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SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED FUEL STATION AND CAR WASH
CHICAGO AVENUE AND VAN BUREN BOULEVARD
RIVERSIDE, CALIFORNIA

SALEM PROJECT NO. 3-219-0749
SEPTEMBER 30, 2019

PREPARED FOR:

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September 30, 2019 | Project No. 3-219-0749

Mr. Oscar Etemadian
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SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED FUEL STATION AND CAR WASH
CHICAGO AVENUE AND VAN BUREN BOULEVARD
RIVERSIDE, CALIFORNIA

Dear Mr. Etemadian:

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

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED FUEL STATION AND CAR WASH
CHICAGO AVENUE AND VAN BUREN BOULEVARD
RIVERSIDE, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation for the Proposed Fuel Station and Car Wash to be located at NEC Chicago Avenue and Van Buren Boulevard in Riverside, 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 did not include a slope stability analysis.

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 September 11, 2019 and included the drilling of seven (7) small-diameter soil borings to a maximum depth of 24 feet at the site. Additionally, two (2) percolation tests were performed at depths of approximately 5 and 10 feet below existing grade. The locations of the soil borings and infiltration tests are depicted on Figure 2, Site Plan. A detailed discussion of our field investigation and exploratory boring logs are presented in Appendix A.

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

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

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

2. PROJECT DESCRIPTION

We understand that the proposed development of the site will include construction of a fueling station consisting of a convenience store, 8-pump canopy, underground storage tanks, and a 70-foot tunnel car wash. 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 60 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 proposal. As the existing is gently sloping with a ditch along the northern boundary and bisecting the northeast portion of the site, we anticipate that cuts and fills during earthwork will be moderate in order to provide a 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 triangular in shape and encompasses approximately 2.4 acres. The site is located on the northeast corner of Chicago Avenue and Van Buren Boulevard in the County of Riverside, California (see Vicinity Map, Figure 1).

The site is currently a vacant land with shrubs, trees, and weeds. The site is gently sloping to the southeast with a ditch along the northern boundary and bisecting the northeast portion of the site. Dense trees are present at the watercourse area. Site elevations range from approximately 1581 to 1563 feet above mean sea level based on Google Earth imagery. Site grades are estimated to be no greater than about 10H to1V in the area of the planned development.

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-7) were drilled on September 11, 2019 in the areas shown on the Site Plan, Figure 2. The test borings were advanced with a 6-inch diameter solid flight auger rotated by a truck-mounted CME 45C drill rig. The test borings were extended to a maximum depth of 24 feet below existing grade. The depth of drilling was limited due to auger refusal on very dense soil or bedrock.

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

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

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

5. LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory-testing program was formulated with emphasis on the evaluation of natural moisture, density, shear strength, consolidation potential, expansion index, 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 northern part of the Peninsular Ranges Geomorphic Province of California. The province varies in width from approximately 30 miles to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks and Cretaceous igneous rocks of the Southern California batholith.

The Peninsular Ranges Province is divided into three northwest-trending fault-bounded structural blocks, from west to east, the Santa Ana Mountains, Perris, and San Jacinto Mountains. The Santa Ana Mountains block (southwest of the subject site) extends from the coast to the Elsinore Fault zone. The northern margin of the Perris structural block underlies the subject site. Paleocene to Pliocene sedimentary rocks underlie the western portion of the Santa Ana Mountains structural block. The eastern portion, a highly faulted structural anticline, is cored by a basement assemblage of Mesozoic metasedimentary and Cretaceous batholithic and volcanic rocks.

A thick section of primarily upper Cretaceous marine and Paleocene marine and nonmarine rocks overly this basement. The Perris structural block is a large mass of granitic rock generally bounded by the San Jacinto Fault, the Elsinore Fault, the Santa Ana River and a non-defined southeast boundary. The Perris Block has had a history of vertical land movements of several thousand feet due to shifts in the Elsinore and San Jacinto Faults. Several erosional and depositional surfaces are developed on the Perris block and thin to relatively thick sections of nonmarine, mainly Quaternary sediments discontinuously cover the basement.

Based on review of the Geologic Map of the Riverside East 7.5' Quadrangle [1]the site is mapped in an area of Val Verde tonalite (Kvt). This material is described as "Gray-weathering, relatively homonenous, massive to well foliated, medium to coarse grained, hypautomorphic granular biotite hornblende tonalite; principal rock type of Val Verde pluton."

The materials encountered during drilling are generally similar to those mapped in the vicinity of the site.

Morton, Douglas M., and Cox, Brett F. (2001), Geologic Map of the Riverside East 7.5' quadrangle, Riverside County, California, Version 1.0: U.S. Geological Survey, scale 1:24,000.

7. GEOLOGIC HAZARDS

7.1 Faulting and Seismicity

The Peninsular Range has historically been a province of relatively high seismic activity. The nearest faults to the project site are associated with the San Jacinto fault system located approximately 11.0 miles from the site. There are no known active fault traces in the project vicinity. Based on mapping and historical seismicity, the seismicity of the Peninsular Range has been generally considered high by the scientific community.

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

To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 33.8869° North; site longitude is -117.3483° West. The ten closest active faults are summarized below in Table 7.1.

TABLE 7.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
San Jacinto; SBV+SJV+A+CC+B+SM	11.0	7.9
San Jacinto; SBV	11.0	7.1
Elsinore; W+GI+T+J+CM	11.7	7.9
Chino, alt 2	13.3	6.8
San Jacinto; A+CC+B+SM	13.8	7.6
Elsinore; W	14.5	7.0
Chino, alt 1	14.6	6.7
Elsinore; T+J+CM	15.3	7.6
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO	19.5	8.2
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB	19.5	8.0

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

Based on the 2016 CBC, a Site Class C was selected for the site based on soil conditions encountered and our experience in the vicinity of the subject site. Table 9.2.1 includes design seismic coefficients and spectral response parameters, based on the 2016 California Building Code (CBC) for the project foundation design.

Based on Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.6g (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 24 feet on the project site consisted predominately of soft to stiff sandy silt; and loose to very dense clayey sand, silty sand, poorly graded sand, and well-graded sand. However, it should be noted that the materials encountered greater than about 1 to 5 feet below site grade are considered very dense (N-values greater than 50 blows per foot). The very dense materials encountered are consistent with tonalite bedrock material mapped in the vicinity of the site.

The historically highest groundwater is estimated to be at a depth of approximately 12 to 15 feet below ground surface according to nearby monitoring well data. Based on the presence of very dense material encountered it is anticipated that the groundwater depth reported is due to perched water conditions.

Low to very low cohesion strength is associated with the sandy soil. A seismic hazard, which could cause damage to the proposed development during seismic shaking, is the post-liquefaction settlement of the liquefied sands. The Riverside County Office of Information Technology GIS website: http://mmc.rivcoit.org/MMC_Public/ Viewer.html?Viewer=MMC_Public, shows the subject site to be in a very low liquefaction potential area (Figure 5, Liquefaction Potential Zone Map).

Furthermore, based on the very dense soil/rock conditions encountered the liquefaction potential of the site is considered to be low due the relatively dense/stiff soil conditions.

7.5 Lateral Spreading

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

7.6 Flood and Dam Inundation

The Riverside County Office of Information Technology GIS website shows the subject site is not located in a flood zone (see Figure 6, Flood Zone Map).

7.7 Subsidence/Fissure Potential Zones

The Riverside County Office of Information Technology GIS website shows the subject site is not within a susceptible subsidence potential area (see Figure 7, Subsidence Zone Map). SALEM is not aware of subsidence issues in the immediate project site vicinity.

7.8 Collapsible/Expansive or Hydroconsolidatable Soils

Test data in this geotechnical report show that soil samples consolidated from approximately 6 to 13 percent after a maximum 16 ksf load. Hydroconsolidation (collapse upon wetting) at a load of 2 ksf was from approximately 0.4 to 1.6 percent. Therefore, the on-site soils have slight collapse potential. Soil samples collected from surface to the proposed foundation depths are considered to have a low expansion potential and the sample tested returned and Expansion Index value of 11. The proposed site preparation methods recommended on our geotechnical report should address these geotechnical issues and no additional mitigation measures are required.

7.9 Landslides/Slope Instability/Debris Flow

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. The subject site is on a gently (<5%) sloping grade, over ½ mile from the nearest significant topographic change. As such, landslide/slope instability/rock fall issues pose a very low risk. Due to the site’s distance from significant topography, topography-related debris flows are a low risk.

Furthermore, it is our understanding graded slopes will be no steeper than 2H to 1V and with heights of about 10 feet of less. Provided the recommendations for grading included in this geotechnical report are followed, slope stability and/or surficial instability are not a concern for the planned development.

7.10 Wind and Water Erosion

Based on SALEM’s soil boring logs for the subject site, surface soils consist predominantly of soft to stiff sandy silt; and loose to very dense clayey sand, silty sand, poorly graded sand, and well-graded sand. Soils of this composition and consistency have been shown to possess good resistance to wind and water erosion. The site is essentially flat, minimizing the potential for water erosion. The site will be mostly covered by buildings, pavement or landscaping after development, minimizing long-term wind erosion potential.

7.11 Tsunamis and Seiches

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

8. SOIL AND GROUNDWATER CONDITIONS

8.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils within the depth of exploration consisted predominately of alluvium deposits of soft to stiff sandy silt; and loose to very dense clayey sand, silty sand, poorly graded sand, and well-graded sand.

Fill soils may present onsite between our test borings. Verification of the extent of fill should be determined during site grading. Field and laboratory tests suggest that the deeper native soils are moderately strong and slightly compressible. These soils extended to the termination depth of our borings.

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

8.2 Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was encountered at a depth of 19 feet during this investigation. The historically highest groundwater is anticipated to be at a depth of approximately 12 to 15 feet below existing grade based on the local monitoring 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 107 mg/kg. ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class. ACI 318 requirements for site concrete based upon soluble sulfate are summarized in Table 8.3 below.

TABLE 8.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Water Soluble Sulfate (SO_4) in Soil, % by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Min. Concrete Compressive Strength	Cementations Materials Type
0.0107	Not Severe	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 19 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 Infiltration Testing

Two (2) 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. 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 percolation rate data are presented in tabular format at the end of this Report. The difference in the percolation rates are reflected by the varied type of soil materials at the bottom of the test holes. The test results are shown on the table below.

Test No.	Depth (Feet)	Measured Percolation Rate (min/inch)	Infiltration Rate* (inch/hour)	Soil Type**
P-1	10	6.4	**1.16**	Silty SAND (SM)
P-2	5	2.2	**4.61**	Silty SAND (SM)

*** Tested infiltration Rate = $(\Delta H\ 60\ r) / (\Delta t(r + 2H_{avg}))$**
**** At bottom of drilled holes**

Based on the results of the infiltration test performed and relative density of the materials encountered, an infiltration rate of 1.16 inches per hour may be used in design. 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 infiltration 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 infiltration testing services in relatively small areas (double rings) that the testing may not fully model the actual full scale long term performance of a given site. This is particularly true where infiltration test data is to be used in the design of large infiltration areas such as those proposed for the site. Subsurface conditions, including infiltration 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 undocumented potentially compressible materials at the site. Recommendations to mitigate the effects of these soils are provided in this report.

9.1.3 Undocumented fill materials may be present onsite between our boring locations. Undocumented fill materials are not suitable to support any future structures and should be replaced with Engineered Fill. The extent and consistency of the fills should be verified during site construction. Prior to fill placement, Salem Engineering Group, Inc. should inspect the bottom of the excavation to verify the fill condition.

9.1.4 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 6 to 8 inches of the soils containing, vegetation, roots and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. The stripped vegetation, will not be suitable for use as Engineered Fill or within 5 feet of building pads or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

9.1.5 Tree roots were present in some of the samples collected from the borings. Tree root systems in the proposed improvement areas should be removed to a minimum depth of 4 feet and to such an extent which would permit removal of all roots. 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.

9.1.6 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.

9.1.7 The near-surface onsite soils are moisture-sensitive and are s compressible, and exhibited slight collapse potential under saturated conditions. Structures within the project vicinity have experienced excessive post-construction settlement, when the foundation soils become near saturated. The collapsible or weak soils should be removed and recompacted according to the recommendations in the Grading section of this report (Section 9.5).

9.1.8 The scope of our services for the investigation does not include a slope stability evaluation of the site. Slopes should be constructed in accordance with the typical figures and details as shown in the General Earthwork and Pavement Specifications, Appendix C (i.e. Stabilization Fill, Buttress Fill, Daylight Shear key, Shear Key, Fill Slope above Natural Ground, Fill Slope

Above Cut Slope, Backdrain, Geofabric Subdrain, Benching for Compacted Fill, Rock Disposal, Canyon Subdrain and Transition Lot).

9.1.9 Where fill slopes are to be constructed on original ground that slopes steeper than 6:1 (horizontal to vertical), the ground should be stepped or benched. The benches should be cut into the dense slope as the grading operations proceed. The first bench (base or key bench) should be at least 15 feet wide. Each bench should consist of a minimum 8 feet wide of level terrace, with the rise to the next bench held for 4 feet or less.

9.1.10 The horizontal distance between the outer edges of the footing bottom and the adjacent firm/compacted slope face should be at least 5 feet.

9.1.11 To reduce the erosion of graded slopes, it is recommended that all slopes be planted with ground cover vegetation and deep rooted vegetation as soon as practical. The proper maintenance of proper lot drainage and vegetation should be performed. Over-irrigation should be prevented. A rodent control program should be established and maintained.

9.1.12 All surface runoff should be directed away from the slope and toward approved drainage devices.

9.1.13 All infiltration facilities or retention basins shall be located a minimum of 10 feet away from any foundations and/or slopes (descending or ascending).

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

9.1.15 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.

9.1.16 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 2016 CBC, our recommended parameters are shown below. These parameters were determined using California's Office of Statewide Health Planning and Development (OSHPD) (https://seismicmaps.org/) in accordance with the 2016 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
Site Coordinates (Datum = NAD 83)		33.8869 Lat -117.3483 Lon
Site Class	--	C
Soil Profile Name	--	Very Dense Soil and Soft Rock
Risk Category	--	II
Site Coefficient for PGA	F_{PGA}	1.2
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.6 g
Seismic Design Category	SDC	D
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	1.5 g
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.562 g
Site Class Modified Site Coefficient	F_a	1.2
Site Class Modified Site Coefficient	F_v	1.438
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	1.8 g
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	0.808 g
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.2 g
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	0.539 g

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

9.3 Soil and Excavation Characteristics

9.3.1 Based on the soil conditions encountered in our soil borings, the upper soils can be excavated with moderate effort using conventional heavy-duty earthmoving 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 potentially 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 expansion and collapse potential. As recommended in Section 9.5, the collapsible soils should be overexcavated and recompacted. Mitigation measures will not eliminate post-construction soil movement, but will reduce the soul 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, damp to moist due to the absorption characteristics of the soil. Earthwork operations may encounter 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 have an Expansion Index of 20 or less (EI≤20) and do not contain deleterious matter, organic material, or rock material larger than 3 inches in maximum dimension.

