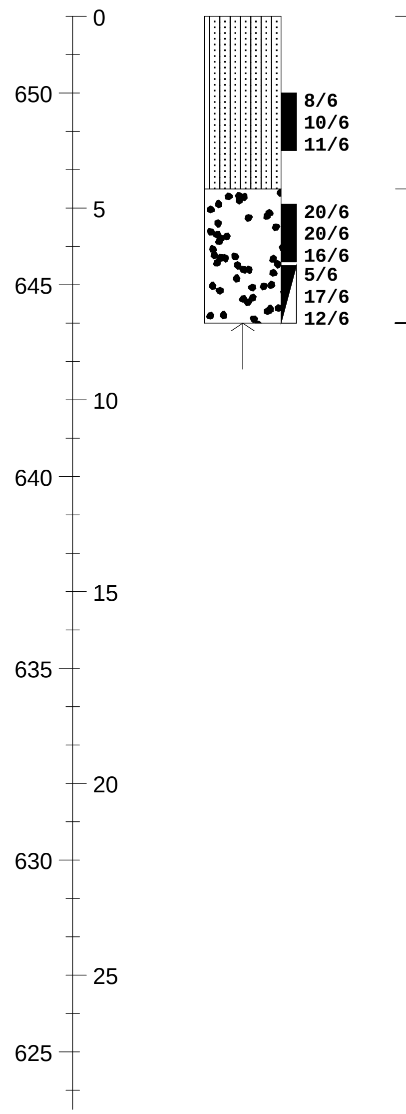
Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 652'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		SM	Silty SAND Medium dense; slightly moist; brown; fine to coarse grain sand.				
650	8/6 10/6 11/6			21	4.9	85.8	
5	20/6 20/6 16/6	GP	Sandy GRAVEL Medium dense; slightly moist; grayish brown; fine to coarse gravel; fine to coarse grain sand.	36	2.5	-	Disturbed sample.
645	5/6 17/6 12/6			20	1.9	-	
			Refusal at 8 feet BSG due to gravel/cobbles.				



Notes:

Figure Number A-3

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

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

SALEM engineering group, inc.

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

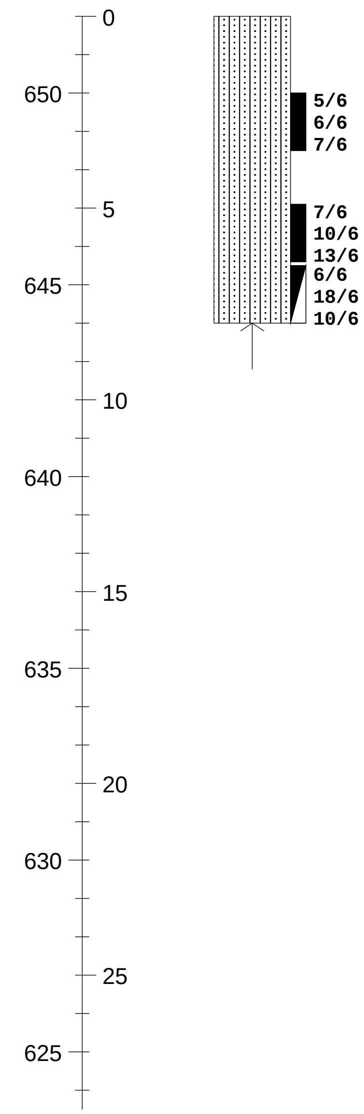
Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 652'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
		SM	Silty SAND Loose; slightly moist; brown; fine to coarse grain sand; with gravel.	13	3.2	110.5	
			Grades as above; medium dense.	23	2.9	115.8	
			Grades as above; moist.	28	10.7	-	
			Refusal at 8 feet BSG due to gravel/cobbles.				



Notes:

Figure Number A-4

SALEM engineering group, inc.

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

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 658'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 3.75 in.				
		AB	Aggregate Base = 2.5 in.				
655	7/6 10/6 9/6	SM	Silty SAND Medium dense; slightly moist; mottled brown/gray; fine to medium grain sand; with gravel.	19	4.8	116.7	
5	7/6 9/6 11/6		Grades as above; moist; brown; trace gravel.	20	7.9	107.4	
650							
10	7/6 9/6 11/6		Grades as above; slightly moist; fine to coarse grain sand; with gravel.	20	4.2	-	
645							
15	5/6 6/6 10/6		Grades as above; mottled brown/ reddish brown/gray.	16	4.6	-	
			End of boring at 16.5 feet BSG.				
640							
20							
635							
25							
630							

Notes:

Figure Number A-5

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

SALEM engineering group, inc.

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

Project: Proposed Retail Development

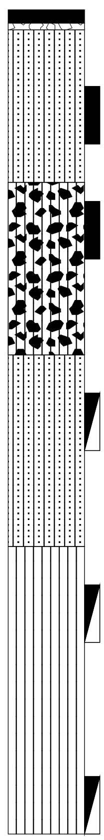
Location: NWC 6th Street & Main Street, Corona, California

Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 663'

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

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



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 3.25 in.				
		AB	Aggregate Base = 3 in.				
660	15/6 10/6 11/6	SM	Silty SAND Medium dense; slightly moist; brown; fine to coarse grain sand; with fine gravel.	21	5.4	106.0	
5	6/6 7/6 9/6	GM	Sandy, Silty GRAVEL Medium dense; moist; brown; fine to coarse gravel; fine to coarse grain sand. Grades as above; trace gravel.	16	4.6	116.1	
655 / 10	11/6 10/6 10/6	SM	Silty SAND Medium dense; moist; mottled brown/gray; fine to coarse grain sand; with fine gravel.	20	6.1	-	
650 / 15	8/6 10/6 15/6	ML	Sandy SILT Very stiff; moist; reddish brown; fine grain sand.	25	12.6	-	
645 / 20	10/6 13/6 12/6		Grades as above.	25	11.8	-	
			End of boring at 21.5 feet BSG.				
640 / 25 / 635							

Notes:

Figure Number A-6

SALEM engineering group, inc.

Test Boring: B-7 **Page** 1 **Of:** 1

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 654'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	7/6 6/6 9/6	SM	Silty SAND Medium dense; slightly moist; grayish brown; fine to coarse grain sand; with gravel.	15	4.4	-	
			End of boring at 3 feet BSG.				

Depth scale: 0, 5, 10, 15, 20, 25; Elevation scale: 650, 645, 640, 635, 630

Notes:

Figure Number A-7

Test Boring: B-8 **Page** 1 **Of:** 1

SALEM engineering group, inc.

Project Number: 3-222-1216

Date: 12/01/2022

Client: Northgate Gonzales Real Estate, LLC

Project: Proposed Retail Development

Location: NWC 6th Street & Main Street, Corona, California

Drilled By: SALEM **Logged By:** CC

Drill Type: CME 75 **Elevation:** 660'

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
660 — 0		AC	Asphalt Concrete = 5.25 in.				
	3/6 6/6 8/6	SM	*No Aggregate Base Silty SAND	14	7.3	117.3	
	3/6 4/6 7/6		Loose; moist; dark brown; fine to coarse grain sand; with gravel.	11	4.4	-	
			End of boring at 4 feet BSG.				
655 — 5							
650 — 10							
645 — 15							
640 — 20							
635 — 25							

Notes:

Figure Number A-8

KEY TO SYMBOLS

Symbol Description

Strata symbols

Asphaltic Concrete

Silty sand

Silt

Poorly graded gravel

Aggregate Base

Silty gravel

Misc. Symbols

Drill rejection

Soil Samplers

California sampler

Standard penetration test

Notes:

Granular Soils
Blows Per Foot (Uncorrected)

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

Cohesive Soils
Blows Per Foot (Uncorrected)

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

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

Percolation Test Worksheet

Project: **Proposed Retail Development**
NWC of 6th Street & Main Street
Corona, California

Job No.: **3-222-1216**
Date Drilled: **12/1/2022**
Soil Classification: **Silty SAND (SM)**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Total Depth of Hole: 36 in.

Test Hole No.: **P-1**
Tested by: **CC**
Drilled Hole Depth: **3** ft.

Presoaking Date: **12/1/2022**
Test Date: **12/2/2022**

Pipe Stick up: 2 ft.

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
10:55	11:25	5.0	Y	0:30	3.02	3.43	4.92	30	6.1	23.8	18.8	21.3	0.84
11:25	11:55	5.0	N	0:30	3.43	3.72	3.48	30	8.6	18.8	15.4	17.1	0.73
11:55	12:25	5.0	N	0:30	3.72	3.95	2.76	30	10.9	15.4	12.6	14.0	0.69
12:25	12:55	5.0	N	0:30	3.95	4.15	2.40	30	12.5	12.6	10.2	11.4	0.72
12:55	13:25	5.0	N	0:30	4.15	4.31	1.92	30	15.6	10.2	8.3	9.2	0.68
13:25	13:55	5.0	N	0:30	4.31	4.44	1.56	30	19.2	8.3	6.7	7.5	0.66
13:57	14:27	5.0	Y	0:30	2.88	3.24	4.32	30	6.9	25.4	21.1	23.3	0.68
14:27	14:57	5.0	N	0:30	3.24	3.54	3.60	30	8.3	21.1	17.5	19.3	0.68
14:57	15:27	5.0	N	0:30	3.54	3.79	3.00	30	10.0	17.5	14.5	16.0	0.67
15:27	15:57	5.0	N	0:30	3.79	4.00	2.52	30	11.9	14.5	12.0	13.3	0.66
15:57	16:27	5.0	N	0:30	4.00	4.18	2.16	30	13.9	12.0	9.8	10.9	0.67
16:27	16:57	5.0	N	0:30	4.18	4.33	1.80	30	16.7	9.8	8.0	8.9	0.66
											Infiltration Rate		**0.66**

SALEM
engineering group, inc.

