

Appendix 5 Geotechnical Evaluation Report

**GEOTECHNICAL EVALUATION
WILDOMAR 20-ACRE NATURE PARK
PALOMAR STREET AND SOUTH PASADENA STREET
CITY OF WILDOMAR, RIVERSIDE 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

June 18, 2024
Project No. 3901-CR

Architerra Design Group

10221-A Trademark
Rancho Cucamonga, California 91730

Attention: Mr. Peter Dalman

Subject: **Geotechnical Evaluation**
Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
City of Wildomar, Riverside County, California

Dear Mr. Dalman:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical Evaluation for the Wildomar 20-Acre Nature Park project to be located easterly of the intersection of Palomar Street and South Pasadena Street, in the City of Wildomar, Riverside 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

Kase I. Gebbie, EIT
Staff Engineer

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

\\geotekfs1\Riverside\Projects\3901 to 3950\3901CR Architerra Design Group Wildomar 20-Acre Nature Park Wildomar\Geotechnical Evaluation\3901CR Geotechnical Evaluation Wildomar 20-Acre Nature Park.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	3
3.1 FIELD EXPLORATION.....	3
3.2 LABORATORY TESTING.....	3
4. GEOLOGIC AND SOILS CONDITIONS	4
4.1 REGIONAL SETTING.....	4
4.2 GENERAL SOIL CONDITIONS	5
4.2.1 Disturbed Soil.....	5
4.2.2 Alluvium.....	5
4.2.3 Pauba Formation Sedimentary Bedrock.....	5
4.3 SURFACE WATER AND GROUNDWATER	6
4.3.1 Surface Water	6
4.3.2 Groundwater.....	6
4.4 FAULTING AND SEISMICITY	6
4.4.1 Faulting.....	6
4.4.2 Lineament Analysis	7
4.4.3 Seismic Design Parameters.....	8
4.5 LIQUEFACTION.....	9
4.6 OTHER SEISMIC HAZARDS.....	9
5. CONCLUSIONS AND RECOMMENDATIONS	10
5.1 GENERAL.....	10
5.2 EARTHWORK CONSIDERATIONS.....	10
5.2.1 General.....	10
5.2.2 Site Clearing.....	10
5.2.3 Site Preparation	10
5.2.4 Engineered Fill.....	11
5.2.5 Trench Excavations and Backfill	11
5.2.6 Shrinkage and Bulking.....	12
5.2.7 Grading Plan Review	12
5.3 DESIGN RECOMMENDATIONS.....	12
5.3.1 Shallow Foundation Design Criteria.....	12
5.3.2 Miscellaneous Shallow Foundation Recommendations.....	15
5.3.3 Foundation Setbacks.....	16
5.3.4 Deep Foundation Design Criteria – Drilled Piers.....	16
5.3.5 Soil Corrosivity.....	18
5.3.6 Soil Sulfate Content.....	18
5.3.6 Import Soils	18
5.4 RETAINING WALL DESIGN AND CONSTRUCTION	18
5.4.1 General Design Criteria.....	18

TABLE OF CONTENTS

5.4.2	Cantilevered Walls.....	19
5.4.3	Retaining Wall Backfill and Drainage.....	20
5.4.4	Restrained Retaining Walls.....	21
5.4.5	Additional Retaining Wall Design Considerations	21
5.5	PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS	21
5.5.1	Asphalt Concrete (AC) Pavement	21
5.5.2	Permeable Interlocking Concrete Pavers.....	22
5.5.3	Portland Cement Concrete (PCC) Pavement.....	23
5.5.4	Porous Asphalt Concrete	24
5.5.4	Gravel Surfaced Concrete Parking Lot	24
5.5.5	Pavement Construction	24
5.6	CONCRETE CONSTRUCTION.....	25
5.6.1	General.....	25
5.6.2	Concrete Mix Design	25
5.6.3	Concrete Flatwork.....	25
5.6.4	Concrete Performance.....	26
5.7	POST CONSTRUCTION CONSIDERATIONS	27
5.7.1	Landscape Maintenance and Planting	27
5.7.2	Drainage	27
5.8	PLAN REVIEW AND CONSTRUCTION OBSERVATIONS	27
6.	INTENT	28
7.	LIMITATIONS	28
8.	SELECTED REFERENCES	29

