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

SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED PETROLEUM STATION
16720 MONTEREY HIGHWAY
MORGAN HILL, CALIFORNIA 95037

SALEM PROJECT NUMBER 5-220-0505
AUGUST 7, 2020

PREPARED FOR:

MR. JOHN HUNDLEY
WORLD OIL CORP
9302 GARFIELD AVENUE
SOUTH GATE, CALIFORNIA 90280

PREPARED BY:

SALEM ENGINEERING GROUP, INC.
4729 W. JACQUELYN AVENUE
FRESNO, CA 93722
P: (559) 271-9700
F: (559) 275-0827
www.salem.net

SALEM engineering group, inc.

4729 W. Jacquelyn Avenue
Fresno, CA 93722
Phone (559) 271-9700
Fax (559) 275-0827

August 7, 2020 Project Number 5-220-0505

Mr. John Hundley
World Oil Corp
9302 Garfield Avenue
South Gate, California 90280

Subject: GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED PETROLEUM STATION
16720 MONTEREY HIGHWAY
MORGAN HILL, CALIFORNIA 95037

Dear Mr. Hundley:

As requested and authorized, SALEM Engineering Group, Inc. (SALEM) has prepared this geotechnical engineering investigation report for the proposed petroleum station planned within the existing development lot located at 16720 Monterey Highway in Morgan Hill, California.

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

Joshua R. Marroquin, EIT
Geotechnical Staff Engineer
Central / Northern California

Dean B. Ledgerwood II, CEG
Northern California Geotechnical Manager
CEG 2613

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

4729 W. Jacquelyn Avenue
Fresno, CA 93722
Phone (559) 271-9700
Fax (559) 275-0827

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED PETROLEUM STATION
16720 MONTEREY HIGHWAY
MORGAN HILL, CALIFORNIA 95037

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical engineering investigation for the proposed petroleum station and convenience store planned within the existing development lot located at 16720 Monterey Highway in Morgan Hill, California, as depicted on Figure 1, Vicinity Map.

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

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

2. PROJECT DESCRIPTION

We understand that development of the site includes the construction of a Petroleum Station with an approximate 2,114 square feet convenience store, a fuel canopy with fuel dispensers, and underground storage tanks. The proposed 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 existing underground tanks will be removed. 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 flat grades at the project site during our field exploration, it is anticipated that cuts and fills will be on the order of 1 to 2 feet. 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 fuel station is planned in the northeast corner of the intersection of Monterey Highway and San Pedro Avenue. The area of the proposed fuel station is currently developed with an existing fuel station. The site was noted to be covered with asphaltic concrete pavements, with isolated landscape islands, an existing building, underground utilities, and underground fuel tanks. It is our understanding the existing improvements will be demolished prior to construction of the proposed fuel station. The site is bounded Monterey Highway to the west, San Pedro Avenue to the south, and existing commercial developments to the east and north.

Existing underground utilities, including existing fuel tanks were noted are within the footprint of the proposed building pad. The project site area is relatively flat. Based on review of Google Earth aerial imagery, site elevation is approximately 337 feet above mean sea level (AMSL).

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-2) were drilled on July 15, 2020, within or near the proposed petroleum station areas to the depths ranging from 16.5 feet to the maximum depth explored of 24 feet below site grade. At 24 feet BSG, auger refusal due to very dense gravel/cobble materials were noted. Two (2) percolation test borings were drilled to depths of about 3.3 feet and 5 feet BSG. The results of the percolation tests are discussed in section 5.4 of this report. The approximate locations of the test borings are shown on Figure No. 2, Site Plan. The test borings were advanced with 8-inch diameter hollow stem auger and 6-inch diameter solid flight auger rotated by a truck-mounted CME-55 drill rig.

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

A 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.

5. LABORATORY TESTING

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

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

6. SOIL AND GROUNDWATER CONDITIONS

6.1 Subsurface Soil Conditions

The test borings drilled encountered approximately 3.5 inches of asphalt concrete over 3.5 inches of aggregate base. Below the pavements, the subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils encountered included clayey sand with gravel and clayey sand, to the maximum depth explored of 24 feet below site grade where practical refusal was encountered due to dense gravel and/or cobbles.

A consolidation test performed on a near surface sample, resulted in about 10¼ percent consolidation under a load of 8 kips per square foot. When wetted under a load of 2.0 kips per square foot, the sample exhibited about 2¾ percent collapse. A direct shear test resulted in internal angle of friction of 38 degrees with cohesion value of 202 pound per square-feet. An Atterberg Limits test performed on a near surface soil sample resulted in a plasticity index of 15 and liquid limit value of 30. An expansion index test performed on a sample collected from between 0 and 3 feet below site grade resulted in an expansion index of 30.

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.

6.2 Groundwater

The test borings were checked for the presence of groundwater during and after the excavation operations. Groundwater was encountered during the time of our subsurface investigation at a depth of approximately 15 feet below site grade.

Based on review of available groundwater depth records with the California Department of Water Resources Groundwater (www.water.ca.gov./waterlibrary) indicate State Well No 09S03E20K003M., located approximately 1.28 miles northwest of the project site, reported a historical high groundwater depth of 5.92 feet in April 2017.

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

6.3 Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate and chloride levels and how they relate to cement reactivity with soil and/or water. A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil samples was detected to be 497 mg/kg.

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

TABLE 6.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Dissolved Sulfate (SO_4) in Soil % by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementitious Materials Type
0.0493	N/A	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 24 mg/kg. In addition, testing performed on a near surface soil resulted in a minimum resistivity value of 2,188 ohm-centimeters. Based on the results, these soils would be considered to have a "highly corrosive" 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>.

6.4 Results of Percolation Testing

The approximate locations of the percolation tests are shown on the attached Figure 2. Approximately 6-inch diameter percolation boreholes were advanced using a truck mounted drill rig. Approximately 2 inches of gravel was placed in the bottom of each hole followed by a 3-inch diameter perforated pipe. The annulus surrounding the perforated pipe was backfilled with gravel. The holes were pre-saturated before percolation testing commenced. The following table includes a summary of the percolation tests:

Location	Depth, BSG (feet)	Gravel Pack Corrected Unfactored Percolation Rate (minutes per inch)	Estimated Unfactored Infiltration Rate (inches per hour)	Soil Type (USCS)
P1	3.30	68.5	0.12	SC
P2	5.00	50.9	0.09	SC

The results of the percolation tests performed generally indicate the soils tested have very low infiltration characteristics. Based on the infiltration tests performed an average infiltration rate of 0.09 inches per hour should be considered for design. The estimated infiltration rates included in this report are unfactored. An appropriate factor of safety should be selected for design. At a minimum a factor of safety of 3 should be considered.

SALEM Engineering should be provided plans showing the limits and calculations used for design of the proposed drainage for review. During construction, the bottom of the proposed drainage system should be inspected and/or tested for infiltration to determine if the system has been design appropriate.

7. GEOLOGIC SETTING

The subject site is located in the Coast Ranges Geomorphic Province of California. The Coast Ranges province comprises a series of northwest-trending, low (2,000 to 4,000 feet above sea level) mountains and valleys that trend sub-parallel to the San Andreas Fault. The San Andreas Fault, the most prominent geologic feature of the province, separates two distinct bedrock regions. To the west is the Salinian Block, composed of a granitic core overlain by Mesozoic and Cenozoic sedimentary strata. To the east of the fault (including the subject site) lies the Franciscan Complex, which is a complexly folded mélange of Mesozoic marine sedimentary deposits. In several areas, the Franciscan rocks are overlain by Cenozoic volcanic cones and flows.

The subject site is mapped by the Geologic Map of the Mt. Madonna Quadrangle, as underlain by Quaternary alluvial deposits (Qa)[1]. Generally the materials encountered throughout the depths explored are generally consistent with the alluvium mapped within the site.

8. GEOLOGIC HAZARDS

8.1 Faulting and Seismicity

Based on the proximity of several dominant active faults and seismogenic structures, as well as the historic seismic record, the area of the subject site is considered subject to relatively moderate seismicity.

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

To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 37.1215° North; site longitude is -121.6473° West. The ten closest active faults are summarized below in Table 8.1.

TABLE 8.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Calaveras;CN+CC+CS	4.22	7.0
N. San Andreas;SAS	9.83	7.1
Monte Vista-Shannon	12.19	6.5
Zayante-Vergeles	13.17	7.0
Calaveras;CS	13.41	5.8
N. San Andreas;SAP	20.08	7.2
Quien Sabe	20.63	6.6
San Andreas fault - creeping segment	21.95	N/A
Ortigalita	22.64	7.1
Calaveras;CN	24.00	6.9

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.

