



SECTION H

EXHIBITS

COUNTY SERVICE AREA 20 JOSHUA TREE PHASE II DESERT VIEW CONSERVATION AREA RECREATIONAL TRAILS PROJECT

FOR

**COUNTY SERVICE AREA (CSA) 20
JOSHUA TREE, CALIFORNIA**

Exhibit A

SUBSTITUTION REQUEST FORM

DATE: _____

TO: San Bernardino County Public Works - Special District

ATTENTION: John Hernandez , Project Manager

PROJECT: CSA 20 Joshua Tree Phase II Desert View Conservation Area Recreational Trails Project

We hereby submit for your consideration the following product as a substitute for the specified product for the above project:

Section No.	Paragraph	Specified Product
_____	_____	_____
Proposed Substitution:		_____
_____		_____

Product Data:

Attach complete technical data for proposed substitution.

Include complete information on changes to Contract Documents which proposed substitution will require for its proper installation.

Samples:

Attached Will be furnished upon request

Does the substitution affect dimensions shown on drawings?

No Yes (explain on attachment)

Adjustment to the Contract Sum or Schedule

Sum _____

Schedule _____

Estimated Cost of engineering, design, or agency fees:

Sum _____

Effects of proposed substitution on other trades:

Differences between proposed substitution and specified product:

Other projects where the proposed substitution was successful.

Manufacturer's warranties of the proposed and specified products are:

Same Different (explain on attachment)

Maintenance services and spare parts are available for proposed product from:

Previous installations where proposed product may be seen:

Project: _____ Project: _____
Owner: _____ Owner: _____
Architect: _____ Architect: _____
Date Installed: _____ Date Installed: _____

Cost savings to be realized by Owner, if proposed substitution is accepted:

Change to Contract Time, if proposed substitution is accepted:

No Change Add ____ days Deduct ____ days

Substitution Request Form-2

By submitting this substitution request, Contractor represents that he has read and agrees to the provisions of Section 016000.

Submitted by Contractor:

Signature

Firm

For Use By Owner:

Based on the information supplied by the Contractor, the Owner has reviewed the proposed substitution on the basis of design concept of the Work and conformance with information given in Contract Documents.

Accepted Accepted as Noted Rejected

Submit Additional Information: _____

By: _____ Date: _____

Exhibit B

**GEOTECHNICAL EVALUATION
SAN BERNARDINO COUNTY PHASE II DESERT VIEW CONSERVATION AREA (DVCA)
RECREATIONAL TRAILS PROJECT
62539 ONAGA TRAIL
JOSHUA TREE AREA OF SAN BERNARDINO COUNTY, CALIFORNIA**

PREPARED FOR

**ARCHITERRA DESIGN GROUP
10221-A TRADEMARK
RANCHO CUCAMONGA, CALIFORNIA 91730**

PREPARED BY

**GEOTEK, INC.
1548 NORTH MAPLE STREET
CORONA, CALIFORNIA 92878**



GeoTek, Inc.
1548 North Maple Street, Corona, California 92878
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

August 29, 2024
Project No. 3910-CR

Architerra Design Group

10221-A Trademark
Rancho Cucamonga, California 91730

Attention: Mr. Andrew Krumwiede

Subject: **Geotechnical Evaluation**
San Bernardino County Phase II Desert View Conservation Area (DVCA)
Recreational Trails Project
62539 Onaga Trail
Joshua Tree area of San Bernardino County, California

Dear Mr. Krumwiede:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical Evaluation for the San Bernardino County Phase II Desert View Conservation Area (DVCA) Recreational Trails project to be located at 62539 Onaga Trail, in the Joshua Tree area of San Bernardino County, California. This report presents the results of GeoTek's evaluation, discussion of findings, and provides geotechnical recommendations for foundation design and construction.

Based upon review and evaluation, site development appears feasible from a geotechnical viewpoint provided that the recommendations included in this report are incorporated into the design and construction phases of the project.

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

Respectfully submitted,
GeoTek, Inc.



Bruce A. Hick
GE 2284, Exp. 12/31/24
Geotechnical Engineer

Edward H. LaMont
CEG 1892, Exp. 07/31/26
Principal Geologist

Anna M. Scott
Project Geologist

Distribution: (1) Addressee via email (one PDF file)

\\geotekfs\Riverside\Projects\3901 to 3950\3910CR Architerra Design Group Phase II DVCA Joshua Tree\Geotechnical Evaluation\3910CR Geotechnical Evaluation DVCA Phase 2.doc

TABLE OF CONTENTS

1. PURPOSE AND SCOPE OF SERVICES	1
2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT	1
2.1 SITE DESCRIPTION	1
2.2 PROJECT DESCRIPTION	2
3. FIELD EXPLORATION AND LABORATORY TESTING	2
3.1 FIELD EXPLORATION.....	2
3.2 LABORATORY TESTING.....	3
4. GEOLOGIC AND SOILS CONDITIONS	3
4.1 REGIONAL SETTING.....	3
4.2 GENERAL SOIL CONDITIONS	4
4.2.1 Alluvium.....	4
4.2.2 Granitic Bedrock.....	4
4.3 SURFACE WATER AND GROUNDWATER	4
4.3.1 Surface Water	4
4.3.2 Groundwater.....	4
4.4 FAULTING AND SEISMICITY	5
4.4.1 Faulting.....	5
4.4.2 Seismic Design Parameters.....	5
4.5 LIQUEFACTION.....	6
4.6 OTHER SEISMIC HAZARDS.....	7
5. CONCLUSIONS AND RECOMMENDATIONS.....	7
5.1 GENERAL.....	7
5.2 EARTHWORK CONSIDERATIONS.....	7
5.2.1 General.....	7
5.2.2 Site Clearing.....	7
5.2.3 Site Preparation	8
5.2.4 Engineered Fill.....	8
5.2.5 Excavation Characteristics	8
5.2.6 Trench Excavations and Backfill	9
5.2.7 Shrinkage and Bulking.....	10
5.2.8 Grading Plan Review	10
5.3 DESIGN RECOMMENDATIONS.....	10
5.3.1 Shallow Foundation Design Criteria.....	10
5.3.2 Moisture and Vapor Retarding System.....	12
5.3.3 Miscellaneous Shallow Foundation Recommendations.....	13
5.3.4 Foundation Setbacks.....	13
5.3.5 Deep Foundation Design Criteria – Drilled Piers.....	14
5.3.6 Soil Corrosivity.....	15
5.3.7 Soil Sulfate Content.....	15
5.3.6 Import Soils.....	16
5.4 RETAINING WALL DESIGN AND CONSTRUCTION	16
5.4.1 General Design Criteria.....	16

TABLE OF CONTENTS

5.4.2	Cantilevered Walls.....	17
5.4.3	Retaining Wall Backfill and Drainage.....	17
5.4.4	Restrained Retaining Walls.....	18
5.4.5	Additional Retaining Wall Design Considerations	18
5.5	CONCRETE CONSTRUCTION.....	19
5.5.1	General.....	19
5.5.2	Concrete Mix Design	19
5.5.3	Concrete Flatwork.....	19
5.5.4	Concrete Performance.....	20
5.6	POST CONSTRUCTION CONSIDERATIONS	20
5.6.1	Landscape Maintenance and Planting.....	20
5.6.2	Drainage.....	21
5.7	PLAN REVIEW AND CONSTRUCTION OBSERVATIONS	21
6.	INTENT	21
7.	LIMITATIONS	22
8.	SELECTED REFERENCES.....	22

ENCLOSURES

Figure 1 – Site Location and Topography Map

Figures 2a, 2b, 2c and 2d – Trench Location Map

Figure 3 – Overall Conceptual Trail Plan

Appendix A – Logs of Exploratory Trenches

Appendix B – Laboratory Test Results

Appendix C – General Earthwork Grading Guidelines

I. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical engineering and geologic conditions at the project site, as outlined in GeoTek’s proposal P-0311424-CR, dated March 28, 2024. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- Perform a site reconnaissance,
- Site exploration consisting of the excavation, logging, and sampling of six (6) exploratory test trenches extending to depths of approximately 6.5 to 10 feet below existing grades,
- Laboratory testing of soil samples collected during the field investigation,
- Review and evaluation of site seismicity, and
- Preparation of this Geotechnical Evaluation report which presents GeoTek’s findings, conclusions, and recommendations for this site.

The intent of this report is to aid in the evaluation of the site for future proposed improvements from a geotechnical perspective. The professional opinions and geotechnical information contained in this report may need to be updated based upon review of the 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 Deset View Conservation Area (DVCA) is comprised of approximately 605 acres of largely undisturbed native land located south of State Highway 62 and addressed as 62539 Onaga Trail, in the Joshua Tree area of San Bernardino County, California (see Figure 1, Site Location and Topography Map).

Access to the site is from Onaga Trail, an unimproved roadway located adjacent to the western boundary of the conservation area. The conservation area currently consists of vacant, undeveloped land, with a moderate covering of native grasses and brushes.