Percolation Test Worksheet

Project: **Proposed Retail Development**
NWC of 6th Street & Main Street
Corona, California

Job No.: **3-222-1216**
Date Drilled: **12/1/2022**
Soil Classification: **Silty SAND (SM)**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Total Depth of Hole: 48 in.

Test Hole No.: **P-2**
Tested by: **CC**
Drilled Hole Depth: **4.0** ft.

Presoaking Date: **12/1/2022**
Test Date: **12/2/2022**

Pipe Stick up: 0.5 ft.

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
9:50	10:20	4.5	Y	0:30	2.75	2.98	2.76	30	10.9	21.0	18.2	19.6	0.51
10:20	10:50	4.5	N	0:30	2.98	3.17	2.28	30	13.2	18.2	16.0	17.1	0.48
10:50	11:20	4.5	N	0:30	3.17	3.33	1.92	30	15.6	16.0	14.0	15.0	0.45
11:20	11:50	4.5	N	0:30	3.33	3.47	1.68	30	17.9	14.0	12.4	13.2	0.44
11:50	12:20	4.5	N	0:30	3.47	3.60	1.56	30	19.2	12.4	10.8	11.6	0.46
12:20	12:50	4.5	N	0:30	3.60	3.71	1.32	30	22.7	10.8	9.5	10.1	0.43
12:53	13:23	4.5	Y	0:30	3.03	3.20	2.04	30	14.7	17.6	15.6	16.6	0.44
13:23	13:53	4.5	N	0:30	3.20	3.35	1.80	30	16.7	15.6	13.8	14.7	0.43
13:53	14:23	4.5	N	0:30	3.35	3.48	1.56	30	19.2	13.8	12.2	13.0	0.42
14:23	14:53	4.5	N	0:30	3.48	3.60	1.44	30	20.8	12.2	10.8	11.5	0.43
14:53	15:23	4.5	N	0:30	3.60	3.71	1.32	30	22.7	10.8	9.5	10.1	0.43
15:23	15:53	4.5	N	0:30	3.71	3.81	1.20	30	25.0	9.5	8.3	8.9	0.44
											Infiltration Rate		**0.42**

SALEM engineering group, inc.

APPENDIX

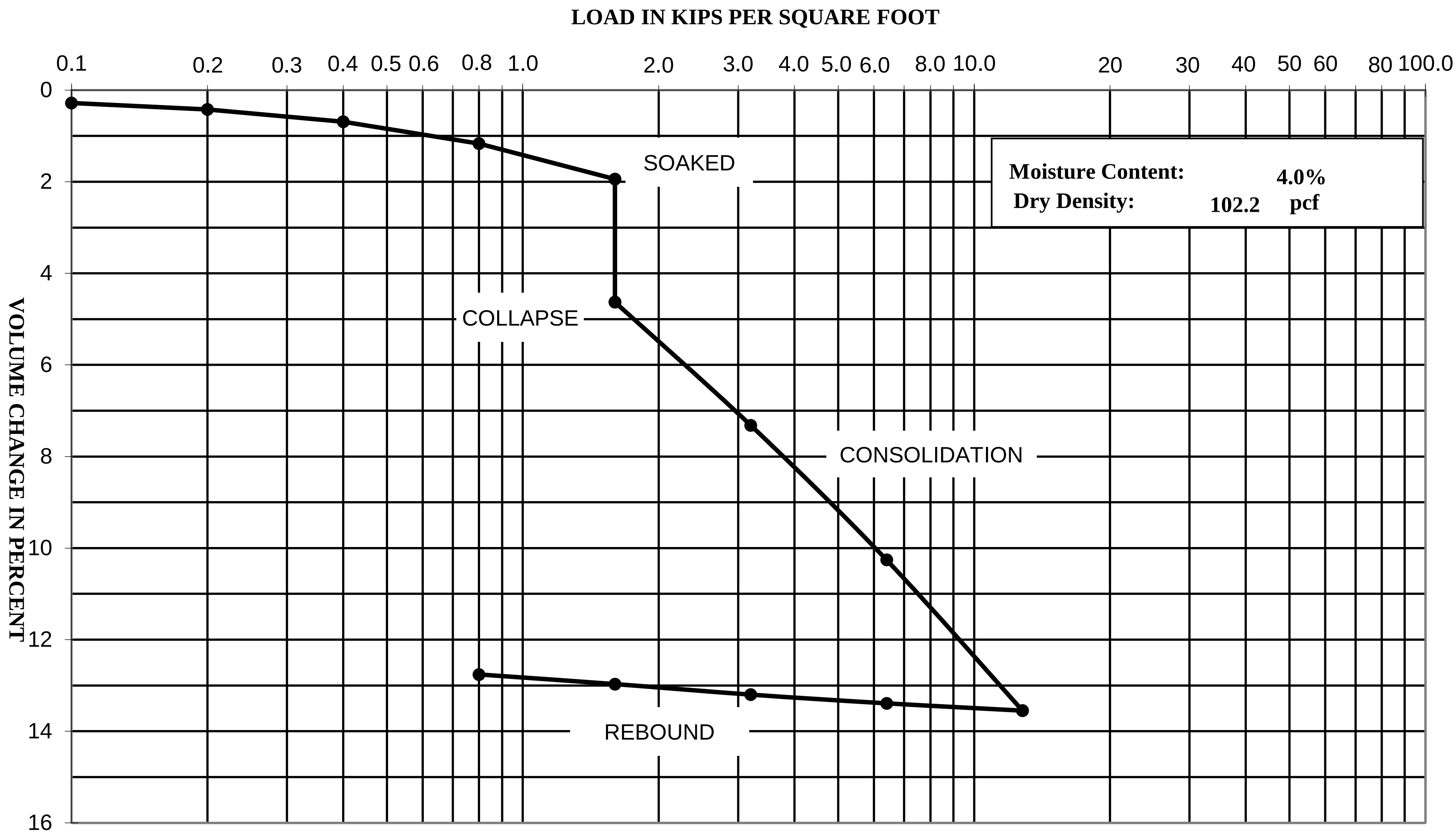
B

APPENDIX B
LABORATORY TESTING

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

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435



Project Name: Proposed Retail Development - Corona, CA

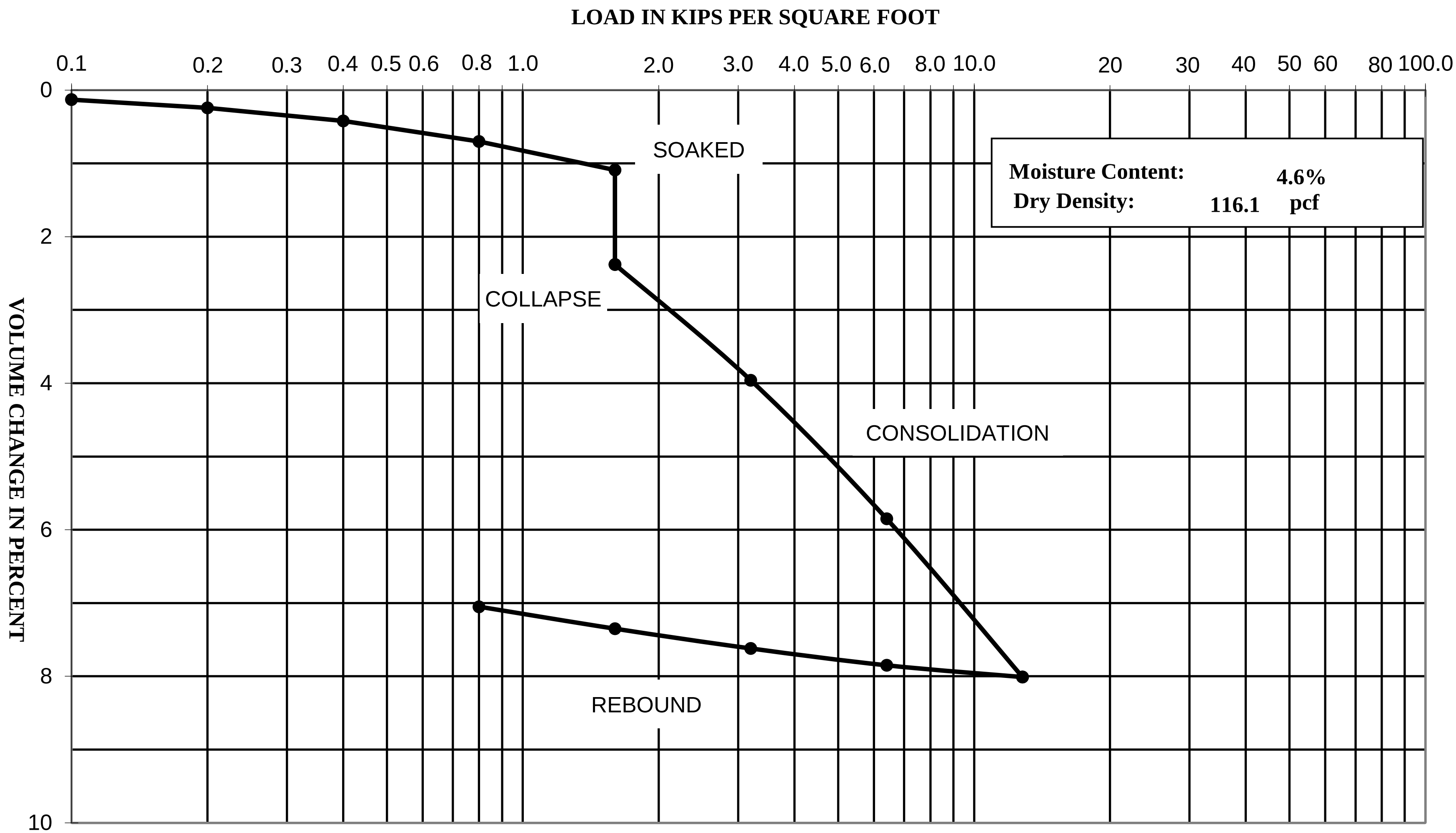
Project Number: 3-222-1216

Boring: B-1 @ 2'

SALEM engineering group, inc.