8.2 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active faults with the potential for surface fault rupture are known to pass directly beneath the

[1] Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Mt. Madonna quadrangle, Santa Clara and Santa Cruz Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-159, scale 1:24,000

site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low.

8.3 Ground Shaking

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

Based on Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.712g (based on both probabilistic and deterministic seismic ground motion).

8.4 Liquefaction

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

In general, the soils encountered included clayey sand with gravel to the maximum depth explored of 24 feet below site grade. Free groundwater was encountered at a depth of 15 feet below site grade during this investigation. Based on available water well data, historic groundwater depths are reported to be around 6 feet BSG.

A 50 foot deep boring was included in scope of this investigation, but practical refusal was encountered at 24 feet due to dense gravel and cobbles. Based on the County of Santa Clara Department of Planning and Development, the project site is not located in an area mapped for liquefaction. Based on medium dense to dense soils encountered and our experience in the site vicinity liquefaction potential at the project site is considered to be low.

8.5 Lateral Spreading

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

8.6 Landslides

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

8.7 Tsunamis and Seiches

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

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

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General Conclusions

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

9.1.2 The critical geotechnical concern identified during this investigation is the presence of existing underground fuel tanks within the proposed building pad area. Removal of existing fuel tanks followed by backfill with an approved fill material compacted as engineered fill will be imperative to support of the proposed building. In addition, the tank excavation will be required to be benched out within the building pad to limit differential fill thickness below the pad to no more than 1 foot vertical over a horizontal distance of 5 feet.

9.1.3 In general, the soil encountered included clayey sand with gravel to the maximum depth explored of 24 feet below site grade where practical refusal was encountered due to dense gravel and/or cobbles.

9.1.4 Based on the results of laboratory testing these near surface soils were determined to have a high compressibility, moderate collapse potential, and a low expansion potential.

9.1.5 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.

9.1.6 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 to shallow spread foundations on the order of about 1-inch and differential static of ½ inch in 40 feet, should be anticipated for design.

9.1.7 Based on the chemistry testing performed, the near surface soils have ‘negligible’ potential for sulfate attack on concrete, and a “highly corrosive” potential to buried metal.

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

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

9.2 Surface Drainage

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

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

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

9.3 Site Grading

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

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

9.3.3 Site demolition activities shall include removal of all surface obstructions not intended to be incorporated into final site design. In addition, undocumented fill, underground buried structures, 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.

9.3.4 Site preparation should begin with removal of existing surface/subsurface structures, pavements, underground utilities (as required), underground tanks, 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.

Existing underground fuel tanks were noted within the limits of the proposed building pad. Upon removal of the underground tanks, the subsequent excavation should be backfilled with on-site soils or approved imported fill material compacted as engineered fill. **<u>Within the building pad the tank excavation shall be benched into adjacent native soils to limit the thickness of differential fill below the building pad to no more than 1 foot vertical over a horizontal distance of 5 feet.</u>**

9.3.5 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.

9.3.6 Structural building pad areas and over-build zone should be considered as areas extending a minimum of 5 feet horizontally beyond the outside dimensions of buildings. The over-build zone for shallow canopy foundations may extend horizontally to 5 feet beyond foundations.

9.3.7 To provide uniform support for the proposed building, it is recommended that over-excavation extend to at least 24 inches below preconstruction site grade, to 18 inches below foundations, or to the depth required to remove any undocumented fills (if encountered), whichever is greater. The resulting bottom of excavation shall be scarified to a minimum depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to at least 1 percent above optimum moisture, 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 5 feet beyond the outer edges of the proposed building pad.

As described above in section 9.3.4, backfill of tank excavations within the building pad should be performed to limit differential fill thickness within the pad to no more than 1 foot vertical over 5 feet horizontal.

9.3.8 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 moisture conditioned engineered fill extending below foundations.

9.3.9 If desired to support the canopies on shallow foundations, to provide uniform support for the proposed canopy foundations, it is recommended that over-excavation extend to at least 24 inches below preconstruction site grade, 24 inches below the bottom of proposed foundations, or to the depth required to remove any undocumented fills (if encountered), whichever is greater. The resulting bottom of excavation shall be scarified to a minimum depth of at least 8 inches, worked until uniform and free from large clods, moisture-conditioned to slightly above optimum moisture, and compacted to 92 percent of the maximum density. The horizontal limits of the over-excavation should extend laterally to a minimum of 5 feet beyond the outer edges of the proposed footings.

9.3.10 After stripping of the asphalt, areas of exterior concrete slabs on grade located outside the building pad over-build zone, should be prepared by over-excavation to a depth of 12 inches below existing grade, 12 inches below the bottom of concrete slabs on grade, or the depth required to remove undocumented fills, whichever is greater. The zone of over-excavation should extend a minimum of 3 feet beyond these improvements. These soils should be moisture conditioned to at least 1 percent above optimum 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.

9.3.11 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 footing/over-excavation shall be scarified to a depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to slightly above optimum moisture, and compacted to 92 percent of the maximum density. The horizontal limits of the over-excavation should extend, laterally to a minimum of 3 feet beyond the outer edges of the proposed footings.

9.3.12 Areas of new asphaltic concrete or Portland cement concrete pavements should be prepared by over-excavation to 12 inches below preconstruction site grade or 12 inches below the proposed aggregate base layer, whichever provides greater fill. The bottom of excavation should be scarified a minimum of 12 inches and compacted as engineered fill. The horizontal limits of the over-excavation should extend, laterally to a minimum of 3 feet beyond pavements. The upper 12 inches below aggregate base sections should be compacted to a minimum of 95 percent relative compaction.

9.3.13 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 slightly above optimum and compacted as engineered fill.

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

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

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

9.3.17 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.

9.4 Soil and Excavation Characteristics

9.4.1 Based on the soil conditions encountered in our borings, the onsite soils can be excavated with conventional excavation equipment. However, if elected to support canopies on CIDH piers, excavations extending greater than 20 to 25 feet BSG may encounter very dense cobbles.

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

9.4.3 Due to the existing site development, undocumented fill material may be encountered throughout the site. This report includes recommendations that all abandoned subsurface structures, and undocumented fill material, be fully removed and/or compacted as engineered fill.

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

9.5 Materials for Fill

9.5.1 On-site soils are suitable for use as general Engineered Fill in structural areas, and below the aggregate base section recommended below concrete slabs on grade, provided they do not contain deleterious matter, organic material, or particles larger than 3 inches in maximum dimension.

9.5.2 Imported 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 9.5.2.

TABLE 9.5.2
IMPORT FILL REQUIREMENTS

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

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

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

9.5.4 On-Site soils used as engineered fill soils should moisture conditioned to at least 1 percent above optimum moisture content, and compacted to at least 92 percent relative compaction (ASTM D1557).

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

9.5.6 Engineered fill placed at depths greater than 5 feet BSG should be compacted to a minimum of 95 percent relative compaction (ASTM D1557).

9.5.7 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.

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

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

9.5.10 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

9.6 Seismic Design Criteria

9.6.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2019 CBC, our recommended parameters are shown below. These parameters were determined using 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 9.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2016 ASCE 7 or 2019 CBC Reference
Site Coordinates (Datum = NAD 83)		37.1215 Lat -121.6473 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}	1.100	ASCE 7 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.712 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	1.550 g	CBC Figure 1613.3.1(1-6)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.600 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}	1.550 g	CBC Equation 16-37
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	1.020 g*	CBC Equation 16-38
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.034 g	CBC Equation 16-39
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	0.680 g*	CBC Equation 16-40
Short Period Transition Period (S_{D1}/S_{DS}), Seconds	T_S	0.658	ASCE 7-16, Section 11.4.6
Long Period Transition Period (seconds)	T_L	12	ASCE 7-16, Figures 22-14 through 22-17

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

Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, Structures on Site Class D, with S1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific ground motion analysis may not be required based on Exceptions listed in ASCE 11.4.8. The Structural Engineer should verify whether Exception No. 2 of ASCE 7-16, Section 11.4.8 is valid for the site. In the event a site specific ground motion analysis is required, SALEM should be contacted for these services.

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

9.7 Shallow Foundations

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

9.7.2 The bearing wall footings considered for the building should be continuous with a minimum width of 15 inches and extend to minimum depths of 12 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.

9.7.3 Shallow foundations for the planned fueling canopies, if utilized, should have a minimum width of 24 inches and extend to the minimum depth of 24 inches below 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.

