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SALEM engineering group, inc.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED VERIZON 105' MONOPOLE TOWER
PROJECT ID: NIGHTMARE ROCK
1203 LUBKEN CANYON ROAD
NEAR COORDINATE: 36.5574°, -118.0978°
LONE PINE, CALIFORNIA

SALEM PROJECT NO. 1-224-0478
JULY 17, 2024

PREPARED FOR:

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July 17, 2024 | Project No. 1-224-0478

Mr. Mike Shubin
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SUBJECT: GEOTECHNICAL ENGINEERING INVESTIGATION
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LONE PINE, CALIFORNIA

Dear Mr. Shubin:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this geotechnical engineering investigation report for the Proposed Verizon 105' Monopole Tower planned at 1203 Lubken Canyon Road, in Lone Pine, California.

The accompanying report presents our findings, conclusions, and recommendations regarding the geotechnical aspects of designing and constructing the project as presently proposed.

In our opinion, the proposed project is feasible from a geotechnical viewpoint, provided our recommendations are incorporated into the design and construction of the project.

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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LONE PINE, CALIFORNIA

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical engineering investigation for the proposed Verizon 105-foot monopole tower planned near area addressed at 1203 Lubken Canyon Road, and within parcel with APN#: 026-430-12-000, near coordinate: 36.5574°,-118.0978°, in Lone Pine, California. (see Figure 1, Vicinity Map).

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

The scope of this investigation included a field exploration, laboratory testing, engineering analysis and the preparation of this report. The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions.

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

2. SITE LOCATION AND DESCRIPTION

The project site is located at near area addressed at 1203 Lubken Canyon Road, and within parcel with (APN#: 026-430-12-000), near coordinate: 36.5574°,-118.0978°, in Lone Pine, California (see Site Plan, Figure 1).

The area of tower is planned in southern portion of parcel (APN#: 026-430-12-000), that is covered with dirt and wild bushes. The entire parcel site is bounded by a chain link fence. The site of tower is bounded to north by undeveloped area and 350 feet a way to the north by a residence structure, and ends with chain link fence. To the east open dirt lot; to the west by open dirt field ending with chain link fence, some trees and Tuttle Creek Road beyond; to the south open dirt lot beyond and ends with chain link fence, and Lubken Canyon road around 3,800 feet beyond.

The immediate project site was observed to be relatively flat. During this investigation several large boulders (3 feet in diameter or larger) were noted directly adjacent to the area of the planned tower. In addition, exposures of granitic rock (likely boulders) were noted within the near surface. Based on review of available aerial imagery, the existing building development located around 345 feet north from project site appears to have been constructed before 1993. The project site elevation, based upon information obtained from google earth, is about 4,617 feet above mean sea level.

3. PROJECT DESCRIPTION

We understand that the project will consist of the construction of Verizon wireless unmanned telecommunications facility. The facility will consist of 105’-0” Tall Monopole Tower. Due to the presence of shallow rock conditions, it is anticipated the Monopine tower will be supported on shallow spread foundations. Use of CIDH piers does not appear feasible due to the hard rock conditions. Further evaluation, including rock coring would be needed to determine the feasibility of CIDH piers.

In addition, the project will be within a 900 square foot rectangular shape new chain link fenced (CLF) enclosure area and will include lightly loaded equipment slabs and infrastructure for supporting of the communication tower facility.

At the time of this investigation, foundation loads for the proposed tower structure had not been provided for review. It is anticipated concentrated loads will be no greater than about 30 kips. Based on our experience lateral loading typically governs design.

A site grading plan was not available at the time of preparation of this report. We anticipate that cuts and fills during earthwork will be minimal and limited to providing 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.

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

4. FIELD EXPLORATION

Our field exploration consisted of a site surface reconnaissance and a subsurface exploration. One exploratory test boring (B-1), and two attempted borings (B-1A and B-1B) were drilled on June 10, 2024, in the area shown on the Site Plan, Figure 2. The test borings were advanced with 6 5/8-inch diameter hollow-stem auger rotated by a truck mounted CME-45C drill rig to the maximum depth explored of 4-feet below site grade, where practical refusal was encountered due to dense soils and possible boulders.

The materials encountered in the test boring was visually classified in the field, and the log was 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 boring was generally made in accordance with the Unified Soil Classification System (ASTM D2487). The boring locations can be found on the Site Plan, attached at the end of this report.

A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix "A." The Test Boring Log is presented in Appendix "A." The Boring Log includes the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The location of the test boring was 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 Log in Appendix "A" should be consulted. Subsurface soil samples were obtained by driving a Modified California sampler (MCS) and a Standard Penetration Test (SPT) sampler. Penetration resistance blow counts were obtained by dropping an automated 140-pound trip hammer through a 32.5-inch free fall to drive the sampler to a maximum depth of 18 inches. The number of blows required to drive the last 12 inches is recorded as Penetration Resistance (blows/foot) on the logs of the boring. In case very high penetration resistance is encountered, the number of blows recorded may be for less than 12 inches.

Soil samples were obtained from the test boring at the depths shown on the log of boring. 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.

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, consolidation potential, shear strength, density, expansion index, Atterberg Limits, and gradation of the materials encountered.

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

6. SOIL AND GROUNDWATER CONDITIONS

6.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. The near surface soils encountered comprised of silty sand with gravel to 4 feet below ground surface, where practical refusal encountered at 4 feet depth BSG. It should be noted that auger refusal due to very dense materials occurred at depths between 1.5 and 4 feet BSG.

One (1) consolidation test performed on a near surface sample, resulted in about 8.8 percent consolidation under a load of 8 kips per square foot. When wetted under a load of 2.0 kips per square foot, the samples exhibited about 5.2 percent collapse. One (1) direct shear tests resulted in internal angles of friction of 46 degrees with cohesion values of 257 pounds per square foot. One (1) Atterberg limits tests performed on a near surface samples resulted in plasticity index of 0 with liquid limits value of 15. An expansion index test performed on a near surface soil sample indicated the sample tested has an expansion potential (EI = 0).

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

6.2 Groundwater

The test boring location was checked for the presence of groundwater during and after the drilling operations. Free groundwater was not encountered at the time of our investigation to a depth of 4 feet after the completion of drilling.

Based on review of available groundwater depth records with the California Department of Water Resources Groundwater (https://wdl.water.ca.gov) State Well Code 365788N1180533W001, located approximately 2.75-mile northeast of the project site, and recorded a historical high groundwater depth of 5.7 feet below ground surface in April 16, 2018. It should be noted that the referenced well has around 791 feet lower in elevation than the project site area.

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.

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

6.3 Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2019 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 167 mg/kg.

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

TABLE 6.3
WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS

Dissolved Sulfate (SO_4) in Soil, % by Weight	Exposure Severity	Exposure Class	Maximum w/cm Ratio	Minimum Concrete Compressive Strength	Cementitious Materials Type (ASTM C150)
0.0167	Not Applicable	S0	N/A	2,500 psi	No Restriction

The water-soluble chloride concentration detected in saturation extract from the soil samples was 31 mg/kg. In addition, testing performed on a near surface soil resulted in a minimum resistivity value of 11,110 ohm-centimeters. Based on the results, these soils would be considered to have a “Negligible” potential to buried metal objects (per National Association of Corrosion Engineers, Corrosion Severity Ratings).

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

7. GEOLOGIC SETTING

The subject site is located at the western end of Basin and Range Province and approximately 3.9 miles southwest of Lone Pine, California, situated mainly between Sierra Nevada Mountains to the west, Alabama Hills, Owens Valley and Inyo Mountains beyond to the east.

The Basin and Range province is characterized by interior drainage with lakes and playas, and a horst and graben structure of subparallel, fault-bounded ranges separated by down-dropped basins. The horst and graben structure of the Basin and Range province is the result of extensional tectonics throughout the region and has resulted in a topography that is dominated by isolated, north-trending mountain ranges separated by desert basins. Indian Wells Valley and Searles Valley are both graben structures and are the most southern and western most of the Basin and Range desert basins. The Indian Wells Valley region including Porterville and Inyokern areas bordered by the Sierra Nevada to the west, the Argus Range to the east, the Coso Range to the north, and the El Paso Mountains and Spangler Hills to the south. The Searles Valley is teardrop shaped and is generally bounded on the west by the Argus Range and Spangler Hills, on the east and northeast by the Slate Range, and on the south by the Garlock fault zone. Both the Indian Wells and Searles Valleys are closed topographical depression with drainage from the surrounding hills and mountains directed inward toward China and Searles Lakes, respectively.

