GEOTECHNICAL EVALUATION
PROPOSED FACILITY IMPROVEMENTS – THE RIVERS EDGE RANCH
ASSESSOR'S PARCEL NUMBER (APN) 0453-062-14-0000
33433 HAYNES ROAD
LUCERNE VALLEY, SAN BERNARDINO COUNTY, CALIFORNIA

PREPARED FOR

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PROJECT NO. 3977-CR **JULY 29, 2024**

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July 29, 2024
Project No. 3977-CR

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Attention: Ms. Fayres Hall

Subject: **Geotechnical Evaluation**
Proposed Facility Improvements – The Rivers Edge Ranch
Assessor's Parcel Number (APN) 0453-062-14-0000
33433 Haynes Road
Lucerne Valley, San Bernardino County, California

Dear Ms. Hall:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical Evaluation for proposed improvements to be constructed on the Rivers Edge Ranch (Assessor's Parcel Number (APN) 0453-062-14-0000), which is addressed as 33433 Haynes Road, in the Lucerne Valley area of San Bernardino County, California. This report presents the results of GeoTek's evaluation and discussion of findings.

Based upon review, it is GeoTek's opinion that site development appears feasible from a geotechnical viewpoint. Final site development and grading plans should be reviewed by this firm as they become available, as it will be necessary to provide appropriate recommendations for intended specific site development as those plans become refined.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call GeoTek.

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the general geotechnical conditions on the project site and provide geotechnical recommendations as deemed appropriate. Services for this study included the following:

- Research and review of available geologic and geotechnical data pertinent to the site,
- Perform a reconnaissance of the site,
- Excavation of six (6) exploratory borings to depths ranging between about 19.5 to 51 feet below existing grades,
- Collection of bulk and relatively undisturbed in-situ samples of the onsite materials for classification and laboratory testing,
- Laboratory testing of select soil samples obtained during the site exploration to determine pertinent physical and chemical soil properties,
- Review and evaluation of site seismicity and geologic hazards, and
- Compilation of this Geotechnical Evaluation report which presents GeoTek's findings, conclusions and recommendations for the site development.

The intent of this report is to aid in the evaluation of the site for future development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report will likely need to be updated based on review of final site development plans. These plans should be provided to GeoTek for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The approximate 17.97-acre roughly rectangular-shaped project site is addressed as 33433 Haynes Road, in the Lucerne Valley area of San Bernardino County, California. The project site is also identified as San Bernardino County Assessor's Parcel Number (APN) 0453-062-014-0000. The site facility is a working ranch (The Rivers Edge Ranch) that provides services to men in recovery. The site can generally be accessed from Haynes Road, a paved, improved street located adjacent to the northern site boundary (see Figure 1 – Site Location and Topography Map).

The Rivers Edge Ranch facility currently consists of, but is not necessarily limited to, an administration building, a garage, a chapel, sports courts and numerous animal corrals. The site can be considered as having a relatively flat topography, with surface drainage generally directed down to the south-southwest.

The site is located at an elevation of approximately 2,895 feet above mean sea level within the northeastern portion of the site, with an elevation relief of about ten (10) feet down to the southwestern portion.

The project site is located within an area characterized by vacant land, and sparse single-family residences. The site is bounded by Haynes Road, followed by vacant land and the remnants of a single-family residence to the north. A single-family residence bounds the site to the west. Verdugo Road, a poorly maintained dirt road, followed by vacant land and a single-family residence bounds the site to the east. An easement for the future alignment of Gypsy Lane, followed by vacant land and a single-family residence bound the site to the south.

2.2 PROPOSED DEVELOPMENT

Based on review of the *Rivers Edge Ranch Conceptual Grading Plan*, prepared by Albert A. Webb Associates and dated June 12, 2024, GeoTek understands that it is proposed to expand the existing administration building for housing, construct a bunk house, construct an all-weather fire access road and associated improvements. Stormwater facilities are not anticipated for the project.

The proposed structure(s) are anticipated to be of wood-framed construction, to be supported by conventional shallow spread footings which will most likely include a conventional slab-on-grade floor. Maximum foundation loads of 40 kips and 2.5 klf for column and wall loads, respectively, are anticipated for the proposed structure(s).

Sewage disposal is anticipated to be by construction of five (5) new septic systems, each with either a 1,000 gallon or 2,000-gallon septic tank, and seepage pits. This Evaluation does not address the design recommendations related to the proposed On-Site Wastewater Treatment System (OWTS). A separate report will need to be prepared for the design of the OWTS.

Cuts and fills of less than about five (5) feet are expected to be required to reach design grades. Cut and fill slopes, as well as significant retaining walls are not anticipated for the subject site.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Final site development plans should be reviewed by GeoTek when they become available. Additional geotechnical field exploration, analyses, and recommendations may be necessary upon review of site development plans.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

After completing an initial site reconnaissance, GeoTek excavated six (6) exploratory hollow-stem auger borings on July 5, 2024, with a truck-mounted drill rig equipped with hollow-stem augers to depths ranging from approximately 19.5 to 51 feet below existing grades at the boring locations. The approximate locations of the GeoTek excavations are shown on the Boring Location Map (Figure 2). A geologist from GeoTek logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory borings are included in Appendix A.

Relatively undisturbed soil samples were recovered at various intervals in the geotechnical borings with a California sampler. The California sampler is a 3-inch outside diameter, 2.5-inch inside diameter, split barrel sampler lined with brass rings. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for testing and evaluation. The California sampler test data are presented on the boring logs in Appendix A.

In Boring B-1, standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, and split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The sampler penetration test data are presented on the Logs of Boring for Boring B-1.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected bulk and relatively undisturbed soil samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the subsurface materials encountered and to evaluate the soil physical properties for use in the engineering design and analysis. GeoTek's test results along with a brief description and relevant information regarding testing procedures are included in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The property is situated in the Mojave Desert geomorphic province. The Mojave Desert province is a wedge-shaped area that is enclosed on the southwest by the San Andreas fault zone, the Transverse Ranges province and the Colorado Desert province, on the north and northeast by the Garlock fault zone, the Tehachapi Mountains and the Basin and Range province, and on the east by the Nevada and Arizona state lines, and the Colorado River. The area is dominated by broad alluviated basins that are mostly aggrading surfaces that are receiving non-marine continental deposits from the adjacent upland areas.

The primary fault zones of the area are found in the western half of the province and have a general northwest-southeast trend. These zones are the San Andreas, Helendale, Lenwood and Lockhart in the subject site vicinity. In addition to these major zones, there are numerous secondary fault zones in the area and many smaller fault zones in the eastern half of the province. Many of the secondary fault zones in the province have a general east-west trend.

Based on review of available geologic mapping by others, and the subsurface exploration performed, the site is underlain by alluvium (Dibblee, T.W., 1964). The nearest known active faults are the Helendale-South Lockhart fault zone (Helendale section) and the Lenwood-Lockhart fault zone (Lenwood section) located approximately 5.3 and 8.3 miles southeast and northeast of the site, respectively.

4.2 EARTH MATERIALS

A brief description of the earth materials encountered in GeoTek's explorations is presented in the following sections. Based on the recent site exploration performed, the project site was observed to be underlain by alluvium.

4.2.1 Alluvium

Alluvial deposits were encountered in all exploratory borings to the maximum depth explored of about 51 feet below existing grades. These alluvial deposits consist of interbedded silty sands and sandy silts (SM and ML soil types based on the Unified Soil Classification System). The alluvial soils were generally observed to be brown to yellow brown in color, slightly moist, and medium dense to relatively dense. Detailed logs of the subsurface conditions of the site are presented in Appendix A.

Based on the results of laboratory testing, the surficial soils are considered to have a "Very Low" ($0 \leq EI \leq 20$) Expansion Index (EI) when tested in general accordance with ASTM D 4829 test procedures. Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327), classifying the soils as having a "negligible" (i.e., "S0") sulfate exposure classification per Table 19.3.1.1 of ACI 318-19. The upper site soils exhibit a "very low" potential for hydroconsolidation (settlement upon wetting with or without additional loading). The laboratory test results are provided in Appendix B.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

Surface water was not noted during GeoTek's field investigations. If encountered during earthwork construction, surface water on this site will most likely be the result of precipitation or possibly some minor surface run-off from immediately surrounding properties. Overall site area drainage is generally to the south-southwest, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

Based on review of the Federal Emergency Management Agency (FEMA), Flood Insurance Rate Map No. 06071C-5900H, the project site is located in a "Zone D" Flood Hazard Zone, which corresponds to "Areas with possible but undetermined flood hazards. No flood hazard analysis has been conducted".

4.3.2 Groundwater

Groundwater was not encountered within any of the borings performed, to the maximum depth explored (51 feet below existing grades) at the time of drilling. Based on a review of current groundwater information provided by the California Department of Water Resources, Water Data Library, the historic depth to high groundwater is estimated to be greater than 50 feet below existing grade. Based on the above, groundwater is not anticipated to be a factor during the site grading.

4.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within a State of California designated *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007).

The nearest known active faults are the Helendale-South Lockhart fault zone (Helendale section) and the Lenwood-Lockhart fault zone (Lenwood section) located approximately 5.3 and 8.3 miles southeast and northeast of the site, respectively.

The site has not yet been evaluated by the County of San Bernardino as being located within an earthquake fault zone, or liquefaction and landslide hazard zone.

4.4.1 Seismic Design Parameters

The site is located at approximately 34.54461 North Latitude and -116.93305 West Longitude. A Site Class "D" is considered appropriate for this site due to the presence of relatively dense soil across the site. Site spectral accelerations (Ss and S_1), for 0.2 and 1.0 second periods for a Class "D" site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. As noted, using the ASCE 7-16 option on the SEAOC/OSHPD website, the values for S_{M1} and S_{D1} are reported as "null-See Section 11.4.8 (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

For a site Class D, an exception to performing a site-specific ground motion analysis is allowed in ASCE 7-16 where S_1 exceeds 0.2 provided the value of the seismic response coefficient, Cs, is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of $T \leq 1.5Ts$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5Ts$ or Eq. 12.8-4 for $T > T_L$.

