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

SALEM
engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED COMMERCIAL DEVELOPMENT
NWC N. SANDERSON AVENUE & COTTONWOOD AVENUE
SAN JACINTO, CALIFORNIA

SALEM PROJECT NO. 3-216-0492
MAY 16, 2016

PREPARED FOR:

MR. INO CRUZ
J&T BUSINESS MANAGEMENT
139 RADIO ROAD
CORONA, CA 92879

PREPARED BY:

SALEM ENGINEERING GROUP, INC.
11650 MISSION PARK DR., #108
RANCHO CUCAMONGA, CA 91730
P: (909) 980-6455
F: (909) 980-6435
www.salemeng.com

SALEM
engineering group, inc.

11650 Mission Park Dr., #108
Rancho Cucamonga, CA 91730
Phone (909) 980-6455
Fax (909) 980-6435

May 16, 2016

Project No. 3-216-0492

Mr. Ino Cruz
J&T Business Management
139 Radio Road
Corona, CA 92879

Subject: Geotechnical Engineering Investigation
Proposed Commercial Development
NWC N. Sanderson Avenue & Cottonwood Avenue
San Jacinto, California

Dear Mr. Cruz:

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

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Clarence Jiang, GE
Senior Geotechnical Engineer
RGE 2477

REGISTERED PROFESSIONAL ENGINEER
CLARENCE T. JIANG
No. 2477
Exp. 6/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

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

REGISTERED PROFESSIONAL ENGINEER
R. SAMMY SALEM
No. 2549
Exp. Dec. 31, 2016
GEOTECHNICAL
STATE OF CALIFORNIA

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1
2. PROJECT DESCRIPTION 1
3. SITE LOCATION AND DESCRIPTION 2
4. FIELD EXPLORATION 2
5. LABORATORY TESTING 3
6. GEOLOGIC SETTING 3
7. GEOLOGIC HAZARDS 4
   7.1 Faulting and Seismicity 4
   7.2 Surface Fault Rupture 5
   7.3 Ground Shaking 5
   7.4 Liquefaction 5
   7.5 Seismic Densification 6
   7.6 Lateral Spreading 6
   7.7 Subsidence 6
   7.8 Collapsible/Expansive or Hydroconsolidatable Soils 6
   7.9 Flood and Dam Inundation 7
   7.10 Landslides/Slope Instability/Debris Flow 7
   7.11 Wind and Water Erosion 7
   7.12 Tsunamis and Seiches 7
8. SOIL AND GROUNDWATER CONDITIONS 7
   8.1 Subsurface Conditions 7
   8.2 Groundwater 8
   8.3 Soil Corrosion Screening 8
   8.4 Percolation Testing 9
9. CONCLUSIONS AND RECOMMENDATIONS 10
   9.1 General 10
   9.2 Seismic Design Criteria 12
   9.3 Soil and Excavation Characteristics 13
   9.4 Materials for Fill 14
   9.5 Grading 15
   9.6 Option 1 - Shallow Foundations with Geogrid 17
   9.8 Caisson Foundations for Canopy 20
   9.9 Concrete Slabs-on-Grade 21
   9.10 Lateral Earth Pressures and Frictional Resistance 22
   9.11 Retaining Walls 23
   9.12 Temporary Excavations 24
   9.13 Underground Utilities 25
   9.14 Surface Drainage 25
   9.15 Pavement Design 26
10. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING 27
   10.1 Plan and Specification Review 27

TABLE OF CONTENTS (cont.)

10.2 Construction Observation and Testing Services 27

11. LIMITATIONS AND CHANGED CONDITIONS 27

FIGURES

Figure 1, Vicinity Map
Figure 2, Site Plan
Figure 3A, Regional Geologic Map
Figure 3B, Regional Geologic Map Explanation
Figure 4, Regional Fault Map
Figure 5, Flood Zone Map

APPENDIX A – FIELD INVESTIGATION

Figures A-1 through A-12, Logs of Exploratory Soil Borings B-1 through B-12
Percolation Test Results, P-1 through P-4
Seismic Densification Settlement Analysis

APPENDIX B – LABORATORY TESTING

Consolidation Test Results
Direct Shear Test Results
Gradation Curves
Expansion Index Test Results
Corrosivity Test Results
Maximum Density and Optimum Moisture Proctor Test Results

APPENDIX C – EARTHWORK AND PAVEMENT SPECIFICATIONS

SALEM engineering group, inc.
11650 Mission Park Drive, Suite 108
Rancho Cucamonga, CA 91730
Phone (909) 980-6455
Fax (909) 980-6435

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
NWC N. SANDERSON AVENUE & COTTONWOOD AVENUE
SAN JACINTO, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation for the Proposed Commercial Development to be located at the northwest corner of N. Sanderson Avenue and Cottonwood Avenue in the City of San Jacinto, California (see Figure 1, Vicinity Map).

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

The scope of this investigation included a field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on May 5, 2016 and included the drilling of twelve (12) small-diameter soil borings to a maximum depth of 50 feet at the site. Additionally, four (4) percolation tests were performed on May 6, 2016 at depths between 5 to 8 feet below existing grade for determination of the percolation rate. The locations of the soil borings and percolation tests are depicted on Figure 2, Site Plan. A detailed discussion of our field investigation, exploratory boring logs are presented in Appendix A.

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

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions.

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

2. PROJECT DESCRIPTION

We understand that the development of the site will include construction of a total of nine (9) buildings and one (1) 8-pump gasoline canopy on a 6.28-acre vacant land. The buildings will include a 3,800 square-foot convenience store, a 2,800 square-foot fast-food drive-thru restaurant #1, a 3,200 square-foot fast-food drive-thru restaurant #2, a 2,800 square-foot fast food restaurant #3, a 1,500 square-foot fast food restaurant #4, a 2,800 square-foot fast food restaurant #5, a 2,400 square-foot retail #1, a 12,500 square-foot retail #2, and 2,080 square-foot carwash. The canopy will be 4,395 square-foot 8-pump gasoline canopy. On-site parking and landscaping are planned to be associated with the development.

Maximum wall load is expected to be on the order of 2.5 kips per linear foot. Maximum column load is expected to be on the order of 50 kips. Floor slab soil bearing pressure is expected to be on the order of 150 psf. A site grading plan was not provided at the time of preparation of this proposal. As the existing project area is essentially level, we anticipate that cuts and fills during the earthwork will be minimal and limited to providing level building pad and positive site drainage.

Concrete and asphaltic concrete pavement for parking area, customers travel lanes, and truck lane are to be designed for standard duty and heavy-duty traffic loading based on an Equivalent Single Axle Load (ESAL) of 18 kips, a maximum load of 60,000 ESAL and a design life of 20 years. The pavement design recommendations provided herein are based on the State of California Department (CALTRANS) design manual.

A site grading plan was not available at the time of preparation of this report. 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 site is semi-triangular in shape and encompasses approximately 6.28 acres. The site is located on the northwest corner of the intersection of N. Sanderson Avenue and Cottonwood Avenue in the City of San Jacinto, Riverside County, California (see Vicinity Map, Figure 1).

The site is currently utilized for agricultural purposes for the cultivation of raw crops. Two palm trees were observed on the southeast portion of the site. The site is bounded by a pump-station facility to the north, N. Sanderson Avenue to the east, Cottonwood Avenue to the south, and agricultural land to the west. The site is relatively flat with no major changes in grade.

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-12) were drilled on May 5, 2016 in the area shown on the Site Plan, Figure 2. The test borings were advanced with a 4-inch diameter solid flight auger rotated by a truck-mounted CME-45C drill rig. The test borings were extended to a maximum depth of 50 feet below existing grade.

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

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

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

5. LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory-testing program was formulated with emphasis on the evaluation of natural moisture, density, shear strength, consolidation potential, expansion 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 width. 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 (Morton and Miller, 2006). The Santa Ana Mountains block (west of the subject site) extends from the coast to the Elsinore Fault zone, which is located approximately 18 miles southwest of the subject site. The northeastern margin of the Perris structural block, which underlies the subject site, is approximately 5 miles northeast of the site.

The Perris block is underlain by lithologically diverse prebatholithic metasedimentary rocks intruded by plutons of the Cretaceous Peninsular Ranges batholith. Supra-batholithic volcanic rocks are preserved in the western part of the block. 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 (see Regional Geologic Map, Figure 3).

The most notable geomorphic/geologic feature in the site vicinity is a fault graben associated with movement along the Casa Loma Fault and Claremont Fault, the two major strands of the larger San Jacinto Fault Zone in the area. This graben is represented by the San Jacinto Basin, a broad sediment-filled valley containing scattered bedrock hills.

The near-surface deposits in the vicinity of the subject site are comprised of recent alluvium consisting of unconsolidated sands, silt, and clays derived from erosion of local mountain ranges. The geologic map in Figure 3 shows the site to be underlain by Quaternary alluvium (map unit Qa) consisting of alluvial sand and clay. Deposits encountered on the subject site during exploratory drilling are discussed in detail in this report.

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 0.7 miles from the site (see Figure 4, Regional Fault Map). 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 D in accordance with Chapter 16 of the California Building Code.

The proposed structures are determined to be in Seismic Design Category E. To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 33.7880° North; site longitude is 117.0079° 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; A+CC+B+SM	0.7	7.6
San Jacinto; SJV+A+CC+B+SM	1.7	7.9
San Jacinto; SBV+SJV	3.3	7.4
S. San Andreas; PK+CH+CC+BB+NM +SM+NSB+SSB+BG+CO	16.0	8.2
S. San Andreas; BG+CO	16.4	7.4
Elsinore; W+GI	19.0	7.3
Elsinore; W+GI+T+J+CM	19.4	7.9
San Jacinto; SBV	20.6	7.1
Pinto Mtn	24.7	7.3
S. San Andreas; PK+CH+CC+BB+NM +SM+NSB	27.8	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

We used the USGS web-based application *US Seismic Design Maps* to estimate the peak ground acceleration adjusted for site class effects (PGA_M). Because of the proximity to the subject site and the maximum probable events for these faults, it appears that a maximum probable event along the fault zones could produce a peak horizontal acceleration of approximately 0.926 g (2% probability of being exceeded in 50 years). While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

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 50 feet on the project site consisted predominately of silty sand/sand, silty sand with varying amounts of clay, silty sand/sandy silt with varying amounts of clay, sandy silt with varying amounts of clay, and clayey silt. Based on the County of Riverside Geologic Hazards Map (2004). The historically highest groundwater is estimated to be at a depth of approximately 100 feet below existing grade. 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 moderate liquefaction potential area. 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 site was evaluated for liquefaction potential. The liquefaction analysis indicated that the soils had a low potential for liquefaction under seismic conditions due to the historically highest groundwater being at a depth of approximately 100 feet below existing grade. Therefore, no mitigation measures are warranted.

7.5 Seismic Densification

One of the most common phenomena during seismic shaking accompanying any earthquake is the induced settlement of loose unconsolidated soils. Based on site subsurface conditions and the high seismicity of the region, any loose granular materials at the site could be vulnerable to this potential hazard. Our analysis of dynamic densification of "dry" soil in the upper 50 feet of existing soil profile was performed.

For the analysis, a maximum earthquake magnitude of 7.9 M_w and a peak horizontal ground surface acceleration of 0.93 g (with a 2 percent probability of exceedance in 50 years) were considered appropriate for the analysis. The seismic densification of dry to damp alluvial sandy soils due to onsite seismic activity is calculated to have a total settlement of approximately 3.07 inches. For relatively uniform soil conditions, differential settlement is expected to be approximately one-half of the total settlement. Therefore, the differential settlement is estimated to be 1.54 inches. The proposed site preparation methods recommended on our geotechnical report should address these geotechnical issues. The settlement analysis is included in Appendix A.

7.6 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.

7.7 Subsidence

The Riverside County Office of Information Technology GIS website shows the subject site to be in an active subsidence potential area. Widespread land subsidence has been observed in the San Jacinto basin as the area and its groundwater resources have been developed. In the fault graben to the east of the subject site, tectonic subsidence has averaged 0.2 in/yr over the past 40,000 years and subsidence due to groundwater withdrawal and aquifer compaction is 1 - 1.2 in/yr (Morton, 1995). Lofgren (1975, 1976) reported in studies that periods of subsidence tend to correspond to the periods of increased groundwater extraction, and that subsidence has been greater within the graben than on either side.

SALEM is not aware of subsidence issues in the project site vicinity. No subsidence-related ground fissures are mapped at or near the site (Bedrosian and Roffers, 2010). Based on the depth to groundwater exceeding 100 feet near the subject site, subsidence-related differential settlement on the scale of the project site is considered negligible.

7.8 Collapsible/Expansive or Hydroconsolidatable Soils

Test data in this geotechnical report show that soil samples consolidated from approximately 9 to 10 percent after a maximum 12.8 ksf load. Hydroconsolidation (collapse upon wetting) at a load of 1.6 ksf was approximately 1.5 to 2 percent. Soil samples collected from surface to the proposed foundation depths are considered to have a low to moderate expansion potential — the sample tested returned and Expansion Index value of 36. The proposed site preparation methods recommended on our geotechnical report should address these geotechnical issues – no additional mitigation measure should be required.

7.9 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 5, Flood Zone Map). The subject site is not located in areas of Riverside County Integrated Project map S-10, "Dam Failure Inundation Zones."

7.10 Landslides/Slope Instability/Debris Flow

The subject site is on a gently (<5%) sloping grade, over 3/4 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.

7.11 Wind and Water Erosion

Based on SALEM's soil boring logs for the subject site, surface soils consist predominantly of medium dense to slightly loose silty sand with varying amounts of clay, medium dense to slightly loose silty sand/sandy silt with varying amounts of clay, and firm to slightly soft sandy silt with varying amounts of clay. 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 completely covered by buildings, pavement or landscaping after development, minimizing long-term wind erosion potential.

7.12 Tsunamis and Seiches

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

8. SOIL AND GROUNDWATER CONDITIONS

8.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils within the depth of exploration consisted of alluvium deposits of loose to dense silty sand/sand, loose to medium dense silty sand with varying amounts of clay, loose to medium dense silty sand/sandy silt with varying amounts of clay, soft to stiff sandy silt with varying amounts of clay, and very stiff clayey silt.

No significant fill was encountered in our borings. Fill soils may be present onsite between our test boring locations. Verification of the extent of fill should be determined during site grading. Field and laboratory tests suggest that the deeper native soils are moderately strong and slightly compressible. These soils extended to the termination depth of our borings.

The soils were classified in the field during the drilling and sampling operations. The stratification lines were approximated by the field engineer on the basis of observations made at the time of drilling. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted.

The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The locations of the test borings were determined by measuring from feature shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants.

8.2 Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was not encountered during this investigation. The historically highest groundwater is anticipated to be at a depth of approximately 100 feet below existing grade.

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 2011 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 50 mg/kg. ACI 318 Tables 4.2.1 and 4.3.1 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, Percentage by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementations Materials Type
0.0050	Not Applicable	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 46 mg/kg. This level of chloride concentration is not considered to be corrosive. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, applicable manufacturer's recommendations for corrosion protection of buried metal pipe be closely followed.

8.4 Percolation Testing

Four percolation tests (P-1 through P-4) were performed within assumed infiltration areas and were conducted in accordance with 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.

Four 8-inch diameter boreholes were advanced to depths of approximately between 5 to 8 feet below existing grade. The holes were pre-saturated a minimum of 18 hours and maximum of 24 hours before percolation testing commenced.

Percolation rates were measured by filling the test holes with clean water and measuring the water drops at a certain time interval. The percolation rate data are presented in tabular format at the end of this Report. The difference in the percolation rates are reflected by the varied type of soil materials at the bottom of the test holes. The test results are shown on the table below.

TABLE 8.4
PERCOLATION TEST RESULTS

Test No.	Depth (feet)	Measured Percolation Rate (min/inch)	Tested Infiltration Rate* (inch/hour)	Soil Type
P-1	8	35.7	0.13	Sandy SILT (ML) w/ clay
P-2	8	6.4	0.70	Silty SAND (SM)
P-3	7	25.0	0.17	Sandy SILT (ML)w/ trace clay
P-4	5	20.8	0.23	Sandy SILT (ML)w/ trace clay

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

The soil infiltration or percolation rates are based on tests conducted with clear water. The infiltration/percolation rates may vary with time as a result of soil clogging from water impurities. The infiltration/percolation rates will deteriorate over time due to the soil conditions and an appropriate factor of safety (FS) may be applied. The owner or civil engineer may elect to use a lower FS for the design; however, more frequent maintenance will be expected. The soils may also become less permeable to impermeable if the soil is compacted. Thus, periodic maintenance consisting of clearing the bottom of the drainage system of clogged soils should be expected.

The infiltration/percolation rate may become slower if the surrounding soil is wet or saturated due to prolonged rainfalls. Additional percolation tests may be conducted at bottom of the drainage system during construction to verify the infiltration/percolation rate. Groundwater, if closer to the bottom of the drainage system, will also reduce the infiltration/percolation rate.

The scope of our services did not include a groundwater study and was limited to the performance of percolation testing and soil profile description, and the submitted data only. Our services did not include those associated with septic system design. Neither did services include an Environmental Site Assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater, or atmosphere; or the presence of wetlands.

Any statements, or absence of statements, in this report or on any boring logs regarding odors, unusual or suspicious items, or conditions observed, are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous and/or toxic assessment. The geotechnical engineering information presented herein is based upon professional interpretation utilizing standard engineering practices. The work conducted through the course of this investigation, including the preparation of this report, has been performed in accordance with the generally accepted standards of geotechnical engineering practice, which existed in the geographic area at the time the report was written. No other warranty, express or implied, is made.

Please be advised that when performing percolation testing services in relatively small diameter borings, that the testing may not fully model the actual full scale long term performance of a given site. This is particularly true where percolation test data is to be used in the design of large infiltration system such as may be proposed for the site. The measured percolation rate includes dispersion of the water at the sidewalls of the boring as well as into the underlying soils. Subsurface conditions, including percolation rates, can change over time as fine-grained soils migrate. It is not warranted that such information and interpretation cannot be superseded by future geotechnical engineering developments. We emphasize that this report is valid for the project outlined above and should not be used for any other sites.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

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

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

9.1.3 Fill materials may be present on site between boring locations. Undocumented fill materials are not suitable to support any future structures and should be excavated and replaced with Engineered Fill. Prior to fill placement, SALEM should inspect the bottom of the excavation to verify the bottom condition.

9.1.4 The site is currently utilized for agricultural purposes for the cultivation of crops. Two palm trees were observed on the southeast portion of the site. 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 10 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 root systems in proposed improvement areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots greater than ½ inch in diameter. Tree roots removed in parking areas may be limited to the upper 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 majority of the upper soils within the project site are identified primarily as silty sand with varying amounts of clay, silty sand/sandy silt with varying amounts of clay, and sandy silt with varying amounts of clay. The clayey soils exhibit a low to moderate swell potential (EI=36) and are subject to volumetric changes if moisture contents vary. The clayey soil, in its present condition, possess minor to moderate hazards to construction in terms of possible post-construction movement of the foundations and floor systems if no mitigation measures are employed. The estimated swell pressures of the clayey material may cause movement affecting slabs and brittle exterior finishes. Accordingly, measures are considered necessary to reduce anticipated soil movement.

9.1.8 To minimize the potential soil movement due to expansive soil conditions, it is recommended that the upper 12 inches of soil beneath the required granular aggregate subbase within slab on grade and exterior flatwork areas be removed and replaced with Non-Expansive Engineered Fill meeting the requirements of section 9.4. The soils with an EI greater than 20 (EI>20) may be placed below a depth of 12 inches within the building pad and exterior flatwork areas or in the parking and non-structural areas.

9.1.9 Loose to medium dense sandy soils were encountered within the soil borings at the project site. A seismic hazard, which could cause damage to the proposed development during seismic shaking, is the seismic induced settlement of loose unconsolidated soils. Our analysis of dynamic densification of "dry" soil in the upper 50 feet of existing soil profile was performed. The seismic densification of slightly moist alluvial sandy soils due to onsite seismic activity is calculated to have a total settlement of approximately 3.07 inches. Mitigation measures are recommended to minimize structural damage due to the seismic settlement. The potential for structural damage at the site can be minimized by using geogrid, a structural slab system, stone columns, or supporting the building on a deep foundation system.

9.1.10 Geogrid is a commonly and economically method to reduce structural damage due to seismic induced settlement. This method has been accepted by cities and counties throughout California, and implemented into design and construction of many retail buildings. However, this method may not be accepted by some local jurisdictions. We have no control for the acceptance of this method for this project. To use the geogrid method, it's recommended the proposed buildings be designed and the structural drawings be prepared after this report is approved by the City of San Jacinto or County of Riverside.