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435



Project Name: Proposed Retail Development - Corona, CA

Project Number: 3-222-1216

Boring: B-6 @ 5'

SALEM engineering group, inc.

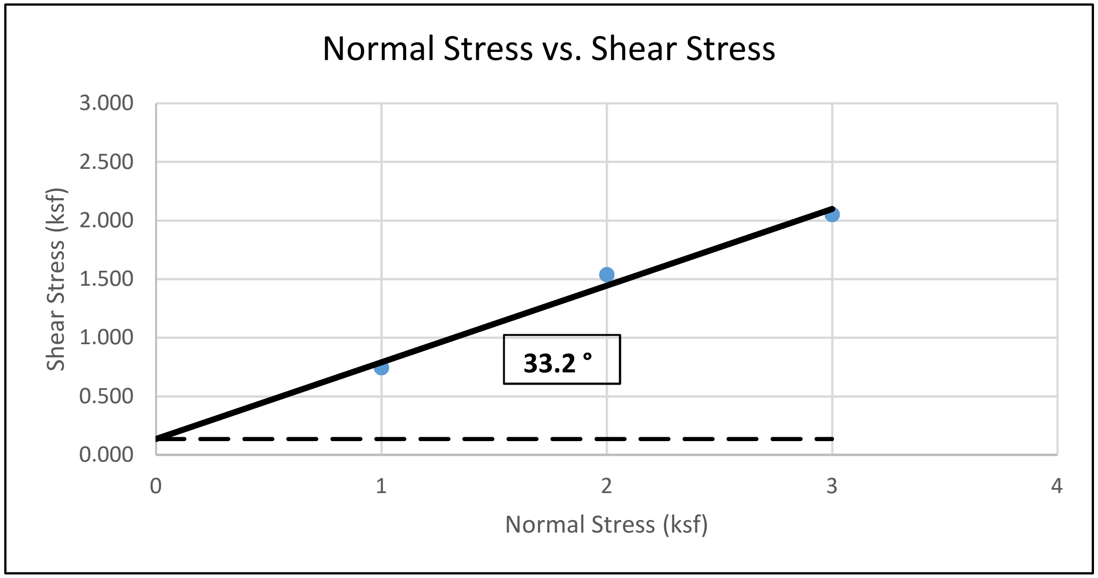
Direct Shear Test (ASTM D3080)

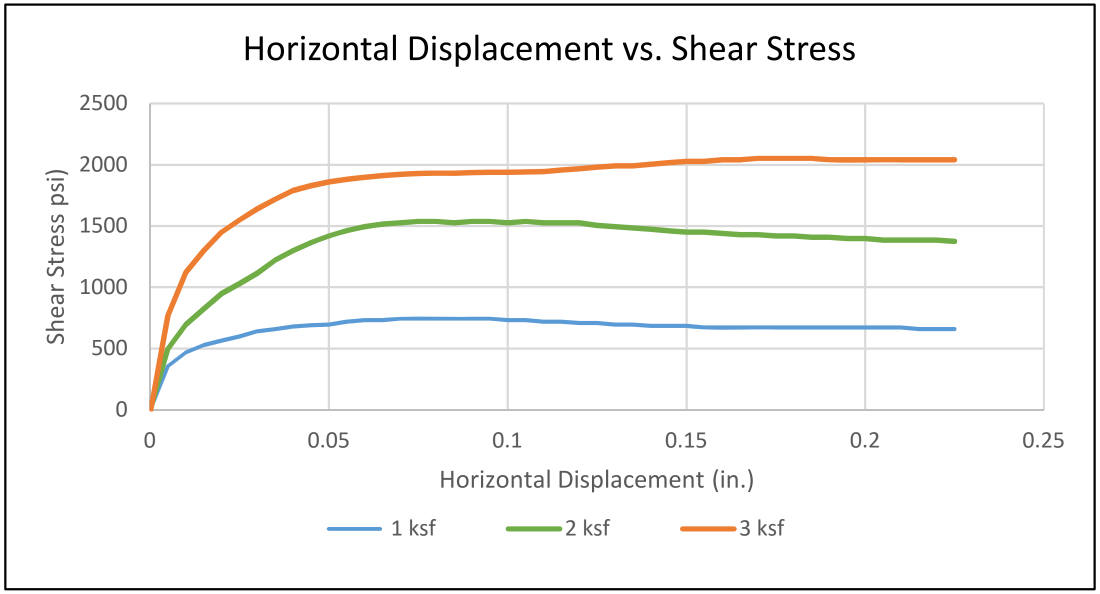
Project Name:	Proposed Retail Development - Corona, CA
Project Number:	3-222-1216
Client:	Northgate Gonzales Real Estate, LLC
Sample Location:	B-2 @ 5'
Sample Type:	Undisturbed Ring
Soil Classification:	Sandy SILT (ML) w/trace Gravel
Tested By:	M. Noorzay
Reviewed By:	CJ
Date:	12/13/2022
Equipment Used:	Geomatic Direct Shear Machine

	Sample 1	Sample 2	Sample 3
Normal Stress (ksf)	1.000	2.000	3.000
Shear Rate (in/min)	0.004		
Peak Shear Stress (ksf)	0.744	1.538	2.052
Residual Shear Stress (ksf)	0.000	0.000	0.000

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in.)	1	1	1
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	10.3		
Final Moisture Content (%)	20.1	16.8	18.5
Dry Density (pcf)	110.2	114.6	112.8

Peak Shear Strength Values	
Slope	0.65
Friction Angle	33.2
Cohesion (psf)	137





SALEM engineering group, inc.

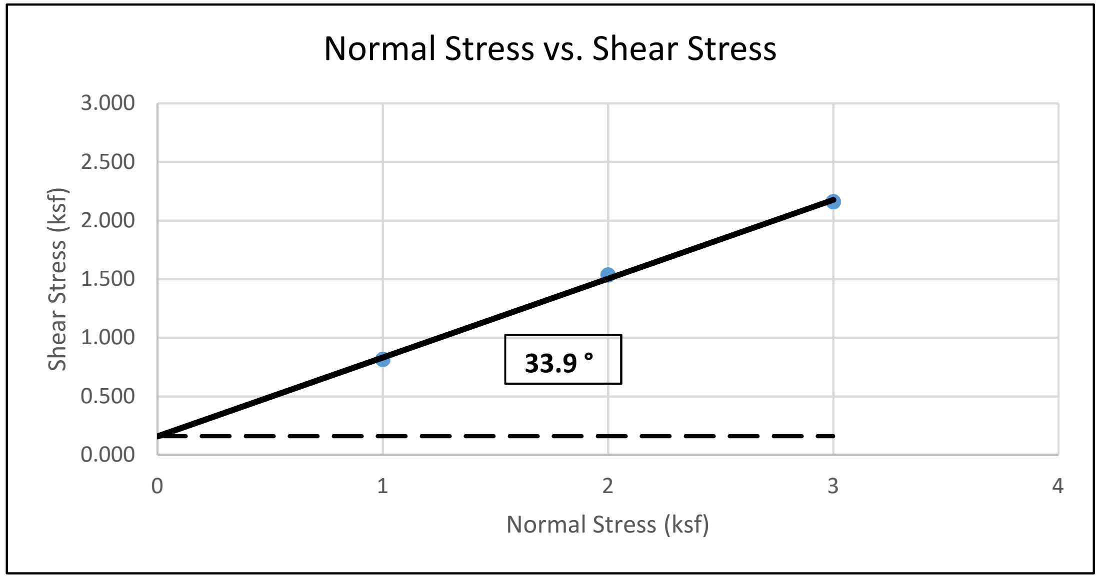
Direct Shear Test (ASTM D3080)