Based on review of Geologic Map of the Lone Pine 15' quadrangle[1], Inyo County, California, the site is underlain by of Inactive Alluvial fan gravels (Qai), and sandy deposits. Shallow refusal was encountered at the site around 1.5 to 4 feet BSG. The shallow auger refusal appears to be due to possible boulder material. Large boulders of approximately 3 feet wide were noted near the test borings at the surface. In addition, adjacent geologic unit, Qgy, to the immediate west were described as Younger alluvial and debris flow gravels (Pleistocene), comprised of abundant subangular to subrounded cobble sand boulders (noted to have diameters greater than 2 meters) of plutonic rocks. It appears the younger alluvial fan gravels (Qai) are relatively thin and overlying the Pleistocene younger alluvial and debris flow gravel (Qgy) deposits.

8. GEOLOGIC HAZARDS

8.1 Faulting and Seismicity

Based on mapping and historical seismicity, the seismicity of the Lone Pine Area has been generally considered to be in a moderate to high seismic area. The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards nor within an Alquist-Priolo Earthquake Fault (Special Studies) Zone, therefore, a site-specific fault study investigation by an Engineering Geologist is not required. No active faults with the potential for surface fault rupture are

[1] Stone, Paul, Dunne, G.C., Moore, J.G., and Smith, G.I., 2000, Geologic map of the Lone Pine 15' quadrangle, Inyo County, , California: US Geological Survey: Geologic Investigations Series Map I-2617, scale 1:62,500

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.

Soils on site are classified as Site Class D (Default) in accordance with Chapter 16 of the California Building Code. The proposed structures are determined to be in Seismic Design Category D.

To determine the distance of known active faults within 100 miles of the site, we used the United States Geological Survey (USGS) web-based application *2008 National Seismic Hazard Maps - Fault Parameters*. Site latitude is 36.5574° North; site longitude is -118.0978° West. The ten closest active faults are summarized below in Table 9.1.

TABLE 9.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Independence	2.64	7.2
Owens Valley	2.80	7.3
Hunter Mountain Connected	16.85	7.6
So Siera Nevada	24.06	7.5
Birch Creek	31.80	6.6
White Mountains	32.27	7.4
Panamint Valley	39.00	7.4
Little Lake	46.41	7.4
Deep Springs	46.47	6.8
Death Valley (No)	51.03	7.3

The faults tabulated above and numerous other faults in the region are sources of potential ground motion. However, earthquakes that might occur on other faults throughout California are also potential generators of significant ground motion and could subject the site to intense ground shaking.

8.2 Surface Fault Rupture

Based on mapping and historical seismicity, the seismicity of the Lone Pine Area has been generally considered high by the scientific community. The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards nor within an Alquist-Priolo Earthquake Fault (Special Studies) Zone. Therefore, a site specific fault study investigation by an Engineering Geologist is not required. 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.

The nearest faults to the project site are primarily associated with Owens Valley Faults (Lone Pine) around 2.0 miles to the east and Independence fault to the west 2.4 miles from the project site. There are no known active fault traces in the immediate project vicinity.

8.3 Ground Shaking

Based on the 2022 CBC, a Site Class D (Default) was selected for the site based on soil conditions encountered during this investigation and our understanding of local geologic deposts. Table 9.6.1

includes design seismic coefficients and spectral response parameters, based on the 2022 California Building Code (CBC) for the project foundation design.

Based on Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps, the estimated design peak ground acceleration adjusted for site class effects (PGA_M) was determined to be 0.766 g (based on both probabilistic and deterministic seismic ground motion).

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

8.4 Liquefaction

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

In general, the near surface soils encountered consisted of silty sand with gravel to 4 feet below ground surface, where practical refusal encountered at 4 feet depth BSG.

Based on review of the California Earthquake Hazard Zone application (EQ Zapp: California Earthquake Hazards Zone Application) and Hazard Maps, the project site is not located within a known liquefaction hazard zone.

Based on the lack of groundwater within the upper 50 feet BSG, the potential for liquefaction/seismic settlement to impact the site is considered low.

8.5 Lateral Spreading

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

8.6 Landslides

There are no known landslides located 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

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

9.1.2 The near surface soils encountered comprised of medium dense to very dense silty sand with some gravel to around 2 feet BSG, the silty sand is underlain by dense to very dense silty sand with trace of clay and gravel to the maximum depth explored of 4 feet BSG. It should be noted that auger refusal was encountered due to very dense materials and possible cobbles. Groundwater was not encountered during this investigation.

9.1.3 Based on the laboratory testing results, the near surface soils are considered to have a very low expansion potential, moderate compressibility and when wetted under a nominal load of 2 kips per square foot, exhibited a moderate collapse potential.

9.1.4 The proposed tower may be supported on either shallow spread foundations or cast in drilled hole pier (CIDH) foundations. <u>Due to the presence of very dense shallow materials (either bedrock or boulder material), the use of CIDH piers may require specialty drilling equipment, such as core barrels or rock bits to achieve the required pier depth.</u> Tower improvement foundations constructed in accordance with the recommendations included in this report may be designed considering total and differential static settlement of 1 inch total and ½ inch differential in 40 feet.

9.1.5 Based on the chemistry testing performed, the near surface soils have 'negligible' potential for sulfate attack on concrete. Also, these soils would be considered to have a "negligible corrosive" potential to buried metal objects.

9.1.6 Provided the recommendations included in this report are followed, the proposed tower foundation may be designed utilizing a drilled pier caisson foundation system.

9.1.7 Provided the site is graded in accordance with the recommendations of this report and foundations constructed as described herein, we estimate that total static settlement of about 1 inch and differential static settlement of ½ inch in 40 feet should be anticipated.

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

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

9.2 Surface Drainage

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

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

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

9.3.4 Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report.

9.3.5 Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. The upper 6 to 8 inches of soils containing, vegetation, roots and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. In addition, any existing concrete and asphalt materials shall be removed from areas of proposed improvements and stockpiled separately from excavated soil material. The stripped vegetation, asphalt and concrete materials will not be suitable for use as Engineered Fill or within 5 feet of building pads or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

9.3.6 If desired to support the proposed communication tower on shallow spread mat foundations, the area of the proposed shallow foundation should be over-excavated to 12 inches below the bottom of foundation or 36 inches below preconstruction site grade, whichever is greater. The bottom of excavation should be scarified 12 inches, moisture conditioned slightly above optimum, and compacted as engineered fill. <u>Over-excavation for shallow foundations intended for tower support should include an over-build zone extending a minimum of 5 feet beyond foundations.</u>

It should be noted that boulder sized materials may be present within 2 to 4 feet BSG. If encountered, these boulder sized materials should be removed from where they protrude the bottom of planned over-excavation. Due to potential for uneven surface due to boulder removal, voids due to boulder material may be backfilled with a 2-sack sand/cement slurry to the bottom of the proposed over-excavation. After placement of the 2-sack sand/cement slurry, the remaining excavation may be backfilled with approved soils compacted as engineered fill. At a minimum, foundations should be supported on a uniform layer of 12 inches of compacted engineered fill prepared as recommended above.

9.3.7 Areas of proposed lightly loaded structures such as equipment shelters or retaining walls, should be prepared by over-excavation to 12 inches below foundations, 12 inches below adjacent site grade, or to the depth required to remove any undocumented fills (if encountered), whichever is greater. Upon approval, the bottom of excavation should be scarified a minimum of 8 inches, moisture conditioned to slightly above optimum, and compacted to 92 percent of the maximum density. The over-excavation zone should extend horizontally a minimum of 3 feet beyond foundations.

9.3.8 Areas of equipment slabs, should be prepared by over-excavation to 12 inches below the bottom of the recommended aggregate base section, 12 inches below adjacent site grade, or to the depth required to remove any undocumented fills (if encountered), whichever is greater. Upon approval, the bottom of excavation should be scarified a minimum of 12 inches, moisture conditioned to slightly above optimum, and compacted to 92 percent of the maximum density. The over-excavation zone should extend horizontally a minimum of 3 feet beyond equipment slabs.

Equipment slabs on grade should be supported on a minimum of 4 inches of Class 2 aggregate base compacted to 95 percent relative compaction over subgrade soils prepared as recommended above.

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

9.3.10 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.

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

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

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

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

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

9.4 Soil and Excavation Characteristics

9.4.1 Based on the soil conditions encountered in our soil boring, the upper (within 2 to 3 feet BSG) near surface soils can be excavated with moderate to high effort using conventional excavation equipment.

The Contractor should note the potential for boulder sized material or possible bedrock to be encountered during drilling. Excavations and CIDH pier excavations will likely encounter these materials which would require specialized equipment, such as rock teeth, rock core barrels, etc. to achieve required excavation depths. In addition, large soil cemented particles or rock boulders will require mechanical processing to reduce particle size and produce a well-blended mixture prior to use as engineered fill.