ENCLOSURES

Figure 1 – Site Location and Topography Map

Figure 2 – Boring Location Map

Figure 3 – California Geological Survey Earthquake Zones of Required Investigation Map

Figure 4 – Lineament Air Photo Assessment Map

Appendix A – Logs of Exploratory Borings

Appendix B – Laboratory Test Results

Appendix C – Historic Aerial Photographs

Appendix D – 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-0407924-CR, dated April 24, 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 eight (8) exploratory test borings extending to depths of approximately 11.5 to 51.5 feet below existing grades,
- Laboratory testing of soil samples collected during the field investigation,
- Review and evaluation of site seismicity and geologic hazards, including a fault lineament study based on research of available topographic maps, geologic reports for the area, and aerial photography information for the project site, 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 approximate 20.54-acre irregular shaped project site is located easterly of the intersection of Palomar Street and South Pasadena Street, in the City of Wildomar, Riverside County, California (see Figure 1, Site Location and Topography Map). The project site is comprised of



two (2) parcels identified as Riverside County Assessor's Parcel Numbers (APNs) 380-050-025 and -029. The two parcels are separated by a concrete-lined drainage channel in the western portion of the project site.

Access to the site is from Palomar Street, a paved, improved street located adjacent to the southwestern boundary of the site. The site currently consists of vacant, undeveloped land, with a light scattering of trees and moderate covering of native grasses and brush.

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

The site is bound by the concrete-lined drainage channel, Palomar Street, and Anne Sullivan Preschool to the southwest, a single-family residential development to the northeast, mostly by vacant land associated with a single-family residence to the southeast, and by vacant land for the future extension of Gierson Avenue or Como Street to the northwest.

2.2 PROJECT DESCRIPTION

Based upon review of the referenced *Wildomar 20 Acre Nature Park Conceptual Master Plan* prepared by Architerra Design Group, GeoTek understands that the project will include, but not necessarily be limited to, two (2) prefabricated restroom buildings; a shade structure; a pedestrian bridge over the aforementioned adjacent concrete-lined drainage channel; parking areas; monument signs; walking trails; landscaping; and other associated improvements. Gierson Avenue will be extended into the northeast corner of the site and is to be terminated by the construction of a partially asphalt concrete paved and gravel surfaced parking lot area. A porous asphalt concrete parking lot is also proposed with access from Palomar Street.

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

The pedestrian bridge abutments are anticipated to be supported by deepened shallow foundations and possibly by cast-in-drilled-hole (CIDH) deep foundations and are to be constructed in the southern portion of the project site adjacent to the concrete lined drainage channel. Current structural loading for the proposed bridge abutments is unknown.

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 May 30 and June 11, 2024, and included the excavating of eight (8) geotechnical exploratory borings with a track-mounted hollow-stem auger drill rig to depths between about 11.5 and 51.5 feet each below grade at the boring locations.

The approximate locations of the GeoTek excavations are shown on the Geotechnical Investigation Map (Figure 2). A geologist from GeoTek logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory borings are included in Appendix A.

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

In Borings B-2 and B-3, standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, and split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The sampler penetration test data are presented on the Logs of Borings for B-2 and B-3.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected relatively undisturbed ring and 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 the 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 borings logs included in Appendix A and in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends approximately 975 miles south of the Transverse Ranges geomorphic province to the tip of Baja California. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is in an area geologically mapped to be underlain by alluvium and sedimentary bedrock of the Pauba Formation (Morton, D.M., Miller, F.K., 2006).