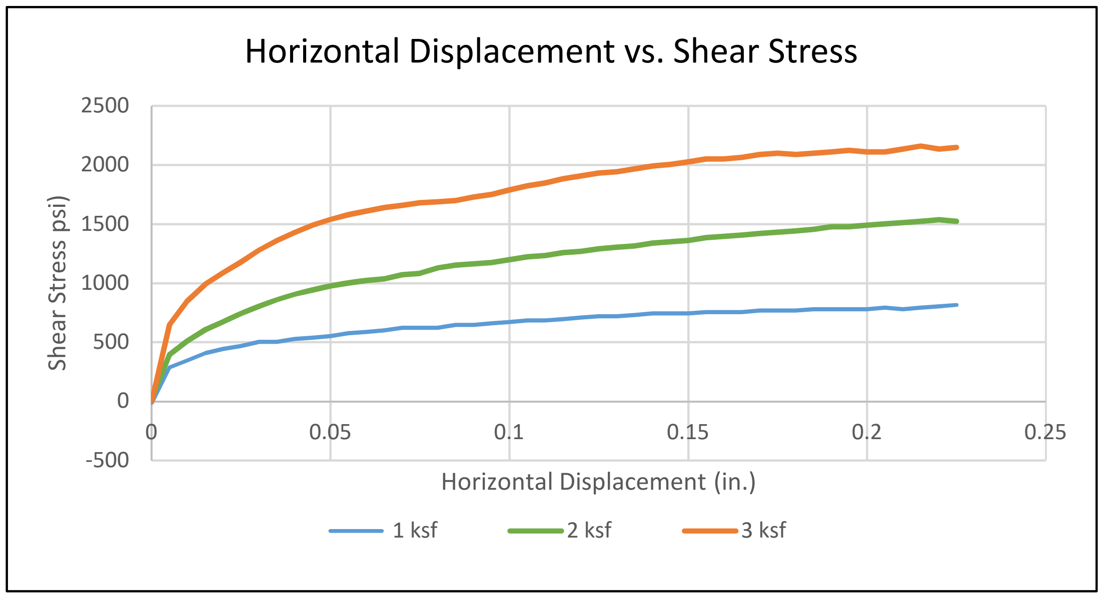
Project Name:	Proposed Retail Development - Corona, CA
Project Number:	3-222-1216
Client:	Northgate Gonzales Real Estate, LLC
Sample Location:	B-6 @ 2'
Sample Type:	Undisturbed Ring
Soil Classification:	Silty SAND (SM) w/Gravel
Tested By:	M. Noorzay
Reviewed By:	CJ
Date:	12/14/2022
Equipment Used:	Geomatic Direct Shear Machine

	Sample 1	Sample 2	Sample 3
Normal Stress (ksf)	1.000	2.000	3.000
Shear Rate (in/min)	0.004		
Peak Shear Stress (ksf)	0.816	1.536	2.160
Residual Shear Stress (ksf)	0.000	0.000	0.000

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in.)	1	1	1
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	5.2		
Final Moisture Content (%)	17.0	16.9	14.9
Dry Density (pcf)	105.6	106.4	107.7

Peak Shear Strength Values	
Slope	0.67
Friction Angle	33.9
Cohesion (psf)	160

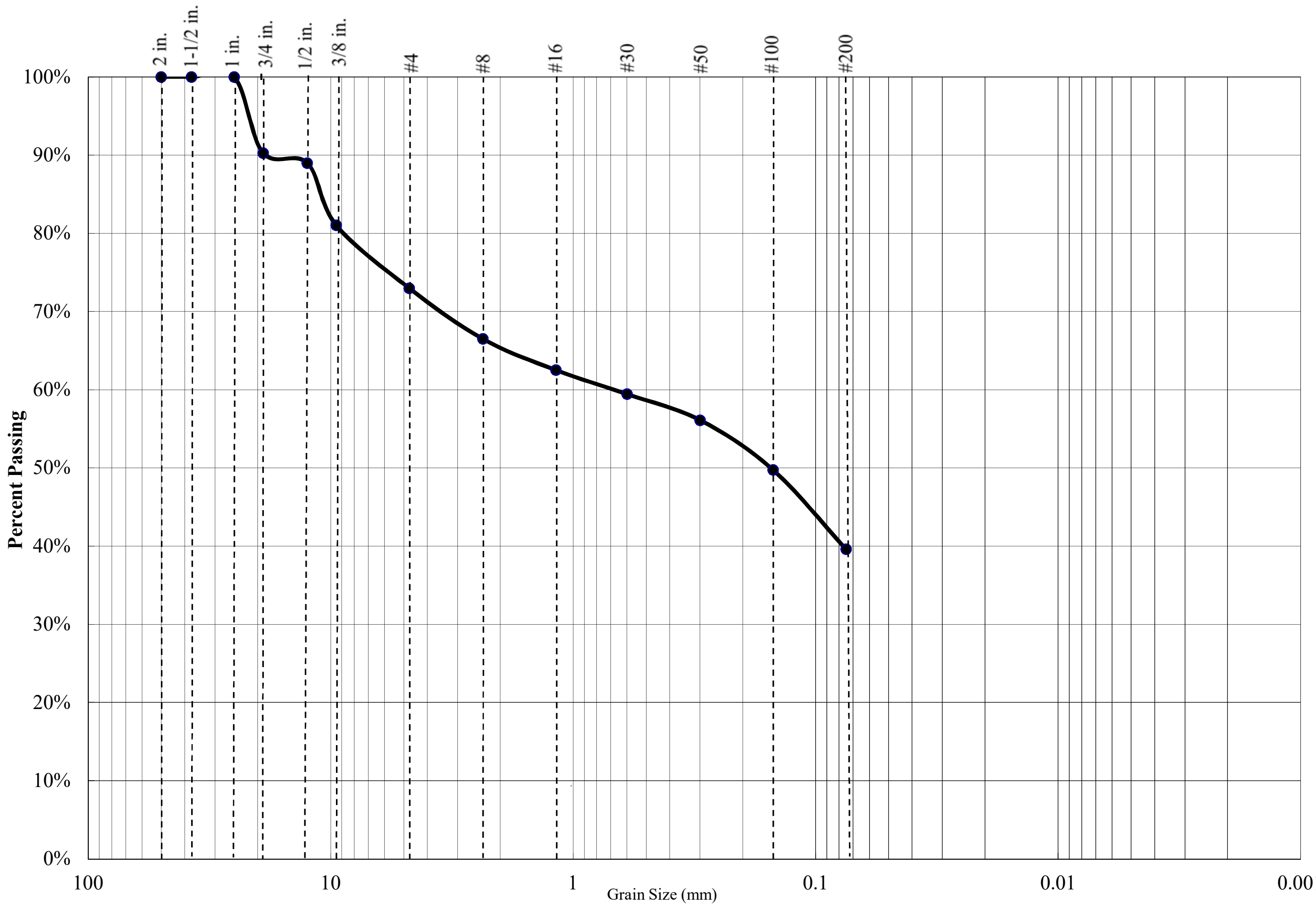




SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
27%	33%	40%

Sieve Size	Percent Passing
3/4 inch	90.3%
1/2 inch	89.0%
3/8 inch	81.0%
#4	73.0%
#8	66.5%
#16	62.5%
#30	59.4%
#50	56.1%
#100	49.8%
#200	39.6%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Gravelly, Silty SAND (SM)

Project Name: Proposed Retail Development - Corona, CA

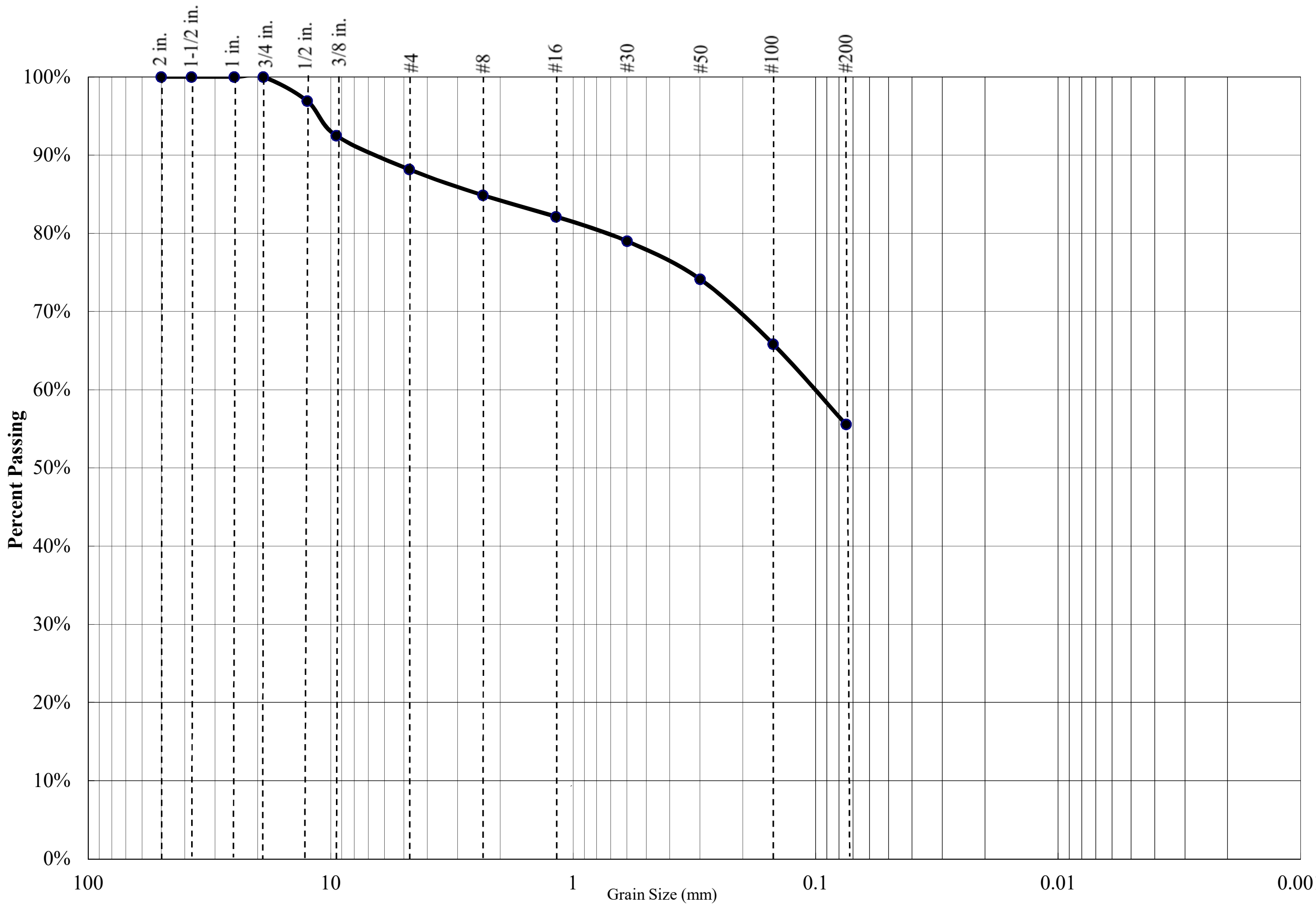
Project Number: 3-222-1216

Boring: B-1 @ 2'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
12%	33%	56%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	96.9%
3/8 inch	92.5%
#4	88.2%
#8	84.9%
#16	82.1%
#30	79.0%
#50	74.1%
#100	65.8%
#200	55.6%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy SILT (ML)