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

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

TABLE 9.4.3
IMPORT FILL REQUIREMENTS

Minimum Percent Passing No. 200 Sieve	20
Maximum Percent Passing No. 200 Sieve	50
Minimum Percent Passing No. 4 Sieve	80
Maximum Particle Size	3"
Maximum Plasticity Index	12
Maximum CBC Expansion Index	20

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

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

9.5 Grading

9.5.1 A SALEM representative 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 4 to 8 inches of the soils containing vegetation, roots, and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. In addition, existing concrete and asphalt materials shall be removed from areas of proposed improvements and stockpiled separately from excavated soil material. The stripped vegetation, asphalt, and concrete materials will not be suitable for use as Engineered Fill or within 5 feet of building pads or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

9.5.5 Fill soils may be present onsite between our test boring locations. All fill materials encountered during grading should be removed and replaced with engineered fill. The actual depth of the overexcavation and recompaction should be determined by our field representative during construction.

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

9.5.7 To minimize post-construction soil movement and provide uniform support for the proposed buildings, it is recommended that the overexcavation and recompaction within the proposed building areas be performed to a minimum depth of **three (3)** 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.

9.5.8 Within pavement areas, overexcavation and recompaction should be performed to a minimum depth of **two (2) feet** below existing grade or proposed grade, whichever is deeper. The overexcavation and recompaction should also extend laterally to a minimum of 2 feet beyond the pavement edges.

9.5.9 Prior to placement of fill soils, the upper 8 to 10 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 percent (90% for fine grained, cohesive soils) of the maximum dry density based on ASTM D1557-07 Test Method.

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

9.5.11 Engineered Fill soils should be moisture conditioned to slightly above optimum moisture content, and compacted to at least 95% relative compaction.

9.5.12 Non-Expansive Engineered Fill should be moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction.

9.5.13 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.

9.5.14 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.

9.5.15 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.

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

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

To expedite the stabilizing process, crushed rock may be utilized for stabilization provided this method is approved by the owner for the cost purpose. If the use of crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of ¾-inch to 1-inch crushed rocks. The thickness of the rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of crushed rock material will provide a stable platform. It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock. A layer of geofabric is recommended to be placed on top of the compacted crushed rock to minimize migration of soil particles into the voids of the crushed rock, resulting in soil movement. Although it is not required, the use of geogrid (e.g. Tensar TX7) below the crushed rock will enhance stability and reduce the required thickness of crushed rock necessary for stabilization.

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

9.6 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 18 inches and extend to a minimum depth of 18 inches below the lowest adjacent grade. Isolated column footings should have a minimum width of 24 inches and extend a minimum depth of 18 inches below the lowest adjacent grade.

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

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	2,500 psf
Dead-Plus-Live Load	3,000 psf
Total Load, Including Wind or Seismic Loads	4,000 psf

9.6.5 For design purposes, total settlement due to static loading on the order of 1 inch may be assumed for shallow footings. Differential settlement due to static loading, 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 0.50 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 450 pounds per cubic foot acting against the appropriate vertical native footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combinations that includes wind or earthquake loads.

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

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

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

9.7 Concrete Slabs-on-Grade

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

9.7.2 Granular aggregate subbase material shall conform to ASTM D-2940, Latest Edition (Table 1, bases) with at least 95 percent passing a 1½-inch sieve and not more than 8% passing a No. 200 sieve or its approved equivalent to prevent capillary moisture rise. Crushed Miscellaneous Base (CMB) should <u>not</u> be used as subbase material 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 200 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 Pier Foundations

9.8.1 It is recommended that the Cast in Drilled Hole (CIDH) Pier foundation should have a minimum depth of 10 feet below the lowest adjacent grade.

9.8.2 The CIDH Piers may be designed using an allowable sidewall friction of 250 psf. This value is for dead-plus-live loads. An allowable end bearing capacity of 3,000 psf may be used provided that the bottom of the pier is cleaned with the use of a clean-out bucket or equivalent and inspected by our representative prior to placement of reinforcement and concrete. An increase of one-third is permitted when using the alternate load combination that includes wind or earthquake loads.

9.8.3 Uplift loads can be resisted by piers using an allowable sidewall friction of 200 psf of the surface area and the weight of the pier.

9.8.4 The total static settlement of the pier foundation is expected to be less than 1 inch. Differential static settlement should be less than ½ inch over 20 feet. Most of the settlement is expected to occur during construction as the loads are applied.

9.8.5 The CIDH piers may be designed for a lateral capacity of 450 pounds per square foot per foot of depth below the lowest adjacent grade to a maximum of 6,750 psf.

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

9.8.7 These values may be increased by one-third when using the alternative load combinations in that include wind or earthquake loads. The lateral loading criteria is based on the assumption that the load application is applied at the ground level and flexible cap connections applied.

9.8.8 Sandy soils and groundwater were encountered at the site. Casing will be required during drilling of the pier footings.

9.9 Lateral Earth Pressures and Frictional Resistance

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

Lateral Pressure Conditions	Equivalent Fluid Pressure, pcf
Active Pressure, Drained	30
At-Rest Pressure, Drained	48
Passive Pressure	450
Related Parameters	
Allowable Coefficient of Friction	0.50
In-Place Soil Density (lbs/ft^3)	120

9.9.2 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.

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

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

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

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

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

9.9.8 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

9.10 Retaining Walls

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

9.10.2 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.

9.10.3 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.

9.10.4 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.

9.10.5 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.

9.11 Temporary Excavations

9.11.1 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.

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

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

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

RECOMMENDED EXCAVATION SLOPES

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

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

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

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

9.12 Underground Utilities

9.12.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than 3 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches and compacted to at least 95% (90% for fine grained, cohesive soils) relative compaction at or above optimum moisture content.

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

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

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

9.13 Surface Drainage

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

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

9.13.3 Impervious surfaces within 10 feet of the 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.

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

9.14 Pavement Design

9.14.1 Based on site soil conditions, an R-value of 45 was used for the preliminary flexible asphaltic concrete pavement design. The R-value may be verified during grading of the pavement areas.

9.14.2 The pavement design recommendations provided herein are based on the State of California Department of Transportation (CALTRANS) design manual. The asphaltic concrete (flexible pavement) is based on a 20-year pavement life utilizing 1200 passenger vehicles, 10 single unit trucks, and 2 multi-unit trucks. The following table shows the recommended pavement sections for various traffic indices.

TABLE 9.14.2
ASPHALT CONCRETE PAVEMENT

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

95% compaction based on ASTM D1557-07 Test Method

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

TABLE 9.14.3
PORTLAND CEMENT CONCRETE PAVEMENT

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

**** Minimum Compressive Strength of 4,000 psi***
***** 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 aggregate to the site should be screened to determine the potential for corrosion to concrete and buried metal piping.

The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report. If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (909) 980-6455.

Respectfully Submitted,

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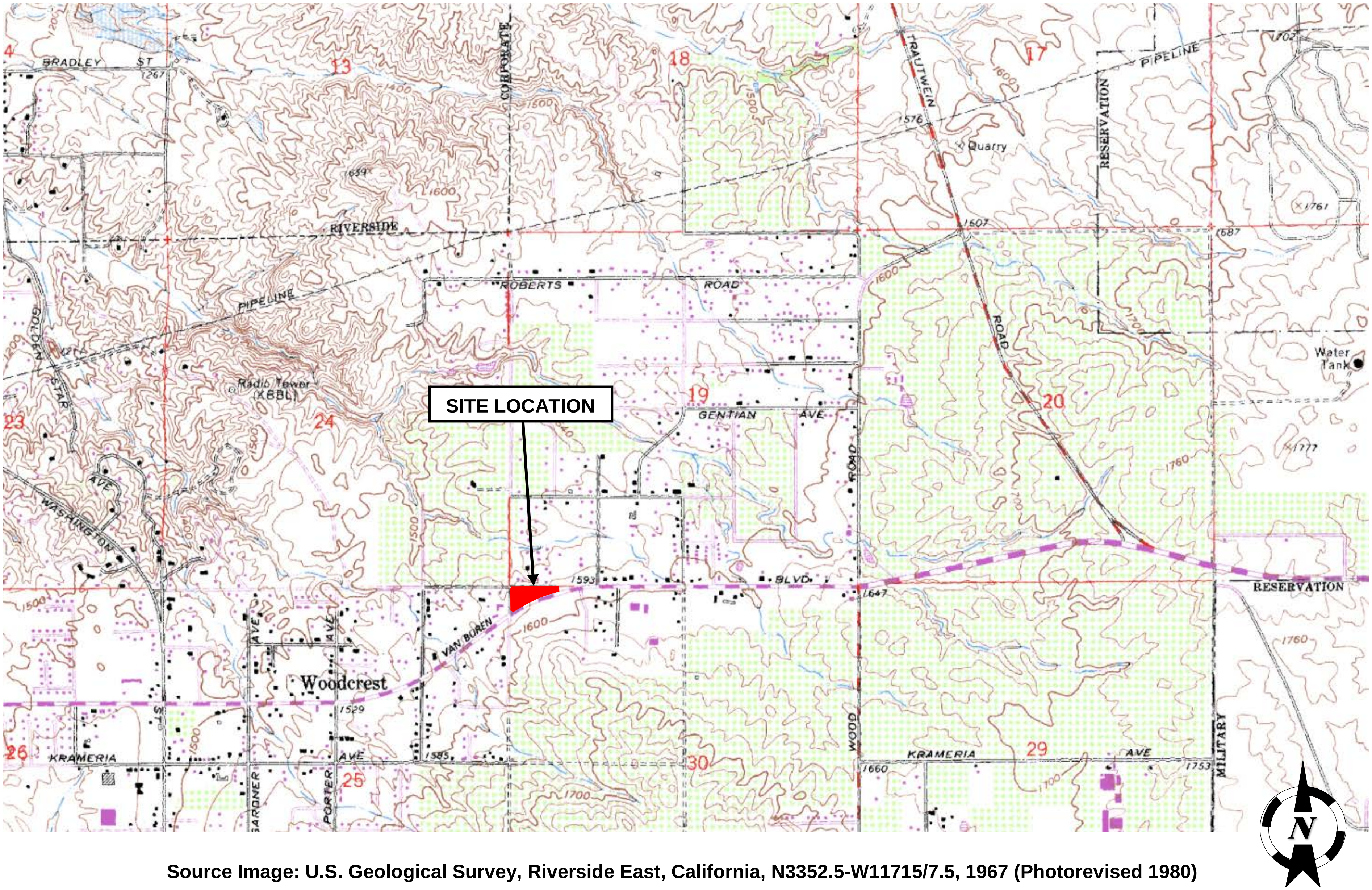
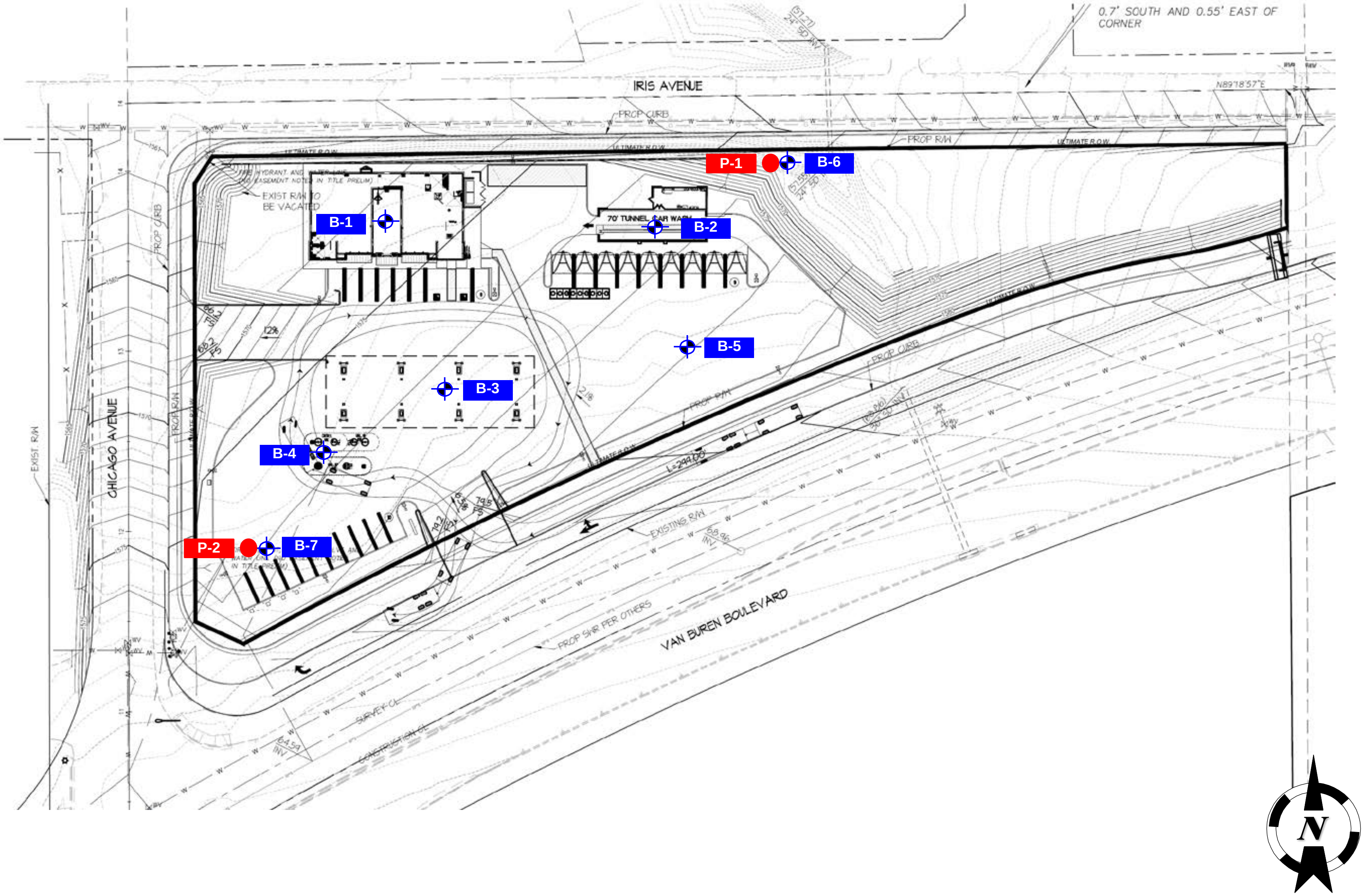


Source Image: U.S. Geological Survey, Riverside East, California, N3352.5-W11715/7.5, 1967 (Photorevised 1980)

VICINITY MAP	SCALE: NOT TO SCALE	DATE: 10/2019	SALEM engineering group, inc.
GEOTECHNICAL ENGINEERING INVESTIGATION Proposed Fuel Station, C-Store & Car Wash NEC Chicago Avenue & Van Buren Boulevard Riverside, California	DRAWN BY: KV	APPROVED BY: CJ	
	PROJECT NO. 3-219-0749	FIGURE NO. 1	