The site is partially located within an “*Alquist-Priolo*” fault hazard zone along Palomar Street and the approximate southwestern half of the project site. The County of Riverside has indicated the site to be located within a “County Fault Zone” identified as the Elsinore Fault Zone. The site has not yet been mapped to be located within a California Geological Survey identified liquefaction or seismic-induced landslide zone. Additionally, the County of Riverside has indicated the site is located within a fault line identified as the “Elsinore Fault | Wildomar Fault”, as having a “Moderate/Very Low” liquefaction potential hazard and as “susceptible” with regards to subsidence.

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 some surficial disturbed soils underlain by alluvium and sedimentary bedrock of the Pauba Formation.

4.2.1 Disturbed Soil

Disturbed soils, consisting of relatively loose clayey to silty sands, sandy silts and sandy clays (SC, SM, ML, and CL soil types based upon the Unified Soil Classification System), were encountered in all borings to depths of about 6 inches to 3 feet below existing grades. Deeper or other deposits of disturbed soils may be present in areas of the site that were not explored. The upper site soils were likely disturbed as a result of previous land uses. Undocumented fill soils, while not encountered within any of the borings performed, may be present at the site in areas that were not explored, especially in the vicinity of any existing improvements.

4.2.2 Alluvium

Alluvial soils were encountered beneath the surficial disturbed soils in all of the borings performed. The alluvium was found to generally consist of medium dense to very dense silty sands, clayey sands, and relatively stiff to hard silty and sandy clays (SM, SC, and CL soil types based upon the Unified Soil Classification System). Alluvial soils were encountered within all borings performed, at depths ranging between about 1.5 to 16.5 feet below existing grades.

Based on the laboratory test results, the near surface materials have a “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). Based upon the consolidation tests, the upper site soils generally demonstrated a “very low” potential for hydro-consolidation (settlement due to the addition of water with or without additional loading). Based on the R-Value test result obtained from a select sample in the proposed parking lot area in the northern portion of the site, the anticipated subgrade soils generally exhibit a relatively poor pavement support capacity, when recompacted. The laboratory test results are provided in Appendix B.

4.2.3 Pauba Formation Sedimentary Bedrock

Late Pleistocene-aged sedimentary bedrock of the Pauba Formation was encountered within all borings performed with the exception of Boring B-6. The sedimentary bedrock of the Pauba Formation was encountered as shallow as about 1.5 feet below existing grades within Boring B-4 and up to about 15 feet below existing grades near the southern portion of the site.

The sedimentary bedrock, as encountered in the borings, generally consists of very pale brown to brown and grey-brown to olive-grey, slightly moist to moist, dense to hard, indurated, massive silty sandstone to sandy siltstone with minor amounts of clay. Caliche stringers were observed to be present within the bedrock units. Sedimentary bedrock is regionally mapped by others as being located within the southern portion of the site, and within the site vicinity to the north, east, and west.

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 south-southwesterly 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 borings performed to a maximum depth of about 51.5 feet below existing grades within Borings B-2 or B-3 at the time of drilling. The California Department of Water Resources, Water Data Library indicates multiple groundwater wells within a 0.5-mile radius of the project site, which suggests groundwater levels in excess of 50 feet below existing grades. Therefore, 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.

The site is partially situated within an “Alquist-Priolo” Earthquake Fault Zone. The County of Riverside has designated the site as in a “County Fault Zone” identified as the “Elsinore Fault

Zone”, within the “Elsinore Fault” and “Wildomar Fault” fault lines, as having a “Moderate” to “Very Low” potential for liquefaction and as “Susceptible” to subsidence.

4.4.2 Lineament Analysis

In order to identify possible mapped and unmapped faults and to evaluate the topographic expressions of published or previously reported fault traces, a lineament analysis was performed for the project site.

GeoTek reviewed aerial photographs provided by Environmental Data Resources, Inc. (EDR) dated 1938, 1949, 1953, 1961, 1967, 1978, 1985, 1989, 1994, 1997, 2002, 2005, 2009, 2012, 2016 and 2020. The aerial photographs reviewed were at a scale of 1” = 500’. Source Maps were provided by USDA, USGS/DOQQ and USDA/NAIP.

Geologic field mapping was also utilized in GeoTek’s lineament analysis.