9.1.11 Recommendations for the geogrid system (option 1) are provided herein. As an alternative to the use of geogrid, the proposed structures may be supported by a structural slab system. A structural slab system will help reduce structural damage caused by seismic densification. Recommendations for a structural slab system (option 2) are provided in the Foundation's section of this report.

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

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

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

9.1.15 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 2013 CBC, our recommended parameters are shown below. These parameters are based on Probabilistic Ground Motion of 2% Probability of Exceedance in 50 years. The Site Class was determined based on the results of our field exploration.

TABLE 9.2.1
2013 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2010 ASCE 7 or 2013 CBC Reference
Site Coordinates (Datum = NAD 83)		33.7880 Lat -117.0079 Lon	
Site Class	--	D	ASCE 7 Table 20.3
Soil Profile Name	--	Stiff Soil	ASCE 7 Table 20.3
Risk Category	--	II	CBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.000	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.926 g	ASCE 7 Equation 11.8-1
Seismic Design Category	SDC	E	ASCE 7 Table 11.6-1 & 2
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	2.409 g	CBC Figure 1613.3.1(1-6)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	1.048 g	CBC Figure 1613.3.1(1-6)
Site Class Modified Site Coefficient	F_a	1.000	CBC Table 1613.3.3(1)
Site Class Modified Site Coefficient	F_v	1.500	CBC Table 1613.3.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	2.409 g	CBC Equation 16-37
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	1.572 g	CBC Equation 16-38
Design Spectral Response Acceleration $S_{DS}=\frac{2}{3}S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.606 g	CBC Equation 16-39
Design Spectral Response Acceleration $S_{D1}=\frac{2}{3}S_{M1}$ (1.0 sec. period)	S_{D1}	1.048 g	CBC Equation 16-40

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 onsite soils can be excavated with moderate effort using conventional excavation equipment.

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

9.3.3 The upper soils within the project site are identified primarily as loose to medium dense silty sand with varying amounts of clay, loose to medium dense silty sand/sandy silt with varying amounts of clay, soft to stiff sandy silt with varying amounts of clay. The clayey soils are moisture-sensitive and moderately expansive.

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

9.4 Materials for Fill

9.4.1 The upper on-site, native soils are predominately silty sand with varying amounts of clay, silty sand/sandy silt with varying amounts of clay, and sandy silt with varying amounts of clay. The test results indicate that the clayey soils have a low to moderate expansion potential (EI=36).

9.4.2 It is recommended that the upper 12 inches of soil within the building slab and exterior flatwork areas be replaced with "non-expansive" fill of silty sand or sandy silt with an Expansion Index equal to or less than 20. The replacement soils should extend 5 feet beyond the perimeter of the building. The soils with an EI greater than 20 (EI>20) may be placed below a depth of 12 inches within the buildings pad and exterior flatwork areas or in the parking and non-structural areas. The exposed native soils in the excavation should not be allowed to dry out and should be kept continuously moist prior to backfilling.

9.4.3 Import soil intended for use as Non-Expansive Engineered Fill 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. A sandy soil will allow the surface water to drain into the expansive clayey soils below, which may result in unacceptable swelling. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 9.4.3.

TABLE 9.4.3
IMPORT FILL REQUIREMENTS

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

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

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

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

9.5 Grading

9.5.1 A 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 The site is currently utilized for agricultural purposes for the cultivation of crops. Two palm trees were observed on the southeast portion of the site. 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 10 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.

9.5.4 Tree root systems in proposed improvement areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots greater than ½ inch in diameter. Tree roots removed in parking areas may be limited to the upper 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.

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

9.5.6 To minimize the potential soil movement due to expansive soil condition, it is recommended that the upper 12 inches of soil beneath the required granular aggregate subbase within slab on grade and exterior flatwork areas be removed and replaced with Non-Expansive Engineered Fill meeting the requirements of section 9.4. The soils with an EI greater than 20 (EI>20) may be placed below a depth of 12 inches within the building pad and exterior flatwork areas or in the parking and non-structural areas.

9.5.7 Any 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.

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

9.5.9 All Engineered Fill soils should be moisture conditioned to near optimum moisture content and compacted to at least 95% (90 % for cohesive soils) relative compaction.

9.5.10 Non-Expansive Engineered Fill and non-cohesive soils should be placed, moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction.

9.5.11 Within pavement and canopy areas, it is recommended that overexcavation and recompaction be performed to at least 18 inches below existing grade or finish grade, whichever is deeper. In addition, the upper 12 inches of final pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned to no less than the optimum moisture content and compacted to at least 95% (90% for cohesive soils) relative compaction.

9.5.12 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 percent for cohesive soils) of the maximum dry density based on ASTM D1557-07 Test Method.

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 moths of the year (typically summer and fall). However, groundwater and soil moisture conditions could be significantly different during the wet season (typically winter and spring) as surface soil becomes wet; perched groundwater conditions may develop. Grading during this time period will likely encounter wet materials resulting in possible excavation and fill placement difficulties. Project site winterization consisting of placement of aggregate base and protecting exposed soils during construction should be performed. If the construction schedule requires grading operations during the wet season, we can provide additional recommendations as conditions warrant.

9.5.17 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 BX 1100 or TX 140) below the crushed rock will enhance stability and reduce the required thickness of crushed rock necessary for stabilization. Our firm should be consulted prior to implementing remedial measures to provide appropriate recommendations.

9.6 Option 1 - Shallow Foundations with Geogrid

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

9.6.2 Subsurface soils within the site are prone to densification under high ground shaking acceleration during an earthquake. Our preliminary calculations indicated that the building areas, and at least 5 feet beyond, should be over-excavated to a depth of 5 feet below finish grade or 3 feet below proposed footings, whichever is greater, and the resulting excavation should be backfilled with a layered system of Engineered Fill and geogrid reinforcing fabric. Any undocumented and uncompacted fills encountered during grading should be removed and replaced with engineered fill. The depth of the over-excavation should be measured from existing ground or rough pad grade, whichever is greater. A preliminary design procedure is provided below. Final design will be provided by the geogrid manufacturer along with our office. Global seismic induced settlement of the site is still anticipated when seismic densification occurs.

Prior to placing the geogrid, the bottom of the subgrade should be scarified to a depth of 8 inches, moisture conditioned to near optimum moisture, and recompacted to a minimum of 95 percent (90% for cohesive soil) relative compaction based on ASTM D 1557 (Latest Edition).

The first layer of geogrid reinforcement will be placed directly on the prepared subgrade at a depth of 5 feet below finish grade or 3 feet below proposed footings, whichever is deeper. The geogrid material should be overlapped a minimum of 3 feet in all directions. The interlock between the geogrid and Engineered Fill will provide load transfer. No vehicles may traverse the geogrid prior to placement of the Engineered Fill cover. The next layer of geogrid should be placed on top of the compacted Engineered Fill. This and subsequent layers need only be overlapped a minimum of 1 foot on all sides. The fill soils excavated from the area beneath the structure may be moisture conditioned and recompacted between geogrid layers as reinforced fill. The reinforced fill should be moisture conditioned to near optimum moisture content and recompacted to a minimum of 95 percent (90% for cohesive soil) of the maximum dry density based on ASTM D 1557 Test Method.

A total of three (3) geogrid layers, including the layer at the base of the excavation should be installed at vertical increments of 1 foot. The geogrid layers should extend to a minimum of 5 feet beyond the exterior footing perimeter of the structure. The geogrid reinforcement fabric should consist of Tensar® TX7 Geogrid. Any additional unstable soils within building areas should be excavated and backfilled with Engineered Fill. It is recommended that the entire site be excavated at once, and soils be stockpiled on adjacent or nearby properties. The geogrid and excavated soil may then be placed and recompacted as recommended herein. It is further recommended that flexible utility connections be used for the project.

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

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

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

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

9.6.6 For design purposes, total static settlement not exceeding 1 inch may be assumed for shallow foundations. Differential static settlement should not exceed 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. The total settlement due to severe seismic loads is expected to be on the order of 3.07 inches. With the geogrid reinforcement, the seismic induced differential settlement is expected to be reduced to approximately ½ inch.

9.6.7 Resistance to lateral footing displacement can be computed using an allowable coefficient of friction factor of 0.37 acting between the base of foundations and the supporting native subgrade.

9.6.8 Lateral resistance for footings can alternatively be developed using an equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical native footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combination in Section 1605.3.2 of the 2012 IBC/2013 CBC that includes wind or earthquake loads.

9.6.9 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.

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

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

9.7 Option 2 – Structural Slabs

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

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

9.7.3 The structural slab should have a minimum depth of 12 inches below the lowest adjacent exterior grade. The structural slab should be supported by at least 2 feet of "non-expansive" Engineered Fill. Any undocumented and uncompacted fills encountered during grading should be removed and replaced with engineered fill.

9.7.4 The total settlement due to foundation loads (static) is not expected to exceed 1 inch. Differential settlement due to static loads should be less than ½ inch over 20 feet. Most of the settlement is expected to occur during construction as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated.

9.7.5 The seismic-induced total and differential settlements are expected to be on the order of 3.07 inches and 1.54 inches, respectively. It is further recommended that flexible utility connectors be used for this project.

9.7.6 Resistance to lateral footing displacement can be computed using an allowable friction factor of 0.37 acting between the base of foundations and the supporting subgrade. Lateral resistance for footings can alternatively be developed using an equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical slab faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance.

9.8 Caisson Foundations for Canopy

9.8.1 It is recommended that the caisson foundation should have a minimum depth of 10 feet below the lowest adjacent grade.

9.8.2 The caissons may be designed using an allowable sidewall friction of 150 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 caisson 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 in Section 1605.3.2 of the CBC that includes wind or earthquake loads.

9.8.3 Uplift loads can be resisted by caissons using an allowable sidewall friction of 120 psf of the surface area and the weight of the caisson.

9.8.4 The total static settlement of the caisson footing is not expected to exceed 1 inch. Differential settlement should be less than ½ inch. Most of the settlement is expected to occur during construction as the loads are applied.

9.8.5 Lateral loads for caissons may be designed utilizing the Isolated Pole Formula and Specifications shown on Table 1804.2, Sections 1804.3.1 and 1808.2.2 of the California Building Code. The drilled caissons may be designed for a lateral capacity of 350 pounds per square foot per foot of depth below the lowest adjacent grade to a maximum of 5,250 psf.

9.8.6 These values may be increased by one-third when using the alternative load combinations in Section 1605.3.2 of the IBC that include wind or earthquake loads. These values should not be doubled since the values given herein are higher than the tabular values shown on the Table 1804.2. The lateral loading criteria is based on the assumption that the load application is applied at the ground level, flexible cap connections applied and a minimum embedment depth of 10 feet.

9.8.7 Sandy soils were encountered at the site. Casing of the drilled caisson will be required if seepage is encountered or the drilled hole has to be left open for an extended period of time.

9.9 Concrete Slabs-on-Grade

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

9.9.2 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 to prevent capillary moisture rise.

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

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

9.9.5 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 full depth construction joints or control joints be provided at a maximum spacing of 15 feet in each direction for 5-inch thick slabs and 12 feet for 4-inch thick slabs.

9.9.6 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.

9.9.7 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.

9.9.8 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.

9.9.9 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.

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

9.9.11 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.

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

9.10 Lateral Earth Pressures and Frictional Resistance

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

Lateral Pressure Conditions	Equivalent Fluid Pressure, pcf
Active Pressure, Drained	40
At-Rest Pressure, Drained	60
Passive Pressure	350
Related Parameters	
Allowable Coefficient of Friction	0.37
In-Place Soil Density (lbs/ft^3)	120

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

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

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

9.10.5 For stability against lateral sliding, which is resisted solely by the passive pressure, we recommend a minimum safety factor of 1.5. For stability against lateral sliding, which is resisted by the combined passive and frictional resistance, a minimum safety factor of 2.0 is recommended. For lateral stability against seismic loading conditions, we recommend a minimum safety factor of 1.1.

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

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

9.11 Retaining Walls

9.11.1 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.

9.11.2 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.

9.11.3 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.

9.11.4 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.

9.11.5 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.

9.12 Temporary Excavations

9.12.1 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.

9.12.2 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.

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

9.12.4 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

9.12.5 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.

9.12.6 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.

9.12.7 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.

9.13 Underground Utilities

9.13.1 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 cohesive soil) relative compaction at or above optimum moisture content.

9.13.2 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.

9.13.3 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.

9.13.4 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.

9.14 Surface Drainage

9.14.1 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.

9.14.2 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.

9.14.3 Impervious surfaces within 10 feet of the buildings foundation shall be sloped a minimum of 2 percent away from the building and drainage gradients maintained to carry all surface water to collection facilities and off site. These grades should be maintained for the life of the project. Ponding of water should not be allowed adjacent to the structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed.

9.14.4 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.

9.15 Pavement Design

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

9.15.2 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 9.15.2
ASPHALT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Asphaltic Concrete	Class II Aggregate Base*	Compacted Subgrade**
5.0 (Parking & Vehicle Drive Areas)	3.0"	5.0"	18.0"
6.0 (Heavy Truck Areas)	3.0"	8.5"	18.0"

*****95% compaction based on ASTM D1557 Test Method***
*****95% (90% for cohesive soils) compaction based on ASTM D1557 Test Method***

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

TABLE 9.15.3
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Portland Cement Concrete*	Class II Aggregate Base**	Compacted Subgrade**
5.0 (Light Duty)	5.0"	4.0"	18.0"
6.0 (Heavy Duty)	6.5"	6.0"	18.0"

**** Minimum Compressive Strength of 4,000 psi***
***** 95% compaction based on ASTM D1557 Test Method***
******95% (90% for cohesive soils) compaction based on ASTM D1557 Test Method***

10. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

10.1 Plan and Specification Review

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

10.2 Construction Observation and Testing Services

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

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

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

11. LIMITATIONS AND CHANGED CONDITIONS

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

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

If site conditions change due to natural processes or human intervention on the property or adjacent to the site, or changes occur in the nature or design of the project, or if there is a substantial time lapse between the submission of this report and the start of the work at the site, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed by SALEM and the conclusions of our report are modified or verified in writing. The validity of the recommendations contained in this report is also dependent upon an adequate testing and observations program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction. SALEM has prepared this report for the exclusive use of the owner and project design consultants. SALEM does not practice in the field of corrosion engineering. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, that manufacturer's recommendations for corrosion protection be closely followed. Further, a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of concrete slabs and foundations in direct contact with native soil. The importation of soil and or aggregate materials to the site should be screened to determine the potential for corrosion to concrete and buried metal piping. The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Ibrahim Ibrahim, MS, EIT
Geotechnical Staff Engineer

Clarence Jiang, GE
Senior Geotechnical Engineer
RGE 2477

REGISTERED PROFESSIONAL ENGINEER
CLARENCE T. JIANG
No. 2477
Exp. 6/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

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

REGISTERED PROFESSIONAL ENGINEER
R. SAMMY SALEM
No. 2549
Exp. Dec. 31, 2016
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

Hemet San Jacinto Water Management Plan: http://www.water.ca.gov/groundwater/docs/GWMP/SC-12_Hemet-SanJacintoGWMArea_WMP_2007.pdf

Bedrossian, T.L. and Roffers, P., 2010, Geologic Compilation of Quaternary Surficial Deposits in Southern California; California Geological Survey Special Report 217; Plate 16, Santa Ana 30 X 60 Quadrangle, scale 1:100,000.

California State Water Resources Control Board [SWRCB]; Geotracker database web page; Availability: http://geotracker.swrcb.ca.gov

Morton, D.M. and F.K. Miller (2006) Geologic map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California: U.S. Geological Survey Open-File Report 2006-1217, scale 1:100,000 .

Riverside County General Plan (2003) Chapter 6: Safety Element; Transportation and Land Management Agency (http://www.rctlma.org/genplan/content/gp/chapter06.html)

Riverside County Information Technology (RCIT), Map My County (ver. 4) website: http://mmc.rivcoit.org/MMC_Public/Custom/disclaimer/Default.htm

Lofgren, Ben. (1976). Land Subsidence & Aquifer System Compaction, Journal Research USGS

Lofgren & Rubin. (1975). Radiocarbon Dates Indicate Rates of Garden Downfaulting, Journal Research USGS

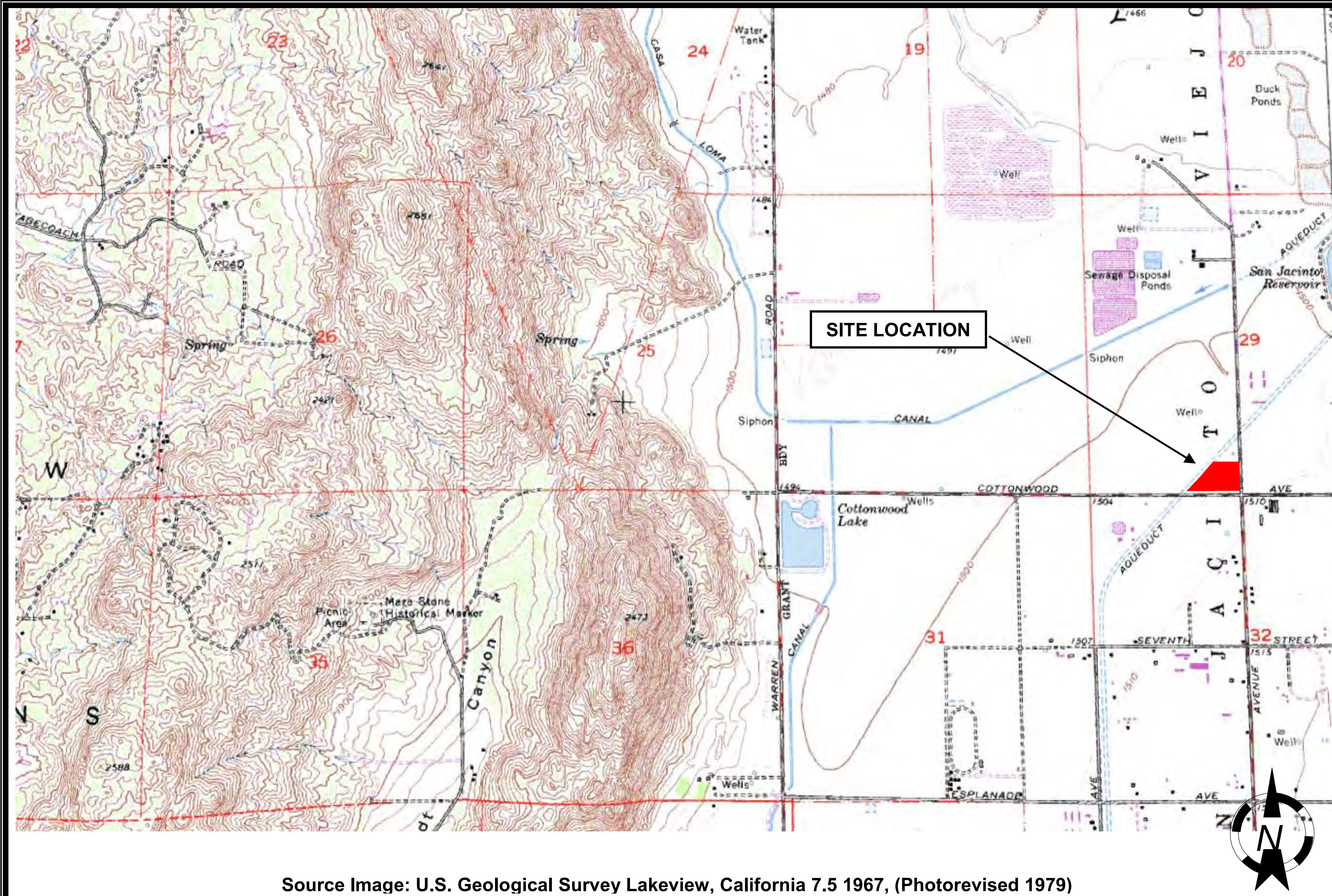
Bryant, W. A. [compiler] (2005) Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: California Geological Survey Web Page, <http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.htm

Morton, D.M. (1995) Subsidence and Ground Fissuring in the San Jacinto Basin, Southern California: In K. R. Prince, D. L. Galloway and S. A. Leake (eds.), U.S. Geological Survey Subsidence Interest Group Conference, Edwards Air Force Base, Antelope Valley, California.