The site consists of relatively flat to gently sloping terrain, with surface drainage generally directed to the west-northwest direction, as dictated by site topography.

The conservation area is generally bound by undeveloped land.

2.2 PROJECT DESCRIPTION

Based upon review of the referenced *Overall Conceptual Trail Plan*, prepared by Architerra Design Group, GeoTek understands that the project will include, but not necessarily be limited to, two (2) shade shelters, a rock outcrop view platform, trailside kiosks with a roof and bench, monument signs, walking trails and other associated improvements. A copy of the *Overall Conceptual Trail Plan* is included as Figure 3 of this report.

Structural plans are not yet available for review for any of the proposed structures. However, GeoTek has assumed that the structures will be supported by conventional shallow foundations and a conventional slab-on-grade floor. Relatively light foundation loading is anticipated for the proposed structures (shade shelters, rock outcrop view platform, trailside kiosks, etc.). Maximum structural loads estimated for the structures are 2 kips per lineal foot for continuous footings and 25 kips for isolated pad foundations.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

The field exploration for this report was conducted on August 8, 2024, and included the excavating of six (6) geotechnical exploratory trenches with a rubber-tired backhoe to depths between about 6.5 and 10 feet below grade at the trench locations.

The approximate locations of the GeoTek excavations are shown on the Trench Location Maps (Figures 2a, 2b, 2c and 2d). A geologist from GeoTek logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory trenches are included in Appendix A.

Bulk soil samples were collected at various intervals in the geotechnical trenches and were returned to the laboratory for testing and evaluation.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected bulk samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the materials encountered and to evaluate their physical properties for use in engineering design and analysis. Results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included on the exploratory trench logs included in Appendix A and in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The project 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.

More specific to the subject property, the site is in an area geologically mapped to be underlain by older alluvium and granitic bedrock (Dibblee, T.W., 1967).

4.2 GENERAL SOIL CONDITIONS

A brief description of the earth materials encountered is presented in the following section. Based on the site reconnaissance, the exploratory excavations and review of published geologic maps, the area investigated is locally underlain by alluvium and granitic bedrock.

4.2.1 Alluvium

Alluvial soils were encountered in all of the trenches performed. The alluvium was found to generally consist of medium dense to very dense silty sands and sands (SM and SP soil types based upon the Unified Soil Classification System).

Based on the laboratory test results, the near surface materials have a “Very Low” ($0 \leq EI \leq 20$) Expansion Index (EI) when tested in general accordance with ASTM D 4829. Based on the laboratory test results, the near surface materials have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). The laboratory test results are provided in Appendix B.

4.2.2 Granitic Bedrock

Granitic bedrock was encountered within two (2) of the exploratory trenches (Trenches T-3 and T-6) below the alluvial materials at depths of 5.5 and 5 feet, respectively, below the existing ground surface. The bedrock was found to be highly weathered at the soil/bedrock contact but becomes less weathered with depth.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

If encountered during earthwork operations, surface water on this site will most be likely the result of precipitation or possibly some minor surface run-off from the surrounding areas. Overall site area drainage is generally to the west-northwest direction. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was not encountered within any of the trenches performed to a maximum depth of about 10 feet below existing grades at the time of the field exploration. The depth to historic groundwater at the site is estimated to be greater than 50 feet below existing grades.

Based on the above, groundwater is not anticipated to be a factor during the site grading. However, seasonal perched groundwater may be encountered during grading within portions of the site.

4.4 FAULTING AND SEISMICITY

4.4.1 Faulting

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 an “Alquist-Priolo” Earthquake Fault Zone. The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced landsliding. The nearest zoned fault is the Morongo Valley fault, located about 2.2 miles to the north.

The County of San Bernardino Geologic Hazard Overlay Map indicates that the site is not located within a liquefaction susceptible or an earthquake fault zone area.

4.4.2 Seismic Design Parameters

The site is located at approximately 33.1113 degrees Latitude and -116.2938 degrees Longitude. Due to the dense/stiff nature of the site soils a Site Class “D” is considered appropriate for this site. Site spectral accelerations (S_a 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. Using the ASCE 7-16 option on the SEAOC/OSHPD website results in the values for S_{M1} and S_{D1} 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, C_s , is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_s$ or Eq. 12.8-4 for $T > T_L$.

The results, based on the 2015 NEHRP and the 2022 CBC, are presented in the following table as it is assumed that the exception as allowed in ASCE 7-16 is applicable. If the exception is deemed not appropriate, a site-specific ground motion analysis will be required.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, Ss	1.931g
Mapped 1.0 sec Period Spectral Acceleration, S1	0.693g
Site Coefficient for Site Class “D”, Fa	1.0
Site Coefficient for Site Class “D”, Fv	1.7
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, SMS	1.931g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, SMI	1.177g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, SDS	1.288g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, SDI	0.785g
Peak Ground Acceleration (PGAM)	0.898g
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.

4.5 LIQUEFACTION

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 project site is not located within an area mapped by the County of San Bernardino as having a potential for liquefaction. Based on the dense alluvial soils, relatively shallow bedrock, as well as the relatively deep estimated historic groundwater at the project site, the liquefaction hazard potential and unsaturated (“dry sand”) settlement potential at the project site is considered minimal.

4.6 OTHER SEISMIC HAZARDS

Due to the general flat terrain, the potential for seismic induced landslides or lateral spreading is considered nil. The potential for secondary seismic hazards such as a seiche and tsunami is considered negligible due to site elevation and distance from an open body of water.

Evidence of ancient landslides or slope instability at this site was not observed during the site reconnaissance. Therefore, the potential for landslides is considered negligible for design purposes.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Development of the site appears feasible from a geotechnical engineering viewpoint. The following recommendations should be incorporated into the design and construction phases of development. The site contains some non-uniform surficially loose alluvium. These soils will require removal and recompaction as engineered fill in order to provide a uniform bearing surface for the proposed structures.

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 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.

5.2.2 Site Clearing

Initial site preparation should commence with removal of debris, any deleterious materials and vegetation within the limits of the planned improvements. These materials should be properly disposed of off-site. Voids resulting from removing any materials should be replaced with engineered fill materials with expansion characteristics similar to the onsite materials.

5.2.3 Site Preparation

Due to the presence of upper disturbed site soils as encountered in the trenches performed, it is recommended that the upper two (2) feet of soil should be removed beneath all areas to receive new fill and/or surface improvements. The precise depths of over-excavation should be determined by a GeoTek representative during site construction based on actual soil conditions encountered. The lateral extent of this recommendation should include the area extending at least three (3) feet beyond the structure limits, where obtainable. The bottoms of all remedial excavations should be reviewed and approved by a GeoTek representative. Removal bottoms terminating in alluvial soils should be relatively uniform in soil type, which is not visibly 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. These removal recommendations apply to the proposed structures at the project site. Removals encountering granitic bedrock should extend to competent bedrock.

Once the base of the over-excavation is approved, the exposed soils should be scarified to a depth of about 12 inches, be moisture treated to slightly above the soil's optimum moisture content (ASTM D1557) and then be compacted to at least 90 percent of the soil's maximum dry density (ASTM D1557).

Testing by GeoTek is recommended to document that the engineered fill soils have been properly moisture conditioned and compacted as recommended.

5.2.4 Engineered Fill

The on-site materials are generally considered suitable for reuse as engineered fill provided, they are free from vegetation, debris, oversized materials (6-inch diameter or greater) and other deleterious material. All areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in 6-to-8-inch loose lifts, moisture conditioned to slightly above the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D-1557 test procedures.

If wet soils are encountered during remedial grading, methods for drying soils such as stockpiling or mixing with dry soils may be required to bring the soils to the required moisture content for placement as engineered fill. Placement of engineered fill should be observed and tested on a full-time basis by a GeoTek representative during grading activities.

5.2.5 Excavation Characteristics

Excavations in alluvial materials and weathered bedrock should be readily accomplished with heavy-duty earthmoving or excavating equipment in good operating condition.

Special excavation techniques may be required in less weathered bedrock areas and possibly other areas may require special excavation techniques as well.

It should be realized that the ability of any particular contractor to excavate the materials encountered will vary based on factors that may or may not be considered in GeoTek's evaluation. All methods available to evaluate rock hardness and associated rippability/excavatability are interpretive to some extent. As such, experience and judgment are primary factors in such evaluations.

Oversized rocks (>6 inches) should be anticipated within the area of the rock outcrop view deck area and hard floaters/corestones may be encountered at varying depths. The alluvial materials may contain significant quantities of oversize (i.e., greater than six inches maximum size). Special excavation techniques and/or equipment (screens) may be required to create acceptable fill.

5.2.6 Trench Excavations and Backfill

Temporary trench excavations within the on-site materials should be stable at a 1.5:1 inclination for short durations during construction and where cuts do not exceed 10 feet in height. Deeper temporary excavations should be reviewed by GeoTek prior to their planned excavation to determine if supplemental recommendations or analysis are warranted. It is anticipated that temporary cuts to a maximum height of four (4) 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 by 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 (6) 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 properly moisture conditioned prior to placement in trenches.