Lineaments can generally be classified as strong, moderate or weak. A strong lineament is a well-defined feature, which can be continuously traced several hundred feet to a few thousand feet. A moderate lineament is less well defined, somewhat discontinuous, and can be traced for only a few hundred feet. A weak lineament is discontinuous, poorly defined, and can be traced for a few hundred feet or less.

The subject property appears to be vacant land on all of the aerial photographs dating back to 1938. Multiple aerial photographs show evidence of some minor ground disturbance, particularly in the 1985 aerial photograph.

A strong northwest-trending lineament is visible on the southern portion of the subject site. The photo-lineament is identified by a faint to very sharp tonal color change that is best observed in the southern portion of the site in the 1949 aerial photograph. The 1949 aerial photograph and subsequent lineaments are shown on Figure 4 – Lineament Air Photo Assessment Map.

In the parcels to the northwest of the site the lineament continues on a northwest trend; showing faint to moderate tonal color change. More strikingly, a strong northwest trending lineament identified by a very sharp tonal color change is visible on the parcels to the southeast of the site. The strong tonal change is interpreted by GeoTek to be likely caused by a subsurface groundwater barrier, likely associated with strike-slip faulting in the region. The moisture appears to be trapped on the northern side of the lineament, thus possibly causing the vastly darker tones in the aerial photographs. The development of the planned

improvements for the Nature Park are not anticipated to be adversely impacted by faulting due to the absence of planned habitable structures.

4.4.3 Seismic Design Parameters

The site is located at approximately 33.6004 degrees Latitude and -117.2666 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 $T \leq 1.5T_L$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_L$ 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, S_s	1.682g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.625g
Site Coefficient for Site Class “D”, F_a	1.0
Site Coefficient for Site Class “D”, F_v	1.7
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.682g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	1.063g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.121g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.709g
Peak Ground Acceleration (PGA_M)	0.810g
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 located within an area mapped by the County of Riverside as having a “moderate/very low” for liquefaction potential. Based on the relatively shallow sedimentary 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 disturbed soils, and relatively 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 City of Wildomar, the County of Riverside, the 2022 California Building Code (CBC) and recommendations contained in this report. The Grading Guidelines included in Appendix D 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 D.

5.2.2 Site Clearing

In areas of planned grading and improvements, the locations of existing utilities should be determined. Existing utilities should be relocated or abandoned. 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 borings 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. Deeper removals may be necessary within the areas of planned bridge abutments. The removals for the planned abutments should extend until competent materials are reached. 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 sedimentary 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 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.

5.2.6 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 0.2 feet 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.

5.2.7 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 “Low” ($21 \leq EI \leq 50$) Expansion Index (EI) in accordance with ASTM D 4829. Typical design criteria for the site based upon a “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	“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” – W2.9 x W2.9 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 110% 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 2,000 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 400 psf for each additional 12 inches of embedment depth and by 200 psf for each additional 12 inches in width to a maximum of 3,250 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 300 psf per foot of depth, to a maximum earth pressure of 3,000 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 0.30 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. 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 $1/2$ 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 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 150 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.

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.

5.3.2 Miscellaneous Shallow Foundation Recommendations

5.3.2.1 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.

5.3.2.2 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.

5.3.3 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2022 CBC, County of Riverside, or City of Wildomar 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.

5.3.4 Deep Foundation Design Criteria – Drilled Piers

Drilled piers used to support the proposed pedestrian bridge should extend to a minimum depth of ten (10) 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 200 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. This value 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 (130 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 300 pounds per square foot per foot of depth to a maximum value of 4,500 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 ½ 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.

5.3.5 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” (5,226 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.

5.3.6 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

Recommendations presented in this report apply to typical masonry or concrete vertical retaining walls to a maximum height of up to six (6) feet. Additional review and recommendations should be requested for higher walls. These are typical design criteria and are not intended to supersede the design by the structural engineer. Retaining wall foundations should be embedded a minimum of 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	48
2:1	72

*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 $12H^2$ [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 $20H^2$ 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 to 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 $3/4$ - 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 71 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 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

5.5.1 Asphalt Concrete (AC) Pavement

Although planned final grades beneath the proposed parking, access roads and adjacent street improvements within the site are not yet known, the following preliminary pavement design recommendations are based on assumed Traffic Indexes of 5.0 for local streets and car parking areas and 6.0 for access drives, in accordance with the County of Riverside Standard No. 114.