9.7.4 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 3,000 pounds per square foot. This value may be increased by 1/3 for short term wind and seismic loading.

9.7.5 Resistance to lateral footing displacement can be computed using a coefficient of friction factor of 0.38 acting between the base of foundations and engineered fill soils.

9.7.6 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 375 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.

9.7.7 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.

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

9.7.9 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.

9.8 Cast in Drilled Hole Pier Foundations for Canopies

9.8.1 Cast in Drilled Hole Pier foundations should have a minimum diameter of 24 inches and extend a minimum depth of 12 feet below the lowest adjacent grade.

9.8.2 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

9.8.3 Casing of the drilled pier will be required if 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.

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

9.8.5 Skin friction within the upper 1 foot BSG should be neglected in design. The downward load capacity of the piers (extending to at least 12 feet BSG), may be designed based on an allowable skin friction value of 350 pounds per square foot. Provided the CIDH excavation is cleaned of loose soils, an allowable end bearing value of 4,000 pounds per square foot may be considered for design. These values may be increased by 1/3 for short duration wind and seismic loading.

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

9.8.7 The drilled Cast in Drilled Hole Piers should be designed neglecting the lateral capacity within the upper foot. Below a depth of one (1) foot, the lateral capacity can be designed for 375 pounds per square foot per foot of depth below the lowest adjacent grade to a maximum of 3,750 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 6 feet.

9.8.8 If desired, the drilled Cast in Drilled Hole Piers may be designed using LPILE. The soil parameters for LPILE lateral pile analysis are provided as follows:

TABLE 9.8.8.
LPILE PARAMETERS

USCS Soil Type	Effective Unit Weight (pcf)	Angle of Internal Friction (degrees)	Undrained Shear Strength, Cohesion, (psf)	Modulus of Subgrade Reaction, K (pci)	Soil Strain Ratio, ε_{50}
SC/CL	125	38	200	90	--

9.9 Interior Concrete Slabs-on-Grade

9.9.1 Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that non-structural slabs-on-grade be at least 4 inches thick and underlain by four (4) inches of class 2 aggregate base compacted to 95 percent relative compaction over engineered fill extending to the depth recommended below foundations. Based on the low expansive potential of the on-site soils, concrete slabs on grade should be anticipated to be subject to up to ½ inch of heave. If this is not acceptable, the upper 8 inches of engineered fill below the recommended base section should comprise of an approved imported non expansive engineered fill. If elected, this should be specified by the Structural Engineer on the project drawings.

9.9.2 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.

9.9.3 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.

9.9.4 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.

9.9.5 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.

9.9.6 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. To minimize moisture vapor intrusion, it is recommended that a vapor retarder be installed in accordance with manufacturer's recommendations and/or ASTM guidelines, whichever is more stringent. In addition, ventilation of the structure is recommended to reduce the accumulation of interior moisture.

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

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

9.9.9 Avoid use of stakes driven through the vapor retarder.

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

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

9.10 Exterior Concrete Slabs on Grade

9.10.1 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 9.3 of this report.

9.10.2 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.

9.10.3 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.

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

9.11 Lateral Earth Pressures and Frictional Resistance

9.11.1 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	32
At-Rest Pressure, Drained, pcf	52
Allowable Passive Pressure, pcf	375
Allowable Coefficient of Friction	0.38
Maximum Unit Weight (pcf) [γ_{max}]	135
Minimum Unit Weight (pcf) [γ_{min}]	105

9.11.2 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.

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

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

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

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

Dynamic Seismic Lateral Loading Equation
Dynamic Seismic Lateral Load = $\frac{3}{8}\gamma K_h H^2$
Where: γ = Maximum In-Place Soil Density (Section 9.11.1 above)
K_h = Horizontal Acceleration = $\frac{2}{3}PGA_M$ (Section 9.6.1 above)
H = Wall Height

9.12 Temporary Excavations

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

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

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

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

RECOMMENDED EXCAVATION SLOPES

Depth of Excavation (ft)	Slope (Horizontal : Vertical)
0-5	1:1
5-10	1½:1
10-15	2:1
15-20	2½:1

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

9.12.6 Braced shorings should be designed for a maximum pressure distribution of 20H, (where H is the depth of the excavation in feet). The foregoing does not include excess hydrostatic pressure or surcharge loading. Fifty percent of any surcharge load, such as construction equipment weight, should be added to the lateral load given herein. Equipment traffic should concurrently be limited to an area at least 3 feet from the shoring face or edge of the slope.

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

9.13 Underground Utilities

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

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

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

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

9.14 Pavement Recommendations

9.14.1 The pavement design recommendations provided herein are based on the State of California Department of Transportation (CALTRANS) design manual. Based on the results of the Atterberg limits testing and our experience in the site vicinity, an R-value of 20 was assumed for design. During grading subgrade samples should be tested to verify the recommendations included in this report remain valid.

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

TABLE 9.14.2
ASPHALT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Asphaltic Concrete, (inches)	Class 2 Aggregate Base, (inches)*	Compacted Subgrade, (inches)*
4.0	2.5	4.5	12.0
5.0	2.5	8.0	12.0
6.0	3.0	10.5	12.0
7.0	4.0	12.0	12.0

95% compaction based on ASTM D1557 Test Method

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

TABLE 9.14.3
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

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

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

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

9.14.5 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.

9.14.6 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.

9.14.7 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.

9.14.8 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.

10. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

10.1 Plan and Specification Review

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

10.2 Construction Observation and Testing Services

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

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

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

11. LIMITATIONS AND CHANGED CONDITIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the test boring drilled at the approximate locations shown on the Site Plan, Figure 1. 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.

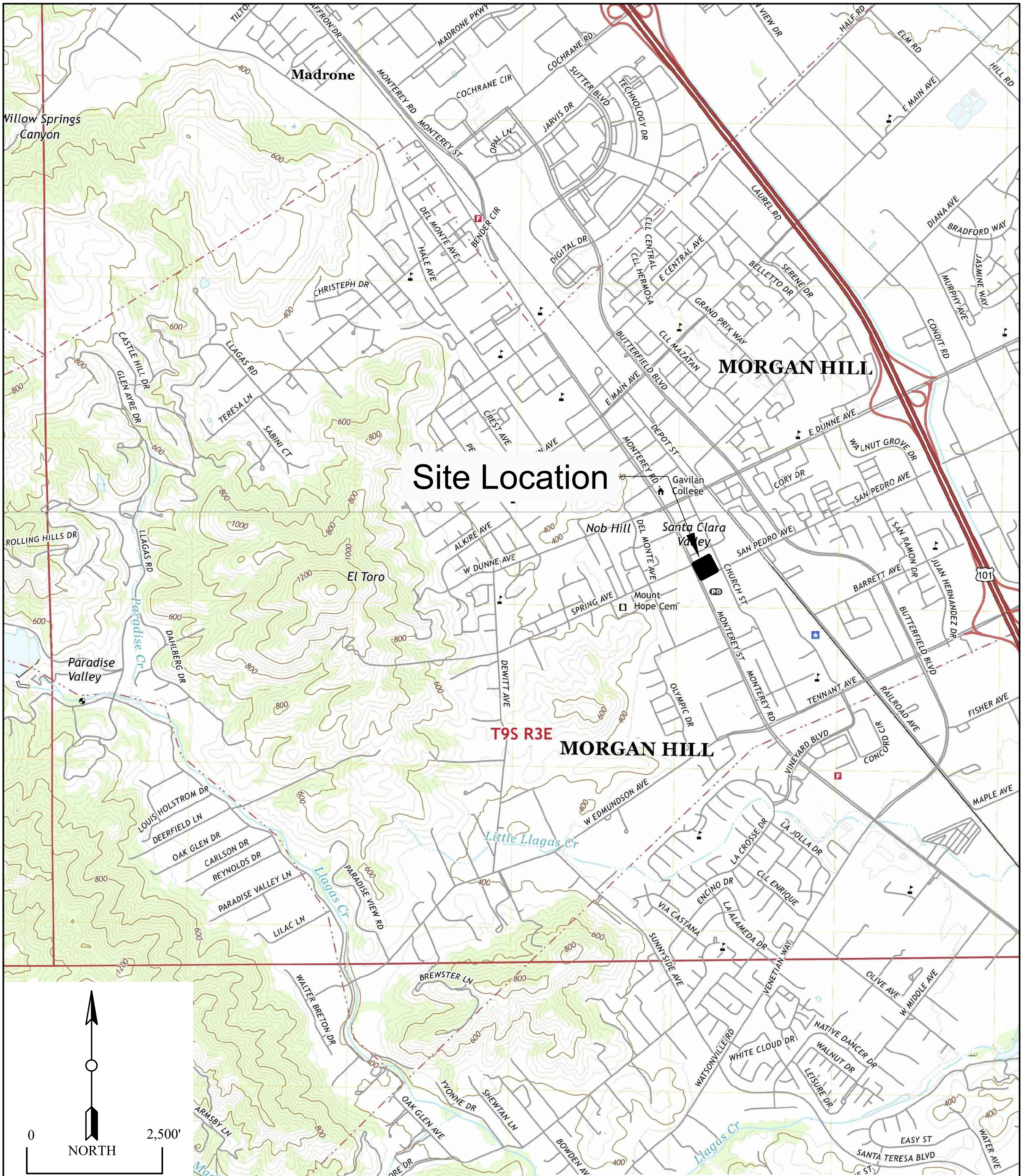
Joshua R. Marroquin, EIT
Geotechnical Project Engineer
Central / Northern California