Assuming that the Cs value calculated by and used by the structural engineer allows for the exclusion per ASCE 7-16, noted above, then a site-specific ground motion analysis is not required. For this assumption and condition, the following seismic design parameters, based on the 2015 National Earthquake Hazards Reduction Program (NEHRP), are presented on the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.124g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.405g
Site Coefficient for Site Class "D", F_a	1.05
Site Coefficient for Site Class "D", F_v	1.895
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.181g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.767g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.787g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.511g
Site Modified Peak Ground Acceleration, PGA_M	0.533g
Seismic Design Category	D

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

4.4.2 Surface Fault Rupture

The site is in a seismically active region; however, no active or potentially active faults are known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007). Based on review of the current geologic mapping, the potential for surface fault rupture at the site is considered remote.

4.4.3 Liquefaction and Seismically Induced Settlement

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, consolidation and settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

The site has not been identified by the California Geological Survey or the County of San Bernardino as being located within an earthquake induced liquefaction hazard zone. Due to the relatively dense nature of the subsurface soils and depth to historic high groundwater in exceedance of 50 feet, it is GeoTek's opinion that the liquefaction potential at the site is very low.

4.4.4 Other Seismic Hazards

Due to the relatively dense nature of the underlying alluvium as encountered in the borings, the seismic induced ("dry sand") settlements are estimated to be minimal.

Evidence of ancient landslides or slope instabilities at this site was not observed during GeoTek's field investigations. Based on the relatively flat nature of the site, landslides and slope instabilities are not expected to adversely impact the subject development.

The potential for a secondary seismic hazard such as a seiche (seismically-induced waves in a closed body of water) is considered negligible due to the distance from a closed body of water.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Development of the site appears feasible from a geotechnical viewpoint. The following recommendations should be incorporated into the design and construction phases of development.

Site development plans should be reviewed by GeoTek prior to construction, and additional recommendations may be required as plans become available.

The upper site soils were found to be disturbed, non-uniform, and are considered unsuitable for support of foundation loading in their current state. Remedial grading, in the form of removal and replacement of the upper site soils as engineered fill will be required for the support of the proposed structure(s) and associated improvements.

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the County of San Bernardino, the 2022 California Building Code (CBC), and recommendations contained in this report. The General Grading Guidelines included in Appendix C outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix C.

Final site grading plans should be reviewed by this office when they become available. Additional recommendations will likely be offered subsequent to review of these plans.

5.2.2 Site Clearing

In areas of planned grading and improvements, the locations of existing utilities should be determined. Existing utilities should be relocated or abandoned. The site should be cleared of existing structures, pavements, slabs, trees (including root balls), vegetation and other deleterious materials. Debris should be properly disposed of off-site. Voids resulting from site clearing should be backfilled with engineered fill.

Any on-site wastewater treatment system encountered during site grading should be properly disposed of in accordance with the County of San Bernardino Department of Environmental Health guidelines.

5.2.3 Remedial Grading

Due to the presence of disturbed soils and anticipated disturbance of the upper site soils from the site clearing and demolition operations and to provide a more uniform bearing for the proposed structure foundation and slabs-on-grade, it is recommended that the soils be removed beneath the planned building footprint to a depth of at least three (3) feet below existing grade, or two (2) feet below the base of the proposed foundations, whichever is greater. The lateral extent of this recommended over-excavation should extend at least five (5) feet beyond the building limits, where obtainable. Removal bottoms should be relatively uniform in soil type and not adversely porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures.

Following site clearing operations, over-excavation and lowering of site grades, where necessary, it is recommended that the exposed subgrade soils beneath all surface improvements be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the geotechnical engineering representative. The proof rolling equipment should include at least 4 passes, two in each perpendicular direction. All soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative. Following proof rolling and removal of any unsuitable bearing soil, the exposed subgrade should be scarified to a depth of about 12 inches, be moisture conditioned to slightly above the soil's optimum moisture content and then be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM D-1557.

5.2.4 Oversized Rock Disposal

Although unlikely, oversized cobbles, boulders and rock fragments may be encountered during rough grading and utility trench operations. If encountered, on-site disposal of oversized materials is possible, provided the oversized materials are placed as recommended within Appendix C. Alternatively, over-sized materials can be exported from the site.

5.2.5 Pavement Areas

Any undocumented fill should be removed below proposed pavement areas. If no undocumented fill is encountered or is relatively shallow, the natural soils should be overexcavated to a depth of 12 inches below existing grade or 12 inches below proposed finished grade, whichever is deeper. Finished grade is defined as the top of the subgrade. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.6 Hardscape Areas

Any undocumented fill should be removed below hardscape areas. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.7 Preparation of Excavation Bottoms

A representative of this firm should observe the bottom of all excavations. Upon approval, the exposed soils should be scarified to a depth of approximately 12 inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.8 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Portland cement concrete (PCC) that is to be removed from the site may be pulverized into fragments not exceeding three inches in greatest dimension and incorporated into the fill at all levels. Asphalt concrete (AC) should not be incorporated into site fill and should instead be disposed of off-site. Engineered fill should be placed in loose lifts with a thickness of eight inches or less and moisture conditioned to at least two percent above the optimum moisture content.

Below and within the proposed building area, engineered fill should be moisture conditioned to at least optimum moisture content and be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures if a shallow foundation system is used.

Below other structural elements, such as hardscape areas and walls independent of the building, engineered fill should be moisture conditioned to at least optimum moisture content and be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.9 Excavation Characteristics

Excavation of the on-site soils are expected to be feasible utilizing heavy-duty grading equipment in good operating condition. All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the on-site materials should be stable at 1.5:1 (horizontal: vertical) inclinations for cuts less than five feet in height.

5.2.10 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 10 to 15 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.1 foot may be anticipated for the underlying soils.

5.2.11 Trench Excavations and Backfill

Temporary trench excavations within the on-site materials should be stable at 1.5:1 (h:v) inclinations for short durations during construction and where cuts do not exceed ten feet in height. It is anticipated that temporary cuts to a maximum height of four feet can be excavated vertically.

Trench excavations should conform to Cal-OSHA regulations. The contractor should have a competent person, per OSHA requirements, on site during construction to observe conditions and to make the appropriate recommendations.

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined per ASTM D 1557 test procedures). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for road pavements should be compacted to at least 95 percent relative compaction. On-site materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than six inches are removed.

Compaction should be achieved with a mechanical compaction device. Ponding or jetting of trench backfill is not recommended. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2022 CBC, are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Based on the results of GeoTek's laboratory testing, the on-site materials are classified as having "Very Low" ($0 \leq EI \leq 20$) Expansion Index per ASTM D 4829. Additional laboratory testing should be performed at the completion of site grading to verify the expansion potential of the near-surface soils.

The foundation elements for the proposed structures should bear entirely in engineered fill soils as recommended in this report. Foundations should be designed in accordance with the *2022 California Building Code* (CBC). A summary of the foundation design recommendations is presented in the following table:

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONALLY REINFORCED FOUNDATIONS	
Design Parameter	"Very Low" Expansion Index (0≤EI≤20)
Minimum Foundation Depth (inches below lowest adjacent grade)	12
Minimum Foundation Width (Inches)*	12
Minimum Slab Thickness (Inches)	4 – Actual
Minimum Slab Reinforcing	6" x 6" – W1.4 x W1.4 welded wire fabric, or No. 3 reinforcing bars placed at 18-inch o.c. each way, placed in middle of slab
Minimum Reinforcement for Continuous Footings, Grade Beams, and Retailing Wall Footings	Two No. 4 reinforcing bars, one placed near the top and one near the bottom
Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)	Minimum 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

*Code minimums per Table 1809.7 of the 2022 CBC should be complied with.

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

An allowable bearing capacity of 2,500 pounds per square foot (psf) may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 18 inches deep. This value may be increased by 400 psf for each additional 12 inches in depth and by 250 psf for each additional 12 inches in width to a maximum value of 3,500 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads). These bearing values contain a minimum factor of safety of three (3).

The recommended allowable bearing capacity is based on an estimated maximum post-construction settlement of 1-inch. Differential settlement of about one-half of the total settlement over a horizontal distance of 40 feet could result. Seismically induced settlements are anticipated to be negligible. The project structural engineer, foundation engineer, and earth retention structure designer should incorporate these settlement estimates into the design, as appropriate.

The on-grade slabs may be designed as beams on an elastic foundation, based on a subgrade modulus of reaction (k-value) of 200 pounds per cubic inch (pci).

The passive earth pressure may be computed as an equivalent fluid having a density of 400 psf per foot of depth, to a maximum earth pressure of 4,000 psf for footings founded on engineered fill. The allowable passive earth pressure contains a factor of safety of 1.5. A coefficient of friction between soil and concrete of 0.40 may be used with dead load forces. Passive pressure and frictional resistance may be combined without reduction. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements.

A grade beam, a minimum of 12 inches wide and 12 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

It is recommended that control joints be placed in two directions spaced approximately 24 to 36 times the thickness of the slab in inches. These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer.

5.3.2 Slab Moisture and Vapor Retarding System

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2022 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2022 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as the result of construction related punctures (e.g., stake penetrations, tears, punctures from walking on the aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. It is GeoTek's opinion that a minimum ten mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and atmospheric conditions.

Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeance) to achieve the desired performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

5.3.3 Miscellaneous Foundation Recommendations

To minimize moisture penetration beneath the slab-on-grade areas, utility trenches should be backfilled with engineered fill, lean concrete, or concrete slurry where they intercept the perimeter footing or thickened slab edge.

Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

5.3.4 Foundation Setbacks

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least 5 feet and need not exceed 40 feet.
- The outside bottom edge of all footings should be set back a minimum of H/2 (where H is the slope height) from the face of any ascending slope. The setback should be at least 7 feet and need not to exceed 15 feet. Where a retaining wall is constructed at the toe of the slope, the height of the slope should be measured from top of the wall to the top of the slope.
- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom inside edge of the wall footing.
- The bottom of any proposed foundations for structures should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom of the nearest excavation.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1.1 General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete vertical retaining walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations should be embedded a minimum of 12 inches into engineered fill.

Retaining wall foundations should be designed in accordance with Section 5.3 of this report. Structural needs may govern and should be evaluated by the project structural engineer.

All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization.

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of Section 5.2.8 in this report.

In general, cantilever earth retention structures, which are designed to yield at least 0.001H, where H is equal to the height of the earth retention structure, may be designed using the "active" condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at top, such as typical basement walls) should be designed using the "at-rest" condition.

In addition to the design lateral forces due to retained earth, surcharges due to improvements, such as an adjacent building or traffic loading, should be considered in the design of the earth retention structures. Loads applied within a 1:1 (horizontal: vertical) projection from the surcharge on the stem of the earth retention structure should be considered in the design.

Final selection of the appropriate design parameters should be made by the designer of the earth retention structures.

5.4.1.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (horizontal: vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	41
2:1	64

*The design pressures assume the backfill material has an expansion index less than or equal to 20. Backfill zone includes area between back of the wall to a plane (1:1 horizontal: vertical) up from bottom of the wall foundation (on the backside of the wall) to the ground surface.

For walls with a retained height greater than six (6) feet, an incremental seismic pressure should be included into the wall design. Where needed, it is recommended that an incremental seismic load for unrestrained walls with level backfill of $8H^2$ [Units: pounds per lineal foot of wall] should be included into the wall design to account for seismic loading conditions, where H is the retained height of the wall. For unrestrained walls with a retained height greater than six (6) feet with backfill of a 2:1 [horizontal: vertical] gradient, a dynamic load increment of $14H^2$ should be included in the wall design. These incremental seismic loads may be assumed to be applied at a point 1/3H above the base of the wall.

5.4.1.3 Retaining Wall Backfill and Drainage

The wall backfill should also include a minimum one (1) foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The upper 24 inches should consist of compacted on-site materials. The rock should be separated from the earth with filter fabric. The presence of other materials might necessitate revision of the parameters provided and modification of the wall designs. The backfill materials should be placed in lifts no greater than eight (8) inches in thickness and compacted to a minimum of 90% relative compaction as determined by ASTM D 1557 test procedures. Proper surface drainage needs to be provided and maintained.

As an alternative to the drain, rock and fabric, a pre-manufactured wall drainage product (example: Mira Drain 6000 or approved equivalent) may be used behind the retaining wall. The wall drainage product should extend from the base of the wall to within two (2) feet of the ground surface. The subdrain should be placed in direct contact with the wall drainage product.

Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four (4)-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one (1) cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

5.4.1.4 Restrained Retaining Walls

Retaining walls that will be restrained at the top that support level backfill or that have reentrant or male corners, should be designed for an equivalent at-rest fluid pressure of 62 pcf, plus any applicable surcharge loading. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.4.1.5 Additional Retaining Wall Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

5.5.1 Asphalt Concrete (AC) Pavement

Although planned final grades beneath the street improvements within the site are not yet known, the following preliminary pavement design recommendations are based on typical traffic indices associated with similar developments. Additionally, an R-Value of 22 was obtained from testing in general accordance with CA Test 301 test procedures. This R-Value has been assumed for the preliminary design of the project pavement sections. Preliminary pavement thickness design is based on the Caltrans Highway Design Manual (2018).

Once the traffic loading information becomes more defined, revision to the pavement design recommendations may be warranted. It is recommended that the final pavement design be based on R-value testing of the as-graded subgrade soils within the pavement areas.

Based on the assumptions noted above the following preliminary pavement recommendations are provided for the site:

Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0	3.5	6.0
6.0	4.0	8.0

Traffic Indices (TIs) used in the pavement design are designated by the County of San Bernardino and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure. Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D1557 test procedures. All materials and methods of construction should conform to the requirements of the County of San Bernardino.

5.5.2 Portland Cement Concrete (PCC) Pavement

For the proposed vehicle parking areas, it is recommended that a minimum of 5 inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures be utilized. For truck access/truck delivery/emergency vehicle drives, it is recommended that a minimum of 6 inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density be utilized. This section should also be used in heavy truck traffic areas such as loading docks, fire lanes, and trash dumpster pads and approaches.

Requirements of Section 90 of Caltrans Standard Specifications, and various ACI and ASTM standards regarding mixing and placing concrete should be followed. The PCC pavement should have a minimum modulus of rupture of 500 pounds per square inch and a minimum 28-day compressive strength of 4,000 pounds per square inch. Concrete should incorporate 1-inch maximum size aggregate and should be proportioned to achieve a maximum slump of four inches. Instead of increasing the water content, a plasticizing admixture may be utilized to increase the workability of the concrete. The concrete should be properly cured after placement. Concrete should not be placed during hot and windy weather.

Crack control joints should be provided in the transverse direction spaced at horizontal intervals ranging from 24 to 36 times the thickness of the concrete.

5.5.3 Base Surfaced Fire Truck Access Road

A proposed base surfaced fire truck access road should be at least twelve (12) inches in thickness. The subgrade beneath the base surface access road should consist of at least 12 inches of compacted engineered fill.

Removals within native materials should expose competent alluvium, with an in-place density of at least 85 percent of the maximum dry density (ASTM D 1557), and not visibly porous. The subgrade soils beneath the base surface should be moisture conditioned to at least optimum moisture content and compacted to at least 95 percent of the maximum dry density (ASTM D 1557). The base should consist of Class II Aggregate Base, or equivalent, and be compacted in thin lifts (less than about 6 to 8 inches in loose thickness) to at least 95 percent of the maximum dry density of the aggregate base (ASTM D 1557).

The aggregate base surface should be sloped to drain to the edges. Periodic maintenance to the exposed AB will be needed. Maintenance may require occasional moisture treatment and surface compaction of the AB material that has been subject to weathering.

5.5.4 Pavement Construction

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete should be done in accordance with the County of San Bernardino specifications, and under the observation and testing of GeoTek and a County Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 95 percent of the maximum dry density as determined by ASTM D1557 test procedures. If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.
All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 test procedures. All materials and methods of construction should conform to the requirements of the County of Riverside.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. The pavement base should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum dry density (modified proctor).

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with County of San Bernardino specifications, and under the observation and testing of GeoTek and a County Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

5.6 SITE CONSTRUCTION

5.6.1 Soil Corrosivity

The soil resistivity at this site was tested in the laboratory on one sample collected during the field investigation. The results of the testing indicate that the on-site soils are considered "*highly corrosive*" (2,278 ohm-cm) (Roberge, 2000) to buried ferrous metal in accordance with current standards used by corrosion engineers. It is recommended that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site.

5.6.2 Soil Sulfate Content

The sulfate content was determined in the laboratory on one sample collected during the field investigation. The results indicate that the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327), classifying the soils as having a "negligible" (S0) sulfate exposure category as per ACI 318-19. Based on the test results and Table 19.3.1.1 of ACI 318-19. Based on the lab results and ACI 318-19, no other special recommendations for concrete are required for this project due to soil sulfate exposure.

Additional soil sampling, laboratory testing and analysis regarding soil corrosion and soil sulfate content should be conducted following completion of the project rough grading operation.

5.6.3 Import Soils

Import soils should have expansion characteristics similar to the on-site soils. GeoTek also recommends that the proposed import soils be tested for expansion and sulfate potential. GeoTek should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

5.6.4 Concrete Flatwork

5.6.4.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum actual thickness. Concrete used for this project should have a minimum compressive strength of 2,500 psi. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in industrial construction.

Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior flatwork having a "Very Low" Expansion Index should be pre-saturated to a minimum of 100 percent of optimum moisture content to a depth of at least 12" below subgrade.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the County of San Bernardino specifications, and under the observation and testing of GeoTek and a County inspector, if necessary.

5.6.4.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are hairline to more than 1/8 inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent upon a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek suggests that control joints be placed in two orthogonal directions and located a distance approximately equal to 24 to 36 times the slab thickness.

5.7 POST CONSTRUCTION CONSIDERATIONS

5.7.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decrease the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundations. This type of landscaping should be avoided. Due to the presence of high expansive soils, irrigation should be minimized adjacent to the buildings. Planters within 30 feet of the buildings should be above ground and underlain by a concrete slab. Waterproofing of the foundation and/or subdrains may be warranted and advisable. GeoTek could discuss these issues, if desired, when plans are made available.

5.7.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times, as directed by the project civil engineer. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings and floor-slabs. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

Roof gutters should be installed that will direct the collected water at least 20 feet from the buildings.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

5.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

It is recommended that site grading, specifications, retaining wall/shoring plans and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations contained in this report. Additional recommendations may be necessary based on these reviews. It is also recommended that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek's representative perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement and collect soil samples for laboratory testing when necessary.
- Observe the fill for uniformity during placement including utility trenches.
- Test the fill for field density and relative compaction.
- Test the near-surface soils to verify proper moisture content.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek, which can comply with the requirements of the governmental agencies having jurisdiction over the project. It is recommended that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

6. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of GeoTek's evaluation is limited to the area explored that is shown on the Boring Location Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to GeoTek by the client. Further, no evaluation of any existing site improvements is included. The scope is based on GeoTek's understanding of the project and the client's needs, GeoTek's proposal (Proposal No. P-0400124-CR) dated April 1, 2024, and geotechnical engineering standards normally used on similar projects in this region.

Since the recommendations contained in this report are based on the site conditions observed and encountered, and laboratory testing, GeoTek's conclusions and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

7. LIMITATIONS

GeoTek's findings are based on site conditions observed and the stated sources. Thus, GeoTek's comments are professional opinions that are limited to the extent of the available data.

