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SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED SAM'S CLUB PETROLEUM STATION
STORE #6630
7370 WEST 52ND AVENUE
ARVADA, COLORADO

SALEM PROJECT NO. 7-219-1010
DECEMBER 4, 2019

PREPARED FOR:

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Mr. John DeGunya
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Subject: **GEOTECHNICAL ENGINEERING INVESTIGATION**
PROPOSED SAM'S CLUB PETROLEUM STATION
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ARVADA, COLORADO

Dear Mr. DeGunya:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Geotechnical Engineering Investigation report for the proposed petroleum station planned at the existing Sam's Club Store located at 7370 West 52nd Avenue in Arvada, Colorado. The overall site currently supports a retail shopping center stores, while the immediate area to support the petroleum station supports asphaltic concrete pavements used as parking.

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

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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Geotechnical Manager

TABLE OF CONTENTS

1

TABLE OF CONTENTS (cont.)

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED SAM'S CLUB PETROLEUM STATION
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ARVADA, COLORADO

1. PURPOSE AND SCOPE

This report presents the results of our Geotechnical Engineering investigation for the proposed petroleum station planned within the existing Sam's Club shopping center located at 7370 West 52nd Avenue in Arvada, Colorado (see Figure 1, Vicinity Map).

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

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

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

2. SITE LOCATION AND DESCRIPTION

The project site is located at the existing Sam's Club Store located at 7370 West 52nd Avenue in Arvada, Colorado. The Sam's Club shopping center is bound to the north, west, and east by commercial developments supporting asphalt parking, and to the south by interstate 70. The area of the proposed petroleum station is planned in the northern area of the Sam's Club property, south of existing commercial developments.

The immediate project location is bounded by existing asphalt paved parking areas, landscape islands with mature trees, and nearby underground utilities.

The site has an elevation of approximately 5,311 feet above mean sea level (AMSL) based on Google Earth Imagery.

3. PROJECT DESCRIPTION

We understand that development of the site includes construction of a petroleum station of unknown square-footage. It is our understanding the development will include fuel dispensers with associated canopy, underground storage tanks, and a fuel attendant building. The area proposed for the facility is located within Sam's Club #6630 property north of the existing Sam Club warehouse building.

A site grading plan was not available at the time of preparation of this report. Based on the current development of the site, we anticipate that cuts and fills will be on the order of 1 to 2 feet in order to provide a level pad and positive site drainage. In the event that changes occur in the nature or design of the project, the conclusions and recommendations contained in the report will not be considered valid unless the changes are reviewed, and the conclusions of our report are modified.

Based on previous experience, the petroleum station service building will have maximum column loads on the order of 20 kips and maximum wall loads on the order of 1 kip per linear feet. The estimated maximum uniform floor slab live load is 100 psf. the petroleum station canopy, with an assumed multi-column layout, will have a maximum column load of 40 kips.

It is anticipated that isolated landscape areas, asphaltic and Portland cement concrete pavements, and underground utilities will be planned as part of the proposed site development.

The site configuration and locations of proposed improvements are shown on the Site Plan, Figure 2.

4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. On November 15, 2019, a total of three (3) exploratory test borings (B-1 through B-3) were drilled to depths ranging from 15 to 25.5 feet below site grade. The approximate locations of the borings are shown on Figure No. 2, Site Plan. The test borings were advanced using 6-inch diameter hollow stem 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 and stratification lines were approximated on the basis of observations made at the time of drilling. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2487). A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix A. The test boring logs are presented in Appendix A. The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The location of the test borings were determined by measuring from features shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix A should be consulted.

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

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 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 boring logs.

Soil samples were obtained from the test borings at the depths shown on the test 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 cuttings and patched with asphaltic concrete cold patch to match existing site conditions.

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, consolidation, shear strength, resistance value (R-value), gradation, plasticity index, and expansion index. 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 Surface and Subsurface Conditions

The existing pavement sections encountered at each of the three (3) test boring locations included 3 to 4 inches of asphaltic concrete over 3 inches of aggregate base material. Within the asphalt section a paving fabric was encountered.

Below the existing pavement sections, the soils encountered included clayey sands, sandy lean clay, silty sand with gravel, poorly-graded gravel with sand, sandy silt, and clayey gravel to the maximum depth explored of 25½ feet BSG. It should be noted in test boring B-1 that weathered rock material was encountered at a depth beginning at 21 feet BSG.

A consolidation tests resulted in about and 6 percent consolidation under a load of 8 kips per square foot, and when wetted under a load of 2.0 kips per square foot, the sample exhibited less than 1 percent collapse. A direct shear test performed on a clay sample at 3.5 feet BSG, resulted in an internal angle of friction of 16 degrees with a cohesion value of 673 pounds per square foot, respectively. One (1) Atterberg limit test was performed on near surface sample resulted in a plasticity index of 5 and a liquid limit of 22. An expansion index test performed on a near surface sample resulted in an expansion index of 10. An R-value test performed on a sample collected in the upper 3 feet BSG resulted in an R-value of 50.

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 boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was encountered at a depth of 13 feet BSG in test borings B-2 and B-3 during the time of the field exploration. Based on review of available groundwater data from State of Colorado Division of Water Resources, Well name STEPP-1 located 1,400 feet northwest of the property reported a groundwater depth of 6.62 feet BSG in July 14 1993.

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.

In accordance with Walmart requirements, due to the presence of groundwater, a sample was collected at a depth of about 20 feet BSG for BTEX testing. The results of the sample tested resulted in none detected levels of benzene, ethylbenzene, and xylenes (total). Toluene level of 0.108 mg/kg were reported in the sample tested. The results of the sample tested is included in the laboratory testing section 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 sample was detected to be less than 50 mg/kg.

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

TABLE 6.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Dissolved Sulfate (SO_4) in Soil percent by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementitious Materials Type
< 0.005	Not Applicable	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 44 mg/kg. In addition, testing performed on a near surface soil resulted in a minimum resistivity value of 3,117 ohmcentimeters. Based on the results, these soils would be considered to have a “corrosive” potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings).

It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, applicable manufacturer’s recommendations for corrosion protection of buried metal pipe be closely followed. Additional corrosion testing for minimum resistivity may need to be performed if required by the pipe manufacturer.

7. GEOLOGIC SETTING

Based on review of the USGS Geologic map of the Arvada quadrangle, Adams, Denver, and Jefferson Counties, Colorado, dated 1979[1], the site is located in an area mapped as Qpp “Post-Piney Creek Alluvium (Holocene)”. The geologic unit is described as “Light to dark-grayish-brown clay, silt, sand, and small amounts of gravel. Dark-brown and dark-bluish-black humic bog clays, interbedded in places with sand and silt. Mapped chiefly in Clear Creek and South Platte River valleys, but thin deposits are present in most minor tributary stream valleys.” The subsurface conditions encountered appear typical of those found in the geologic region of the site.

8. GEOTECHNICAL AND GEOLOGIC

8.1 Faulting and Seismicity

Based on the lack of dominant active faults and seismogenic structures within 100 miles of the site area, the subject site is considered subject to relatively low seismicity. Soils on site are classified as Site Class 5 in accordance with the 2015 International Building Code. The proposed structures are determined to be in Seismic Design Category B.

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 39.7898° North; site longitude is 105.0780° West. There are no “active faults” within 100 miles of the site.

8.2 Surface Fault Rupture

No active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low.

8.3 Ground Shaking

Based on the 2015 IBC, a Site Class C was selected for the site based on soil conditions with standard penetration resistance, N-values, averaging between greater than 50 blows per foot. Table 9.6.1 includes design seismic coefficients and spectral response parameters, based on the 2015 International Building Code (IBC) for the project foundation design.

[1] Lindvall, R.M, 1979, Geologic map of the Arvada quadrangle, Adams, Denver, and Jefferson Counties, Colorado: U.S. Geological Survey, Geologic Quadrangle Map GQ-1453, scale 1:24,000

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 0.115g (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. However, liquefaction has occurred in soils other than clean sand.

A seismic hazard, which could cause damage to the proposed development during seismic shaking, is the post-liquefaction settlement of the liquefied sands. Based on the low peak ground acceleration value, liquefaction is not a concern for the subject site.

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 lack of faults in the area and low peak ground acceleration value for the site, we judge the likelihood of lateral spreading to be negligible.

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 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, as outlined in the project description section.

9.1.2 Below the existing pavement sections, the soils encountered included clayey sands, sandy lean clay, silty sand with gravel, poorly-graded gravel with sand, sandy silt, and clayey gravel to the maximum depth explored of 25½ feet BSG. It should be noted in test boring B-1 that weathered rock material was encountered at a depth beginning at 21 feet BSG.

9.1.3 Based on the results of the Atterberg Limits testing and Expansion Index testing performed, the near surface soils have a very low expansion potential. When compacted as engineered fill, the near surface soils have excellent pavement support characteristics.

9.1.4 Provided the site is graded in accordance with the recommendations of this report and foundations constructed as described herein, we estimate that total settlement due to static loads utilizing conventional shallow foundations of less than 1-inch in 40 feet.

9.1.5 Based on the chemistry testing performed, the near surface soils have 'negligible' potential for sulfate attack on concrete and are considered "corrosive" to buried metal objects.

9.1.6 To minimize the potential soil movement due to settlement, and provide uniform support for new foundations, the building pad area and over-build zone should be prepared as recommended in section 9.3 of this report.

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

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

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 All site drainage should be collected and transferred away from improvements in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site (with the exception of designed bio-swale areas), and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers are not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within 5 feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

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.

9.2.4 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2 percent away from structures.

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 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 pre-construction 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, 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 The immediate project site is currently developed. Site preparation should begin with removal of existing surface/subsurface structures, underground utilities (as required), disturbed soil, any existing uncertified/undocumented fill, and debris. It is expected demolition activities will disturb the surficial subgrade. Excavations or depressions resulting from demolition, site clearing operations, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report.

9.3.5 At the time of our investigation the site was paved with asphalt, however, if vegetation is encountered 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 but is not likely for this site. The stripped vegetation will not be suitable for use as Engineered Fill. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site

9.3.6 Although none were noted, existing trees within the building pad should be removed and their root systems should be thoroughly cleared of root balls as well as isolated roots greater than ¼-inch in diameter. The root system removal may disturb a significant quantity of soil. Following tree removal, all loose and disturbed soil should be removed from the tree wells. Any areas or pockets of soft or loose soils, void spaces made by burrowing animals, undocumented fill, or other disturbed soil (i.e. soil disturbed by root removal) that are encountered, should be excavated to expose approved firm native material. Care should be taken during site grading to mitigate (e.g. excavate and compact as engineered fill) all soil disturbed by demolition and tree removal activities.

9.3.7 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, including footings and non-cantilevered overhangs carrying structural loads, where feasible.

9.3.8 To minimize the potential soil movement due to settlement, and provide uniform support for the proposed building it is recommended that over-excavation extend to a minimum of 18 inches below preconstruction site grade, 12 inches below the bottom of proposed foundations, or the depth required to remove undocumented fill/disturbed subgrade, whichever is greater. The resulting bottom of over-excavation shall be scarified to a depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to between 1 and 3 percent above optimum moisture, and compacted to a minimum of 95 percent relative compaction per ASTM D689. 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 footings, where feasible.

9.3.9 Interior slab on grade should be supported on a minimum of 6 inches of CDOT Class 6 Aggregate Base Coarse over the depth of engineered fill recommended below foundations.

9.3.10 If elected to support canopy structures on shallow spread foundations, it is recommended that over-excavation extend to a minimum of 24 inches below preconstruction site grade, 24 inches below the bottom of proposed foundations, or the depth required to remove undocumented fill/disturbed subgrade, whichever is greater. The resulting bottom of over-excavation shall be scarified to a depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to between 1 and 3 percent above optimum moisture, and compacted to a minimum of 95 percent relative compaction per ASTM D689. The horizontal limits of the over-excavation should extend throughout the footing over-build zone, laterally to a minimum of 5 feet beyond the outer edges of the proposed footings, where feasible.

9.3.11 Areas of exterior concrete slabs on grade located outside the building pad over-build zone, should be prepared by over-excavation to a minimum of 12 inches below existing grade or 12 inches below the bottom of the recommended aggregate base section, whichever is greater. The zone of over-excavation should extend a minimum of 3 feet beyond these improvements. These exposed bottom of excavation should be moisture conditioned to between 1 and 3percent above optimum and compacted as engineered fill. Exterior concrete slabs on grade should be supported on a minimum of 4 inches of Class 6 Aggregate Base Coarse over subgrade soils prepared as recommended above.

9.3.12 After stripping, areas of proposed asphaltic concrete pavements should be over-excavated to the bottom of the proposed aggregate base layer. The over-excavation should extend horizontally to a minimum of 3 feet beyond the limits of these improvements. The bottom of excavation should be scarified 12 inches and compacted as engineered fill. The upper 12 inches of final pavement subgrade, should be compacted to a minimum of 98 percent relative compaction per ASTM D698.