Dibblee, T W. and Minch W.A., (2003) Geologic Map of the Lakeview Quadrangle, Riverside County, California, D.M. (1995) Dibblee Geologic Foundation, Dibblee Foundation Map DF-115, scale 1:24,000.

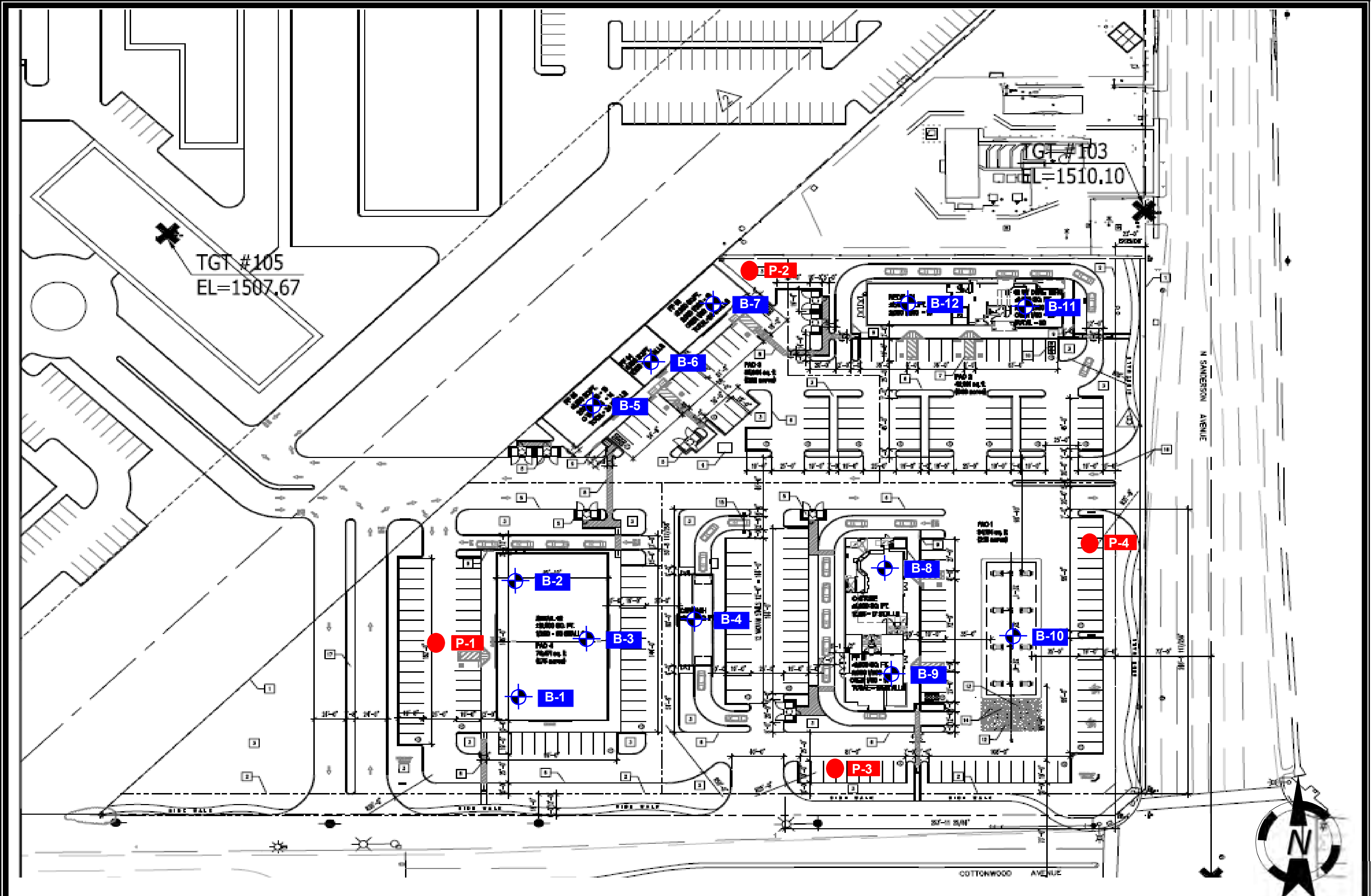
USGS National Seismic Hazard Mapping Program Java applet tool to determine site-specific accelerations (available at http://earthquake.usgs.gov/designmaps/us/application.php/).

Wills, C.J., Weldon, R.J., II, and Bryant, W.A. (2008) California fault parameters for the National Seismic Hazard Maps and Working Group on California Earthquake Probabilities, *Appendix A in* The Uniform California Earthquake Rupture Forecast, version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437A, and California Geological Survey Special Report 203A, 48 p. [http://pubs.usgs.gov/of/2007/1437/a/].



Source Image: U.S. Geological Survey Lakeview, California 7.5 1967, (Photorevised 1979)

SVICINITY MAP **GEOTECHNICAL ENGINEERING INVESTIGATION** **Proposed Commercial Development** **NWC N. Sanderson Avenue and Cottonwood Avenue** **San Jacinto, California**	SCALE: NOT TO SCALE	DATE: 05/2016	SALEM engineering group, inc.
	DRAWN BY: AA	APPROVED BY: CJ	
	PROJECT NO. 3-216-0492	FIGURE NO. 1	



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Commercial Development
NWC N. Sanderson Avenue and Cottonwood Avenue
San Jacinto, California

SCALE: NOT TO SCALE	DATE: 05/2016
DRAWN BY: AA	APPROVED BY: CJ
PROJECT NO. 3-216-0492	FIGURE NO. 2

LEGEND:
B-1 Soil Boring Locations
P-1 Percolation Test Locations
All Locations Approximate

SALEM engineering group, inc.

Dibblee, T W. and Minch W.A., (2003) Geologic Map of the Lakeview Quadrangle, Riverside County, California, D.M. (1995) Dibblee Geologic Foundation, Dibblee Foundation Map DF-115, scale 1:24,000.

Regional Geologic Map

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Commercial Development
NWC N. Sanderson Avenue and Cottonwood Avenue
San Jacinto, California

SCALE: NOT TO SCALE	DATE: 05/2016
DRAWN BY: II	APPROVED BY: CJ
PROJECT NO. 3-216-0492	FIGURE NO. 3A

SALEM engineering group, inc.

Symbol explination

GEOLOGIC SYMBOLS

not all symbols shown on each map

FORMATION CONTACT *dashed where inferred or indefinite dotted where concealed*

MEMBER CONTACT *between units of a formation* — Prominent bed

CONTACT BETWEEN SURFICIAL SEDIMENTS *located only approximately in places*

FAULT: *Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth are on upper plate of low angle thrust fault.* U D 25 ?

FOLDS: *overturned* ANTICLINE SYNCLINE

arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

Strike and dip of Sedimentary rocks: 18 *inclined*; 20, 3 *inclined (approximate)*; 80 *overturned*; *horizontal*; *vertical*

Strike and dip of metamorphic or igneous rock foliation or flow banding or compositional layers: 75 *inclined*; *inclined (approximate)*; *vertical*; 80 *overturned*

OTHER SYMBOLS: *Direction of landslide movement*; *outline of water bodies shown on map*; *water well*; *oil well*

LEGEND

Qa Qg Qc

SURFICIAL SEDIMENTS

Alluvial sediments, unconsolidated, undissected

Qg *Alluvial sand of major stream channels*
Qa *Alluvial sand and clay of valley areas, covered by gray soil*
Qc *Clay of lake bed*

Qls

LANDSLIDE DEBRIS

Qls *Landslide of rock rubble*

Qoa

OLDER SURFICIAL SEDIMENTS

Dissected older alluvial deposits, undeformed; late Pleistocene age

Qoa *Alluvial fan gravel and sand derived from adjacent bedrock areas, slightly dissected locally, grades down slope into* **Qa**, *with gradational contact shown approximately*

Tst

SAN TIMOTEO FORMATION

(San Timoteo beds of Frick, 1921) stream-laid alluvial sediments of detritus derived from plutonic and metamorphic rocks of San Bernadino Mountains area, weakly indurated; only basal part of formation exposed at NE corner of this quadrangle; age, Pliocene

Tst *Sandstone, light grey to greenish gray, fine to medium grained, arkosic, and some pebble conglomerate of granitic clasts*

— *UNCONFORMITY* —

Tme Tmeg Tmeb

MOUNT EDEN FORMATION

(of Fraser, 1931), moderately lithified, derived from basement rocks of San Jacinto Mountains; age, upper Miocene (Clarendonian Stage of Fraser, 1921)

Tme *Sandstone, light greenish gray to reddish, bedded, arkosic, includes lenses of claystone, gray to light reddish brown, and lenses of pebble-cobble conglomerate*
Tmeg *Boulder conglomerate lenses, unsorted, angular clasts of light gray quartz diorite,* (**qdi**) *near top of Formation near northeast corner of quadrangle*
Tmeb *Diorite breccia, of shattered masses and breccia, dark to medium gray dioritic and granitic detritus, unsorted rubble mostly of dark hornblende-biotite rich quartz diorite, probably landslide masses of* **qdh** & **qdx**, *exposed only near north border of quadrangle, grades laterally NW through breccia into* **Tme**

Holocene — Pleistocene — ? — Pliocene — ? — Miocene

QUATERNARY — ? — TERTIARY

Geologic Unit Explanation

— *UNCONFORMITY* —

gr p
qm qdh qdi qdx

PLUTONIC ROCKS

Medium grained holocrystalline granitic rocks, part of Peninsular Range batholith; age, Cretaceous

gr *Granite of Mt. Eden area, leucocratic gray-white, mostly dike rocks, massive, of granite to quartz monzonite composition, of quartz, potassic feldspar and sodic plagioclase feldspar in nearly equal amounts, and less than 5% mica; massive, includes aplite and pegmatite dikes too small to map*
p *Pegmatite dike rocks, leucocratic, very coarse grained granitic rock, of quartz, alkali feldspar, biotite, and mica; dikes as wide as 2 m, forms dike swarm in quartz diorite in central west area, and scattered dikes elsewhere*
qm *Quartz monzonite (of Morton, 1969), Leucocratic light gray, massive to somewhat gneissoid, composed of quartz, potassic feldspar and sodic plagioclase feldspar in about equal amounts and minor biotite, porphyritic with small phenocrysts of potassic feldspar*
qdh *Quartz diorite, hornblende–rich; Lakeview Mountain quartz diorite (of Dudley, 1935), Lakeview Mountain Tonalite (of Larsen, 1948), gray, gneissoid, about 1/4 quartz, 1/2 sodic plagioclase feldspar , no potassic feldspar, and about 1/4 biotite and hornblende; gneissoid structure from parallel orientation of biotite and hornblende, contains many dark gray discoid inclusions (xenoliths) oriented parallel to gneissoid foliation of rock; includes gneiss-like schlieren from shearing during crystallization, most oriented parallel to gneissoid foliation; radiometric ages of rock (K/A of biotite) range from 87.0 to 98.1 +- 10 MA (Morton, 1969). Unit interpreted by Morton 1969, Morton and Matti, 2001, as Lakeview Mountain Pluton, intrusive as a vertical crescent-shaped mass convex southeastward into older plutonic and metamorphic rocks, but mass is accordant to, not cross-cutting, the regional structural grain of this part of Peninsular Range batholith; suggestive that this rock may have in part recrystallized from a preexisting metasedimentary protolith; this rock not exposed elsewhere in Peninsular Range batholith*
qdi *Quartz diorite, light to medium gray rocks of mostly quartz diorite to granodiorite; composed mostly of sodic plagioclase feldspar, and variable amounts of quartz, potassic feldspar, biiotite and hornblende, contains some discoid inclusions, locally includes mixtures (migmatites) of schist/gneiss*
qdx *Quartz diorite, similar to* **qdi**, *but mostly gray, more variable in composition, more gneissoid with more abundant dark gray discoid inclusions (xenoliths), and locally includes migmatites*

mig

MIGMATITE

mig *Mixtures of metasedimentary rocks (schist–gneiss–***ms***) described below with quartz diorite* (**qdi**) *described above; complexly migmatized, occur mostly around margins of unit* **gdh** *of Lakeview Mountain Pluton in some places*

ms ml

METASEDIMENTARY ROCKS

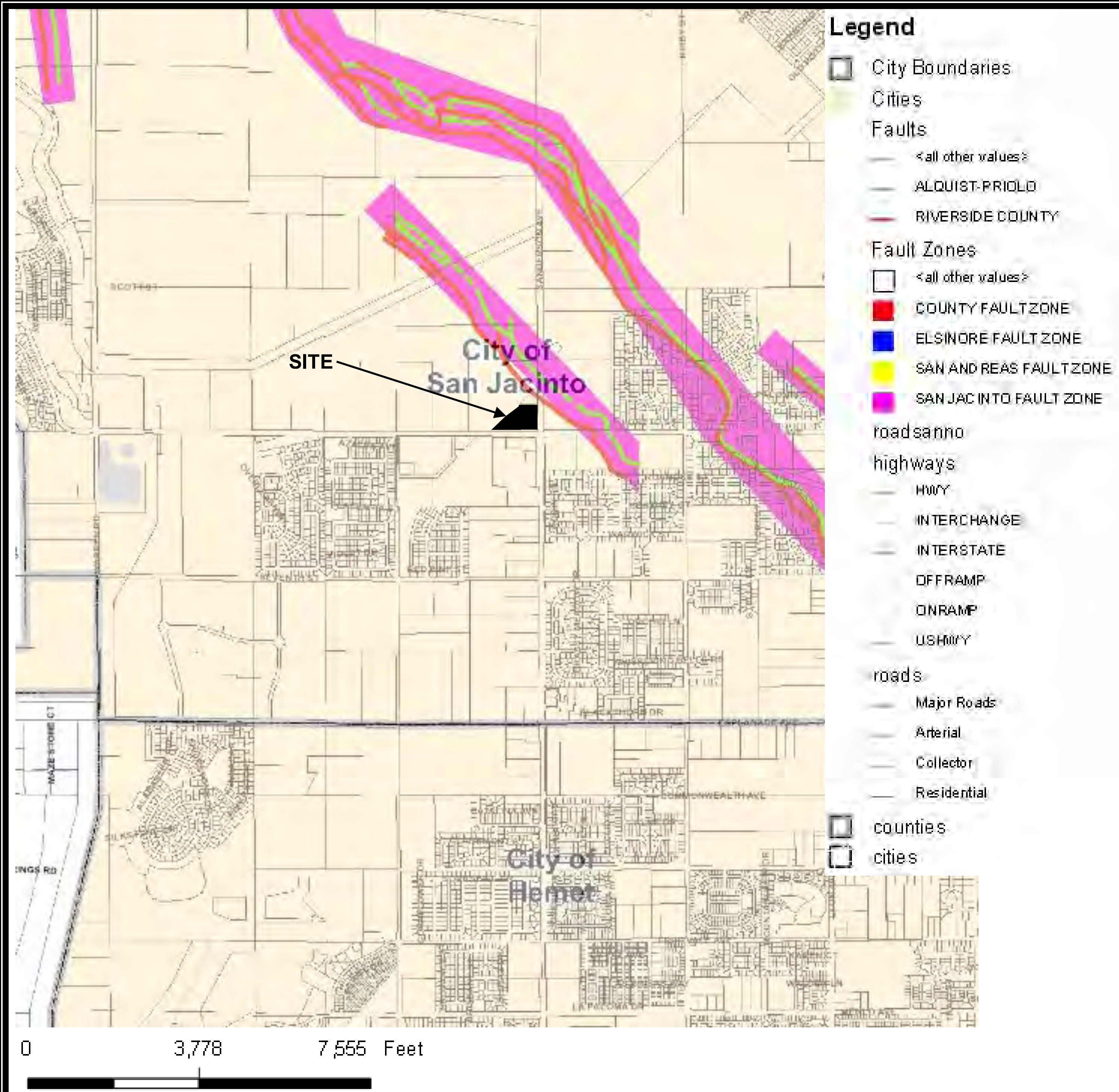
Rocks crystallized at depth from deformed sedimentary rocks, mostly argillaceous; of presumed Paleozoic? or Mesozoic age

ms *Schist, dark gray, fine-medium grained, foliated, of mica (mostly biotite), feldspar and quartz; prominently emposed in northeast area where it includes marble; in other areas schist in part crystalline to fine grained gneiss*
ml *Marble, white to light gray, fine crystalline, of calcite or dolomite*

CRETACEOUS — MESOZOIC — ? — PALEOZOIC - MESOZOIC

Dibblee, T W. and Minch W.A., (2003) Geologic Map of the Lakeview Quadrangle, Riverside County, California, D.M. (1995) Dibblee Geologic Foundation, Dibblee Foundation Map DF-115, scale 1:24,000.

Regional Geologic Map GEOTECHNICAL ENGINEERING INVESTIGATION Proposed Commercial Development NWC N. Sanderson Avenue and Cottonwood Avenue San Jacinto, California	SCALE: NOT TO SCALE	DATE: 05/2016	SALEM engineering group, inc.
	DRAWN BY: II	APPROVED BY: CJ	
	PROJECT NO. 3-216-0492	FIGURE NO. 3B	



IMPORTANT Maps and data are to be used for reference purposes only. Map features are approximate, and are not necessarily accurate to surveying or engineering standards. The County of Riverside makes no warranty or guarantee as to the content (the source is often third party), accuracy, timeliness, or completeness of any of the data provided, and assumes no legal responsibility for the information contained on this map. Any use of this product with respect to accuracy and precision shall be the sole responsibility of the user.

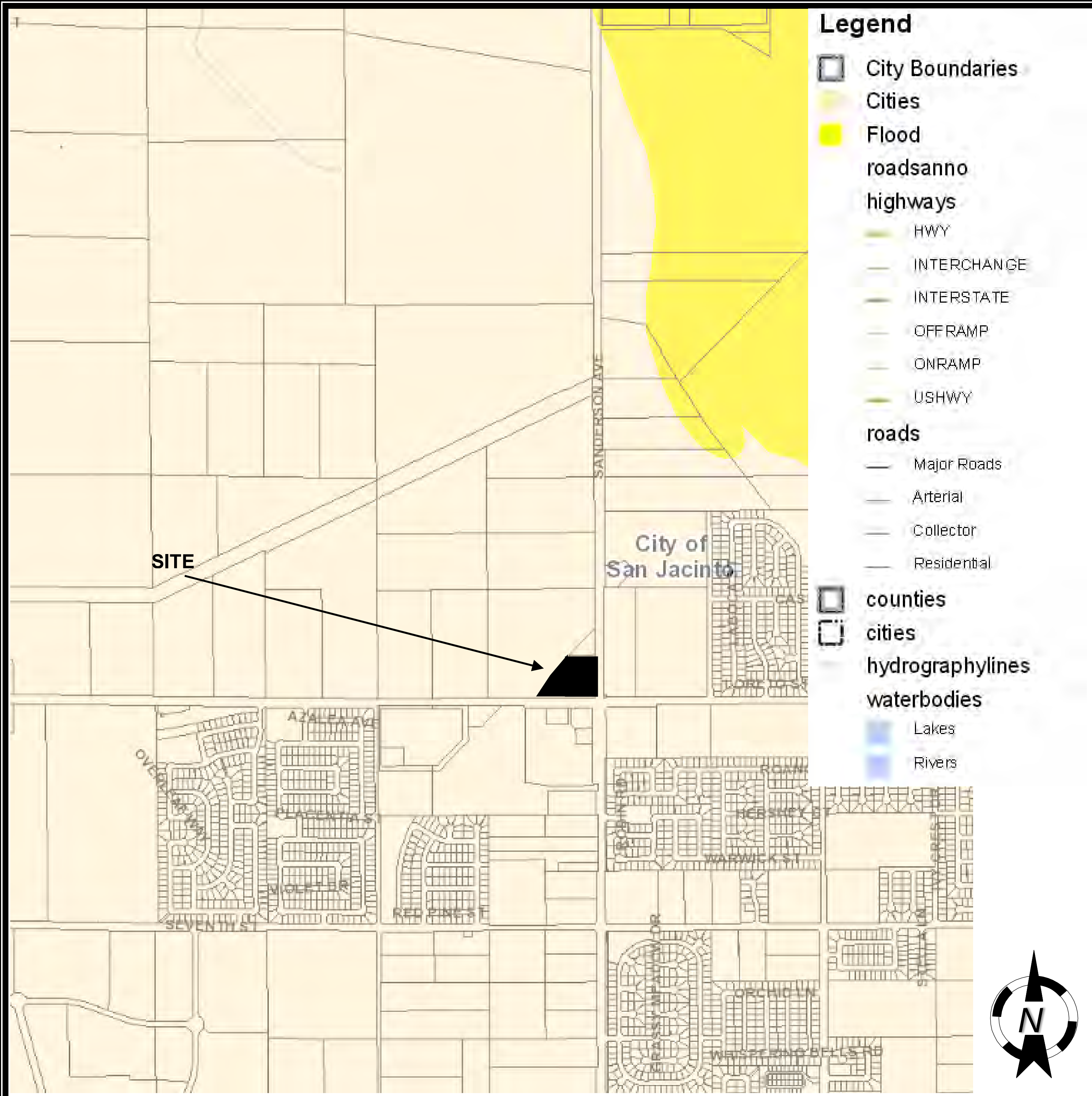
REPORT PRINTED ON... 5/13/2016 10:20:57 AM

Regional Fault Map

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Commercial Development
NWC N. Sanderson Avenue and Cottonwood Avenue
San Jacinto, California

SCALE:	DATE:
AS SHOWN	05/2016
DRAWN BY:	APPROVED BY:
II	CJ
PROJECT NO.	FIGURE NO.
3-216-0492	4

SALEM engineering group, inc.