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 The near surface soils identified as part of our investigation are generally considered damp to moist. Earthwork operations conducted during inclement periods of the year are likely to encounter moist potentially unstable soils which may require removal to a stable bottom. Exposed native soils exposed as part of site grading operations shall not be allowed to dry out and should be kept continuously moist prior to placement of subsequent fill.

9.5 Materials for Fill

9.5.1 On-site soils are considered suitable for use as engineered fill below foundations and below the aggregate base layer recommended below concrete slabs on grade. On-site soils used as engineered fill should not contain deleterious matter, organic material, or rock material larger than 3 inches in maximum dimension. On-site soils should be well blended to prevent nesting of larger particles.

9.5.2 Import soil intended for use as Imported Engineered Fill soil, should be well-graded, slightly cohesive silty sand or sandy silt. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 9.5.2

TABLE 9.5.2
IMPORT ENGINEERED FILL REQUIREMENTS

Percent Passing 3-inch Sieve	100
Percent Passing No.4 Sieve	75-100
Percent Passing No 200 Sieve	15-40
Maximum Plasticity Index	15
Organic Content, Percent by Weight	Less than 3%
Maximum Expansion Index (ASTM D4829)	10

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 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.

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

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

9.5.6 On-site soils used as Engineered Fill should be moisture conditioned to slightly above optimum moisture content, and compacted to 92 percent relative compaction (ASTM D 1557).

9.5.7 Imported Engineered Fill should be moisture conditioned to slightly above optimum moisture content, and compacted to 92 percent relative compaction (ASTM D1557).

9.5.8 All Engineered Fill should be placed in lifts no thicker than will allow for adequate bonding and compaction (typically a maximum of 6 to 8 inches in loose thickness).

9.5.9 Caltrans Class 2 Aggregate Base shall meet the minimum requirements of Section 26 of the Caltrans Standard Specifications (Current Edition). Prior to importing, the Contractor should provide documentation that the aggregate base meets the requirements for Class 2 aggregate base (i.e. gradation, durability, R-value, sand equivalent, etc.) to the Owner and Salem for review. All aggregate base should be compacted to a minimum of 95 percent relative compaction.

9.5.10 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 2022 CBC, our recommended parameters are shown below. These parameters were determined using Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps by location website (https://seismicmaps.org/), in accordance with the 2022 CBC. The Site Class was determined based on the soils encountered during our field exploration.

TABLE 9.6.1
2022 CBC SEISMIC DESIGN PARAMETERS

Seismic Item	Symbol	Value	2016 ASCE 7 or 2022 CBC Reference
Site Coordinates (Datum = NAD 83)		36.5574 Lat -118.0978 Lon	
Site Class	--	D	ASCE 7-16 Table 20.3
Soil Profile Name	--	(Default)	ASCE 7-16 Table 20.3
Risk Category	--	I/II	CBC Table 1604.5
Site Coefficient for PGA	F_{PGA}	1.200	ASCE 7-16 Table 11.8-1
Peak Ground Acceleration (adjusted for Site Class effects)	PGA_M	0.766 g	ASCE 7-16 Equation 11.8-1

Seismic Item	Symbol	Value	2016 ASCE 7 or 2022 CBC Reference
Seismic Design Category	SDC	D	ASCE 7-16 Table 11.6-1 & 2
Mapped Spectral Acceleration (Short period - 0.2 sec)	S_S	1.969 g	CBC Figure 1613.2.1(1)
Mapped Spectral Acceleration (1.0 sec. period)	S_1	0.508 g	CBC Figure 1613.2.1(3)
Site Class Modified Site Coefficient	F_a	1.200	CBC Table 1613.2.3(1)
Site Class Modified Site Coefficient	F_v	**1.792***	CBC Table 1613.2.3(2)
MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$	S_{MS}	1.675g	CBC Equation 16-20
MCE Spectral Response Acceleration (1.0 sec. period) $1.5{*}S_{M1} = 1.5 (F_v S_1)$	$1.5{*}S_{M1}$	**1.366g***	CBC Equation 16-21 / ASCE 7-16 Supplement 3
Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec)	S_{DS}	1.117g	CBC Equation 16-22
Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period)	S_{D1}	**0.910 g***	CBC Equation 16-23
Short Period Transition Period (S_{D1}/S_{DS}), Seconds	T_S	**0.815***	ASCE 7-16, Section 11.4.6
Long Period Transition period (seconds)	T_L	8	ASCE 7-16, Figures 22-14 through 22-17

Note: * Values Fv, SM1, and SD1 determined per ASCE 7-16, Supplements 1 and 3. Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, Structures on Site Class D, with S1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. The value reported for SM1 includes a 50% increase in accordance with exceptions listed in ASCE 7-16 - Supplement 3. In the event a site specific ground motion analysis is required, SALEM should be contacted for these services.

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

9.7 Shallow Foundations for Proposed Tower (if selected)

9.7.1 The site is suitable for use of a shallow foundation for the equipment pads consisting of isolated spread footings bearing in compacted engineered fill as recommended in this report.

9.7.2 Shallow foundations used for support of communication towers, should be at least 24 inches wide extend to at least 36 inches below lowest adjacent grade.

9.7.3 Foundations supported on the depths of engineered fill recommended in this report may be designed based on an allowable bearing capacity of 3,000 pounds per square foot. This value may be increased by 1/3 for wind and seismic loading.

9.7.4 Total static settlement of 1 inch and differential static settlement of ½ inch in 40 feet should be anticipated for design.

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

9.7.6 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical footing faces. The frictional and passive resistance of the soil may be combined provided that a 50 percent reduction of the frictional resistance factor is used in determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combination, as applicable, of the 2022 CBC that includes wind and earthquake loads. The upper 12 inches should be neglected in design.

9.7.7 Reinforcement for spread footings should be designed by the project structural engineer.

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

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

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

9.8 Cast in Drilled Hole (CIDH) Pier Foundations

9.8.1 Tower footings should be designed by the project Structural Engineer based on design service loads and maximum lateral forces expected at site. Larger piers and deeper embedment may be needed to resist the lateral seismic forces or winds per the 2022 California building Code. We recommend that each of the tower legs bear on CIDH piers with a minimum diameter of 24 inches and extend to a minimum depth of 10 feet below site grade. As described in this report, based on shallow refusal depth and potential to encounter boulders, the Contractor should anticipate the need for specialized equipment, i.e. rock teeth, rock core barrels, etc., to achieve the recommended pier embedment depth of 10 feet.

9.8.2 Based on the granular nature of the soils encountered collapse should be anticipated. The Contractor should anticipate the need for temporary casing. The casing should be bedded into the soil unit near the design depth prior to placement of the reinforcing steel and concrete, and casing extraction.

9.8.3 Cast in drilled hole pier foundations may be designed based on total static settlement of 1 inch and differential static settlement of ½ inch in 40 feet or between piers, whichever is less.

9.8.4 Skin friction within the upper 1 foot BSG should be neglected in design. The downward load capacity of the piers (extending to at least 10 feet BSG), may be designed based on an allowable skin friction value of 250 pounds per square foot. Provided the CIDH piers are cleaned of loose soils. An end bearing value of 5,000 pounds per square foot may be considered for design. These values may be increased by 1/3 for short duration wind and seismic loading.

9.8.5 The allowable uplift resistance of the pier foundations may be assumed to be 150 pounds per square foot, plus the weight of the CIDH pier.

9.8.6 Provided the approval of the project structural engineer, passive resistance in the upper portion of the piles to a depth of 1 foot should be neglected for design. The allowable passive resistance of the soils below a depth of 1 foot below site grade may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot to a maximum of 3,500 pounds per square foot. These values may be increased by one-third for short duration wind and seismic loads. No other increases should be applied to the allowable passive pressure.

9.8.7 If desired, the cast in drilled hole piers may be designed using LPILE and the parameters presented in Table 9.8.7. The lateral loading criteria is based on the assumption that the load application is applied at the ground level, flexible cap connections applied.

TABLE 9.8.7 -LPILE PARAMETERS

Depth, BSG (Feet)	L-Pile Soil Type	Effective Unit Weight (pcf)	Angle of Internal Friction (degrees)	Static Modulus of Subgrade Reaction, K (pci)
1-4	Sand (Reese)	125	37	90
>4	Sand (Reese)	130	40	225

9.9 CIDH Pier Construction

9.9.1 The project structural engineer should prepare a specification for the construction of the deep foundations as part of the construction documents. The specifications should be consistent with the recommendations included in this report.

9.9.2 Concrete should be placed in the drilled shaft as soon as possible following drilling. Concrete should be placed by tremie pipe method from the bottom of the drilled shaft.