9.3.13 Areas of miscellaneous lightly loaded foundations, such as screen walls, retaining walls, etc., should be over-excavated to the bottom of foundations, 18 inches below existing site grade, or the depth required to remove undocumented fill, whichever is deeper. The resulting bottom of over-excavation should be scarified to a depth of at least 12 inches, worked until uniform and free from large clods, moisture-conditioned to between 1 and 4 percent above optimum moisture, and compacted to a minimum of 95 percent relative compaction (ASTM D698). Horizontal limits of over-excavation should extend a minimum of 5 feet beyond all sides of the foundations

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

9.3.15 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 The on-site soils were noted to be moist to very moist. Therefore, the Contractor should anticipate the need for drying/aeration of the on-site materials prior to compaction as engineered fill.

9.3.17 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 in the event that wet soil of free water is encountered, dewatering and/or stabilization of the excavation may be required. It should be noted that 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.

9.3.18 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 or near the optimum moisture content by having the subgrade soils scarified and aerated or mixed with drier soils prior to compacting. However, the drying process may require an extended period of time and delay the construction operation. To expedite the stabilizing process, crushed rock may be utilized for stabilization provided this method is approved by the owner for the cost purpose.

If the use of crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of ¾-inch to 1-inch crushed rocks. The thickness of the rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of crushed rock material will provide a stable platform. It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock. A layer of geofabric is recommended to be placed on top of the compacted crushed rock to minimize migration of soil particles into the voids of the crushed rock, resulting in soil movement. Although it is not required, the use of geogrid (e.g. Tensar BX 1100, 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.

9.3.19 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 should 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.

9.4 Soil and Excavation Characteristics

9.4.1 Based on the soil conditions encountered in our borings, the on-site soils will require moderate excavation effort utilizing conventional equipment. Excavations greater than 10 feet BSG should be anticipated to encounter very dense, excavation resistant material.

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 Prior to fill placement, a representative of Salem Engineering Group, Inc. should inspect the bottom of the excavation to verify whether additional excavation will be required. Limits of removal and compaction should extend 5 feet beyond structural elements. Fill material should be worked until uniform and free from large clods, moisture-conditioned to above optimum moisture, and compacted to a minimum of 90 percent of maximum density based on ASTM Test Method D1557.

9.4.4 The near surface soils identified as part of our investigation are, generally, moist to very 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 Engineered Fill

9.5.1 On-site soils are considered for use as engineered fill provided these materials do not contain deleterious matter, organic material, or material larger than 3 inches in maximum dimension.

9.5.2 Imported Non-Expansive Engineered Fill soil, should be well-graded, low-to-non-expansive slightly cohesive silty sand or sandy silt. A clean sand or very sandy soil is not acceptable for this purpose. A sandy soil will allow the surface water to drain into the expansive clayey soils below, which may result in unacceptable swelling. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 9.5.2.

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	< 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 between 1 and 3 percent above optimum moisture content, and compacted to at least 95 percent relative compaction ASTM D698.

9.5.5 Import Engineered Fill, if selected, should be placed, moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction (ASTM D698).

9.5.6 The preferred materials specified for Imported 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.

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

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

9.5.9 All engineered fill soils placed at depths greater than 5 feet BSG should be moisture conditioned to above optimum and compacted to a minimum of 98 percent relative compaction (ASTM D698).

9.5.10 CDOT Class 6 Aggregate Base shall meet the minimum requirements of Section 703 of the Colorado Department of Transportation Standard Specifications for Road and Bridge Construction (2019 Edition). Prior to importing, the Contractor should provide documentation that the aggregate base meets the requirements for Class 6 Aggregate Base to the Owner and Salem for review. All aggregate base should be compacted to a minimum of 98 percent relative compaction (ASTM D698).

9.5.11 Open graded gravel and rock material (i.e. ¾ inch or ½ inch crushed gravel) should not be used as backfill including utility trenches. If required by local agency or for use in subgrade stabilization, to prevent migration of fines, open graded materials should be fully encapsulated in a geotextile fabric such as Mirafi 140N or equivalent. Open graded rock should be placed in loose lifts no greater than about 6 to 8 inches, and vibrated in-place to a firm non-yielding condition.

9.6 Seismic Design Criteria

9.6.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2015 IBC, our recommended parameters are shown below. These parameters were determined using the Office of Statewide Health Planning and Development (OSHPD) (https://seismicmaps.org/) in accordance with the ASCE 7-10. The Site Class was determined based on the soils encountered during our field exploration.

TABLE 9.6.1
SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2010 ASCE 7 or 2015 IBC Reference
Site Coordinates (Datum = NAD 83)		39.7898 Lat -105.0780 Lon	
Site Class	--	C	ASCE 7 Table 20.3
Soil Profile Name	--	"Very Dense Soil and Soft Rock"	ASCE 7 Table 20.3
Risk Category	--	II	IBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.200	ASCE 7-10 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.115	ASCE 7 Table 11.8-1
Seismic Design Category	SDC	B	ASCE 7 Table 11.6-1 & 2
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	0.188 g	IBC Figure 1613.3.1(1-8)

Seismic Item	Symbol	Value	2010 ASCE 7 or 2015 IBC Reference
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.059 g	IBC Figure 1613.3.1(1-8)
Site Class Modified Site Coefficient	F_a	1.200	IBC Table 1613.3.3(1)
Site Class Modified Site Coefficient	F_v	1.700	IBC Table 1613.3.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	0.226 g	IBC Equation 16-37
MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$	S_{M1}	0.101 g	IBC Equation 16-38
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	0.150 g	IBC Equation 16-39
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	0.067 g	IBC Equation 16-40

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 Foundation Recommendations

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 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 structure should be continuous with a minimum width of 12 inches, and isolated column footings should have a minimum width of 18 inches. Based on the Climatic and Geographic Design Criteria a frost depth for Arvada, Jefferson County, foundations should extend to the minimum depth of 36 inches below site grade to avoid frost damage.

9.7.3 Shallow spread foundations may be designed based on an allowable bearing capacity of 2,500 pounds per square foot (dead plus live load). This value may be increased by 1/3 due to wind and/or seismic design.

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

9.7.5 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 250 pounds per cubic foot acting against the appropriate vertical native 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.

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

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

9.7.8 The footing excavations should not be allowed to dry out any time prior to pouring concrete. The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Prior to placing rebar reinforcement, foundation excavations should be evaluated by a representative of SALEM for appropriate support characteristics and moisture content. Moisture conditioning may be required for the materials exposed at footing bottom, particularly if foundation excavations are left open for an extended period.

9.8 Interior Concrete Slabs-on-Grade

9.8.1 For non-structural interior concrete slabs on grade, slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading. We recommend that interior non-structural slabs-on-grade be at least 4 inches thick and underlain by 6 inches of compacted Class 6 Aggregate Base Course over the depth of engineered fill recommended below foundations.

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

9.8.3 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.

9.8.4 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.

9.8.5 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.

9.8.6 Exterior finish grades should be sloped at a minimum of 1 to 1½ percent away from all interior slab areas to preclude ponding of water adjacent to the structures and should be maintained throughout the life of the structure. Ponding of water should not be allowed adjacent to the structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed. In addition, ventilation of the structure is recommended to reduce the accumulation of interior moisture.

9.8.7 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.

9.8.8 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.

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

9.8.10 Avoid use of stakes driven through the vapor retarder.

9.8.11 The recommendations of this report are intended to reduce the potential for cracking of slabs. 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. 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.

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

9.9 Exterior Slabs on Grade

9.9.1 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 6 Aggregate Base

Course over subgrade soils prepared in accordance with the recommendations in section 9.3 of this report.

9.9.2 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.

9.9.3 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.

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

9.10 Cast In Drilled Hole Pier Foundations for Canopies

9.10.1 If elected, cast in drilled hole pier foundations for the canopy footings should have a minimum embedment depth of 10 feet below the lowest adjacent grade.

9.10.2 Casing will be required if groundwater is encountered. Casing should be bedded into the soil near the design depth prior to placement of the reinforcing steel and concrete, and casing extraction. If free water is encountered, concrete should be placed from the bottom of the CIDH pier.

9.10.3 The upper 1 foot of CIDH piers should be neglected in design. Drilled piers can be designed using an allowable sidewall friction of 300 pounds per square foot. This value is for dead-plus-live loads. This value is for dead-plus-live loads. An increase of one-third may be applied when using the alternate load combinations that include wind or earthquake loads.

9.10.4 Uplift loads can be resisted by drilled piers using an allowable sidewall friction of 240 pounds per square foot of the surface area and the weight of the drilled piers.

9.10.5 The total settlement of the drilled piers is not expected to exceed ½ inch. Most of the settlement is expected to occur during construction as the loads are applied.

9.10.6 The CIDH drilled piers may be designed for an allowable lateral capacity of 250 pounds per square foot per foot of depth starting from 1 foot below the lowest adjacent grade to a maximum of 2,500 psf. These values may be increased by one-third when using the alternative load combinations that include wind or earthquake loads. The lateral loading criteria is based on the assumption that the load application is applied at the ground level and flexible cap connections applied. Provided the piers are spaced greater than three (3) pier diameters the lateral bearing pressure may be assumed to act over two pier diameters due to passive arching.

9.11 Lateral Earth Pressures and Frictional Resistance

9.11.1 Lateral earth pressures, friction coefficient, and in-place density of soils against footings and walls are summarized in the Table 9.11.1 below.

TABLE 9.11.1
GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Soil Equivalent Fluid Pressure, pcf
Active Pressure, Drained	50
At-Rest Pressure, Drained	73
Allowable Passive Pressure	250
Related Parameters	
Allowable Coefficient of Friction	0.30
Minimum Unit Weight (lbs/ft^3) [γ_{min}]	105
Maximum Unit Weight (lbs/ft^3) [γ_{max}]	130

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 should 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 (Section 9.6.1 above)
H = Wall Height

9.12 Retaining Walls

9.12.1 Retaining and/or below grade walls should be drained with either perforated pipe encased in free-draining gravel or a prefabricated drainage system. The gravel zone should have a minimum width of 12 inches wide and should extend upward to within 12 inches of the top of the wall. The upper 12 inches of backfill should consist of native soils, concrete, asphaltic-concrete or other suitable backfill to minimize surface drainage into the wall drain system. The gravel should conform to Section 801 of the current Colorado Standard Specifications.

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

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

9.12.4 The top of the perforated pipe should be placed at or below the bottom of the adjacent floor slab or pavements. The pipe should be placed in the center line of the drainage blanket and should have a minimum diameter of 4 inches. Slots should be no wider than 1/8-inch wide, while perforations should be no more than ¼-inch in diameter.

9.12.5 If retaining walls are less than 6 feet in height, the perforated pipe may be omitted in lieu of weep holes on 4 feet maximum spacing. The weep holes should consist of 4-inch diameter holes (concrete walls) or unmortared head joints (masonry walls) and placed no higher than 18 inches above the lowest adjacent grade. Two 8-inch square overlapping patches of geotextile fabric should be affixed to the rear wall opening of each weep hole to retard soil piping.

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

9.13 Temporary Excavations

9.13.1 We anticipate that the majority of the dense site soils will be classified as OSHA "Type B" 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 OSHA standards. The contractor should have a OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and make appropriate recommendations where necessary.

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

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

9.13.4 Open, unbraced excavations in undisturbed soils should be made according to the slopes presented in Table 9.13.4 below.

TABLE 9.13.4
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

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

9.13.6 Braced shorings should be designed for a maximum pressure distribution of 40H, (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.

9.13.7 The excavation and shoring recommendations provided herein are based on soil characteristics derived from the borings within the area. Variations in soil conditions will likely be encountered during the excavations. SALEM 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.

9.14 Underground Utilities

9.14.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than 3 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches and compacted to at least 90 percent relative compaction (ASTM D1557) or 95 percent ASTM D698 at or above optimum moisture content. The upper 12 inches of trench backfill within asphalt or concrete paved areas should be moisture conditioned to at or above optimum moisture content and compacted to at least 95 percent relative compaction (ASTM D1557).

9.14.2 The contractor should anticipate that screening of excavated material from trench excavations will be required to produce material suitable for backfill of utilities.

9.14.3 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.

9.14.4 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.

9.14.5 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.

9.15 Pavement Thickness Design

9.15.1 New pavement subgrade soils should be prepared as recommended in section 9.3 of this report.

9.15.2 Based upon the site soil conditions and the R-Value test result, the table below presents minimum sections recommended for flexible asphaltic concrete pavement design. One (1) Resistance Value (R-Value) tests RV-1, performed on a near surface sample resulted in an R-value of 50. Based on the R-value test performed, a correlated resilient modulus of subgrade (M_{RSG}) of 12,000 was estimated for design.