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Fuel Station, C-Store & Car Wash
NEC Chicago Avenue & Van Buren Boulevard
Riverside, California

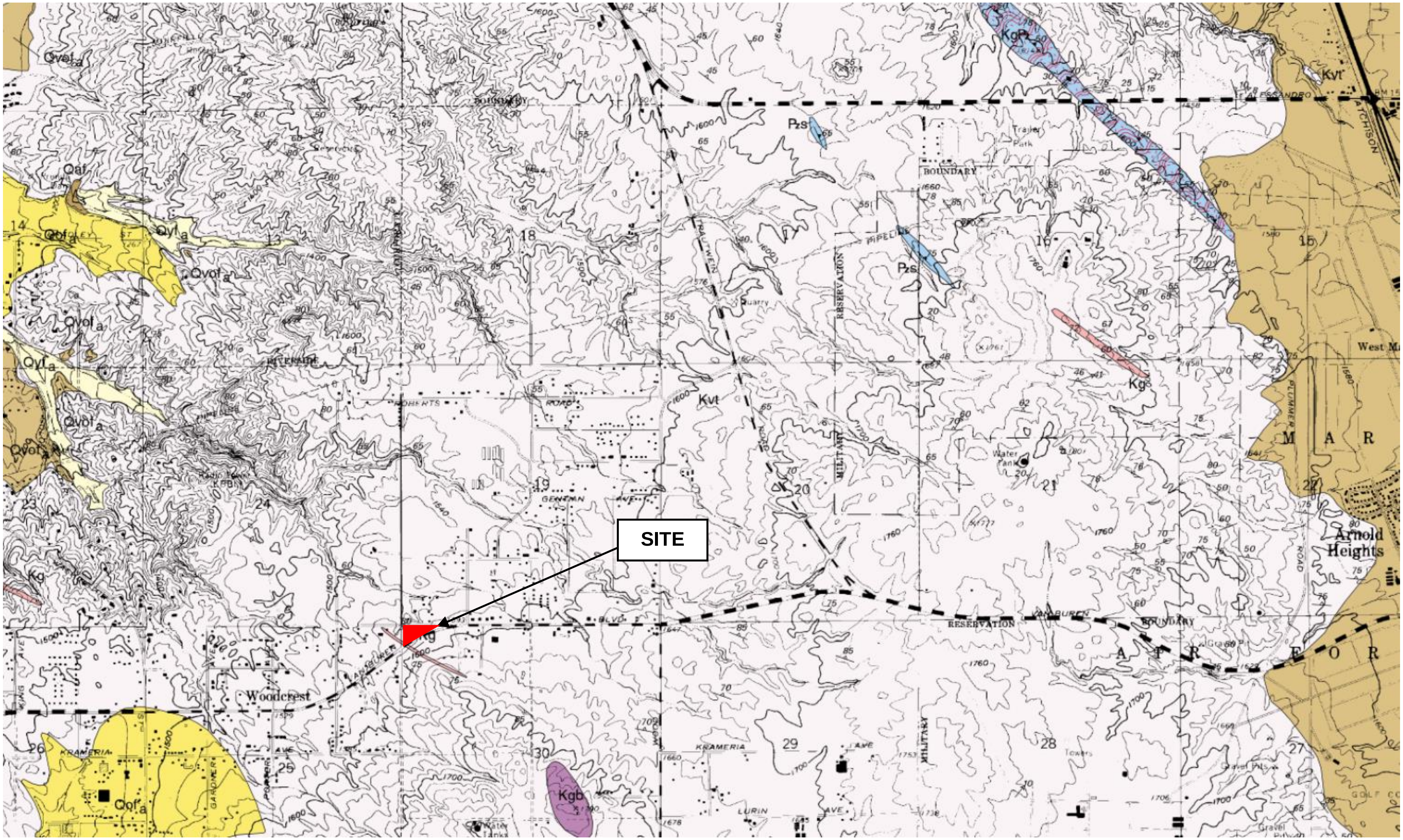
SCALE: NOT TO SCALE	DATE: 10/2019
DRAWN BY: KV	APPROVED BY: CJ
PROJECT NO. 3-219-0749	FIGURE NO. 2

LEGEND:

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

All Locations Approximate

SALEM engineering group, inc.

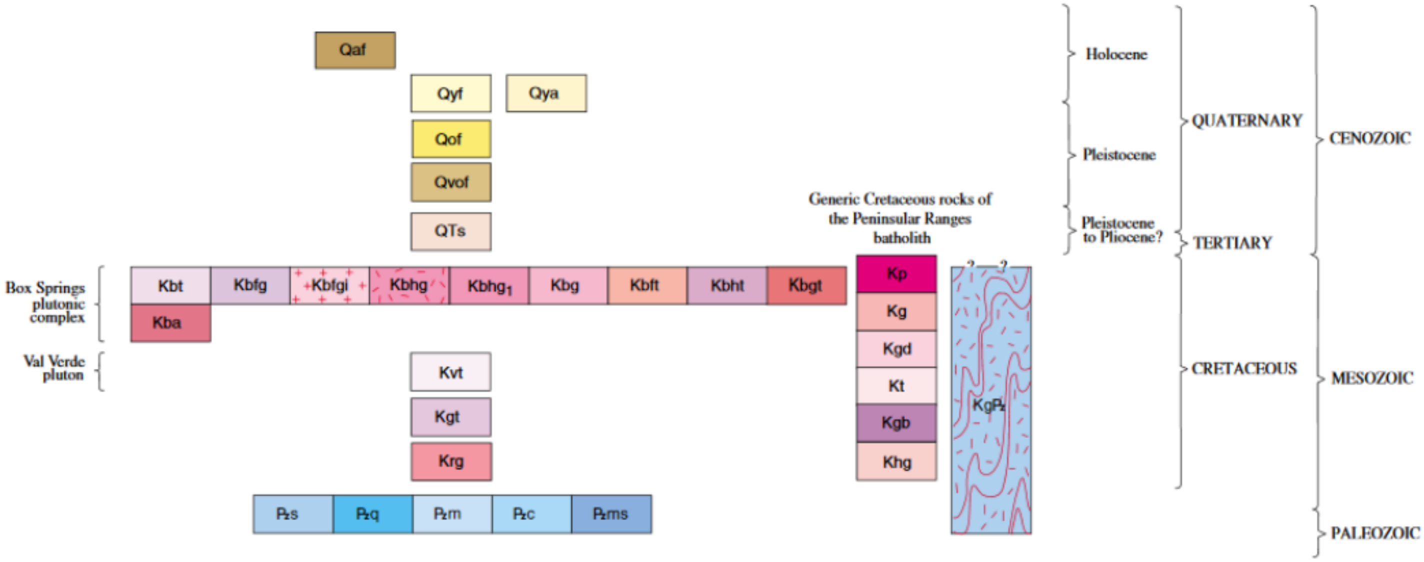


Morton, Douglas M., and Cox, Brett F. (2001), Geologic Map of the Riverside East 7.5’ quadrangle, Riverside County, California, Version 1.0: U.S. Geological Survey, scale 1:24,000.

Regional Geologic Map	SCALE: NOT TO SCALE	DATE: 10/2019
GEOTECHNICAL ENGINEERING INVESTIGATION **Proposed Fuel Center, Convenience Store, & Car Wash** **NEC Chicago Avenue & Van Buren Boulevard** **Riverside, California**	DRAWN BY: JC	APPROVED BY: CJ
	PROJECT NO. 3-219-0749	FIGURE NO. 3A

SALEM engineering group, inc.

Geologic Unit Explination



Symbol Explination

Contact—Generally located within ±15 meters

70

Fault—High angle. Strike-slip component on all faults is right-lateral; dip-slip component is unknown, but probably reflects valley-highland relations. Dashed where located within ±30 meters; dotted where concealed. Arrow and number indicate measured dip of fault plane that was exposed in trench.

Strike and dip of igneous foliation

70 **Inclined**

Vertical

Strike and dip of metamorphic foliation

70 **Inclined**

Vertical

Strike and dip of joints in igneous rocks

70 **Inclined**

Vertical

Bearing and plunge of linear features

70 **Inclined**

Morton, Douglas M., and Cox, Brett F. (2001), Geologic Map of the Riverside East 7.5' quadrangle, Riverside County, California, Version 1.0: U.S. Geological Survey, scale 1:24,000.

Regional Geologic Map **GEOTECHNICAL ENGINEERING INVESTIGATION** **Proposed Fuel Center, Convenience Store, & Car Wash** **NEC Chicago Avenue & Van Buren Boulevard** **Riverside, California**	SCALE: NOT TO SCALE	DATE: 10/2019
	DRAWN BY: JC	APPROVED BY: CJ
	PROJECT NO. 3-219-0749	FIGURE NO. 3B

SALEM engineering group, inc.

Geologic Unit Explination - Details

VERY YOUNG SURFICIAL DEPOSITS—Sediment recently transported and deposited in channels and washes, on surfaces of alluvial fans and alluvial plains, and on hillslopes. Soil-profile development is non-existant. Includes:

Qaf — **Artificial fill (late Holocene)**—Deposits of fill resulting from human construction or mining activities; restricted to large area of regrading related to residential development in west central part of quadrangle and several smaller areas nearby

YOUNG SURFICIAL DEPOSITS—Sedimentary units that are slightly consolidated to cemented and slightly to moderately dissected. Alluvial fan deposits (Qyf series) typically have high coarse:fine clast ratios. Younger surficial units have upper surfaces that are capped by slight to moderately developed pedogenic-soil profiles (A/C to A/AC/$B_{cambric}C_{ox}$ profiles). Includes:

Qyf — **Young alluvial fan deposits (Holocene and late Pleistocene)**—Gray-hued sand and cobble- and gravel-sand deposits derived chiefly from rocks of Peninsular Ranges batholith. Found in restricted drainages along west edge of quadrangle, but contiguous with much more extensively developed deposits west of quadrangle

Qya — **Young axial channel deposits (Holocene and late Pleistocene)**—Gray, unconsolidated alluvium consisting of medium- to fine-grained sand and lesser silt flooring several low relief valleys and their tributaries in northwestern and northeastern part of quadrangle. Includes sediments in Tequesquite Arroyo and Pigeon Pass Valley

OLD SURFICIAL DEPOSITS—Sedimentary units that are moderately consolidated and slightly to moderately dissected. Older surficial deposits have upper surfaces that are capped by moderately to well-developed pedogenic soils (A/AB/B/Cox profiles and Bt horizons as much as 1 to 2 m thick and maximum hues in the range of 10YR 5/4 and 6/4 through 7.5YR 6/4 to 4/4 and mature Bt horizons reaching 5YR 5/6). Includes:

Qof — **Old alluvial fan deposits (late to middle Pleistocene)**—Indurated, sandy alluvial fan deposits developed extensively in western part of quadrangle. Most of unit is slightly to moderately dissected and reddish-brown. Some Qof includes thin, discontinuous surface layer of Holocene alluvial fan material

VERY OLD SURFICIAL DEPOSITS—Sediments that are slightly to well consolidated to indurated, and moderately to well dissected. Upper surfaces are capped by moderate to well developed pedogenic soils (A/AB/B/C_{OX} profiles having Bt horizons as much as 2 to 3 m thick and maximum hues in the range 7.5YR 6/4 and 4/4 to 2.5YR 5/6)

Qvof — **Very old alluvial fan deposits (early Pleistocene)**—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Forms large area in southeastern part of quadrangle in area of March Air Force Base, and numerous smaller areas in northern part of quadrangle. Derived chiefly from rocks of southern California batholith

QTs — **Unnamed late Cenozoic sedimentary rocks in Riverside and Corona areas (early Pleistocene to late Pliocene?)**—Lithologically diverse, moderately indurated, gray to brown, coarse-grained sandstone, pebbly sandstone, and conglomerate. Restricted to two small areas near southeast end of Box Springs Canyon. In the Riverside West 7.5' quadrangle, most clasts in unit were derived from San Bernardino Mountains. In Riverside East 7.5' quadrangle, appears to be derived from units found in Santa Ana River drainage. Southeast of Riverside, clasts are locally derived from Peninsular Ranges sources

Box Springs plutonic complex (Cretaceous)—Box Springs plutonic complex is an elliptical, horizontally-floored basin-shaped granitic complex centered on Box Springs Mountains; apparently lower part of granitic diapir. Layering and foliation in granitic rocks is primary. Complex consists of essentially massive to indistinctly primary layered biotite tonalite in core, surrounded by layer of foliated biotite granodiorite to tonalite. Further outward in complex is discontinuous layer of foliated, heterogeneous, porphyritic granodiorite, succeeded by uniform porphyritic granodiorite. Other compositionally and texturally diverse granitic rocks also occur within complex, but in relatively small amounts. All rocks of complex were included in Perris quartz diorite by Dudley (1935) and in Bonsall tonalite by Larsen (1948). Except for dike rocks, units are described in general order from core outward. Includes:

Kbt — **Biotite tonalite**—Massive, fine- to medium-grained, equigranular biotite tonalite. Much has faintly to moderately developed, very regular compositional layering. Rocks contain about 35 to 40 percent quartz and 6 to 12 percent biotite. Hornblende is absent and potassium feldspar ranges from 1 to 4 percent. Mineral alignment is poorly developed or absent, but much of rock has incipient to well-developed primary layering defined by mineral concentrations. Unit contains sparse equant- to elliptical-shaped, fine-grained, mesocratic inclusions; some have relatively mafic rims. Inclusions tend to be aligned parallel to compositional layering. Zircon ages of rock are 98.6 Ma_{id} and 100.4 Ma_{ip}

Kbfg — **Biotite granodiorite and tonalite**—Light gray, medium- to coarse-grained, foliated biotite granodiorite and tonalite. Contains 25 to 35 percent quartz, 8 to 15 percent biotite, and minor hornblende. Potassium feldspar occurs as small interstitial grains and sparse, subhedral phenocrysts up to 1.5 cm in diameter. Potassium feldspar content appears to decrease progressively inward; tonalite most abundant in inner part. Mesocratic discoidal inclusions oriented parallel to foliation are common, but not abundant. Grades into biotite tonalite unit (Kbt)

+Kbfgi — **Biotite granodiorite and tonalite containing abundant inclusions**—Biotite granodiorite and tonalite that contains abundant discoidal, mafic inclusions; restricted to east side of complex

Kbhg — **Heterogeneous porphyritic granodiorite**—Heterogeneous porphyritic granodiorite and subordinate tonalite. In most places surrounds biotite granodiorite and tonalite unit (Kbfg). Pinches out along west side of complex. Medium- to coarse-grained, light gray, foliated, and porphyritic. Quartz ranges from 25 to 35 percent; mafic minerals, biotite and subordinate hornblende, from 10 to 15 percent. Mafic minerals unevenly distributed imparting heterogeneous appearance to rock. Subhedral potassium feldspar crystals are up to 2.5 cm in length. Widespread discoidal mesocratic inclusions oriented parallel to foliation. Cut by numerous dikes of leucocratic granite and pegmatite