IMPORTANT Maps and data are to be used for reference purposes only. Map features are approximate, and are not necessarily accurate to surveying or engineering standards. The County of Riverside makes no warranty or guarantee as to the content (the source is often third party), accuracy, timeliness, or completeness of any of the data provided, and assumes no legal responsibility for the information contained on this map. Any use of this product with respect to accuracy and precision shall be the sole responsibility of the user.

REPORT PRINTED ON...5/13/2016 10:42:23 AM

Flood Zone Map

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Commercial Development
NWC N. Sanderson Avenue and Cottonwood Avenue
San Jacinto, California

SCALE: Not To Scale	DATE: 05/2016
DRAWN BY: II	APPROVED BY: CJ
PROJECT NO. 3-216-0492	FIGURE NO. 5

SALEM engineering group, inc.

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation (drilling) was conducted on May 5, 2016 and included a site visit, subsurface exploration, and soil sampling. Percolation tests were performed on May 6, 2016. The locations of the exploratory borings and percolation tests 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 an 8-inch hollow stem auger and a 4-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 drill cuttings.

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

Unified Soil Classification System

Major Divisions			Letter	Symbol	Description
Coarse-grained Soils More than ½ retained on the No. 200 Sieve	**Gravels** More than ½ coarse fraction retained on the No. 4 sieve	Clean Gravels	GW		Well-graded gravels and gravel-sand mixtures, little or no fines.
			GP		Poorly-graded gravels and gravel-sand mixtures, little or no fines.
		Gravels With Fines	GM		Silty gravels, gravel-sand-silt mixtures.
			GC		Clayey gravels, gravel-sand-clay mixtures.
	Sands More than ½ passing through the No. 4 sieve	Clean Sands	SW		Well-graded sands and gravelly sands, little or no fines.
			SP		Poorly-graded sands and gravelly sands, little or no fines.
		Sands With Fines	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sandy-clay mixtures.
Fine-grained Soils More than ½ passing through the No. 200 Sieve	**Silts and Clays** Liquid Limit less than 50%		ML		Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL		Organic clays of medium to high plasticity.
	Silts and Clays Liquid Limit greater than 50%		MH		Inorganic silts, micaceous or diatomaceous fines sands or silts, elastic silts.
			CH		Inorganic clays of high plasticity, fat clays.
			OH		Organic clays of medium to high plasticity.
Highly Organic Soils			PT		Peat, muck, and other highly organic soils.

Consistency Classification

Granular Soils

Description - Blows Per Foot (Corrected)

Description	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

Description - Blows Per Foot (Corrected)

Description	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

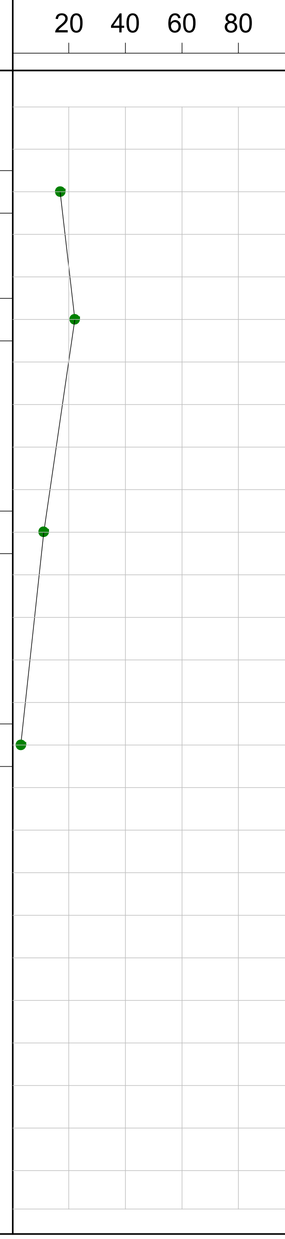
Boring No. B-1

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-1
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
0	Ground Surface				
0–4	**Silty SAND (SM)** Medium dense; slightly moist; light brown; fine grained.	106.9	5.4	MCS	17
4–12	**Silty SAND/SAND (SM/SP)** Medium dense; slightly moist; grayish yellow; fine-medium grained.	98.6	4.5	MCS	22
10	Grades as above; loose.	-	-	SPT	11
12–20	**Sandy SILT (ML)** Soft; very moist; dark brown; fine grained.	-	23.5	SPT	3
20	Grades as above.				
20	End of Borehole				

Penetration Test



Water Level

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

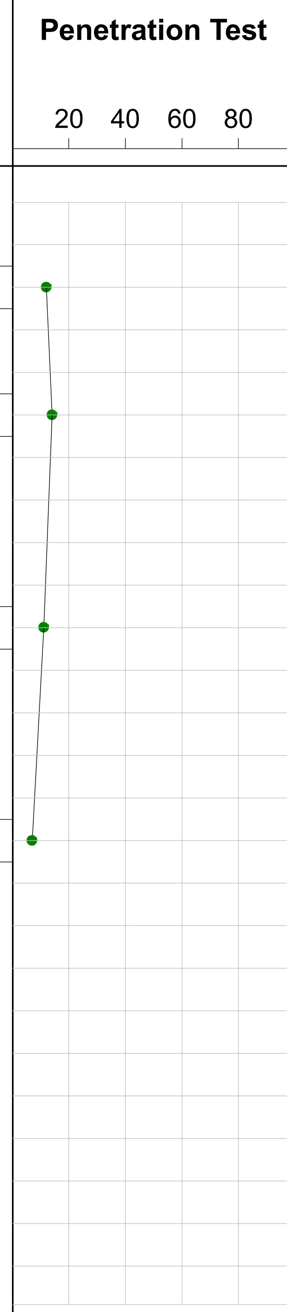
SALEM

Boring No. B-2

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-2
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count	Penetration Test	Water Level
		Ground Surface							
0		**Silty SAND (SM)** Loose; moist; dark brown; fine grained; with trace clay.	115.9	7.5	MCS		12		
5		**Silty SAND/SAND (SM/SP)** Loose; slightly moist; grayish yellow; fine-medium grained.	97.9	4.3	MCS		14		
10		**Sandy SILT (ML)** Stiff; very moist; dark brown; fine grained; with clay.	-	26.1	SPT		11		
15		Grades as above; firm; with less clay.	-	12.4	SPT		7		
20		Grades as above.							
		End of Borehole							
25									



Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

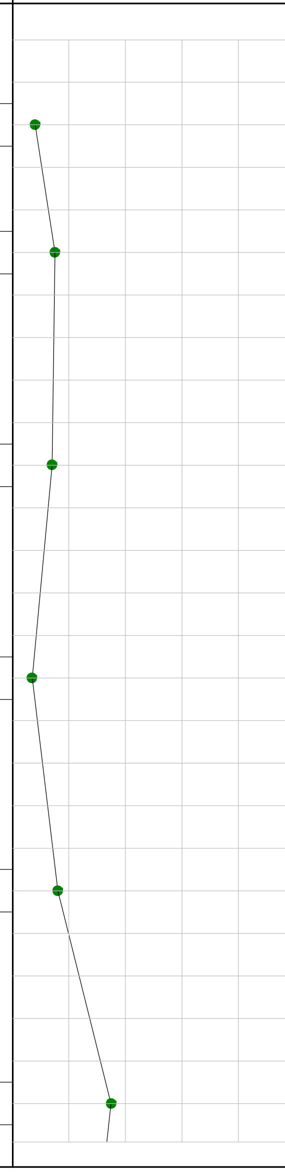
SALEM

Boring No. B-3

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-3
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	**Symbol**	**Description**	**Dry Density (pcf)**	**Moisture Content (%)**	**Sampler Type**	**Penetration**	**Blow Count**	**Penetration Test** (20 40 60 80)	**Water Level**
		Ground Surface							
0		**Silty SAND/Sandy SILT (SM/ML)** Loose; very moist; dark brown; fine grained; with clay.	116.0	14.2	MCS		8		
5		**Silty SAND (SM)** Loose; slightly moist; dark brown; fine grained.	109.3	3.8	MCS		15		
10		**Silty SAND/SAND (SM/SP)** Medium dense; slightly moist; grayish yellow; fine-medium grained.	-	4.2	SPT		14		
15		**Sandy SILT (ML)** Stiff; very moist; dark brown; fine grained; with clay.	-	20.6	SPT		7		
20		**Clayey SILT (ML)** Very stiff; very moist; dark brown; fine grianed.	-	23.2	SPT		16		
25		**Silty SAND (SM)** Dense; moist; dark brown; fine-medium grained; with trace clay.	-	10.4	SPT		35		



Drill Method: Hollow Stem Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 2

Drill Date: 5/5/16
Borehole Size: 8 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

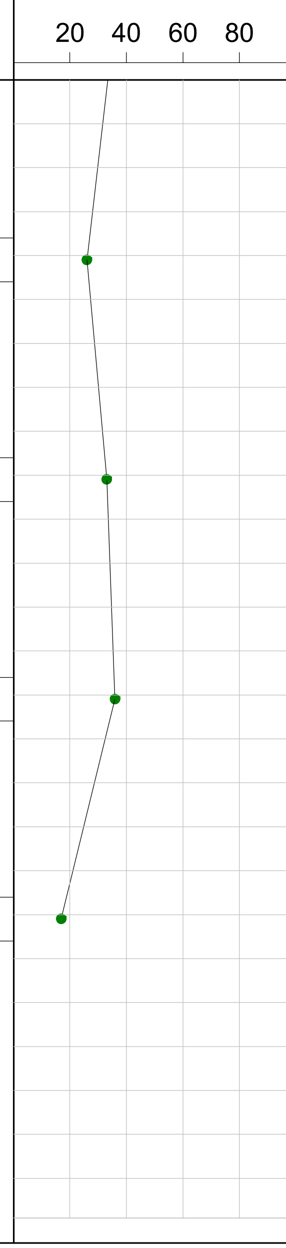
SALEM

Boring No. B-3

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-3
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count	Penetration Test	Water Level
		Grades as above.							
30		**Silty SAND/SAND (SM/SP)** Medium dense; slightly moist; grayish brown; fine-medium grained.	-	4.1	SPT		26		
35		Grades as above; dense.	-	3.4	SPT		33		
40		Grades as above; grayish yellow.	-	2.0	SPT		36		
45		**Silty SAND (SM)** Medium dense; moist; light brown; fine-medium grained.	-	8.9	SPT		17		
		Grades as above.							
50		End of Borehole							



Drill Method: Hollow Stem Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 2 of 2

Drill Date: 5/5/16
Borehole Size: 8 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

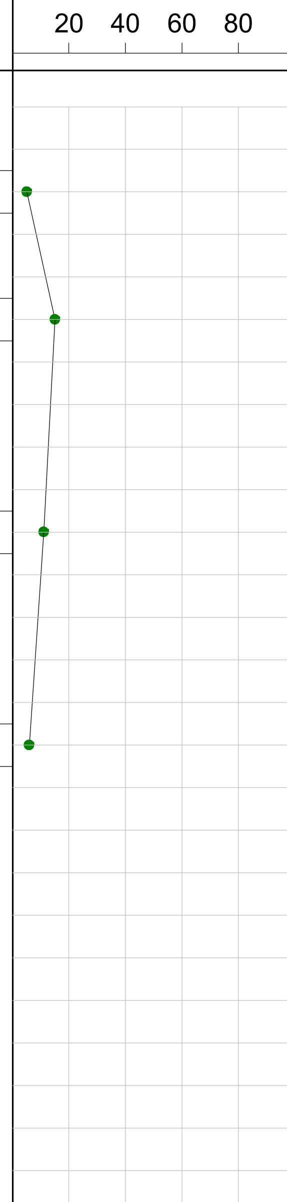
Boring No. B-4

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-4
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

Depth (ft)	Symbol	Description (Subsurface Profile)	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count	Water Level
0		Ground Surface						
0–2	ML	**Sandy SILT (ML)** Soft; moist; brown; fine grained; with minor clay.	110.0	12.4	MCS		5	
5		Grades as above; stiff.	108.2	9.9	MCS		15	
10		Grades as above; with trace clay.	-	17.0	SPT		11	
15		Grades as above; firm; very moist; with clay.	-	23.8	SPT		6	
20		Grades as above.						
20		End of Borehole						

Penetration Test



Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

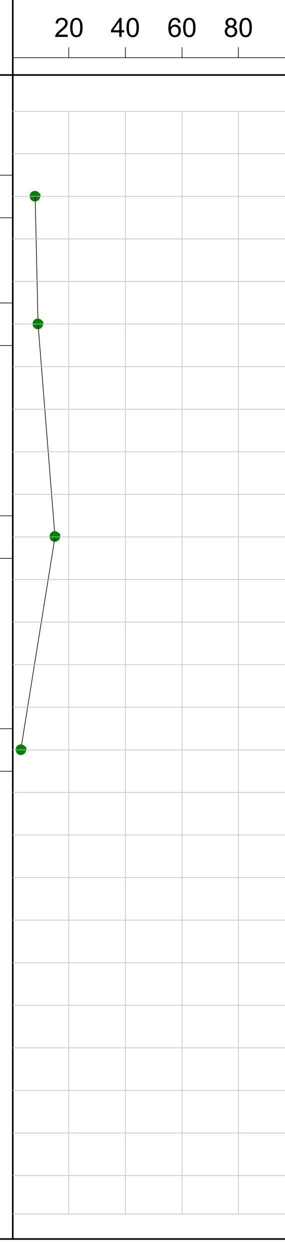
Boring No. B-5

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-5
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
0	Ground Surface				
0–10	**Silty SAND (SM)** Loose; very moist; dark brown; fine grained.				
2		110.4	17.0	MCS	8
5	Grades as above.	111.7	14.6	MCS	9
10–20	**Sandy SILT (ML)** Stiff; very moist; dark brown; fine-medium grained; with trace clay.	-	21.4	SPT	15
15	Grades as above; no recovery; soft.	-	-	SPT	3
20	Grades as above.				
20	End of Borehole				

Penetration Test



Water Level

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

Boring No. B-6

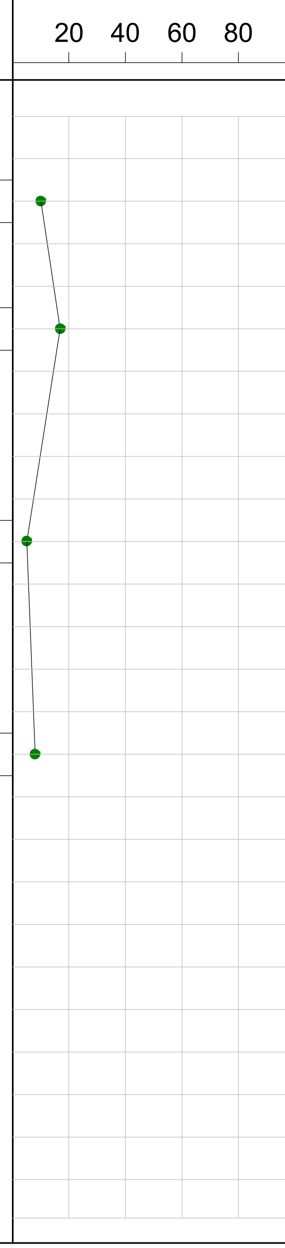
Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-6
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE / SAMPLE

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
0	Ground Surface				
0–10	**Silty SAND (SM)** Loose; very moist; brown; fine-medium grained; with trace clay.	113.2	14.5	MCS	10
5	Grades as above; medium dense; less clay.	116.0	12.3	MCS	17
10–20	**Sandy SILT (ML)** Firm; wet; brown; fine-medium grained; with clay.	-	29.2	SPT	5
15	Grades as above; with trace clay.	-	19.8	SPT	8
	Grades as above.				
20	End of Borehole				

Penetration Test



Water Level

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

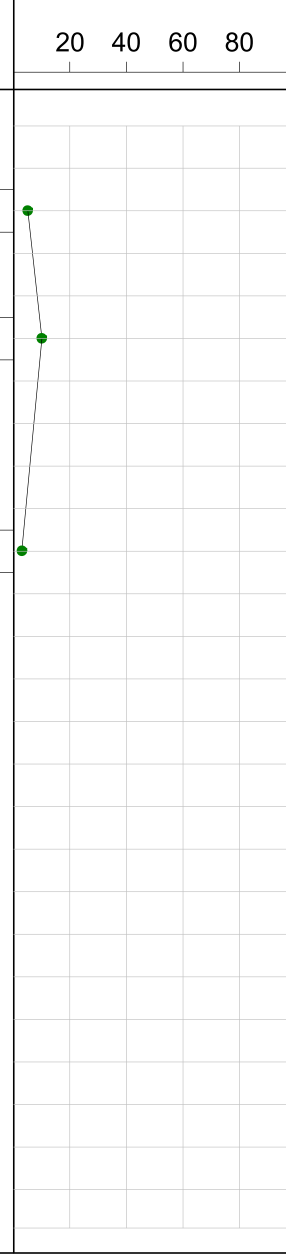
SALEM

Boring No. B-7

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-7
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count	Penetration Test (20 40 60 80)	Water Level
		Ground Surface							
0		**Sandy SILT (ML)** Soft;very moist; dark brown; fine grained grained; with trace clay.	98.2	20.6	MCS		5		
5		**Silty SAND (SM)** Loose; very moist; brown; fine-medium grained.	105.4	15.5	MCS		10		
10		**Sandy SILT (ML)** Soft; wet; dark brown; fine grained; with clay. Grades as above.	-	30.0	SPT		3		
15		End of Borehole							
20									
25									



Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

Boring No. B-8

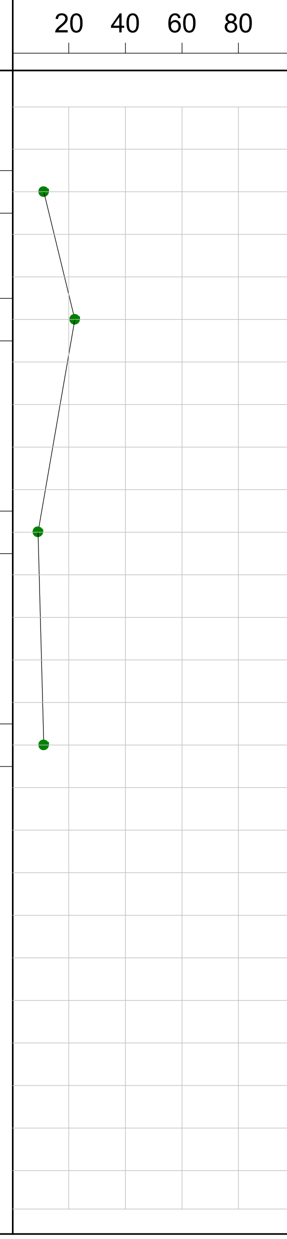
Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-8
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE / SAMPLE

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
0	Ground Surface				
0–4	**Sandy SILT (ML)** Stiff; moist; brown; fine grained; with trace clay.	113.4	12.3	MCS	11
4–8.5	**Silty SAND/Sandy SILT (SM/ML)** Medium dense; moist; brown; fine grained.	108.3	11.6	MCS	22
8.5–20	**Sandy SILT (ML)** Stiff; moist; dark brown; fine-medium grained; with trace clay.	-	14.6	SPT	9
15	Grades as above; very moist.	-	23.5	SPT	11
18.5	Grades as above.				
20	End of Borehole				

Penetration Test



Water Level

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

Boring No. B-9

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-9
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
	Ground Surface				
0	**Sandy SILT (ML)** Stiff; moist; brown; fine grained.				
		113.4	12.3	MCS	12
	Silty SAND/Sandy SILT (SM/ML) Medium dense; moist; brown; fine grained.				
5		108.3	11.6	MCS	20
	Sandy SILT (ML) Stiff; moist; dark brown; fine-medium grained; with trace clay.				
10		-	14.6	SPT	10
15	Grades as above; very moist.	-	23.5	SPT	12
20	Grades as above.				
	End of Borehole				
25					

SUBSURFACE PROFILE: Depth (ft), Symbol, Description. SAMPLE: Dry Density (pcf), Moisture Content (%), Sampler Type, Penetration, Blow Count. Penetration Test (20, 40, 60, 80). Water Level.