9.15.3 The following table includes pavement design recommendations based on AASHTO Pavement Design Procedure (20 year design life), 18 kip Equivalent Single Axel Loads for Standard Duty (ESAL=2,200) and Heavy Duty (ESAL= 18,000) pavements, a reliability factor of 85%, initial serviceability factor of 4.2, terminal serviceability of 2.0, an overall standard deviation of 0.45 for flexible pavements and 0.35 for rigid pavements.

TABLE 9.15.3.1
ASPHALT CONCRETE PAVEMENT THICKNESSES

Pavement Area	ESAL	Asphaltic Concrete, (inches)*	Graded Aggregate Base Course, (inches)**	Compacted Subgrade, (inches)**
Standard Duty (Sam's Club Petroleum Station)	2,200	3.0	4.0	12.0
Heavy Duty (Sam's Club Petroleum Station)	18,000	4.0	4.0	12.0

**** 1" wearing surface over tack coat over 2" inch binder course over prime coat***

***** 98% compaction based on ASTM D698 Test Method***

The following recommendations are for light-duty and heavy-duty Portland Cement Concrete pavement sections. Based on the R-value test performed, a correlated k-value corresponding to modulus of subgrade reaction of 200 pounds per cubic inch per inch, was estimated for design.

TABLE 9.15.3.2
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

Pavement Area	ESAL	Portland Cement Concrete, (inches)*	Graded Aggregate Base Course, (inches)**	Compacted Subgrade. (inches)**
Standard Duty (Sam's Club Petroleum Station)	2,200	5.0	4.0	12.0
Heavy Duty (Sam's Club Petroleum Station)	18,000	6.0	4.0	12.0

**** Minimum Compressive Strength of 4,000 psi***
***** 98% compaction based on ASTM D698 Test Method***

9.15.4 Asphalt concrete should conform to Division 400 of the Colorado Standard Specifications for Road and Bridge Construction.

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

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

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

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

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 borings excavated at the approximate locations shown on the Site Plan, Figure 1. The report does not reflect variations which may occur between borings. The nature and extent of such variations may not become evident until construction is initiated.

If variations then appear during construction, 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.

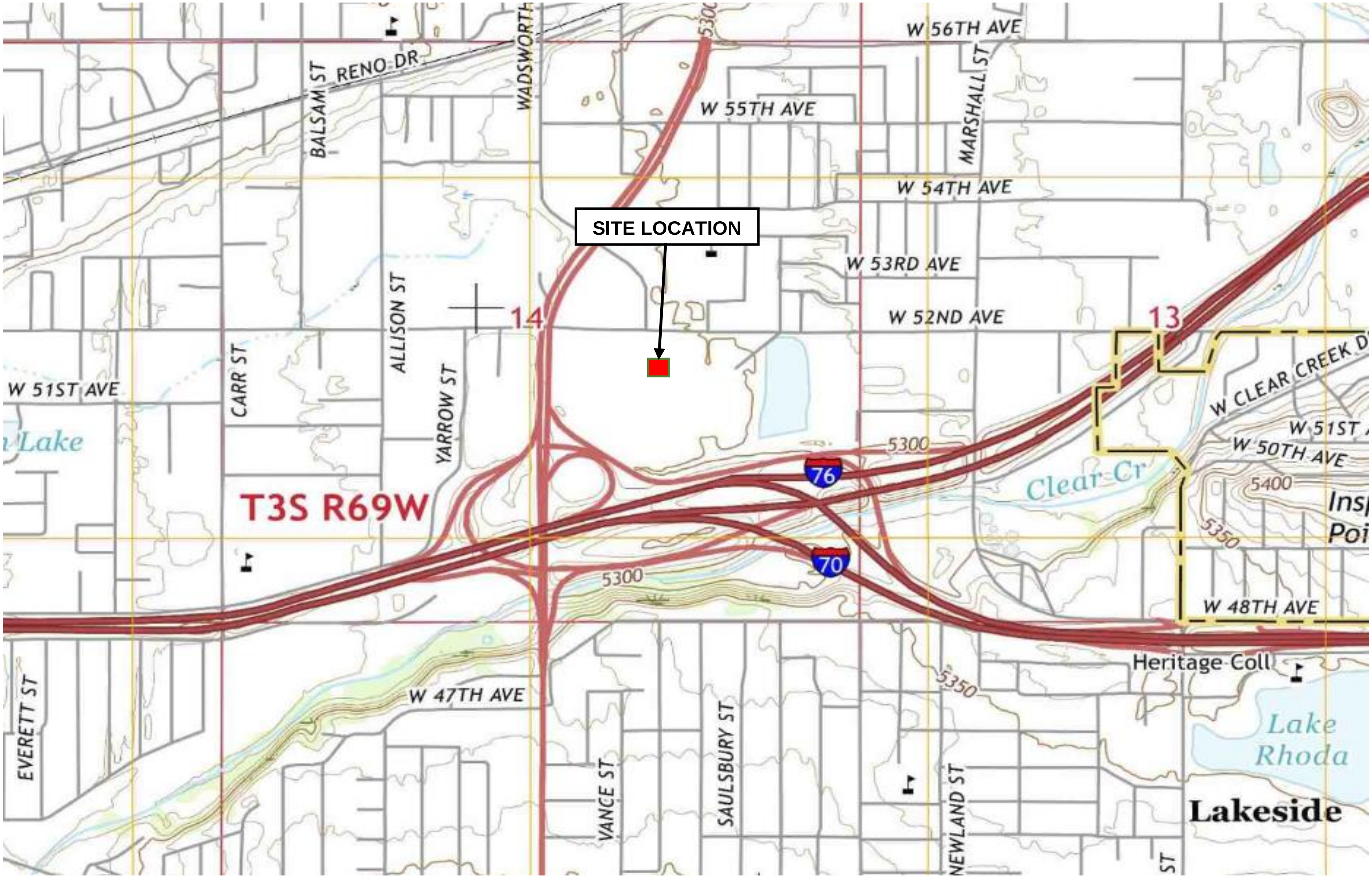
Joshua R Marroquin, EIT
Geotechnical Staff Engineer

Dean B. Ledgerwood II
Geotechnical Manager

COLORADO LICENSED
RIFAAT S. SALEM
45178
PROFESSIONAL ENGINEER

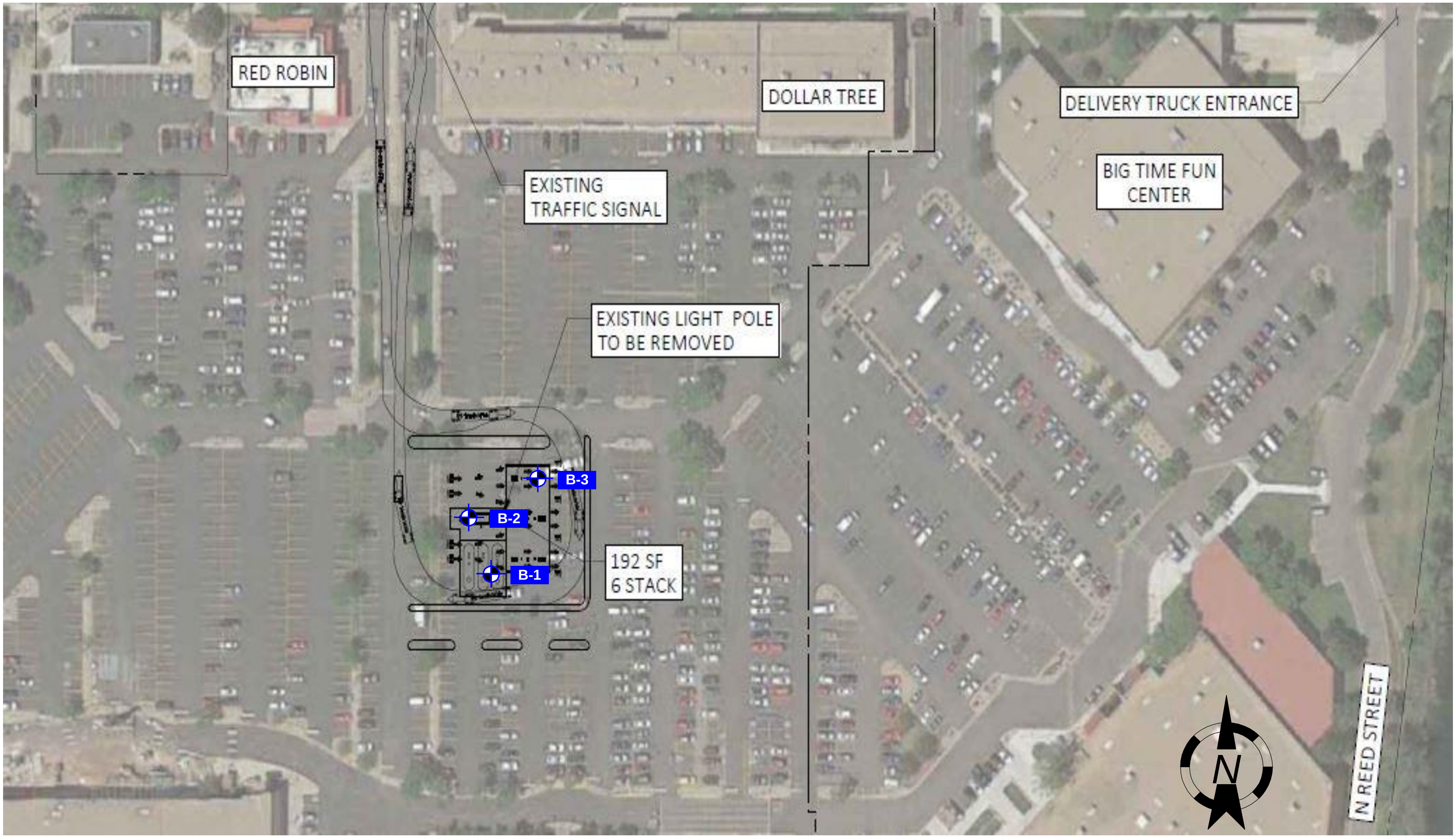
R. Sammy Salem, MS, PE
Principal Managing Engineer
PE No. 45178 – Expires 04/30/2021

Date Signed: 12 / 04 / 2019



Source Image: U.S. Geological Survey, Arvada, CO. 7.5' Quadrangle, 2016 (US Topo)

VICINITY MAP **GEOTECHNICAL ENGINEERING INVESTIGATION** **Proposed Sam's Club Petroleum Station** **7370 W. 52nd Avenue** **Arvada, California**	SCALE: NOT TO SCALE	DATE: 11/29/2019	SALEM engineering group, inc.
	DRAWN BY: VT	APPROVED BY: JRM	
	PROJECT NO. 7-219-1010	FIGURE NO. 1	



SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Sam's Club Petroleum Station
7370 W. 52nd Avenue
Arvada, Colorado

SCALE: NOT TO SCALE	DATE: 11/29/2019
DRAWN BY: VT	APPROVED BY: JRM
PROJECT NO. 7-219-1010	FIGURE NO. 2

LEGEND:
B-1 Soil Boring Location

All Locations Approximate

SALEM engineering group, inc.

AASHTO FLEXIBLE PAVEMENT DESIGN

Layer Thickness Determination Using Layered Analysis Approach

	Layer No.	Description	Layer Coefficient, a_i	Drainage Coefficient, m_i	Elastic Modulus, psi	SN Using E of next lower layer in inputs box below	Min. Layer Thickness, D, inches	Practical Layer Thickness, D, inches	Associated SN	
(topmost)	Layer 1	AC Layer	0.44	1.00	400,000	1.60	3.64	4.00	1.76	
	Layer 2	Gran Base A	0.14	0.90	27,000	0.00	-13.97	4.00	0.50	
	Layer 3						0.00	0.00	0.00	
	Layer 4						0.00	0.00	0.00	
(bottommost)	Layer 5						0.00	0.00	0.00	
	Subgrade	Subgrade	N/A	N/A	3,450	N/A	N/A	N/A	N/A	
						Total Pavement Thickness, inches,	-10.33	8.00	2.26	Calculated SN
									1.60	SN to Match

Design is sufficient

Standard Duty AC Pavement

Inputs Box

W18 =	2,200	*ESALs Applications Over Design Period*
R =	85 %	*Reliability*
So =	0.45	*Standard Deviation*
MR =	12,000 psi	*Subgrade Resilient Modulus*
Pi =	4.2	*Initial Serviceability*
Pt =	2	*Terminal Serviceability*

SN on top of layer =	1.60

AASHTO RIGID PAVEMENT DESIGN

Standard Duty PCC Pavements

Design Inputs

W18 =	2,200		*ESALs Applications Over Design Period*	Typ. Range 0.5 to 100 million
PCC MR =	550	psi	*Concrete Modulus of Rupture*	Typ. Range 550 to 750 psi
E =	4,000,000	psi	*Concrete Elastic Modulus*	Typ. Range 3 to 6 million psi
k-value =	200	psi/in	*Modulus of Subgrade Reaction*	Typ. Range 100 to 500 psi/in
R =	85	%	*Reliability*	Typ. Range 80 to 95%
So =	0.35		*Standard Deviation*	Typ. Range 0.3 to 0.5
J =	3.2		*Load Transfer Coefficient*	Typ. Range 2.2 to 4.4
Cd =	1		*Drainage Coefficient*	Typ. Range 0.9 to 1.1
Pi =	4.2		*Initial Serviceability*	Typ. Range 4.5 to 4.8
Pt =	2		*Terminal Serviceability*	Typ. Range 2.0 to 3.0