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

Dean B. Ledgerwood II, CEG
Northern California Geotechnical Manager
CEG 2613

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



Mt Madonna, Morgan Hill, CA. 2018 USGS 7.5' Topo

VICINITY MAP	SCALE: 1 : 2500'	DATE: JUNE 2020	SALEM engineering group, inc.
PROPOSED PETROLEUM STATION 16720 MONTEREY HIGHWAY MORGAN HILL, CALIFORNIA	DRAWN BY: VT	APPROVED BY: DL	
	PROJECT NO. 5-220-0505	FIGURE NO. 1	

SITE PLAN	SCALE: 1"=40'	DATE: JUNE 2020	SALEM engineering group, inc.
PROPOSED PETROLEUM 16720 MONTEREY HIGHWAY MORGAN HILL, CALIFORNIA	DRAWN BY: VT	APPROVED BY: DL	
	PROJECT NO. 5-220-0505	FIGURE NO. 2	

APPENDIX

A

APPENDIX A

FIELD EXPLORATION

Fieldwork for our investigation was conducted on July 15, 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-55 drilling rig and 8-inch diameter hollow stem auger. 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 cuttings, thus, some settlement should be anticipated.

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.

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

SALEM engineering group, inc.

Project Number: 5-220-0505

Date: 07/15/2020

Client: World Oil Corp.

Project: Proposal Petroleum Station

Location: 16720 Monterey Highway, Morgan Hill, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** A. Ghoneim

Drill Type: CME 55 **Elevation:** 337 feet AMSL

Auger Type: 6 5/8 inch Hollow Stem Auger **Initial Depth to Groundwater:** 20 feet

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** 20 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 3.5 inches				
		AB	Aggregate Base = 3.5 inches				
335	4/6 2/6 2/6	SC	Clayey sand; loose, brown, moist, medium to fine-grained.	4	12.6	111.5	
5	8/6 11/6 17/6		Clayey sand with gravel; medium dense, brown, moist, coarse to fine-grained.	28	11.5	116.7	
330							
10	7/6 13/6 13/6		Grades as above.	26	10.7	--	
325							
15	6/6 9/6 14/6		Grades as above; grey.	23	10.2	--	
320							
20	9/6 12/6 16/6		Grades as above; wet.	28	13.0	--	
315	10/6 17/6 23/6		Grades as above; dense.	40	16.5	--	
25			End of boring at 24 feet below site grade due to dense gravel and/or cobbles.				
310							

Notes:

Figure Number A-1

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

SALEM engineering group, inc.

Project Number: 5-220-0505

Date: 07/15/2020

Client: World Oil Corp.

Project: Proposal Petroleum Station

Location: 16720 Monterey Highway, Morgan Hill, CA.

Drilled By: Salem Engineering Group, Inc. **Logged By:** A. Ghoneim

Drill Type: CME 55 **Elevation:** 337 feet AMSL

Auger Type: 6in. Solid Flight Auger **Initial Depth to Groundwater:** 15 feet

Hammer Type: Automatic Trip - 140lbs./30in. **Final Depth to Groundwater:** 15 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0		AC	Asphalt Concrete = 3.5 inches				
		AB	Aggregate Base = 3.5 inches				
335	2/6 4/6 5/6	SC	Clayey sand with gravel; loose, dark brown, moist, medium to fine grained.	9	13.5	--	
5							
330							
10	11/6 14/6 24/6		Grades as above; medium dense, brown.	38	11.2	122.8	
325							
15	6/6 14/6 15/6		Grades as above; wet, with medium gravel.	29	19.2	--	
320			End of boring at 16.5 feet below site grade.				
20							
315							
25							
310							

Notes:

Figure Number A-2

KEY TO SYMBOLS

Symbol Description

Strata symbols

Asphaltic Concrete

Aggregate Base

Clayey sand

Misc. Symbols

Water table during drilling

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

APPENDIX

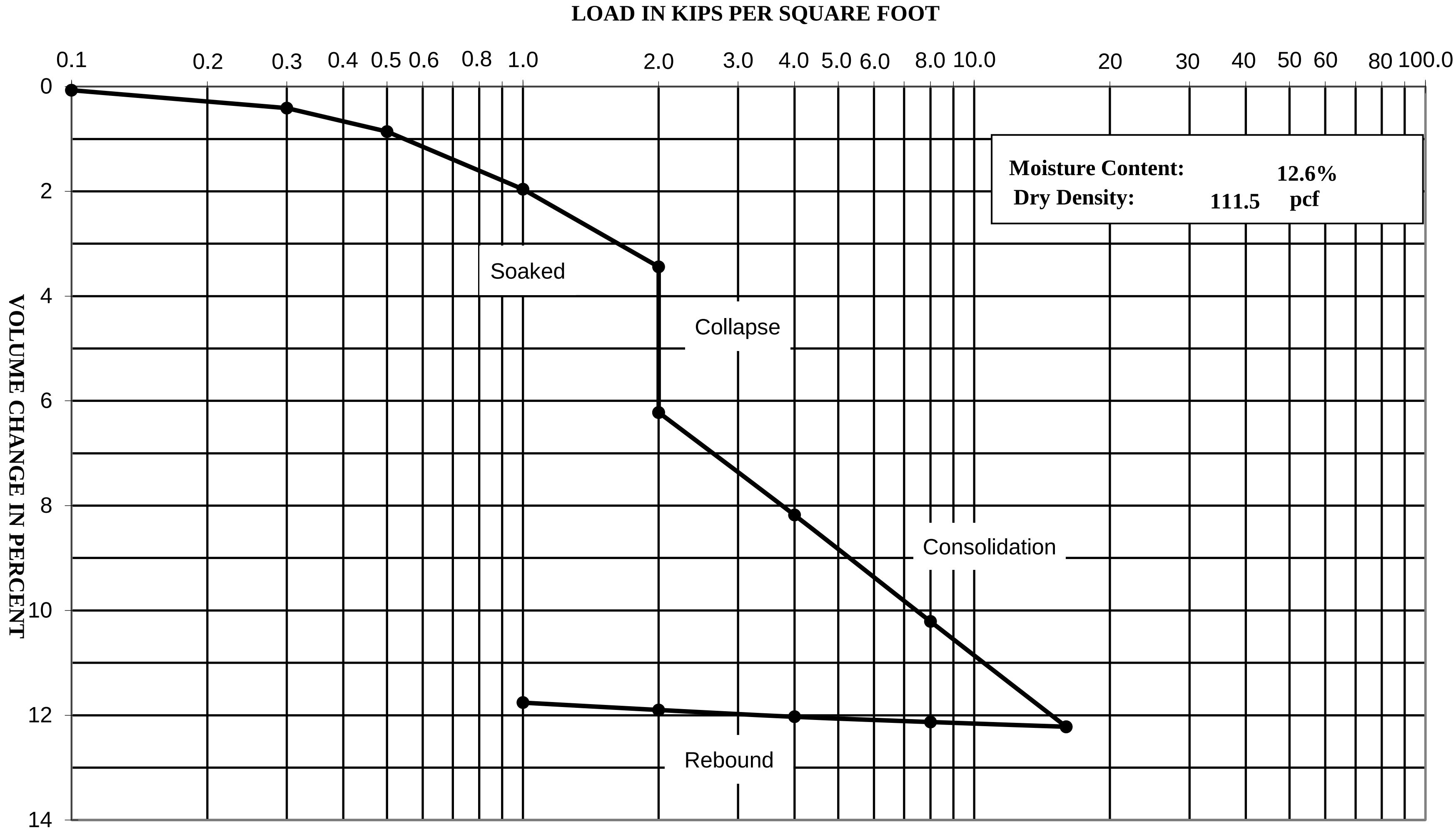
B

APPENDIX B

LABORATORY TESTING

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

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Petroleum Station - Morgan Hill, CA