Preliminary pavement thickness design is based on the Caltrans Highway Design Manual (2018). An R-value of 12 was tested by GeoTek in general accordance with ASTM D 2844 test procedures, which was then assumed for the determination of preliminary pavement sections for this report.

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

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

PRELIMINARY MINIMUM PAVEMENT SECTION*		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0 (Car Parking Areas)	4.0	6.5
6.0 (Access/Truck Delivery/Emergency Vehicle Drives)	4.0	10.0
	5.0	8.0

*Shall be in accordance with the County of Riverside Standard No. 114.

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

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

5.5.2 Permeable Interlocking Concrete Pavers

Interlocking concrete pavers can be used for this project. The pavers are assumed to be approximately 3.2 inches (80 millimeters) in thickness. Concrete pavers should be underlain by 1.0 inch to 1.5 inches of bedding sand overlying four inches of aggregate base founded on compacted subgrade soils. The aggregate base should be compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

Where possible, the aggregate base should extend beyond the perimeter of the pavers a minimum distance of four inches. The bedding sand should be placed and lightly moistened and

compacted. Since this compaction cannot be tested it should be observed by a representative of this firm.

Historically, paver systems have experienced failures in areas where water has degraded the support characteristics of the underlying base and/or subgrade soils. Since paver systems are permeable and allow transmission of water into the underlying materials, it may be prudent to discuss with the paver designer/manufacturer what methods may be employed to address the issue of potential water introduction into the underlying materials.

Underdrain systems, local subgrade reinforcement, tilting the pavement subgrade to drain away from paver areas and not pond, or additional structural elements such as geotextiles can be considered, particularly in high traffic areas and/or low areas where water will tend to collect. The recommendations of the designer/manufacturer should then be implemented into the design and construction of the paver system.

5.5.3 Portland Cement Concrete (PCC) Pavement

For the proposed vehicle parking and access lanes, it is recommended that a minimum of five and one-half (5.5) inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures be utilized. For vehicle service lanes, it is recommended that a minimum of six (6.0) inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density be utilized. This section should also be used in heavy truck traffic areas such as fire lanes, trash dumpster pads and approaches.

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

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

5.5.4 Poured Asphalt Concrete

Poured asphalt concrete is proposed for the surface of the southwestern parking lot to be located adjacent to Palomar Street. Permeable pavements, unlike traditional pavements that are impermeable, reduce the volume and peak of stormwater runoff, provided the underlying soils can accept infiltration. Permeable pavement surfaces generally function best under relatively flat to very gently sloping ground conditions. The permeable asphalt surface layer is placed on top of a reservoir layer that holds the water quality stormwater volume, which infiltrates from the reservoir layer to the native subsoil. Compaction of the subgrade soils will likely significantly reduce the infiltration capacity of the subgrade soils, and therefore the use of poured asphalt concrete may not be suitable for high trafficked areas and drive aisles and may be more suitable for parking spaces.

Sediment clogging the soil below the permeable pavement surface and reservoir layer should be expected over time, even with proper surface maintenance. Therefore, the reservoir layer is recommended by Riverside County LID BMP guidelines (RCFC, 2018) to not exceed 12 inches in depth to ensure adequate drainage. The minimum thickness of the poured asphalt should be at least 4 inches in thickness and be underlain by at least 10 inches of AASHTO #57 gravel (or equivalent) vibrated in place, with a minimum void ratio of 0.40.

5.5.4 Gravel Surfaced Concrete Parking Lot

Gravel surface parking is proposed for a portion of the parking lot located near the extension of Gierson Avenue in the northern portion of the site. The gravel surface should consist of at least twelve (12) inches in thickness. The subgrade beneath the gravel surface parking lot should consist of at least 18 inches of engineered fill.

