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SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED WALMART PETROLEUM STATION
STORE NO. 5156
1540 EAST 2ND STREET
BEAUMONT, CALIFORNIA

SALEM PROJECT NO. 3-220-0783
OCTOBER 19, 2020

PREPARED FOR:

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Mr. John P. DeGunya
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SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED WALMART PETROLEUM STATION
STORE NO. 5156
1540 EAST 2ND STREET
BEAUMONT, CALIFORNIA

Dear Mr. DeGunya:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation report for the Proposed Walmart Petroleum Station 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 (559) 271-9700.

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
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BEAUMONT, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering Investigation for the Proposed Walmart Petroleum Station to be located at 1540 East 2nd Street in Beaumont, California (see Figure 1, Vicinity Map).

SALEM Engineering Group, Inc. (SALEM) has completed this geotechnical engineering investigation with the purpose to observe and sample the subsurface conditions encountered at the site, and provide conclusions and recommendations relative to the geotechnical aspects of constructing the project as presently proposed. The recommendations presented herein are based on analysis of the data obtained during the investigation and our local experience with similar soil and geologic conditions.

If project details vary significantly from those described herein, SALEM should be contacted to determine the necessity for review and possible revision of this report.

2. PROJECT DESCRIPTION

We understand that development of the site includes the construction of a Walmart Petroleum Station on an approximately 0.51 acre parcel area. The petroleum station will include a 1,440 square-foot associated attendant building, new fuel canopies with fuel dispensers, and underground storage tanks. Parking areas, driveways, and landscaping are expected to be associated with the development.

The proposed attendant building is anticipated to include either wood framed or CMU wall construction, with slabs on grade and conventional shallow spread foundations. We also understand that the proposed fueling canopy will including 8 fuel dispensers. At the time of this report, precise foundation loads were unknown. However, based on our experience, maximum column loads of about 30 kips and wall bearing loads of 2 to 3 kips per linear feet are anticipated.

Based on the existing slightly sloping grades to the south at the project site during our field exploration, it is anticipated that cuts and fills will be minimal and limited to providing building pads and positive site drainage. In the event that changes occur in the nature or design of the project, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions of our report are modified. The site location and approximate locations of proposed improvements are shown on the Site Plan, Figure 2.

3. SITE LOCATION AND DESCRIPTION

The subject site is located within an existing Walmart Shopping Center at 1540 East 2nd Street in Beaumont, California (see Vicinity Map, Figure 1). The Walmart Shopping Center parcel is irregular in shape, with Riverside County parcel number 419-260-081

At the time of our field exploration, the site area of the proposed fuel station and new pavements consisted of a parking lot that is part of the Walmart Shopping Center. The site was covered with asphaltic concrete pavements with isolated landscape areas, and overhead lighting. Mature trees were noted within the landscape island areas. Existing underground utilities were noted around the perimeter of the site and within the vicinity of the proposed fueling station.

The site is bounded by Walmart parking to the north and west, a commercial building to the east, and 2nd Street Marketplace to the south. The site area slightly sloping to the south. Based on review of Google Earth aerial imagery, site elevation ranges from approximately 2568 to 2575 feet above mean sea level (AMSL).

4. SITE HISTORY

Based on review of available on-line aerial imagery, since at least 1996, the site area was an agriculture land. In 2005, the western portion of the site appeared to be developed as an asphalt-paved parking lot, and the eastern portion of the site appeared as a staging area for the on-going development. In 2006, the eastern portion of the site appeared to be developed as an asphalt-paved parking lot as well. The site remained unchanged since 2006.

5. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-3) were drilled on September 23, 2020 within or near the proposed fueling station and parking lot areas to the depths ranging from 12 feet to the maximum depth explored of 16½ feet below existing grade (BEG). The depth of drilling was limited due to auger refusal on the dense soil. The approximate locations of the test borings are shown on Figure No. 2, Site Plan. The test borings were advanced with 6-inch hollow stem augers and 4-inch solid flight augers rotated by a truck-mounted CME-45C drill rig.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer at that time. Visual classification of the materials encountered in the test borings was generally made in accordance with the Unified Soil Classification System (ASTM D2488).

A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix "A." The test boring logs are presented in Appendix "A." Subsurface soil samples were obtained by driving a Modified California sampler (MCS) or a Standard Penetration Test (SPT) sampler.

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

Soil samples were obtained from the test borings at the depths shown on the boring logs. The MCS samples were recovered and capped at both ends to preserve the samples at their natural moisture content; SPT samples were recovered and placed in a sealed bag to preserve their natural moisture content. At the completion of drilling and sampling, the test borings were backfilled with soil cuttings and patched with cold asphalt.

6. 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, resistance value (R-value), organic content, expansion index, gradation, and maximum density and optimum moisture content of the materials encountered.

In addition, chemical tests and resistivity 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.

7. SOIL AND GROUNDWATER CONDITIONS

7.1 Subsurface Soil Conditions

Approximately 4 to 6 inches of asphaltic concrete underlain by approximately 2 to 3 inches of aggregate base material was encountered at our test boring locations. The soils encountered below the existing pavements generally consisted of medium dense to very dense silty sand and sandy gravel; and stiff to hard sandy silt to the maximum depth explored of 16½ feet BEG. The subsurface conditions encountered appear typical of those found in the geologic region of the site.

One consolidation test was performed on a near surface sample, and resulted in an approximately 10 percent consolidation under a load of 16 kips per square foot. When wetted under a load of 2.0 kips per square foot, the samples exhibited a collapse of 2.5 percent, respectively. Two (2) direct shear tests performed on relatively undisturbed soil samples resulted in internal angles of friction of 33.1 and 32.5 degrees with cohesion values of 206 and 286 pounds per square foot, respectively. In addition, an expansion index test performed on a near surface soil sample resulted in an expansion index of 0. An R-value test performed on a near surface sample resulted in an R-value of 44. An organic content test was also performed to determine the organic content of the near surface soils and concluded that the near surface soils contained 1.8 percent organic matter, by weight.

Soil conditions described in the previous paragraphs are generalized. Therefore, the reader should consult exploratory boring logs included in Appendix A for soil type, color, moisture, consistency, and USCS classification of the materials encountered at specific locations and elevations.

7.2 Groundwater

The test borings were checked for the presence of groundwater during and after the excavation operations. Groundwater was not encountered during our investigation. The historically highest groundwater is anticipated to be deeper than 50 feet below existing grade, based on local groundwater well data. According to California Department of Water Resources State Well Number (03S01W12L001S) located approximately 0.8 miles southeast of the site, groundwater level was measured at a depth of approximately 116 to 122 feet below existing grade from 2011 to 2019.

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

7.3 Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate and chloride levels and how they relate to cement reactivity with soil and/or water. A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil samples was detected to be 113 mg/kg.

ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class. ACI 318 requirements for site concrete based upon soluble sulfate are summarized in Table 7.3 below.

TABLE 7.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Dissolved Sulfate (SO_4) in Soil % by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementitious Materials Type
0.0113	Negligible	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil sample tested is 14 mg/kg. In addition, testing performed on a near surface soil resulted in a minimum resistivity value of 3,017 ohm-centimeters. Based on the results, these soils would be considered to have a "corrosive" corrosion potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings).

It is recommended that, at a minimum, applicable manufacturer's recommendations for corrosion protection of buried metal pipe be closely followed. Corrosion is dependent upon a complex variety of conditions, which are beyond the Geotechnical practice. <u>Consequently, a qualified corrosion engineer should be consulted if the owner desires more specific recommendations</u>.

7.4 Percolation Testing

One percolation test (P-1) was performed within assumed infiltration areas. The approximate location of the percolation test are shown on the attached Site Plan, Figure 2. An approximately 8-inch diameter borehole was advanced to the depth shown on the percolation test worksheet. The hole was pre-saturated before percolation testing commenced. Percolation rates were measured by filling the test holes with clean water and measuring the water drops at a certain time interval. The difference in the percolation rates are reflected by the varied type of soil materials at the bottom of the test hole. The percolation data are presented in tabular format at the end of this report. The test results are shown in the table below.

TABLE 7.4
PERCOLATION TEST RESULTS

Test No.	Depth (feet)	Field Percolation Rate (min/inch)	Unfactored Infiltration Rate* (inch/hour)	Soil Type**
P-1	10	62.5	**0.11**	Sandy SILT (ML)

*** Tested infiltration Rate = (ΔH 60 r) / (Δt(r + 2H_{avg})), with no factor of safety**
****At bottom of test hole**

<u>Based on the soil condition and percolation test results, the site is considered to be technically **infeasible** to attain an infiltration rate necessary to achieve reliable performance of infiltration or bioretention BMPs in retaining the stormwater quality design volume (SWQDV) on site.</u>

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) shall be applied to the tested infiltration rate for the final design infiltration rate. The FS should be determined by the civil engineer based on Worksheets/tables provided in the Design Handbook for Low Impact Development, Best Management Practices, prepared by Riverside County Flood Control.

The soils may also become less permeable to impermeable if the soil is compacted. Thus, periodic maintenance consisting of clearing the bottom of the drainage system of clogged soils should be expected.

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

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.

8. GEOLOGIC SETTING

The site is located within the central portion of the San Gorgonio pass within the northernmost portion of the Peninsular Ranges Geomorphic Province. The San Gorgonio Pass is a tectonic physiographic feature that separates the San Bernardino Mountains of the Transverse Ranges on the north and the San Jacinto Mountains on the south. The San Gorgonio Pass is expressed as a narrow notch that cuts through the mountains into the Colorado Desert to the east. Most of the site's vicinity is underlain by a thick sequence of terrestrial sediments that rest on the basement comprising igneous-metamorphic rocks. Younger alluvium occurs in active channels of San Timoteo Wash and tributary canyons, where the alluvium has been deposited on sediments of San Timoteo Formation. The near-surface deposits in the vicinity of the subject site are mapped as (Qf) alluvial fan of San Gorgonio Pass, sand and gravel of plutonic and gneissic detritus derived from rising San Bernardino mountains to the north, dissected at lower south margins. Deposits encountered on the subject site during exploratory drilling are discussed in detail in this report.

9. GEOLOGIC HAZARDS

9.1 Faulting and Seismicity

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

To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 33.9238° North; site longitude is 116.9508° West. The ten closest active faults are summarized below in Table 9.1.

TABLE 9.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
San Jacinto; SBV+SJV	6.2	7.4
San Jacinto; SBV+SJV+A+CC+B+SM	6.9	7.9
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO	7.3	8.2
San Jacinto; A+CC+B+SM	8.2	7.6
S. San Andreas; BG+CO	8.8	7.4
Pinto Mtn	16.0	7.3
San Jacinto; SBV	17.7	7.1
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB	20.1	8.0
Elsinore; W+GI	27.7	7.3
Elsinore; W+GI+T+J+CM	28.7	7.9

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.

9.2 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.

9.3 Ground Shaking

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

Based on the 2019 CBC, a Site Class D was selected for the site based on soil conditions with standard penetration resistance, N-values, averaging between 15 and 50 blows per foot. A table providing the recommended design acceleration parameters for the project site, based on a Site Class D designation, is included in Section 10.6.1 of this report.

Based on the Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.867g (based on both probabilistic and deterministic seismic ground motion).

9.4 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.

Below the existing pavements, the soils encountered generally consisted of medium dense to very dense silty sand and sandy gravel; and stiff to hard sandy silt to the maximum depth explored of 16½ feet BEG. Free groundwater was not encountered during our field exploration. The historically highest groundwater is estimated to be at a depth of more than 50 feet below existing grade.

The scope of work did not include a 50-foot deep test boring. A 50-foot test boring at a minimum, is required to evaluate the potential for liquefaction and seismic settlement. However, the potential for liquefaction at the site is considered to be low, due to the historically highest groundwater being at a depth of more than 50 feet below the existing grade. Additionally, the Riverside County GIS (Map My County) online tool shows the site located within a low liquefaction potential area.

9.5 Lateral Spreading

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

9.6 Landslides

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

9.7 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

10. CONCLUSIONS AND RECOMMENDATIONS

10.1 General Conclusions

10.1.1 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.

10.1.2 Below the existing pavements, the native soils encountered generally consisted of medium dense to very dense silty sand and sandy gravel; and stiff to hard sandy silt to the maximum depth explored of 16.5 feet BEG. Groundwater was not encountered during our field exploration.

10.1.3 The soils tested exhibited moderately compressibility and when wetted under a load of 2 kips per square foot exhibited a moderate collapse potential. The near surface soils exhibited a very low expansion potential. When compacted as engineered fill the soils encountered are anticipated to have a good pavement support characteristics.

10.1.4 Based on the subsurface conditions at the site, we assume that the proposed fueling station will be supported using conventional shallow foundations. However, the fuel canopies may be supported on either Cast in Drilled Hole (CIDH) or shallow spread foundations.

10.1.5 Provided the site is graded in accordance with the recommendations of this report and any new foundations constructed as described herein, we estimate that total settlement due to static loads utilizing conventional shallow foundations will be approximately 1 inch and the corresponding differential static settlement will be approximately ½ inch over a horizontal distance of 30 feet.

10.1.6 Based on the soluble sulfate testing performed on near surface samples, the near surface soils have 'negligible' potential for sulfate attack on concrete.

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

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

10.2 Surface Drainage

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

10.2.2 The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than 5 percent for a minimum distance of 10 feet. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building and drainage gradients maintained to carry all surface water to collection facilities and off site. These grades should be maintained for the life of the project. Ponding of water should not be allowed adjacent to the structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed.

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

10.3 Site Grading

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

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

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

10.3.4 Site preparation should begin with removal of existing surface/subsurface structures, pavements, underground utilities (as required), foundations, disturbed soil, any existing uncertified/undocumented fill, and debris. Underground utilities within the limits of the proposed building pad should be removed and relocated. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report.

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

10.3.6 Structural building pad areas and over-build zone should be considered as areas extending a minimum of 2.5 feet horizontally beyond the outside dimensions of buildings, including footings and non-cantilevered overhangs carrying structural loads.