$Kbhg_1$ — **Layered heterogeneous porphyritic granodiorite**—Heterogeneous porphyritic granodiorite having pronounced layering that is defined chiefly by variations in grain size. Restricted to single mass west of Sugarloaf Mtn in north-central part of quadrangle

Kbg — **Porphyritic granodiorite**—Coarse-grained, light gray, foliated, porphyritic biotite granodiorite and subordinate tonalite. In most places grades into heterogeneous porphyritic granodiorite unit (Kbhg). Groundmass is plagioclase, quartz (30 to 40 percent), and mafic minerals (5 to 10 percent). Mafic minerals are biotite and sparse hornblende, which are more evenly distributed than in heterogeneous granodiorite (Kbhg). Subhedral potassium feldspar phenocrysts are up to 2.5 cm in length. Discoidal mesocratic inclusions are oriented parallel to foliation

Kbft — **Biotite-hornblende tonalite**—Light to medium gray, medium- to coarse-grained, foliated tonalite. Forms discontinuous, pod-shaped masses surrounding, but not in contact with, biotite tonalite (Kbt). Contains 20 to 25 percent quartz and about 25 percent biotite and hornblende in subequal amounts. Hornblende and biotite occur as ragged crystals. Potassium feldspar present, but very sparse. Anhedral, interstitial sphene is conspicuous accessory mineral. Contains abundant, fine-grained, mesocratic, ellipsoidal- to discoidal-shaped mafic inclusions aligned parallel to foliation

Kbht — **Heterogeneous biotite tonalite**—Light-gray, inequigranular, medium- to coarse-grained, foliated biotite tonalite; restricted to northwestern Box Springs Mountains. Leucocratic, containing 1 to 4 percent biotite, which occurs as thin, subhedral plates, irregularly concentrated and aligned to produce wispy, swirled foliation. Leucocratic tonalite encloses pods and lenses of tonalite containing about 15 percent biotite as large ragged plates. Both types of tonalite contain abundant quartz (30 to 40 percent) and very sparse potassium feldspar (1 percent or less). Contains dispersed, mesocratic, discoidal inclusions. Granitic pegmatite dikes are abundant

Kbgt — **Heterogeneous granodiorite and tonalite**—Light- to medium-gray, medium- to coarse-grained, texturally heterogeneous, foliated, hornblende-biotite tonalite and granodiorite; restricted to northern Box Springs Mountains near Pigeon Pass. Common discoidal, mesocratic inclusions oriented parallel to foliation

Kba — **Amphibolitic gabbro**—Dark-gray to black, fine- to medium-grained, foliated, hornblende-rich amphibolitic gabbro forming lenses and elongate masses within biotite granodiorite and tonalite (Kbfg). Foliation is parallel to foliation in that unit

Val Verde pluton (Cretaceous)—Relatively uniform pluton composed of biotite-hornblende tonalite. Termed Perris quartz diorite by Dudley (1935), Val Verde tonalite by Osborn (1939), and included within Bonsall tonalite by Larsen (1948). Name Val Verde adopted by Morton (1999) based on detailed study of Osborn (1939) near Val Verde, a former settlement and railway siding midway between Perris and Riverside. Apparently steep-walled Val Verde pluton is eroded to mid-pluton level. Emplacement age of the pluton is 105.7 Ma_{id}. $^{40}Ar/^{39}Ar$ age of hornblende is 100 Ma, biotite 95 Ma and potassium feldspar 88.5 Ma. Includes:

Kvt — **Val Verde tonalite**—Gray-weathering, relatively homogeneous, massive- to well-foliated, medium- to coarse-grained, hypautomorphic-granular biotite-hornblende tonalite; principal rock type of Val Verde pluton. Contains subequal biotite and hornblende, quartz and plagioclase. Potassium feldspar generally less than two percent of rock. Where present, foliation typically strikes northwest and dips moderately to steeply northeast. Northern part of pluton contains younger, intermittently developed, northeast-striking foliation. In central part of pluton, tonalite is mostly massive, and contains few segregational masses of mesocratic to melanocratic tonalite. Elliptical- to pancake-shaped, meso-to melanocratic inclusions are common.

Krg — **Granite of the Riverside area (Cretaceous)**—Medium- to coarse-grained, massive- to faintly-foliated, leucocratic biotite granite. Contains about 1 to 3 percent biotite. Inclusions are sparse or absent except locally in western part of body, west of quadrangle, where rock contains 2 to 8 percent biotite and sparse to abundant inclusions of quartz diorite, granodiorite, and fine-grained mafic rock. At Mount Rubidoux, west of quadrangle, rocks contain sparse hypersthene and fayalitic olivine and moderately abundant equant inclusions of dark-gray fine-grained rock

Generic Cretaceous granitic rocks of the Peninsular Ranges batholith

Kp — **Granitic pegmatite dikes (Cretaceous)**—Leucocratic, mostly tabular, pegmatitic-textured granitic dikes. Most dikes range in thickness from a few centimeters to over a meter. Larger dikes are typically zoned compositionally and texturally, having a border and wall zone consisting of coarse-grained biotite, quartz, and alkali feldspars. Intermediate zone consists of large to giant crystals of quartz and alkali feldspars, and commonly contain muscovite, schorl, and garnet. Core zone consists of quartz and alkali feldspars. Line-rock layering is rare

Kg — **Granitic dikes (Cretaceous)**—Includes texturally diverse group of leucocratic granitic dikes composed mainly of quartz and alkali feldspars. Dikes range in thickness from few centimeters to over a meter and are up to several hundred meters in length. Most are tabular; some are texturally and compositionally unzoned, irregular-shaped bodies. Some dike rock has a foliated or gneissoid fabric. Textures are mostly coarse grained and equigranular granitic, but range from aplitic to pegmatitic. Accessory minerals include biotite, muscovite, and garnet

Kgd — **Granodiorite, undifferentiated (Cretaceous)**—Intermediate composition granitic rocks, mainly biotite-hornblende and biotite granodiorite; most is massive and medium grained. Restricted to single area just east of Sycamore Canyon

Kt — **Tonalite, undifferentiated (Cretaceous)**—Mainly biotite-hornblende tonalite not associated with specific plutons. Gray, medium-grained, typically foliated. Forms relatively large mass on south side of Box Springs Mountains

Kgb — **Gabbro (Cretaceous)**—Mainly hornblende gabbro. Typically brown-weathering, medium-to very coarse-grained hornblende gabbro. Very large poikilitic hornblende crystals in some rocks; locally pegmatitic. Commonly heterogeneous in composition and texture. Includes noritic and dioritic composition rocks. Exposed in southern part of quadrangle and as small masses in biotite granodiorite and tonalite (Kbfg)

Khg — **Heterogeneous granitic rocks (Cretaceous)**—Includes heterogeneous, compositionally diverse granitic rocks mostly of tonalite and granodiorite composition, but includes some monzogranite and gabbro. Mapped in and east of Sycamore Canyon, and near west edge of quadrangle

End rocks of Peninsular Ranges batholith

KgPz — **Intermixed Paleozoic(?) schist and gneiss and Cretaceous granitic rocks (Cretaceous and Paleozoic?)**—Intermixed Paleozoic(?) schist and gneiss and Cretaceous granitic rocks, mostly tonalite and granodiorite. Forms elongate mass within Val Verde tonalite (Kvt) west of Sycamore Canyon and small mass south of Tequesquite Arroyo

Pzs — **Biotite Schist (Paleozoic?)**—Medium-to dark-gray, fine-grained biotite schist and biotite-quartz-feldspar schist. Locally contains sillimanite and cordierite. Commonly includes minor amounts of quartzite and calc-silicate hornfels. Limited exposures in hills south of Tequesquite Arroyo, and as pendants in Val Verde tonalite

Pzq — **Impure quartzite (Paleozoic?)**—Quartzite; impure, light-gray to light-greenish-gray, fine-to medium-grained, layered to massive. Limited exposures in hills south of Tequesquite Arroyo

Pzm — **Marble (Paleozoic?)**—Marble; white to light-gray, locally bluish-gray and blue, coarse to extremely coarse grained

Pzc — **Calc-silicate rocks (Paleozoic?)**—Heterogeneous, massive to well-layered calc-silicate rocks

Pzms — **Marble and schist, undifferentiated (Paleozoic?)**—Intermixed marble, calc-silicate rock, and biotite schist. Mapped on North Hill in northwestern part of quadrangle

Morton, Douglas M., and Cox, Brett F. (2001), Geologic Map of the Riverside East 7.5' quadrangle, Riverside County, California, Version 1.0: U.S. Geological Survey, scale 1:24,000.

Regional Geologic Map

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Fuel Center, Convenience Store, & Car Wash
NEC Chicago Avenue & Van Buren Boulevard
Riverside, California

SCALE:	DATE:
NOT TO SCALE	10/2019
DRAWN BY:	APPROVED BY:
JC	CJ
PROJECT NO.	FIGURE NO.
3-219-0749	3C

SALEM engineering group, inc.

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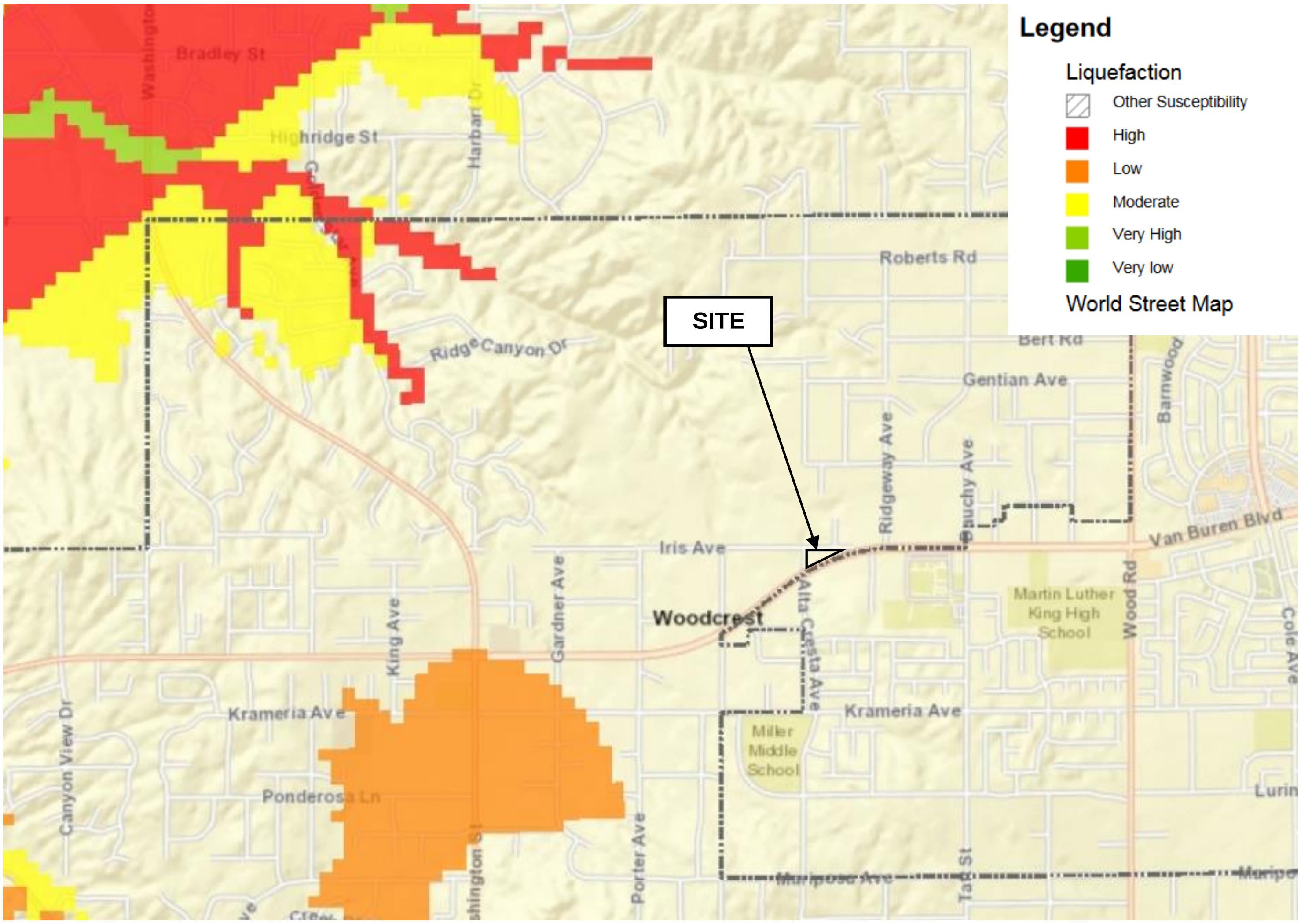
Fault Map **GEOTECHNICAL ENGINEERING INVESTIGATION** **Proposed Fuel Center, Convenience Store, & Car Wash** **NEC Chicago Avenue & Van Buren Boulevard** **Riverside, California**	SCALE: NOT TO SCALE	DATE: 10/2019	SALEM engineering group, inc.
	DRAWN BY: JC	APPROVED BY: CJ	
	PROJECT NO. 3-219-0749	FIGURE NO. 4	