9.9.3 Temporary casing (if required) used for support of drilled pile excavations during construction should be slowly removed from the shaft excavation during placement of concrete while ensuring the casing is not raised above the level of the concrete during shaft construction. The bottom of the casing should be lifted slowly as the concrete is deposited and kept at least two feet below the top of the concrete to avoid sloughing soils from mixing with the concrete.

9.9.4 Casing (where used) should be able to withstand the external pressures of the caving soils. The outside diameter of the casing should not be less than the diameter of the cast-in-drilled hole concrete pile.

9.9.5 Drilled holes for pile foundations should be drilled within 2 degrees of vertical. The rebar cage should be suspended within 2 degrees of vertical in the center of the excavation. Minimum concrete cover, as specified by the project design engineer, should be maintained throughout the length of the excavation. These conditions should be verified and documented by Salem Engineering Group during construction.

9.9.6 Salem Engineering Group should inspect the drilling of the shafts to verify that the materials encountered are consistent with those evaluated during our geotechnical engineering investigation. **This inspection should be conducted during drilling and prior to placement of reinforcing steel and concrete.**

9.9.7 All loose materials should be removed from the drilled shaft excavations prior to placement of reinforcing steel and concrete by use of a clean-out bucket or other acceptable methods to ensure removal of all loose materials.

9.10 Shallow Foundations for Lightly Loaded Improvements

9.10.1 The site is suitable for use of conventional shallow foundations for equipment shelter structures consisting of continuous strip footings bearing in compacted engineered fill prepared in accordance with section 9.3 of this report.

9.10.2 It is recommended that continuous footings to be utilized for lightly loaded foundations such as the equipment shelter should have a minimum width of 12 inches, and a minimum embedment depth of 12 inches below lowest adjacent pad grade.

9.10.3 Shallow foundations supported on engineered fill as recommended in this report may be designed based on an allowable bearing capacity of 2,500 pounds per square foot. This value may be increased by 1/3 for wind and seismic loading.

9.10.4 Total static settlement of 1 inch and differential static settlement of ½ inch in 40 feet should be anticipated for design. The footing excavations should not be allowed to dry out any time prior to pouring concrete.

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

9.10.6 Lateral resistance for footings can alternatively be developed using an estimated allowable equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical footing faces. An increase of one-third is permitted for wind and earthquake and seismic loading. The upper 6 inches should be neglected in design.

9.10.7 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.

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

9.10.9 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.

9.11 Concrete Equipment Slabs-on-Grade

9.11.1 Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading.

9.11.2 The equipment shelter slab should have a minimum thickness of 6 inches. The slab should be reinforced as a minimum with No. 4 reinforcement bars at 18 inches on center, each way. Thicker floor slabs with increased concrete strength and reinforcement should be designed wherever large vehicular loads, heavy concentrated loads, heavy equipment, or machinery is anticipated.

9.11.3 Equipment slabs supported on a minimum of 4 inches of Class 2 aggregate base over subgrade soils prepared in accordance with Section 9.3 of this report may be designed based on an allowable bearing capacity of 1,500 pounds per square foot.

9.11.4 Equipment slabs should be designed with thickened edges extending to a minimum of 6 inches below the bottom of slab or to the bottom of the recommended aggregate base section, or as required by the structural engineer, whichever is greater.

9.11.5 Resistance to lateral footing displacement can be computed using an estimated allowable coefficient of friction factor of 0.37 acting between the base of foundations and the supporting Engineered Fill.

9.11.6 Lateral resistance can be developed using an estimated allowable equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical native footing faces. The upper 1 foot of subgrade soils should be neglected in design.

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

9.12 Temporary Excavations

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

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

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

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

RECOMMENDED EXCAVATION SLOPES

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

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

9.12.6 Braced shorings should be designed for a maximum pressure distribution of 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.

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

9.13 Underground Utilities

9.13.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than 3 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches and compacted to at least 90 percent relative compaction to at least 1 percent above optimum moisture content. The upper 12 inches of trench backfill within asphalt or concrete paved areas shall be moisture conditioned to at or above optimum moisture content and compacted to at least 95 percent relative compaction.

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

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

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

10. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

10.1 Plan and Specification Review

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

10.2 Construction Observation and Testing Services

10.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design.

If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

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

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

11. LIMITATIONS AND CHANGED CONDITIONS

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

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

If site conditions change due to natural processes or human intervention on the property or adjacent to the site, or changes occur in the nature or design of the project, or if there is a substantial time lapse between the submission of this report and the start of the work at the site, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed by SALEM and the conclusions of our report are modified or verified in writing.

The validity of the recommendations contained in this report is also dependent upon an adequate testing and observations program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction. SALEM has prepared this report for the exclusive use of the owner and project design consultants.

SALEM does not practice in the field of corrosion engineering. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, that manufacturer's recommendations for corrosion protection be closely followed. Further, a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of concrete slabs and foundations in direct contact with native soil. The importation of soil and or aggregate materials to the site should be screened to determine the potential for corrosion to concrete and buried metal piping.

The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either express or implied, are made as to the professional advice provided under the terms of our agreement and included in this report.

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

Respectfully Submitted,

SALEM ENGINEERING GROUP, INC.

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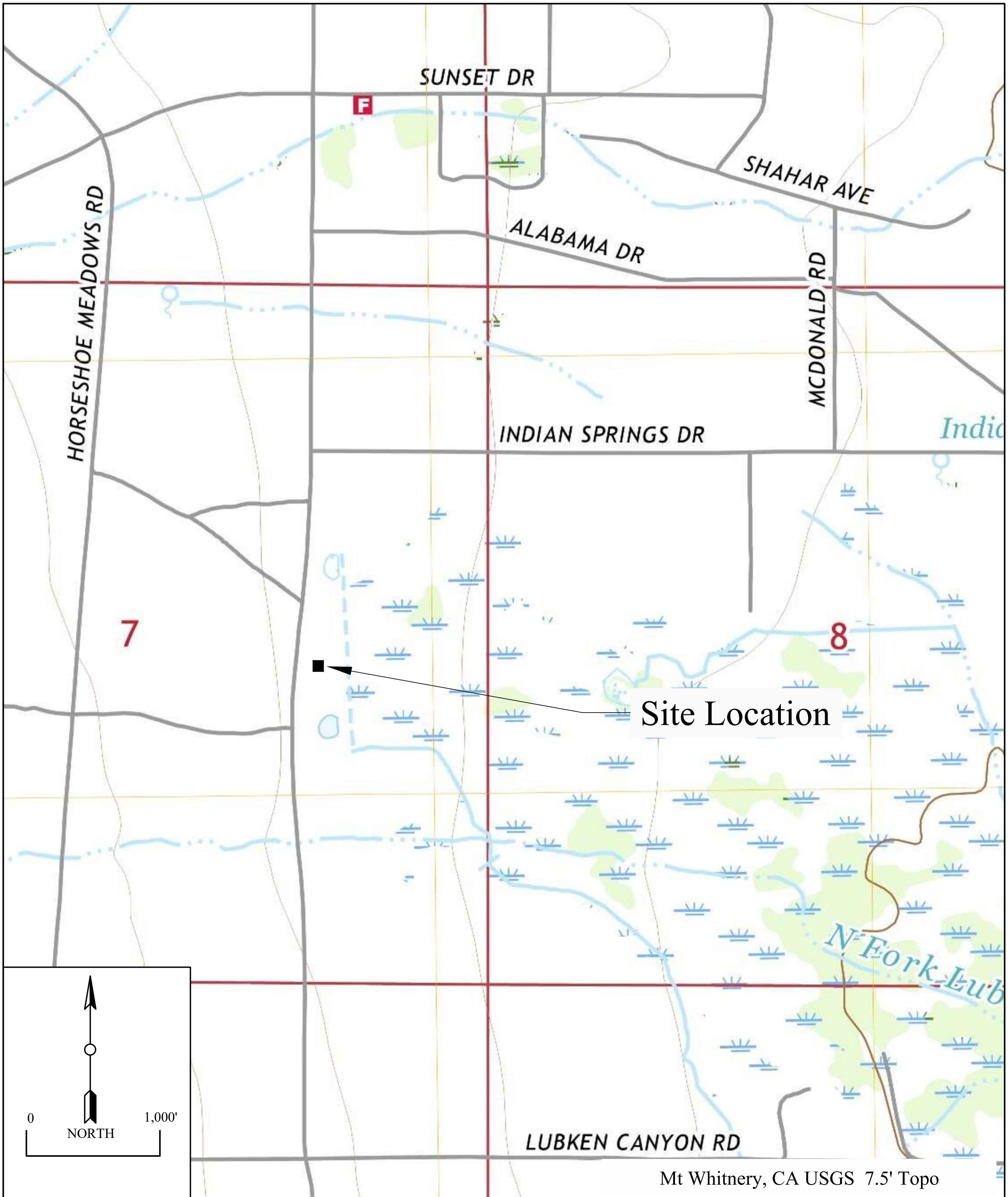


VICINITY MAP	SCALE: 1" = 1,000'	DATE: June 14, 2024	SALEM engineering group, inc.
GEOTECHNICAL ENGINEERING INVESTIGATION Proposed Verizon 105' Monopole Tower 1203 Lubken Canyon Road Lone Pine, California	DRAWN BY: PC	APPROVED BY: AD	
	PROJECT NO. 1-224-0478	FIGURE NO. 1	

SITE PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
Proposed Verizon 105' Monopole Tower
1203 Lubken Canyon Road
Coordinates: (36.557419, -118.097589)
Lone Pine, California

SCALE: 1" = 200'	DATE: June 14, 2024
DRAWN BY: PC	APPROVED BY:AD
PROJECT NO. 1-224-0478	FIGURE NO. 2

SALEM
engineering group, inc.