5.2.7 Shrinkage and Bulking

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.

For planning purposes, a shrinkage factor of about ten (10) to twenty (20) percent may be considered for disturbed soil and alluvial materials that need to be removed and replaced. Subsidence up to about 0.1 foot is estimated in soil areas during site preparation. No subsidence is expected in bedrock areas. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork construction.

5.2.8 Grading Plan Review

Upon completion of the site grading plans, it is recommended that those plans be provided to GeoTek for review. Based on that review, some modifications to the recommendations provided in this report may be necessary.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Shallow Foundation Design Criteria

The near surface materials can be classified as having a “Very Low” ($0 \leq EI \leq 20$) Expansion Index (EI) in accordance with ASTM D 4829. Typical design criteria for the site based upon a “Very Low” expansion index are tabulated below. These are minimal recommendations and are not intended to supersede the design by the project structural engineer. Once structural loading information is provided, revisions to the recommendations provided in this report may be necessary.

The conventional foundation elements for the proposed structures/improvements should bear entirely in engineered fill soils or bedrock (not a combination of the two). Foundations should be designed in accordance with the 2022 CBC.

Expansion index and soluble sulfate evaluation of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions.

A summary of the foundation design recommendations is presented in the following table:

GEOTECHNICAL RECOMMENDATIONS FOR FOUNDATION DESIGN

Design Parameter	“Very Low” Expansion Index
Minimum Foundation Depth (Inches below lowest adjacent grade)	12 – One- and two-stories
Minimum Foundation Width (Inches)*	12 – One- and two stories
Minimum Slab Thickness (actual)	4 – Actual
Minimum Slab Reinforcing	6” x 6” – W1.4 x W1.4 welded wire fabric placed in middle of slab or No. 3 bars at 18-inch centers, each way
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 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.

An allowable bearing capacity of 2,500 pounds per square foot (psf) may be used for design of building foundations for a footing depth of 12 inches and a footing width of 12 inches. This allowable soil bearing capacity may be increased by 500 psf for each additional 12 inches of embedment depth and by 400 psf for each additional 12 inches in width to a maximum of 3,750 psf. The allowable bearing capacity may also be increased by one-third when considering short-term wind and seismic loads. These bearing values contain a minimum factor of safety of three (3).

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 or competent native soil. 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. The coefficient of friction contains a reduction value of one-third (1/3). 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 or isolated pad foundations. 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 in concrete slabs 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.



For footings designed in accordance with the recommendations presented in this report, it is anticipated that a maximum static settlement of less than an inch and a maximum differential static settlement of about ½ the total static settlement in a 40-foot span will occur. Dynamic (seismic-induced) settlements are estimated to be minimal due to the presence of relatively dense alluvial soils and/or shallow bedrock. These static and dynamic settlement estimates should be accounted for in the proposed building design by the project structural engineer.

Where the slab is to be designed as a beam on an elastic foundation, a modulus of subgrade reaction (k-value) of 250 pounds per cubic inch (pci) may be considered for design based on a presumed value for a 1-foot by 1-foot plate load test. Dependent upon how the mat slab is loaded, the subgrade modulus value may need to be geometrically modified.

5.3.2 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 a result of construction related punctures (e.g., stake penetrations, tears, punctures from walking on the vapor retarder placed atop the underlying 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. Although the CBC specifies a 6-mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

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 environmental 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 permeability) to achieve the desired performance level.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek recommends that a qualified person, such as the flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the building be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person (or persons) should provide recommendations relative to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate.

In addition, the recommendations in this report and GeoTek's services in general are not intended to address mold prevention; since GeoTek, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

5.3.3 Miscellaneous Shallow Foundation Recommendations

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

5.3.3.2 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

Minimum setbacks for all foundations should comply with the 2022 CBC or County of San Bernardino requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements.

If large enough, these movements can compromise the integrity of the improvements. The top outside 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 five (5) feet and need not exceed 40 feet.

5.3.5 Deep Foundation Design Criteria – Drilled Piers

Drilled piers used to support the proposed structures should extend to a minimum depth of eight (8) feet below finish grade, or a minimum embedment of at least five (5) feet into competent materials and be a minimum of 12 inches in diameter. This minimum depth incorporates any potential down drag (negative skin friction) associated with the existing disturbed soils. The drilled piers can be designed using an average allowable skin friction value of 350 pounds per square foot.

The top foot of embedment depths for the drilled pier foundations should be neglected unless the ground surface is covered by concrete or pavement. The allowable skin friction value provided incorporates a safety factor of 2.0 and may be increased by one-third for temporary wind and/or seismic loading. Alternatively, the drilled pier can be designed using an allowable end-bearing capacity of 6,000 psf. End-bearing and skin friction values cannot be combined.

A representative of the geotechnical engineer should be present during installation to verify the embedment depth. The capacities are based on soil strength considerations, and stresses within the piles and piers may impose more severe limitations. The recommendations are for single-acting piles and piers. If necessary, the capacities for closely spaced piles and piers should be reduced in accordance with the efficiency formula provided below:

$$E = 1 - \frac{\phi[(n-1)m+(m-1)n]}{90mn}$$

where:

E = Pile efficiency

$\phi = \tan^{-1}(d/s)$

d = pile diameter

s = spacing of piles center to center

m = number of rows

n = number of piles in a row

For insertion into the efficiency formula, groups containing three piles or piers should be considered to have two rows and two piles or piers per row.

If caving soils are encountered, drilled piers should be provided with temporary steel casing. The temporary casing may be withdrawn as concrete is placed within the drilled pier, keeping a concrete head of at least two feet above the bottom of the casing as it is being removed.

After excavating the drilled piers, the excavations should be observed by the geotechnical consultant of record and concrete placed as quickly as possible to avoid exposure of the

foundation sidewalls to wetting and drying. Surface run-off water should be drained away and not be allowed to pond adjacent to the excavations. It is recommended that foundation concrete be placed within 24 hours of the drilled pier excavation being completed.

If it is required that foundation excavations be left open for more than one day, they should be properly covered to prevent personal injury and fall hazards and be protected to reduce evaporation or entry of moisture. Where drilled piers are to be placed closer than 2.5 diameters center to center, the closely spaced piers should be constructed on different days.

The vertical uplift capacity of the piers due to friction is considered two-thirds ($2/3$) of the download friction capacity (230 psf), plus the weight of the pier.

Lateral bearing capacity of drilled piers may be computed using any accepted code-allowed “pole” formula. The allowable lateral passive earth pressure for drilled piers may be considered to develop at a rate of 400 pounds per square foot per foot of depth to a maximum value of 6,000 psf. This value may be increased by one-third for wind or seismic loading. However, the allowable passive pressure can be doubled where the piers will not be adversely affected by $1/2$ inch lateral movements at the ground surface. The effective width of the drilled piers can be assumed to be twice the pier diameter for passive resistance calculations.

Moment distribution and lateral deflection analyses for the drilled piers can be provided, if needed, when vertical and lateral loads are known.

For drilled piers designed and constructed as recommended in this report, it is anticipated a maximum static settlement of less than one inch will occur with a maximum differential static settlement of $1/2$ inch over a 50-foot span.

5.3.6 Soil Corrosivity

The resistivity of the near surface soils at this site was tested in the laboratory on one (1) soil sample collected during the field investigation. The results of the testing indicate that the near surface soils are considered “moderately corrosive” (8,040 ohm-cm) (Roberge, 2000) to buried ferrous metal in accordance with current standards used by corrosion engineers. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer. Additional corrosion testing should be performed at the time of site grading to assess the corrosion potential of the as-graded soils.

5.3.7 Soil Sulfate Content

The sulfate content of the near surface soils was determined in the laboratory on one (1) soil sample collected during the field investigation. The results indicate that the water-soluble

sulfate content is less than 0.1 percent by weight, which is considered “negligible” (“S0” exposure classification) as per Table 19.3.1.1 of ACI 318-19. Based on the test results and Table 19.3.1.1 of ACI 318-19, no special recommendations for concrete are required for this project due to soil sulfate exposure.

5.3.6 Import Soils

Import soils should consist of soils similar or better than the on-site soils and should be tested for expansion and corrosivity potential prior to their use. GeoTek, Inc. should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

The following retaining wall design recommendations 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 18 inches into engineered fill or competent bedrock. 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 5.2.4 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.

5.4.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to six (6) feet high. Active earth pressure may be used for design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the pressure against the wall.

Appropriate fluid unit weights are given below for specific slope gradients of 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	35
2:1	50

*The design pressures assume the backfill material has an expansion index less than or equal to 30. 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 $13H^2$ [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 $22H^2$ 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.

5.4.3 Retaining Wall Backfill and Drainage

The wall backfill should also include a minimum one (1) foot wide section of $3/4$ - 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 percent 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 $\frac{3}{4}$ - 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.

5.4.4 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 55 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.

5.4.5 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 CONCRETE CONSTRUCTION

5.5.1 General

Concrete construction should follow the 2022 CBC and ACI guidelines regarding design, mix placement and curing of the concrete. If desired, GeoTek could provide quality control testing of the concrete during construction.