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

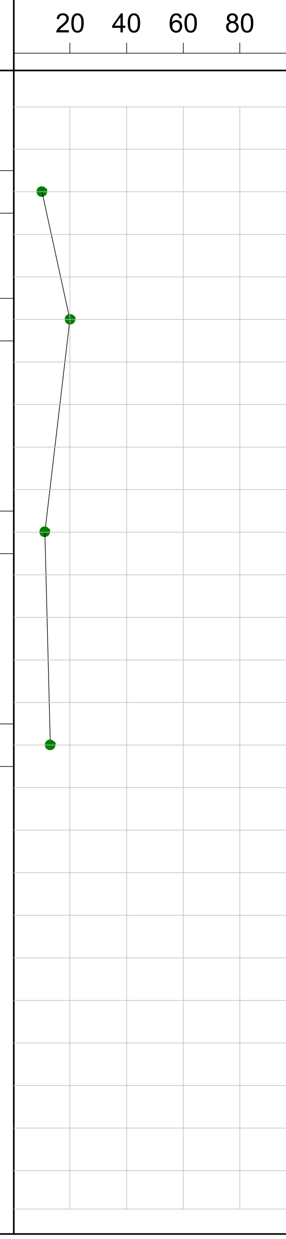
Boring No. B-10

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-10
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

Depth (ft)	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
0	Ground Surface				
0	**Sandy SILT (ML)** Firm; moist; brown; fine grained.				
2		108.5	9.2	MCS	10
5	Grades as above; stiff.	109.4	10.7	MCS	20
6.5	**Silty SAND (SM)** Moist; light brown; fine grained.				
8.5	**Sandy SILT (ML)** Stiff; very moist; grayish brown; fine grained; with trace clay.				
10		-	19.9	SPT	11
15	Grades as above; with less clay.	-	19.1	SPT	13
20	Grades as above.				
20	End of Borehole				

Penetration Test



Water Level

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

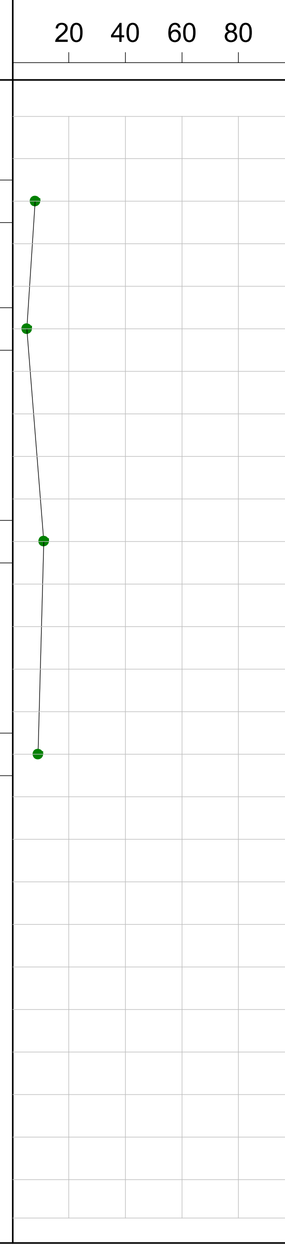
Boring No. B-11

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-11
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

Depth (ft)	Description (Subsurface Profile)	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Blow Count
0	Ground Surface				
0–4.5	**Silty SAND/Sandy SILT (SM/ML)** Loose; very moist; dark brown; fine grained; with trace clay.	103.0	17.6	MCS	8
4.5–15	**Sandy SILT (ML)** Soft; very moist; dark brown; fine grained; with trace clay.	104.6	18.1	MCS	5
10	Grades as above; firm; with trace clay.	-	23.6	SPT	11
15–20	**Silty SAND (SM)** Loose; slightly moist; light brown; with trace clay.	-	3.4	SPT	9
19.5	Grades as above.				
20	End of Borehole				

Penetration Test



Water Level

Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

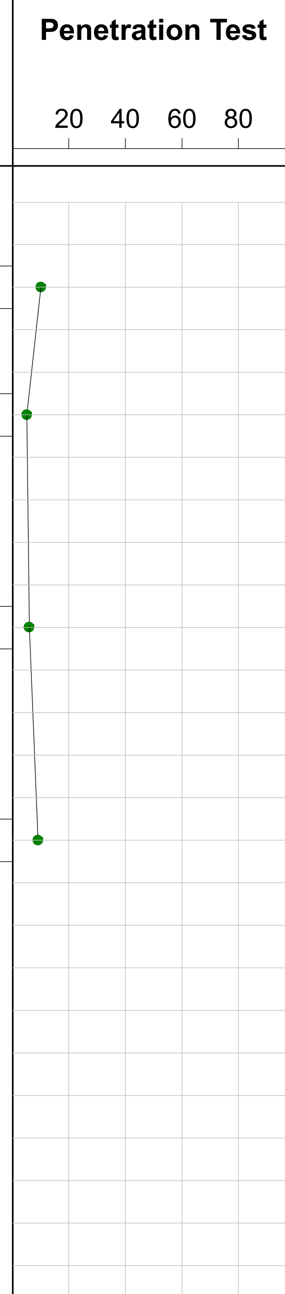
SALEM

Boring No. B-12

Project: Proposed Commercial Development
Client: J&T Business Management
Location: NWC N. Sanderson Ave & Cottonwood Ave, San Jacinto, CA
Grnd. Surf. Elev. (Ft. MSL) N/A

Project No: 3-216-0492
Figure No.: A-12
Logged By: II
Depth to Water> **Initial:** None
At Completion: None

SUBSURFACE PROFILE			SAMPLE						
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count	Penetration Test (20 40 60 80)	Water Level
		Ground Surface							
0		**Sandy SILT (ML)** Firm; very moist; dark brown; fine-medium grained; with clay.	99.7	21.9	MCS		10		
5		Grades as above; soft.	94.7	24.1	MCS		5		
10		Grades as above; firm.	-	24.3	SPT		6		
15		Grades as above; firm.	-	28.3	SPT		9		
20		Grades as above.							
		End of Borehole							
25									



Drill Method: Solid Flight Auger
Drill Rig: CME 45C
Driller: Salem Engineering Group, Inc.
Sheet: 1 of 1

Drill Date: 5/5/16
Borehole Size: 4 inches
Hammer Type: Auto Trip.
Weight & Drop: 140lbs./30in.

SALEM

Percolation Test Worksheet

Project: **Proposed Commercial Development**
NWC N. Sanderson Ave & Cottonwood Ave
San Jacinto, CA

Job No.: **3-216-0492**
Date Drilled: **5/5/2016**
Soil Classification: **Sandy SILT (ML) with clay**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Totoal Depth of Hole: 96 in.

Test Hole No.: **P-1**
Tested by: **SK**
Drilled Hole Depth: **8** ft.

Presoaking Date: **5/5/2016**
Test Date: **5/6/2016**

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
9:55	10:25	8.0	Y	0:30	5.01	5.18	2.04	30	14.7	35.9	33.8	34.9	0.22
10:25	10:55	8.0	N	0:30	5.18	5.32	1.68	30	17.9	33.8	32.2	33.0	0.19
10:55	11:25	8.0	N	0:30	5.32	5.44	1.44	30	20.8	32.2	30.7	31.4	0.17
11:25	11:55	8.0	N	0:30	5.44	5.56	1.44	30	20.8	30.7	29.3	30.0	0.18
11:55	12:25	8.0	N	0:30	5.56	5.67	1.32	30	22.7	29.3	28.0	28.6	0.17
12:25	12:55	8.0	N	0:30	5.67	5.76	1.08	30	27.8	28.0	26.9	27.4	0.15
12:55	13:25	8.0	N	0:30	5.76	5.85	1.08	30	27.8	26.9	25.8	26.3	0.15
13:25	13:55	8.0	N	0:30	5.85	5.93	0.96	30	31.3	25.8	24.8	25.3	0.14
13:55	14:25	8.0	N	0:30	5.93	6.01	0.96	30	31.3	24.8	23.9	24.4	0.15
14:25	14:55	8.0	N	0:30	6.01	6.08	0.84	30	35.7	23.9	23.0	23.5	0.13
14:55	15:25	8.0	N	0:30	6.08	6.15	0.84	30	35.7	23.0	22.2	22.6	0.14
15:25	15:55	8.0	N	0:30	6.15	6.22	0.84	30	35.7	22.2	21.4	21.8	0.14
Recommended for Design:										**Infiltration Rate**			**0.13**

SALEM engineering group, inc.

Percolation Test Worksheet

Project: **Proposed Commercial Development**
NWC N. Sanderson Ave & Cottonwood Ave
San Jacinto, CA

Job No.: **3-216-0492**
Date Drilled: **5/5/2016**
Soil Classification: **Silty SAND (SM)**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Totoal Depth of Hole: 96 in.

Test Hole No.: **P-2**
Tested by: **SK**
Drilled Hole Depth: **8** ft.

Presoaking Date: **5/5/2016**
Test Date: **5/6/2016**

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill-Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
9:52	10:17	8.0	Y	0:25	5.26	5.86	7.20	25	3.5	32.9	25.7	29.3	1.10
10:18	10:43	8.0	Y	0:25	4.89	5.47	6.96	25	3.6	37.3	30.4	33.8	0.93
10:52	11:02	8.0	Y	0:10	4.30	4.57	3.24	10	3.1	44.4	41.2	42.8	0.87
11:02	11:12	8.0	N	0:10	4.57	4.81	2.88	10	3.5	41.2	38.3	39.7	0.83
11:12	11:22	8.0	N	0:10	4.81	5.02	2.52	10	4.0	38.3	35.8	37.0	0.77
11:22	11:32	8.0	N	0:10	5.02	5.20	2.16	10	4.6	35.8	33.6	34.7	0.71
11:32	11:42	8.0	N	0:10	5.20	5.37	2.04	10	4.9	33.6	31.6	32.6	0.71
11:42	11:52	8.0	N	0:10	5.37	5.53	1.92	10	5.2	31.6	29.6	30.6	0.71
11:52	12:02	8.0	N	0:10	5.53	5.68	1.80	10	5.6	29.6	27.8	28.7	0.70
12:02	12:12	8.0	N	0:10	5.68	5.82	1.68	10	6.0	27.8	26.2	27.0	0.70
12:12	12:22	8.0	N	0:10	5.82	5.96	1.68	10	6.0	26.2	24.5	25.3	0.74
12:22	12:32	8.0	N	0:10	5.96	6.09	1.56	10	6.4	24.5	22.9	23.7	0.73
Recommended for Design:										**Infiltration Rate**			**0.70**

SALEM engineering group, inc.

Percolation Test Worksheet

Project: **Proposed Commercial Development**
NWC N. Sanderson Ave & Cottonwood Ave
San Jacinto, CA

Job No.: **3-216-0492**
Date Drilled: **5/5/2016**
Soil Classification: **Sandy SILT (ML) with trace clay**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Totoal Depth of Hole: 84 in.

Test Hole No.: **P-3**
Tested by: **SK**
Drilled Hole Depth: 7 ft.

Presoaking Date: **5/5/2016**
Test Date: **5/6/2016**

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill-Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
10:02	10:32	7.0	Y	0:30	3.13	3.34	2.52	30	11.9	46.4	43.9	45.2	0.21
10:32	11:02	7.0	N	0:30	3.34	3.54	2.40	30	12.5	43.9	41.5	42.7	0.21
11:02	11:32	7.0	N	0:30	3.54	3.71	2.04	30	14.7	41.5	39.5	40.5	0.19
11:32	12:02	7.0	N	0:30	3.71	3.87	1.92	30	15.6	39.5	37.6	38.5	0.19
12:02	12:32	7.0	N	0:30	3.87	4.02	1.80	30	16.7	37.6	35.8	36.7	0.19
12:32	13:02	7.0	N	0:30	4.02	4.16	1.68	30	17.9	35.8	34.1	34.9	0.18
13:02	13:32	7.0	N	0:30	4.16	4.29	1.56	30	19.2	34.1	32.5	33.3	0.18
13:32	14:02	7.0	N	0:30	4.29	4.41	1.44	30	20.8	32.5	31.1	31.8	0.17
14:02	14:32	7.0	N	0:30	4.41	4.53	1.44	30	20.8	31.1	29.6	30.4	0.18
14:32	15:02	7.0	N	0:30	4.53	4.64	1.32	30	22.7	29.6	28.3	29.0	0.17
15:02	15:32	7.0	N	0:30	4.64	4.75	1.32	30	22.7	28.3	27.0	27.7	0.18
15:32	16:02	7.0	N	0:30	4.75	4.85	1.20	30	25.0	27.0	25.8	26.4	0.17
Recommended for Design:										**Infiltration Rate**			**0.17**

SALEM engineering group, inc.

Percolation Test Worksheet

Project: **Proposed Commercial Development**
NWC N. Sanderson Ave & Cottonwood Ave
San Jacinto, CA

Job No.: **3-216-0492**
Date Drilled: **5/5/2016**
Soil Classification: **Sandy SILT (ML) with trace clay**

Hole Radius: 4 in.
Pipe Dia.: 3 in.
Totoal Depth of Hole: 60 in.

Test Hole No.: **P-4**
Tested by: **SK**
Drilled Hole Depth: **5** ft.

Presoaking Date: **5/5/2016**
Test Date: **5/6/2016**

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
10:04	10:34	5.0	Y	0:30	1.05	1.38	3.96	30	7.6	47.4	43.4	45.4	0.33
10:34	11:04	5.0	N	0:30	1.38	1.64	3.12	30	9.6	43.4	40.3	41.9	0.28
11:04	11:34	5.0	N	0:30	1.64	1.87	2.76	30	10.9	40.3	37.6	38.9	0.27
11:34	12:04	5.0	N	0:30	1.87	2.08	2.52	30	11.9	37.6	35.0	36.3	0.26
12:04	12:34	5.0	N	0:30	2.08	2.28	2.40	30	12.5	35.0	32.6	33.8	0.27
12:34	13:04	5.0	N	0:30	2.28	2.46	2.16	30	13.9	32.6	30.5	31.6	0.26
13:04	13:34	5.0	N	0:30	2.46	2.62	1.92	30	15.6	30.5	28.6	29.5	0.24
13:34	14:04	5.0	N	0:30	2.62	2.77	1.80	30	16.7	28.6	26.8	27.7	0.24
14:04	14:34	5.0	N	0:30	2.77	2.91	1.68	30	17.9	26.8	25.1	25.9	0.24
14:34	15:04	5.0	N	0:30	2.91	3.04	1.56	30	19.2	25.1	23.5	24.3	0.24
15:04	15:34	5.0	N	0:30	3.04	3.16	1.44	30	20.8	23.5	22.1	22.8	0.23
15:34	16:04	5.0	N	0:30	3.16	3.28	1.44	30	20.8	22.1	20.6	21.4	0.25
Recommended for Design:										**Infiltration Rate**			**0.23**

SALEM engineering group, inc.

DRY SETTLEMENT DUE TO EARTHQUAKE SHAKING

Job No. 3-216-0492 **Job Name** Proposed Commercial Development
Boring No. B-3 **Drill Date** 05/05/2016

* Use Fig. 11 of Tokimatsu & Seed (1987)
** Use Fig. 13 of Tokimatsu & Seed (1987)
*** $MSF=10^{2.24}/Mw^{2.56}$
\# $C_N=2.2/(1.2+\sigma'_o/P_a)$

User Input Section

Earthquake Data			
Mag. (M_w)	7.9	Drilling GW Depth (ft)	50
a_{max}/g	0.93	Earthquake GW Depth (ft)	50
MSF***	0.88	Rod Stick-Up (ft)	3

SPT N-Value Correction Factors

Energy Ratio	C_E	1.60	Notes
Borehole Dia.	C_B	1.15	Notes
Sampling Method	C_S	1.2	Notes
Factor of Safety	FS	1.0	
Rod Length	C_R	Calculated	
Overburden Press	C_N	Calculated	

Lookup Tables

% Fines	ΔN	Length	C_R
0	0	1	0.75
10	1	12	0.85
25	2	20	0.95
50	4	30	0.98
75	5	33	1

13.64

Depth (ft)	USCS	Dry Unit Wt (pcf)	w (%)	Fines %	SPT Field N	Layer (ft)	Unit Wt (pcf)	Total σ_o bottom (psf)	Total σ_o mid-pt. (psf)	During Drilling Eff. σ'_o (psf)	C_N#	SPT $(N_1)_{60}$	ΔN	Fines Corct'd SPT $(N_1)_{60f}$	During EQ Eff. σ'_{oeq} (psf)	$\sigma_o/\sigma_{o'eq}$	r_d	Shear Modulus G_{max} ##	Cyclic Shear Stress T_{av}	Eff. Shear Strain γ(%)*	Vol. Strain (1-way) V%**	Vol. Strain Mw Corct'd V%*	S (2-way) in.
2	SM/ML	116	14.2	45.7	5	2.0	132.5	265	132	132	1.74	14.4	2.0	16.4	132	1.000	0.997	4.71E+05	79.9	2.6E-01	3.2E-1	0.35	0.17
5	SM	110	3.8	19.0	9	3.0	114.2	607	436	436	1.55	23.1	1.0	24.1	436	1.000	0.990	9.73E+05	261.1	3.5E-01	2.5E-1	0.28	0.20
10	SM/SP	115	4.2	10.5	14	5.0	119.8	1207	907	907	1.33	35.0	1.0	36.0	907	1.000	0.979	1.60E+06	536.8	3.1E-01	1.3E-1	0.15	0.17
15	ML	115	20.6	71.2	7	5.0	138.7	1900	1553	1553	1.11	14.6	4.0	18.6	1553	1.000	0.968	1.68E+06	909.2	1.4E+00	1.4E+0	0.00	0.00
20	ML	115	23.2	81.4	16	5.0	141.7	2608	2254	2254	0.95	31.7	5.0	36.7	2254	1.000	0.956	2.54E+06	1303.3	5.6E-01	2.3E-1	0.00	0.00
25	SM	115	10.4	39.8	35	5.0	127.0	3243	2926	2926	0.83	60.7	2.0	62.7	2926	1.000	0.941	3.46E+06	1664.3	3.0E-01	6.0E-2	0.07	0.08
30	SM/SP	115	4.1	9.5	26	5.0	119.7	3842	3543	3543	0.74	42.5	0.0	42.5	3543	1.000	0.919	3.35E+06	1968.3	5.3E-01	1.8E-1	0.20	0.24
35	SM/SP	115	3.4	6.6	33	5.0	118.9	4436	4139	4139	0.67	49.0	0.0	49.0	4139	1.000	0.888	3.80E+06	2222.6	4.3E-01	1.2E-1	0.13	0.16
40	SM/SP	115	2.0	6.8	36	5.0	117.3	5023	4730	4730	0.62	49.1	0.0	49.1	4730	1.000	0.848	4.06E+06	2423.6	3.9E-01	1.1E-1	0.12	0.14
45	SM	115	8.9	34.6	17	5.0	125.2	5649	5336	5336	0.57	21.3	2.0	23.3	5336	1.000	0.799	3.37E+06	2578.4	9.7E-01	7.2E-1	0.80	0.96
50	SM	115	8.9	34.6	17	5.0	125.2	6275	5962	5962	0.53	19.8	2.0	21.8	5962	1.000	0.748	3.48E+06	2695.5	8.7E-01	7.1E-1	0.79	0.94

The total seismic-induced settlement calculation is based on a water table depth of 50 feet below grade