APPENDIX

A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation was conducted on June 10, 2024 and included a site visit, subsurface exploration, and soil sampling. The location of the exploratory boring is shown on the Site Plan, Figure 2. The boring log for our exploration is presented in figures following the text in this appendix. The boring was located in the field using existing reference points. Therefore, the actual boring location may deviate slightly.

Our boring was performed using a truck-mounted CME-45 drill rig equipped with 6-inch diameter hollow-stem augers. Sampling in the boring was accomplished using a hydraulic 140-pound hammer with a 32.5-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler 18 inches into the soil. Penetration and/or Resistance tests were performed at selected depths. The resistance/N-Value obtained from driving was recorded based on the number of blows required to penetrate the last 12 inches. The driving energy was provided by an auto-trip hammer weighing 140 pounds, falling 4inches. Relatively undisturbed MCS soil samples were obtained while performing this test. Bag samples of the disturbed soil were obtained from the SPT samples and auger cuttings. All samples were returned to our Fresno laboratory for evaluation. The test borings were backfilled with excavated soil upon completion of drilling and sampling.

Subsurface conditions encountered in the exploratory boring 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 log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the log contains both observed and interpreted data. We determined the lines designating the interface between soil materials on the log using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field log was revised based on subsequent laboratory testing.

SALEM engineering group, inc.

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

Project Number: 1-224-0478

Date: June 10, 2024

Client: Sequoia Deployment Services, Inc.

Project: Proposed Verizon 105' Monopole Tower

Location: 1203 Lubken Canyon Road, Lone Pine, CA

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R

Drill Type: CME-45 **Elevation:** 4617 feet AMSL

Auger Type: 6-5/8in Hollow Stem Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	40/6 50/2	SM	Silty SAND with Gravel, very dense, brown, fine to coarse grained.	>50	1.9	115.7	Very Hard Augering
4615							
	50/3		Grades as above;	>50	2.2	--	
			End of boring at 4 feet BSG due to Auger refusal and Very Dense Materials (boulders and possible bedrock).				
5							
4610							
10							
4605							
15							
4600							
20							
4595							
25							
4590							

Notes: *Dirt road with bushes.*

Figure Number

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

Project Number: 1-224-0478

Date: June 10, 2024

Client: Sequoia Deployment Services, Inc.

SALEM engineering group, inc.

Project: Proposed Verizon 105' Monopole Tower

Location: 1203 Lubken Canyon Road, Lone Pine, CA

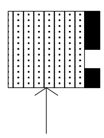
Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R

Drill Type: CME-45 **Elevation:** 4614 feet AMSL

Auger Type: 6-5/8in Hollow Stem Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	40/6 50/6	SM	Silty SAND; very dense, brown, dry, fine to coarse grained.	>50	2.2	115.7	
	50/6		Grades as above; with gravel	>50	1.8	--	
			End of boring at 2 feet BSG due to Very Dense Material. (boulders and possible bedrock)				
4610 / 5							
4605 / 10							
4600 / 15							
4595 / 20							
4590 / 25							



Notes: *Dirt road with bushes.*

Figure Number

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

Project Number: 1-224-0478

Date: June 10, 2024

Client: Sequoia Deployment Services, Inc.

SALEM engineering group, inc.

Project: Proposed Verizon 105' Monopole Tower

Location: 1203 Lubken Canyon Road, Lone Pine, CA

Drilled By: Salem Engineering Group, Inc. **Logged By:** C.R

Drill Type: CME-45 **Elevation:** 4614 feet AMSL

Auger Type: 6-5/8in Hollow Stem Auger **Initial Depth to Groundwater:** N/A

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

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	N-Values blows/ft.	Moisture Content %	Dry Density, PCF	Remarks
0	45/6 50/6	SM	Silty SAND; very dense, brown, dry, fine to coarse grained, with gravel.	>50	2.6	115.7	
			End of boring at 1.5 feet BSG due to very Dense Material. (boulders and possible bedrock)				
4610							
5							
4605							
10							
4600							
15							
4595							
20							
4590							
25							

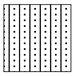
Notes: *Dirt road with bushes.*

Figure Number

KEY TO SYMBOLS

Symbol Description

Strata symbols

 **Silty Sand**

Misc. Symbols

 **Drill rejection**

Soil Samplers

 **California sampler**

Notes:

Granular Soils
Blows Per Foot (Uncorrected)

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

Cohesive Soils
Blows Per Foot (Uncorrected)

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

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

APPENDIX

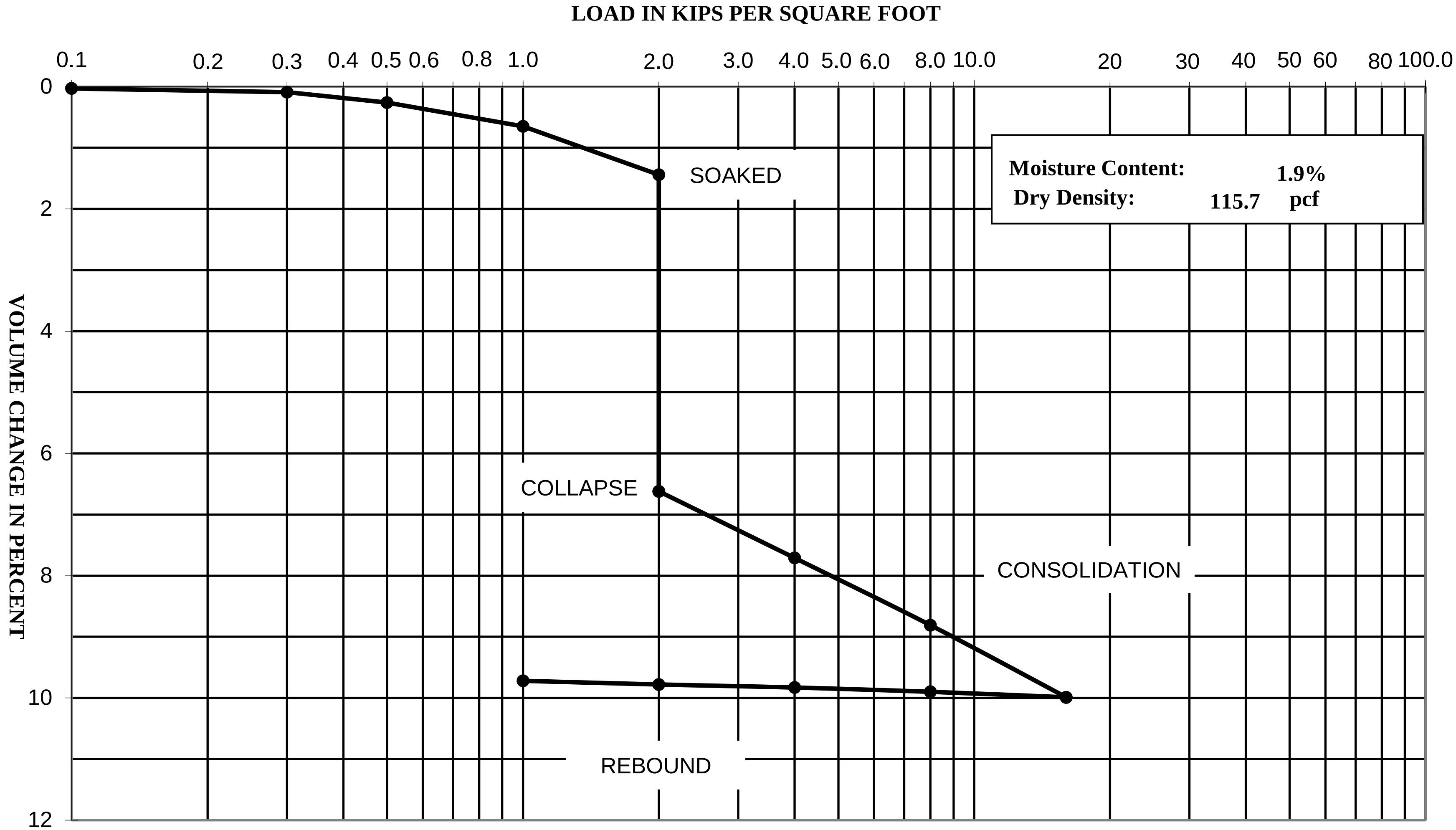
B

APPENDIX B
LABORATORY TESTING

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

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435



Project Name: Verizon 105' Monopole Tower - Lone Pine, CA

Project Number: 1-224-0478

Boring: B-1 @ 0'