DESIGN D, inches, = 5.00

AASHTO FLEXIBLE PAVEMENT DESIGN

Layer Thickness Determination Using Layered Analysis Approach

	Layer No.	Description	Layer Coefficient, a_i	Drainage Coefficient, m_i	Elastic Modulus, psi	SN Using E of next lower layer in inputs box below	Min. Layer Thickness, D, inches	Practical Layer Thickness, D, inches	Associated SN	
(topmost)	Layer 1	AC Layer	0.44	1.00	400,000	1.76	4.00	4.00	1.76	
	Layer 2	Gran Base A	0.14	0.90	27,000	0.00	-13.97	4.00	0.50	
	Layer 3						0.00	0.00	0.00	
	Layer 4						0.00	0.00	0.00	
(bottommost)	Layer 5						0.00	0.00	0.00	
	Subgrade	Subgrade	N/A	N/A	8,500	N/A	N/A	N/A	N/A	
						Total Pavement Thickness, inches,	-9.97	8.00	2.26	Calculated SN
									1.60	SN to Match

Design is sufficient

Heavy Duty AC Pavement

Inputs Box

W18 =	18,000		*ESALs Applications Over Design Period*
R =	85	%	*Reliability*
So =	0.45		*Standard Deviation*
MR =	12,000	psi	*Subgrade Resilient Modulus*
Pi =	4.2		*Initial Serviceability*
Pt =	2		*Terminal Serviceability*

SN on top of layer =	1.60

AASHTO RIGID PAVEMENT DESIGN

Heavy Duty PCC Pavements

Design Inputs

W18 =	18,000		*ESALs Applications Over Design Period*	Typ. Range 0.5 to 100 million
PCC MR =	550	psi	*Concrete Modulus of Rupture*	Typ. Range 550 to 750 psi
E =	4,000,000	psi	*Concrete Elastic Modulus*	Typ. Range 3 to 6 million psi
k-value =	200	psi/in	*Modulus of Subgrade Reaction*	Typ. Range 100 to 500 psi/in
R =	85	%	*Reliability*	Typ. Range 80 to 95%
So =	0.35		*Standard Deviation*	Typ. Range 0.3 to 0.5
J =	3.2		*Load Transfer Coefficient*	Typ. Range 2.2 to 4.4
Cd =	1		*Drainage Coefficient*	Typ. Range 0.9 to 1.1
Pi =	4.2		*Initial Serviceability*	Typ. Range 4.5 to 4.8
Pt =	2		*Terminal Serviceability*	Typ. Range 2.0 to 3.0

DESIGN D, inches, =	**6.00**

GEOTECHNICAL INVESTIGATION FACT SHEET

Include this form in the Geotechnical Report as an Appendix.

PROJECT LOCATION: 7370 West 52nd Avenue, Arvada, Colorado

Engineer: R. Sammy Salem, PE No. 45178

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

Geotechnical Engineering Co.: Salem Engineering Group, Inc. Report 7-219-1010, Dated: December 4, 2019

Ground Water Elevation: 13 feet (historic 6.6 feet) Fill Soils Characteristics: NE

Date Groundwater Measured: 11/15/19 Maximum Liquid Limit: N/A

Topsoil/Stripping Depth: 12” Maximum Plasticity Index: N/A

Undercut (If Required): __See Report_________Specified Compaction: 95 percent ASTM D698

Standard Proctor Results: Not Tested

Moisture Content Range: 1 to 3% above optimum

pH: 7.8

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

Chloride 44 mg/kg Minimum Resistivity 3,117 ohm-cm

Corrective actions required for construction based on resistivity level noted:
Soils are Corrosive to Buried Metal Objects

Cement Type: No Restrictions

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

Colorado DOT Class 6 Aggregate Base Coarse_____________________________

Recommended Compaction Control Tests:

1 Test for Each 2,500 Sq. Ft. each Lift (bldg. area)
1 Test for Each 10,000 Sq. Ft. each Lift (parking area)

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

Subgrade Design R-value = 30.

COMPONENT	ASPHALT		CONCRETE	
	Standard	heavy	standard	heavy
Stabilized Subgrade (If Applicable)	_12”___	12”____	__12”__	_12”___
Base Material (Class 2 AB.)	_4.0	4.0”____	__4”___	__4”___
Asphalt Base Course	__1.5”__	2” __		
Surface Course	___1.5”_	__2”_	__5”__	__6”__

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

FOUNDATION DESIGN CRITERIA

Include this form in the Geotechnical Report as an Appendix.

PROJECT LOCATION: 7370 West 52nd Avenue, Arvada, Colorado

Engineer: R. Sammy Salem, PE No. 45178

Geotechnical Engineering Co.: Salem Engineering Group, Inc. Report 7-219-1010, Dated: December 4, 2019

Foundation type: Conventional Shallow Foundation

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

Factor of Safety: N/A

Minimum footing dimensions: Individual: 18” Wide Continuous: 12” Wide

Minimum footing embedment: Exterior: 36” Interior: 36”

Frost depth: 36 inches

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

Slab: Potential vertical rise: N/A

Capillary Break (not a vapor barrier) describe: 6” Colorado DOT Class 6 Aggregate Base

Subgrade reaction modulus: 150 psi/in Method obtained: ACPA Calculator

Active Equivalent Fluid Pressures: 50 pcf

Passive Equivalent Fluid Pressures: 250 psf

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

Retaining Wall: At rest pressure: 73 psf
Coefficient of friction: 0.30

COMMENTS: ____

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation was conducted on November 15, 2019 and included a site visit, subsurface exploration, and soil sampling. The locations of the exploratory borings are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

Our borings were drilled using a truck-mounted CME-55 drilling rig. Sampling was accomplished by driving a 2-inch Standard Penetration Test (SPT) sampler into the soil. Penetration resistance blow counts were obtained 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 logs of borings. The MCS samples were recovered and capped at both ends to preserve the samples at their natural moisture content; SPT samples were recovered and placed in a sealed bag to preserve their natural moisture content. At the completion of drilling and sampling, the test borings were backfilled with drill cuttings and capped with asphaltic concrete cold patch.

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

Project Number: 7-219-1010

Date: 11/15/2019

Client: CEI Engineering Associates, Inc.

SALEM engineering group, inc.

Project: Proposed Sam's Club Petroleum Station - #6630

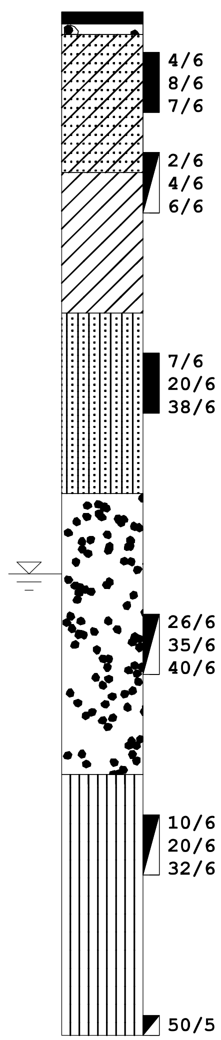
Location: 7370 W. 52nd Avenue, Arvada, Colorado

Drilled By: Site Services Drilling **Logged By:** JRM

Drill Type: CME 55 **Elevation:** 5,312 feet AMSL

Auger Type: 6in. Hollow Stem Auger **Initial Depth to Groundwater:** 14 feet

Hammer Type: Automatic Trip - 140lbs/30in. **Final Depth to Groundwater:** 14 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 with Geotextile embedded.				
		AB	Aggregate Base = 3 inches				
5310	4/6 8/6 7/6	SC	Clayey sand; loose, brown to red, moist, coarse to fine grained.	15	8.3	114.7	
	2/6 4/6 6/6		Grades as above.	10	22.3	--	
5 / 5305		CL	Sandy lean clay; stiff, brown, moist, high plasticity.				
10	7/6 20/6 38/6	SM	Silty sand with gravel; dense, dark brown, moist, coarse to fine grained.	58	5.1	--	
5300 / 15 / 5295	26/6 35/6 40/6	GP	Poorly-graded gravel with sand; very dense, olive to brown, wet, coarse to fine grained.	75	7.9	--	
20 / 5290	10/6 20/6 32/6	ML	Sandy silt; hard, grey, moist, non-plastic.	52	17.7	--	
25	50/5		Grades as above; [weathered bedrock].	>50	20.2	--	
5285			End of boring at 25.5ft. due to bedrock.				

Notes:

Figure Number A-1

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

Project Number: 7-219-1010

Date: 11/15/2019

Client: CEI Engineering Associates, Inc.

SALEM engineering group, inc.

Project: Proposed Sam's Club Petroleum Station - #6630

Location: 7370 W. 52nd Avenue, Arvada, Colorado

Drilled By: Site Services Drilling **Logged By:** JRM

Drill Type: CME 55 **Elevation:** 5,311 feet AMSL

Auger Type: 6in. Hollow Stem Auger **Initial Depth to Groundwater:** 13 feet

Hammer Type: Automatic Trip - 140lbs/30in. **Final Depth to Groundwater:** 13 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 = 4 inches with Geotextile embedded.				
		AB	Aggregate Base = 3 inches				
5308	6/6 8/6 8/6	SC	Clayey sand; medium dense, red to brown, moist, coarse to fine grained.	16	9.2	122.4	
4	5/6 9/6 11/6	CL	Sandy lean clay; stiff, brown, moist, high plasticity.	20	20.7	102.9	
5304 / 8	4/6 20/6 23/6	GC	Clayey gravel; dense, dark brown, moist, coarse to fine grained.	43	14.7	--	
5300 / 12	19/6 36/6 39/6	SP	Poorly-graded sand with gravel; very dense, light brown, moist to wet, coarse to fine grained.	75	8.5	--	
5296 / 16			End of boring at 16.5ft. BSG				
5292 / 20							

Notes:

Figure Number A-2

SALEM engineering group, inc.

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

Project Number: 7-219-1010

Date: 11/15/2019

Client: CEI Engineering Associates, Inc.

Project: Proposed Sam's Club Petroleum Station - #6630

Location: 7370 W. 52nd Avenue, Arvada, Colorado

Drilled By: Site Services Drilling **Logged By:** JRM

Drill Type: CME 55 **Elevation:** 5,310 feet AMSL

Auger Type: 6in. Hollow Stem Auger **Initial Depth to Groundwater:** 13 feet

Hammer Type: Automatic Trip - 140lbs/30in. **Final Depth to Groundwater:** 13 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 = 4 inches with Geotextile embedded.				
		AB	Aggregate Base = 3 inches				
5308	3/6 8/6 7/6	SC	Clayey sand; loose, red, moist, coarse to fine grained.	15	9.3	125.0	
4	3/6 3/6 6/6	CL	Sandy lean clay; firm, light brown, moist, high plasticity.	9	25.4	98.5	
5304							
8	20/6 34/6 40/6	GP	Poorly-graded gravel with sand; very dense, light brown, damp, coarse to fine grained.	74	1.2	--	
5300							
12							
5296	23/6 30/6 40/6		Grades as above; [weathered rock].	70	8.4	--	
			End of boring at 15ft. BSG				
16							
5292							
20							
5288							

Notes:

Figure Number A-3

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Asphaltic Concrete
	Aggregate Base
	Clayey sand
	Lean Clay
	Silty sand
	Poorly graded gravel
	Silt
	Clayey gravel
	Poorly graded sand
Misc. Symbols	
	Water table during drilling

Symbol	Description
Soil Samplers	
	California sampler
	Standard penetration test

Notes:

Consistency Classification

Blows Per Foot (Uncorrected)