Project Name: Proposed Retail Development - Corona, CA

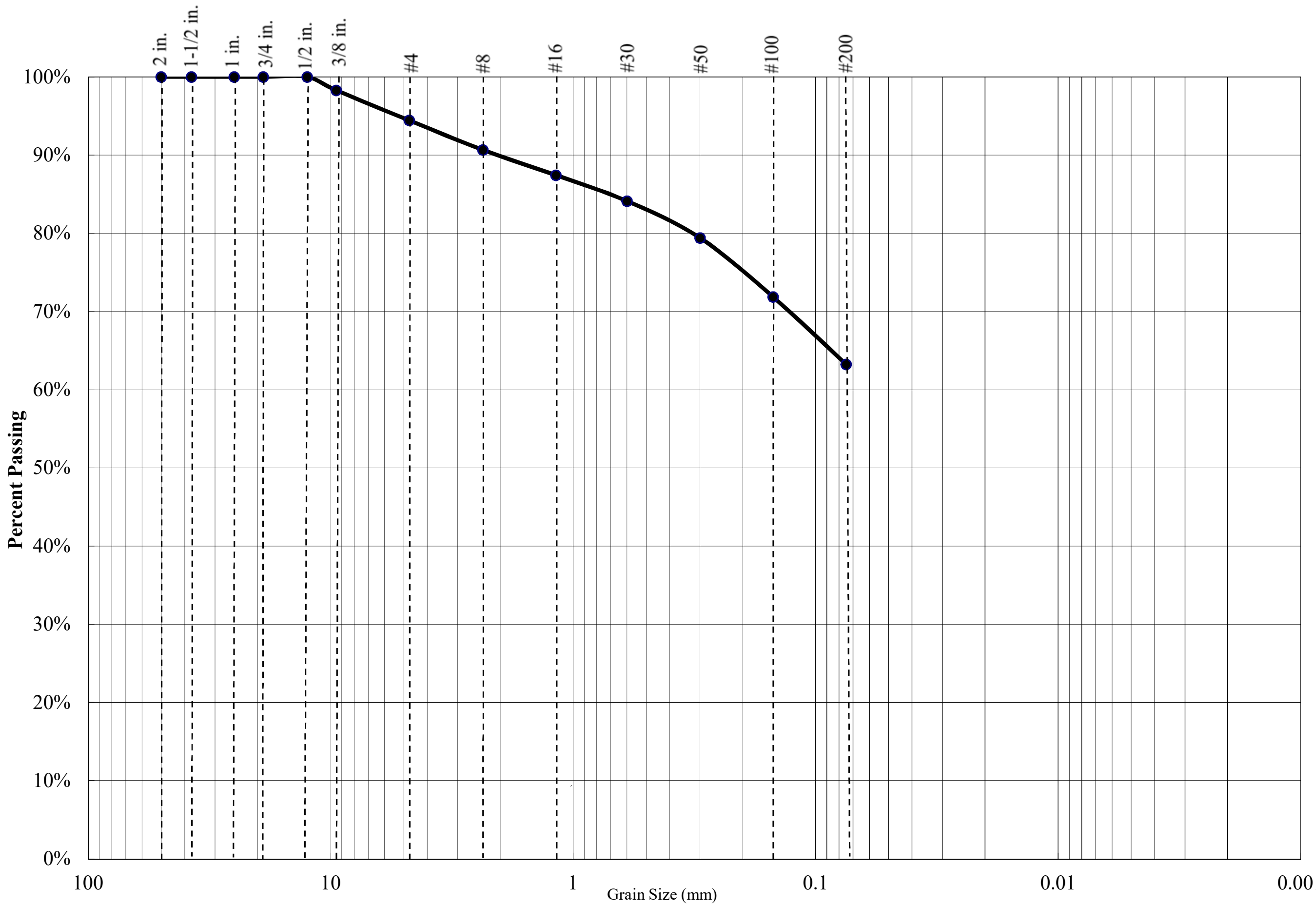
Project Number: 3-222-1216

Boring: B-2 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
6%	31%	63%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	98.3%
#4	94.4%
#8	90.7%
#16	87.4%
#30	84.1%
#50	79.4%
#100	71.9%
#200	63.3%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy SILT (ML)

Project Name: Proposed Retail Development - Corona, CA

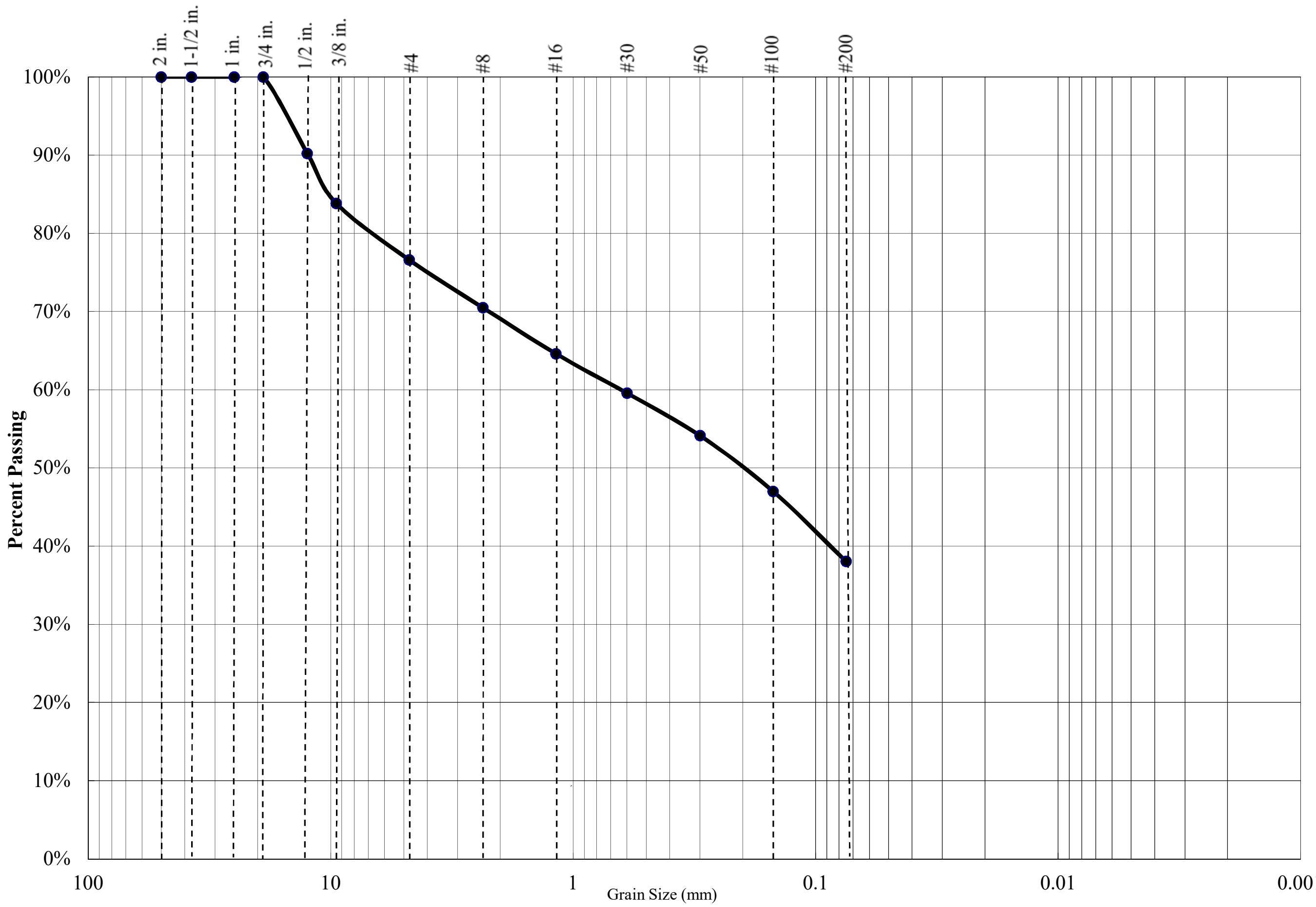
Project Number: 3-222-1216

Boring: B-2 @ 10'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
23%	39%	38%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	90.2%
3/8 inch	83.8%
#4	76.6%
#8	70.5%
#16	64.6%
#30	59.6%
#50	54.1%
#100	47.0%
#200	38.0%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM) w/Gravel