GeoTek has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering at this time and location and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report.

Since GeoTek's recommendations are based on the site conditions observed and encountered at the stated times and laboratory testing. Thus, GeoTek's conclusions and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty of any kind is expressed or implied. Standards of care/practice are subject to change with time.

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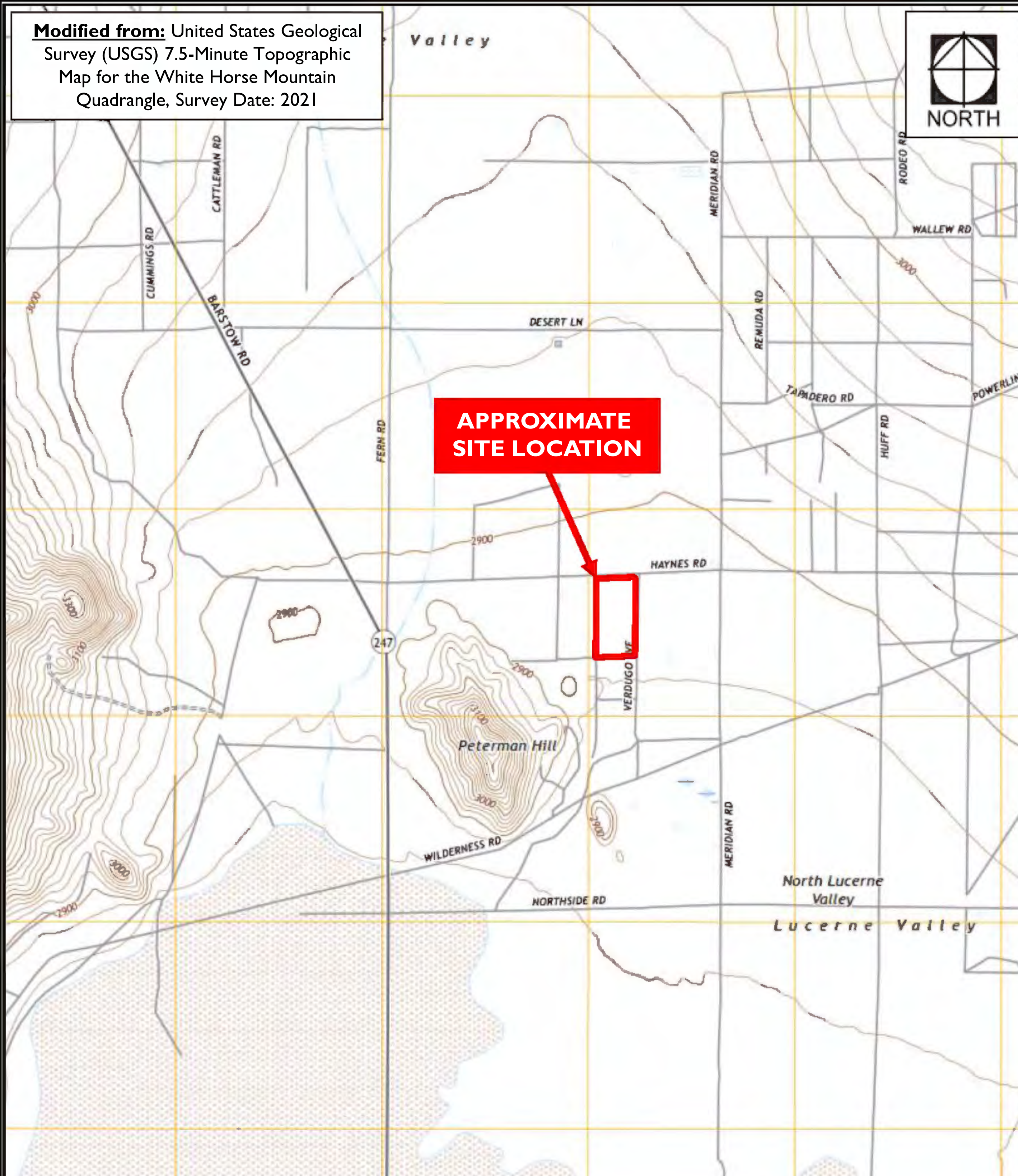


Albert A. Webb Associates
Proposed Facility Improvements – The Rivers Edge Ranch
Assessor's Parcel Number (APN) 0453-062-14-0000
Lucerne Valley, San Bernardino County, California

Project No. 3977-CR

Figure 1

Site Location and Topography Map

GEOTEK

Albert A. Webb Associates
Proposed Facility Improvements – The Rivers Edge Ranch
Assessor's Parcel Number (APN) 0453-062-14-0000
33433 Haynes Road
Lucerne Valley, San Bernardino County, California

Project No. 3977-CR

Figure 2

Boring Location Map

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Geotechnical Evaluation
Proposed Facility Improvements – The Rivers Edge Ranch
Assessor's Parcel Number (APN) 0453-062-14-0000
33433 Haynes Road
Lucerne Valley, San Bernardino County, California
Project No. 3977-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)
The ring sampler is driven into the ground at various depths in accordance with ASTM D 3550 test procedures. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. Disturbed samples are removed from the sample barrel, sealed in a plastic bag, and transported to the laboratory for testing.

Bulk Samples (Large)
These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)
These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – BORING LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip
C: Contact line	
………..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
———	Thick solid line denotes end of the boring

(Additional denotations and symbols are provided on the logs of borings)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'42"N 116°55'58"W	**ELEVATION:**	2,890 ft	**DATE:**	7/5/2024

Boring No.: B-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					**Quaternary Alluvium:**			
		30 27 31	R1	SM	Silty f-c SAND, yellowish brown, slightly moist, dense, trace f gravel	4.3	117.8	EI, MD, SH Expansion Index = 1
		25 36 50/5"	R2		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f gravel	3.4	121.3	
5		50/6"	R3		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f gravel	8.0	118.8	
		42 50/6"	R4		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f gravel	4.5	120.4	
10		50/6"	R5		Silty f-c SAND with fine gravel, yellowish brown, slightly moist, very dense			
		22 50/6"	R6		Silty f-c SAND with fine gravel, yellowish brown, slightly moist, very dense	2.3	114.4	
15								
		22 50/6"	R7	SM/ML	Silty f-c SAND to f-c sandy SILT, brown, moist, very dense/ hard			
20								
25		18 20		SM	Silty f-c SAND, yellowish brown, moist, medium dense, trace f gravel			
		26	S1	ML/SM	F-c sandy SILT, light brown, moist, trace f-c gravel	10.2		
30		20 25 50/4"	S2	SM	Silty f-c SAND, yellowish brown, moist, very dense, trace f-c gravel, trace clay			

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'42"N 116°55'58"W	**ELEVATION:**	2,890 ft	**DATE:**	7/5/2024

Boring No.: B-1 (continued)

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
35	SPT	22 19 19	S3	SM	Silty f-c SAND with gravel, yellowish brown, moist, dense	10.1		
40	SPT	16 50/6"	S4	ML	F-c sandy SILT, yellowish brown, moist, hard, trace f-c gravel, trace clay			
45	SPT	7 19 50/6"	S5	SM	Silty f-c SAND, yellowish brown, moist, very dense, trace f-c gravel	10.5		
50	SPT	25 50/4"	S6	SM/ML	Silty f-c SAND to f-c sandy SILT, yellowish brown, slightly moist, very dense/ hard, trace f-c gravel			
					TERMINATED AT 51 FEET No groundwater encountered Boring backfilled with cuttings			

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'42"N 116°55'58"W	**ELEVATION:**	2,891 ft	**DATE:**	7/5/2024

Boring No.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
0					**Quaternary Alluvium:**			
				SM/ML	Silty f-c SAND to f-c sandy SILT, yellowish brown, slightly moist, medium dense			
	Ring	19 18 25	R1			7.1	120.2	
5	Ring	33 32 38	R2	SM	Silty f-c SAND with gravel, yellowish brown, slightly moist, dense	5.9	117.8	
	Ring	31 50/4"	R3		Silty f-c SAND, yellowish brown, slightly moist, trace f gravel, very dense	3.1	118.7	
10	Ring	50/6"	R4		Silty f-c SAND, yellowish brown, slightly moist, trace f-c gravel, very dense			
15	Ring	50/5"	R5	SM/ML	Silty f-c SAND to f-c sandy SILT, yellowish brown, slightly moist, very dense/ hard, trace f-c gravel	5.0	114.3	
20	Ring	24 50/3"	R6	ML/SM	F sandy SILT to silty f SAND, yellowish brown, slightly moist, very dense/ hard, trace m-c SAND	2.2	118.2	
					TERMINATED AT 21 FEET No groundwater encountered Boring backfilled with cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test

SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'41"N 116°55'58"W	**ELEVATION:**	2,889 ft	**DATE:**	7/5/2024

Boring No.: B-3

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0					**Quaternary Alluvium:**			
	Ring	41 50/5"	R1	SM	Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f gravel	2.6	112.9	
5	Ring	32 50/5"	R2		Silty f-c SAND, yellowish brown, moist, very dense, trace f-c gravel	3.2	118.2	
	Ring	19 35 50/4"	R3	SM/ML	Silty f-c SAND, yellowish brown, slightly moist, dense, trace f-c gravel Silty f-c SAND to f-c sandy SILT, greyish brown, slightly moist, very dense/ hard, trace f-c gravel	2.9	117.6	
10	Ring	17 50/6"	R4		Silty f-c SAND to f-c sandy SILT, greyish brown, slightly moist, very dense/ hard, trace f-c gravel	9.4	111.6	
15	Ring	17 25 47	R5	ML	F sandy SILT, yellowish brown, slightly moist, hard, trace m-c sand	8.0	113	
20	Ring	19 24 32	R6	SM/ML	Silty f-c SAND to f-c sandy SILT with gravel, yellowish brown, moist, very dense/ hard			
					TERMINATED AT 20 FEET No groundwater encountered Boring backfilled with cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'41"N 116°55'57"W	**ELEVATION:**	2,890 ft	**DATE:**	7/5/2024