Project Number: 5-220-0505

Boring: B-1 @ 1.5'

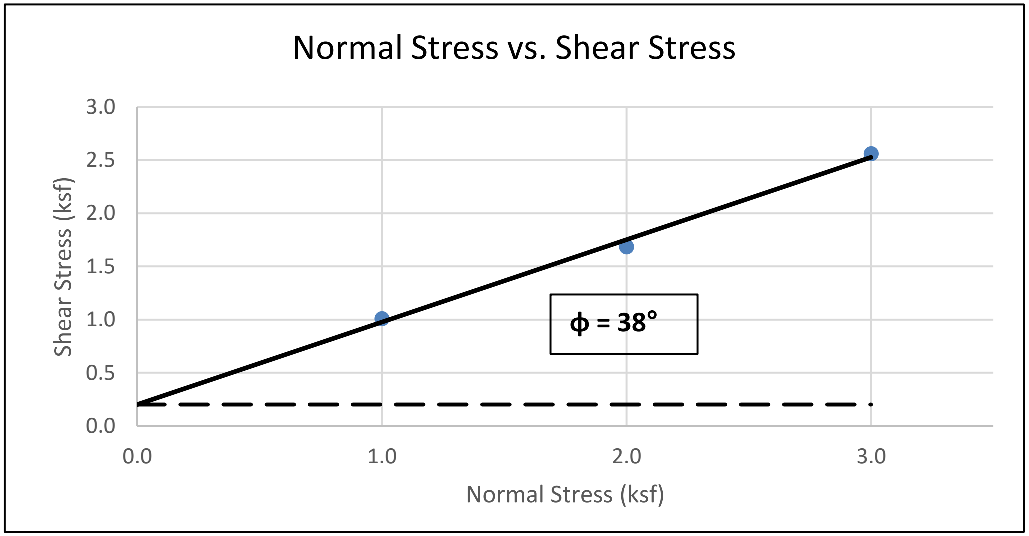
SALEM engineering group, inc.

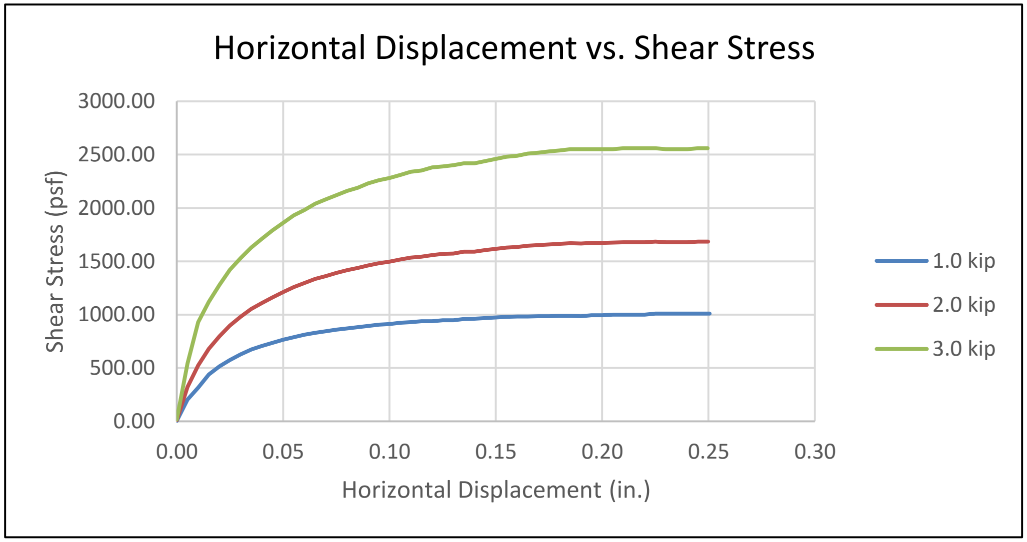
Direct Shear Test (ASTM D3080)

Project Name: Petroleum Station - Morgan Hill, CA
Project Number: 5-220-0505
Client: World Oil Corporation
Boring: B-1 @ 5'
Soil Type: Clayey sand with gravel (SC)
Sample Type: Undisturbed Ring
Tested By: NL
Reviewed By: JRM
Date of Test: 7/23/20
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	2.0 kip	3.0 kip
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	1.01	1.69	2.56

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.930	0.909	0.888
Post-Shear Sample Height (in.)	0.914	0.893	0.873
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	11.5		
Dry Density (pcf)	103.6	104.6	105.6
Saturation %	49.8	51.1	52.4
Void Ratio	0.62	0.61	0.59
Consolidated Void Ratio	0.51	0.46	0.41
Final (post-shear) Values			
Final Moisture Content (%)	21.6	20.8	20.0
Dry Density (pcf)	109.2	111.2	113.3
Saturation %	94.6	101.1	108.8
Void Ratio	0.61	0.55	0.49



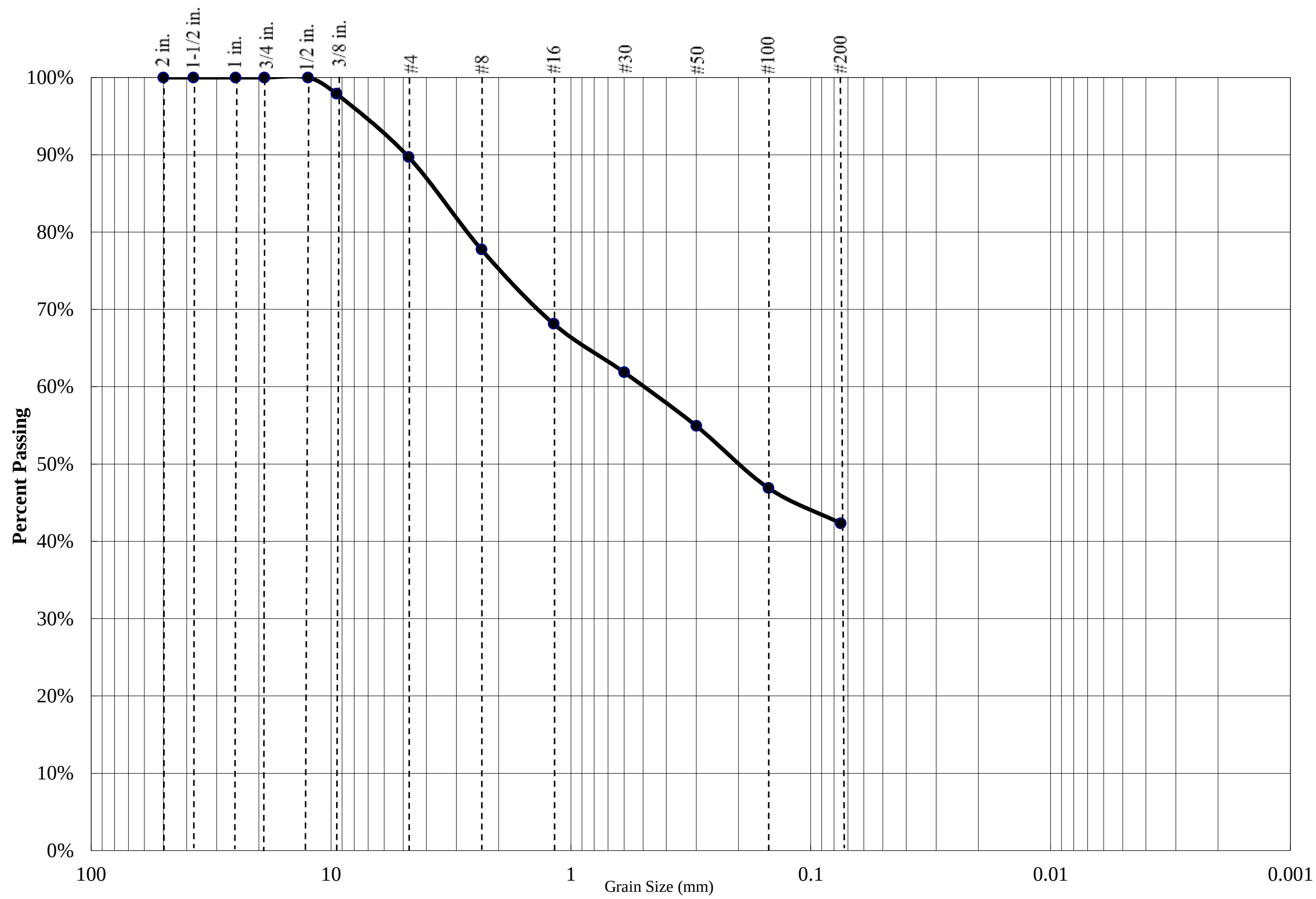


Peak Shear Strength Values	
Slope	0.78
Friction Angle	38
Cohesion (psf)	202

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

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
10%	48%	42%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	97.9%
#4	89.7%
#8	77.8%
#16	68.1%
#30	61.9%
#50	54.9%
#100	46.9%
#200	42.3%