10.3.7 To minimize post-construction soil movement provide uniform support for the proposed structure, it is recommended that over-excavation and recompaction within the proposed building area extend to at least 36 inches below preconstruction site grade, 24 inches below the bottom of proposed foundations, or to the depth required to remove any undocumented fills, whichever is greater. The resulting bottom of over-excavation shall be scarified to a minimum depth of at least 10 inches, worked until uniform and free from large clods, moisture-conditioned to near or slightly above optimum moisture content, and compacted to 92 percent of the maximum density. The horizontal limits of the over-excavation should extend throughout the building over-build zone, laterally to a minimum of 2.5 feet beyond the outer edges of the proposed footings.

10.3.8 Interior slabs on grade should be supported on a minimum of 4 inches of Class 2 aggregate base compacted to 95 percent relative compaction, over the depth of engineered fill extending to the depths recommended within the building pad per section 10.3.7 of this report.

10.3.9 Areas of exterior concrete slabs on grade located outside the building pad over-build zone, should be prepared by over-excavation to at least 12 inches below existing grade, 12 inches below the bottom of concrete slabs on grade, or the depth required to remove undocumented fills (if encountered), whichever is greater. The zone of over-excavation should extend a minimum of 2.5 feet beyond these improvements. These soils should be moisture conditioned to near or slightly above optimum moisture content and compacted as engineered fill. Exterior concrete slabs on grade should be supported on a minimum of 4 inches of Class 2 aggregate base compacted to 95 percent relative compaction over subgrade soils prepared as recommended above.

10.3.10 Areas of lightly loaded foundations such as retaining walls, screen walls, etc., should be prepared by over-excavation to a minimum of 1 foot below foundations, 1 foot below preconstruction site grade, or to the depth required to remove undocumented fills, whichever is greater. The resulting bottom of over-excavation shall be scarified to a depth of at least 10 inches, worked until uniform and free from large clods, moisture-conditioned to between to near or slightly above optimum moisture content, and compacted to 92 percent of the maximum density. The horizontal limits of the over-excavation should extend, laterally to a minimum of 2.5 feet beyond the outer edges of the proposed footings.

10.3.11 Areas of new asphaltic concrete or Portland cement concrete pavements should be prepared by over-excavation to the bottom of the recommended aggregate base section or the depth required to remove undocumented fills (if any) with deleterious material. The bottom of excavation should be scarified a minimum of 12 inches, moisture conditioned to near or slightly above optimum moisture content, and compacted to a minimum of 92 percent relative compaction. The horizontal limits of the over-excavation should extend, laterally to a minimum of 2.5 feet beyond pavements. The upper 12 inches below aggregate base sections should be compacted to a minimum of 95 percent relative compaction.

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

10.3.13 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.

10.3.14 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.

10.3.15 We do not anticipate groundwater or seepage to adversely affect construction if conducted during the drier months of the year (typically summer and fall). However, due to the shallow cemented hardpan soils, 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.

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

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

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

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

10.4 Soil and Excavation Characteristics

10.4.1 Based on the soil conditions encountered in our borings the on-site soils can be excavated with conventional heavy-duty excavation equipment.

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

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

10.4.4 The near surface soils identified as part of our investigation are, generally, 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.

10.5 Materials for Fill

10.5.1 On-site soils are considered suitable for use as general Engineered Fill in structural areas and depths greater than 6 inches below concrete slabs on grade, provided they do not have an expansion index greater than 20 (EI≤20) and do not contain deleterious matter, organic material, or material larger than 3 inches in maximum dimension.

10.5.2 Imported Non-Expansive Engineered Fill soil should be well-graded, very low-to-non-expansive, slightly cohesive silty sand or sandy silt. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 10.5.2.

TABLE 10.5.2
IMPORT NON-EXPANSIVE FILL REQUIREMENTS

Percent Passing 3-inch Sieve	100
Percent Passing No.4 Sieve	75-100
Percent Passing No 200 Sieve	15-40
Maximum Plasticity Index	15
Maximum Expansion Index (ASTM D4829)	20

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

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

10.5.5 On-Site soils used as engineered fill soils should moisture conditioned to slightly above optimum moisture content, and compacted to at least 92 percent relative compaction.

10.5.6 Import Non-Expansive Engineered Fill, should be placed, moisture conditioned to slightly above optimum moisture content, and compacted to at least 92 percent relative compaction.

10.5.7 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.

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

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

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

10.6 Seismic Design Criteria

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

TABLE 10.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2016 ASCE 7 or 2019 CBC Reference
Site Coordinates (Datum = NAD 83)		33.9238 Lat -116.9508 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}	1.1	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.867 g	ASCE 7 Equation 11.8-1
Seismic Design Category	SDC	D	ASCE 7 Table 11.6-1 & 2
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	1.935 g	CBC Figure 1613.3.1(1-6)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.661 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	***1.700**	CBC Table 1613.3.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	1.935 g	CBC Equation 16-37
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	***1.124 g**	CBC Equation 16-38
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.290 g	CBC Equation 16-39
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	***0.749 g**	CBC Equation 16-40
Short Term Transition Period (S_{D1}/S_{DS}), Seconds	T_S	0.581	ASCE 7-16, Section 11.4.6
Long Term Transition Period (seconds)	T_L	8	ASCE 7-16, Figure 22-14

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

10.6.2 Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, structures on Site Class D with S_1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific motion analysis may not be required based on

Exceptions listed in ASCE 11.4.8. The Structural Engineer should verify whether Exception No. 2 of ASCE 7-16, Section 11.4.8, is valid for the site. In the event that a site specific ground motion analysis is required, SALEM should be contacted for these services.

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

10.7 Shallow Foundations

10.7.1 The site is suitable for use of conventional shallow foundations consisting of continuous footings and isolated pad footings supported on engineered fill soils prepared in accordance with Section 10.3 of this report. Shallow foundations supported on engineered fill as recommended in this report may be designed based on total and differential static settlement of 1 inch and ½ inch in 30 feet, respectively.

10.7.2 The bearing wall footings considered for the structure should be continuous with a minimum width of 15 inches and extend to minimum depths of 12 inches below the lowest adjacent grade. Isolated column footings should have a minimum width of 12 inches and extend a minimum depth of 18 inches below the lowest adjacent grade. The bottom of footing excavations should be maintained free of loose and disturbed soil. Footing concrete should be placed into a neat excavation.

10.7.3 Shallow spread foundations supported engineered fill prepared in accordance with the recommendations provided in this report may be designed based on an allowable bearing pressure of 2,500 pounds per square foot. This value may be increased by 1/3 for short term wind and seismic loading.

10.7.4 Resistance to lateral footing displacement can be computed using a coefficient of friction factor of 0.30 acting between the base of foundations and engineered fill soils.

10.7.5 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 400 pounds per cubic foot acting against the appropriate vertical footing faces. The frictional and passive resistance of the soil may be combined provided that a 50% reduction of the frictional resistance factor is used in determining the total lateral resistance.

10.7.6 Foundation reinforcement should be determined by the Structural Engineer. At a 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.

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

10.7.8 The footing excavations should not be allowed to dry out any time prior to pouring concrete. 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.

10.8 Cast in Drilled Hole Pier Foundations for Canopies

10.8.1 Cast in Drilled Hole Pier foundations should have a minimum diameter of 24 inches and extend a minimum depth of 8 feet below the lowest adjacent grade. The contractor should be prepared to utilize special equipment if excessive resistance is encountered during drilling of the Cast in Drilled Hole Pier.

10.8.2 If groundwater is encountered, the shaft should be drilled with care, advancing the casing ahead of the auger and maintaining a water head inside the casing equal to (or higher) than the surrounding water table to limit the potential for drilled shaft hole collapse, when applicable.

10.8.3 Casing of the drilled pier will be required if groundwater, perched water, or caving is encountered, and/or the drilled hole has to be left open for an extended period of time. The casing should be bedded into the soil unit near the design depth prior to placement of the reinforcing steel and concrete, and casing extraction.

10.8.4 The total settlement of the drilled Cast in Drilled Hole Piers are not expected to exceed 1 inch and ½ inch differential between piers.

10.8.5 Skin friction within the upper 1 foot BEG should be neglected in design. The downward load capacity of the piers (extending to at least 8 feet BEG), may be designed based on an allowable skin friction value of 200 pounds per square foot. Provided the CIDH excavation is cleaned of loose soils, an allowable end bearing value of 5,000 pounds per square foot may be considered for design. These values may be increased by 1/3 for short duration wind and seismic loading.

10.8.6 Uplift loads can be resisted by drilled piers using 60 percent of the allowable downward side friction value plus the weight of the pier.

10.8.7 The drilled Cast in Drilled Hole Piers should be designed neglecting the lateral capacity within the upper 1 feet due to asphalt pavements. Below a depth of one (1) foot, the lateral capacity can be designed for 400 pounds per square foot per foot of depth below the lowest adjacent grade to a maximum of 4,000 pounds per square foot. 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 8 feet.

10.8.8 If desired, the drilled Cast in Drilled Hole Piers may be designed using LPILE. The upper 1 foot should be neglected in design. The soil parameters for LPILE lateral pile analysis are provided as follows:

TABLE 10.8.8
LPILE PARAMETERS

Depth (ft)	USCS Soil Type	Effective Unit Weight (pcf)	Angle of Internal Friction (degrees)	Undrained Shear Strength, Cohesion, (psf)	Coefficient of Variation of Lateral Subgrade Reaction*, f (pci)	Soil Strain Ratio, ε_{50}
1-5	ML	130	30	250	10	--
5-12	SM	130	33	180	25	--
12-20	ML	120	30	250	10	--

10.9 Interior Concrete Slabs-on-Grade

10.9.1 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 four (4) inches of class 2 aggregate base over the depth recommended below foundations.

10.9.2 We recommend reinforcing slabs, at a minimum, with No. 3 reinforcing bars placed 18 inches on center, each way. As an alternative, the use of welded wire or fiber mesh reinforcement may be considered by the Structural Engineer.

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

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

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

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

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

10.9.8 Avoid use of stakes driven through the vapor retarder.

10.9.9 In lieu of using a vapor retarder within areas where floor dampness reduction is not of a concern; the contractor may place a four (4) inch thick layer of 3/4-inch crushed rock. The crushed rock layer should be compacted to a condition that is firm and unyielding.

10.9.10 In lieu of using a vapor retarder within areas where floor dampness reduction is desired – as described in paragraph 10.9.6 – the concrete slab-on-grade should be waterproofed to minimize moisture vapor intrusion.

10.9.11 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 or loose 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.

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

10.10 Exterior Concrete Slabs on Grade

10.10.1 The following recommendations are intended for lightly loaded exterior slabs on grade not subject to vehicular traffic. Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that non-structural slabs-on-grade be at least 4 inches thick and underlain by four (4) inches of class 2 aggregate base over subgrade soils prepared in accordance with the recommendations in section 10.3 of this report.

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

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

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

10.11 Lateral Earth Pressures and Frictional Resistance

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

Lateral Pressure Conditions	Soil Equivalent Fluid Pressure
Active Pressure, Drained, pcf	33
At-Rest Pressure, Drained, pcf	50
Passive Pressure, pcf	400
Allowable Coefficient of Friction	0.30
Maximum Unit Weight (pcf) [γ_{max}]	135
Minimum Unit Weight (pcf) [γ_{min}]	105

10.11.2 Active pressure applies to walls, which are free to rotate. At-rest pressure applies to walls, which are restrained against rotation. The preceding lateral earth pressures assume sufficient drainage behind retaining walls to prevent the build-up of hydrostatic pressure. The top one-foot of adjacent subgrade should be deleted from the passive pressure computation.

10.11.3 The foregoing values of lateral earth pressures represent equivalent soil values and a safety factor consistent with the design conditions should be included in their usage.

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

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

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

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

Dynamic Seismic Lateral Loading Equation
Dynamic Seismic Lateral Load = $\frac{3}{8}\gamma K_h H^2$
Where: γ = Maximum In-Place Soil Density (Section 10.11.1 above)
K_h = Horizontal Acceleration = $\frac{2}{3}PGA_M$ (Section 10.6.1 above)
H = Wall Height

10.12 Retaining Walls

10.12.1 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 2 permeable materials graded in accordance with the current Caltrans Standard Specifications.

10.12.2 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.

10.12.3 Drainage pipes should be placed with perforations down and should discharge in a non-erosive manner away from foundations and other improvements.

10.12.4 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.

10.12.5 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.

10.12.6 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.

10.13 Temporary Excavations

10.13.1 We anticipate that the majority of the 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.

10.13.2 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.

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

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

RECOMMENDED EXCAVATION SLOPES

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

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

10.13.6 Braced shorings should be designed for a maximum pressure distribution of 22H, (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.

10.13.7 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.

10.14 Underground Utilities

10.14.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than 3 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches and compacted to at least 92 percent relative compaction at or slightly above optimum moisture content. The upper 12 inches of trench backfill within asphalt or concrete paved areas shall be moisture conditioned to near or slightly above optimum moisture content and compacted to at least 95 percent relative compaction.

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

10.14.3 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.

10.14.4 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.

10.15 Pavement Design

10.15.1 R-Value testing was performed on a sample collected from upper soils within test boring B-1. The sample was tested in accordance with the State of California Materials Manual Test Designation 301. R-Value testing on the near surface sample resulted in an R-value of 44. The pavement design recommendations provided herein are based on an R-value of 40. Using the ACPA Subgrade Resilient Modulus (M_{RSG}) Calculator from an R-Value of 40 the $M_{RSG\,=}$ 8,558 psi.

10.15.2 The following table includes pavement design recommendations based on Walmart Geotechnical Criteria and Caltrans Highway Design Manual (20 year design life), 18 kip Equivalent Single Axel Loads for Standard Duty Division 1 and Fuel Stations (ESAL=2,200) and Fuel Station Heavy Duty (ESAL= 18,000) pavements, a reliability factor of 85%, initial serviceability factor of 4.2, terminal serviceability of 2.0, an overall standard deviation of 0.45 for flexible pavements and 0.35 for rigid pavements.