Boring No.: B-4

Depth (ft)	Samples: Sample Type	Samples: Blows/ 6 in	Samples: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					**Quaternary Alluvium:**			
				SM	Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f-c gravel			SR
		27 50/6"	R1			3.0	118.2	
5		50/5"	R2		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f-c gravel	4.5	110.7	
		23 50/6"	R3		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f-c gravel	3.3	119.4	
10		50/6"	R4		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f gravel	4.1	116.3	
15		15 50/6"	R5	ML	F-c sandy SILT, yellowish brown, slightly moist, hard			
20		13 24 37	R6		F-c sandy SILT, yellowish brown, moist, hard	9.8	116.4	
					TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'44"N 116°55'57"W	**ELEVATION:**	2,891 ft	**DATE:**	7/5/2024

Boring No.: B-5

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0					**Quaternary Alluvium:**			
	Ring	13 34 50/5"	R1	SM	Silty f-c SAND with gravel, yellowish brown, slightly moist, very dense	2.8	110.3	
5	Ring	24 50/5"	R2		Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f-c gravel	2.4	113.4	
	Ring	14 20 31	R3	ML/SM	F sandy SILT to silty f SAND, yellowish brown, slightly moist, dense/ hard, trace m-c sand	4.4	110.7	Collapse
10	Ring	27 50/3"	R4		F sandy SILT to silty f SAND, yellowish brown, slightly moist, very dense/ hard, trace m-c sand	4.1	119	
	Ring	50/5"	R5	SM	Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f-c gravel			
15								
	Ring	31 50/4"	R6	ML/SM	F sandy SILT to silty f SAND, yellowish brown, slightly moist, hard/ very dense, trace f-c gravel	4.1	113.0	
20					**TERMINATED AT 19.5 FEET** No groundwater encountered Boring backfilled with cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Albert A Webb Associates	**DRILLER:**	2R Drilling	**LOGGED BY:**	Jack Small
PROJECT NAME:	33433 Haynes Road	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Adrian
PROJECT NO.:	3977-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck
COORDINATES:	34°32'44"N 116°55'56"W	**ELEVATION:**	2,891 ft	**DATE:**	7/5/2024

Boring No.: B-6

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0	Large Bulk				**Quaternary Alluvium:**			
	Ring	22	R1	SM/ML	Silty f-c SAND to f-c sandy SILT, yellowish brown, moist, very dense/ hard, trace f gravel	11.7	110.3	R-Value = 16
		50/6"		SM	Silty f-c SAND, yellowish brown, slightly moist, very dense, trace f gravel, trace clay			
5	Ring	8 14 13	R2	SM/ML	Silty f-c SAND to f-c sandy SILT, yellowish brown, slightly moist, medium dense/ very stiff, trace f gravel	11.8	110.3	Collapse
	Ring	5 8 33	R3	SM	Silty f-c SAND, yellowish brown, slightly moist, medium dense, trace f gravel	1.6	113.9	Collapse
10	Ring	50/4"	R4	ML/SM	F-c sandy SILT to silty f-c SAND, yellowish brown, slightly moist, very dense/ hard, trace f-c gravel	4.6	114.3	
15	Ring	14 50/6"	R5		F-c sandy SILT to silty f-c SAND, yellowish brown, slightly moist, very dense/ hard, trace f-c gravel			
20	Ring	16 24 50/4"	R6	ML	F-c sandy SILT, yellowish brown, slightly moist, hard, trace f gravel	5.8	119.9	

TERMINATED AT 21.5 FEET

No groundwater encountered
Boring backfilled with cuttings

25

30

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test
SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

APPENDIX B

LABORATORY TEST RESULTS

Geotechnical Evaluation
Proposed Facility Improvements – The Rivers Edge Ranch
Assessor's Parcel Number (APN) 0453-062-14-0000
33433 Haynes Road
Lucerne Valley, San Bernardino County, California
Project No. 3977-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the logs of borings in Appendix A.

Collapse
Collapse testing was performed on select soil samples in general accordance with ASTM D 4546 test procedures. The results of the collapse tests are presented graphically in Appendix B.

Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM D 3080 test procedures. The rate of deformation was approximately 0.035 inch per minute. The sample was sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The test was performed on one soil sample remolded to approximately 90 percent of maximum dry density as determined by ASTM D 1557 test procedures in addition to in-situ samples obtained from the ring sampler. The shear test results are presented graphically in Appendix B.

Expansion Index
Expansion Index testing was performed on one sample collected during the subsurface exploration. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below and in Appendix B.

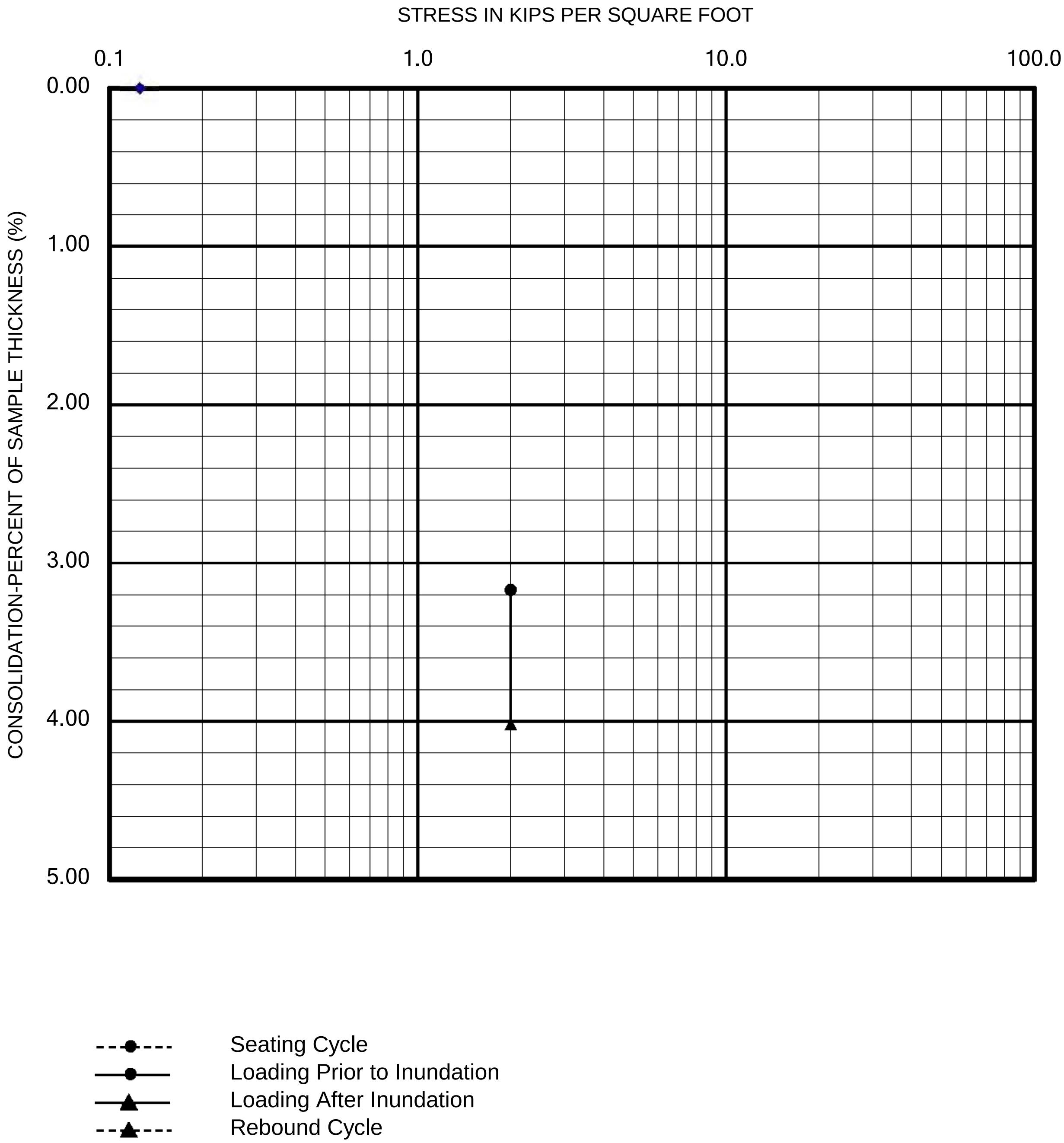
Boring No.	Depth (ft.)	Description	Expansion Index	Classification
B-1	0-5	Silty Sand (SM)	1	Very Low

In-Situ Moisture and Density
The natural water content of sampled soils was determined in general accordance with ASTM D 2216 test procedures on samples of the materials recovered from the subsurface exploration. In addition, in-place dry density of the sampled soils was determined in general accordance with ASTM D 2937 test procedures on relatively undisturbed samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths in Appendix A.

Moisture-Density Relationship
Laboratory testing was performed on one sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with ASTM Test D 1557 test procedures. The results of the testing are presented graphically in Appendix B.

Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content was performed by others for GeoTek in general accordance with ASTM D4327 test procedures. Resistivity testing was completed by others for GeoTek in general accordance with ASTM G187 test procedures. Testing to determine the chloride content was performed by others in general accordance with ASTM D4327 test procedures. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	pH ASTM D4972	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-4	0-5	6.9	60.7	0.0444	2,278



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

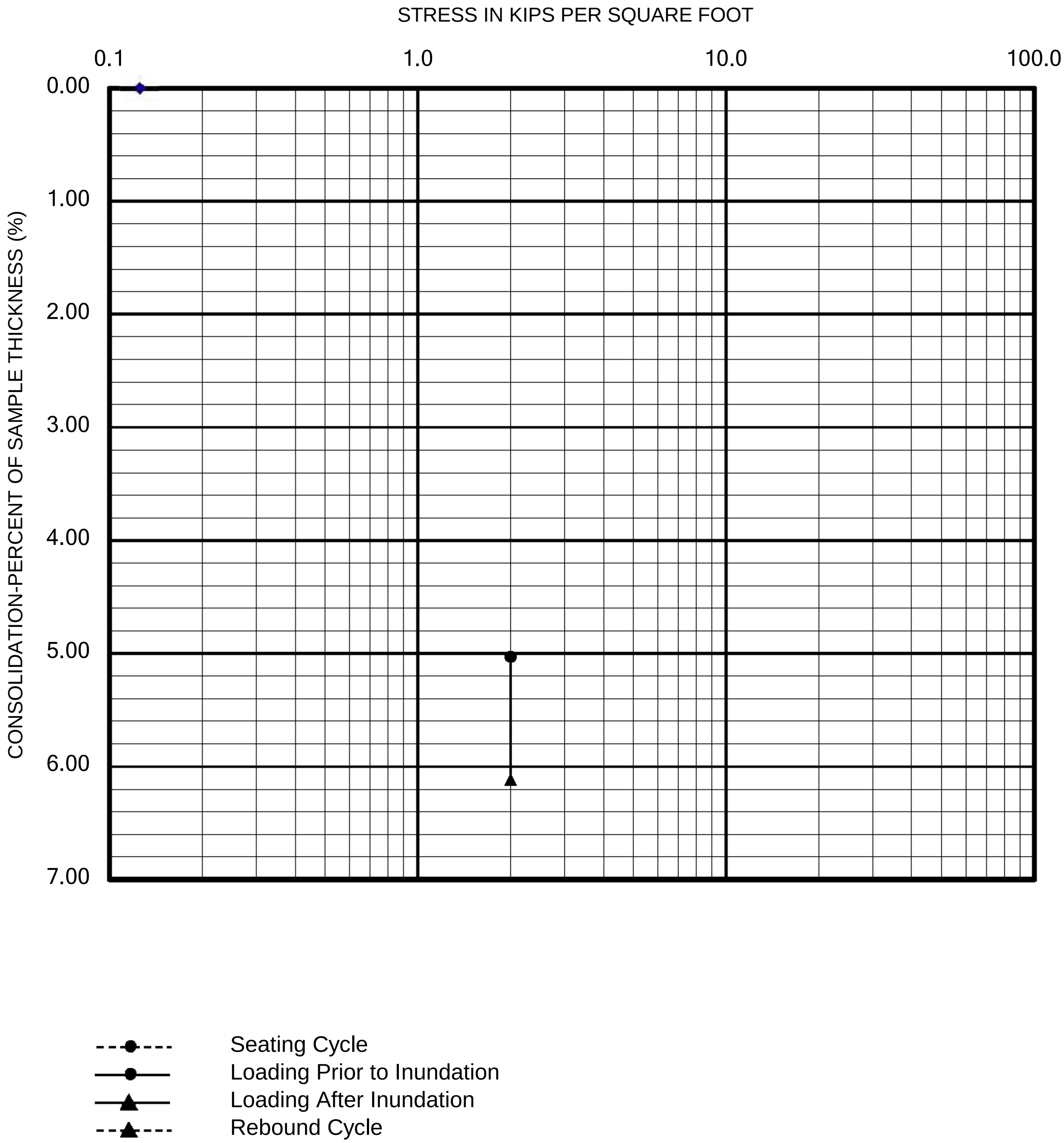
COLLAPSE REPORT

Sample: B-5 @ 6 Feet

33433 Haynes Road Lucerne Valley

Plate B-1

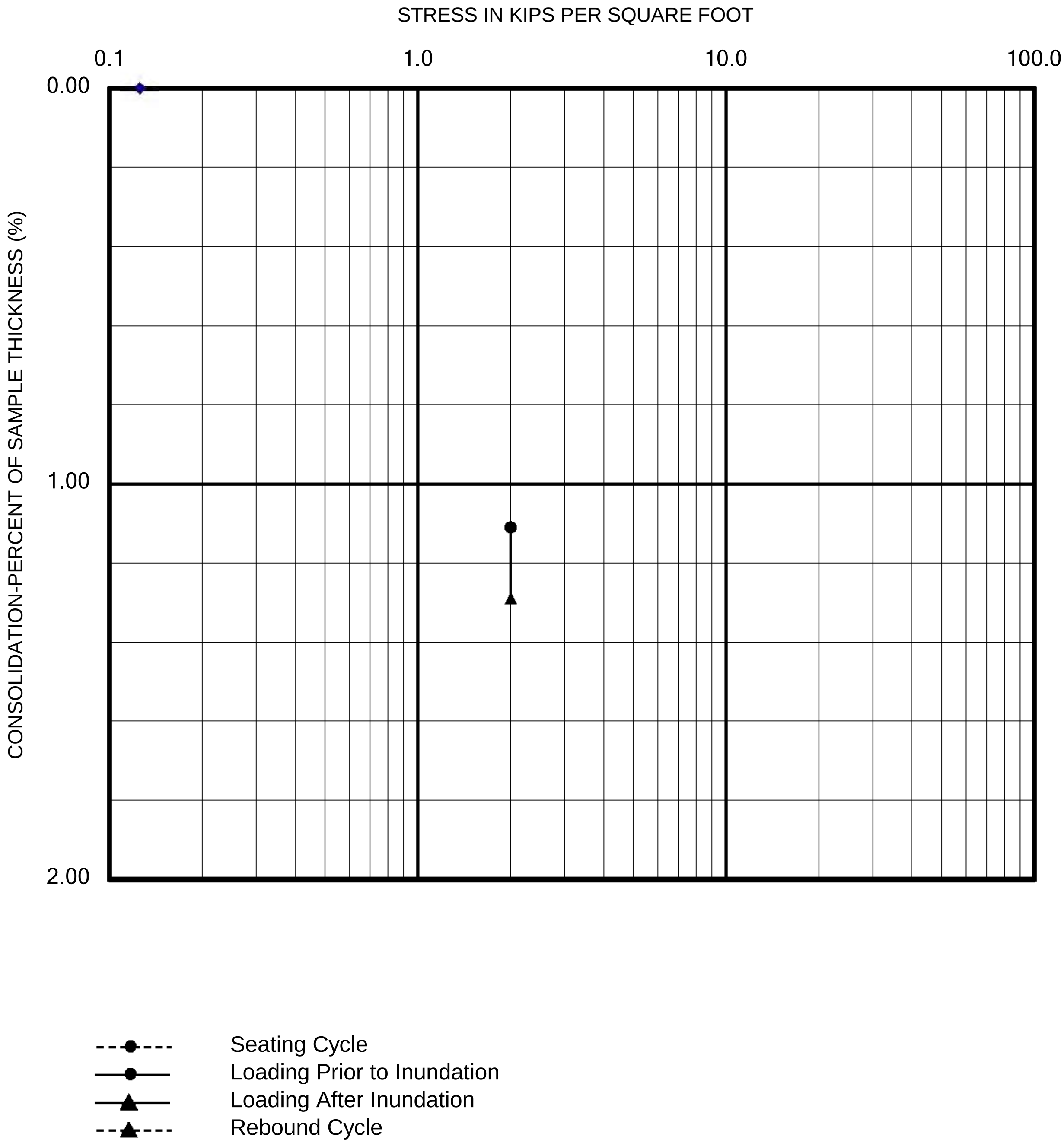
CHECKED BY: EC	Lab: Corona
PROJECT NO.: 3977-CR	Date: 7/18/2024



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

CHECKED BY: EC	Lab: Corona	**COLLAPSE REPORT** **Sample: B-6 @ 5 Feet** **33433 Haynes Road Lucerne Valley**	**Plate B-2**
PROJECT NO.: 3977-CR	Date: 7/18/2024		