Atterberg Limits					
PL=	15	**LL=**	30	**PI=**	15

Coefficients					
D_{85}=	3.5	**D_{60}=**	0.49	**D_{50}=**	0.2
D_{30}=	n/a	**D_{15}=**	n/a	**D_{10}=**	n/a
C_u=	N/A	**C_c=**	N/A		

USCS CLASSIFICATION
Clayey sand (SC)

Project Name: Petroleum Station - Morgan Hill, CA

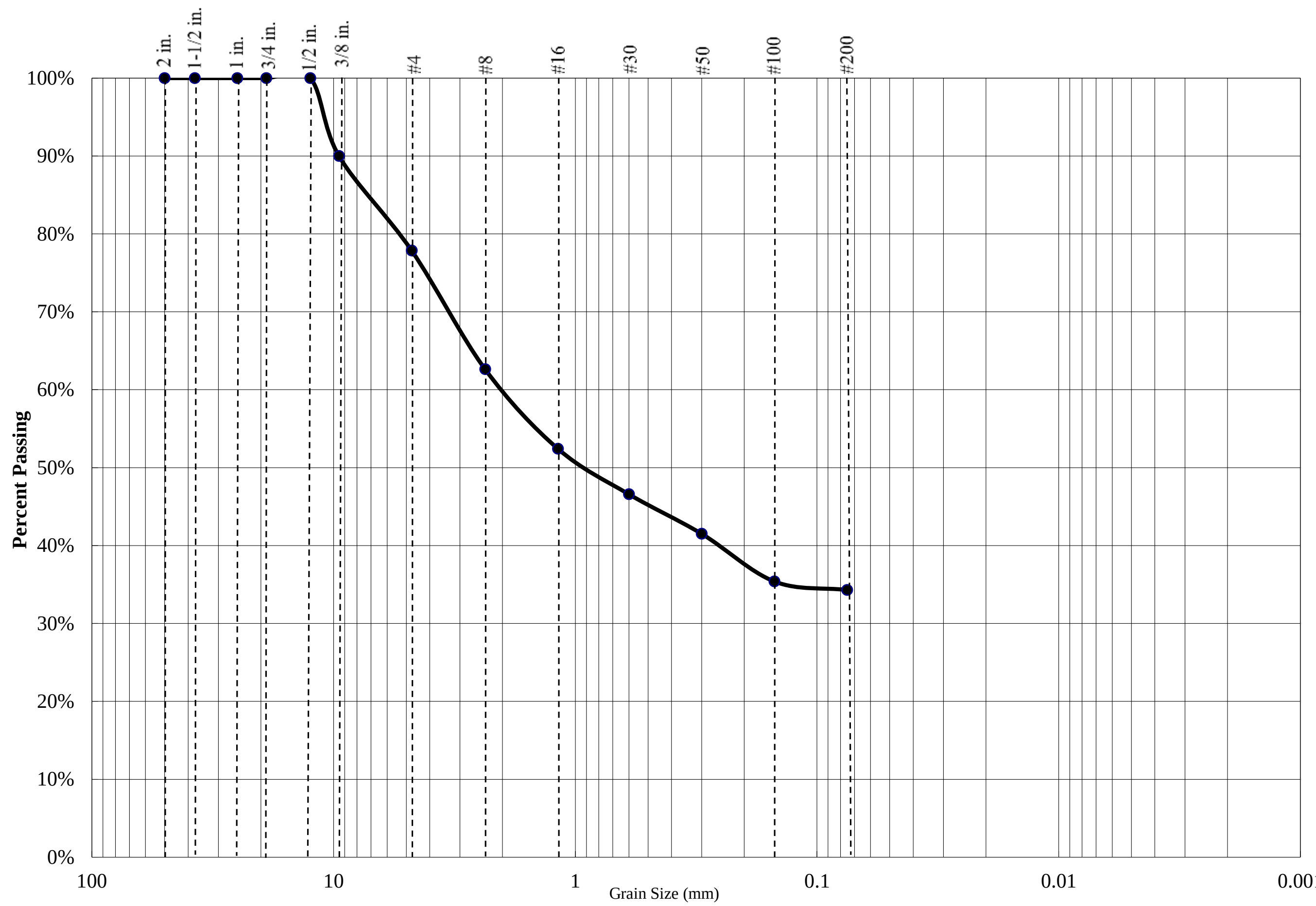
Project Number: 5-220-0505

Boring: B-1 @ 1.5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
22%	44%	34%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	90.0%
#4	77.8%
#8	62.6%
#16	52.4%
#30	46.6%
#50	41.5%
#100	35.4%
#200	34.3%

Atterberg Limits					
PL=	n/a	**LL=**	n/a	**PI=**	n/a

Coefficients					
D_{85}=	7.2	**D_{60}=**	2.1	**D_{50}=**	0.92
D_{30}=	n/a	**D_{15}=**	n/a	**D_{10}=**	n/a
C_u=	N/A	**C_c=**	N/A		

USCS CLASSIFICATION
Clayey sand with gravel (SC)

Project Name: Petroleum Station - Morgan Hill, CA

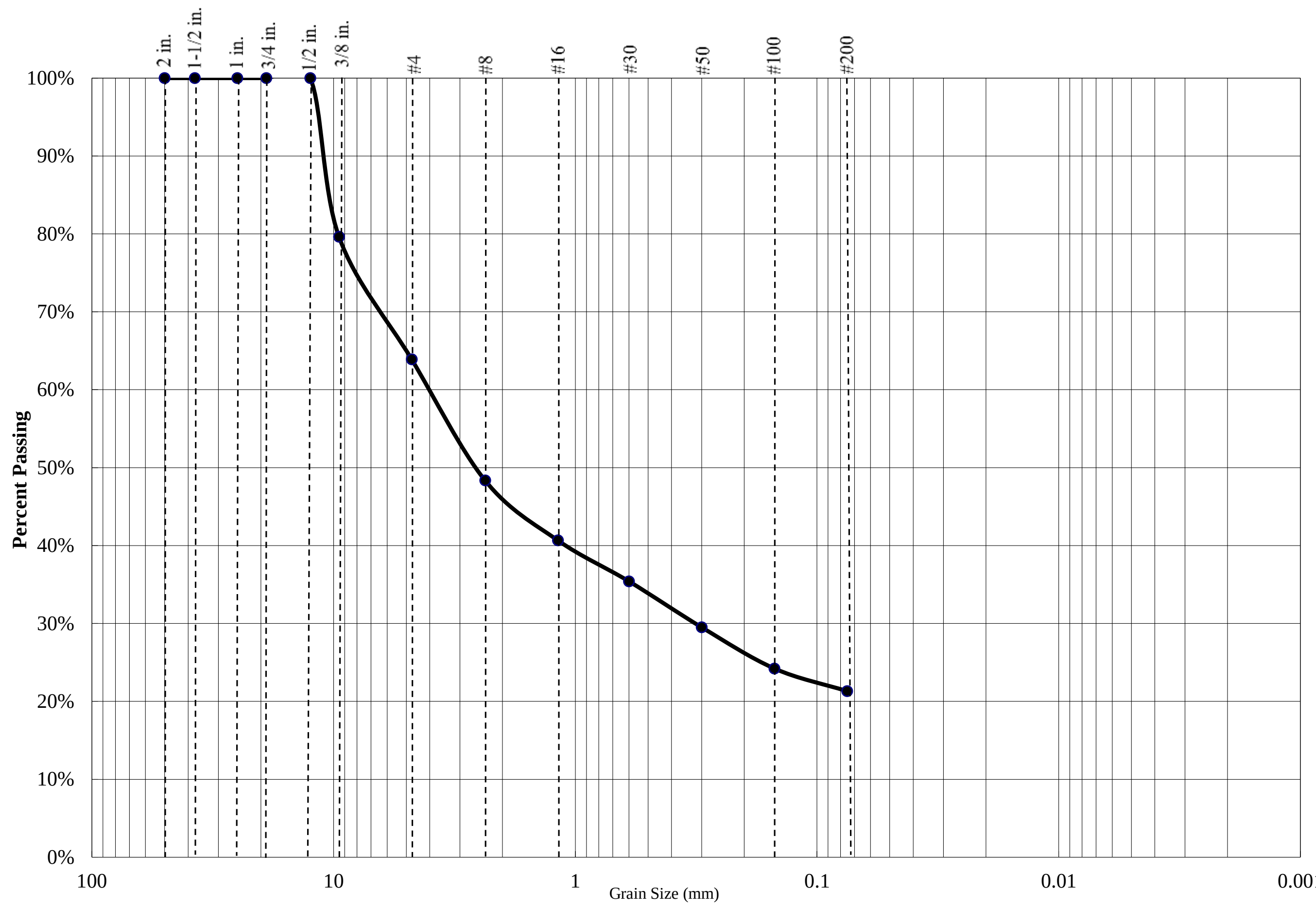
Project Number: 5-220-0505

Boring: B-1 @ 5'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
36%	43%	21%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	79.6%
#4	63.9%
#8	48.3%
#16	40.7%
#30	35.4%
#50	29.5%
#100	24.2%
#200	21.3%