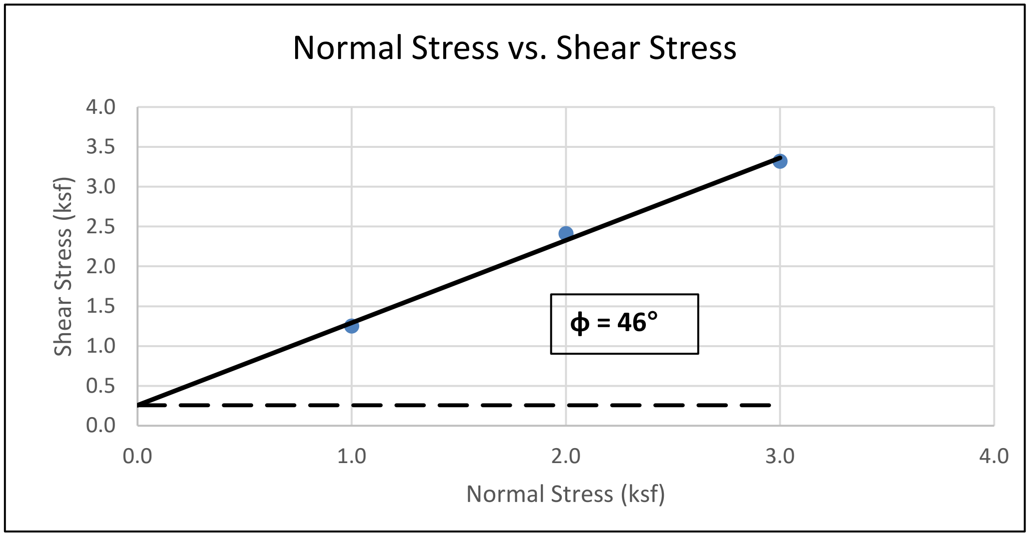
SALEM engineering group, inc.

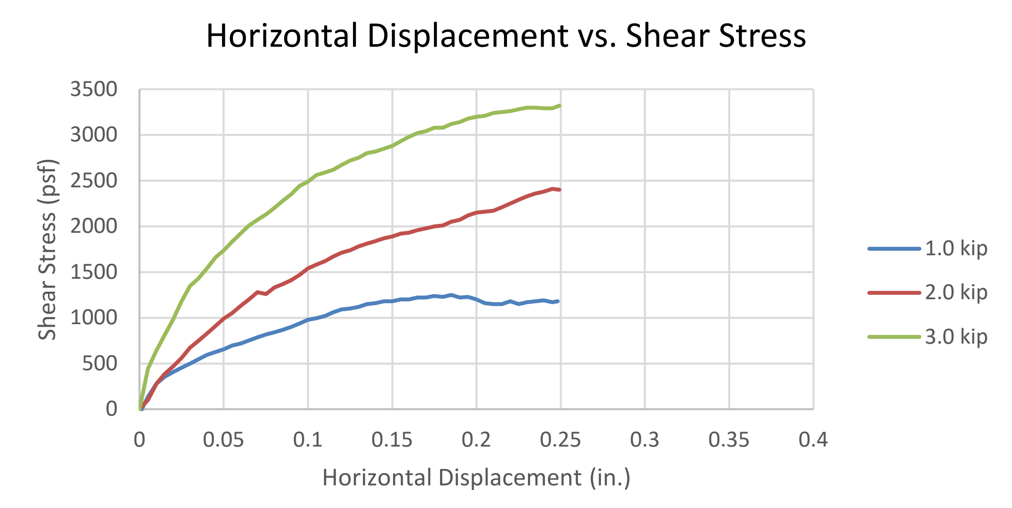
Direct Shear Test (ASTM D3080)

Project Name: Verizon 105' Monopole Tower - Lone Pine, CA
Project Number: 1-224-0478
Client:
Boring: B-1B @ 0'
Soil Type: Silty SAND with Gravel (
Sample Type: Undisturbed Ring
Tested By: NL
Reviewed By:
Date of Test: 6/19/24
Test Equipment: GeoComp ShearTrac II

	Loading		
	1.0 kip	2.0 kip	3.0 kip
Normal Stress (ksf)	1.00	2.00	3.00
Shear Rate (in/min)	0.0040	0.0040	0.0040
Peak Shear Stress (ksf)	1.25	2.41	3.32

Initial Height of Sample (in)	1.000	1.000	1.000
Post-Consol. Sample Height (in.)	0.975	0.954	0.934
Post-Shear Sample Height (in.)	0.975	0.944	0.927
Diameter of Sample (in)	2.4	2.4	2.4
Initial (pre-shear) Values			
Moisture Content (%)	2.6		
Dry Density (pcf)	116.6	115.0	111.7
Saturation %	16.2	15.5	14.1
Void Ratio	0.43	0.45	0.49
Consolidated Void Ratio	0.39	0.38	0.39
Final (post-shear) Values			
Final Moisture Content (%)	15.0	14.7	14.1
Dry Density (pcf)	110.3	115.8	114.7
Saturation %	71.5	74.1	70.0
Void Ratio	0.56	0.53	0.54