Total 3.07

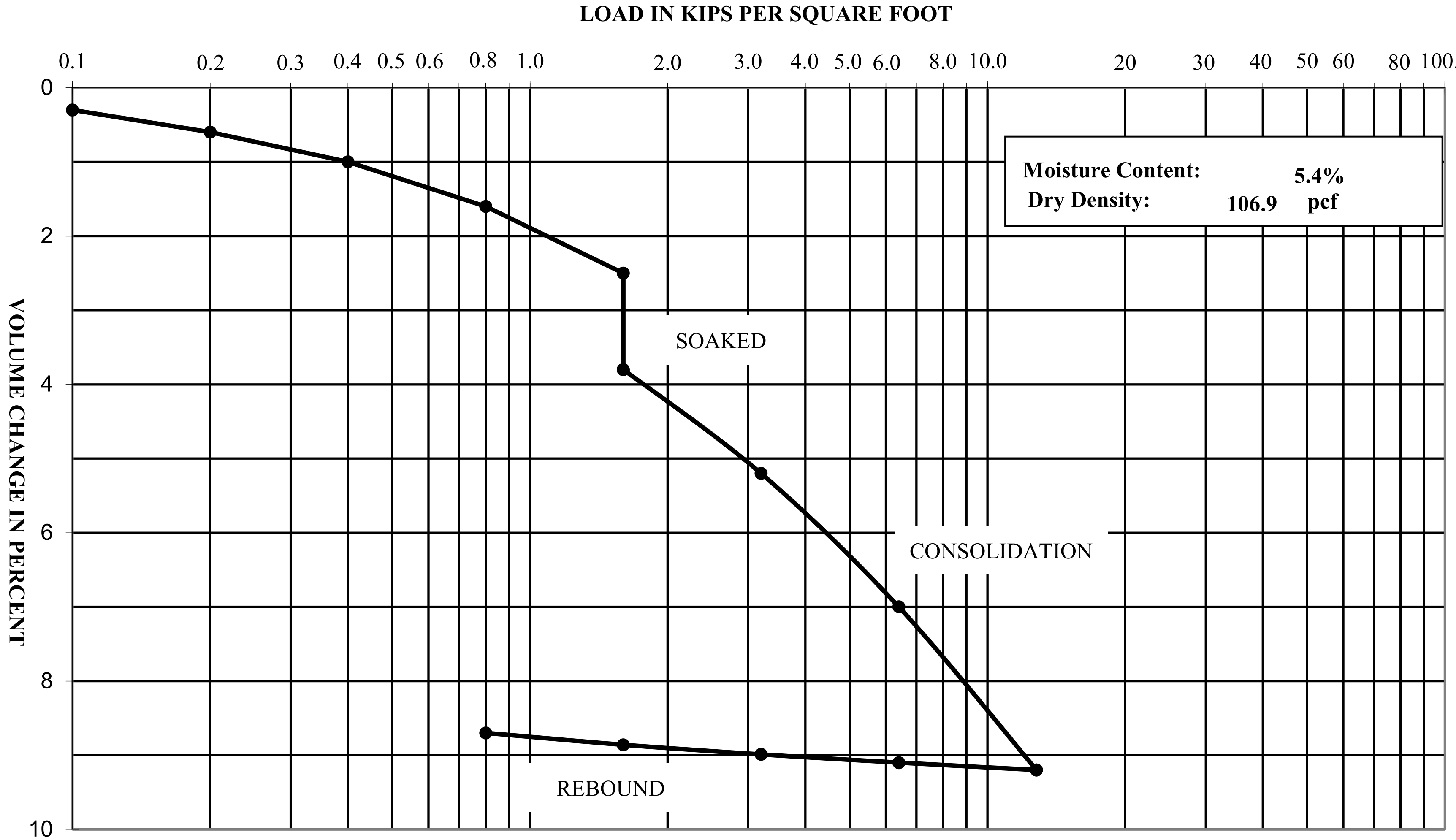
APPENDIX

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 D 2435



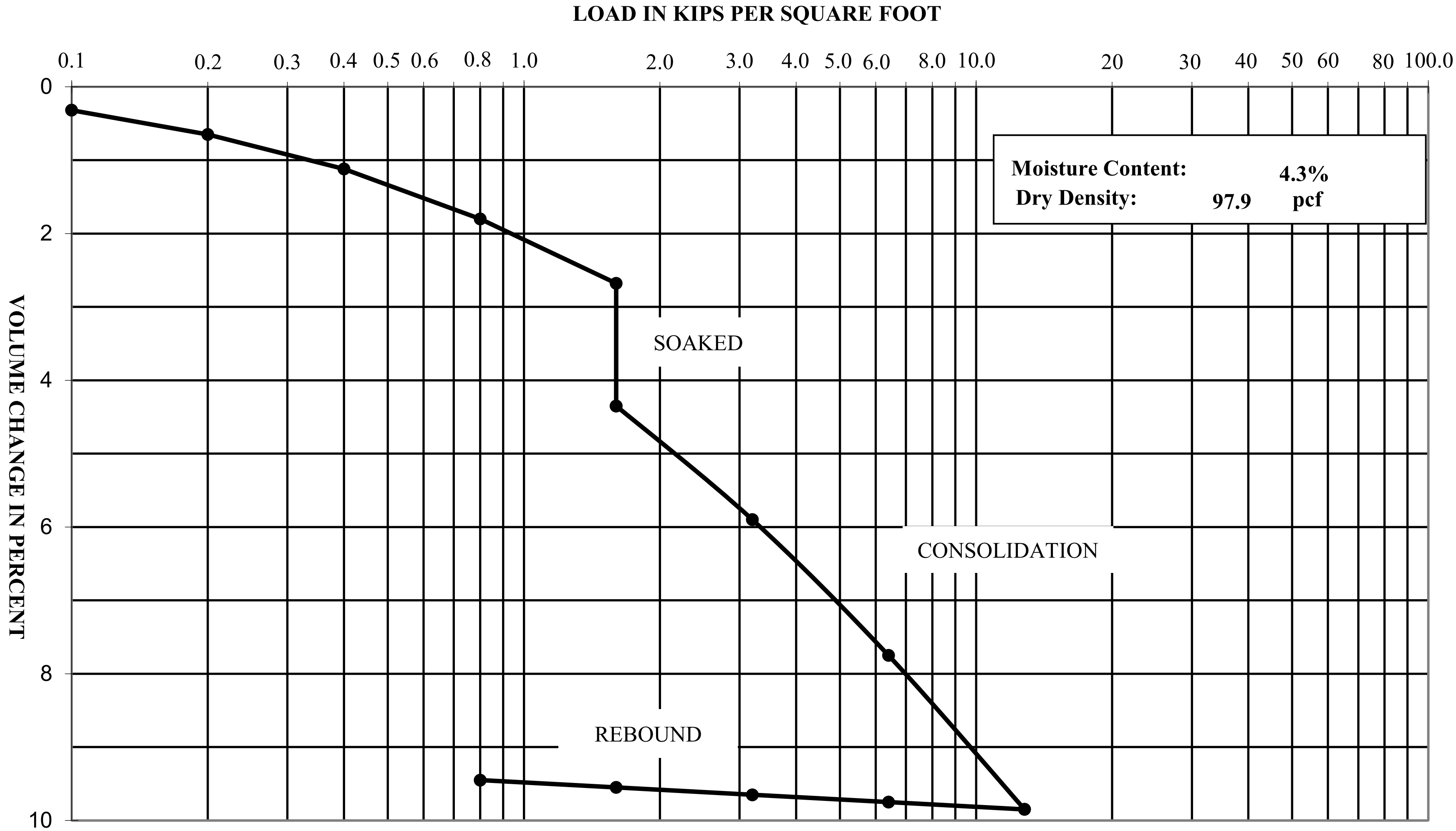
Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-1 @ 2'

SALEM engineering group, inc.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D 2435



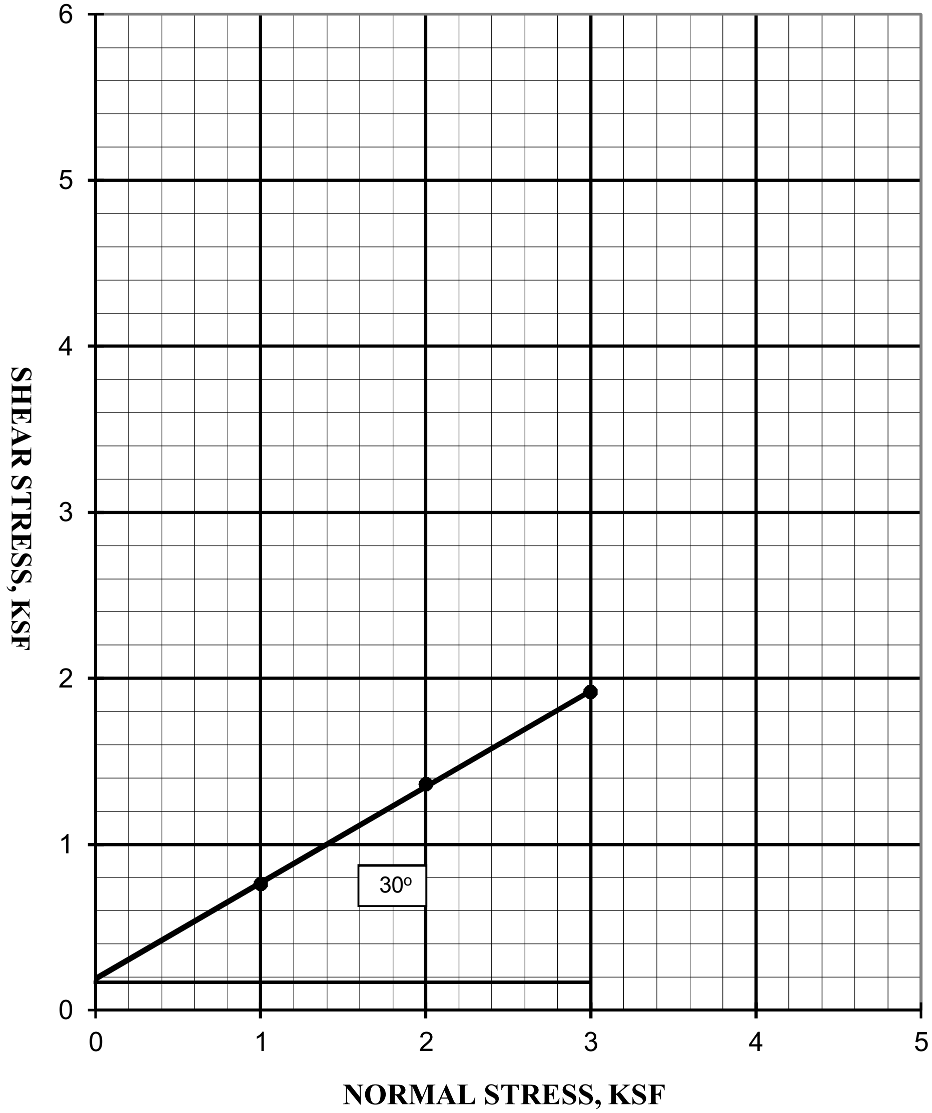
Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-2 @ 5'

SALEM engineering group, inc.

SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM D - 3080



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

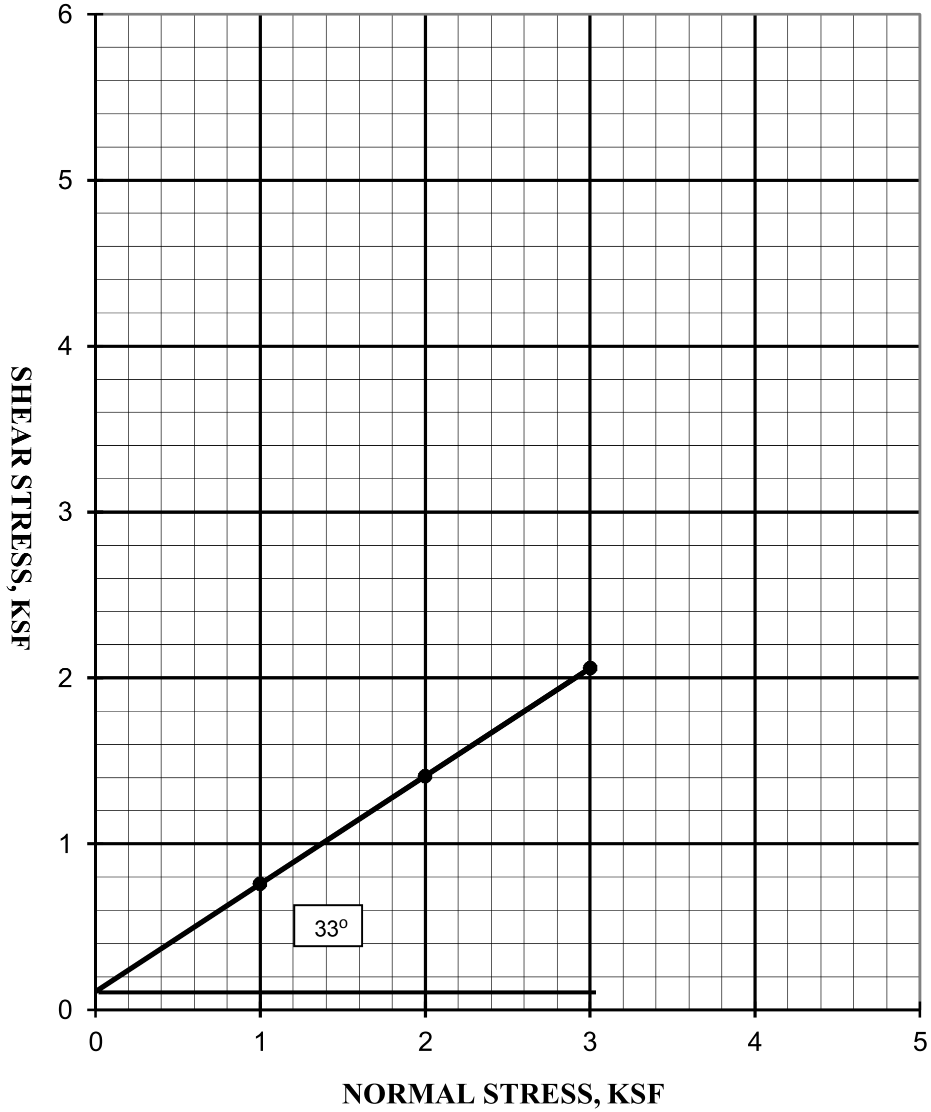
Boring: B-3 @ 2'
Soil Type: Silty SAND/Sandy SILT (SM/ML) with clay

Friction Angle:	**30.0**	**degrees**
Cohesion:	**190**	**psf**

Moisture Content	**14.2%**
Dry Density	**116.0 pcf**

SALEM engineering group, inc.

SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM D - 3080



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 5'
Soil Type: Silty SAND (SM)

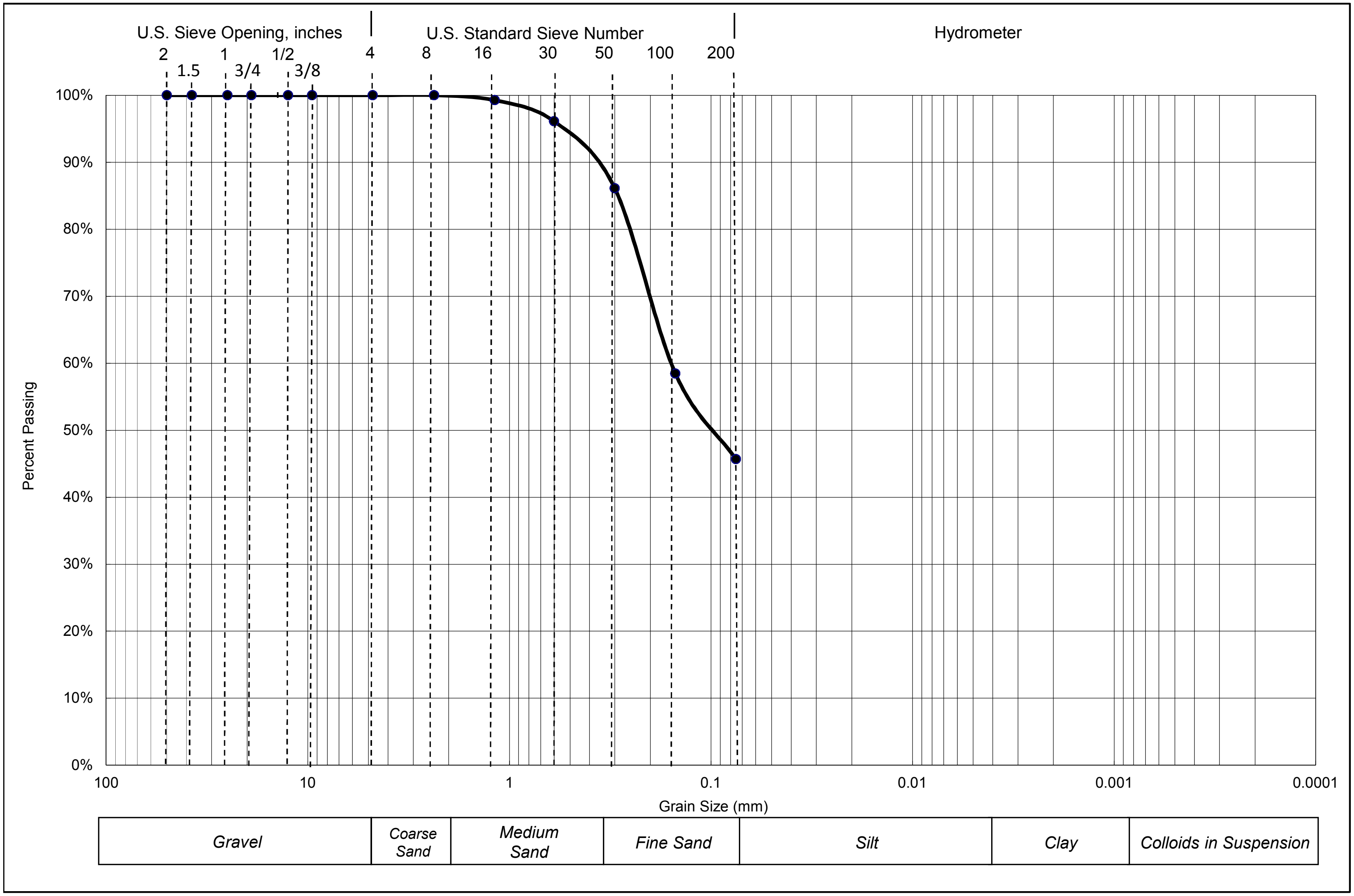
Friction Angle:	**33.0**	**degrees**
Cohesion:	**110**	**psf**

Moisture Content	**3.8%**
Dry Density	**109.3 pcf**

SALEM
engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 2'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

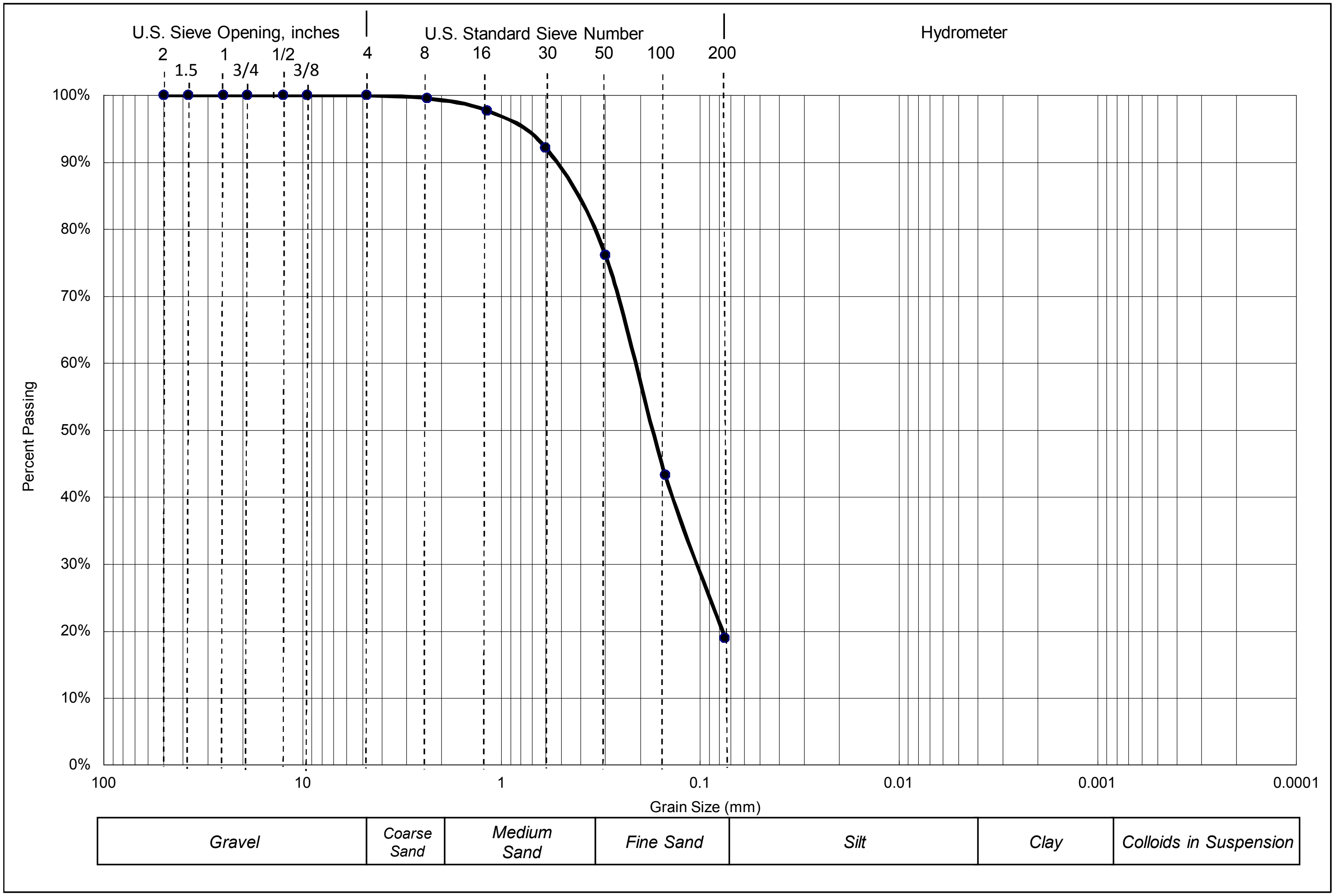
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	100.0%
No. 8	2.36	100.0%
No. 16	1.18	99.3%
No. 30	0.6	96.1%
No. 50	0.3	86.1%
No. 100	0.15	58.4%
No. 200	0.075	45.7%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 2'