5.5.2 Concrete Mix Design

As discussed in Section 5.3.5, no special recommendations for concrete are required for this project due to soil sulfate exposure. Additional testing should be performed during grading so that specific recommendations can be formulated based on the as-graded conditions.

5.5.3 Concrete Flatwork

Exterior concrete flatwork is often one of the most visible aspects of site development. They are typically given the least level of quality control, being considered “non-structural” components. Cracking of these features is common due to various factors. While cracking usually does not affect the structural performance of the concrete, it is unsightly. It is recommended that the same standards of care be applied to these features as to the structure itself.

Exterior concrete slabs and sidewalks should be designed using a four (4) inch minimum thickness. No specific reinforcement is required due to the non-structural nature. However, the use of some reinforcement should be considered. Recommendations can be provided upon request. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in residential 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 herein. The upper 12 inches of hardscape subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 90 percent of the maximum dry density as determined by ASTM D1557 test procedures.

Subgrade soils, classified as having “Very Low” expansion potential, should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. at the subject site should be pre-saturated to a minimum of 100% of optimum moisture content to a depth of 12 inches for “Very Low” expansive soils.

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.

5.5.4 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 undergoes chemical processes that are dependent on 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 at a distance approximately equal to 24 to 36 times the slab thickness.

5.6 POST CONSTRUCTION CONSIDERATIONS

5.6.1 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. An abatement program to control ground-burrowing rodents should be implemented and maintained. 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.

5.6.2 Drainage

Positive site drainage should be maintained at all times. 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.

5.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

It is recommended that site grading, specifications, and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. It is also recommended that GeoTek representatives be present during site grading and foundation construction to observe and document for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of all 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 where necessary.
- Observe the fill for uniformity during placement, including utility trench excavation backfill. Also, test the fill for density, relative compaction and moisture content.
- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

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 Trench 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-0311424-CR) dated March 28, 2024, and geotechnical engineering standards normally used on similar projects in this region.

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.

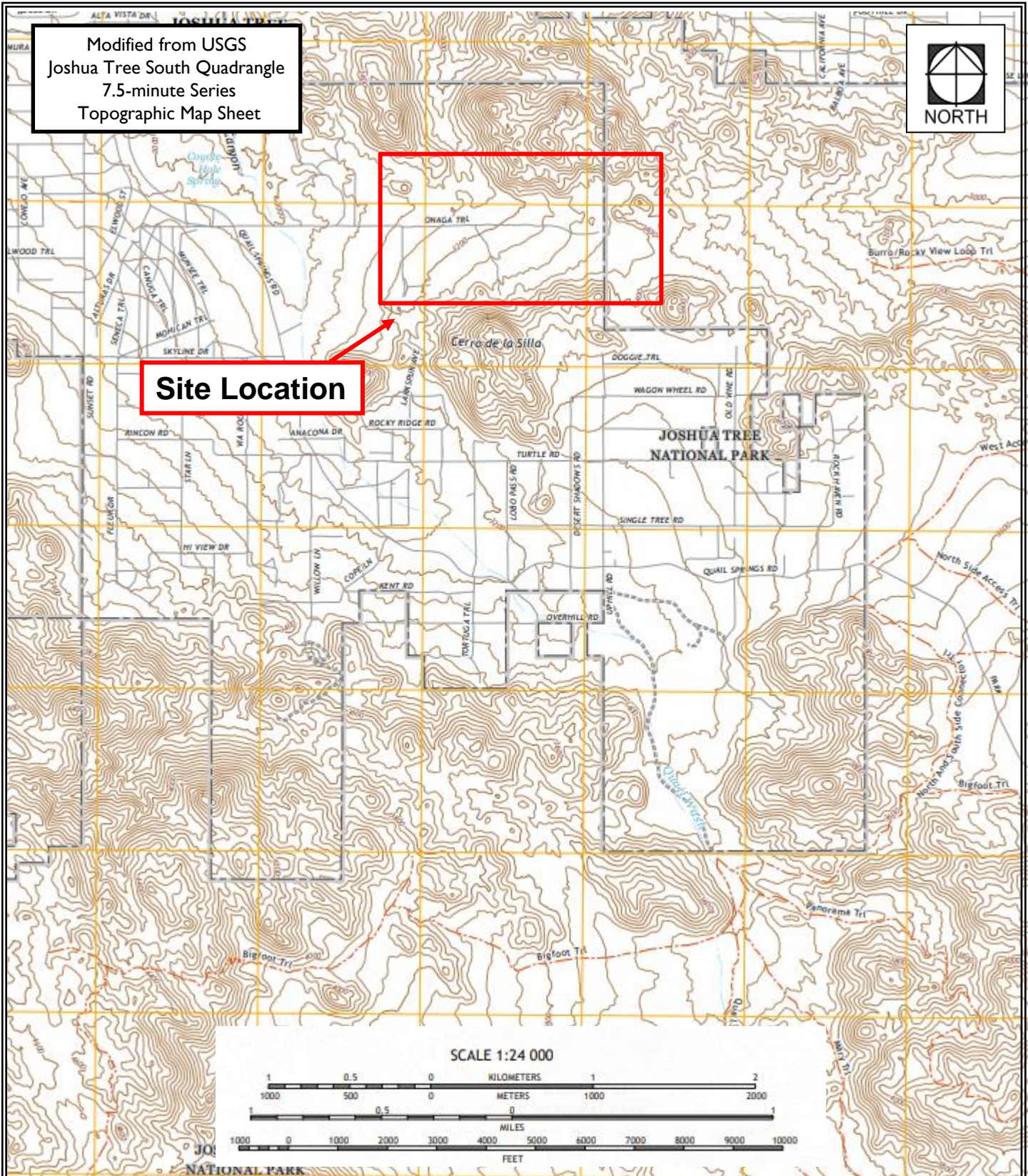
8. SELECTED REFERENCES

American Concrete Institute (ACI), 2006, Publication 302.2R-06, Guide for Concrete Slabs That Receive Moisture Sensitive Flooring Materials.



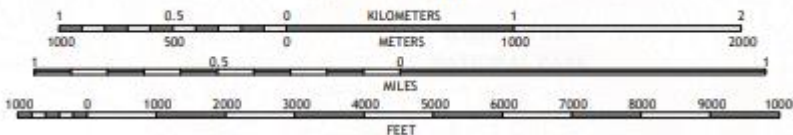
- _____, 2010, Publications 360R-10, Guide to Design of Slabs-On-Ground.
- _____, 2019, Publications 318-19, Building Code Requirements for Structural Concrete.
- American Society of Civil Engineers (ASCE), 2017, “Minimum Design Loads for Buildings and Other Structures,” ASCE/SEI 7-16.
- Architerra Design Group, undated, “Desert View Conservation Area, Overall Conceptual Site Plan”.
- Bowles, J. E., 1977, “Foundation Analysis and Design”, Second Edition.
- Bryant, W.A., and Hart, E.W., 2007, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps, California Geological Survey: Special Publication 42.
- California Code of Regulations, Title 24 Part 2, 2022 “California Building Code,” 2 volumes
- California Geological Survey (CGS, formerly referred to as the California Division of Mines and Geology), 1977, “Geologic Map of California.”
- _____, 2008, “Guidelines for Evaluating and Mitigating Seismic Hazards in California,” Special Publication 117A.
- _____, 2018, “Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Special Publication 42 (Revised).
- Department of the Navy, 1982, “Foundations and Earth Structures”, Design Manual 7.2.
- Dibblee, T.W., 1967, “Geologic Map of the Joshua Tree Quadrangle, San Bernardino and Riverside Counties, California,” U.S. Geological Survey, Miscellaneous Geologic Investigations Map I-516, scale 1:62,500.
- GeoTek, Inc., In-house proprietary information.
- Peck R. B., Hanson W. H. & Thornburn T. H., 1974, “Foundation Engineering”, Second edition.
- Roberge, P. R., 2005, “Handbook of Corrosion Engineering,” 2nd Edition.
- Seismic Design Values for Buildings (<http://geohazards.usgs.gov/designmaps/us/application.php>).