TABLE 10.15.2
ASPHALT CONCRETE PAVEMENT THICKNESSES

Pavement Area	ESAL	Traffic Index	Asphaltic Concrete, Inches	Aggregate Base, Inches*	Compacted Subgrade, Inches*
Standard Duty (Fuel Station)	2,200	**4.5	3.0	4.0	12.0
Heavy Duty (Fuel Station)	18,000	5.5	3.0	5.0	12.0

****95% compaction based on ASTM D1557-07 Test Method***
*****Rounded to nearest ½***

10.15.3 The following recommendations are intended for Portland Cement Concrete pavement sections.

TABLE 10.15.3
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

Pavement Area	ESAL	Traffic Index	Portland Cement Concrete, (inches)*	Aggregate Base, (inches)**	Compacted Subgrade. (inches)**
Standard Duty (Fuel Station)	2,200	***4.5	4.0	4.0	12.0
Heavy Duty (Fuel Station)	18,000	5.5	4.0	4.0	12.0

*** Minimum Compressive Strength of 4,000 psi***
***** 95% compaction based on ASTM D1557-07 Test Method***
******Rounded to nearest ½***

10.15.4 Asphalt concrete should conform to Section 39 of Caltrans' latest Standard Specifications for ½ inch Hot Mix Asphalt (HMA) Type A or B. Asphaltic concrete pavements should be placed and compacted in accordance with Caltrans Standard Specifications.

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

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

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

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

11. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

11.1 Plan and Specification Review

11.1.1 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.

11.2 Construction Observation and Testing Services

11.2.1 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.

11.2.2 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.

11.2.3 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.

12. LIMITATIONS AND CHANGED CONDITIONS

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

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

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Ibrahim Foud Ibrahim, PE
Sr. Managing Engineer
RCE 86724

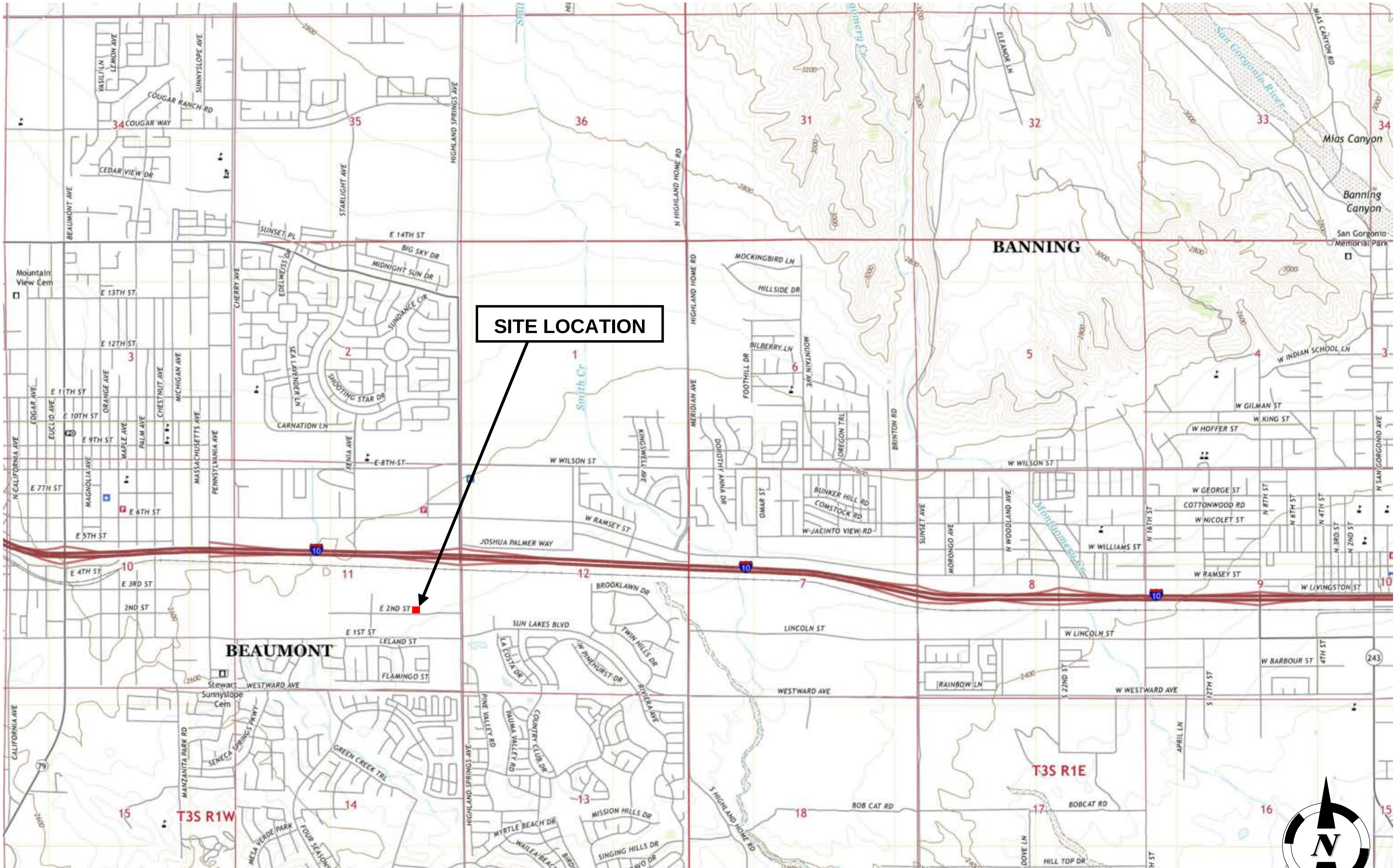
REGISTERED PROFESSIONAL ENGINEER
IBRAHIM FOUD IBRAHIM
C 86724
EXP. 03/31/21
CIVIL
STATE OF CALIFORNIA

Dean B. Ledgerwood II, CEG
Geotechnical Manager
CEG 2613

CERTIFIED ENGINEERING GEOLOGIST
Dean B. Ledgerwood II
No. 2613
Exp 12/31/21
STATE OF CALIFORNIA

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

REGISTERED PROFESSIONAL ENGINEER
R. SAMMY SALEM
GE 2549
EXP. 12-31-2020
GEOTECHNICAL
STATE OF CALIFORNIA



Source Image: U.S. Geological Survey, Beaumont, California, https://ngmdb.usgs.gov/topoview (2018)

VICINITY MAP **GEOTECHNICAL ENGINEERING INVESTIGATION** **Proposed Walmart Petroleum Station, Store #5156** **1540 East 2nd Street** **Beaumont, California**	SCALE: NOT TO SCALE DRAWN BY: JC PROJECT NO. 3-220-0783	DATE: 10/2020 APPROVED BY: II FIGURE NO. 1	SALEM engineering group, inc.

SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Walmart Petroleum Station, Store #5156
1540 East 2nd Street
Beaumont, California

SCALE:	DATE:
NOT TO SCALE	10/2020
DRAWN BY: JC	APPROVED BY: II
PROJECT NO. 3-220-0783	FIGURE NO. 2

LEGEND:

B-1 Soil Boring Locations
P-1 Percolation Locations

All Locations Approximate

SALEM engineering group, inc.

GEOTECHNICAL INVESTIGATION FACT SHEET

Include this form in the Geotechnical Report as an Appendix.

PROJECT LOCATION: 1540 East 2nd Street, Beaumont, California

Engineer: R. Sammy Salem, GE 2549

Geotechnical Manager: Dean Ledgerwood Phone #: 559-271-9700

Geotechnical Engineering Co.: Salem Engineering Group, Inc. Report 3-220-0783, Dated: October 19, 2020

Ground Water Elevation: N/A Fill Soils Characteristics: N/A (If encountered)

Date Groundwater Measured: 09/23/2020 Maximum Liquid Limit: N/A

Topsoil/Stripping Depth: 8” Maximum Plasticity Index: N/A

Undercut (If Required): __See Report_________Specified Compaction: 92 percent ASTM D1557

Modified Proctor Results: 132.0 pcf @7.0% Moisture Content Range: 5.3 to 16.6% (in-situ)

pH: 7.4

Corrective actions required for construction based on pH level noted:_NA___________________

Chloride 14 mg/kg Minimum Resistivity 3,017 ohm-cm

Corrective actions required for construction based on resistivity level noted:
Soils are mildly/moderately corrosive to Buried Metal Objects

Cement Type: No Restrictions

Recommended local DOT subbase/base material (reference section plan in Foundation Subsurface Preparation):

Caltrans Class 2 Aggregate Base

Recommended Compaction Control Tests:

1 Test for Each 2,000 Sq. Ft. each Lift (bldg. area), with two tests minimum if total area is less than 2,000 Sq. Ft
1 Test for Each 2,500 Sq. Ft. each Lift (parking area), with two tests minimum if total area is less than 2,500 Sq. Ft

Structural Fill Maximum Lift Thickness 8 in. (Measured loose)

Subgrade Design R-value=40.

COMPONENT	ASPHALT		CONCRETE	
	Standard	heavy	standard	heavy
Stabilized Subgrade (If Applicable)	_12”___	12”____	__12”__	_12”___
Base Material (Class 2 AB.)	_4.0”	5.0”____	__4”___	__4”___
Asphalt Base Course	__2”__	2” __		
Surface Course	___1.”_	__1”_	__4”__	__4”__

NOTE: This information shall not be used separately from the geotechnical report.

FOUNDATION DESIGN CRITERIA

Include this form in the Geotechnical Report as an Appendix.

PROJECT LOCATION: 1540 East 2nd Street, Beaumont, California

Engineer: R. Sammy Salem, GE 2549

Geotechnical Engineering Co.: Salem Engineering Group, Inc. Report 3-220-0783, Dated: October 19, 2020

Foundation type: Conventional Shallow Foundation

Allowable bearing pressure: Dead + Live = 2,500 psf

Factor of Safety: N/A

Minimum footing dimensions: Individual: 18" Wide Continuous: 15" Wide

Minimum footing embedment: Exterior: 12" Interior: 12"

Frost depth: NA

Maximum foundation settlements: Total: 1", Differential: ½"

Slab: Potential vertical rise: N/A

Capillary Break (not a vapor barrier) describe: 6" crushed rock placed over 4" Class 2 Aggregate Base

Subgrade reaction modulus: 188 psi/in Method obtained: ACPA Calculator

Active Equivalent Fluid Pressures: 33 pcf

Passive Equivalent Fluid Pressures: 400 psf

Perimeter Drains (describe): Building: N/A
Retaining Walls: Drainage Required behind retaining walls per Section 10.12 of GEIR

Retaining Wall: At rest pressure: 50 psf
Coefficient of friction: 0.30

COMMENTS: ____
____
____
____

FUEL STATION FOUNDATION SUBSURFACE PREPARATION
PROPOSED WALMART FUELING STATION (5156)
BEAUMONT, CALIFORNIA
10/19/2020

UNLESS SPECIFICALLY INDICATED OTHERWISE IN THE DRAWINGS AND/OR SPECIFICATIONS, THE LIMITS OF THIS SUBSURFACE PREPARATION ARE CONSIDERED TO BE THAT PORTION OF THE SITE DIRECTLY BENEATH AND 2.5 FEET BEYOND THE FUEL STATION SERVICE BUILDING, DIRECTLY BENEATH AND 2.5 FEET BEYOND CANOPY AND SERVICE BUILDING SLABS, AND DIRECTLY BENEATH AND 2.5 FEET BEYOND CANOPY FOUNDATIONS. AT THE SERVICE BUILDING, THE EXTENTS OF SUBSURFACE PREPARATION SHALL BE SLOPED AWAY FROM THE 5 FOOT PERIMETER AT A MINIMUM 1:1 SLOPE.

AT THE CANOPY AND SERVICE BUILDING FOUNDATIONS, OVER-EXCAVATE TO THE MINIMUM DEPTH OF 24 INCHES BELOW THE BOTTOM OF FOUNDATIONS OR 36 INCHES BELOW PRECONSTRUCTION SITE GRADE, WHICHEVER IS GREATER. THE HORIZONTAL LIMITS OF THE FOUNDATION EXCAVATIONS SHALL EXTEND TO 2.5 FEET BEYOND FOUNDATIONS. THE RESULTING BOTTOM OF OVER-EXCAVATION SHALL BE SCARIFIED TO A DEPTH OF AT LEAST 10 INCHES, WORKED UNTIL UNIFORM AND FREE FROM LARGE CLODS, MOISTURE-CONDITIONED SLIGHLTY ABOVE OPTIMUM MOISTURE, AND COMPACTED TO A MINIMUM OF 92 PERCENT ASTM D1557. ON-SITE SOILS SHALL BE PLACED AS ENGINEERED FILL TO A MINIMUM OF 2 FOOT BELOW FOUNDATIONS. REFERENCE ARCHITECTURAL AND STRUCTURAL DRAWINGS FOR REQUIRED FOUNDATION THICKNESS.

AT THE CANOPY AND SERVICE BUILDING SLABS, OVER-EXCAVATE TO A MINIMUM OF 36 INCHES BELOW SLABS ON GRADE OR 36 INCHES BELOW EXISTING SITE GRADE, OR THE DEPTH REQUIRED TO REMOVE UNDOCMENTED FILLS, WHICHEVER IS GREATER. THE BOTTOM OF EXCAVATION SHALL BE SCARIFIED 10 INCHES, MOISTURE CONDITIONED TO SLIGHTLY ABOVE OPTIMUM AND COMPACTED TO 92 PERCENT (ASTM D1557). ESTABLISH THE FINAL SUBGRADE ELEVATION TO ALLOW FOR THE CONCRETE SLAB AND BASE. REFERENCE ARCHITECTURAL AND STRUCTURAL DRAWINGS FOR REQUIRED SLAB THICKNESS. THE 6 INCH BASE MATERIAL SHALL CONFORM TO CALTRANS CLASS 2 AGGREGATE BASE.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR OBTAINING ACCURATE MEASUREMENTS FOR ALL CUT AND FILL DEPTHS REQUIRED. ANY PROPOSED EQUIVALENT ALTERNATIVE BASE OR SUBBASE MATERIAL MUST BE SUBMITTED FOR APPROVAL WITHIN 30 DAYS AFTER AWARD OF CONTRACT. ANY EQUIVALENT ALTERNATIVE SHALL ONLY BE USED IF APPROVED IN WRITING BY THE CEC AND AOR.