Project Name: Proposed Retail Development - Corona, CA

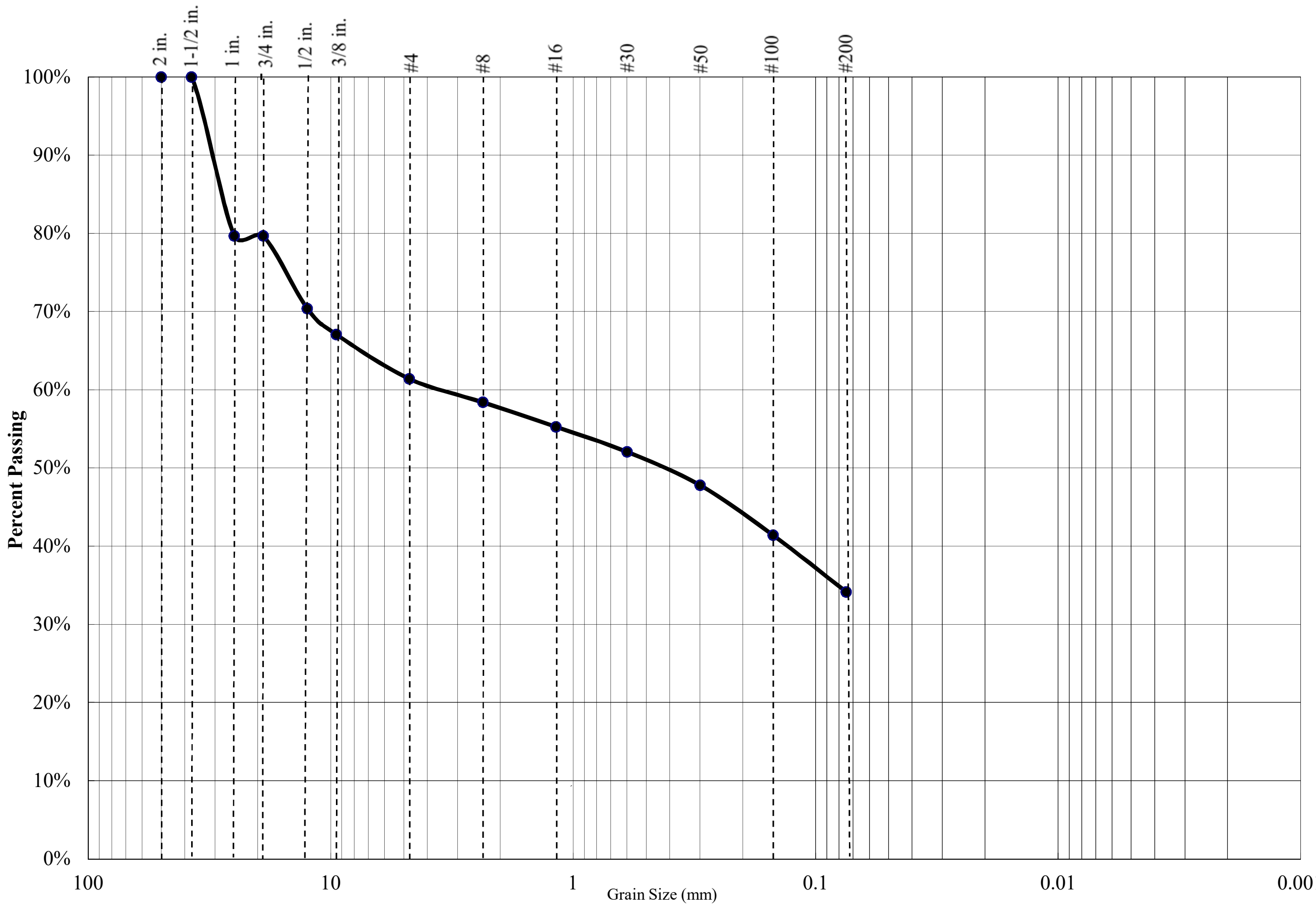
Project Number: 3-222-1216

Boring: B-6 @ 2'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
39%	27%	34%

Sieve Size	Percent Passing
3/4 inch	79.7%
1/2 inch	70.4%
3/8 inch	67.1%
#4	61.4%
#8	58.4%
#16	55.3%
#30	52.0%
#50	47.8%
#100	41.4%
#200	34.2%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy, Silty GRAVEL (GM)

Project Name: Proposed Retail Development - Corona, CA

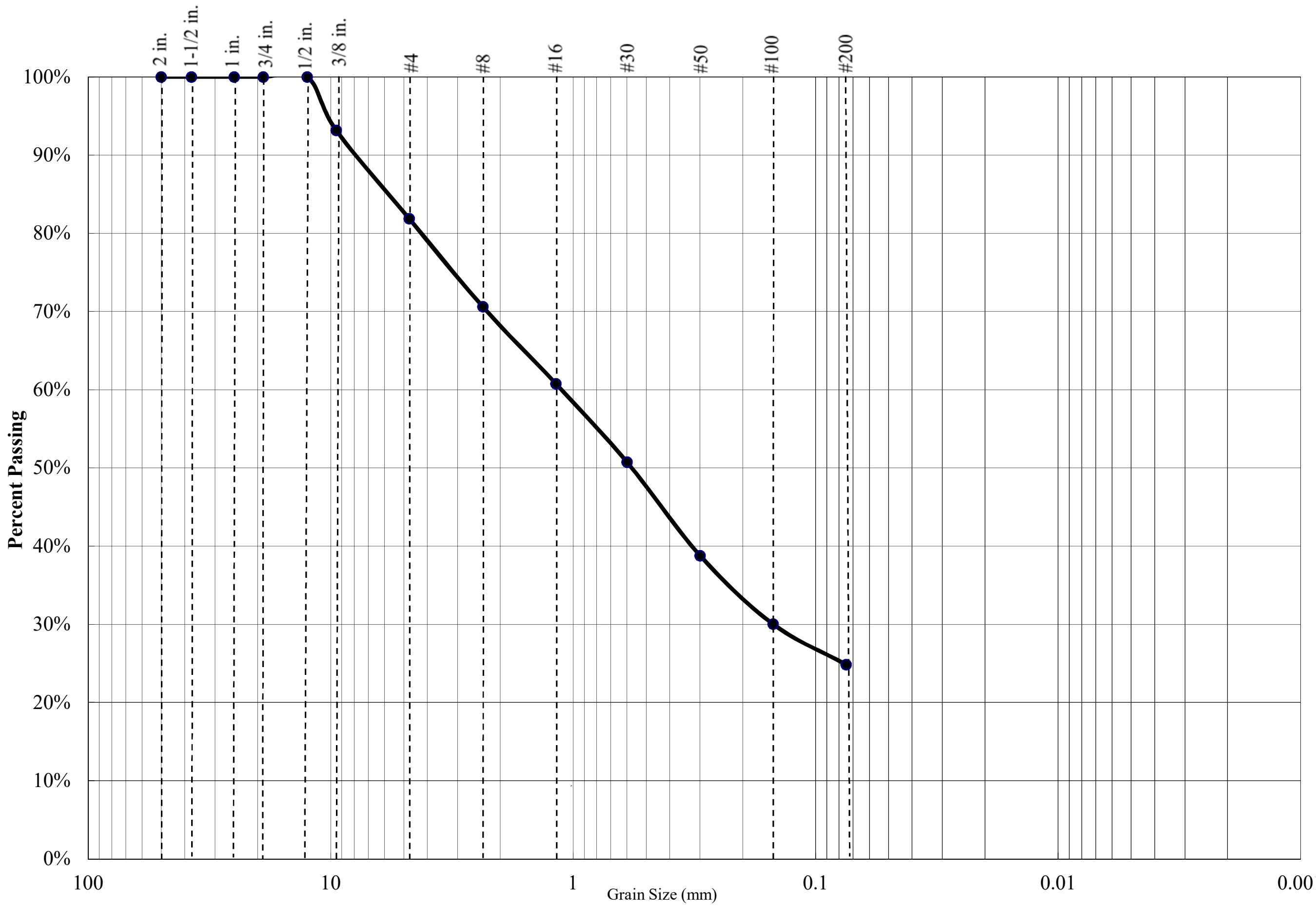
Project Number: 3-222-1216

Boring: B-6 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
18%	57%	25%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	93.2%
#4	81.9%
#8	70.6%
#16	60.8%
#30	50.7%
#50	38.8%
#100	30.0%
#200	24.9%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Retail Development - Corona, CA