Modified from USGS
Joshua Tree South Quadrangle
7.5-minute Series
Topographic Map Sheet



Site Location

SCALE 1:24 000



Architerra Design Group
Desert View Conservation Area Phase 2
Joshua Tree, San Bernardino County, California

Project No. 3910-CR

Figure 1
Site Location
and Topography
Map





5/2025

Desert View Conservation Area

Onaga Trail

T3-T4

T5-T6

T1-T2

LEGEND
(Locations are Approximate)

 T6 Trench Area

Architerra Design Group
Desert View Conservation Area
Joshua Tree, San Bernardino County, California

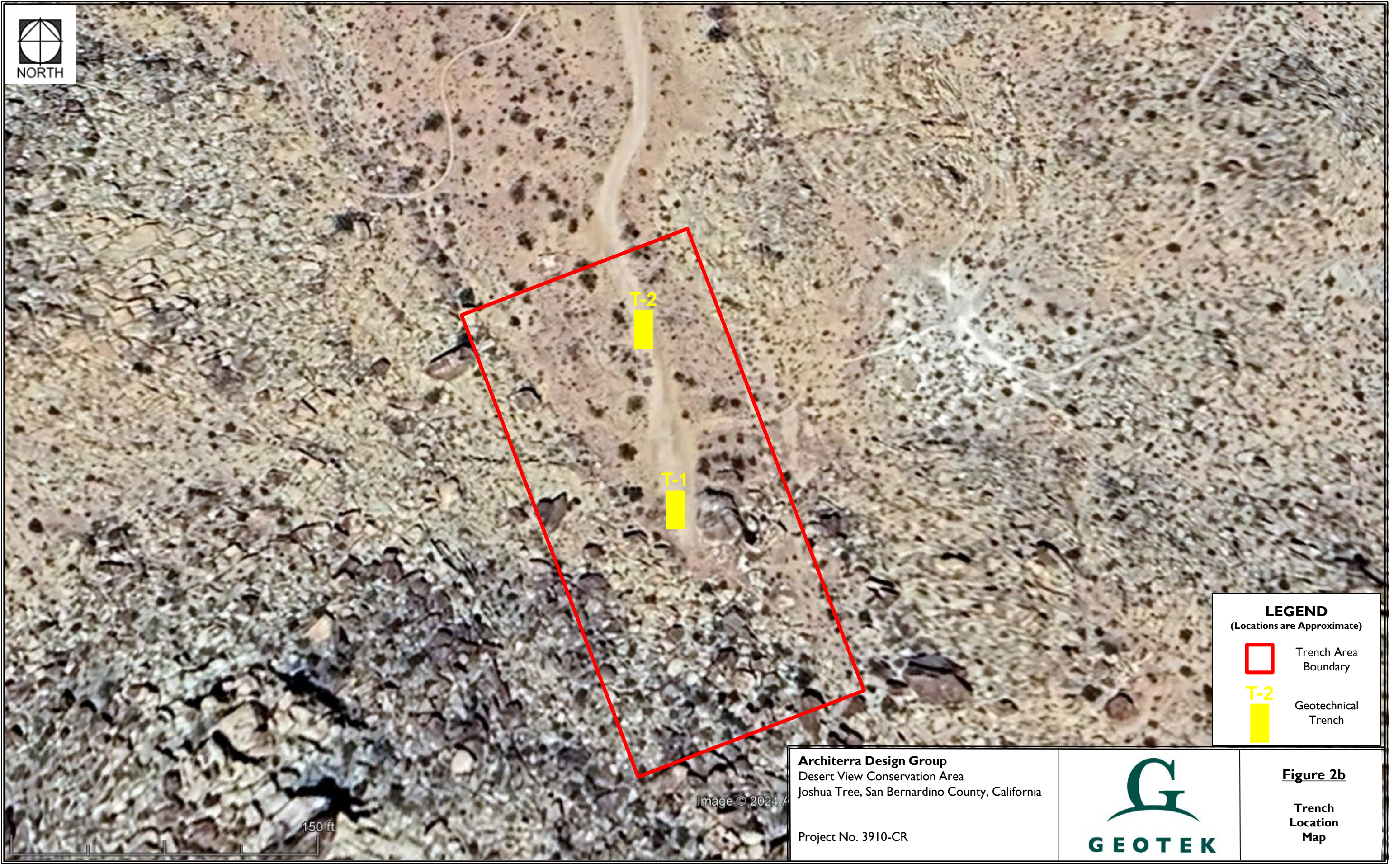
Project No. 3910-CR




Figure 2a
Trench
Location
Map


Image © 2024 Airbus

1000 ft



LEGEND
(Locations are Approximate)

 Trench Area Boundary

 T-2 Geotechnical Trench

Architerra Design Group
Desert View Conservation Area
Joshua Tree, San Bernardino County, California

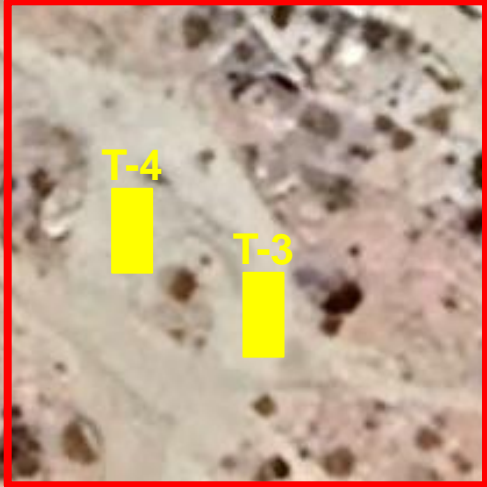
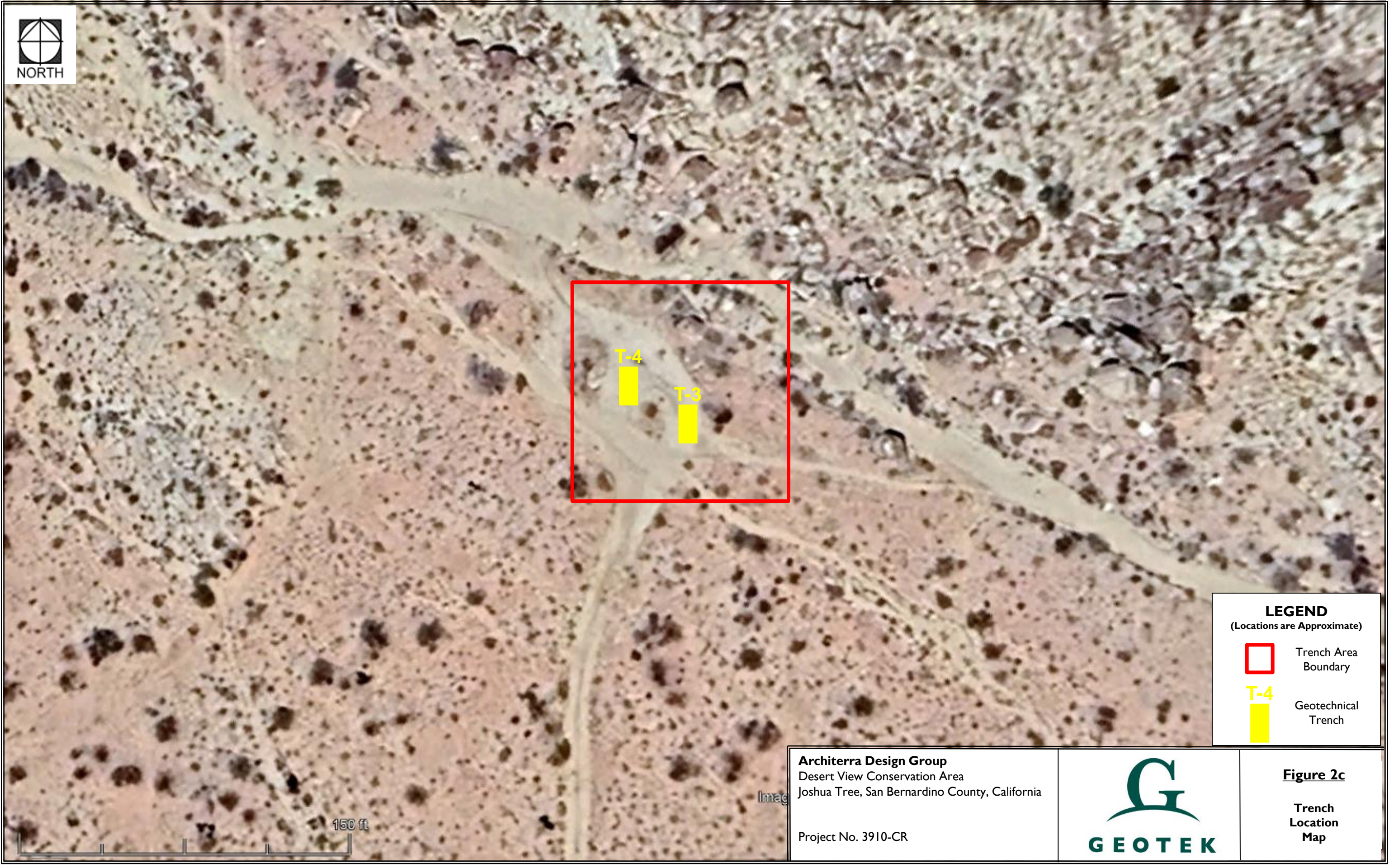
Project No. 3910-CR



Figure 2b
Trench Location Map

150 ft

Image © 2024 A




T-4




T-3



LEGEND
(Locations are Approximate)

 Trench Area Boundary

 T-4 Geotechnical Trench

Architerra Design Group
Desert View Conservation Area
Joshua Tree, San Bernardino County, California