EXISTING FOUNDATIONS, SLABS, PAVEMENTS, AND BELOW-GRADE STRUCTURES SHALL BE REMOVED FROM THE FUEL STATION AREA. REMOVE SURFACE VEGETATIONS, TOPSOIL, ROOT SYSTEMS, ORGANIC MATERIAL, EXISTING FILL, AND SOFT OR OTHERWISE UNSATISFACTORY MATERIAL FROM THE FUEL STATION AREA. PROOFROLL EXPOSED SUBGRADE. REMOVE AND REPLACE UNSATISFACTORY AREAS WITH ENGINEERED FILL PER GEOTECHNICAL REPORT. ENGINEERED FILL MATERIAL SHALL BE FREE OF ORGANIC AND OTHER DELETERIOUS MATERIALS AND SHALL MEET THE FOLLOWING REQUIREMENTS:

LOCATION WITH RESPECT TO FINAL GRADE	P.I.	L.L.
BUILDING AREA, BELOW UPPER 2 FEET	20 MAX.	50 MAX.
BUILDING AREA, UPPER 2 FEET	12 MAX.	40 MAX.

SUBGRADE MATERIAL SHALL BE PLACED IN LOOSE LIFTS NOT EXCEEDING 8 INCHES IN THICKNESS AND COMPACTED TO AT LEAST 92 PERCENT OF THE MODIFIED PROCTOR MAXIMUM DRY DENSITY (ASTM D1557) TO SLIGHTLY ABOVE OPTIMUM MOISTURE CONTENT.

THE FOUNDATION SYSTEM SHALL BE ISOLATED SPREAD FOOTINGS AT CANOPY COLUMNS AND SERVICE BUILDING.

THIS FOUNDATION SUBSURFACE PREPARATION DOES NOT CONSTITUTE A COMPLETE SITE WORK SPECIFICATION. IN CASE OF CONFLICT, INFORMATION COVERED IN THIS PREPARATION SHALL TAKE PRECEDENCE OVER THE SAM'S SPECIFICATIONS. REFER TO THE SPECIFICATIONS FOR SPECIFIC INFORMATION NOT COVERED IN THIS PREPARATION. THIS INFORMATION WAS TAKEN FROM A GEOTECHNICAL REPORT PREPARED BY SALEM ENGINEERING GROUP, INC., DATED OCTOBER 16, 2020 (GEOTECHNICAL REPORT IS FOR INFORMATION ONLY AND IS NOT A CONSTRUCTION SPECIFICATION).

Two-Layer Asphalt Concrete Pavement Design Calculations

Reference: Caltrans Highway Design Manual

Project Name: Proposed Walmart Petroleum Station, Beaumont, California
Project Number: 3-220-0783

Pavement Type: Standard Duty AC Pavements
18-kip Equivalent Single Axle Loading (ESAL): 2,200
Equvelent Traffic Index: 4.5 Rounded to nearest 1/2
Subgrade Design R-value : 40
Aggregate Base R-value : 78

$$\text{Traffic Index(TI)} = 9.0 \times \left(\frac{\text{ESAL} \times \text{LDF}}{10^6}\right)^{0.119}$$

Asphalt Concrete (AC) Layer

$$\text{AC Thickness} = \left(\frac{\text{Gravel Equivalent}(GE_{AC})}{\text{Gravel Factor}(G_{f_{AC}})}\right)$$

$$\text{where: } GE_{AC} = 0.0032 \times (TI) \times [100 - (R - value)_{AB}] + \text{Safety Factor}$$

$$\text{where: Safety Factor} = 0.20 \text{ feet}$$

$$\text{and: } G_{f_{AC}} = \frac{5.67}{(TI^{0.5})}$$

Aggregate Base(AB) Layer

$$\text{AB Thickness} = \left(\frac{\text{Gravel Equivalent}(GE_{AB})}{\text{Gravel Factor}(G_{f_{AB}})}\right)$$

$$\text{where: } GE_{AB} = GE_{(AC+AB)} - GE_{AC_{Actual}}$$

$$\text{where: } GE_{(AC+AB)} = 0.0032 \times (TI) \times [100 - (R - value)_{Subgrade}]$$

$$\text{and: } GE_{AC_{Actual}} = (\text{AC Thickness}) \times G_{f_{AC}}$$

Traffic Index (TI) = 4.50
Lane Distribution Factor (LDF) = 1

Asphalt Concrete (AC) Layer			**Aggregate Base (AB) Layer**		
Required GE =	0.52	feet	Required GE =	0.20	feet
Gravel Factor (GfAC) =	2.67		Gravel Factor (GfAB)= [Class 2 AB]	1.1	
AC Required =	2.3	inches	AB Required =	2.1	inches
Use 3.0 inches AC			**Use 4.0 inches AB**		
Actual GE =	0.67	feet			

SALEM engineering group, inc.

Two-Layer Asphalt Concrete Pavement Design Calculations

Reference: Caltrans Highway Design Manual

Project Name: Proposed Walmart Petroleum Station, Beaumont, California
Project Number: 3-220-0783

Pavement Type: Standard Duty AC Pavements
18-kip Equivalent Single Axle Loading (ESAL): 18,000
Equvelent Traffic Index: 5.5 Rounded to nearest 1/2
Subgrade Design R-value : 40
Aggregate Base R-value : 78

$$\text{Traffic Index(TI)} = 9.0 \times \left(\frac{\text{ESAL} \times \text{LDF}}{10^6}\right)^{0.119}$$

<u>**Asphalt Concrete (AC) Layer**</u>

$$\text{AC Thickness} = \left(\frac{\text{Gravel Equivalent}(GE_{AC})}{\text{Gravel Factor}(G_{f_{AC}})}\right)$$

where: $GE_{AC} = 0.0032 \times (TI) \times [100 - (R-\text{value})_{AB}] + \text{Safety Factor}$

where: $\text{Safety Factor} = 0.20 \text{ feet}$

and: $G_{f_{AC}} = \dfrac{5.67}{(TI^{0.5})}$

<u>**Aggregate Base(AB) Layer**</u>

$$\text{AB Thickness} = \left(\frac{\text{Gravel Equivalent}(GE_{AB})}{\text{Gravel Factor}(G_{f_{AB}})}\right)$$

where: $GE_{AB} = GE_{(AC+AB)} - GE_{AC_{Actual}}$

where: $GE_{(AC+AB)} = 0.0032 \times (TI) \times [100 - (R-\text{value})_{Subgrade}]$

and: $GE_{AC_{Actual}} = (\text{AC Thickness}) \times G_{f_{AC}}$

Traffic Index (TI) = 5.50
Lane Distribution Factor (LDF) = 1

<u>**Asphalt Concrete (AC) Layer**</u>			<u>**Aggregate Base (AB) Layer**</u>		
Required GE =	0.59	feet	Required GE =	0.45	feet
Gravel Factor (GfAC) =	2.42		Gravel Factor (GfAB)= [Class 2 AB]	1.1	
AC Required =	2.9	inches	AB Required =	4.9	inches
Use 3.0 inches AC			**Use 5.0 inches AB**		
Actual GE =	0.60	feet			

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PCC Pavement Section Recommendations for Walmart Fuel Station
Beaumont, California

PCC PAVEMENT DESIGN - AASHTO METHOD

Reference: AASHTO Guide for Design of Pavement Structures 1993

Project Number:	3-220-0783	Date:	10/13/2020
Pavement Section Type:	Standard Duty		

Given Criteria:

Min. Daily ESALs[1]:	0.3	Load Transfer Coefficient (J)[2]:	3.2
Min. 20-yr. Design Life ESALs (W_{18})[1]:	2,200	Concrete Design Strength (psi)[3]:	4,000
Reliability Level (R%)[1]:	85	Concrete Elastic Modulus (E_c) (psi)[4]:	3,120,000
Combined Standard Error (S_o)[1]:	0.35	Concrete Modulus of Rupture (S'_c) (psi)[4]:	500
Initial Serviceability (p_i)[1]:	4.2	Subgrade R-Value[5]:	40
Terminal Serviceability (p_t)[1]:	2.0	Modulus of Subgrade Reaction Value (k)[5]:	188
Drainage Factor (C_d)[2]:	1.0	**Calculated Concrete Thickness (D), Inches:**	**4.0**

1993 AASHTO Empirical Equation for Rigid Pavements:

$$\log_{10} W_{18} = Z_R * S_o + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.624*10^{7}}{(D+1)^{8.46}}} + (4.22 - 0.32P_t) * \log_{10}\left[\frac{S_c^{'} * C_d\left[D^{0.75} - 1.132\right]}{215.63 * J\left[D^{0.75} - \dfrac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

Notes:

[1] Per Walmart Geotechnical Investigation Criteria, September 25, 2018

[2] Assumed value based upon standard conditions

[3] Assumed concrete design 28-day strength

[4] Calculated value based upon normal weight, normal density concrete per (ACI 318)

[5] Assumed value based upon site soil characteristics

PCC Pavement Section Recommendations for Walmart Fuel Station
San Jacinto, California

PCC PAVEMENT DESIGN - AASHTO METHOD

Reference: AASHTO Guide for Design of Pavement Structures 1993

Project Number: 3-220-0783

Date: 10/13/2020

Pavement Section Type: Heavy Duty

Given Criteria:

Min. Daily ESALs[1]:	2.5	Load Transfer Coefficient (J)[2]:	3.2
Min. 20-yr. Design Life ESALs (W_{18})[1]:	18,000	Concrete Design Strength (psi)[3]:	4,000
Reliability Level (R%)[1]:	85	Concrete Elastic Modulus (E_c) (psi)[4]:	3,120,000
Combined Standard Error (S_o)[1]:	0.35	Concrete Modulus of Rupture (S'_c) (psi)[4]:	500
Initial Serviceability (p_i)[1]:	4.2	Subgrade R-Value[5]:	40
Terminal Serviceability (p_t)[1]:	2.0	Modulus of Subgrade Reaction Value (k)[5]:	188
Drainage Factor (C_d)[2]:	1.0	**Calculated Concrete Thickness (D), Inches:**	**4.0**

1993 AASHTO Empirical Equation for Rigid Pavements:

$$\log_{10} W_{18} = Z_R * S_o + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22-0.32P_t)*\log_{10}\left[\frac{S_c' * C_d\left[D^{0.75}-1.132\right]}{215.63*J\left[D^{0.75}-\frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

Notes:

[1] Per Walmart Geotechnical Investigation Criteria, September 25, 2018

[2] Assumed value based upon standard conditions

[3] Assumed concrete design 28-day strength

[4] Calculated value based upon normal weight, normal density concrete per (ACI 318)

[5] Assumed value based upon site soil characteristics

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation was conducted on September 23, 2020 and included a site visit, subsurface exploration, and soil sampling. The locations of the exploratory borings are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

Our borings were drilled using a truck-mounted CME-45C drilling rig equipped with 6-inch hollow stem augers and 4-inch diameter solid flight augers. Sampling was accomplished by driving a 2-inch Standard Penetration Test (SPT) sampler and/or a 3-inch outside diameter Modified California Sampler (MCS) 18 inches into the soil. Penetration and/or Resistance tests were performed at selected depths. The resistance/N-Value obtained from driving was recorded based on the number of blows required to penetrate the last 12 inches. The driving energy was provided by an auto-trip hammer weighing 140 pounds, falling 30 inches. Relatively undisturbed MCS soil samples were obtained while performing this test. Bag samples of the disturbed soil were obtained from the SPT samples and auger cuttings. All samples were returned to our Fresno laboratory for evaluation. At the completion of drilling and sampling, the test borings were backfilled with drill cuttings.

Subsurface conditions encountered in the test 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, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

SALEM engineering group, inc.

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

Project Number: 3-220-0783

Date: 09/23/2020

Client: CEI Engineering Associates, Inc.

Project: Proposed Petroleum Station

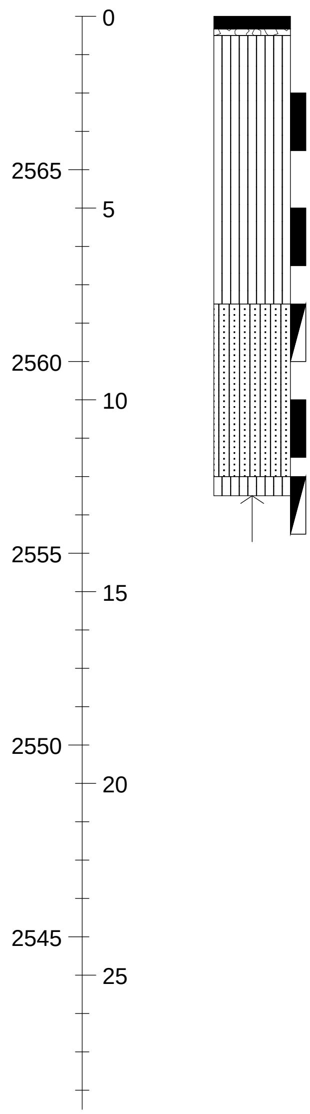
Location: 1540 East 2nd Street, Beaumont, California

Drilled By: SALEM **Logged By:** JC

Drill Type: CME 45C **Elevation:** 2569'

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

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



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
		AC	Asphalt Concrete = 4 in.				
		AB	Aggregate Base = 2 in.				
	4/6 7/6 10/6	ML	Sandy SILT Stiff; moist; reddish brown; fine grain sand.	17	11.8	114.6	
	6/6 7/6 13/6		Grades as above.	20	9.4	121.2	
	13/6 22/6 21/6	SM	Silty SAND Dense; moist; mottled reddish brown/gray/white; fine to medium grain sand; cemented.	43	8.9	-	
	37/6 60/2 -		Very dense @ 10 feet.	60/2"	8.2	127.2	
	13/6 20/6 23/6	ML	Sandy SILT Hard; moist; mottled reddish brown/gray/white; fine to medium grain sand	43	15.3	-	
			Refusal at 12 feet due to hard/ rocky drilling.				