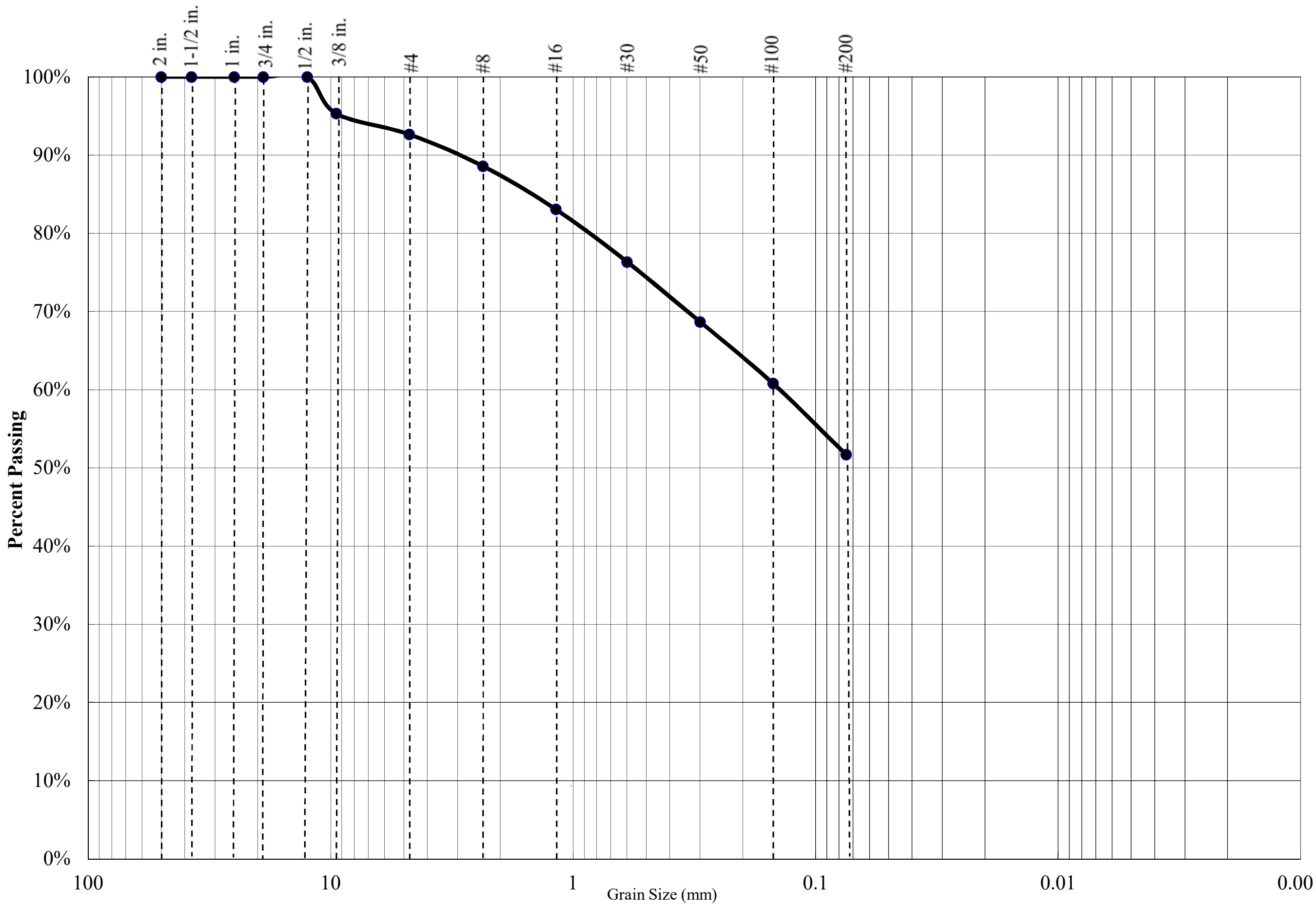
Project Number: 3-222-1216

Boring: B-6 @ 10'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
7%	41%	52%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	95.3%
#4	92.7%
#8	88.6%
#16	83.1%
#30	76.3%
#50	68.7%
#100	60.8%
#200	51.7%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy SILT (ML)

Project Name: Proposed Retail Development - Corona, CA

Project Number: 3-222-1216

Boring: B-6 @ 20'

SALEM engineering group, inc.

CHEMICAL ANALYSIS

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

Project Name: Proposed Retail Development - Corona, CA
Project Number: 3-222-1216
Date Sampled: 12/1/2022 Date Tested: 12/14/2022
Sampled By: CC Tested By: M. Noorzay
Soil Description: Brown Gravelly, Silty SAND (SM)

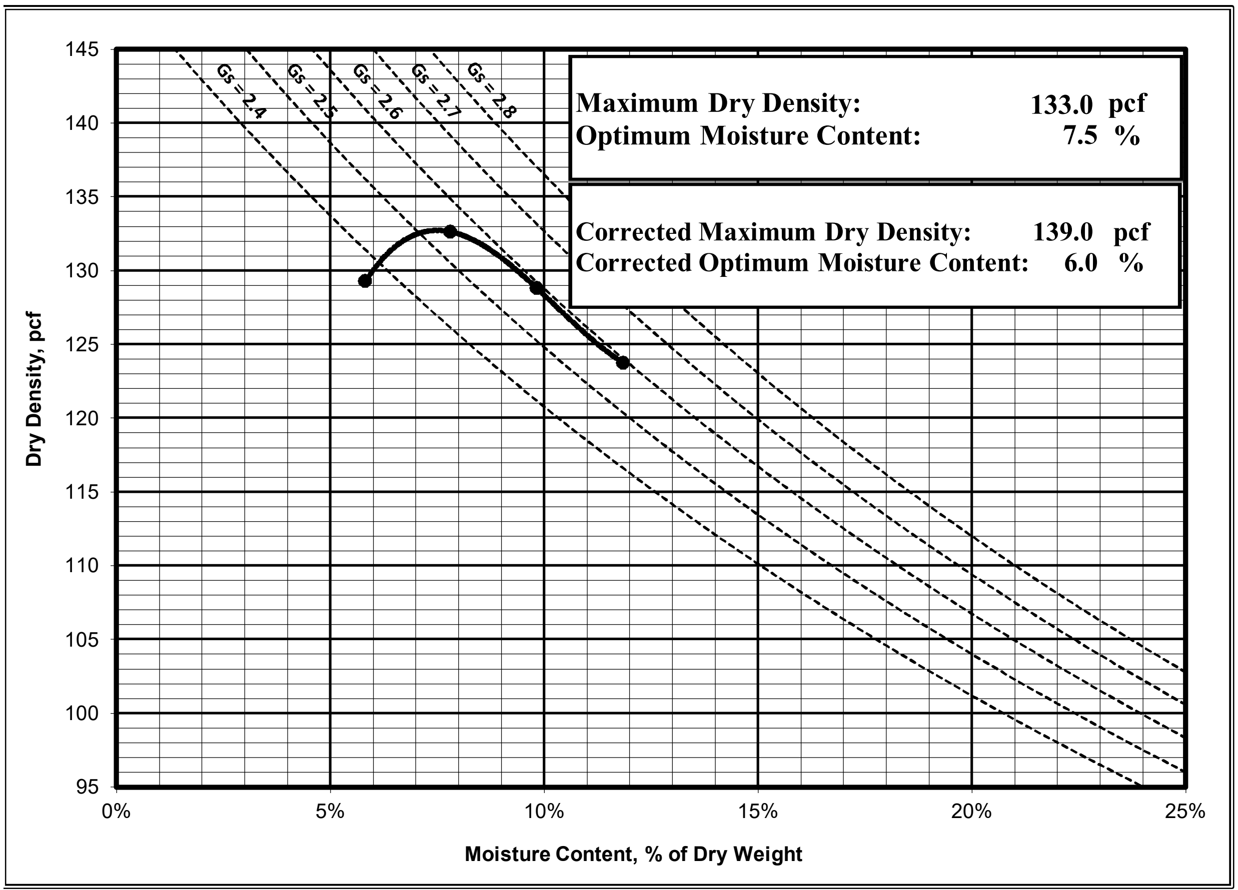
Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 1'-4'	< 50 mg/kg	52 mg/kg	7.9
1b.	B-1 @ 1'-4'	< 50 mg/kg	50 mg/kg	7.9
1c.	B-1 @ 1'-4'	< 50 mg/kg	49 mg/kg	7.9
Average:		**< 50 mg/kg**	**50 mg/kg**	**7.9**

SALEM engineering group, inc.

Laboratory Compaction Curve
ASTM D1557

Project Name: Proposed Retail Development - Corona, CA
Project Number: 3-222-1216
Date Sampled: 12/1/2022 Date Tested: 12/14/2022
Sampled By: CC Tested By: M. Noorzay
Sample Location: B-1 @ 1'-4'
Soil Description: Brown Gravelly, Silty SAND (SM)
Test Method: Method B

	1	2	3	4
Weight of Moist Specimen & Mold, (g)	6348.5	6442.0	6419.0	6372.8
Weight of Compaction Mold, (g)	4280.2	4280.2	4280.2	4280.2
Weight of Moist Specimen, (g)	2068.3	2161.8	2138.8	2092.6
Volume of Mold, (ft^3)	0.0333	0.0333	0.0333	0.0333
Wet Density, (pcf)	136.8	143.0	141.5	138.4
Weight of Wet (Moisture) Sample, (g)	200.0	200.0	200.0	200.0
Weight of Dry (Moisture) Sample, (g)	189.0	185.5	182.1	178.8
Moisture Content, (%)	5.8%	7.8%	9.8%	11.9%
Dry Density, (pcf)	129.3	132.6	128.8	123.7



SALEM engineering group, inc.

APPENDIX

C

APPENDIX C
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

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

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

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

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

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

3.0 TECHNICAL REQUIREMENTS: All compacted materials shall be densified to no less that 95 percent of relative compaction (90 percent for clay soils) 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 recompacted to 95 percent relative compaction (90 percent for clay soils).

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

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

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

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

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

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

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

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 (90 percent for clay soils) based upon ASTM D1557. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

14.0 AGGREGATE BASE - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class II material, ¾-inch or 1½-inches maximum size. The aggregate base material shall be compacted to a minimum relative compaction of 95 percent based upon ASTM D1557 Test Method. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

15.0 AGGREGATE SUBBASE - The aggregate subbase shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate subbase material shall conform to the requirements of Section 25 of the Standard Specifications for Class II Subbase material. The aggregate subbase material shall be compacted to a minimum relative compaction of 95 percent based upon ASTM D1557 Test Method, 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.