Granular Soils

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

Cohesive Soils

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

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

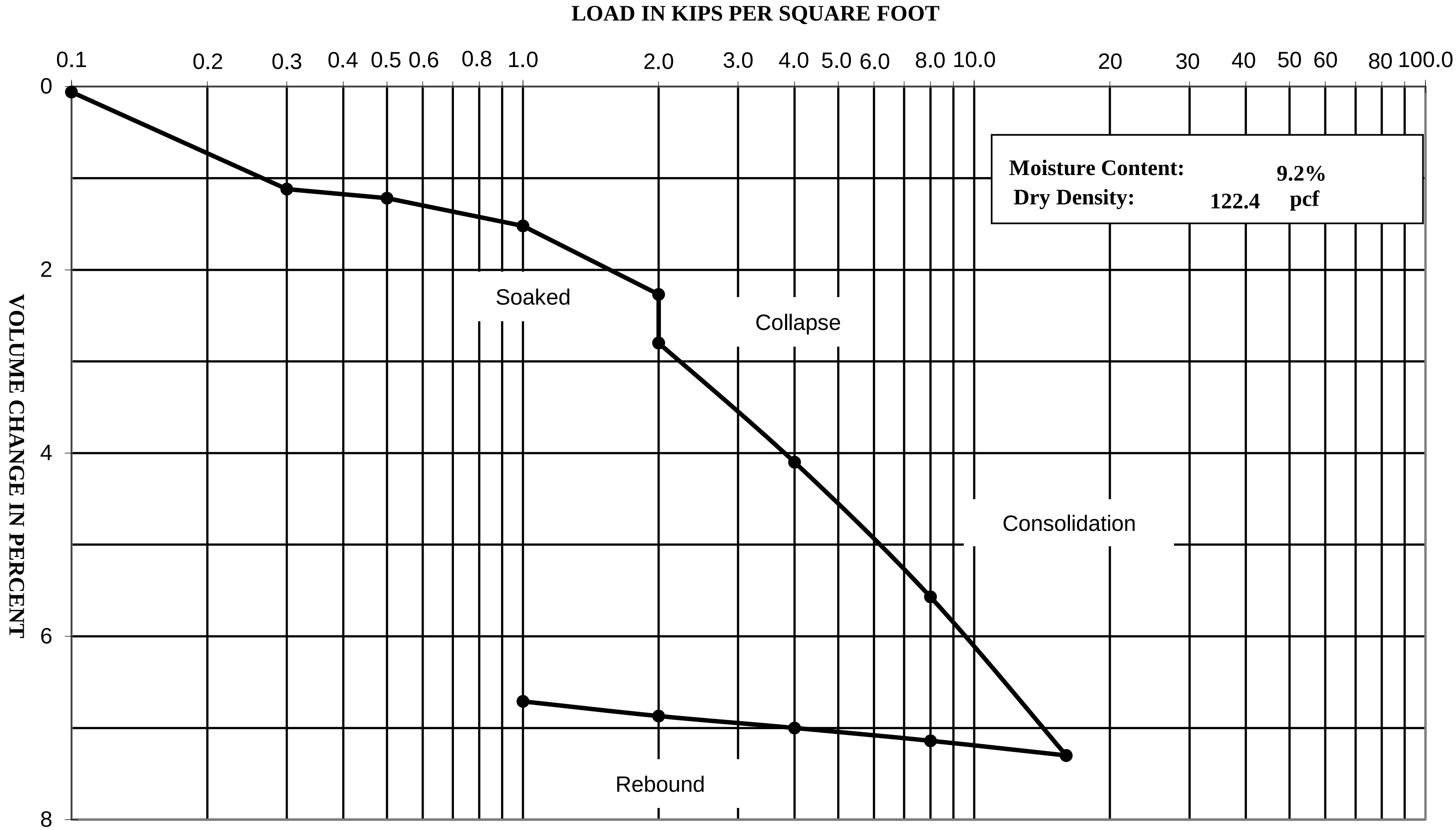
APPENDIX

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), or other suggested procedures. Selected samples were tested for in-situ moisture content, density, consolidation, corrosivity, shear strengths, plasticity index, expansion index, resistance-value, and grain size distribution. The results of the laboratory tests are summarized in the following figures.

CONSOLIDATION - PRESSURE TEST DATA
ASTM D2435



Project Name: Sams Club Petroleum Station - Arvada, CO

Project Number: 7-219-1010

Boring: B-2 @ 2'

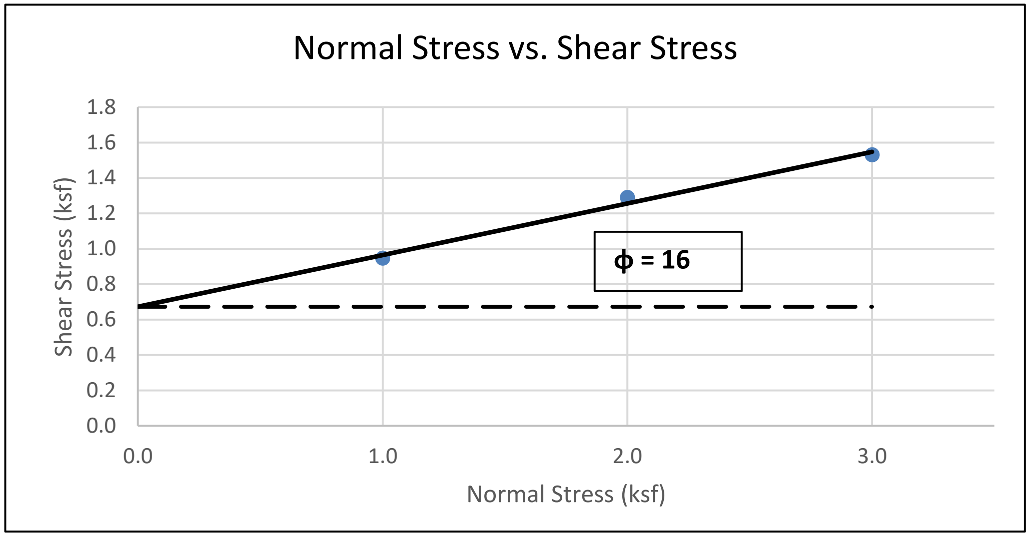
SALEM engineering group, inc.

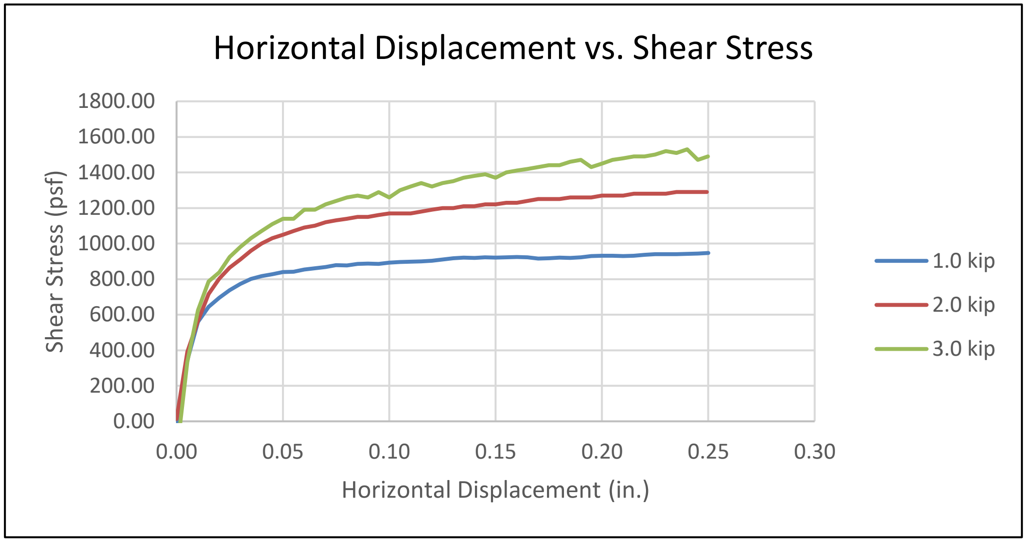
Direct Shear Test (ASTM D3080)

Project Name: Sams Club Petroleum Station - Arvada, CO
Project Number: 7-219-1010
Client: CEI Engineering Associates, Inc.
Boring: B-3 @ 3.5'
Soil Type: Sandy lean clay (CL)
Sample Type: Undisturbed Ring
Tested By: NL / JH
Reviewed By: JRM
Date of Test: 11/21/19
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	**2.0 kip**	**3.0 kip**
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0025	0.0025	0.0025
Peak Shear Stress (ksf)	0.95	1.29	1.53
Residual Shear Stress (ksf)	0.00	0.00	0.00

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.979	0.966	0.909
Post-Shear Sample Height (in.)	0.974	0.958	0.890
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	25.4		
Dry Density (pcf)	99.3	98.8	98.3
Saturation %	97.1	96.0	94.9
Void Ratio	0.71	0.72	0.73
Consolidated Void Ratio	0.68	0.66	0.57
Final (post-shear) Values			
Final Moisture Content (%)	28.2	29.2	27.3
Dry Density (pcf)	98.8	99.1	106.8
Saturation %	109.0	114.1	132.3
Void Ratio	0.70	0.70	0.56



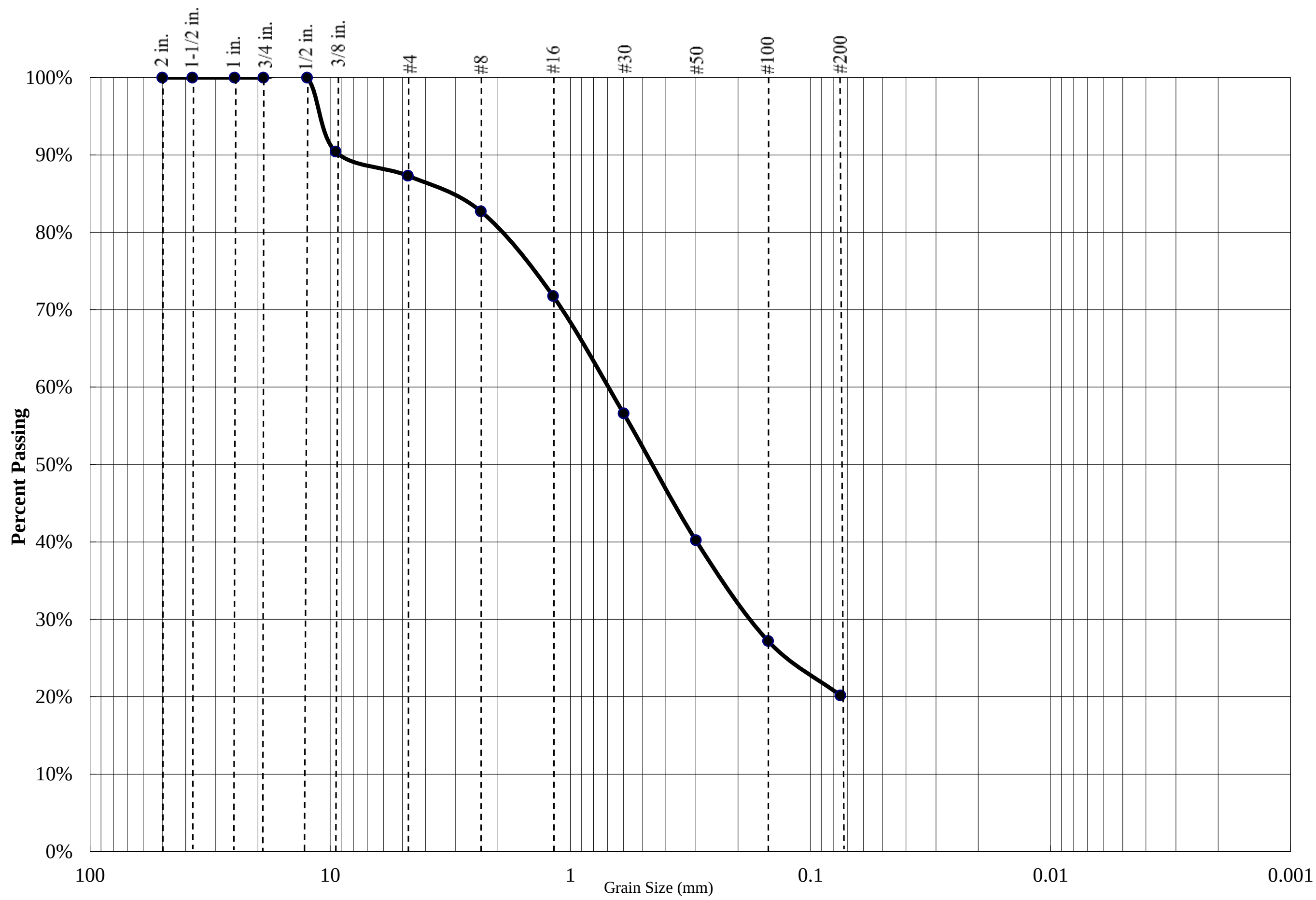


Peak Shear Strength Values		Residual Shear Strength Values	
Slope	0.29	**Slope**	0.00
Friction Angle	16	**Friction Angle**	0
Cohesion (psf)	673	**Cohesion (psf)**	0

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
13%	67%	20%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	90.4%
#4	87.3%
#8	82.7%
#16	71.8%
#30	56.6%
#50	40.2%
#100	27.2%
#200	20.2%

Atterberg Limits					
PL=	17	**LL=**	22	**PI=**	5

Coefficients					
D_{85}=	3	**D_{60}=**	0.7	**D_{50}=**	0.47
D_{30}=	0.19	**D_{15}=**	N/A	**D_{10}=**	N/A
C_u=	N/A	**C_c=**	N/A		

USCS CLASSIFICATION
Clayey sand (SC)