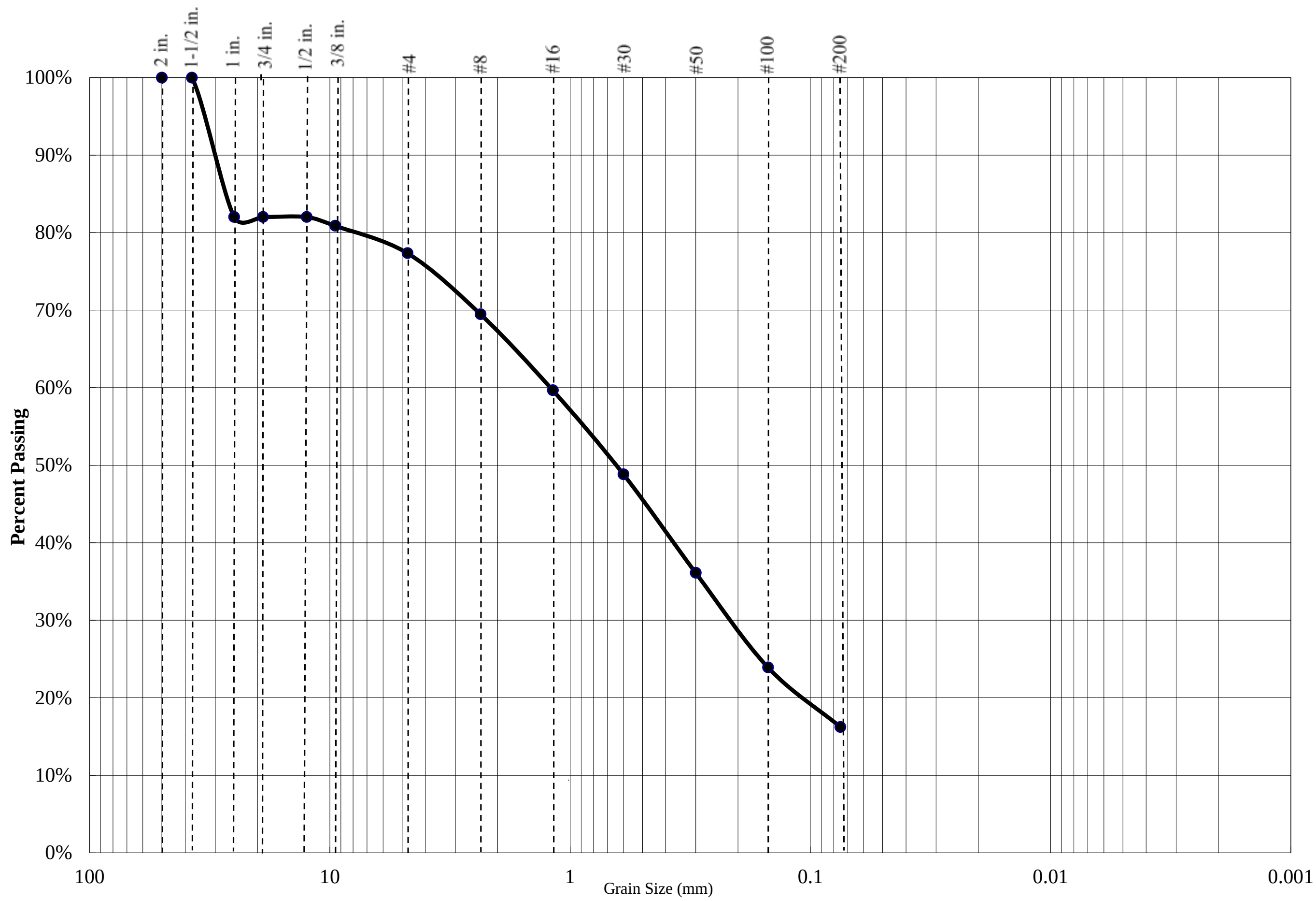


Peak Shear Strength Values	
Slope	1.04
Friction Angle	46
Cohesion (psf)	257

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
23%	61%	16%

Sieve Size	Percent Passing
3/4 inch	82.0%
1/2 inch	82.0%
3/8 inch	80.9%
#4	77.3%
#8	69.5%
#16	59.7%
#30	48.8%
#50	36.1%
#100	23.9%
#200	16.2%

Atterberg Limits		
PL=	**LL=**	**PI=**

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

USCS CLASSIFICATION
Silty SAND with Gravel (SM-GM)

Project Name: Verizon 105' Monopole Tower - Lone Pine, CA

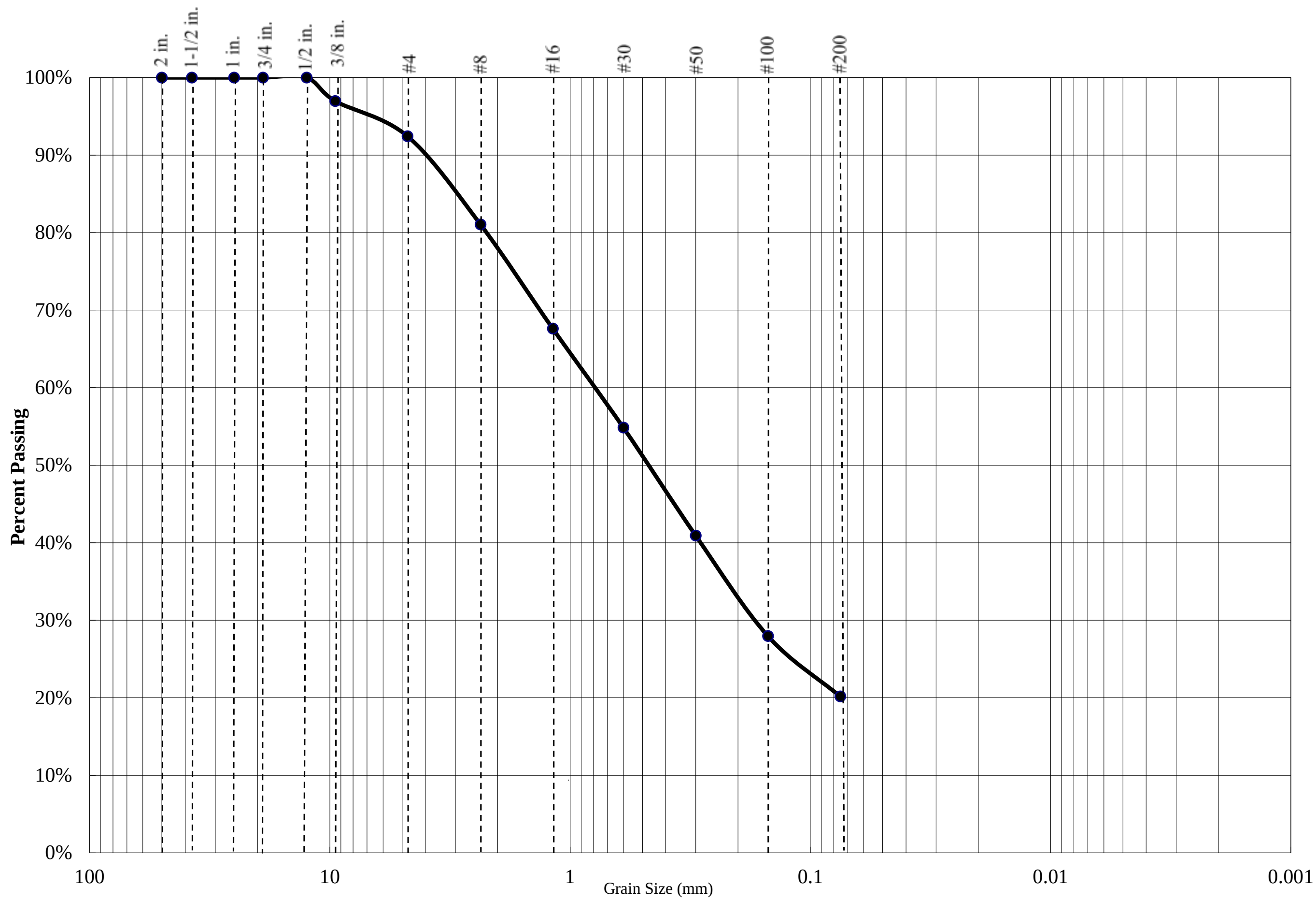
Project Number: 1-224-0478

Boring: B-1 @ 1' - 4'

SALEM engineering group, inc.

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
8%	72%	20%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	97.0%
#4	92.4%
#8	81.1%
#16	67.6%
#30	54.8%
#50	40.9%
#100	27.9%
#200	20.2%

Atterberg Limits		
PL=	LL=	PI=

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

USCS CLASSIFICATION
Silty SAND (SM)

Project Name: Verizon 105' Monopole Tower - Lone Pine, CA

Project Number: 1-224-0478

Boring: B-1A @ 1.5'

SALEM engineering group, inc.

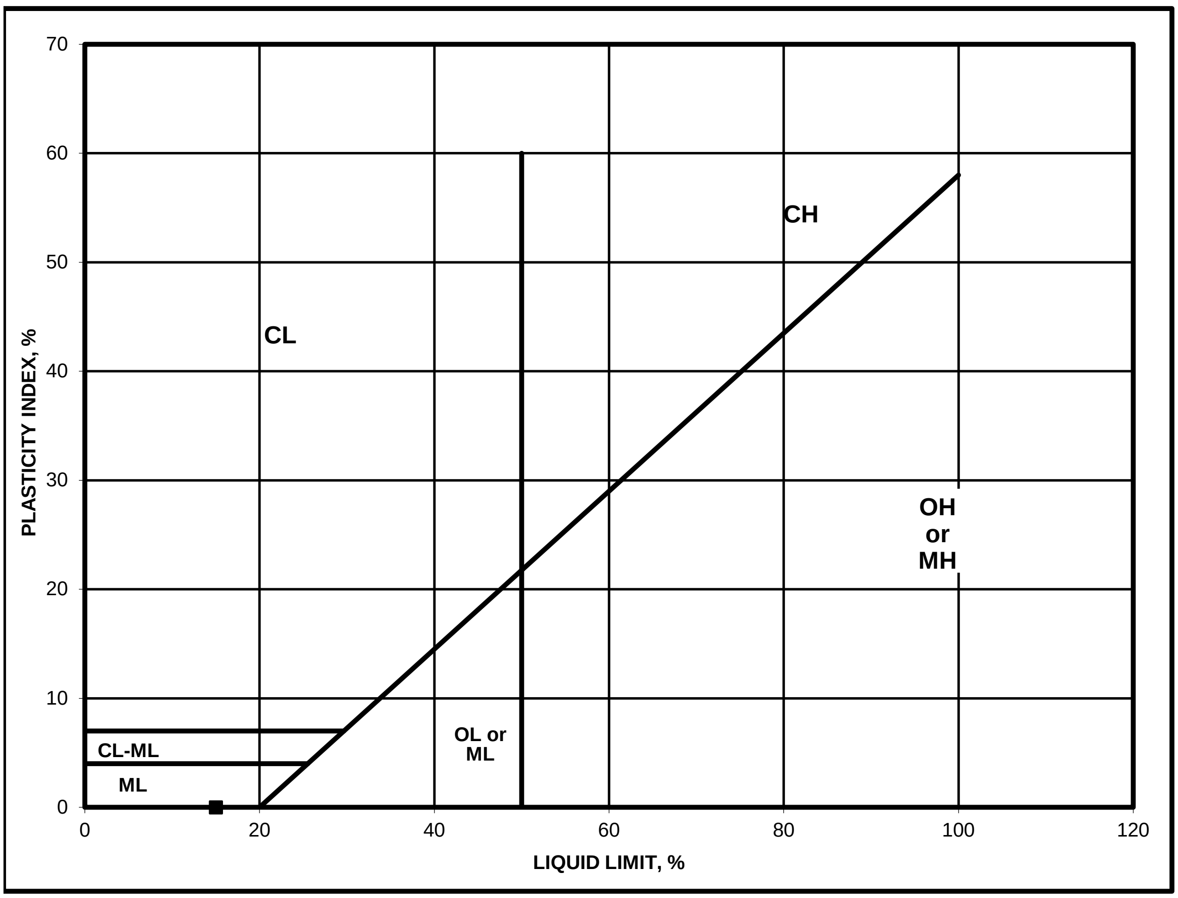
Atterberg Limits Determination
ASTM D4318