Atterberg Limits					
PL=	n/a	LL=	n/a	PI=	n/a

Coefficients					
D_{85}=	10.1	D_{60}=	4	D_{50}=	2.7
D_{30}=	0.33	D_{15}=	n/a	D_{10}=	n/a
C_u=	N/A	C_c=	N/A		

USCS CLASSIFICATION
Clayey sand with gravel (SC)

Project Name: Petroleum Station - Morgan Hill, CA

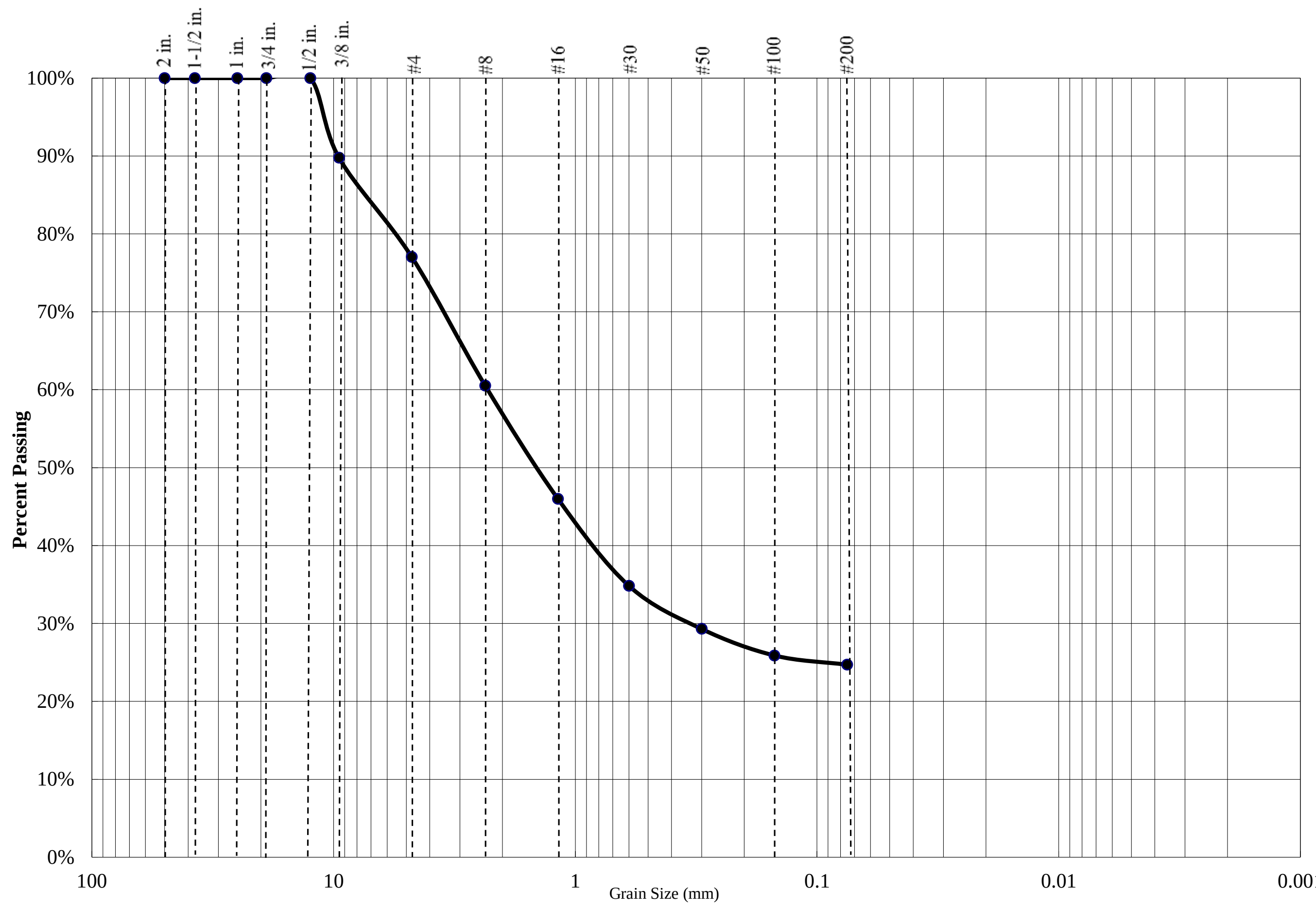
Project Number: 5-220-0505

Boring: B-1 @ 15'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
23%	52%	25%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	89.8%
#4	77.0%
#8	60.5%
#16	46.0%
#30	34.8%
#50	29.3%
#100	25.9%
#200	24.7%

Atterberg Limits					
PL=	n/a	**LL=**	n/a	**PI=**	n/a

Coefficients					
D_{85}=	7.2	**D_{60}=**	2.3	**D_{50}=**	1.6
D_{30}=	0.35	**D_{15}=**	n/a	**D_{10}=**	n/a
C_u=	N/A	**C_c=**	N/A		

USCS CLASSIFICATION
Clayey sand with gravel (SC)

Project Name: Petroleum Station - Morgan Hill, CA

Project Number: 5-220-0505

Boring: B-2 @ 8.5'

SALEM engineering group, inc.

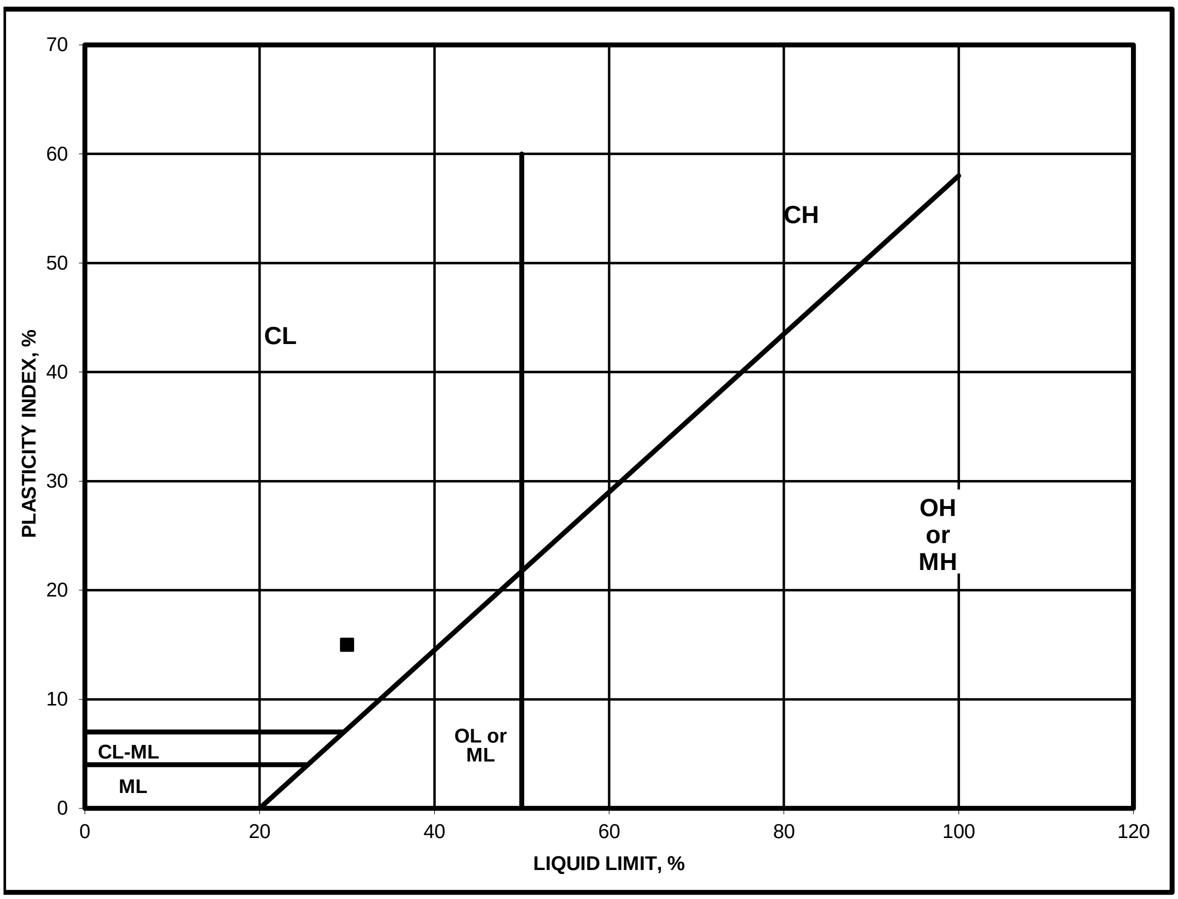
Atterberg Limits Determination
ASTM D4318