Project Name: Sams Club Petroleum Station - Arvada, CO

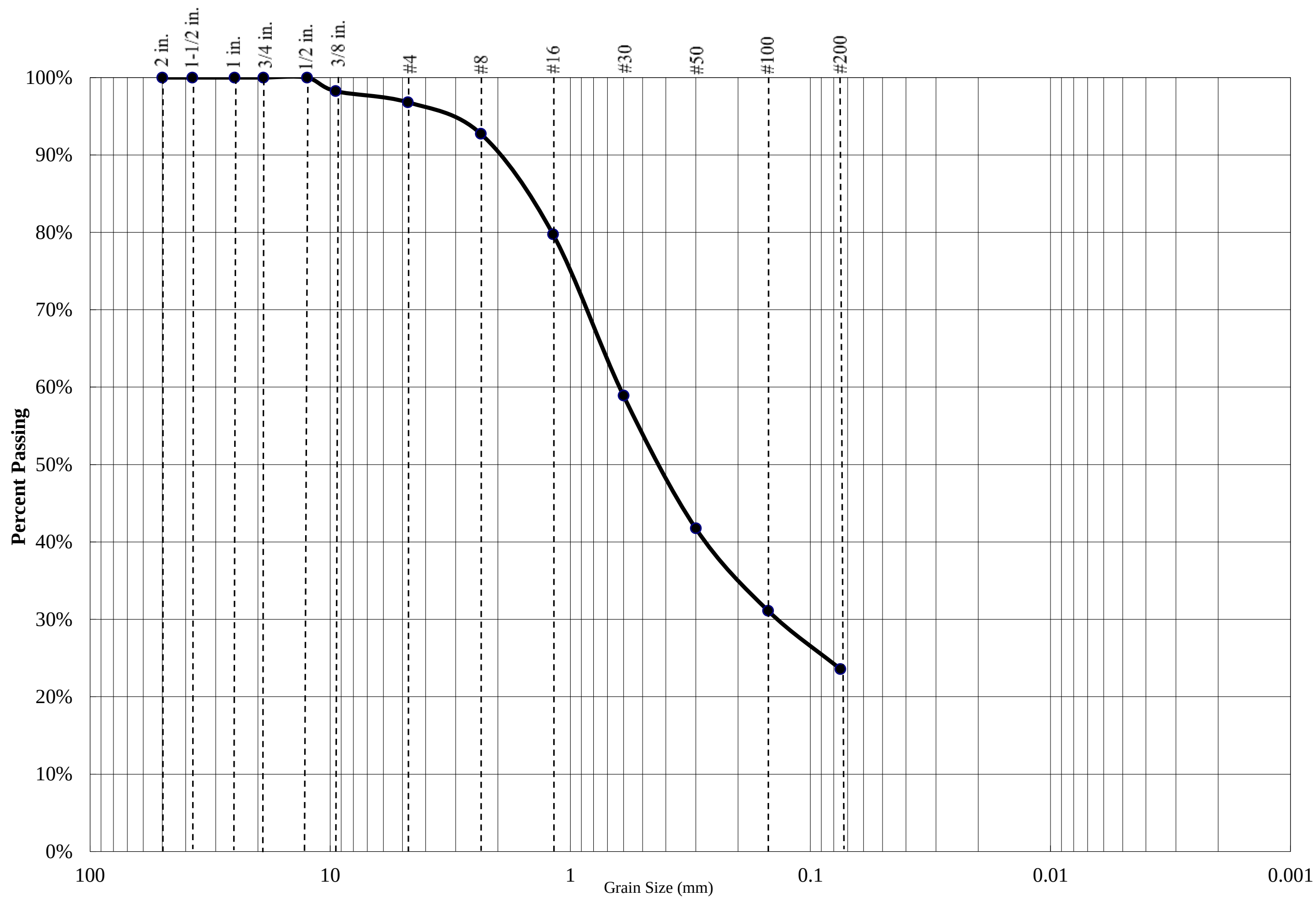
Project Number: 7-219-1010

Boring: B-1 @ 1'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
3%	73%	24%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	98.2%
#4	96.8%
#8	92.7%
#16	79.8%
#30	58.9%
#50	41.7%
#100	31.1%
#200	23.6%

Atterberg Limits					
PL=	N/A	**LL=**	N/A	**PI=**	N/A

Coefficients					
D_{85}=	1.6	**D_{60}=**	0.64	**D_{50}=**	0.43
D_{30}=	0.15	**D_{15}=**	N/A	**D_{10}=**	N/A
C_u=	N/A	**C_c=**	N/A		

USCS CLASSIFICATION
Clayey sand (SC)

Project Name: Sams Club Petroleum Station - Arvada, CO

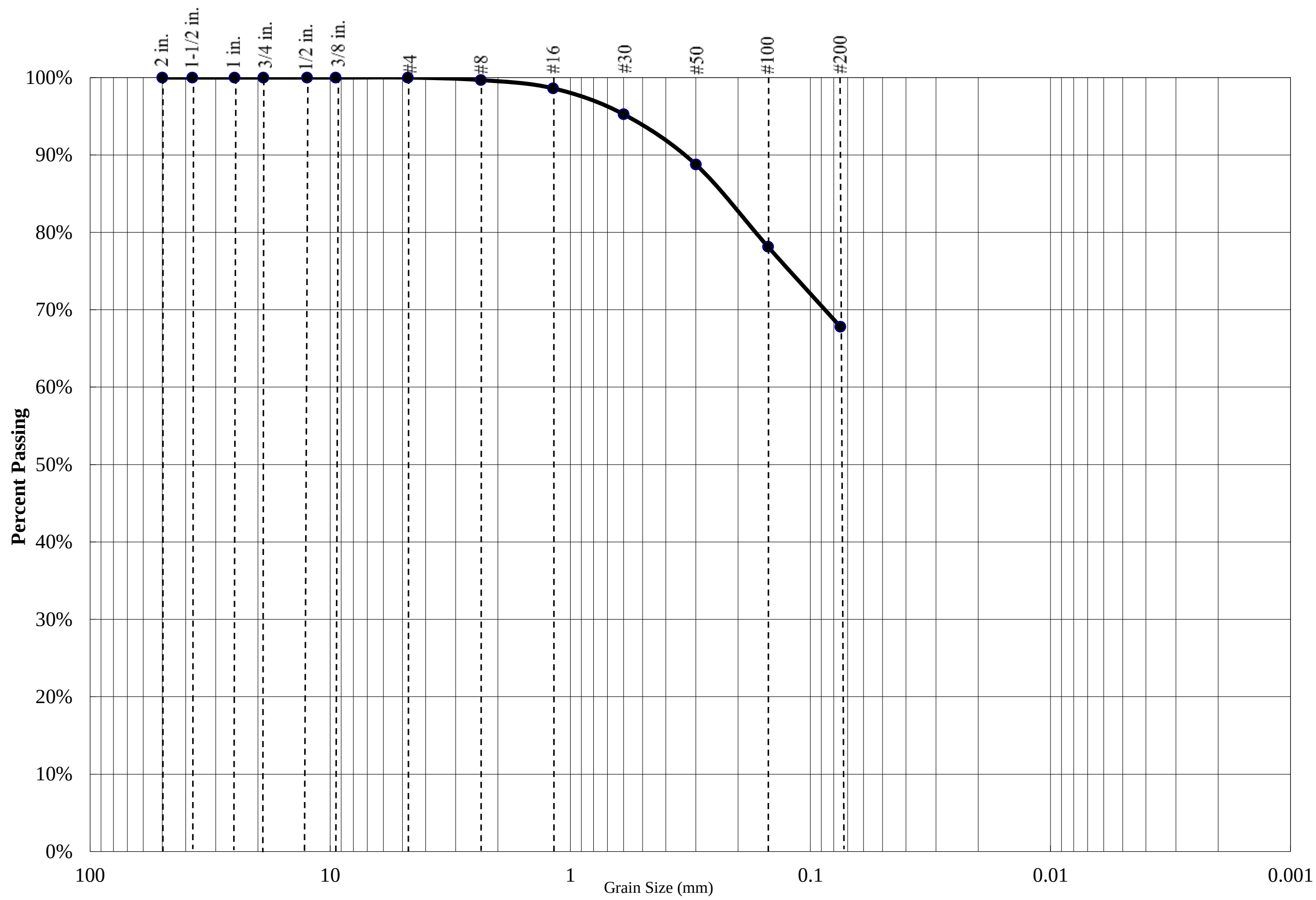
Project Number: 7-219-1010

Boring: B-2 @ 2'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	32%	68%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	99.7%
#16	98.6%
#30	95.3%
#50	88.8%
#100	78.1%
#200	67.8%

Atterberg Limits					
PL=	N/A	**LL=**	N/A	**PI=**	N/A

Coefficients					
D_{85}=	0.23	**D_{60}=**	N/A	**D_{50}=**	N/A
D_{30}=	N/A	**D_{15}=**	N/A	**D_{10}=**	N/A
C_u=	N/A	**C_c=**	N/A		

USCS CLASSIFICATION
Sandy lean clay (CL)

Project Name: Sams Club Petroleum Station - Arvada, CO

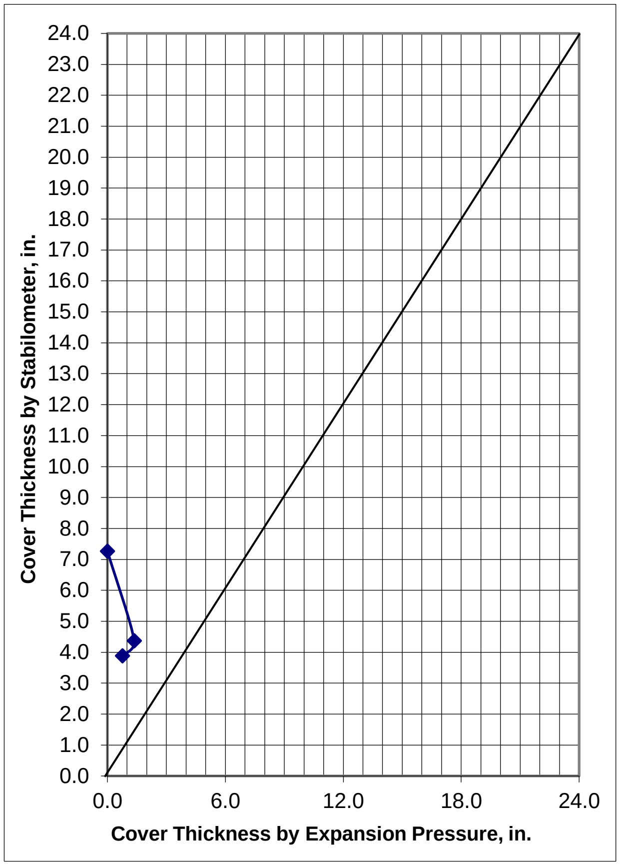
Project Number: 7-219-1010

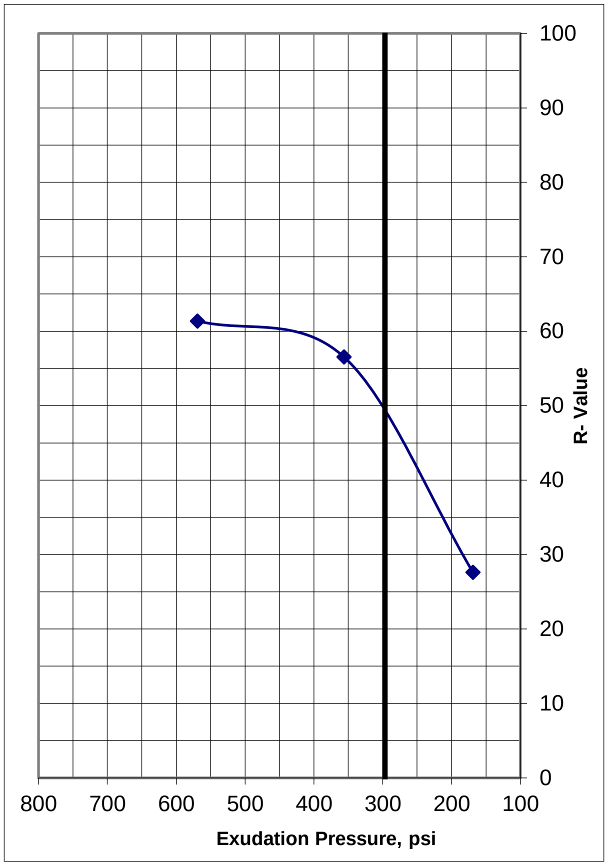
Boring: B-3 @ 3.5'

SALEM engineering group, inc.

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

Project Name: Sams Club Petroleum Station - Arvada, CO
Project Number: 7-219-1010
Date Sampled: 11/15/19 Date Tested:
Sampled By: SEG Tested By: RM
Sample Location: B-1 @ 0 - 3'
Soil Description: Clayey sand (SC)





Specimen	1	2	3
Exudation Pressure, psi	569.3	356.1	168.9
Moisture at Test, %	12.5	13.1	14.4
Dry Density, pcf	127.9	123.8	124.3
Expansion Pressure, psf	82	147	0
Thickness by Stabilometer, in.	3.9	4.4	7.3
Thickness by Expansion Pressure, in.	0.8	1.4	0.0
R-Value by Stabilometer	61	57	28
R-Value by Expansion Pressure	N/A		
R-Value at 300 psi Exudation Pressure	50		

Controlling R-Value	**50**

SALEM
engineering group, inc.