Project No. 3910-CR

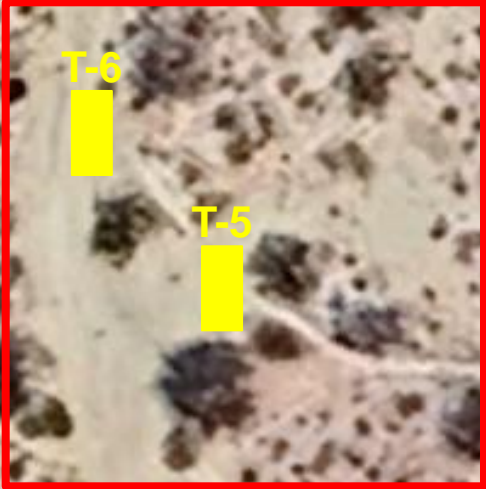


Figure 2c
Trench Location Map







NORTH



LEGEND
(Locations are Approximate)

 Trench Area Boundary

 T-6 Geotechnical Trench

Architerra Design Group
Desert View Conservation Area
Joshua Tree, San Bernardino County, California

Project No. 3910-CR

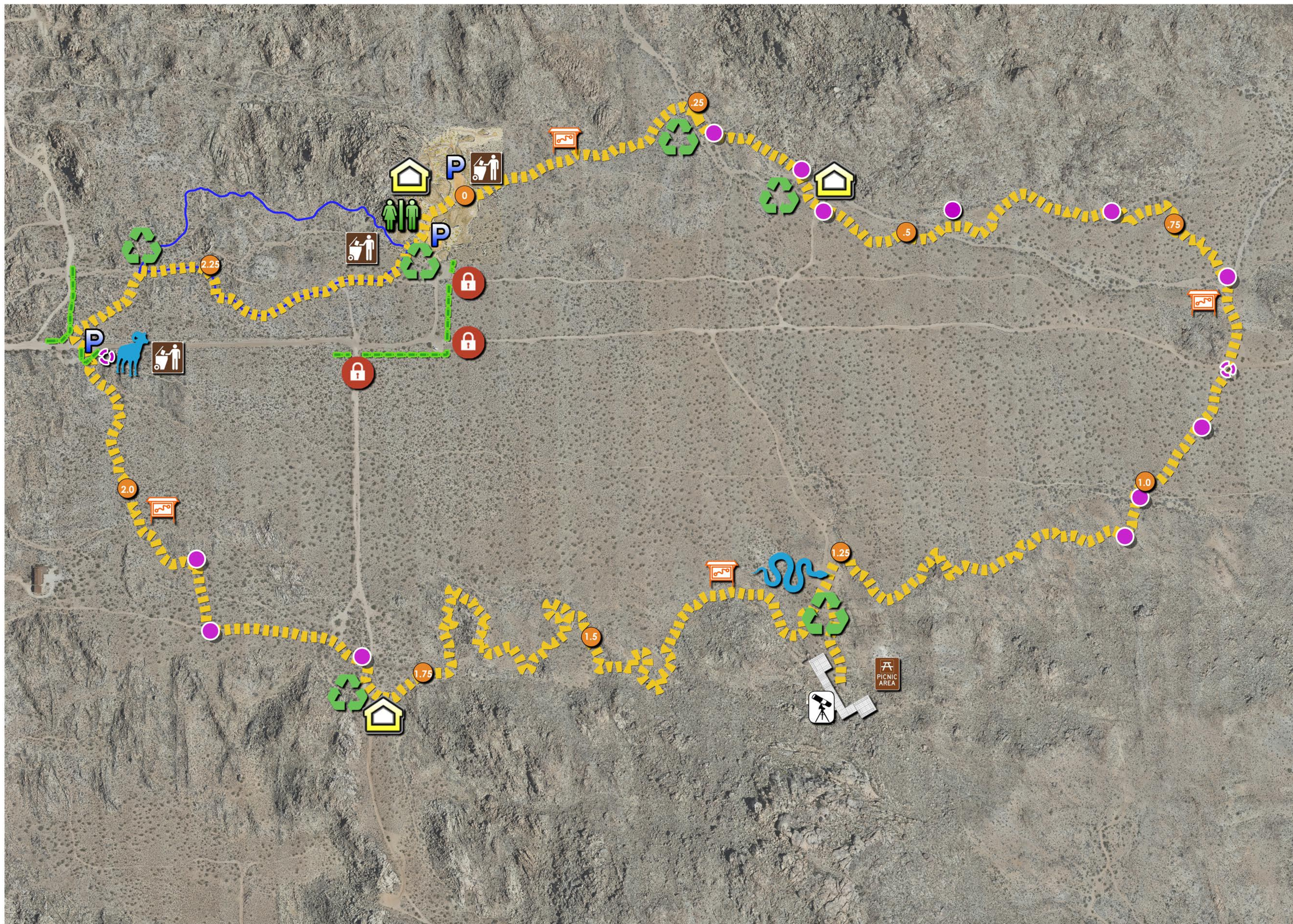


Figure 2d

Trench Location Map

150 ft





FEATURE LEGEND

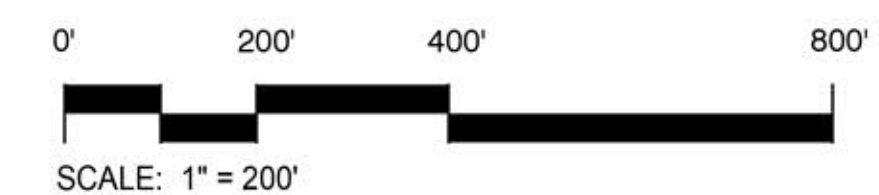
-  DISCOVERY SHELTERS
-(JOSHUA TREE WOODLAND, CREOSOTE, NATIVE AMERICAN)
-  ENVIRONMENT AWARENES SIGN PANEL
(6 TOTAL)
-  TRASH AND RECYCLE RECEPTACLES
(3 EACH)
-  ROCK OUTCROP VIEW DECK WITH
VIEWING TELESCOPE
-  KIOSK LOCATIONS (4 TOTAL)
-  BIG HORN INTERPRETIVE SIGN PANEL
LOCATION
-  .25
DISTANCE MARKERS
(SET EVERY .25 MILE)
-  BIG HORN SHEEP TRAIL MARKER
-  WAY FINDING SIGN POTENTIAL
LOCATIONS (12 TOTAL)
-  BIG HORN TRAIL ROUTE
-  REPTILE INTERPRETIVE SIGN PANEL
LOCATION
-  PICNIC TABLE AREA
-  BARRIER FENCE LOCATION
-  PHASE I - TORTOISE TRAIL ROUTE
-  RESTROOM LOCATION
-  PARKING AREAS
-  CONTROLLED ACCESS / GATES



OVERALL CONCEPTUAL TRAIL PLAN



Desert View Conservation Area



DATE: MAY 31, 2024
SCALE: 1" = 200'

APPENDIX A

LOGS OF EXPLORATORY TRENCHES

**Geotechnical Evaluation
Phase II Desert View Conservation Area
63539 Onaga Trail
Joshua Tree area of San Bernardino County, California
Project No. 3910-CR**



A - FIELD TESTING AND SAMPLING PROCEDURES

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 - TRENCH LOG LEGEND

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

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 trench

(Additional denotations and symbols are provided on the trench logs)

GeoTek, Inc.
LOG OF EXPLORATORY HAND AUGER BORING

CLIENT: Architerra Design Group
PROJECT NAME: Phase II Desert View Conservation Area
PROJECT NO.: 3910-CR
LOCATION: 34.108585, -116.290186

LOGGED BY: DRW
EQUIPMENT: Backhoe
DATE: 8/8/2024
ELEVATION: 3315'

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: T-1 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blow Count			Water Content (%)	Dry Density (pcf)	Others
5	SM		SM	Alluvium: Silty f-c SAND, light grayish brown, dry, trace gravel, trace rootlets			SH, MD, EI EI = 2
5	SM/SP		SM/SP	Silty f-c SAND to f-c SAND, light brown, dry to slightly moist, few gravel, trace cobble			
5	SP		SP	F-c SAND, light brown, slightly moist, few gravel, trace cobble			
10				TRENCH TERMINATED AT 10.0' No groundwater encountered Trench backfilled with excavated materials			
15							

LEGEND	Sample type:	---Ring	---Large Bulk	--Small Bulk	---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 HAND AUGER BORING

CLIENT: Architerra Design Group
PROJECT NAME: Phase II Desert View Conservation Area
PROJECT NO.: 3910-CR
LOCATION: 34.108817, -116.290232

LOGGED BY: DRW
EQUIPMENT: Backhoe
DATE: 8/8/2024
ELEVATION: 3306'

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: T-2	Laboratory Testing		
	Sample Type	Blow Count			MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)
5			SM	Alluvium: Silty f-c SAND, light grayish brown, dry, trace gravel, trace rootlets			
			SM/SP	Silty f-c SAND to f-c SAND, light brown, dry to slightly moist, few gravel, trace cobble			
				Becomes slightly moist			
			SP	F-c SAND, light brown, slightly moist, few gravel, trace cobble			
10				TRENCH TERMINATED AT 10.0'			
				No groundwater encountered Trench backfilled with excavated materials			
15							