0 3,009 6,019 Feet

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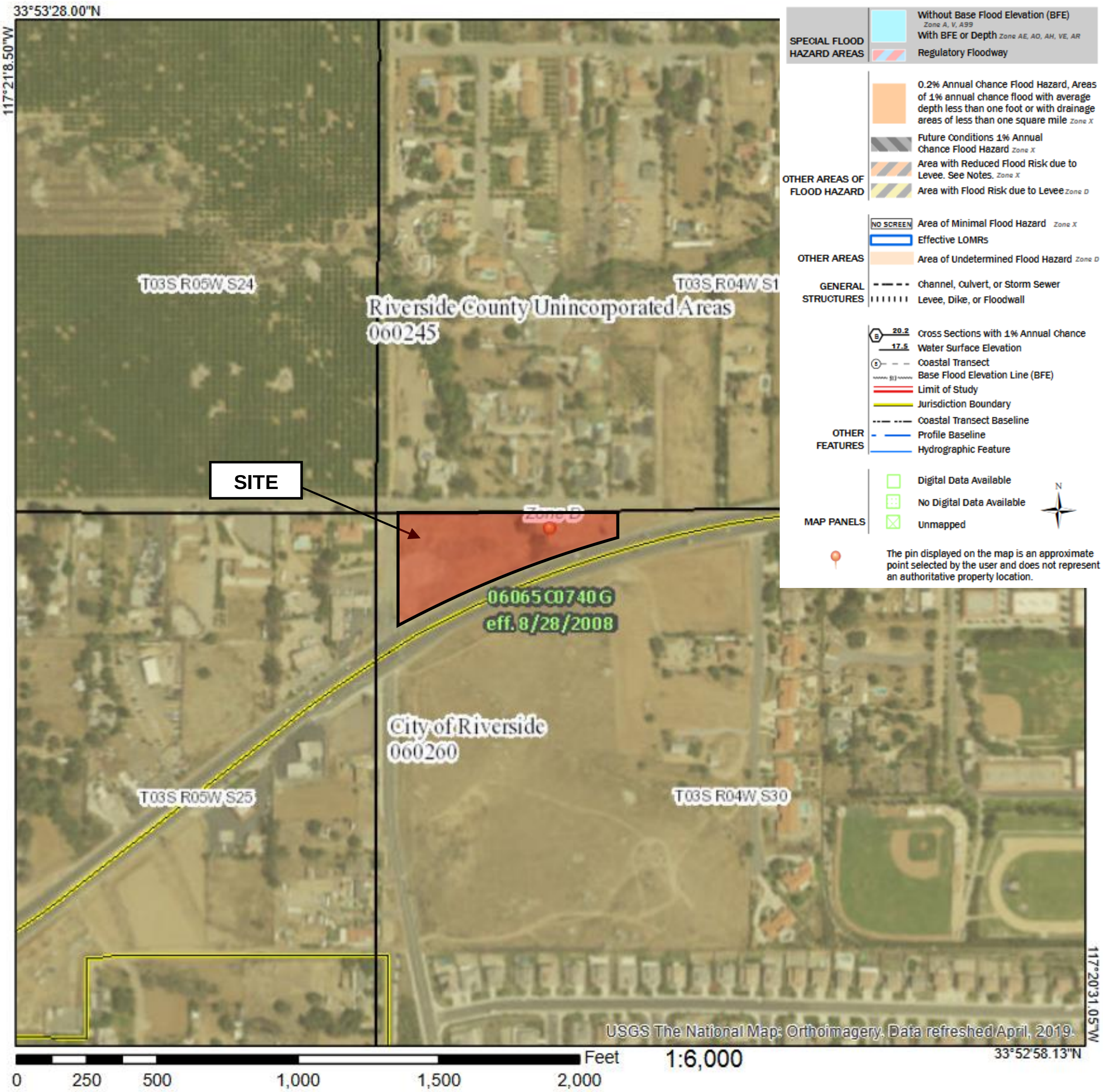
Liquefaction Potential Zone Map

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Fuel Station, Convenience Store, & Car Wash
NEC Chicago Avenue & Van Buren Boulevard
Riverside, California

SCALE:	DATE:
As Shown	10/2019
DRAWN BY: JC	APPROVED BY: CJ
PROJECT NO. 3-219-0749	FIGURE NO. 5

SALEM engineering group, inc.



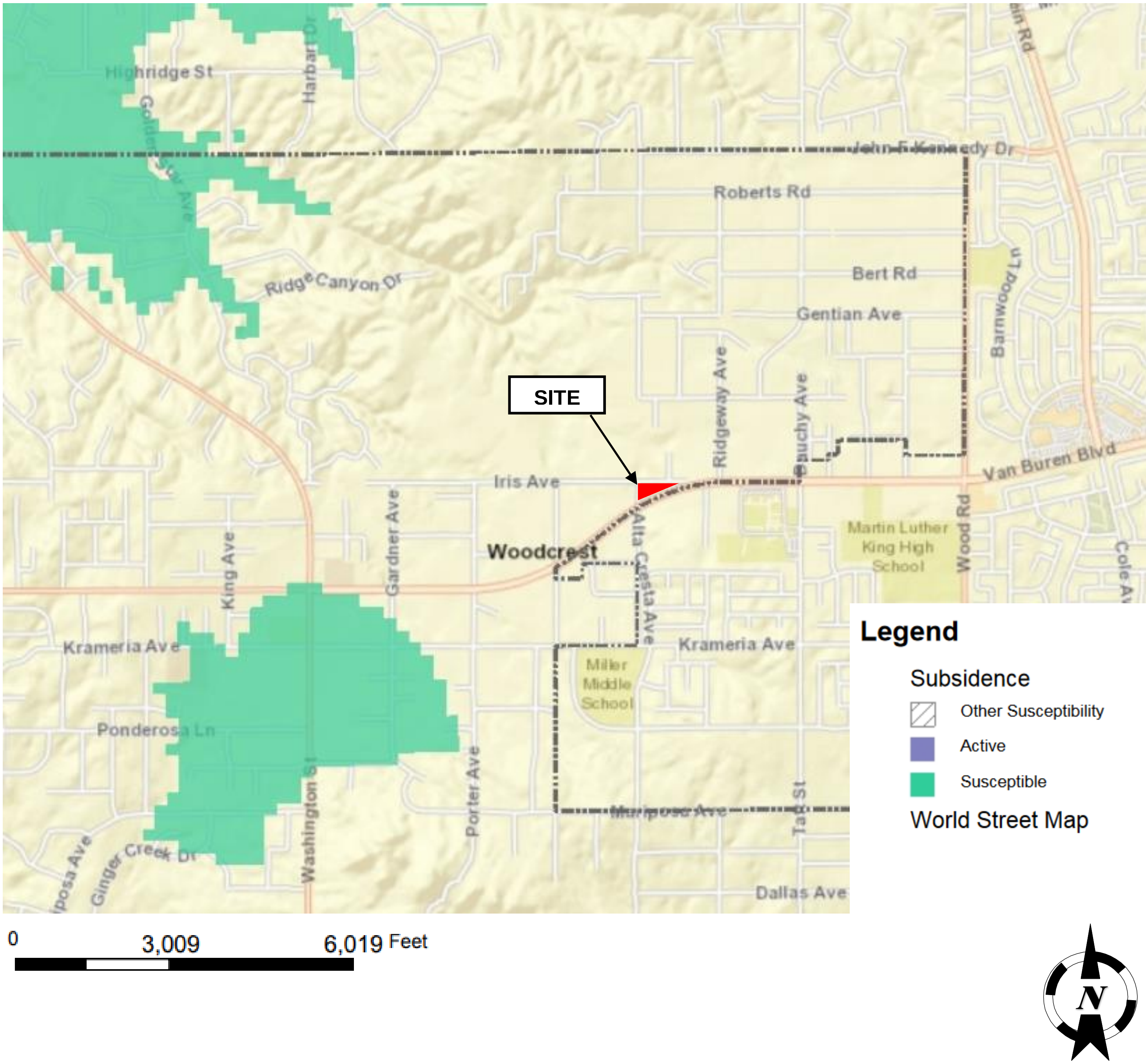
This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards.

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 10/1/2019 at 1:15:19 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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

N

Flood Zone Map	SCALE: As Shown	DATE: 10/2019	SALEM engineering group, inc.
GEOTECHNICAL ENGINEERING INVESTIGATION **Proposed Fuel Center, Convenience Store, & Car Wash** **NEC Chicago Avenue & Van Buren Boulevard** **Riverside, California**	DRAWN BY: JC	APPROVED BY: CJ	
	PROJECT NO. 3-219-0749	FIGURE NO. 6	



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Subsidence Zone Map

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Fuel Center, Convenience Store, & Car Wash
NEC Chicago Avenue & Van Buren Boulevard
Riverside, California

SCALE:	DATE:
As Shown	10/2019
DRAWN BY:	APPROVED BY:
JC	CJ
PROJECT NO.	FIGURE NO.
3-219-0749	7

SALEM engineering group, inc.

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation (drilling) was conducted on September 11, 2019 and included a site visit, subsurface exploration, and soil sampling. The locations of the exploratory borings are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

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

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.

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

SALEM engineering group, inc.

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	3/6 1/6 2/6	ML	Sandy SILT Soft; slightly moist; brown; fine to medium grain sand.	3	4.6	-	
5	35/6 60/6 60/2	SM	Silty SAND Very dense; moist; light brown; fine to coarse grain sand; trace gravel.	120/8	10.2	104.1	
10	25/6 60/4 -		Grades as above; slightly moist.	60/4"	4.8	123.7	
15	50/4 - -		Grades as above; gray; no gravel.	50/4"	4.2	-	
20			Auger refusal at 18 feet due to encountering very dense soil or bedrock.				
25							

Notes:

Figure Number A-1

SALEM engineering group, inc.

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

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

Hammer Type: Automatic Trip - 140 lb/30 in **Final Depth to Groundwater:** 19 feet

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	4/6 4/6 3/6	ML	Sandy SILT Firm; slightly moist; brown; fine grain sand; with clay.	7	4.0	-	
	60/5 - -	SM	Silty SAND Very dense; moist; brown; fine to coarse grain sand; trace gravel.	60/5"	7.3	96.6	
5	50/5 - -		Grades as above; slightly moist.	50/5"	3.6	-	
10	50/5 - -		Grades as above.	50/5"	3.3	-	
15	50/6 - -	SW	Well-graded SAND Very dense; slightly moist; black and white; fine to coarse grain sand.	50/6"	4.5	-	
20	29/6 42/6 50/2		Grades as above; moist; brown.	92/8"	6.5	-	
25			Auger refusal at 24 feet due to encountering very dense soil or bedrock.				

Notes:

Figure Number A-2

SALEM engineering group, inc.

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

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	7/6 4/6 6/6	ML	Sandy SILT Stiff; slightly moist; brown; fine grain sand.	10	4.3	-	
5	42/6 60/3 -	SW-SM	Well-graded SAND with Silt Very dense; slightly moist; brown; fine to coarse grain sand.	60/3"	2.3	115.8	Cu=15.0 Cc=1.1
10	50/5 - -	SM	Silty SAND Very dense; damp; gray; fine to coarse grain sand; with gravel.	50/5"	1.7	-	
15	50/5 - -	SP	Poorly graded SAND Very dense; slightly moist; gray; fine to coarse grain sand.	50/5"	3.7	-	
20	50/5 - -			50/5"			No recovery.
25			Auger refusal at 21.5 feet due to encountering very dense soil or bedrock.				

Notes:

Figure Number A-3

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

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

SALEM engineering group, inc.

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	14/6 13/6 11/6	SC	Clayey SAND Medium dense; slightly moist; brown; fine grain sand; with gravel.	24	2.2	-	
5	12/6 25/6 60/4	ML	Sandy SILT Hard; slightly moist; brown; fine grain sand; with gravel.	85/10	5.1	124.3	
10			Auger refusal at 8 feet due to encountering rocks or bedrock.				
15							
20							
25							

Notes:

Figure Number A-4

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

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	5/6 4/6 5/6	ML	Sandy SILT Stiff; slightly moist; brown; fine grain sand.	9	3.9	-	
	60/4 - -			60/4"	-	-	No Recovery.
5	50/2 - -	SP	Poorly graded SAND Very dense; slightly moist; brown; fine to coarse grain sand; with gravel.	50/2"	4.8	-	
10	50/2 - -	SM	Silty SAND Very dense; slightly moist; gray; fine to medium grain sand.	50/2"	2.8	-	
			Auger refusal at 11.5 feet due to encountering rocks or bedrock.				
15							
20							
25							

Notes:

Figure Number A-5

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

SALEM engineering group, inc.

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	5/6 7/6 8/6	SC	Clayey SAND Medium dense; sliightly moist; light brown; fine grain sand.	15	3.7	-	
5	7/6 57/6 60/2	SM	Silty SAND Very dense; slightly moist; gray; fine to medium grain sand.	117/8	3.8	132.0	
	50/4 - -		Grades as above; damp; fine to coarse grain sand.	50/4"	1.5	-	
10			End of boring at 10 feet BGS.				
15							
20							
25							

Notes:

Figure Number A-6

SALEM engineering group, inc.

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

Project Number: 3-219-0749

Date: 09/11/2019

Client: Riverside Holdings, LLC

Project: Proposed Fuel Station, Convenience Store, & Car Wash

Location: NEC Chicago Avenue & Van Buren Boulevard, Riverside, California

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

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

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

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	8/6 5/6 3/6	ML	Sandy SILT Firm; dry; light brown; fine grain sand.	8	0.9	-	
	3/6 3/6 2/6	SM	Silty SAND Loose; moist; brown; fine to medium grain sand.	5	7.4	-	
5			End of boring at 5 feet BGS.				
10							
15							
20							
25							

Notes:

Figure Number A-7

KEY TO SYMBOLS

Symbol Description

Strata symbols

Silt

Silty sand

Well graded sand

Well graded sand with silt

Poorly graded sand

Clayey sand

Misc. Symbols

Drill rejection

Water table during drilling

Soil Samplers

Standard penetration test

California sampler

Notes:

Consistency Classification

Blows Per Foot (Uncorrected)

Granular Soils

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

Cohesive Soils

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

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

Percolation Test Worksheet

Project: **Proposed Fuel Station, C-Store, & Car Wash**
NEC Chicago Ave. & Van Buren Blvd.
Riverside, California

Job No.: **3-219-0749**
Date Drilled: **9/11/2019**
Soil Classification: **Silty SAND (SM)**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Total Depth of Hole: 120 in.

Test Hole No.: **P-1**
Tested by: **EGR**
Drilled Hole Depth: **10** ft.

Presoaking Date: **9/11/2019**
Test Date: **9/12/2019**

Pipe Stick up: 3.5 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)
14:00	14:25	13.5	Y	0:25	11.50	12.06	6.72	25	3.7	24.0	17.3	20.6	1.42
14:26	14:51	13.5	Y	0:25	11.60	12.10	6.00	25	4.2	22.8	16.8	19.8	1.32
14:55	15:05	13.5	Y	0:10	11.40	11.63	2.76	10	3.6	25.2	22.4	23.8	1.28
15:05	15:15	13.5	N	0:10	11.63	11.83	2.40	10	4.2	22.4	20.0	21.2	1.24
15:15	15:25	13.5	N	0:10	11.83	12.00	2.04	10	4.9	20.0	18.0	19.0	1.16
15:25	15:35	13.5	N	0:10	12.00	12.16	1.92	10	5.2	18.0	16.1	17.0	1.21
15:35	15:45	13.5	N	0:10	12.16	12.30	1.68	10	6.0	16.1	14.4	15.2	1.17
15:45	15:55	13.5	N	0:10	12.30	12.43	1.56	10	6.4	14.4	12.8	13.6	1.20
Recommended for Design:										**Infiltration Rate**			**1.16**

SALEM engineering group, inc.

Percolation Test Worksheet

Project: **Proposed Fuel Station, C-Store, & Car Wash**
NEC Chicago Ave. & Van Buren Blvd.
Riverside, California

Job No.: **3-219-0749**
Date Drilled: **9/11/2019**
Soil Classification: **Silty SAND (SM)**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Total Depth of Hole: 66 in.

Test Hole No.: **P-2**
Tested by: **EGR**
Drilled Hole Depth: **5.5** ft.