CHEMICAL ANALYSIS

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

Project Name: Sams Club Petroleum Station - Arvada, CO
Project Number: 7-219-1010
Date Sampled: 11/15/19 Date Tested: 11/22/19
Sampled By: SEG Tested By: JH
Soil Description: Clayey sand (SC)

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 0 - 3'	< 50 mg/kg	44 mg/kg	7.7
1b.	B-1 @ 0 - 3'	< 50 mg/kg	45 mg/kg	7.7
1c.	B-1 @ 0 - 3'	< 50 mg/kg	44 mg/kg	7.7
Average:		**< 50 mg/kg**	**44 mg/kg**	**7.7**

SALEM engineering group, inc.

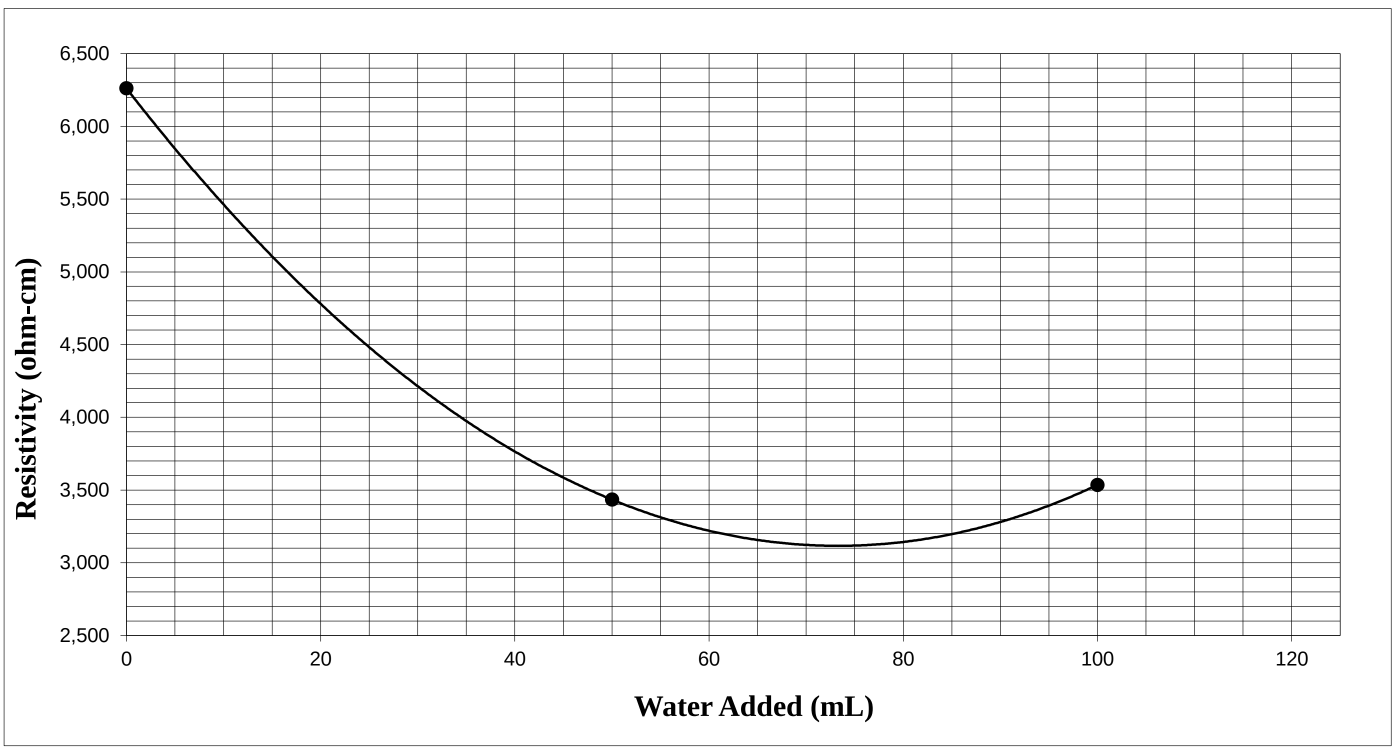
SOIL RESISTIVITY
CTM 643

Project Name: Sams Club Petroleum Station - Arvad Date Sampled: 11/15/19
Project Number: 7-219-1010 Sampled By: SEG
Sample Location: B-1 @ 0 - 3' Date Tested: 11/21/19
Soil Description: Clayey sand (SC) Tested By: JH

Chloride Content: 44 mg/Kg Initial Sample Weight: 700 gms
Sulfate Content: < 50 mg/Kg Test Box Constant: 1.010 cm
Soil pH: 7.7

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm-cm)
1	0	6.2	1,000	6,200	6,262
2	50	3.4	1,000	3,400	3,434
3	100	3.5	1,000	3,500	3,535



Minimum Resistivity:	**3,117** ohm-cm

SALEM engineering group, inc.

EXPANSION INDEX TEST
ASTM D4829

Project Name: Sams Club Petroleum Station - Arvada, CO
Project Number: 7-219-1010
Date Sampled: 11/15/19 Date Tested: 11/21/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 0 - 3'
Soil Description: Clayey sand (SC)

Trial #	1	2	3
Weight of Soil & Mold, g.	617.2		
Weight of Mold, g.	188.2		
Weight of Soil, g.	429.0		
Wet Density, pcf	129.4		
Weight of Moisture Sample (Wet), g.	831.0		
Weight of Moisture Sample (Dry), g.	772.9		
Moisture Content, %	7.5		
Dry Density, pcf	120.3		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	50.7		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.006	0.0085	--	--	0.0095

Expansion Index $_{measured}$ = 9.5
Expansion Index $_{50}$ = 9.8

Expansion Index = $\boxed{10}$

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.

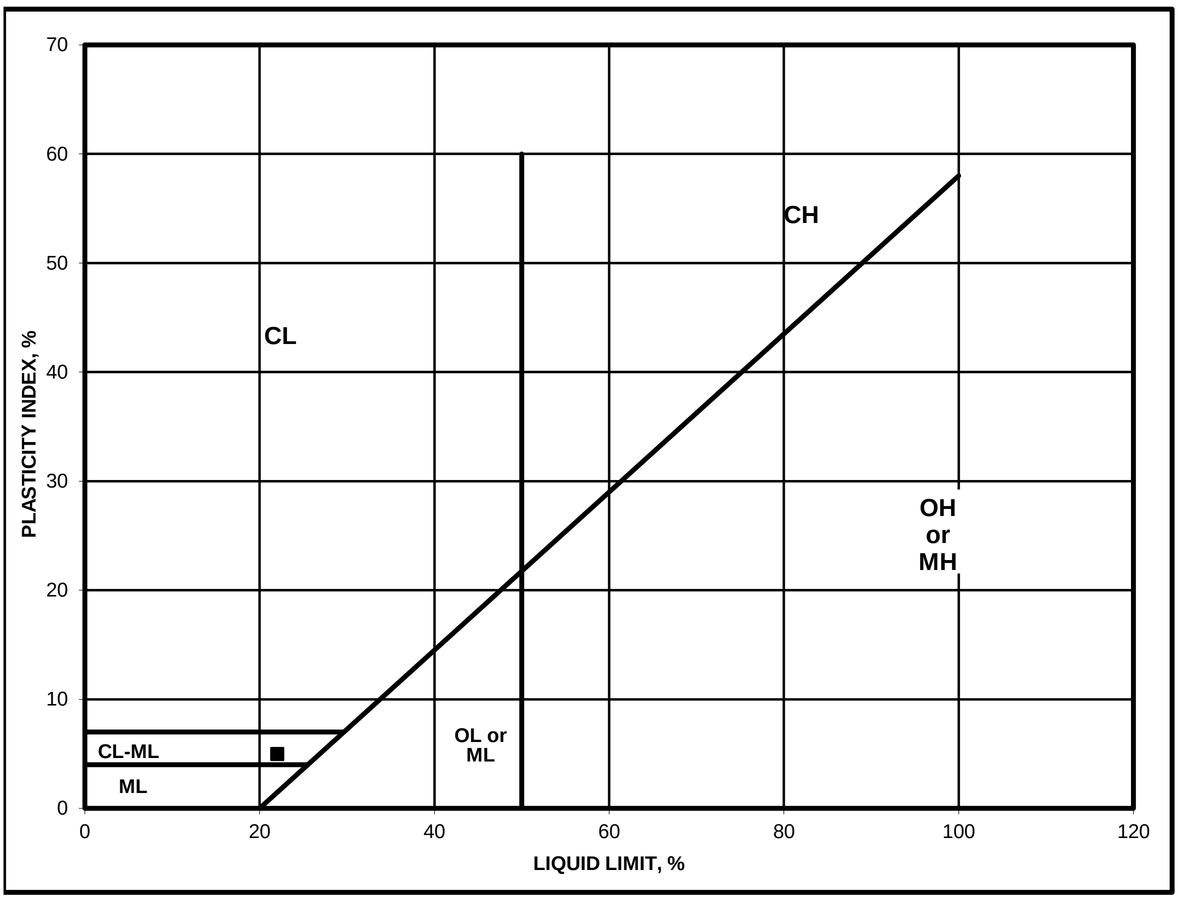
Atterberg Limits Determination
ASTM D4318

Project Name: Sams Club Petroleum Station - Arvada, CO
Project Number: 7-219-1010
Date Sampled: 11/15/19 Date Tested: 11/21/19
Sampled By: SEG Tested By: JH
Sample Location: B-1 @ 1'

	Plastic Limit			Liquid Limit		
Run Number	1	2	3	1	2	3
Weight of Wet Soil & Tare	27.11	28.52	28.42	30.67	33.23	30.40
Weight of Dry Soil & Tare	26.26	27.42	27.24	29.11	31.09	28.63
Weight of Water	0.85	1.10	1.18	1.56	2.14	1.77
Weight of Tare	20.96	20.73	20.50	20.91	20.65	20.71
Weight of Dry Soil	5.30	6.69	6.74	8.20	10.44	7.92
Water Content	16.0	16.4	17.5	19.0	20.5	22.3
Number of Blows				35	30	25

Plastic Limit : 17 **Liquid Limit : 22**

Plasticity Index : 5
Unified Soil Classification : CL/ML



SALEM engineering group, inc.

ORIGINS LABORATORY, INC

November 19, 2019

Salem Engineering Group, Inc.

Joshua Marroquin

7887 East Belleview Suite 1100

Denver CO 80111

Project Name - Sam's Club Petroleum Station Project Number - 7-219-1010

Attached are your analytical results for Sam's Club Petroleum Station received by Origins Laboratory, Inc. November 15, 2019. This project is associated with Origins project number Y911266-01.

The analytical results in the following report were analyzed under the guidelines of EPA Methods. These methods are identified as follows; "SW" are defined in SW-846, "EPA" are defined in 40CFR part 136 and "SM" are defined in the most current revision of Standard Methods For the Examination of Water and Wastewater.

The analytical results apply specifically to the samples and analyses specified per the attached Chain of Custody. As such, this report shall not be reproduced except in full, without the written approval of Origin's laboratory.

Unless otherwise noted, the analytical results for all soil samples are reported on a wet weight basis. All analytical analyses were performed under NELAP guidelines unless noted by a data qualifier.

Any holding time exceedances, deviations from the method specifications or deviations from Origins Laboratory's Standard Operating Procedures are outlined in the case narrative.

Thank you for selecting Origins for your analytical needs. Please contact us with any questions concerning this report, or if we can help with anything at all.

Origins Laboratory, Inc.
303.433.1322
o-squad@oelabinc.com

THE NELAC INSTITUTE

Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

CROSS REFERENCE REPORT

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
B-1@20'	Y911266-01	Soil	November 15, 2019 10:01	11/15/2019 13:18

Origins Laboratory, Inc.

Jen Pellegrini For Noelle Doyle Mathis, President

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

ORIGINS LABORATORY, INC

Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

ORIGINS LABORATORY, INC

Y911266

www.originslaboratory.com

page of

Client: Salem Engineering Group, Inc
Address: 4729 W. Jacquelyn Ave
Fresno, CA, 93722
Telephone Number: (559) 271-9700
Email Address: joshua.mesalem.net

Project Manager: Joshua Marroquin
Project Name: Sam's club Petroleum Station
Project Number: 7-219-1010
Samples Collected By: J. Marroquin

Sample ID Description	Date Sampled	Time Sampled	# of Containers	Preservative: Unpreserved	Preservative: HCl	Preservative: HNO_3	Preservative: Other	Matrix: Groundwater	Matrix: Soil	Matrix: Air Summa Canister #	Matrix: Other	Analysis: BTEX	Analysis	Analysis	Analysis	Analysis	Sample Instructions	
B-1 @ 20'	11/15/19	10:01 AM	1						X									1
																		2
																		3
																		4
																		5
																		6
																		7
																		8
																		9
																		10

Relinquished By:	Date:	Time:	Received By:	Date:	Time:
[signature]	11/15/19	1318	[signature]	11/15/19	1318
Relinquished By:	Date:	Time:	Received By:	Date:	Time:

Turnaround Time: Same Day ☐ 24 Hr ☐ 48 Hr ☐ 72 Hr ☐ Standard ☒

Temp Received- 2.8

Date Results Needed

1725 Elk Place | Denver, CO 80211 | Phone: 303.433.1322 | Fax: 303.265.9645

Origins Laboratory, Inc.