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample: B-6 @ 7 Feet

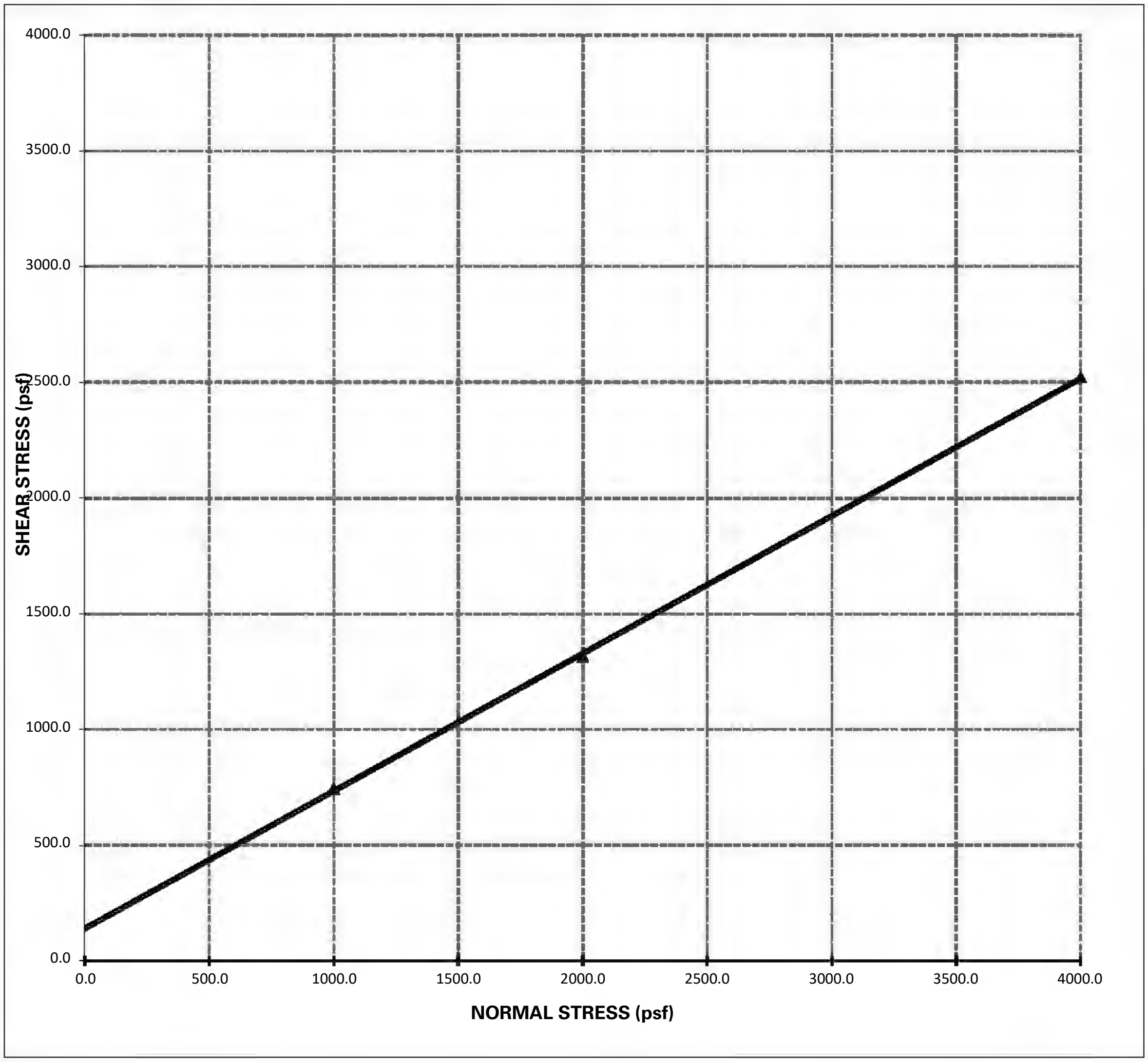
33433 Haynes Road Lucerne Valley

Plate B-3

CHECKED BY: EC	Lab: Corona
PROJECT NO.: 3977-CR	Date: 7/18/2024

DIRECT SHEAR TEST

Project Name: 33433 Haynes Road Lucerne Valley
Project Number: 3977-CR
Sample Location: B-1 @ 0-5 Feet
Date Tested: 7/18/2024



Shear Strength: Φ = 31 ° C = 140 psf

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Albert A.Webb Associates	***Tested/ Checked By:***	AH Lab No Corona
Project Number:	3977-CR	***Date Tested:***	7/10/2024
Project Location:	33433 Haynes Road Lucerne Valley	***Sample Source:***	B-1 @ 0-5 Feet
		Sample Description:	

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	786.7
Weight of ring (gm)	362.7
Net weight of sample (gm)	**424.0**
Wet Density, lb / ft3 (C*0.3016)	**127.9**
Dry Density, lb / ft3 (D/1.F)	**118.6**

SATURATION DETERMINATION

Moisture Content, %	7.8
Specific Gravity, assumed	**2.70**
Unit Wt. of Water @ 20°C, (pcf)	**62.4**
% Saturation	**50.1**

READINGS

DATE	TIME	READING	
7/10/2024		0.9840	Initial
7/10/2024		0.9840	10 min/Dry
7/11/2024		0.9850	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
808.3	**12.9**

EXPANSION INDEX = 1

GeoTek - Corona
1548 N. Maple St.
Corona, CA 92880
Phone: (951) 710-1160
Fax: (951) 710-1167

Report No: PTR:24-00139-S01

Proctor Report

Client: Albert A.Webb Associates
Attn: Jason Ardery
Riverside CA 92506

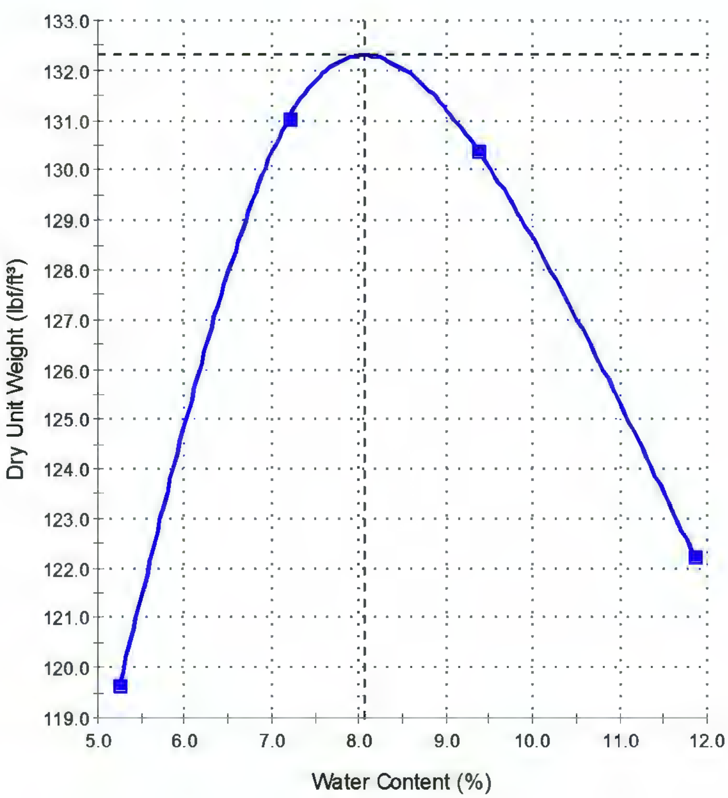
CC:

Project: 3977-CR
33433 Haynes Road Lucerne Valley

Sample Details

Sample ID: 24-00139-S01 **Date Sampled:** 7/5/2024
Sampled By:
Material: Silty Fine to Coarse SAND
Location: B-1 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lbf/ft³):	**132.3**
Optimum Water Content (%):	**8.1**
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	3
Passing Sieve No 4 (4.75mm) (%):	97
Tested By:	Eduardo Cuevas
Date Tested:	7/10/2024

Comments

Results Only Soil Testing for 33433 Haynes Road

July 17, 2024

Prepared for:

Eddy Cuevas
GeoTek USA
1548 N. Maple Avenue
Corona, CA 92878
Ecuevas@geotekusa.com, jbrucelas@geotekusa.com

Project X Job#: S240715P
Client Job or PO#: 3977-CR

Prepared by:
M. Williams

Respectfully Submitted,

Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: GeoTek USA
Job Name: 33433 Haynes Road
Client Job Number: 3977-CR
Project X Job Number: S240715P
July 17, 2024

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates SO_4^{2-}		Chlorides Cl^-		Resistivity As Rec'd	Minimum	pH	Redox	Sulfide S^{2-}	Nitrate NO_3^-	Ammonium NH_4^+	Lithium Li^+	Sodium Na^+	Potassium $K^{\pm}$	Magnesium Mg^{2+}	Calcium Ca^{2+}	Fluoride F_2^{2-}	Phosphate PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ω-cm)	(Ω-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B4	0-5	443.9	0.0444	60.7	0.0061	59,630	2,278	6.9	168	0.1	28.3	3.4	ND	74.3	74.3	12.4	79.6	7.4	4.2

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops. This is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

Try out our new online forms: SOIL CORROSIVITY & THERMAL RESISTIVITY LAB REQUEST FORM & IN-SITU WENNER 4 PIN QUOTE REQUEST FORM

• ANALYSIS
• DESIGN

LaBelle • Marvin

• SOILS, ASPHALT
TECHNOLOGY

PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

July 26, 2024

Ms. Anna Scott
GeoTek Inc.
1548 North Maple Street
Corona, California 92880

Project No. 50199

Attention Ms. Scott:
Laboratory testing of the bulk soil sample delivered to our laboratory on 7/24/2024 has been completed.

P.N.:	W.O.# 3977-CR
Project:	33433 Hayner Road, Albert A. Webb Associates
Sample:	B-6 @ 0'-5'

Data sheets are transmitted herewith for your use and information. Any untested portion of the samples will be retained for a period of sixty (60) days prior to disposal. The opportunity to be of service is appreciated, and should you have any questions, kindly call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:tw
Enclosures

R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 50199

DATE: 7/26/2024

BORING NO. B-6 @ 0'-5'

33433 Hayner Road, Albert A Webb Associates

W.O.# 3977-CR

SAMPLE DESCRIPTION: Brown Sandy Silt

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	10	11	12
Water added, grams	65	47	33
Initial Test Water, %	13.1	11.4	10.0
Compact Gage Pressure,psi	40	85	170
Exudation Pressure, psi	231	409	518
Height Sample, Inches	2.58	2.51	2.47
Gross Weight Mold, grams	3112	3099	3075
Tare Weight Mold, grams	1953	1948	1945
Sample Wet Weight, grams	1159	1151	1130
Expansion, Inches x 10exp-4	0	10	20
Stability 2,000 lbs (160psi)	60 / 135	35 / 83	26 / 58
Turns Displacement	3.80	3.48	3.18
R-Value Uncorrected	11	40	58
R-Value Corrected	11	40	58
Dry Density, pcf	120.4	124.8	126.0

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.91	0.61	0.43
G. E. by Expansion	0.00	0.33	0.67

Equilibrium R-Value	**22** by EXUDATION	Examined & Checked: 7 /26/ 24
REMARKS:	Gf = 1.25 0.0% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 50199

DATE: 7 /26/ 24

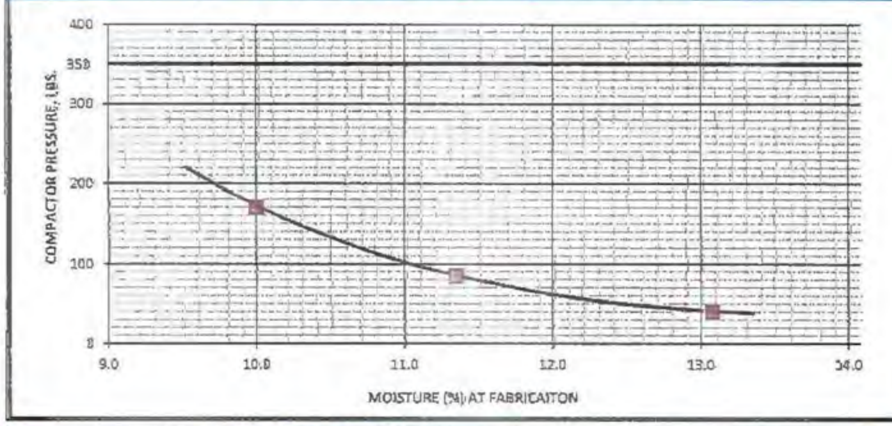
REMARKS:

BORING NO. B-6 @ 0'-5'

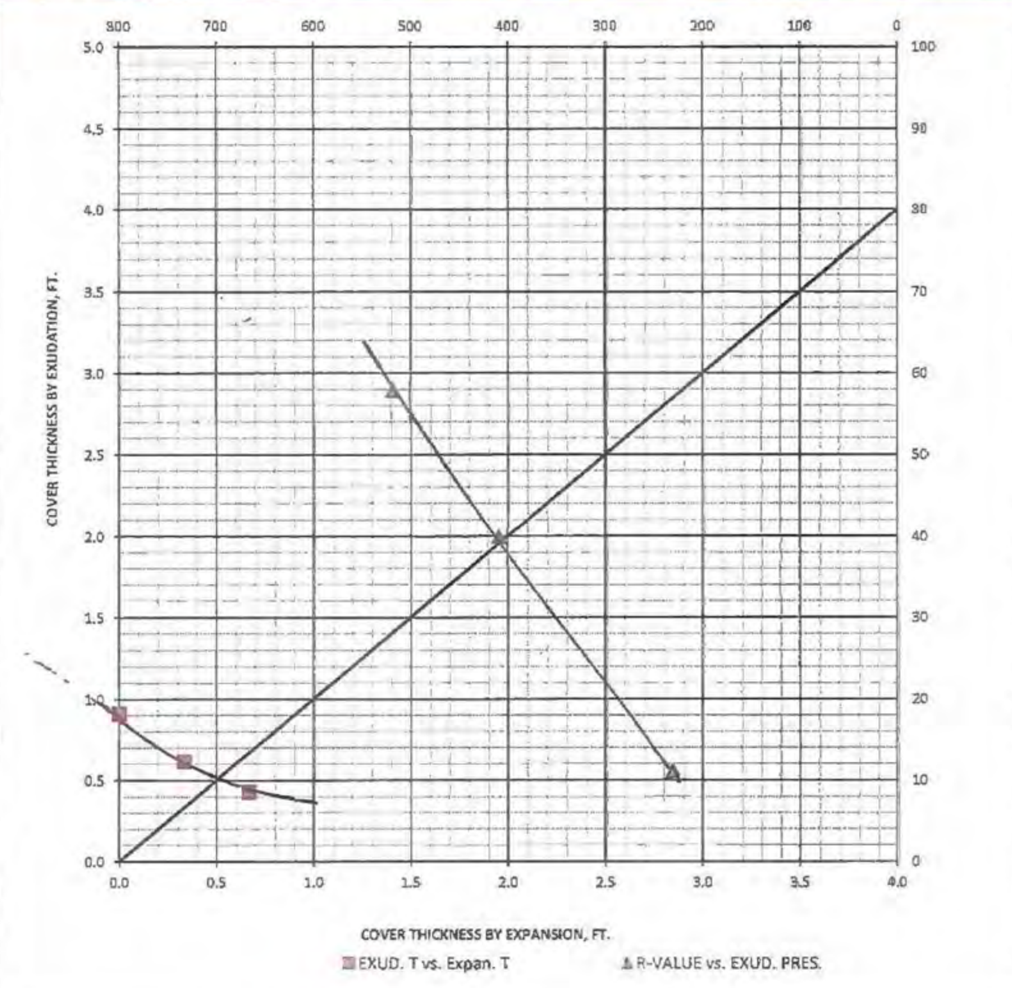
33433 Hayner Road, Albert A Webb Associates

W.O.# 3977-CR

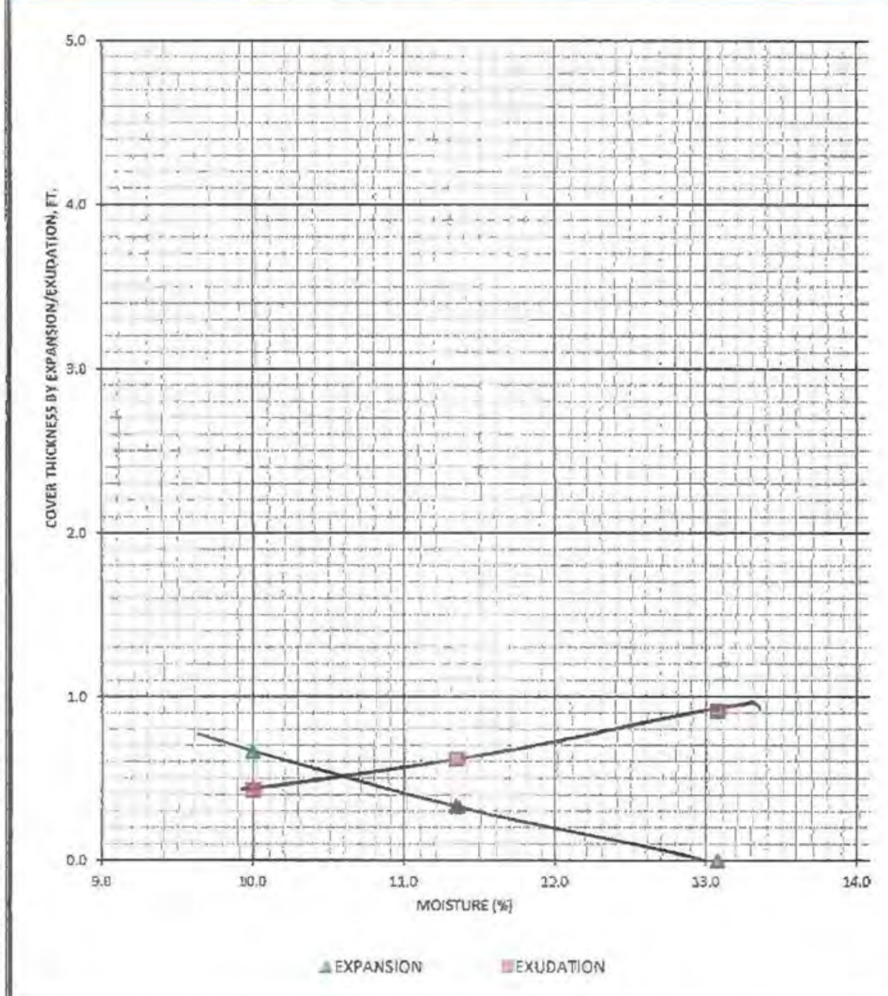
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



APPENDIX C

GENERAL GRADING GUIDELINES

Geotechnical Evaluation
Proposed Facility Improvements – The Rivers Edge Ranch
Assessor's Parcel Number (APN) 0453-062-14-0000
33433 Haynes Road
Lucerne Valley, San Bernardino County, California
Project No. 3977-CR

GEOTEK

GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code, CBC (2022) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.

4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.
6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium, and/or weathered bedrock be removed unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
    a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
    b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
    a) They are not placed in concentrated pockets;
    b) There is a sufficient percentage of fine-grained material to surround the rocks;
    c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated

suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.

6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

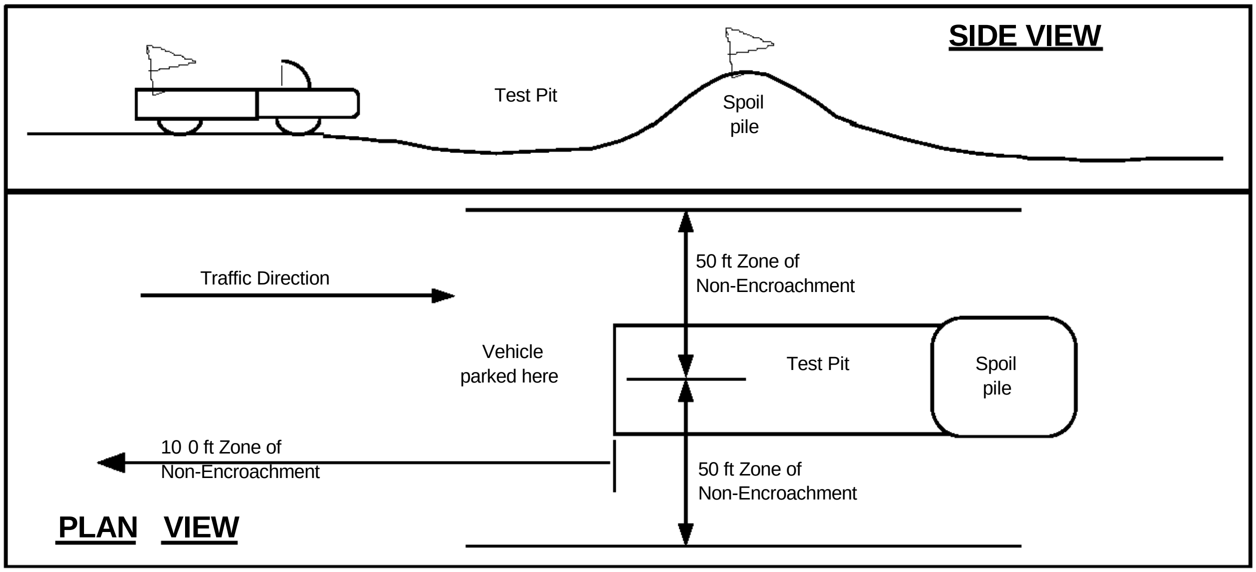
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or

4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractors representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

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