Presoaking Date: **9/11/2019**
Test Date: **9/12/2019**

Pipe Stick up: 1 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)
14:05	14:30	6.5	Y	0:25	3.20	5.60	28.80	25	0.9	39.6	10.8	25.2	5.08
14:31	14:56	6.5	Y	0:25	3.90	5.75	22.20	25	1.1	31.2	9.0	20.1	4.82
15:00	15:10	6.5	Y	0:10	4.00	4.88	10.56	10	0.9	30.0	19.4	24.7	4.74
15:10	15:20	6.5	N	0:10	4.88	5.47	7.08	10	1.4	19.4	12.4	15.9	4.75
15:20	15:30	6.5	N	0:10	5.47	5.86	4.68	10	2.1	12.4	7.7	10.0	4.67
15:31	15:41	6.5	Y	0:10	4.10	4.93	9.96	10	1.0	28.8	18.8	23.8	4.63
15:41	15:51	6.5	N	0:10	4.93	5.49	6.72	10	1.5	18.8	12.1	15.5	4.61
15:51	16:01	6.5	N	0:10	5.49	5.87	4.56	10	2.2	12.1	7.6	9.8	4.62
Recommended for Design:										**Infiltration Rate**			**4.61**

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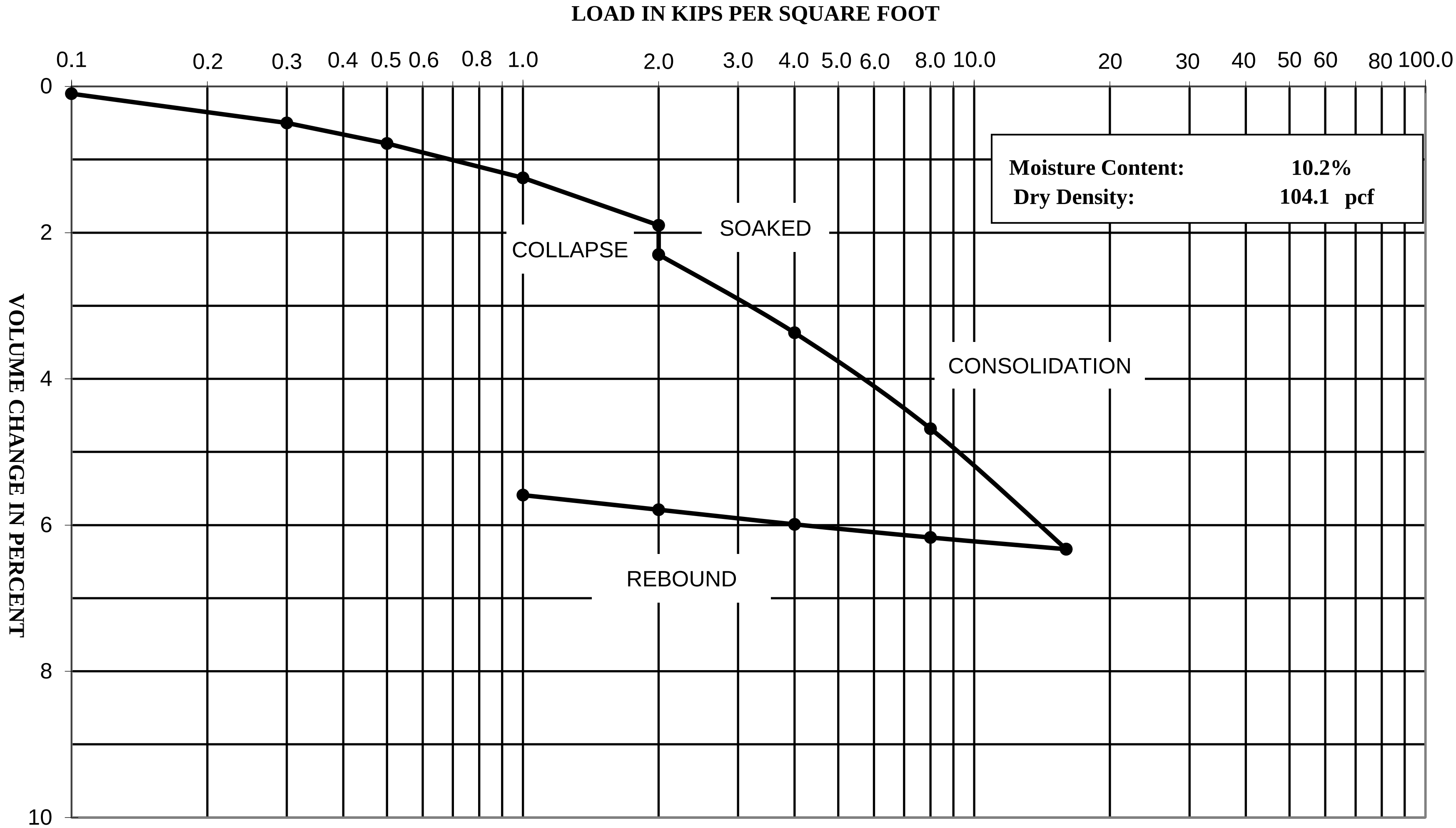
APPENDIX

B

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM), Caltrans, or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, corrosivity, consolidation, shear strength, expansion index, 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 Fuel, C-Store, Car Wash - Riverside, CA

Project Number: 3-219-0749

Boring: B-1 @ 3.5'

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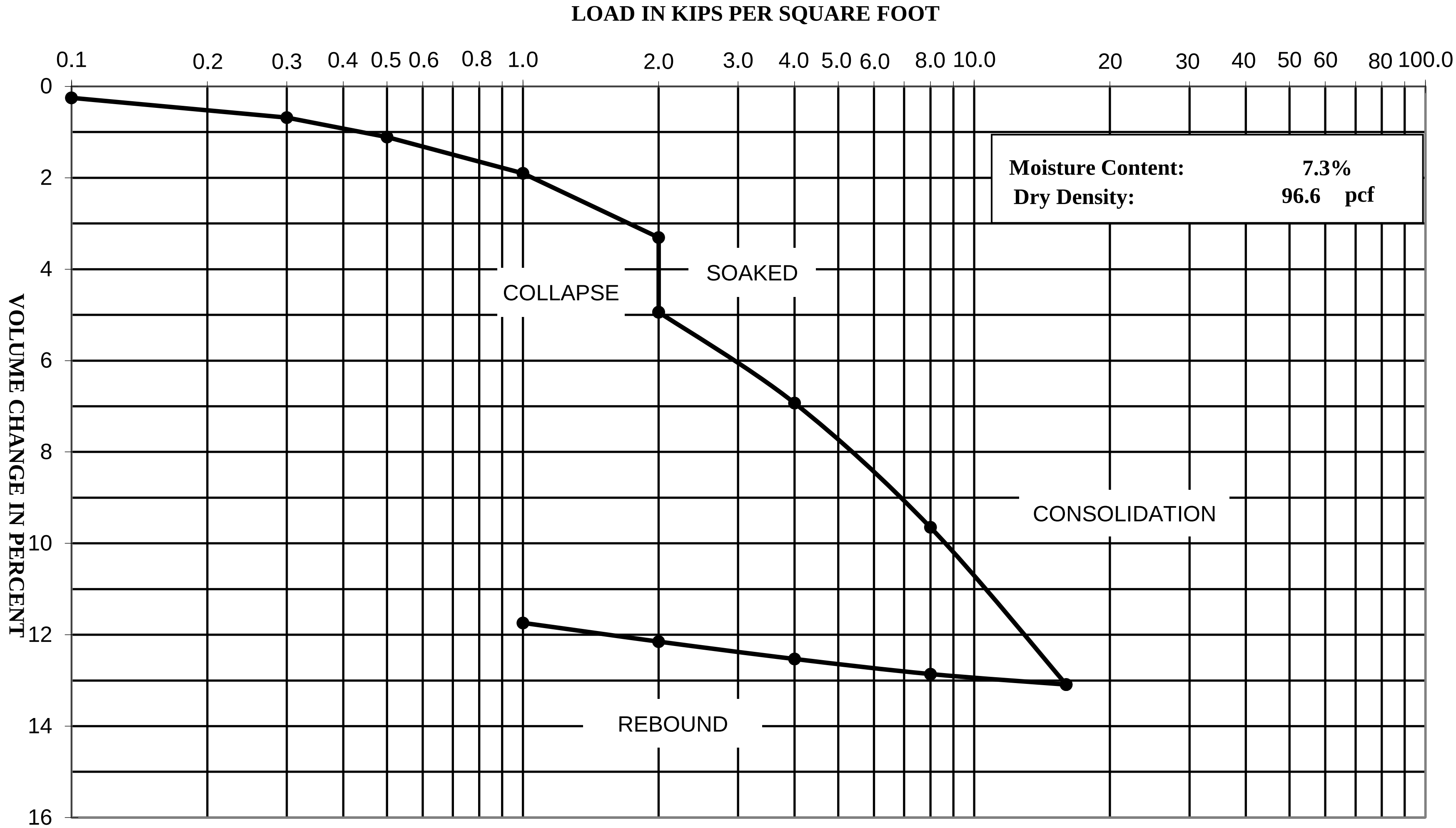
CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA

Project Number: 3-219-0749

Boring: B-2 @ 1.5'

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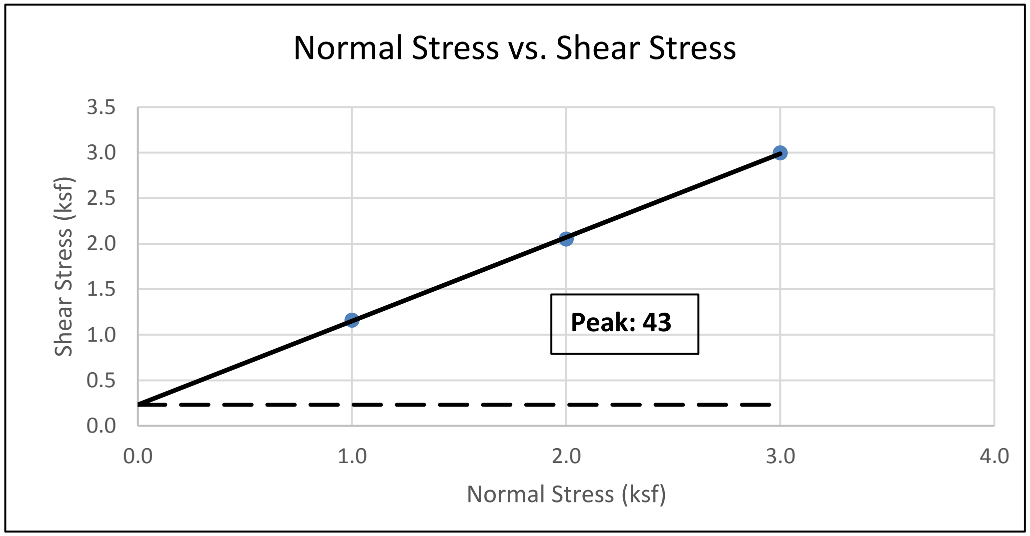
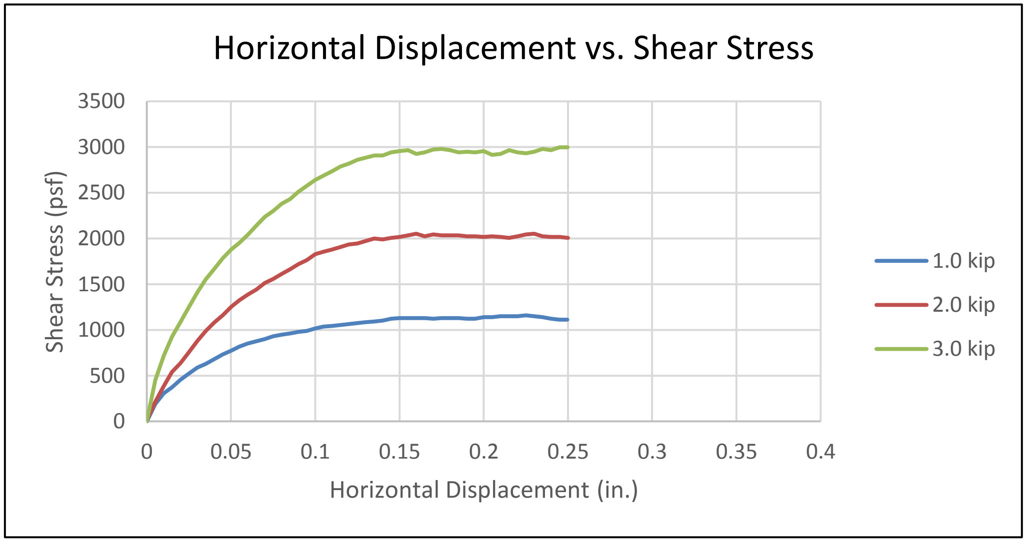
Direct Shear Test (ASTM D3080)

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA
Project Number: 3-219-0749
Client: Riverside Holdings, LLC
Boring: B-3 @ 3.5'
Soil Type: Well-graded SAND with Silt (SP-SM)
Sample Type: Undisturbed Ring
Tested By: NL
Reviewed By:
Date of Test: 9/27/19
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	1.16	2.05	3.00
Residual Shear Stress (ksf)	0.00	0.00	0.00

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.976	0.971	0.953
Post-Shear Sample Height (in.)	0.973	0.976	0.948
Diameter of Sample (in)	2.416	2.416	2.416
Initial (pre-shear) Values			
Moisture Content (%)	2.3		
Dry Density (pcf)	116.0	112.7	102.5
Saturation %	14.1	12.8	9.8
Void Ratio	0.44	0.48	0.63
Consolidated Void Ratio	0.40	0.44	0.55
Final (post-shear) Values			
Final Moisture Content (%)	16.9	17.1	17.3
Dry Density (pcf)	114.0	109.3	110.9
Saturation %	75.4	69.8	60.2
Void Ratio	0.60	0.65	0.77