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

5.5.5 Pavement Construction

All pavement installation, including preparation and compaction of subgrade and base material, placement and rolling of asphaltic concrete and placement of concrete pavement, should be

done in accordance with the City of Wildomar, County of Riverside guidelines, and under the observation and testing of GeoTek and a City/County inspector, where required.

Any aggregate base should consist of crushed rock with an R-Value and gradation in accordance with Crushed Aggregate Base (Section 400-2.4 of the “Greenbook” Regional Supplement Amendments). Any Crushed Miscellaneous Base shall conform to Section 200-2.4 of the Green Book.

Minimum compaction requirements for aggregate base should be 95 percent of maximum dry density as determined by ASTM D 1557 test procedures for both soil subgrade and aggregate base. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern. The upper 12 inches of subgrade should be moisture-conditioned to at least optimum moisture. The top of the subgrade and aggregate base should be graded to drain to the perimeter of the pavement.

5.6 CONCRETE CONSTRUCTION

5.6.1 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.

5.6.2 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.

5.6.3 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 recompact to at least 90 percent of the maximum dry density as determined by ASTM D1557 test procedures.

Subgrade soils, classified as having “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 110% of optimum moisture content to a depth of 12 inches for “Low” expansive soils.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Wildomar specifications, and under the observation and testing of GeoTek and a City Inspector, if necessary.

5.6.4 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.

5.7 POST CONSTRUCTION CONSIDERATIONS

5.7.1 Landscape Maintenance and Planting

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

Overwatering should be avoided. 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.

5.7.2 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.

5.8 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 Geotechnical Investigation 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-0407924-CR) dated April 24, 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, "Wildomar 20 Acre Nature Park Conceptual Master Plan,".

Bowles, J. E., 1977, "Foundation Analysis and Design", Second Edition.

Bray, J.D., Sancio, R.B., 2006, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils," Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, dated September 1.

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.