Notes:

Figure Number A-1

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

SALEM
engineering group, inc.

Project Number: 3-220-0783

Date: 09/23/2020

Client: CEI Engineering Associates, Inc.

Project: Proposed Petroleum Station

Location: 1540 East 2nd Street, Beaumont, California

Drilled By: SALEM **Logged By:** JC

Drill Type: CME 45C **Elevation:** 2573'

Auger Type: 4 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 5 in.				
		AB	Aggregate Base = 3 in.				
2570	3/6 6/6 6/6	ML	Sandy SILT Stiff; moist; reddish brown; fine to medium grain sand.	12	11.4	118.4	
5	5/6 10/6 10/6	SM	Silty SAND Medium dense; moist; mottled reddish brown/white; fine to coarse grain sand; light cementation.	20	9.9	119.8	
2565	8/6 12/6 13/6			25	9.7	-	
10	16/6 19/6 13/6		Grades as above; dense; cemented.	32	12.8	-	
2560		ML	Sandy SILT Very stiff; moist; reddish brown; fine grain sand.				
15	7/6 9/6 12/6			21	16.6	-	
			End of boring at 16.5 feet BSG.				
2555							
20							
2550							
25							
2545							

Notes:

Figure Number A-2

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

Project Number: 3-220-0783

Date: 09/23/2020

Client: CEI Engineering Associates, Inc.

Project: Proposed Petroleum Station

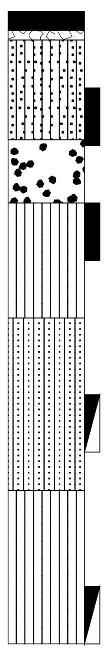
Location: 1540 East 2nd Street, Beaumont, California

Drilled By: SALEM **Logged By:** JC

Drill Type: CME 45C **Elevation:** 2574'

Auger Type: 4 in. Solid Flight Auger **Initial Depth to Groundwater:** N/A

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



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
		AC	Asphalt Concrete = 6 in.				
		AB	Aggregate Base = 3 in.				
	17/6 22/6 18/6	SM/ML	Silty SAND/Sandy SILT Medium dense/Very stiff; moist; reddish brown; fine to medium grain sand.	40	5.3	113.7	
		GP	Sandy GRAVEL Medium dense; dry; light gray; fine gravel; fine to coarse grain sand.				
	12/6 11/6 12/6	ML	Sandy SILT Very stiff; moist; reddish brown; fine to medium grain sand; trace gravel; light cementation.	23	7.7	91.4	
	12/6 12/6 13/6	SM	Silty SAND Medium dense; moist; mottled reddish brown/white; fine to coarse grain sand; trace gravel.	25	8.6	-	
	12/6 12/6 13/6	ML	Sandy SILT Very stiff; moist; light reddish brown; fine to medium grain sand.	25	8.7	-	
			End of boring at 16.5 feet BSG.				

Depth (feet): 0, 5, 10, 15, 20, 25

Elevation: 2570, 2565, 2560, 2555, 2550

Notes:

Figure Number A-3

KEY TO SYMBOLS

Symbol Description

Strata symbols

Asphaltic Concrete

Aggregate Base

Silt

Silty sand

Silty Sand/Sandy Silt

Poorly graded gravel

Misc. Symbols

Drill rejection

Soil Samplers

California sampler

Standard penetration test

Notes:

Granular Soils
Blows Per Foot (Uncorrected)

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

Cohesive Soils
Blows Per Foot (Uncorrected)

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

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

Percolation Test Worksheet

Project: **Proposed Petroleum Station**
1540 East 2nd Street
Beaumont, California

Job No.: **3-220-0783**
Date Drilled: **9/23/2020**
Soil Classification: **Sandy SILT (ML)**

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

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

Presoaking Date: **9/23/2020**
Test Date: **9/23/2020**

Pipe Stick up: 0 ft.

Time Start	Time Finish	Depth of Test Hole (ft)#	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level# (ft)	Final Water Level# (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
12:10	12:40	10.0	Y	0:30	7.91	8.15	2.88	30	10.4	25.1	22.2	23.6	0.45
12:40	13:10	10.0	N	0:30	8.15	8.30	1.80	30	16.7	22.2	20.4	21.3	0.31
13:10	13:40	10.0	N	0:30	8.30	8.40	1.20	30	25.0	20.4	19.2	19.8	0.22
13:40	14:10	10.0	N	0:30	8.40	8.48	0.96	30	31.3	19.2	18.2	18.7	0.19
14:10	14:40	10.0	N	0:30	8.48	8.54	0.72	30	41.7	18.2	17.5	17.9	0.14
14:40	15:10	10.0	N	0:30	8.54	8.59	0.60	30	50.0	17.5	16.9	17.2	0.12
15:10	15:40	10.0	N	0:30	8.59	8.64	0.60	30	50.0	16.9	16.3	16.6	0.13
15:40	16:10	10.0	N	0:30	8.64	8.68	0.48	30	62.5	16.3	15.8	16.1	0.11
Recommended for Design:										**Infiltration Rate**			**0.11**

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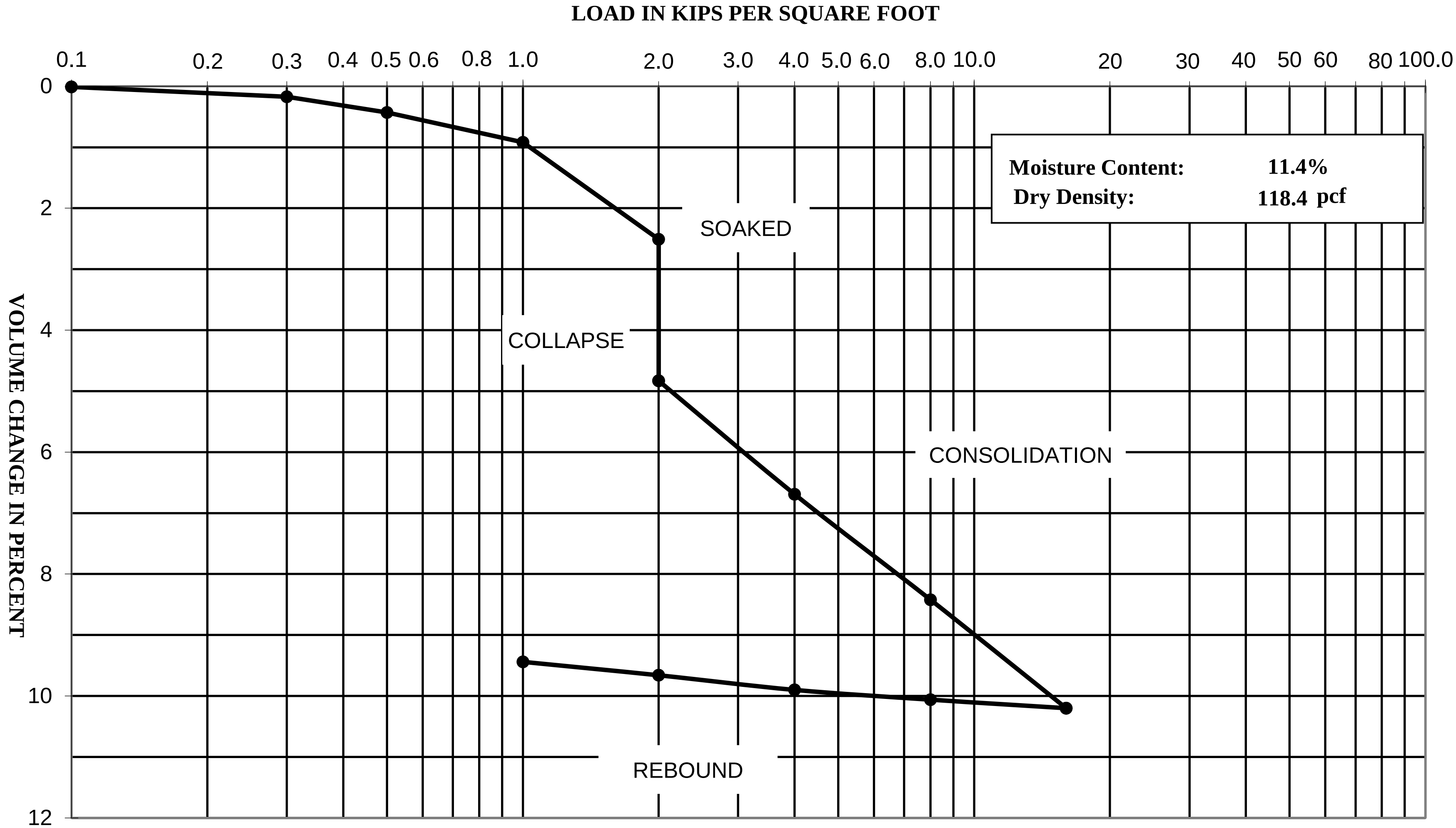
APPENDIX

B

APPENDIX B
LABORATORY TESTING

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

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA

Project Number: 3-220-0783

Boring: B-2 @ 2'

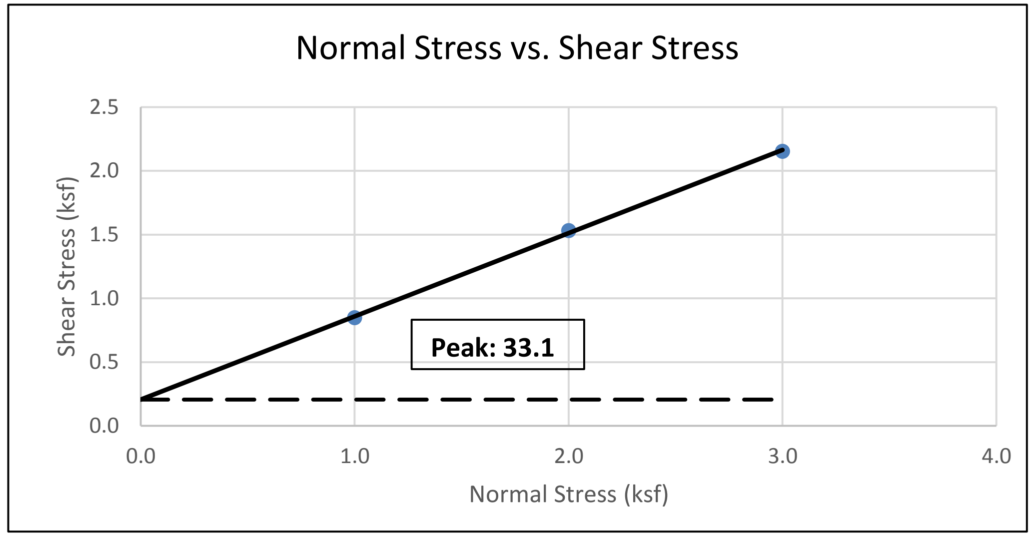
SALEM engineering group, inc.

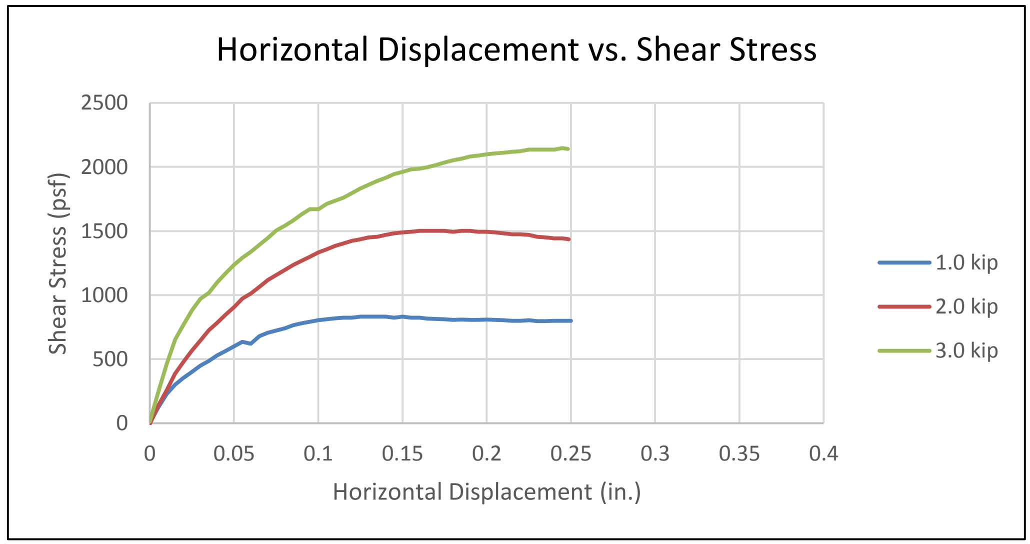
Direct Shear Test (ASTM D3080)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Client: CEI Engineering Associates, Inc.
Boring: B-2 @ 5'
Soil Type: Silty SAND (SM)
Sample Type: Undisturbed Ring
Tested By: NL
Reviewed By: CJ
Date of Test: 9/30/2020
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	2.0 kip	3.0 kip
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	0.85	1.53	2.15

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.975	0.939	0.932
Post-Shear Sample Height (in.)	0.987	0.942	0.930
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	9.9		
Dry Density (pcf)	107.5	115.3	109.5
Saturation %	47.9	59.1	50.5
Void Ratio	0.55	0.45	0.52
Consolidated Void Ratio	0.51	0.36	0.42
Final (post-shear) Values			
Final Moisture Content (%)	17.4	18.1	17.0
Dry Density (pcf)	107.7	115.9	113.2
Saturation %	73.1	104.3	89.7
Void Ratio	0.63	0.46	0.51





Peak Shear Strength Values	
Slope	0.65
Friction Angle	33.1
Cohesion (psf)	206

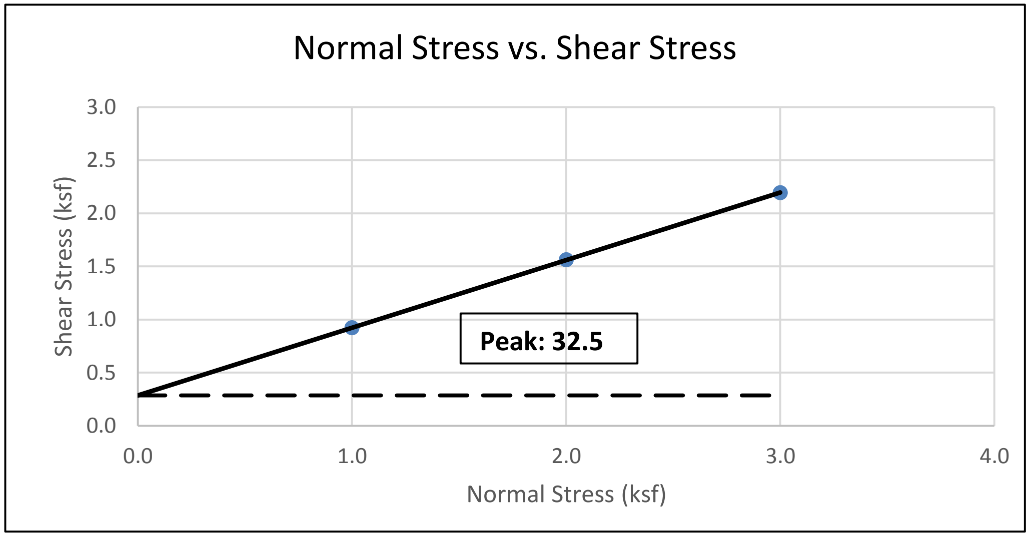
SALEM engineering group, inc.