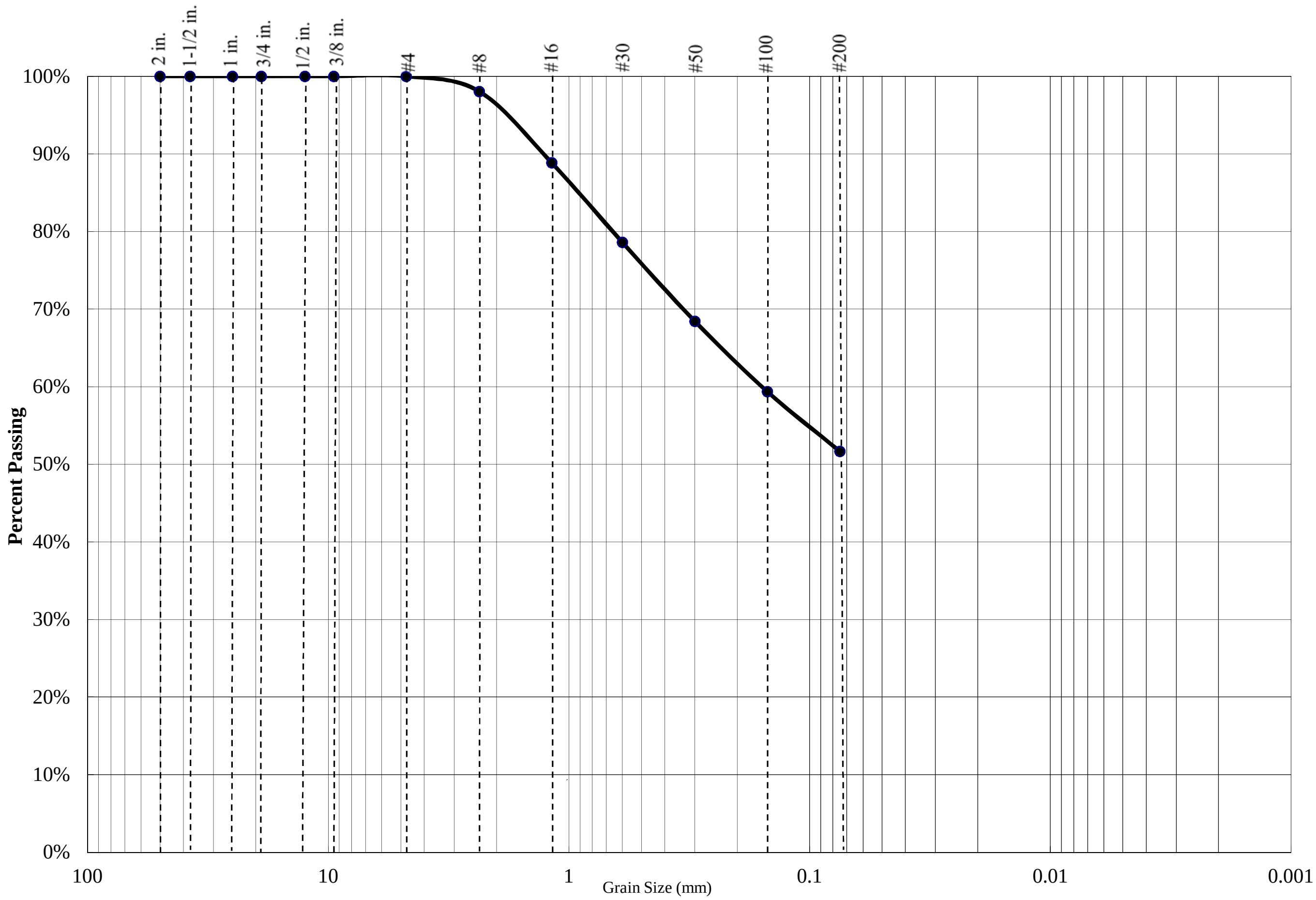
Peak Shear Strength Values	
Slope	0.92
Friction Angle	43
Cohesion (psf)	231

0
0

SALEM engineering group, inc.
8711 Monroe Court, Suite A
Rancho Cucamonga, CA 91730

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	48%	52%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	98.0%
#16	88.9%
#30	78.6%
#50	68.4%
#100	59.4%
#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 Fuel, C-Store, Car Wash - Riverside, CA

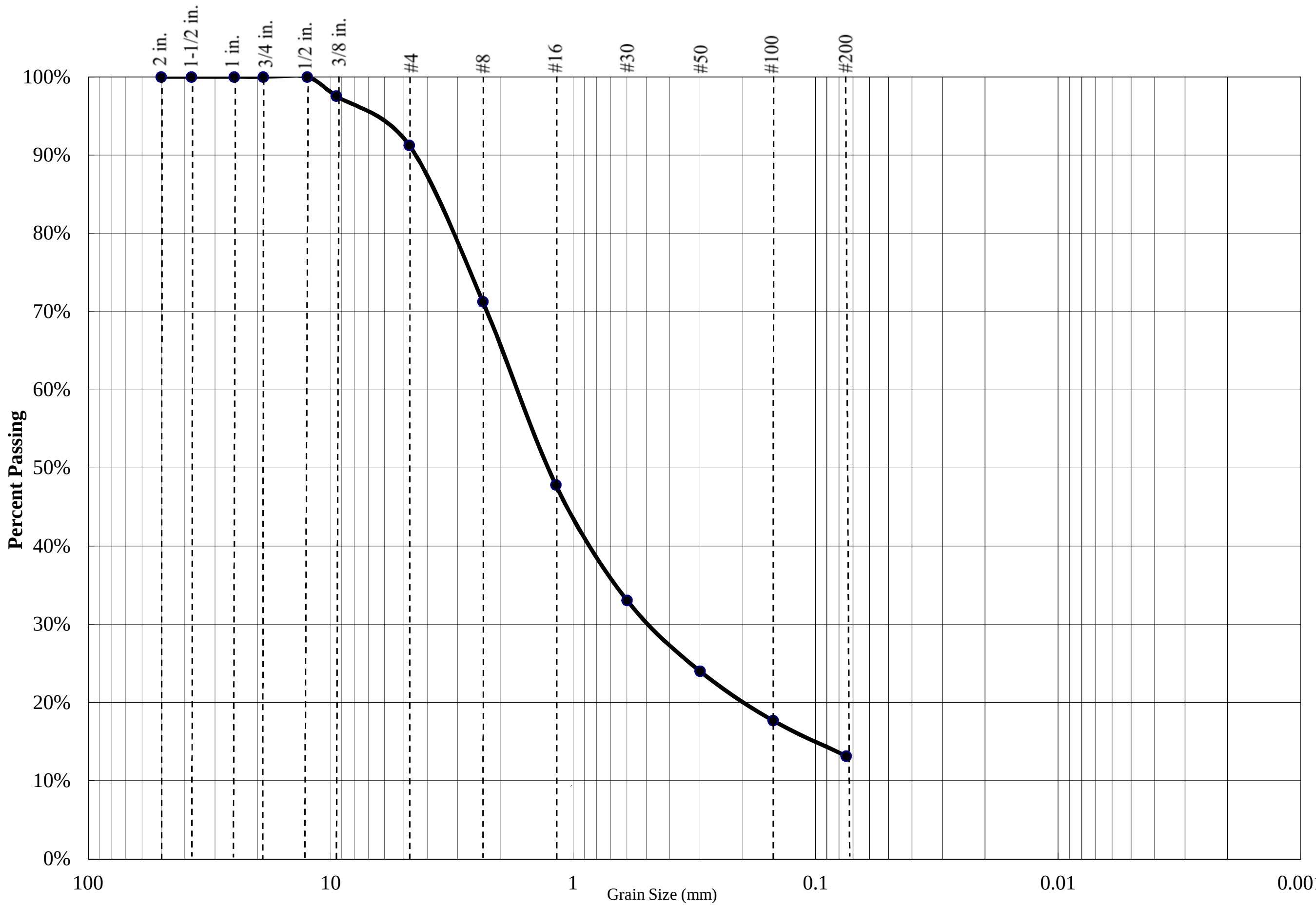
Project Number: 3-219-0749

Boring: B-1 @ 0'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
9%	78%	13%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	97.6%
#4	91.3%
#8	71.3%
#16	47.8%
#30	33.1%
#50	24.0%
#100	17.7%
#200	13.2%

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
Silty SAND (SM)

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA

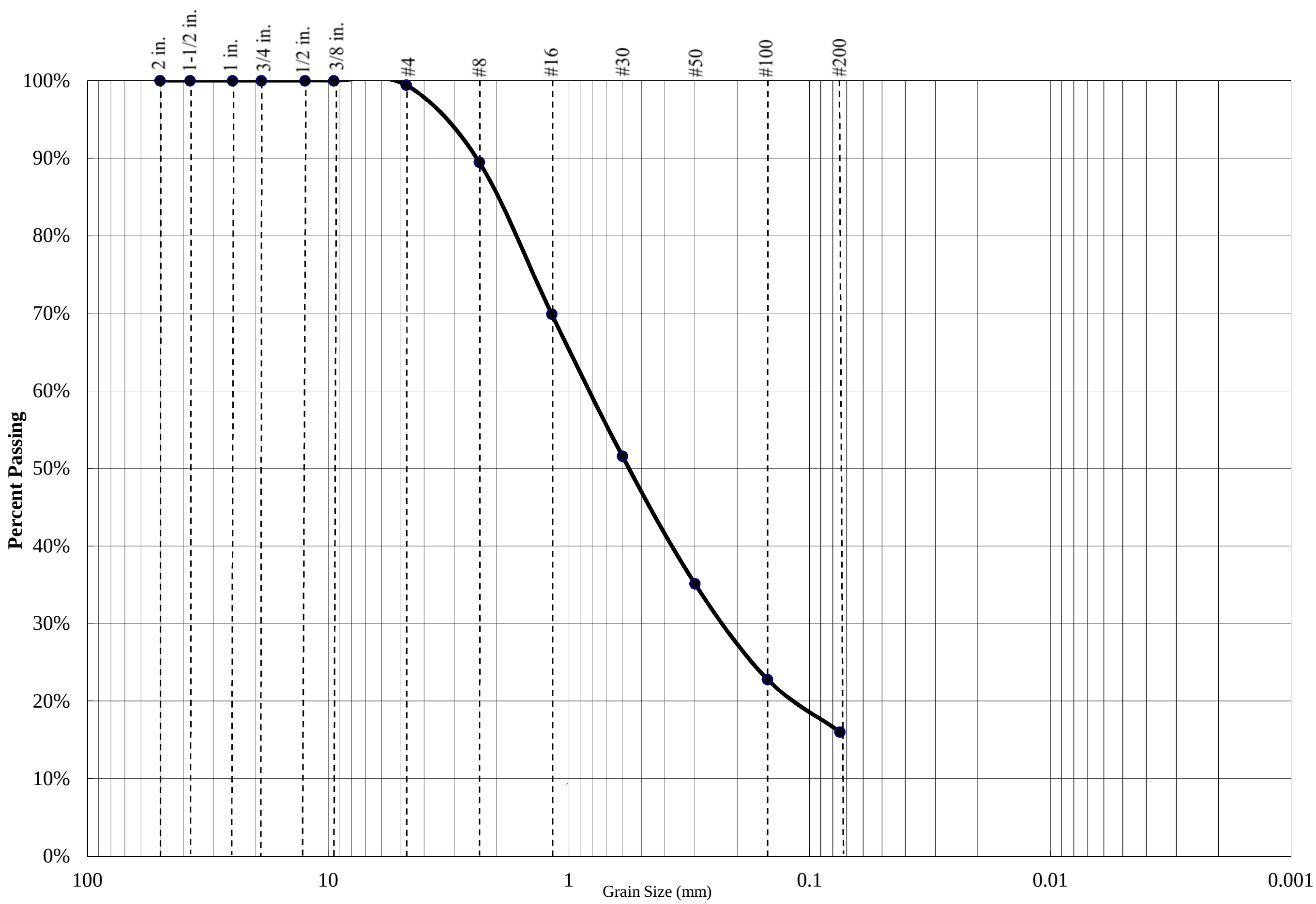
Project Number: 3-219-0749

Boring: B-1 @ 3.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	83%	16%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.4%
#8	89.5%
#16	69.9%
#30	51.6%
#50	35.2%
#100	22.8%
#200	16.0%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA

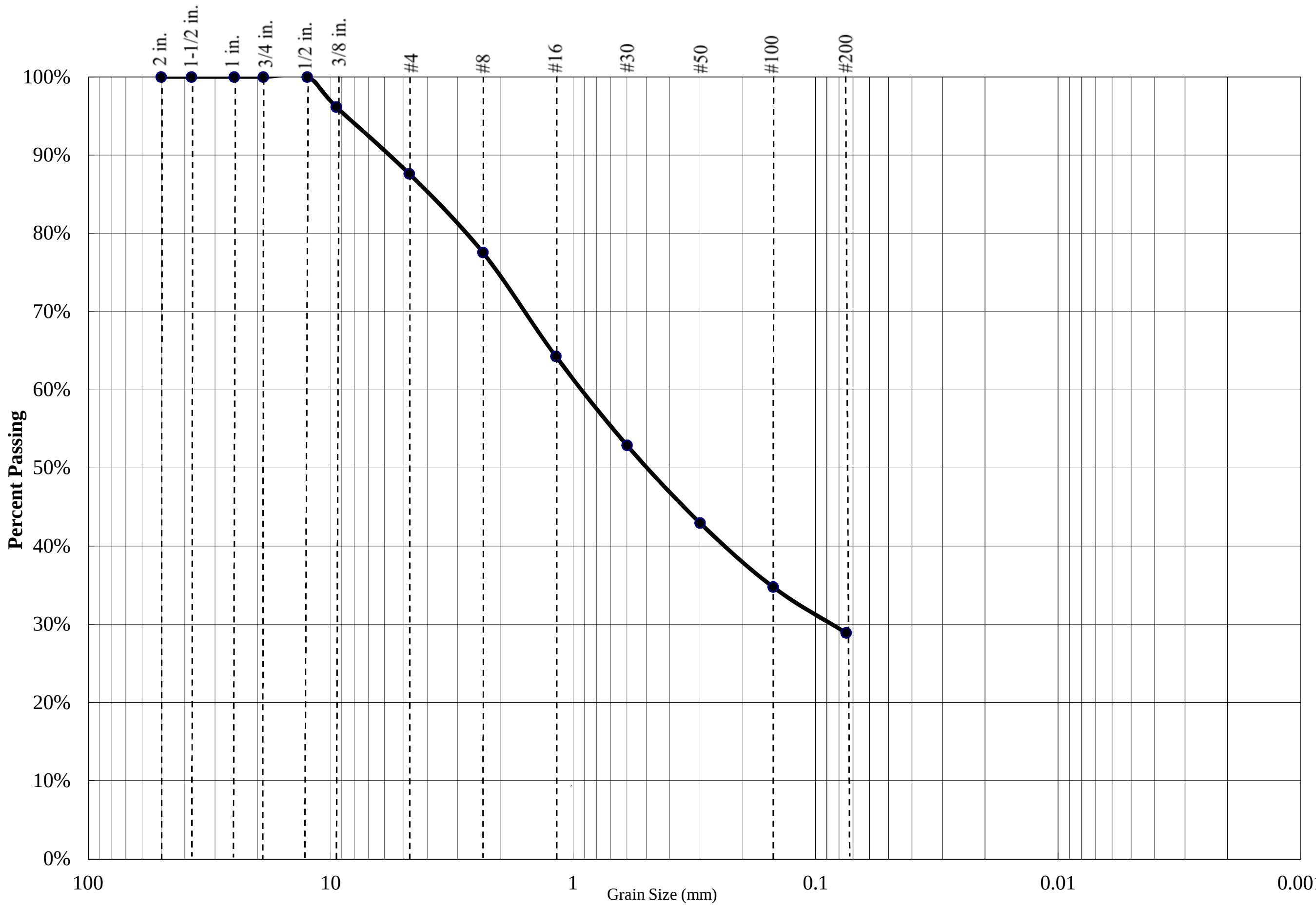
Project Number: 3-219-0749

Boring: B-1 @ 8.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
12%	59%	29%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	96.2%
#4	87.6%
#8	77.6%
#16	64.3%
#30	52.9%
#50	43.0%
#100	34.8%
#200	28.9%