LEGEND	Sample type:	---Ring	---Large Bulk	--Small Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits SR = Sulfate/Resistivity Test	EI = Expansion Index SH = Shear Test	SA = Sieve Analysis HC= Consolidation	RV = R-Value Test MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY HAND AUGER BORING

CLIENT: Architerra Design Group
PROJECT NAME: Phase II Desert View Conservation Area
PROJECT NO.: 3910-CR
LOCATION: 34.114062, -116.290355

LOGGED BY: DRW
EQUIPMENT: Backhoe
DATE: 8/8/2024
ELEVATION: 3171'

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: T-3 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blow Count			Water Content (%)	Dry Density (pcf)	Others
5	X		SP	<u>Alluvium:</u> Silty f-c SAND, light grayish brown, dry, trace gravel, trace rootlets			SR
5	X		SM/SP	Silty f-c SAND to f-c SAND, light brown, dry to slightly moist, few gravel, trace cobble			
5	X			<u>Granitic Bedrock:</u> Indurated, excavates as silty f-c SAND, gray, slightly moist			
10				TRENCH TERMINATED AT 6.5' No groundwater encountered Trench backfilled with excavated materials			
15							

LEGEND	Sample type:	---Ring	---Large Bulk	--Small Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits SR = Sulfate/Resisitivity Test	EI = Expansion Index SH = Shear Test	SA = Sieve Analysis HC= Consolidation	RV = R-Value Test MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY HAND AUGER BORING

CLIENT: Architerra Design Group
PROJECT NAME: Phase II Desert View Conservation Area
PROJECT NO.: 3910-CR
LOCATION: 34.114124, -116.290442

LOGGED BY: DRW
EQUIPMENT: Backhoe
DATE: 8/8/2024
ELEVATION: 3169'

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: T-4 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blow Count			Water Content (%)	Dry Density (pcf)	Others
5			SM	Alluvium: Silty f-c SAND, light grayish brown, dry, trace gravel, trace rootlets			
			SP	F-c SAND, light brown, slightly moist, few gravel, trace cobble			
			SM	Silty f-c SAND, light brown, slightly moist, few gravel			
			SM/SP	Silty f-c SAND to f-c SAND, grayish brown, slightly moist, some gravel, trace cobble			
10				TRENCH TERMINATED AT 10.0' No groundwater encountered Trench backfilled with excavated materials			
15							

LEGEND	Sample type:	---Ring	---Large Bulk	--Small Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits SR = Sulfate/Resisitivity Test	EI = Expansion Index SH = Shear Test	SA = Sieve Analysis HC= Consolidation	RV = R-Value Test MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY HAND AUGER BORING

CLIENT: Architerra Design Group
PROJECT NAME: Phase II Desert View Conservation Area
PROJECT NO.: 3910-CR
LOCATION: 34.10857, -116.296625

LOGGED BY: DRW
EQUIPMENT: Backhoe
DATE: 8/8/2024
ELEVATION: 3181'

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: T-5 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blow Count			Water Content (%)	Dry Density (pcf)	Others
5	SM		SM	Alluvium: Silty f-c SAND, light grayish brown, dry, trace gravel, trace rootlets			
	SM/SP		SM/SP	Silty f-c SAND to f-c SAND, light brown, dry to slightly moist, few gravel, trace cobble			
	SP		SP	F-c SAND, light brown, slightly moist, few gravel, trace cobble			
	SM/SP		SM/SP	Silty f-c SAND to f-c SAND, light brown, dry to slightly moist, few gravel, few cobble			
10				TRENCH TERMINATED AT 10.0'			
15				No groundwater encountered Trench backfilled with excavated materials			

LEGEND	Sample type:	---Ring	---Large Bulk	--Small Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits SR = Sulfate/Resisitivity Test	EI = Expansion Index SH = Shear Test	SA = Sieve Analysis HC= Consolidation	RV = R-Value Test MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY HAND AUGER BORING

CLIENT: Architerra Design Group
PROJECT NAME: Phase II Desert View Conservation Area
PROJECT NO.: 3910-CR
LOCATION: 34.108672, -116.296705

LOGGED BY: DRW
EQUIPMENT: Backhoe
DATE: 8/8/2024
ELEVATION: 3180'

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: T-6 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blow Count			Water Content (%)	Dry Density (pcf)	Others
5			SP	Alluvium: Silty f-c SAND, light grayish brown, dry, trace gravel, trace rootlets			
			SM/SP	Silty f-c SAND to f-c SAND, light brown, dry to slightly moist, few gravel, trace cobble			
				Granitic Bedrock: Indurated, excavates as silty f-c SAND, gray, slightly moist			
10				TRENCH TERMINATED AT 6.5' No groundwater encountered Trench backfilled with excavated materials			
15							

LEGEND	Sample type:	---Ring	---Large Bulk	--Small Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits SR = Sulfate/Resisitivity Test	EI = Expansion Index SH = Shear Test	SA = Sieve Analysis HC= Consolidation	RV = R-Value Test MD = Maximum Density

APPENDIX B

LABORATORY TEST RESULTS

**Geotechnical Evaluation
Phase II Desert View Conservation Area
63539 Onaga Trail
Joshua Tree area of San Bernardino County, California
Project No. 3910-CR**



SUMMARY OF LABORATORY TESTING

Classification

Soil and excavated bedrock materials 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 Trenches in Appendix A.

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 tests were performed on soil samples remolded to approximately 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. The shear test results are presented graphically in Appendix B.

Expansion Index

Expansion Index testing was performed on one (1) soil sample. 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
T-1	0-3	Silty Sand (SM)	2	Very Low

Moisture-Density Relationship

Laboratory testing was performed on one (1) 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 test method ASTM Test Procedure D 1557. The results are presented graphically in Appendix B.

Sulfate Content, Resistivity and Chloride Content

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

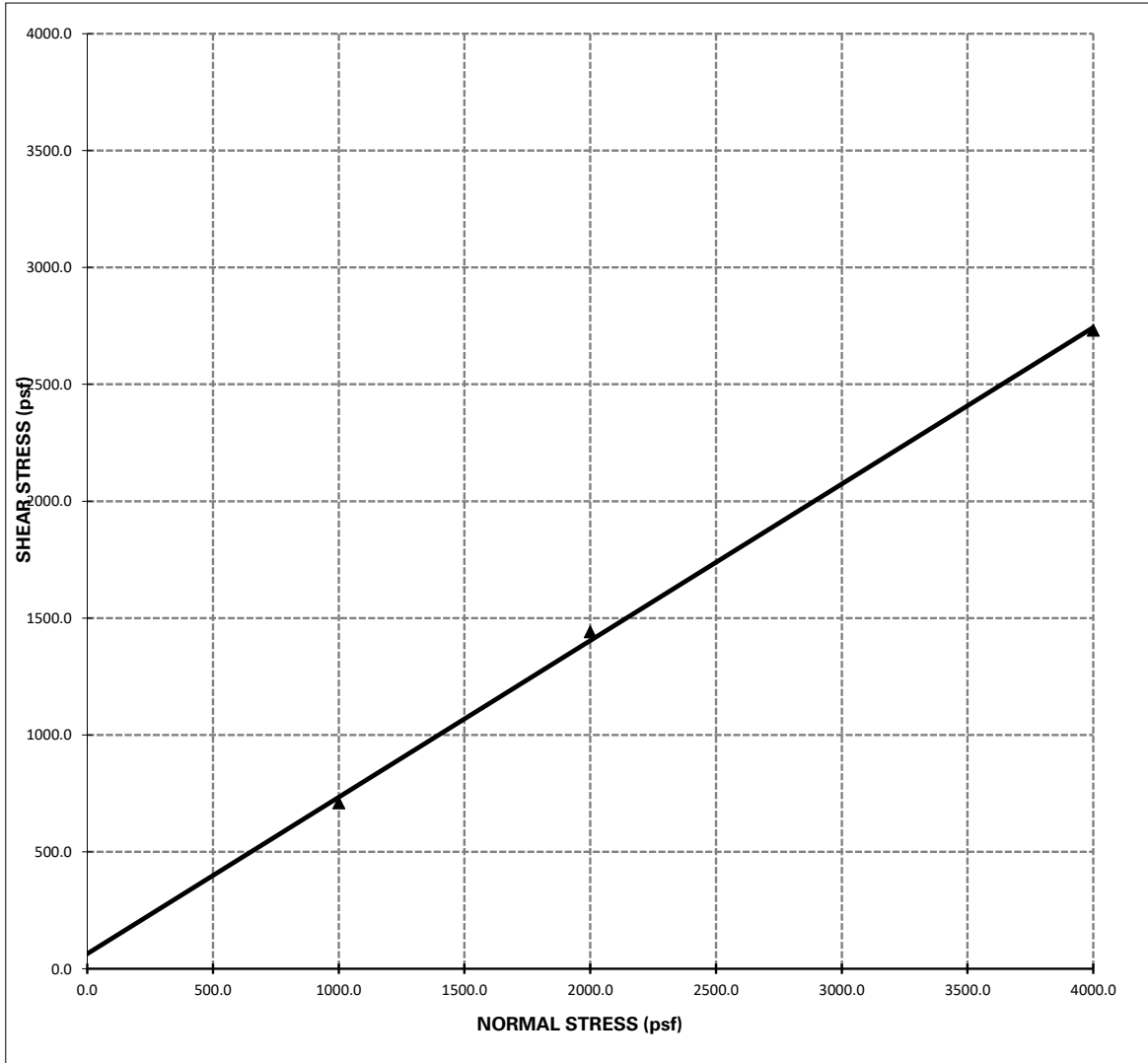
Boring #	Depth (ft.)	pH ASTM G51	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
T-3	1-4	7.1	26.5	0.0052	8,040