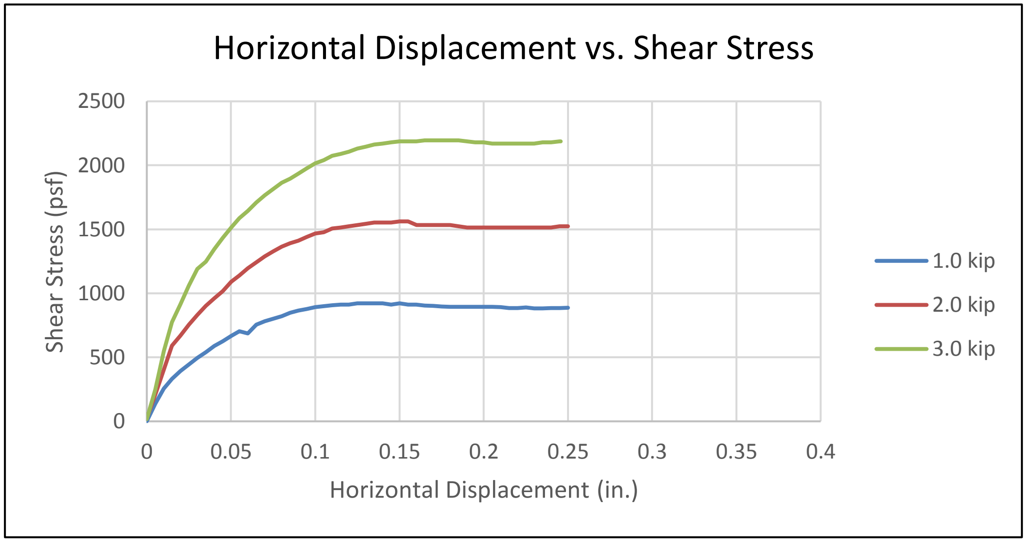
Direct Shear Test (ASTM D3080)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Client: CEI Engineering Associates, Inc.
Boring: B-3 @ 2'
Soil Type: Silty SAND/Sandy SILT (SM/ML)
Sample Type: Undisturbed Ring
Tested By: NL
Reviewed By: CJ
Date of Test: 10/1/2020
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	0.92	1.56	2.20

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.952	0.961	0.945
Post-Shear Sample Height (in.)	0.948	0.952	0.932
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	5.3		
Dry Density (pcf)	110.1	111.0	105.8
Saturation %	27.6	28.3	24.7
Void Ratio	0.51	0.50	0.58
Consolidated Void Ratio	0.44	0.44	0.49
Final (post-shear) Values			
Final Moisture Content (%)	15.5	16.0	17.3
Dry Density (pcf)	111.6	110.4	109.5
Saturation %	72.3	74.5	72.7
Void Ratio	0.57	0.57	0.64





Peak Shear Strength Values	
Slope	0.64
Friction Angle	32.5
Cohesion (psf)	286

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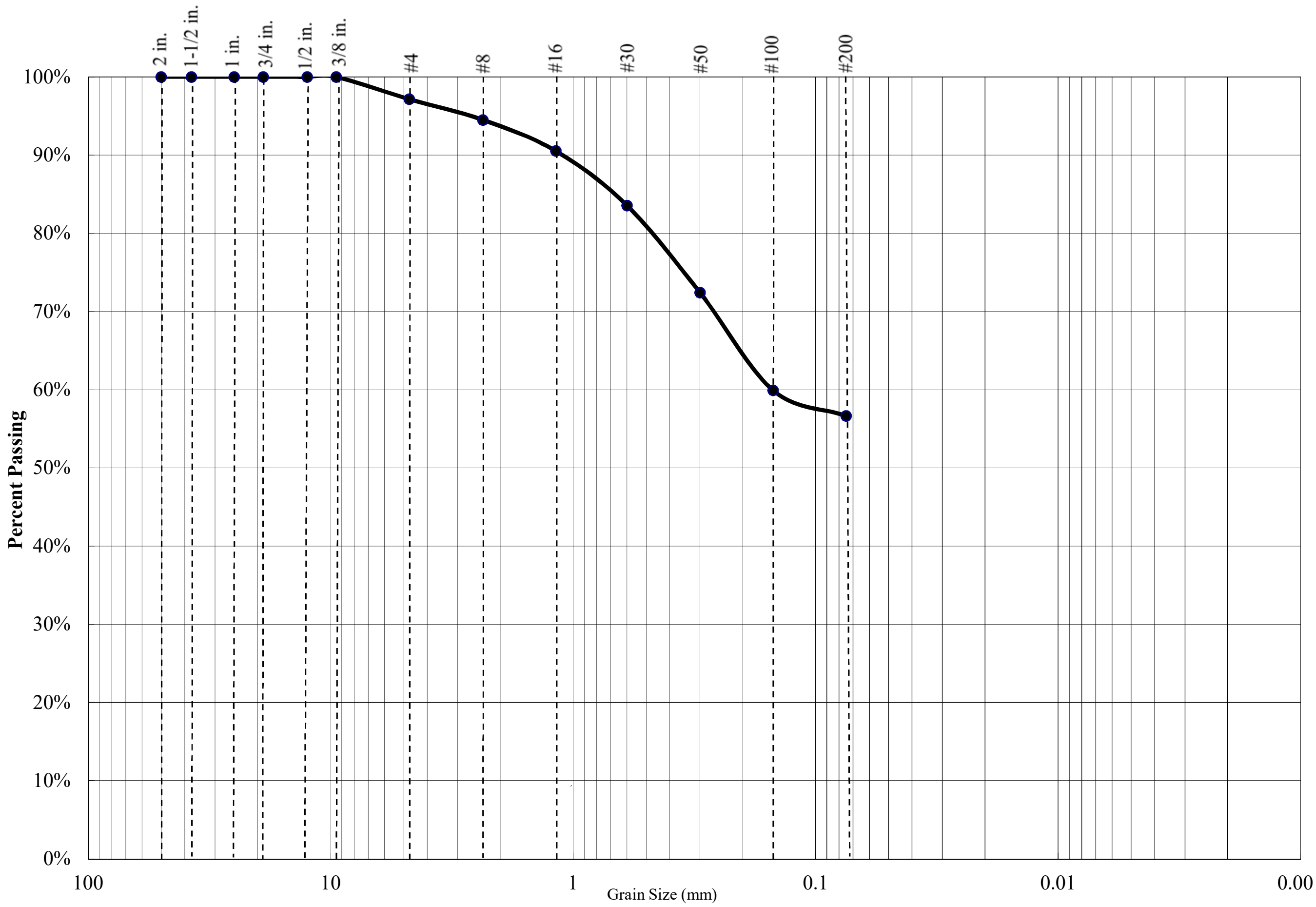
PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
3%	40%	57%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	97.2%
#8	94.5%
#16	90.5%
#30	83.5%
#50	72.4%
#100	59.9%
#200	56.7%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy SILT (ML)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA

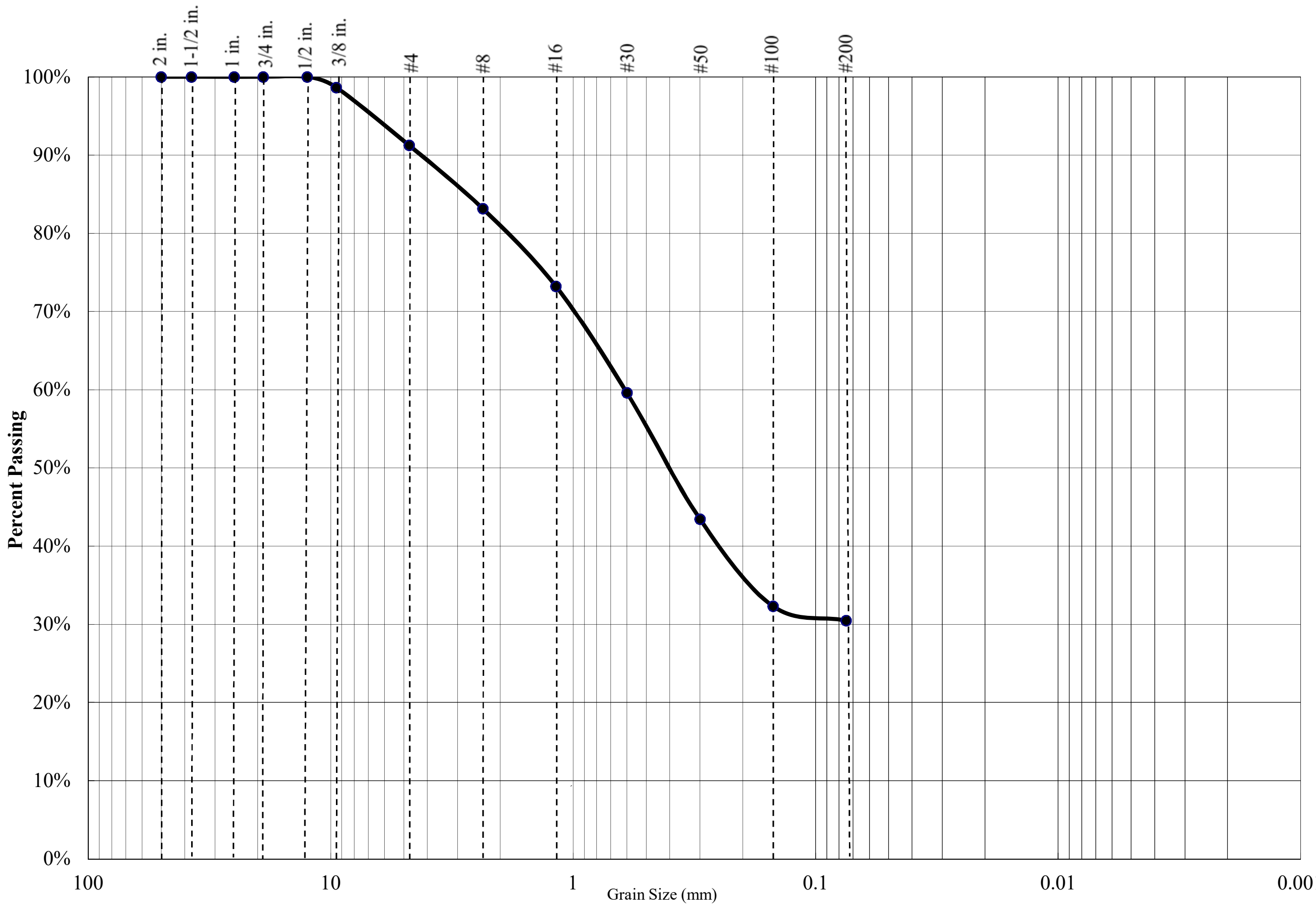
Project Number: 3-220-0783

Boring: B-2 @ 2'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
9%	61%	30%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	98.6%
#4	91.2%
#8	83.1%
#16	73.2%
#30	59.6%
#50	43.4%
#100	32.3%
#200	30.5%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA

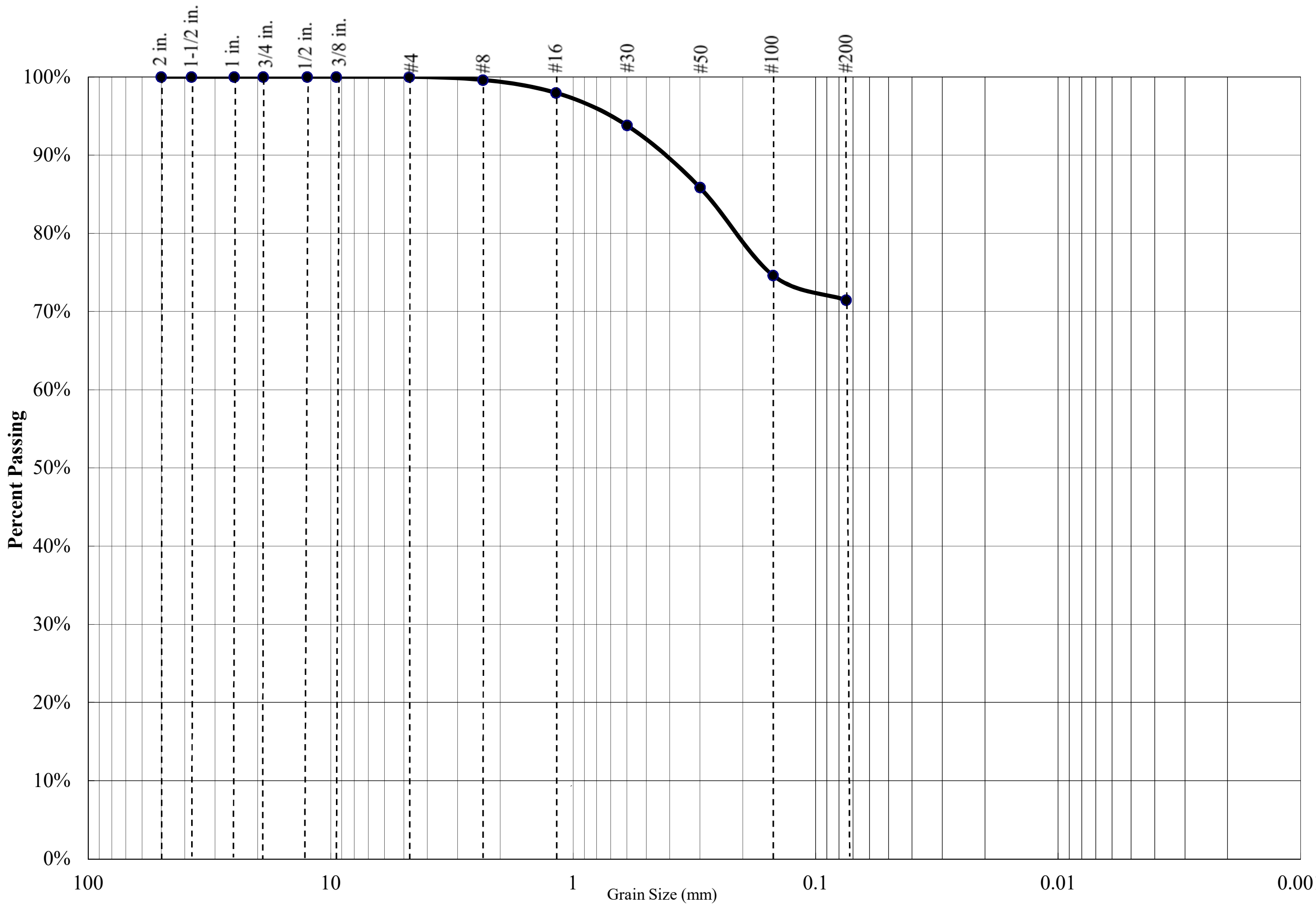
Project Number: 3-220-0783

Boring: B-2 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	29%	71%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	99.6%
#16	98.0%
#30	93.8%
#50	85.8%
#100	74.6%
#200	71.4%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy SILT (ML)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA

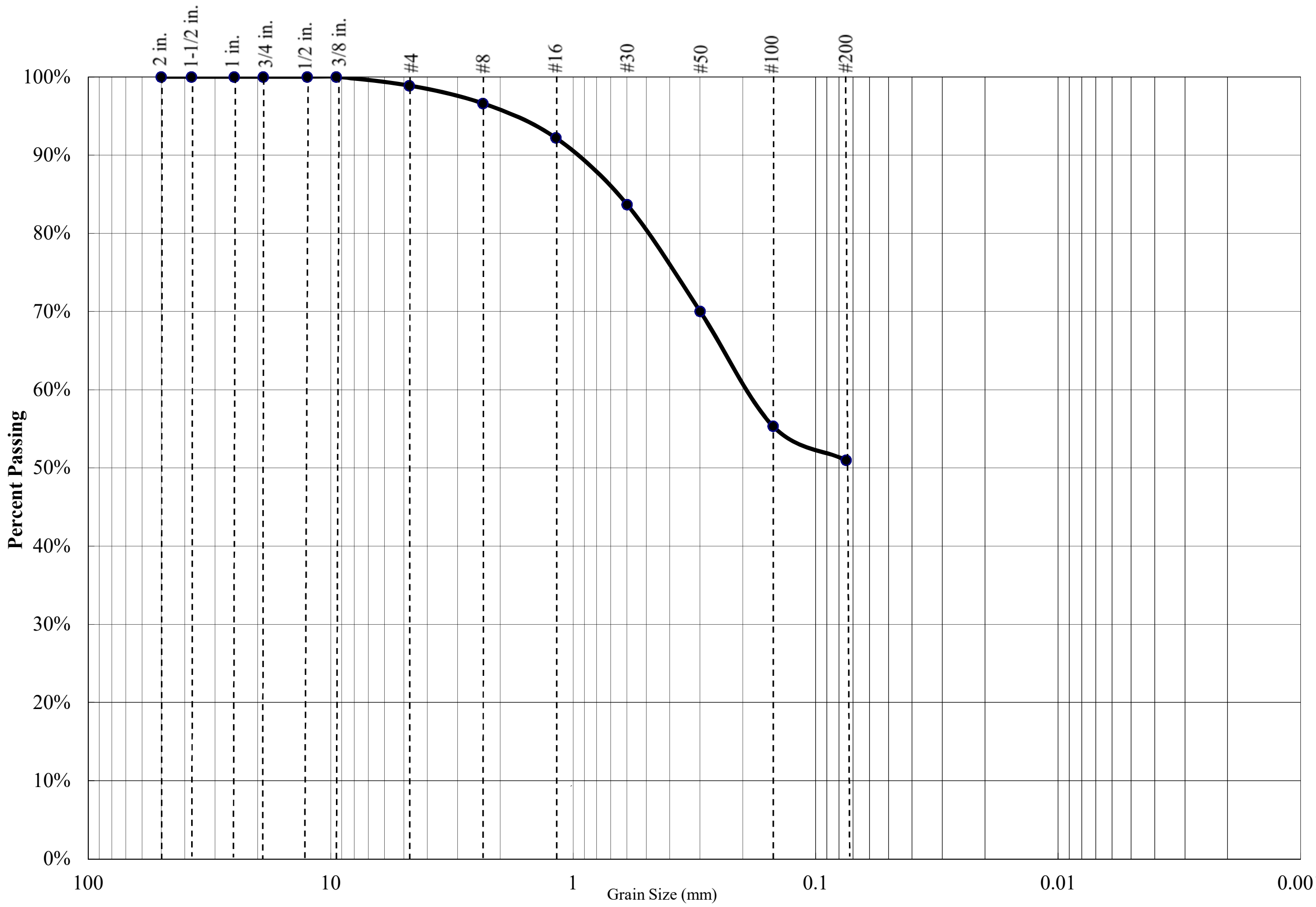
Project Number: 3-220-0783

Boring: B-2 @ 15'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	48%	51%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	98.9%
#8	96.6%
#16	92.2%
#30	83.7%
#50	70.0%
#100	55.3%
#200	51.0%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND/Sandy SILT (SM/ML)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA

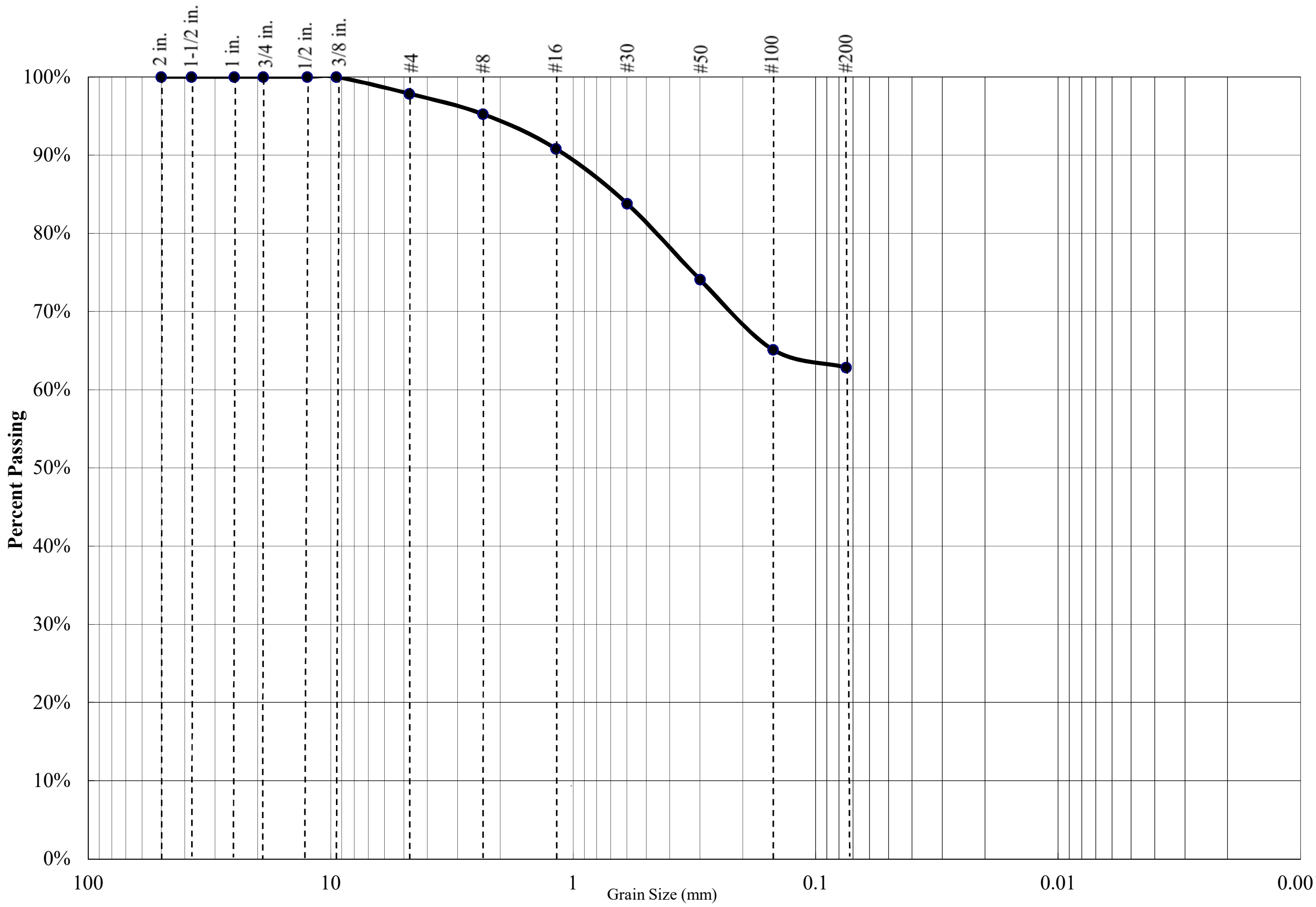
Project Number: 3-220-0783

Boring: B-3 @ 2'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
2%	35%	63%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	97.9%
#8	95.3%
#16	90.8%
#30	83.8%
#50	74.1%
#100	65.1%
#200	62.8%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Sandy SILT (ML)

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA

Project Number: 3-220-0783

Boring: B-3 @ 5'

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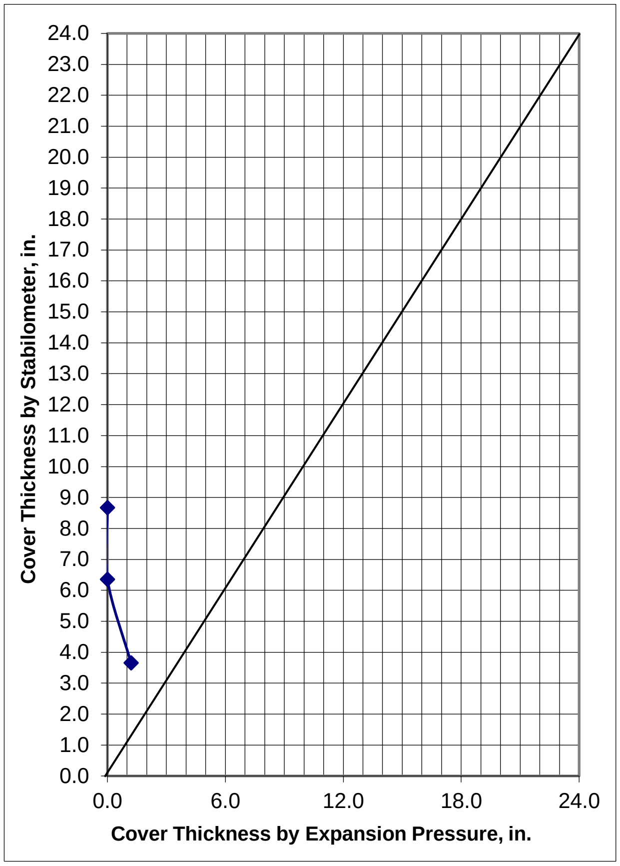
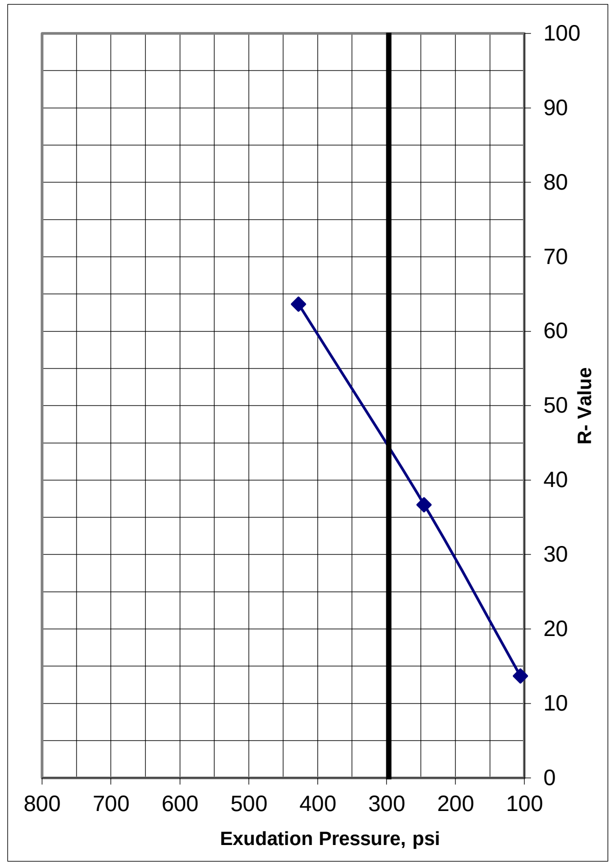
Resistance R-Value
and Expansion Pressure of Compacted Soils
ASTM D2844