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 Fuel, C-Store, Car Wash - Riverside, CA

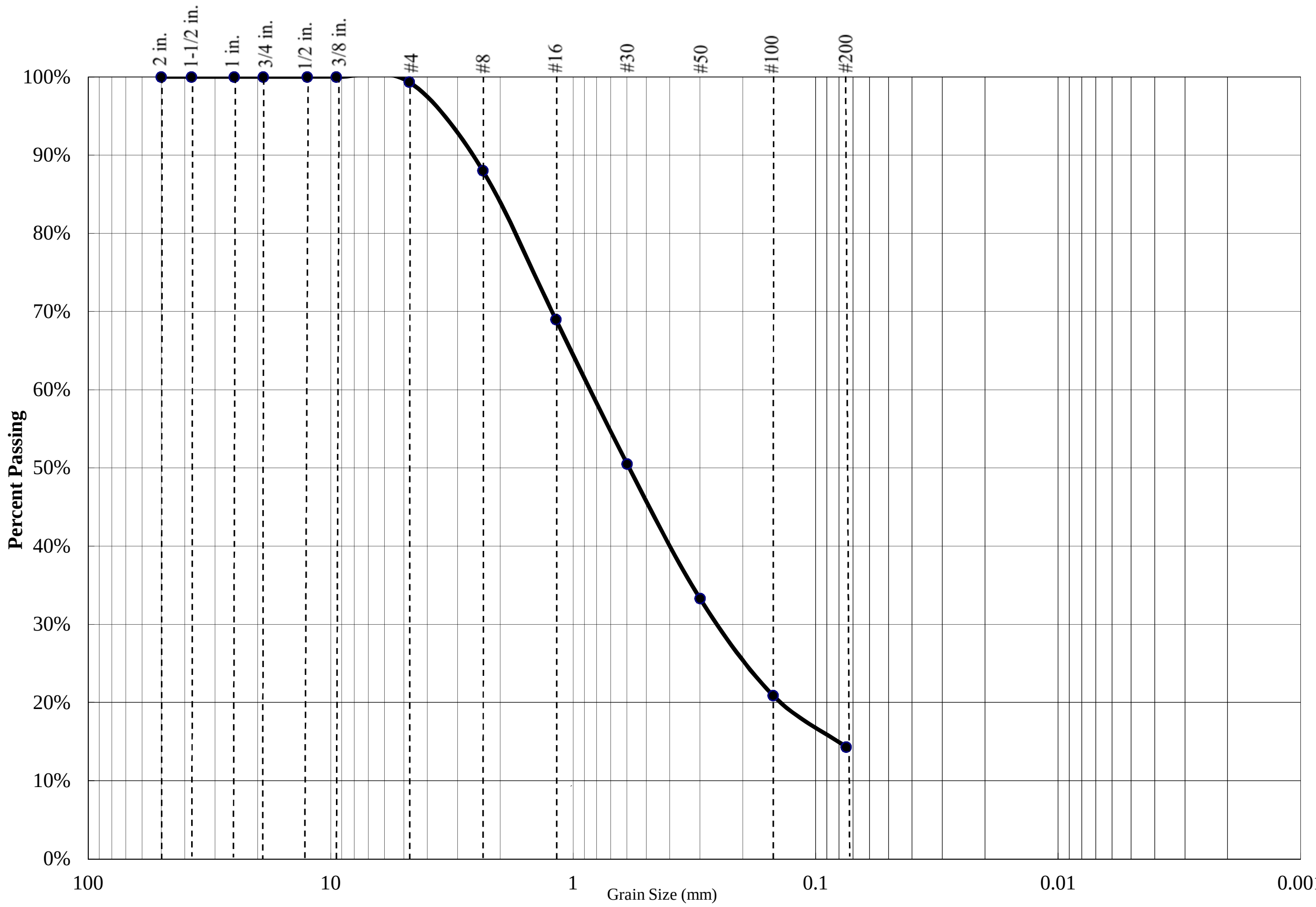
Project Number: 3-219-0749

Boring: B-2 @ 1.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	85%	14%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.3%
#8	88.0%
#16	69.0%
#30	50.5%
#50	33.3%
#100	20.9%
#200	14.3%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA

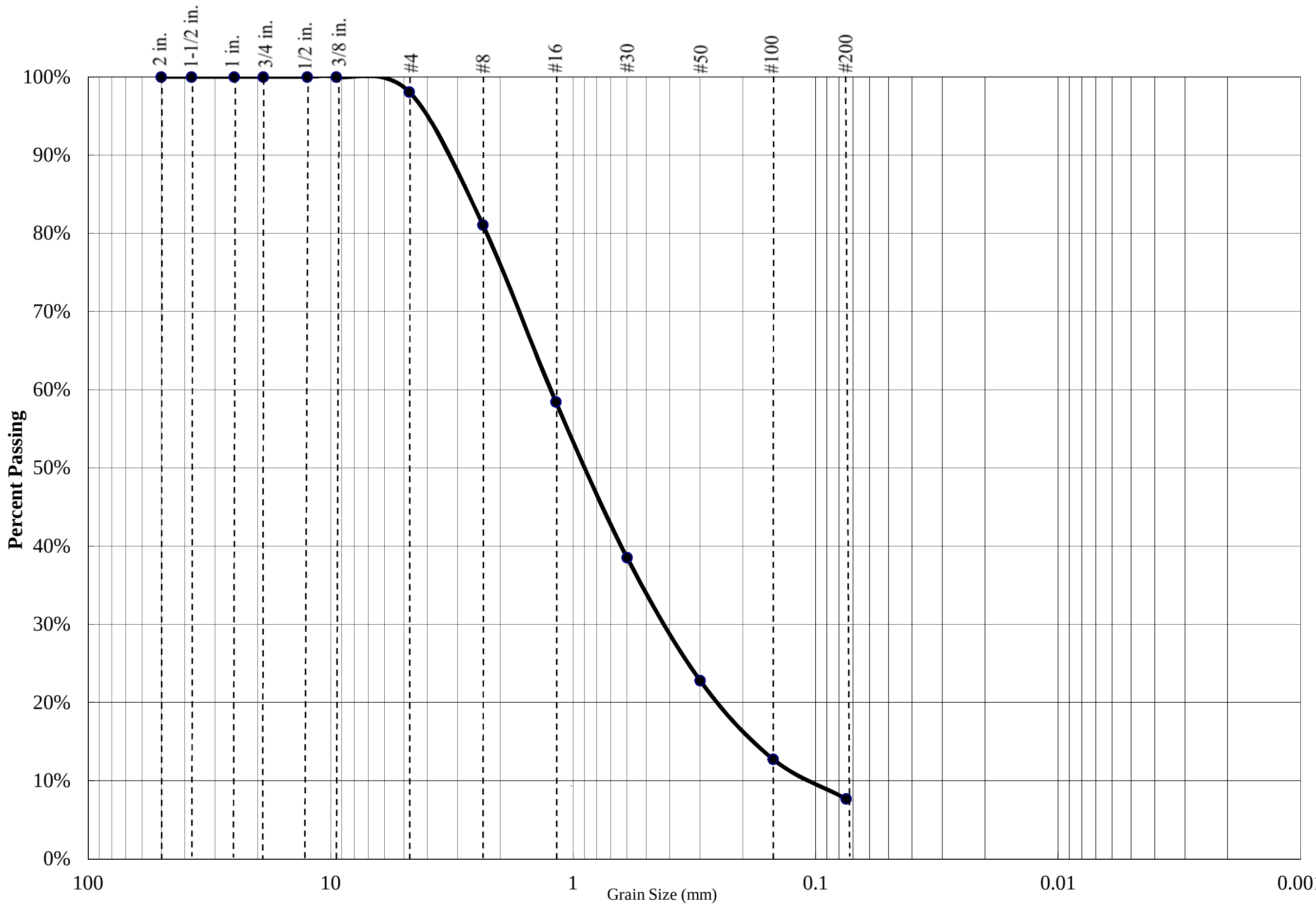
Project Number: 3-219-0749

Boring: B-2 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
2%	90%	8%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	98.1%
#8	81.1%
#16	58.5%
#30	38.5%
#50	22.8%
#100	12.8%
#200	7.7%

Atterberg Limits		
PL=	LL=	PI=

Coefficients					
D_{85}=		D_{60}=	1.5	D_{50}=	
D_{30}=	0.4	D_{15}=		D_{10}=	0.1
C_u=	15.00	C_c=	1.07		

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

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA

Project Number: 3-219-0749

Boring: B-3 @ 3.5'

SALEM engineering group, inc.

EXPANSION INDEX TEST
ASTM D4829

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA
Project Number: 3-219-0749
Date Sampled: 9/11/19 Date Tested: 9/24/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 0'-3'
Soil Description: Dark Brown Silty, Clayey SAND (SC-SM)

Trial #	1	2	3
Weight of Soil & Mold, g.	591.0		
Weight of Mold, g.	188.2		
Weight of Soil, g.	402.8		
Wet Density, pcf	121.5		
Weight of Moisture Sample (Wet), g.	845.0		
Weight of Moisture Sample (Dry), g.	771.5		
Moisture Content, %	9.5		
Dry Density, pcf	110.9		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	49.6		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.0104	0.0104	0.0112	--	0.0116

Expansion Index $_{measured}$ = 11.6
Expansion Index $_{50}$ = 11.4

Expansion Index = 11

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

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CHEMICAL ANALYSIS

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

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA
Project Number: 3-219-0749
Date Sampled: 9/11/19 Date Tested: 9/23/19
Sampled By: SEG Tested By: DZ
Soil Description: Dark Brown Silty, Clayey SAND (SC-SM)

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 0'-3'	100 mg/kg	20 mg/kg	7.2
1b.	B-1 @ 0'-3'	100 mg/kg	19 mg/kg	7.2
1c.	B-1 @ 0'-3'	120 mg/kg	19 mg/kg	7.2
Average:		**107 mg/kg**	**19 mg/kg**	**7.2**

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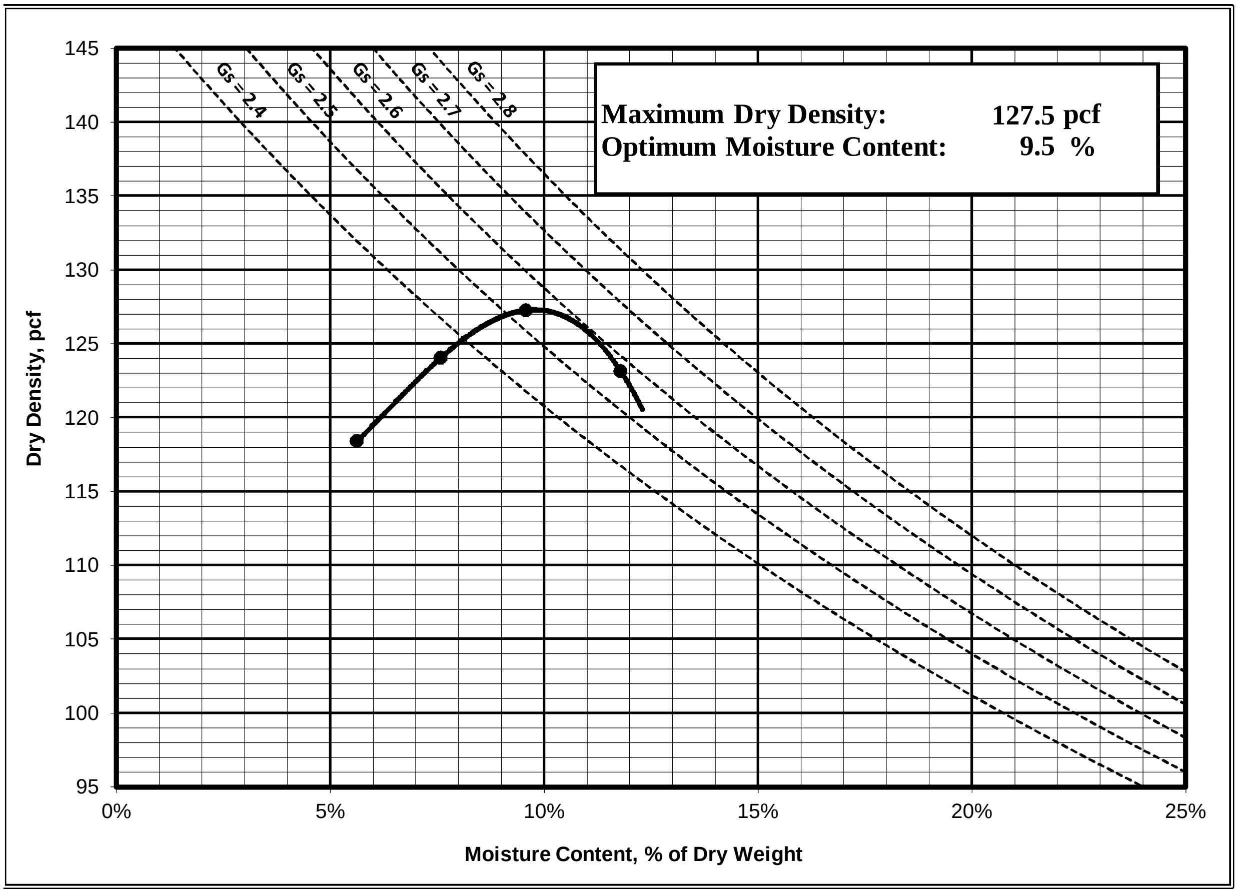
Laboratory Compaction Curve
ASTM D1557

Project Name: Proposed Fuel, C-Store, Car Wash - Riverside, CA
Project Number: 3-219-0749
Date Sampled: 9/11/19 Date Tested: 9/24/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 0'-3'
Soil Description: Dark Brown Silty, Clayey SAND (SC-SM)
Test Method: ASTM D1557 A

	1	2	3	4
Weight of Moist Specimen & Mold, (g)	3864.6	3991.6	4082.3	4055.1
Weight of Compaction Mold, (g)	1974.1	1974.1	1974.1	1974.1
Weight of Moist Specimen, (g)	1890.5	2017.5	2108.2	2081.0
Volume of Mold, (ft^3)	0.0333	0.0333	0.0333	0.0333
Wet Density, (pcf)	125.0	133.4	139.4	137.6
Weight of Wet (Moisture) Sample, (g)	325.0	325.0	325.0	325.0
Weight of Dry (Moisture) Sample, (g)	307.7	302.1	296.6	290.7
Moisture Content, (%)	5.6%	7.6%	9.6%	11.8%
Dry Density, (pcf)	118.4	124.0	127.2	123.1



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APPENDIX

C

APPENDIX C
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The term "Standard Specifications": hereinafter referred to, is the most recent edition of the Standard Specifications of the State of California, Department of Transportation. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as determined by ASTM D1557 Test Method (latest edition) or California Test Method 216 (CAL-216), as applicable.

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

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

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

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