Project Name: Verizon 105' Monopole Tower - Lone Pine, CA
Project Number: 1-224-0478
Date Sampled: 6/10/24 Date Tested: 6/18/24
Sampled By: SEG Tested By: MC
Sample Location: B-1 @ 1' - 4'

Run Number	Plastic Limit			Liquid Limit		
	1	2	3	1	2	3
Weight of Wet Soil & Tare	30.97	27.46	27.07	32.60	33.11	32.93
Weight of Dry Soil & Tare	29.81	26.71	26.28	31.17	31.61	31.44
Weight of Water	1.16	0.75	0.79	1.43	1.50	1.49
Weight of Tare	22.11	21.67	21.05	21.63	21.49	21.61
Weight of Dry Soil	7.70	5.04	5.23	9.54	10.12	9.83
Water Content	15.1	14.9	15.1	15.0	14.8	15.2
Number of Blows				27	23	19

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

Plasticity Index : 0
Unified Soil Classification : ML



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EXPANSION INDEX TEST
ASTM D4829

Project Name: Verizon 105' Monopole Tower - Lone Pine, CA
Project Number: 1-224-0478
Date Sampled: 6/10/24 Date Tested: 6/17/24
Sampled By: SEG Tested By: MC
Sample Location: B-1 @ 1' - 4'
Soil Description: Silty SAND with Gravel (SM-GM)

Trial #	1	2	3
Weight of Soil & Mold, g.	605.1		
Weight of Mold, g.	187.8		
Weight of Soil, g.	417.3		
Wet Density, pcf	125.9		
Weight of Moisture Sample (Wet), g.	850.0		
Weight of Moisture Sample (Dry), g.	784.7		
Moisture Content, %	8.3		
Dry Density, pcf	116.2		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	49.9		

Time	Inital	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	-0.002	-0.0021	--	--	-0.0024

Expansion Index $_{measured}$ = 0
Expansion Index $_{50}$ = 0.0

Expansion Index = 0

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

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CHEMICAL ANALYSIS

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

Project Name: Verizon 105' Monopole Tower - Lone Pine, CA
Project Number: 1-224-0478
Date Sampled: 6/10/24 Date Tested: 6/19/24
Sampled By: SEG Tested By: MC
Soil Description: Silty SAND with Gravel (SM-GM)

Sample Number	Sample Location	Soluble Sulfate SO_4-S	Soluble Chloride Cl	pH
1a.	B-1 @ 1' - 4'	170 mg/kg	32 mg/kg	6.9
1b.	B-1 @ 1' - 4'	170 mg/kg	31 mg/kg	6.9
1c.	B-1 @ 1' - 4'	160 mg/kg	31 mg/kg	6.9
Average:		**167 mg/kg**	**31 mg/kg**	**6.9**

SALEM engineering group, inc.

SOIL RESISTIVITY
CTM 643

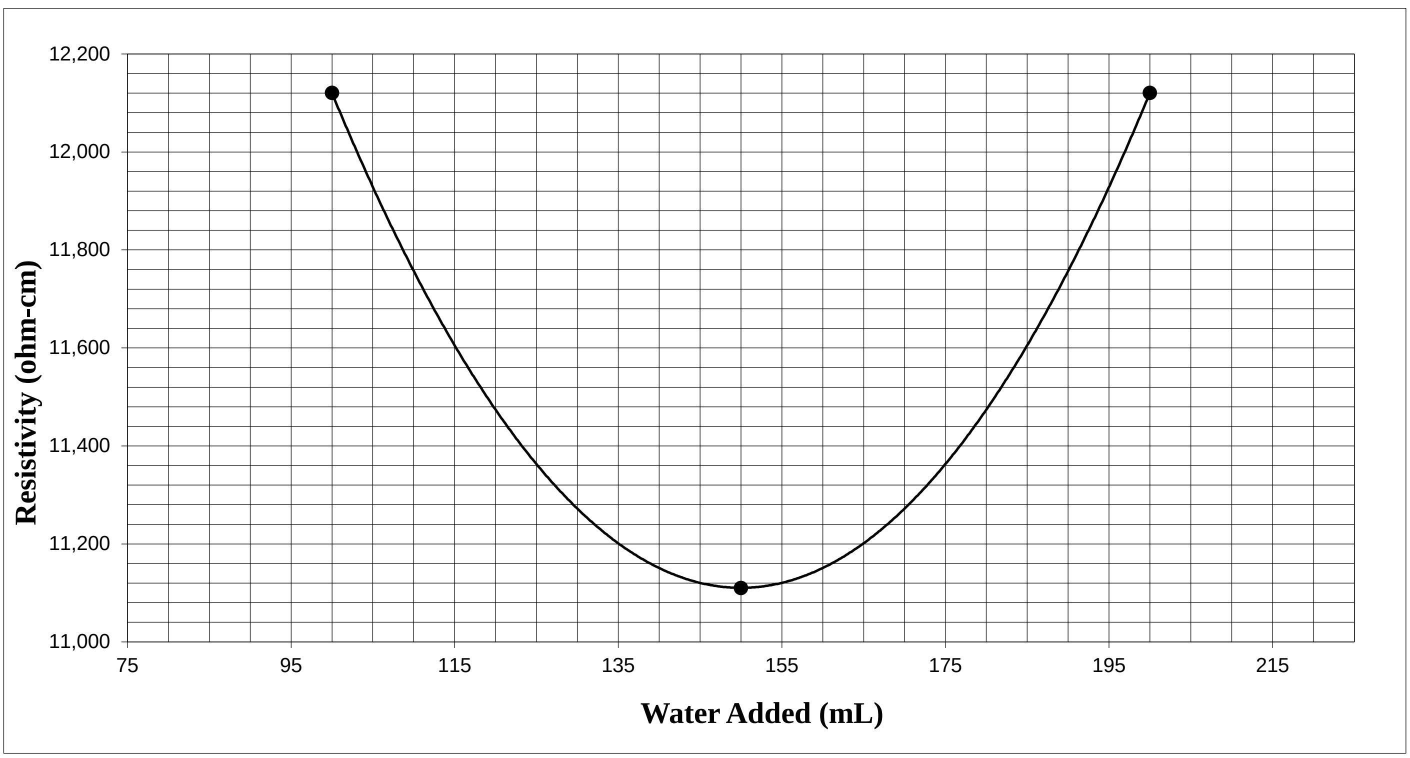
Project Name: Verizon 105' Monopole Tower - Lone Pine, CA
Project Number: 1-224-0478 Date Sampled: 6/10/24
Sample Location: B-1 @ 1' - 4' Sampled By: SEG
Soil Description: Silty SAND with Gravel (SM-GM) Date Tested: 6/18/24 Tested By: AV

Chloride Content: 31 mg/Kg
Sulfate Content: 167 mg/Kg
Soil pH: 6.9

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

Test Data:

Trial #	Water Added (mL)	Meter Dial Reading	Multiplier Setting	Resistance (ohms)	Resistivity (ohm-cm)
1	100	1.2	10,000	12,000	12,120
2	150	1.1	10,000	11,000	11,110
3	200	1.2	10,000	12,000	12,120



Minimum Resistivity:	11,111 ohm-cm

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APPENDIX

C

APPENDIX C
GENERAL EARTHWORK SPECIFICATIONS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

13.0 PREPARATION OF THE SUBGRADE - The Contractor shall prepare the surface of the various subgrades receiving subsequent pavement courses to the lines, grades, and dimensions given on the plans. The upper 12 inches of the soil subgrade beneath the pavement section shall be compacted to a minimum relative compaction of 90 percent based upon ASTM D1557. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

14.0 AGGREGATE BASE - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class 2 material, ¾-inch or 1½-inches maximum size. The aggregate base material shall be compacted to a minimum relative compaction of 90 percent based upon ASTM D1557. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

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

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