Project Name: Petroleum Station - Morgan Hill, CA
Project Number: 5-220-0505
Date Sampled: 7/15/20 Date Tested: 7/23/20
Sampled By: SEG Tested By: HA
Sample Location: B-1 @ 0 - 3'

Run Number	Plastic Limit 1	Plastic Limit 2	Plastic Limit 3	Liquid Limit 1	Liquid Limit 2	Liquid Limit 3
Weight of Wet Soil & Tare	28.12	29.10	28.06	29.51	27.83	28.44
Weight of Dry Soil & Tare	27.18	28.14	27.09	27.46	26.19	26.69
Weight of Water	0.94	0.96	0.97	2.05	1.64	1.75
Weight of Tare	20.85	21.71	20.76	20.72	20.82	20.90
Weight of Dry Soil	6.33	6.43	6.33	6.74	5.37	5.79
Water Content	14.8	14.9	15.3	30.4	30.5	30.2
Number of Blows				26	24	23

Plastic Limit : 15 **Liquid Limit : 30**

Plasticity Index : 15
Unified Soil Classification : CL



SALEM engineering group, inc.

EXPANSION INDEX TEST
ASTM D4829

Project Name: Petroleum Station - Morgan Hill, CA
Project Number: 5-220-0505
Date Sampled: 7/15/20 Date Tested: 7/24/20
Sampled By: SEG Tested By: NL
Sample Location: B-1 @ 0 - 3'
Soil Description: Clayey sand (SC)

Trial #	1	2	3
Weight of Soil & Mold, g.	590.6		
Weight of Mold, g.	187.8		
Weight of Soil, g.	402.8		
Wet Density, pcf	121.5		
Weight of Moisture Sample (Wet), g.	831.0		
Weight of Moisture Sample (Dry), g.	761.2		
Moisture Content, %	9.2		
Dry Density, pcf	111.3		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	48.2		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.0282	0.0289	--	--	0.0307

Expansion Index $_{measured}$ = 30.7
Expansion Index $_{50}$ = 29.7

Expansion Index = 30

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

SALEM engineering group, inc.

CHEMICAL ANALYSIS

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

Project Name: Petroleum Station - Morgan Hill, CA
Project Number: 5-220-0505
Date Sampled: 7/15/20 Date Tested: 7/22/20
Sampled By: SEG Tested By: HA
Soil Description: Clayey sand (SC)

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 0 - 3'	500 mg/kg	25 mg/kg	7.9
1b.	B-1 @ 0 - 3'	490 mg/kg	23 mg/kg	7.9
1c.	B-1 @ 0 - 3'	500 mg/kg	23 mg/kg	7.9
Average:		**497 mg/kg**	**24 mg/kg**	**7.9**

SALEM engineering group, inc.

SOIL RESISTIVITY
CTM 643

Project Name: Petroleum Station - Morgan Hill, CA
Project Number: 5-220-0505
Sample Location: B-1 @ 0 - 3'
Soil Description: Clayey sand (SC)

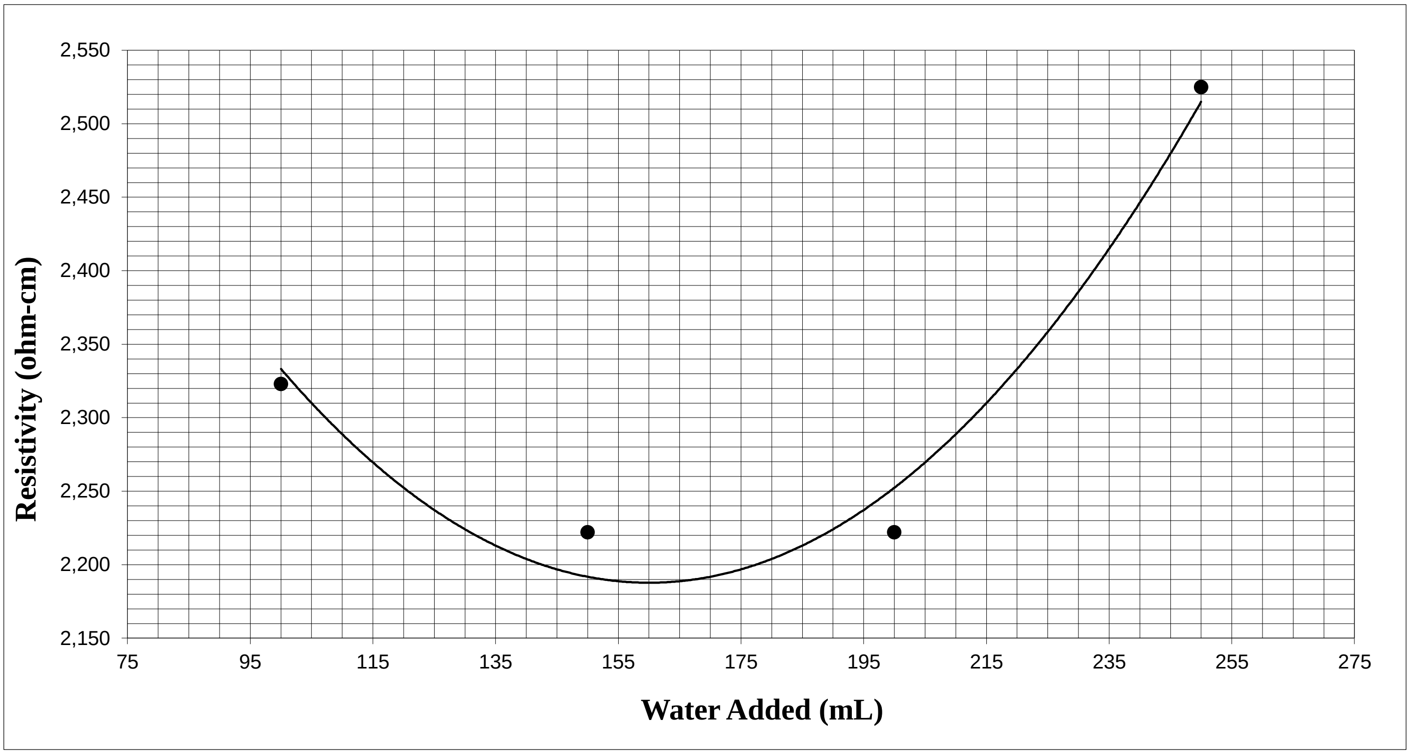
Date Sampled: 7/15/20
Sampled By: SEG
Date Tested: 7/23/20
Tested By: HA

Chloride Content: 24 mg/Kg
Sulfate Content: 497 mg/Kg
Soil pH: 7.9

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	100	2.3	1,000	2,300	2,323
2	150	2.2	1,000	2,200	2,222
3	200	2.2	1,000	2,200	2,222
4	250	2.5	1,000	2,500	2,525



Minimum Resistivity:	**2,188** ohm-cm

SALEM engineering group, inc.

APPENDIX

C

APPENDIX C
GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS

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

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

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

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

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

3.0 TECHNICAL REQUIREMENTS: All compacted materials shall be densified to no less that 90 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 90 percent relative compaction.

Loose soil areas and/or areas of disturbed soil shall be moisture-conditioned as necessary and compacted to 90 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 atmosphere 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.