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Date Sampled: 9/23/2020 Date Tested: 10/6/2020
Sampled By: SEG Tested By: RM
Sample Location: B-1 @ 0'-3'
Soil Description: Reddish Brown Sandy SILT (ML)





Specimen	1	2	3
Exudation Pressure, psi	427.7	245.4	105.6
Moisture at Test, %	9.3	10.1	12.2
Dry Density, pcf	127.7	124.2	122.7
Expansion Pressure, psf	130	0	0
Thickness by Stabilometer, in.	3.7	6.4	8.7
Thickness by Expansion Pressure, in.	1.2	0.0	0.0
R-Value by Stabilometer	64	37	14
R-Value by Expansion Pressure	N/A		
R-Value at 300 psi Exudation Pressure	44		

Controlling R-Value	**44**

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CHEMICAL ANALYSIS

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

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Date Sampled: 9/23/2020 Date Tested: 9/30/2020
Sampled By: SEG Tested By: JH
Soil Description: Reddish Brown Sandy SILT (ML)

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 0'-3'	110 mg/kg	15 mg/kg	7.4
1b.	B-1 @ 0'-3'	120 mg/kg	14 mg/kg	7.4
1c.	B-1 @ 0'-3'	110 mg/kg	14 mg/kg	7.4
Average:		**113 mg/kg**	**14 mg/kg**	**7.4**

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SOIL RESISTIVITY
CTM 643

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Sample Location: B-1 @ 0'-3'
Soil Description: Reddish Brown Sandy SILT (ML)

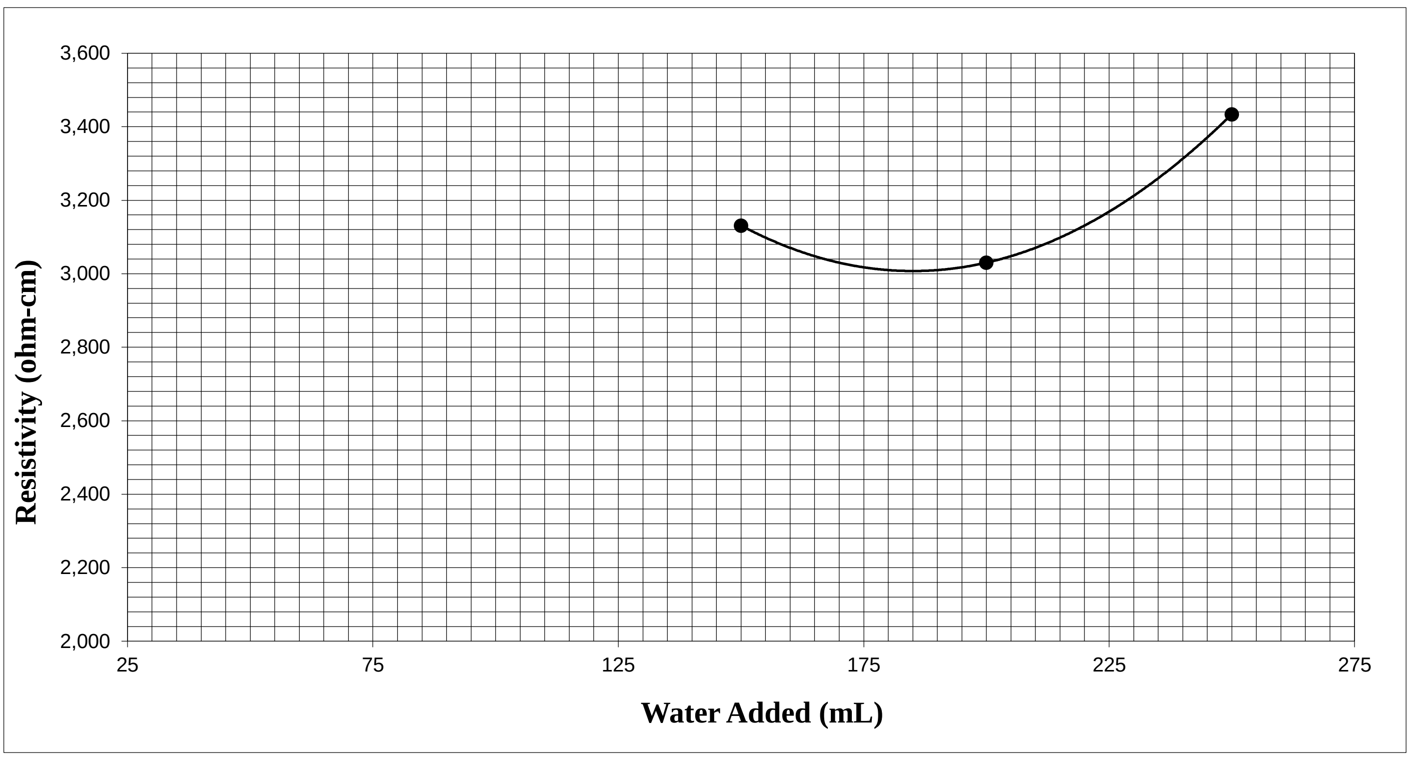
Date Sampled: 9/23/2020
Sampled By: SEG
Date Tested: 9/30/2020
Tested By: IM

Chloride Content: 14 mg/Kg
Sulfate Content: 113 mg/Kg
Soil pH: 7.4

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

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm-cm)
1	150	3.1	1,000	3,100	3,131
2	200	3.0	1,000	3,000	3,030
3	250	3.4	1,000	3,400	3,434



Minimum Resistivity:	**3,017** ohm-cm

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Moisture, Ash, and Organic Matter of Soils

ASTM D2974

Project Name:	Pretroleum Station Beaumont, CA
Project Number:	3-220-0783
Sampled By:	SEG
Sample Date:	9/23/2020
Sample Location:	B-3 @ 0 - 3'
Tested By:	JH
Test Date:	9/30/2020

Test Method A	
Mass of Dish + Foil, (g):	77.54
Mass of Dish + Foil + Soil, (g):	127.60
Mass of as Received Soil, (g):	50.06
Oven Dried Mass of Dish + Foil + Soil, (g):	125.37
Mass of Oven Dried Soil, (g):	47.83

Moisture Content as a Percentage of Oven-Dried Mass, (%):	**4.66%**

Test Method C	
Mass of Dish + Foil, (g):	77.54
Mass of Dish + Foil + Dry Soil, (g):	125.37
Mass of Dry Soil, (g):	47.83
Post-Ignition Mass of Dish + Foil + Ash, (g):	124.52
Mass of Ash, (g):	46.98

Ash Content, (%):	**98.2%**
Organic Matter, (%):	**1.8%**

EXPANSION INDEX TEST
ASTM D4829

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Date Sampled: 9/23/2020 Date Tested: 9/30/20
Sampled By: SEG Tested By: JH
Sample Location: B-3 @ 0'-3'
Soil Description: Reddish Brown Silty SAND/Sandy SILT (SM/ML)

Trial #	1	2	3
Weight of Soil & Mold, g.	607.7		
Weight of Mold, g.	187.8		
Weight of Soil, g.	419.9		
Wet Density, pcf	126.6		
Weight of Moisture Sample (Wet), g.	820.0		
Weight of Moisture Sample (Dry), g.	760.5		
Moisture Content, %	7.8		
Dry Density, pcf	117.4		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	48.6		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.0001	0.0007	0.0009	--	0.0010

Expansion Index $_{measured}$ = 1
Expansion Index $_{50}$ = 0.5

Expansion Index = 0

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

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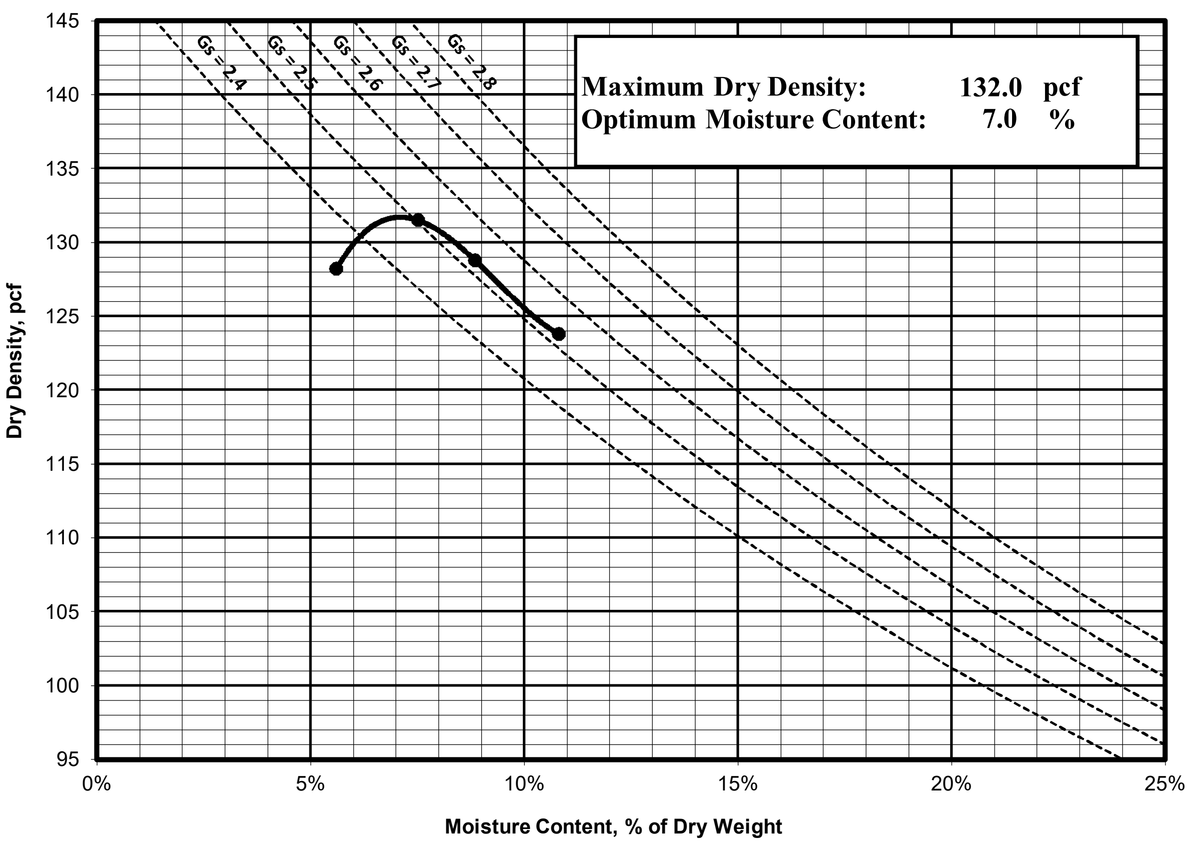
Laboratory Compaction Curve
ASTM D1557

Project Name: Proposed Walmart Pretroleum Station - Beaumont, CA
Project Number: 3-220-0783
Date Sampled: 9/23/2020 Date Tested: 9/29/2020
Sampled By: SEG Tested By: MZ
Sample Location: B-3 @ 0'-3'
Soil Description: Reddish Brown Silty SAND/Sandy SILT (SM/ML)
Test Method: Method A

	1	2	3	4
Weight of Moist Specimen & Mold, (g)	4046.0	4136.7	4118.6	4073.2
Weight of Compaction Mold, (g)	1999.4	1999.4	1999.4	1999.4
Weight of Moist Specimen, (g)	2046.6	2137.3	2119.2	2073.8
Volume of Mold, (ft^3)	0.0333	0.0333	0.0333	0.0333
Wet Density, (pcf)	135.4	141.4	140.2	137.2
Weight of Wet (Moisture) Sample, (g)	325.8	325.8	325.8	325.8
Weight of Dry (Moisture) Sample, (g)	308.5	303.0	299.3	294.0
Moisture Content, (%)	5.6%	7.5%	8.9%	10.8%
Dry Density, (pcf)	128.2	131.5	128.8	123.8



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APPENDIX

C

APPENDIX C
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

14.0 AGGREGATE BASE - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class 2 material, ¾-inch or 1½-inches maximum size. The aggregate base material shall be compacted to a minimum relative compaction of 95 percent based upon ASTM D1557. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

15.0 AGGREGATE SUBBASE - The aggregate subbase shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate subbase material shall conform to the requirements of Section 25 of the Standard Specifications for Class 2 Subbase material. The aggregate subbase material shall be compacted to a minimum relative compaction of 95 percent based on ASTM D1557, and it shall be spread and compacted in accordance with the Standard Specifications. Each layer of aggregate subbase shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

16.0 ASPHALTIC CONCRETE SURFACING - Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at a central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades, and dimensions shown on the plans. The viscosity grade of the asphalt shall be PG 64-10, unless otherwise stipulated or local conditions warrant more stringent grade. The mineral aggregate shall be Type A or B, ½ inch maximum size, medium grading, and shall conform to the requirements set forth in Section 39 of the Standard Specifications. The drying, proportioning, and mixing of the materials shall conform to Section 39. The prime coat, spreading and compacting equipment, and spreading and compacting the mixture shall conform to the applicable chapters of Section 39, with the exception that no surface course shall be placed when the atmosphJohn 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.