DIRECT SHEAR TEST

Project Name: Phase II DVCA, Joshua Tree
Project Number: 3910-CR

Sample Location: T-1 @ 0-3 Feet
Date Tested: 8/16/2024



Shear Strength: $\Phi = 34^\circ$; **C = 64 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: Architerra Design Group, Inc.
Project Number: 3910-CR
Project Location: Phase II DVCA, Joshua Tree

Tested/ Checked By: RK Lab No Corona
Date Tested: 8/13/2024
Sample Source: T-1 @ 0-3 Feet
Sample Description: _____

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	774.5
Weight of ring (gm)	362.5
Net weight of sample (gm)	412.0
Wet Density, lb / ft3 (C*0.3016)	124.3
Dry Density, lb / ft3 (D/1.F)	114.5

SATURATION DETERMINATION

Moisture Content, %	8.5
Specific Gravity, assumed	2.70
Unit Wt. of Water @ 20°C, (pcf)	62.4
% Saturation	48.7

READINGS		
DATE	TIME	READING
8/13/2024		0.9000
8/13/2024		0.9000
8/14/2024		0.9020

Initial
10 min/Dry

Final

FINAL MOISTURE	
Final Weight of wet sample & tare	% Moisture
792.6	12.9

EXPANSION INDEX = 2



Report No: PTR:24-00171-S01

Proctor Report

Client: Architerra Design Group, Inc.
 Attn: Richard Krumwiede
 Rancho Cucamonga CA 91730

CC:

Project: 3910-CR
 Phase II DVCA, Joshua Tree

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

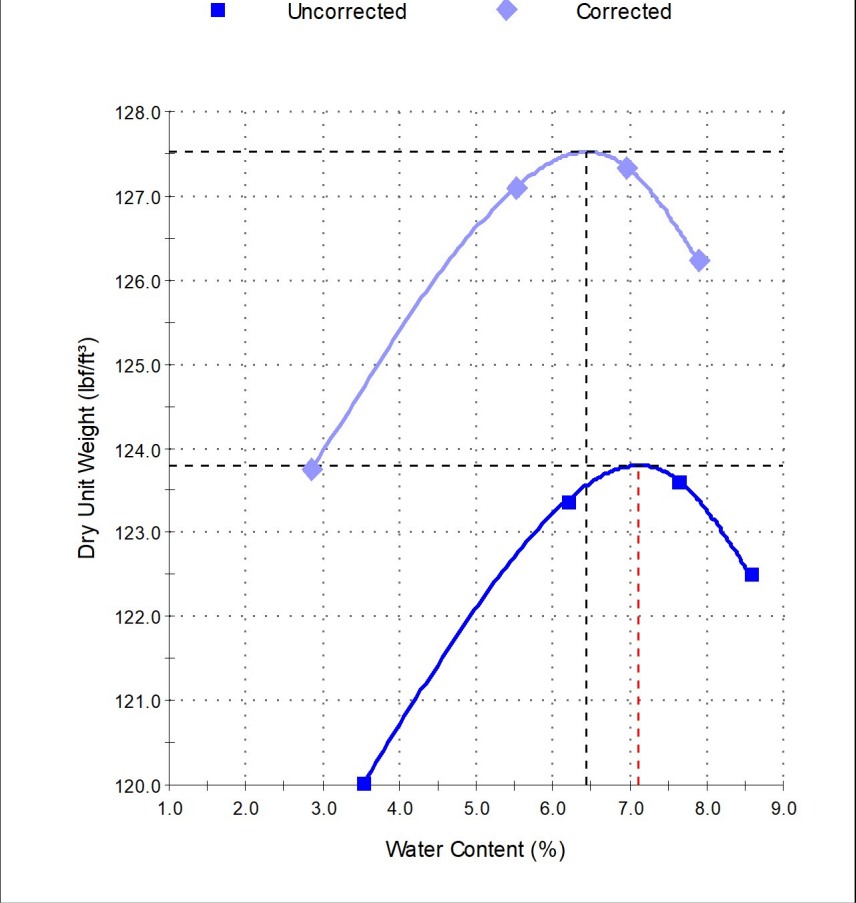
Sample ID: 24-00171-S01 **Date Sampled:** 8/8/2024

Sampled By: Daniel Whitmer

Material: Silty Fine to CXoarse SAND

Location: T-1 @ 0-3 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557	
Maximum Dry Unit Weight (lb/ft³):	123.8
Optimum Water Content (%):	7.1
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	11
Passing Sieve No 4 (4.75mm) (%):	89
Tested By:	Eduardo Cuevas
Date Tested:	8/12/2024
ASTM D 4718	
Corrected Maximum Dry Unit Weight (lb/ft³):	127.5
Corrected Optimum Water Content (%):	6.4
Specific Gravity (Oversize):	2.70
Sieve Size (Oversize):	No 4
Oversize Particles (%):	11

Comments



Results Only Soil Testing for Phase 2 DUCA

August 14, 2024

Prepared for:

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

Project X Job#: S240813C
Client Job or PO#: 3910-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





Soil Analysis Lab Results

Client: GeoTek USA
 Job Name: Phase 2 DUCA
 Client Job Number: 3910-CR
 Project X Job Number: S240813C
 August 14, 2024

Bore# / Description	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 D4327	ASTM D4327	
		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 ⁺	Magnesium Mg ²⁺	Calcium Ca ²⁺	Fluoride F ₂ ⁻
	(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)
T-3	1-4	52.0	0.0052	26.5	0.0026	120,600	8,040	7.1	151	0.7	16.6	3.5	ND	34.3	5.3	19.1	84.3	2.8	0.5

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](#)

APPENDIX C

GENERAL EARTHWORK GRADING GUIDELINES

**Geotechnical Evaluation
Phase II Desert View Conservation Area
63539 Onaga Trail
Joshua Tree area of San Bernardino County, California
Project No. 3910-CR**



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 colluvium as well as weathered or creep effected bedrock, should 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 contractor's 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 contractor's 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 contractor's 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.

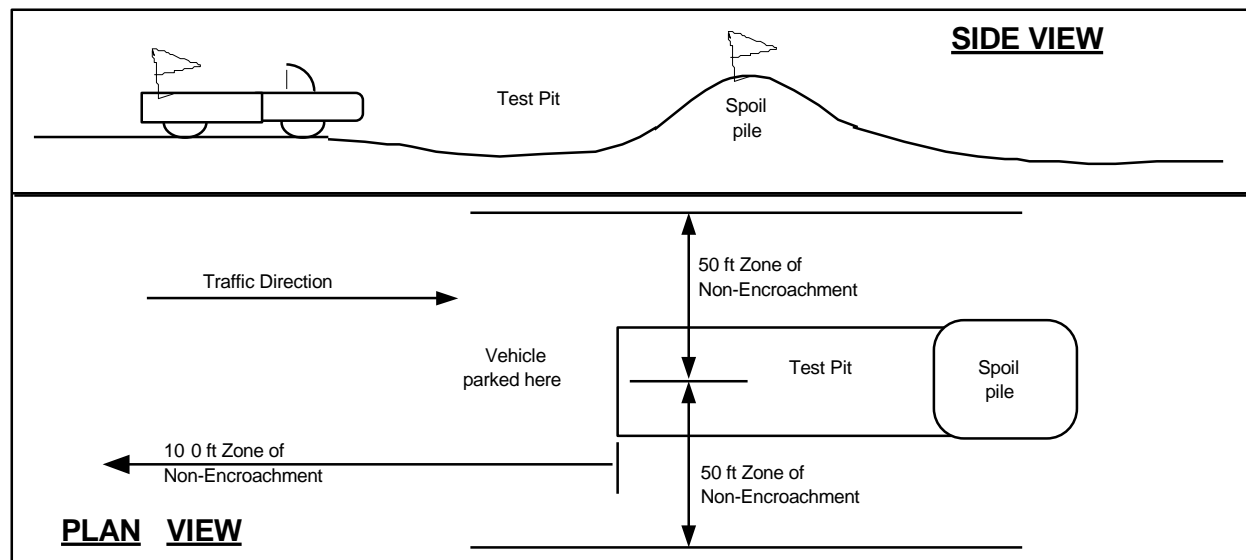
Test Pits Location, Orientation and Clearance

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 contractor's 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.