8711 Monroe Court, Suite A
Rancho Cucamonga, CA 91730
(909) 980-6455 Office
(909) 980-6435 Fax

April 3, 2024

Job No. 3-222-1216

Ms. Elizabeth Resendiz
Northgate Gonzales Real Estate, LLC
1201 N. Magnolia Avenue
Anaheim, CA 92801

**SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION REPORT UPDATE
PROPOSED RETAIL DEVELOPMENT
NWC 6TH STREET & MAIN STREET
CORONA, CALIFORNIA**

Reference: Salem Engineering Group, Inc. (SALEM), Geotechnical Engineering Investigation Report, Proposed Retail Development, NWC 6th Street & Main Street, Corona, California, SALEM Project No. 3-222-1216, Dated December 20, 2022

Dear Ms. Resendiz:

In accordance with your request, we have provided this letter to update the above referenced Report for the proposed Retail Development to be located at the subject site in Corona, California.

The subject site is nearly rectangular in shape and is located at the northwest corner of the intersection of 6th Street and Main Street in the City of Corona, California. The site is gently sloping to the north with elevations ranging from 669 to 647 feet above mean sea level based on Google Earth imagery.

The proposed development of the site will include demolition of an existing commercial building and a bank kiosk, remodel of an existing bank building into two tenants, a 3,633 square-foot bank and a 3,297 square-foot restaurant, and construction of a 40,000 square-foot market building (see attached Site Plan, Figure 1).

Based on available historical imagery, the northern portion of the site was previously occupied by single-family residences and a commercial/industrial building. Those buildings were demolished from around 2005 through 2013.

At the time of our field investigation in December 2022, the site was predominately developed with 3 commercial buildings and a drive-thru kiosk with associated asphalt concrete pavement and landscaping. The site was revisited on April 2, 2024, and the site conditions were found to remain similar to December

2022, except that 5th Street (bisects the site into north and south portions) and 4th street (north end of the site) had been repaved, and the residential property at 323 S. Belle Avenue has been demolished. Photos of the site from April 2, 2024 are presented below:

Looking to the Southwest from 5th Street in middle of site

Looking to the East on 5th Street

Looking to the Northeast from west end of 5th Street

Based on our review of the referenced report and recent site visit, the above mentioned report is considered, from a geotechnical standpoint, to remain valid for the proposed development except the seismic design parameters needed to be updated and grading recommendations for the demolished residential property needed to be provided.

The 2022 report presented seismic design criteria based on the 2019 California Building Code (CBC). Seismic design criteria have now been incorporated into the 2022 California Building Code (CBC) and are applicable as of January 1, 2023.

Site Coefficient

For seismic design of the structures, and in accordance with the seismic provisions of the 2022 CBC, our recommended parameters are shown below. These parameters were determined using Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps by location website (https://seismicmaps.org/), in accordance with the 2022 CBC. The Site Class was determined based on the soils encountered during our field exploration.

2022 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	ASCE 7-16 or 2022 CBC Reference
Site Coordinates (Datum = NAD 83)		33.8774 Lat -117.5679 Lon	
Site Class	--	D	ASCE 7 Table 20.3-1
Soil Profile Name	--	Default	ASCE 7 Table 20.3-1
Risk Category	--	II	CBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.2	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	1.026g	ASCE 7 Equation 11.8-1
Seismic Design Category	SDC	**E**	CBC Table 1613.2.5
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	2.037 g	CBC Figure 1613.2.1(1-10)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.772 g	CBC Figure 1613.2.1(1-10)
Site Class Modified Site Coefficient	F_a	1.2	CBC Table 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	1.7*	CBC Table 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a\ S_S$	S_{MS}	2.444 g	CBC Equation 16-20
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v\ S_1$***1.5**	S_{M1}	1.969 g*	CBC Equation 16-21
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.629 g	CBC Equation 16-22
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	1.312 g*	CBC Equation 16-23
Short Period Transition Period(S_{D1}/S_{DS}), Seconds	T_S	0.806	ASCE 7-16, Section 11.4.6
Long Period Transition period(seconds)	T_L	8	ASCE 7-16, Figures 22-14

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

Site Specific Ground Motion Analysis was not included in the scope of this update. Per ASCE 11.4.8, Structures on Site Class D, with S_1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific ground motion analysis may not be required based on Exceptions listed in ASCE 11.4.8. The Structural Engineer should verify whether Exception No. 2 of ASCE 7-16, Section 11.4.8 is valid for the site. <u>The value reported for S_{M1} includes a 50% increase in accordance with exceptions listed in ASCE 7-16, Supplement 3.</u> In the event a site specific ground motion analysis is required, SALEM should be contacted for these services.

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.

Grading Recommendations for Demolished Residential Building

The residence at 323 S. Belle Street was recently demolished and the ground surface appeared to be approximately 2 to 4 feet lower than the surrounding grade. The subsurface soil condition should be determined by our representative during construction and additional recommendations will be provided accordingly.

Excavated soils generated from cut operations at the site are suitable for use as general Engineered Fill in structural areas provided they do not contain deleterious matter, debris, organic material, or rocks larger than 3 inches in maximum dimension.

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

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

Limitations

The recommendations and limitations provided in the Geotechnical Engineering Investigation Report apply to this letter. If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (909) 980-6455.

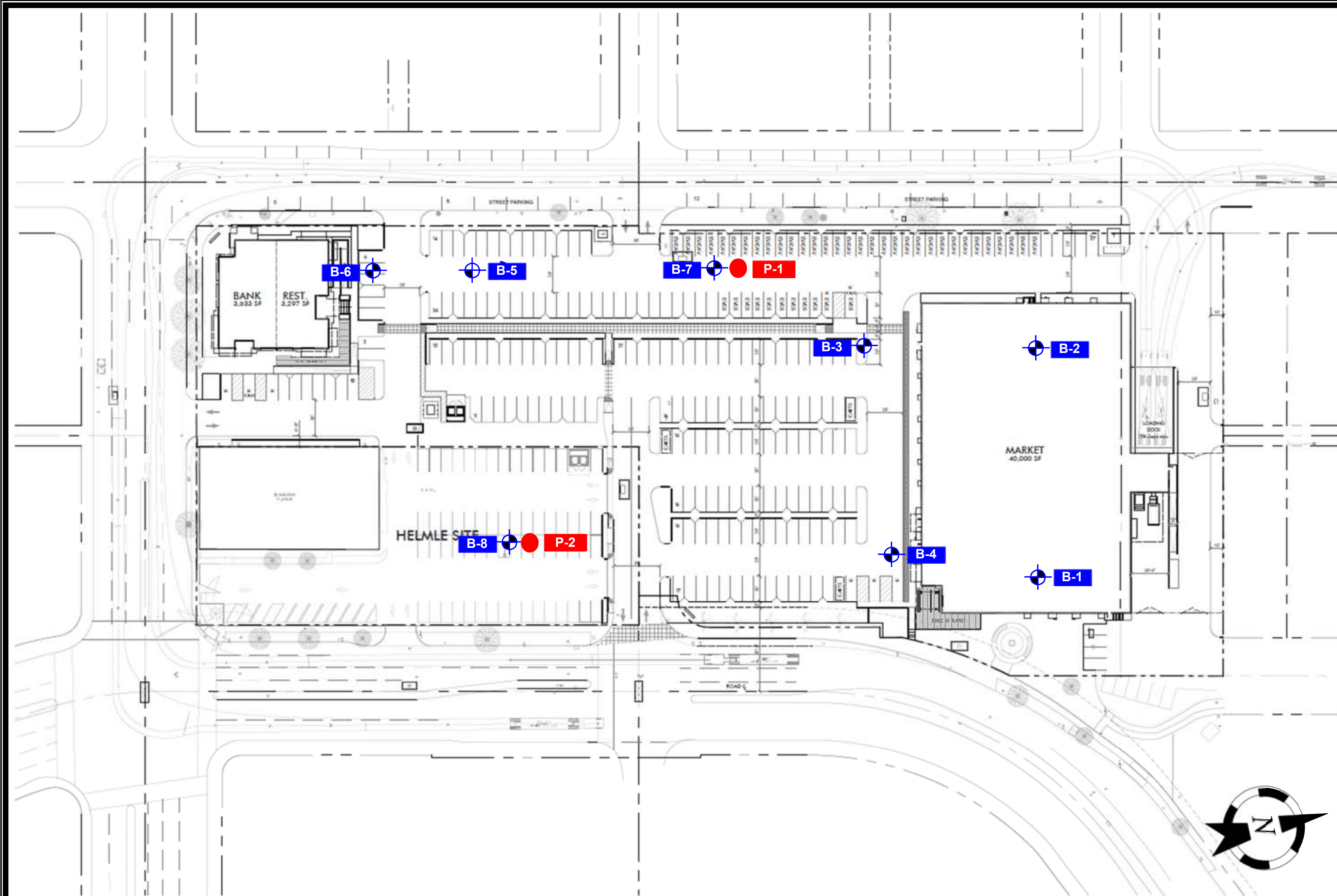
Respectfully submitted,

SALEM ENGINEERING GROUP, INC.

Jared Christiansen, MS, PE
Geotechnical Project Engineer
RCE 94900

Clarence Jiang, GE
Senior Geotechnical Engineer
RGE 2477

Attachment: Site Plan, Figure 1



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Retail Development
NWC of 6th Street and Main Street
Corona, California

SCALE: NOT TO SCALE	DATE: 04/2024
DRAWN BY: JC	APPROVED BY: CJ
PROJECT NO. 3-222-1216	FIGURE NO. 1

LEGEND:

- B-1 Soil Boring Locations
- P-1 Percolation Locations

All Locations Approximate

SALEM engineering group, inc.