SALEM
engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 5'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

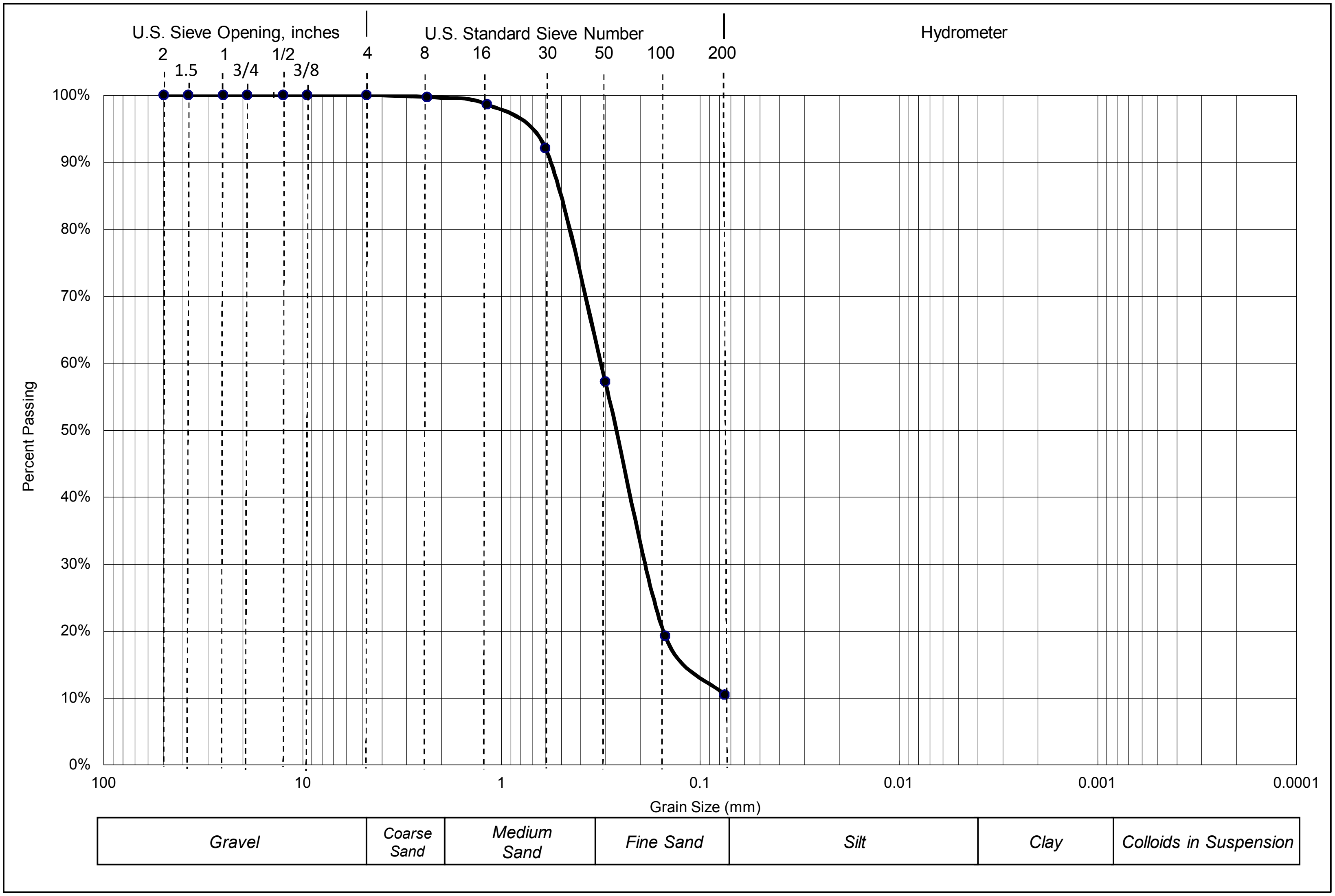
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	100.0%
No. 8	2.36	99.5%
No. 16	1.18	97.7%
No. 30	0.6	92.2%
No. 50	0.3	76.2%
No. 100	0.15	43.4%
No. 200	0.075	19.0%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 10'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

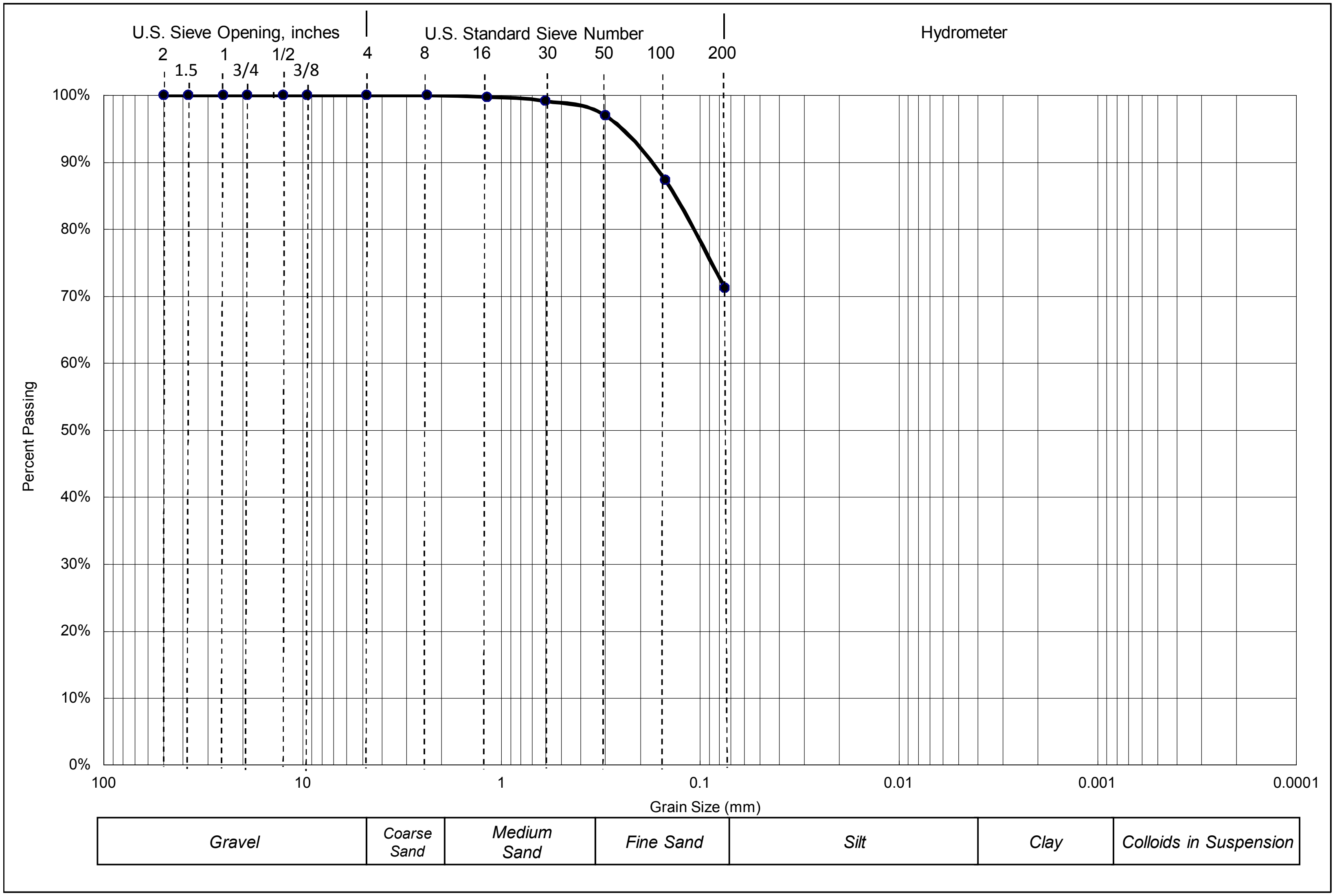
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	100.0%
No. 8	2.36	99.7%
No. 16	1.18	98.6%
No. 30	0.6	92.1%
No. 50	0.3	57.2%
No. 100	0.15	19.3%
No. 200	0.075	10.5%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 10'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 15'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

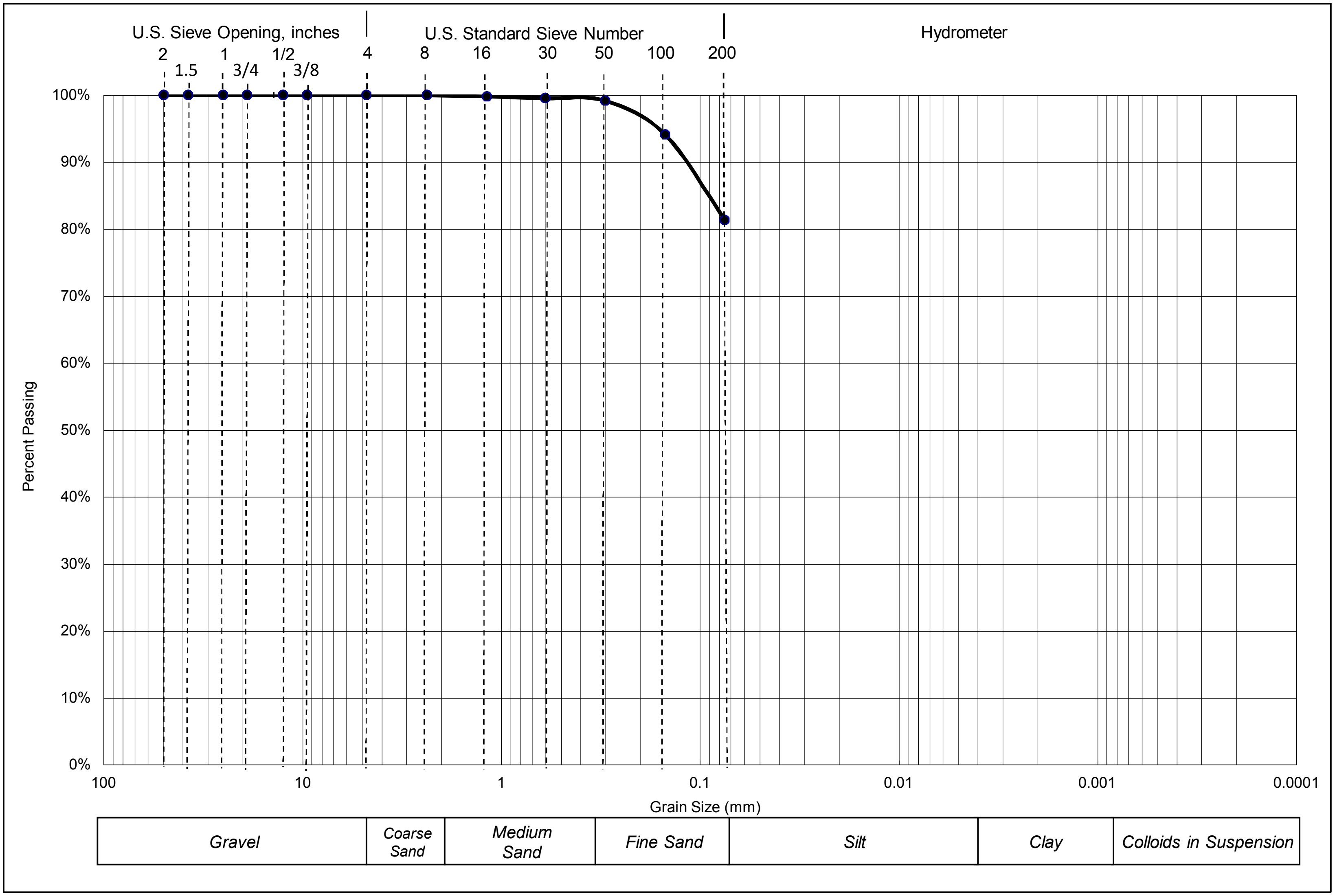
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	100.0%
No. 8	2.36	100.0%
No. 16	1.18	99.8%
No. 30	0.6	99.2%
No. 50	0.3	97.0%
No. 100	0.15	87.3%
No. 200	0.075	71.2%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 15'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 20'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

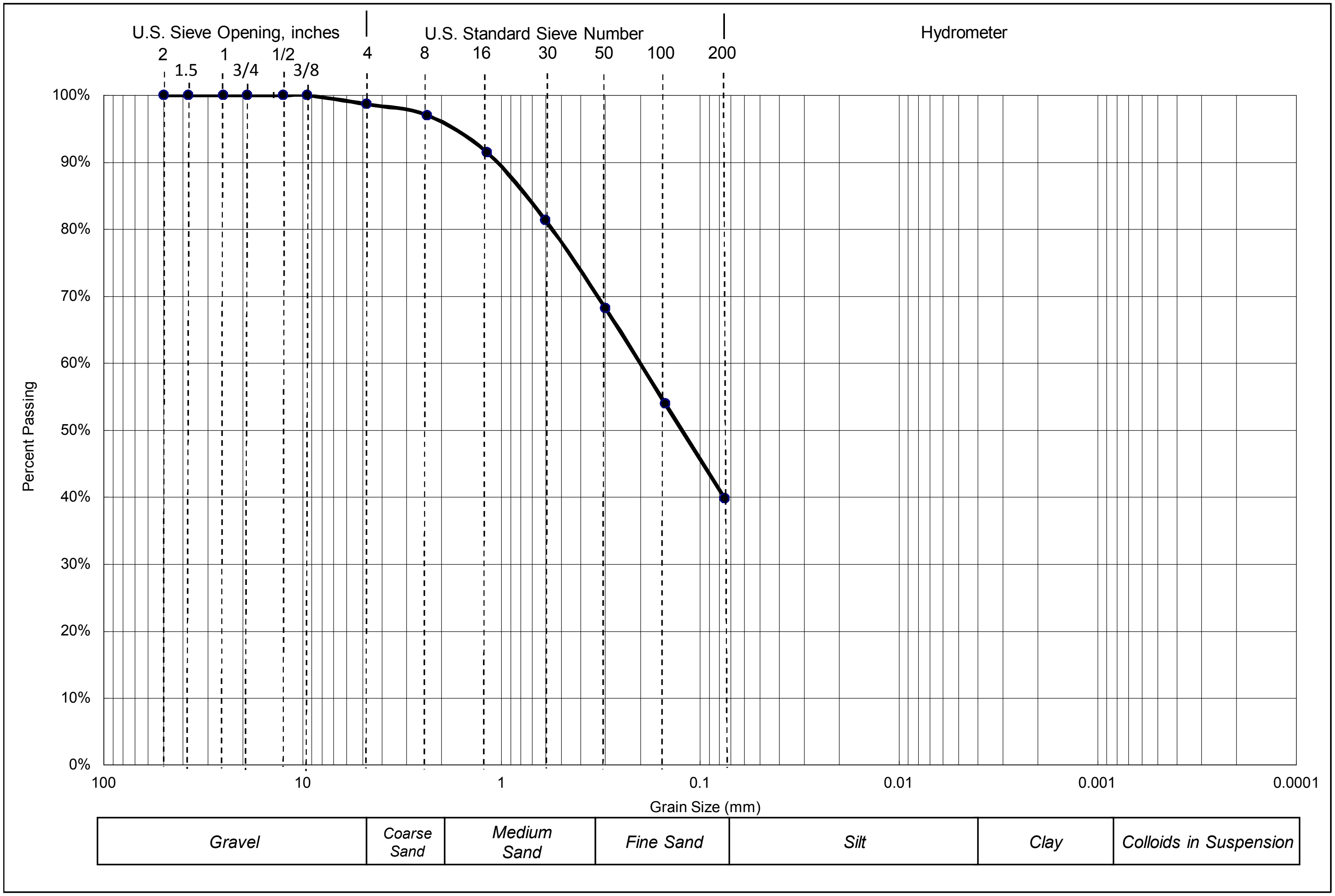
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	100.0%
No. 8	2.36	100.0%
No. 16	1.18	99.8%
No. 30	0.6	99.6%
No. 50	0.3	99.2%
No. 100	0.15	94.1%
No. 200	0.075	81.4%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 20'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 25'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

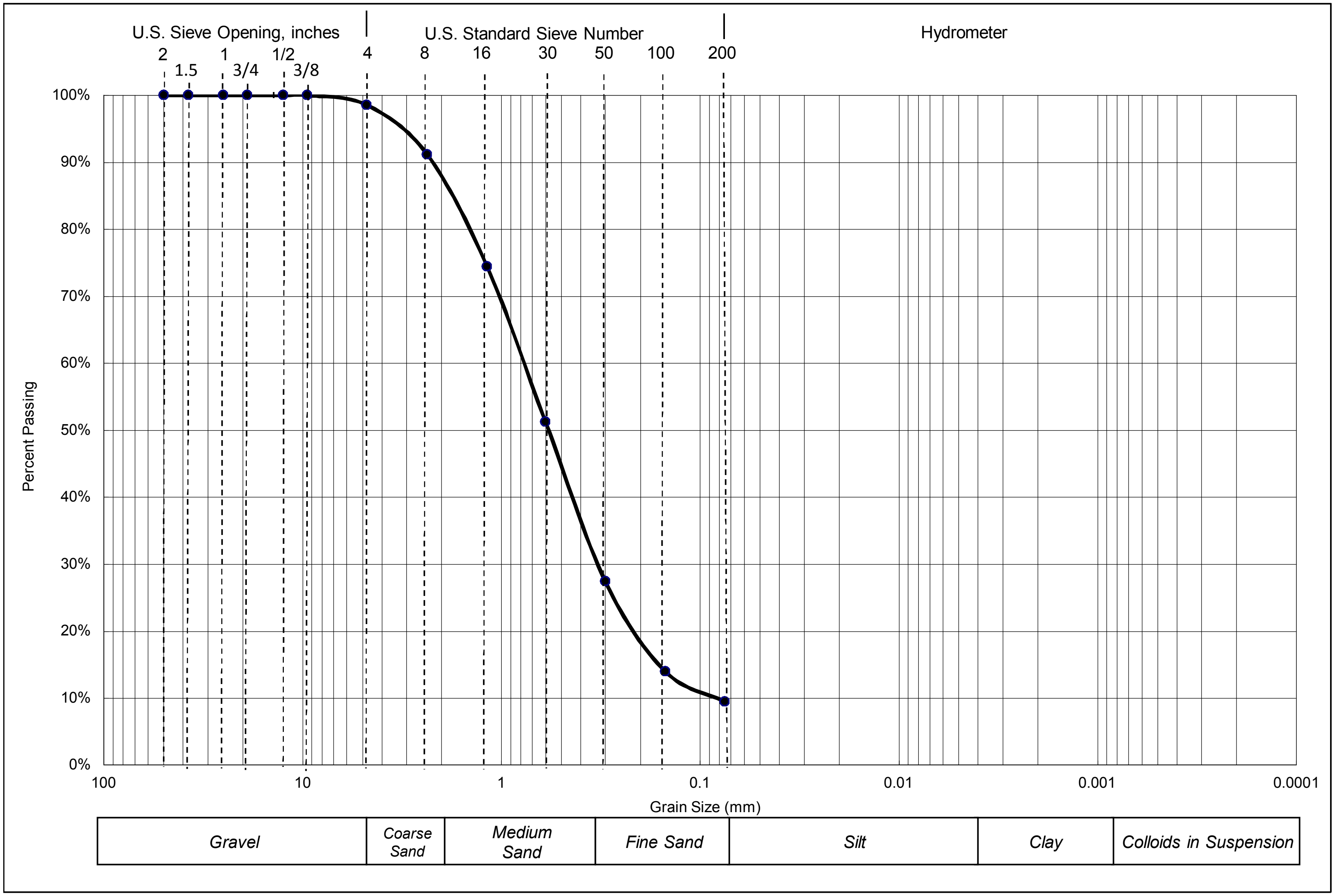
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	98.7%
No. 8	2.36	97.0%
No. 16	1.18	91.4%
No. 30	0.6	81.3%
No. 50	0.3	68.2%
No. 100	0.15	54.0%
No. 200	0.075	39.8%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 25'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 30'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	98.5%
No. 8	2.36	91.2%
No. 16	1.18	74.4%
No. 30	0.6	51.3%
No. 50	0.3	27.4%
No. 100	0.15	14.0%
No. 200	0.075	9.5%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