Jen Pellegrini For Noelle Doyle Mathis, President

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Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

Origins Laboratory

F-012207-01-R1
Effective Date: 01/09/12

Sample Receipt Checklist

Origins Work Order: Y911266

Client: Salem

Client Project ID: Sam's Club

Checklist Completed by: JG

Shipped Via: HD (UPS, FedEx, Hand Delivered, Pick-up, etc.)

Date/time completed: 11/15/19

Airbill #: N/A

Matrix(s) Received: (Check all that apply): X Soil/Solid ____ Water ____ Other: ________ (Describe)

Cooler Number/Temperature: 1 / 2.8 °C ____/____ °C ____/____ °C ____/____ °C

Thermometer ID: T003

Requirement Description	Yes	No	N/A	Comments (if any)
If samples require cooling, was the temperature between 0°C to ≤ 6°C[(1)]?	✓			
Is there ice present (document if blue ice is used)	✓			
Are custody seals present on cooler? (if so, document in comments if they are signed and dated, broken or intact)		✓		
Are custody seals present on each sample container? (if so, document in comments if they are signed and dated, broken or intact)		✓		
Were all samples received intact[(1)]?	✓			
Was adequate sample volume provided[(1)]?	✓			
Are short holding time analytes or samples with HTs due within 48 hours present[(1)]?		✓		
Is a chain-of-custody (COC) present and filled out completely[(1)]?	✓			
Does the COC agree with the number and type of sample bottles received[(1)]?	✓			
Do the sample IDs on the bottle labels match the COC[(1)]?	✓			
Is the COC properly relinquished by the client with date and time recorded[(1)]?	✓			
For volatiles in water – is there headspace (> ¼ inch bubble) present? **If yes, contact client and note in narrative.**			✓	
Are samples preserved that require preservation and was it checked [(1)]? *(note ID of confirmation instrument used in comments) / (preservation is not confirmed for subcontracted analyses in order to insure sample integrity)/(pH <2 for samples preserved with HNO3, HCL, H2SO4) / (pH >10 for samples preserved with NaAsO2+NaOH, ZnAc+NaOH)*			✓	

Additional Comments (if any):

[(1)] If NO, then contact the client before proceeding with analysis and note date/time and person contacted as well as the corrective action to in the additional comments (above) and the case narrative.

Reviewed by (Project Manager): CW

Date/Time Reviewed: 11/18/19

Origins Laboratory, Inc.

Jen Pellegrini

Jen Pellegrini For Noelle Doyle Mathis, President

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Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

B-1@20'
11/15/2019 10:01:00AM

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Analyst	Prepared	Analyzed	Notes

Origins Laboratory, Inc.

Y911266-01 (Soil)

BTEX by EPA 8260D

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Analyst	Prepared	Analyzed	Notes
Benzene	ND	0.050	mg/kg	25	B9K1510	KDK	11/15/2019	11/15/2019	U
Toluene	**0.108**	0.050	"	"	"	KDK	"	"	
Ethylbenzene	ND	0.050	"	"	"	KDK	"	"	U
Xylenes, total	ND	0.050	"	"	"	KDK	"	"	U
Surrogate: 1,2-Dichloroethane-d4	*97.4 %*	*70-130*			*"*	*"*		*"*	
Surrogate: Toluene-d8	*98.0 %*	*70-130*			*"*	*"*		*"*	
Surrogate: 4-Bromofluorobenzene	*101 %*	*70-130*			*"*	*"*		*"*	

Origins Laboratory, Inc.

Jen Pellegrini For Noelle Doyle Mathis, President

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Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

Volatile Organic Compounds by GC/MS SW846 8260D - Quality Control
Origins Laboratory, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B9K1510 - EPA 5030 (soil)										
Blank (B9K1510-BLK1)					Prepared: 11/15/2019 Analyzed: 11/15/2019					
Benzene	ND	0.002	mg/kg							U
Toluene	ND	0.002	"							U
Ethylbenzene	ND	0.002	"							U
Xylenes, total	ND	0.002	"							U
Surrogate: 1,2-Dichloroethane-d4	*0.12*		*"*	*0.125*		*96.7*	*70-130*			
Surrogate: Toluene-d8	*0.12*		*"*	*0.125*		*96.9*	*70-130*			
Surrogate: 4-Bromofluorobenzene	*0.12*		*"*	*0.125*		*98.4*	*70-130*			

Origins Laboratory, Inc.

Jen Pellegrini

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Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

Volatile Organic Compounds by GC/MS SW846 8260D - Quality Control
Origins Laboratory, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B9K1510 - EPA 5030 (soil)										
LCS (B9K1510-BS1)					Prepared: 11/15/2019 Analyzed: 11/15/2019					
Benzene	0.083	0.002	mg/kg	0.100		83.4	70-130			
Toluene	0.083	0.002	"	0.100		83.4	70-130			
Ethylbenzene	0.080	0.002	"	0.100		80.3	70-130			
m,p-Xylene	0.163	0.004	"	0.200		81.4	70-130			
o-Xylene	0.082	0.002	"	0.100		81.8	70-130			
Surrogate: 1,2-Dichloroethane-d4	*0.13*		*"*	*0.125*		*100*	*70-130*			
Surrogate: Toluene-d8	*0.12*		*"*	*0.125*		*99.1*	*70-130*			
Surrogate: 4-Bromofluorobenzene	*0.13*		*"*	*0.125*		*100*	*70-130*			

Origins Laboratory, Inc.

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Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

Volatile Organic Compounds by GC/MS SW846 8260D - Quality Control
Origins Laboratory, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B9K1510 - EPA 5030 (soil)										
Matrix Spike (B9K1510-MS1)		**Source: Y911266-01**			Prepared: 11/15/2019 Analyzed: 11/15/2019					
Benzene	0.094	0.002	mg/kg	0.100	0.028	66.0	70-130			QM-07
Toluene	0.094	0.002	"	0.100	0.108	NR	70-130			QM-07
Ethylbenzene	0.092	0.002	"	0.100	0.018	74.1	70-130			
m,p-Xylene	0.185	0.004	"	0.200	0.096	44.2	70-130			QM-07
o-Xylene	0.091	0.002	"	0.100	0.030	60.6	70-130			QM-07
Surrogate: 1,2-Dichloroethane-d4	*0.12*		*"*	*0.125*		*93.0*	*70-130*			
Surrogate: Toluene-d8	*0.12*		*"*	*0.125*		*99.1*	*70-130*			
Surrogate: 4-Bromofluorobenzene	*0.12*		*"*	*0.125*		*100*	*70-130*			

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Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

Volatile Organic Compounds by GC/MS SW846 8260D - Quality Control
Origins Laboratory, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B9K1510 - EPA 5030 (soil)										
Matrix Spike Dup (B9K1510-MSD1)		**Source: Y911266-01**			Prepared: 11/15/2019 Analyzed: 11/15/2019					
Benzene	0.094	0.002	mg/kg	0.100	0.028	66.1	70-130	0.107	20	QM-07
Toluene	0.094	0.002	"	0.100	0.108	NR	70-130	0.213	20	QM-07
Ethylbenzene	0.094	0.002	"	0.100	0.018	76.4	70-130	2.53	20	
m,p-Xylene	0.191	0.004	"	0.200	0.096	47.1	70-130	3.09	20	QM-07
o-Xylene	0.095	0.002	"	0.100	0.030	65.0	70-130	4.67	20	QM-07
Surrogate: 1,2-Dichloroethane-d4	*0.12*		*"*	*0.125*		*93.5*	*70-130*			
Surrogate: Toluene-d8	*0.12*		*"*	*0.125*		*99.3*	*70-130*			
Surrogate: 4-Bromofluorobenzene	*0.13*		*"*	*0.125*		*101*	*70-130*			

Origins Laboratory, Inc.

Jen Pellegrini

Jen Pellegrini For Noelle Doyle Mathis, President

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Salem Engineering Group, Inc.
7887 East Belleview Suite 1100
Denver CO 80111

Joshua Marroquin
Project Number: 7-219-1010
Project: Sam's Club Petroleum Station

Notes and Definitions

U	Sample is Non-Detect.
QM-07	The spike recovery was outside acceptance limits for the MS and/or MSD. The batch was accepted based on acceptable LCS recovery.
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference

All soil results are reported at a wet weight basis.

Origins Laboratory, Inc.

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Jen Pellegrini For Noelle Doyle Mathis, President

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 should be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work should 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, should 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 should 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 should be made except upon written approval of the Soils Engineer, Civil Engineer, or project Architect.

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

The Contractor should 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 should apply continuously and not be limited to normal working hours; and that the Contractor should 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 should be densified to no less that 90 percent of relative compaction based on ASTM D1557 Test Method (latest edition) as specified in the technical portion of the Soil Engineer's report. The location and frequency of field density tests should be determined by the Soils Engineer. The results of these tests and compliance with these specifications should 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 should make his own interpretation of the data contained in the Geotechnical Engineering Report and the Contractor should 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 should assume all liability, including court costs of codefendants, for all claims related to dust or wind-blown materials attributable to his work. Site preparation should consist of site clearing and grubbing and preparation of foundation materials for receiving fill.

6.0 CLEARING AND GRUBBING: The Contractor should accept the site in this present condition and should 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 should become the property of the Contractor and should 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 should not be permitted.

7.0 SUBGRADE PREPARATION: Surfaces to receive Engineered Fill and/or building or slab loads should be prepared as outlined above, scarified to a minimum of 12 inches, moisture-conditioned as necessary, and compacted to 90 percent relative compaction (ASTM D1557).

Loose soil areas and/or areas of disturbed soil should be moisture-conditioned as necessary and compacted to 90 percent relative compaction (ASTM D1557). All ruts, hummocks, or other uneven surface features should be removed by surface grading prior to placement of any fill materials. All areas which are to receive fill materials should be approved by the Soils Engineer prior to the placement of any fill material.

8.0 EXCAVATION: All excavation should 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 should be backfilled at the Contractor's expense and should be compacted in accordance with the applicable technical requirements.

9.0 FILL AND BACKFILL MATERIAL: No material should 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 should 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 should be the responsibility of the Contractor. Compaction of fill materials by flooding, ponding, or jetting should not be permitted unless specifically approved by local code, as well as the Soils Engineer. Both cut and fill should be surface-compacted to the satisfaction of the Soils Engineer prior to final acceptance.

11.0 SEASONAL LIMITS: No fill material should 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 should 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" should 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 2019 Colorado Department of Transportation Standard Specifications for Road and Bridge Construction. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as determined by ASTM D1557 or ASTM D698 Test Methods (latest edition).

13.0 PREPARATION OF THE SUBGRADE - The Contractor should 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 should be compacted to a minimum relative compaction of 95 percent based upon ASTM D1557 or 98 percent ASTM D698. The finished subgrades should be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

14.0 AGGREGATE BASE - The aggregate base material should be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material should conform to the requirements of Division 700 of the Colorado Standard Specifications for Graded Aggregate Base material. The aggregate base material should be compacted to a minimum relative compaction of 95 percent ASTM D1557 or 98 percent ASTM D698. 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.

15.0 AGGREGATE SUBBASE - The aggregate subbase should be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate subbase material should conform to the requirements of CDOT Division 700. Granular Materials for Fill and Subbase, of the Colorado Standard Specifications. The aggregate subbase material should be compacted to a minimum relative compaction of 95 percent based upon ASTM D1557, and it should be spread and compacted in accordance with the Standard Specifications. Each layer of aggregate subbase should be tested and approved by the Soils Engineer prior to the placement of successive layers.

16.0 ASPHALTIC CONCRETE SURFACING - Asphaltic concrete surfacing should 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 should be Superpave 125, unless otherwise stipulated or local conditions warrant more stringent grade. The mineral aggregate should be Superpave 125, ½ inch maximum size, medium grading, and should conform to the requirements set forth in Division 400 of the Standard Specifications. The drying, proportioning, and mixing of the materials should conform to Division 400. The prime coat, spreading and compacting equipment, and spreading and compacting the mixture should conform to the applicable chapters of Division 400, with the exception that no surface course should be placed when the atmospheric temperature is below 50 degrees F. The surfacing should be rolled with a combination steel-wheel and pneumatic rollers, as described in the Standard Specifications. The surface course should be placed with an approved self-propelled mechanical spreading and finishing machine.