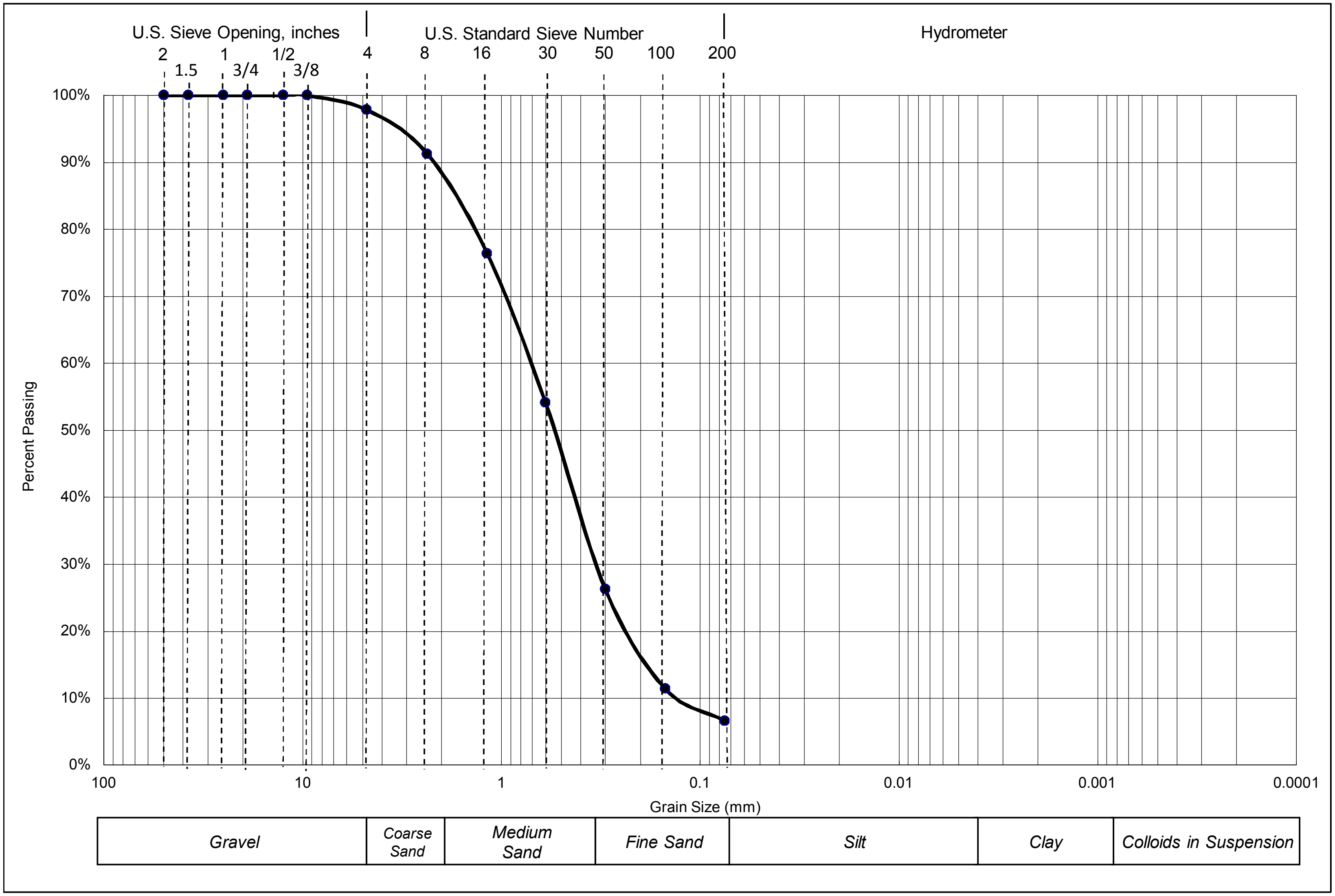
Project Number: 3-216-0492

Boring: B-3 @ 30'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 35'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

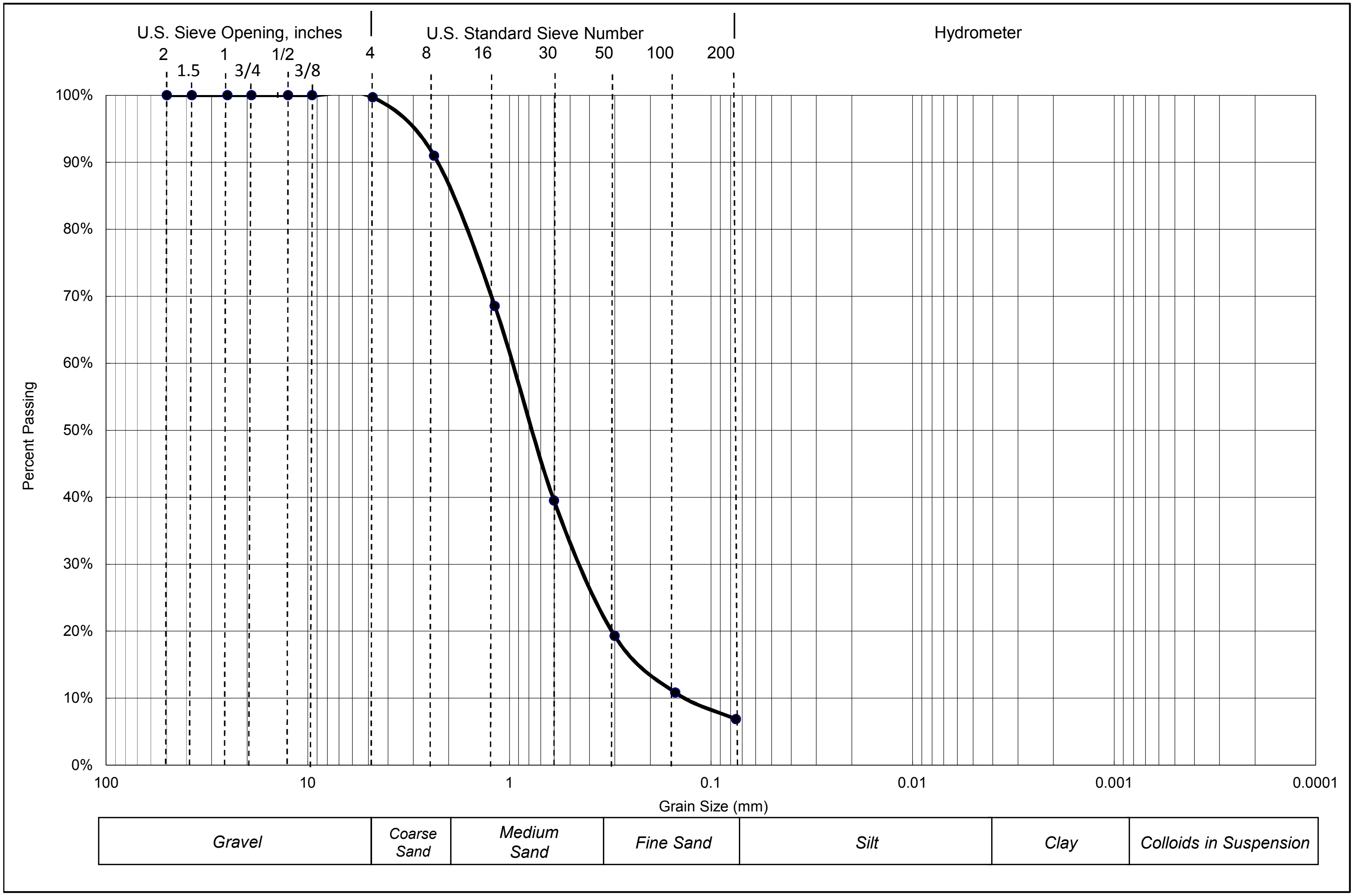
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	97.8%
No. 8	2.36	91.3%
No. 16	1.18	76.4%
No. 30	0.6	54.1%
No. 50	0.3	26.3%
No. 100	0.15	11.4%
No. 200	0.075	6.6%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 35'

SALEM
engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 40'

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

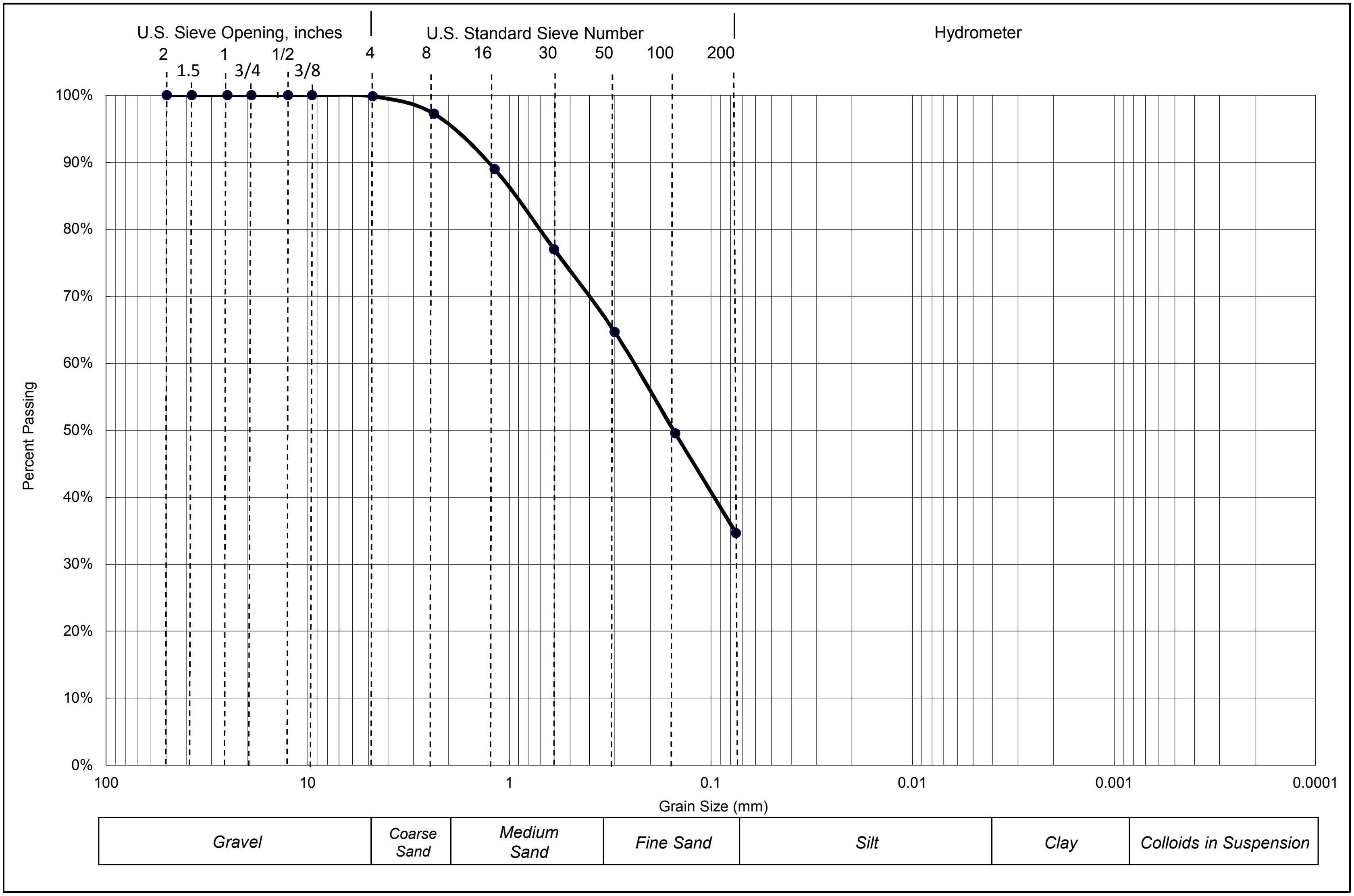
Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	99.7%
No. 8	2.36	91.0%
No. 16	1.18	68.5%
No. 30	0.6	39.5%
No. 50	0.3	19.3%
No. 100	0.15	10.8%
No. 200	0.075	6.8%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 40'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM D 422



Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA

Project Number: 3-216-0492

Boring: B-3 @ 45'

SALEM engineering group, inc.

DRY SIEVE ANALYSIS (ASTM D422 without Hydrometer)

Sieve Size	Particle Size, mm	Percent Passing
1 1/2-in.	37.5	100.0%
1-in.	25	100.0%
3/4-in.	19	100.0%
1/2-in.	12.5	100.0%
3/8-in.	9.5	100.0%
No. 4	4.75	99.8%
No. 8	2.36	97.2%
No. 16	1.18	88.9%
No. 30	0.6	77.0%
No. 50	0.3	64.7%
No. 100	0.15	49.5%
No. 200	0.075	34.6%

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Boring: B-3 @ 45'

SALEM
engineering group, inc.

EXPANSION INDEX TEST
ASTM D 4829 / UBC Std. 29-2

Project Number: 3-216-0492
Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Date Tested: 5/11/16
Sample location/ Depth: B-1 @ 0' - 3'
Sample Number: 1
Soil Classification: Silty SAND/Sandy SILT (SM/ML) with clay

Trial #	1	2	3
Weight of Soil & Mold, gms	570.6		
Weight of Mold, gms	186.7		
Weight of Soil, gms	383.9		
Wet Density, Lbs/cu.ft.	115.8		
Weight of Moisture Sample (Wet), gms	300.0		
Weight of Moisture Sample (Dry), gms	266.7		
Moisture Content, %	12.5		
Dry Density, Lbs/cu.ft.	102.9		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	52.9		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.034

Expansion Index $_{measured}$ = 34
Expansion Index $_{50}$ = 35.7

Expansion Index = 36

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

SALEM engineering group, inc.

CHEMICAL ANALYSIS

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

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Date: 5/11/16
Soil Classification: Silty SAND/Sandy SILT (SM/ML) with clay

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 0' - 3'	50 mg/Kg	46 mg/Kg	7.1
1b.	B-1 @ 0' - 3'	50 mg/Kg	46 mg/Kg	7.1
1c.	B-1 @ 0' - 3'	50 mg/Kg	46 mg/Kg	7.1
Average:		**50 mg/Kg**	**46 mg/Kg**	**7.1**

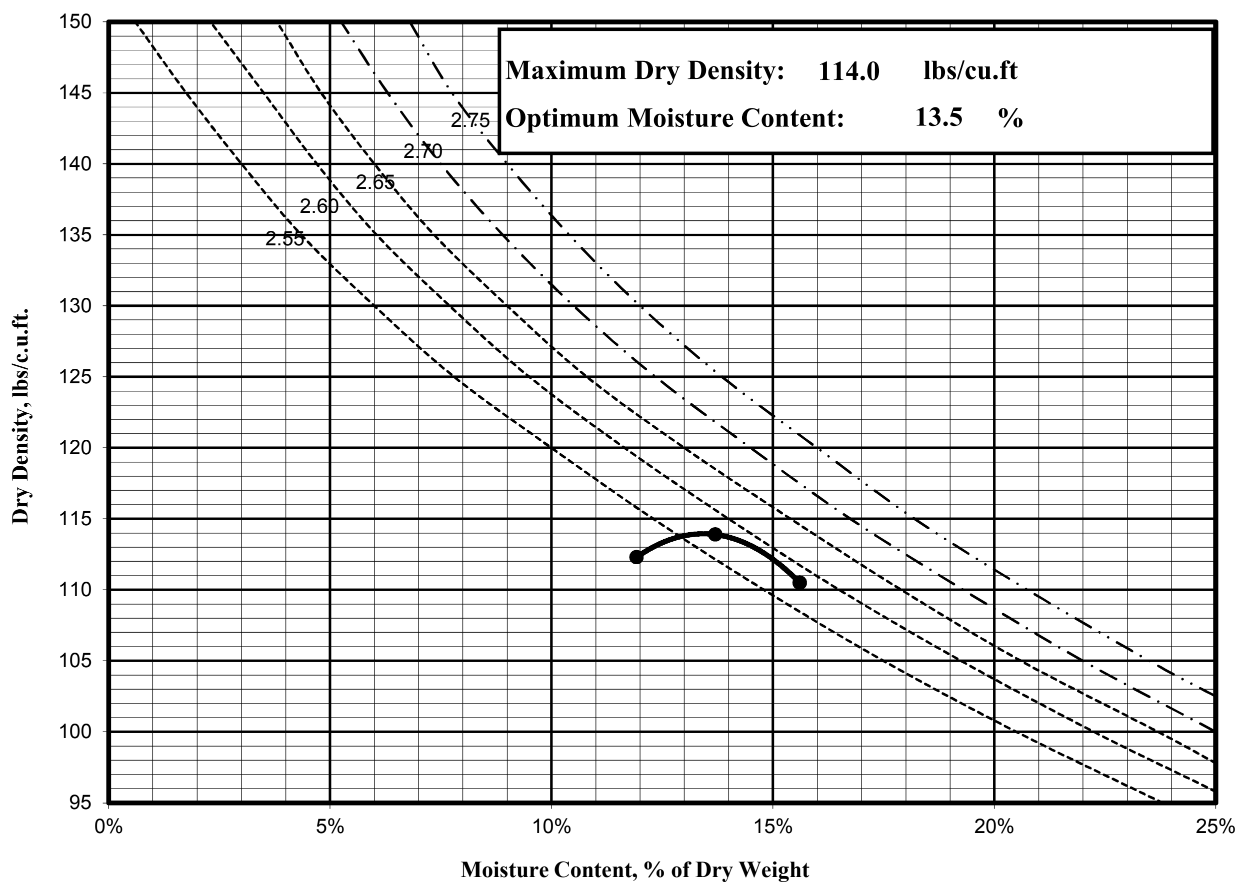
SALEM
engineering group, inc.

LABORATORY COMPACTION CURVE

ASTM - D1557, D698

Proposed NWC N. Sanderson Avenue and Cottonwood Avenue, San Jacinto, CA
Project Number: 3-216-0492
Date Tested: 5/11/16
Sample Location: B-1 @ 0' - 3'
Soil Classification: Silty SAND/Sandy SILT (SM/ML) with clay
Sample/Curve Number: 1
Test Method: 1557 A

	1	2	3
Weight of Moist Specimen & Mold, gm	4187.9	4214.2	4156.6
Weight of Compaction Mold, gm	2258.2	2258.2	2258.2
Weight of Moist Specimen, gm	1929.7	1956.0	1898.4
Volume of mold, cu. ft.	0.0333	0.0333	0.0333
Wet Density, lbs/cu.ft.	127.8	129.5	125.7
Weight of Wet (Moisture) Sample, gm	200.0	200.0	200.0
Weight of Dry (Moisture) Sample, gm	173.0	175.9	178.7
Moisture Content, %	15.6%	13.7%	11.9%
Dry Density, lbs/cu.ft.	110.5	113.9	112.3



SALEM engineering group, inc.

APPENDIX

C

APPENDIX C
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

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

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

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

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

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

3.0 TECHNICAL REQUIREMENTS: All compacted materials shall be densified to no less that 95 percent of relative compaction (90 percent for 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% (90% for cohesive soil) based upon ASTM D1557. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

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

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

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