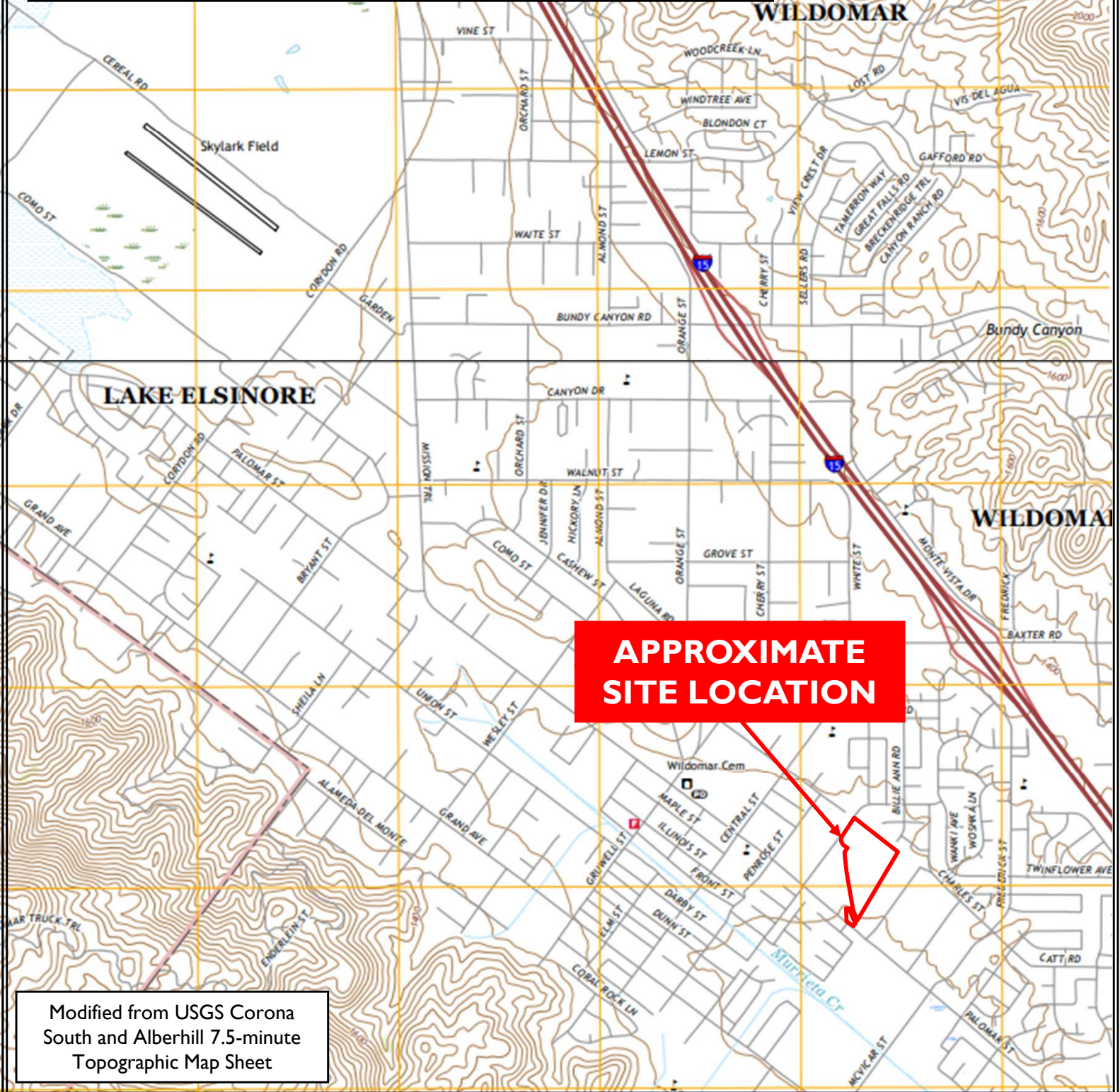
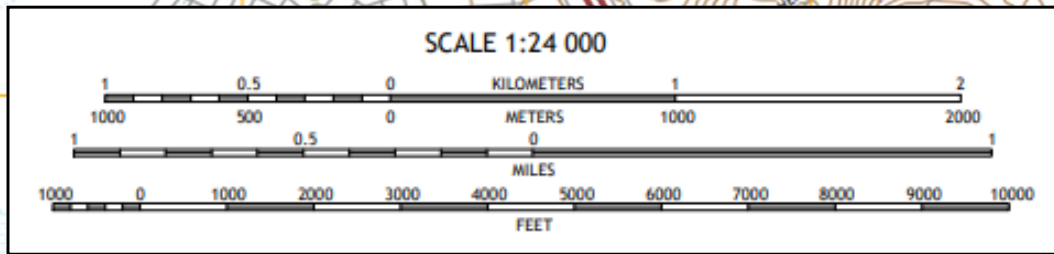
California Department of Transportation, 2018, "Highway Design Manual", 6th Edition.

California Geological Survey (CGS, formerly referred to as the California Division of Mines and Geology), 1977, "Geologic Map of California."

_____, 1978a, "Fault Evaluation Report FER-72," dated September 6.

_____, 1978b, "Fault Evaluation Report FER-76," dated November 16.

- _____, 1979a, "Supplement to Fault Evaluation Report FER-72," dated January 30.
- _____, 1979b, "Supplement No. 1 to Fault Evaluation Report FER-76," dated March 23.
- _____, 1979c, "Supplement No. 2 to Fault Evaluation Report FER-76," dated April 18.
- _____, 1980, "Wildomar Quadrangle, California – Riverside County, 7.5 Minute Series (Topographic), State of California Special Studies Zone," Official Map effective January 1.
- _____, 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.
- GeoTek, Inc., In-house proprietary information.
- Kennedy, M.P., 1977, "Recency and Character of Faulting Along the Elsinore Fault Zone in Southern Riverside County, California: California Division of Mines and Geology Special Report No. 131, 12 pages.
- Lamar, D.L., and Rockwell, T.K., 1986, "An Overview of the Tectonics of the Elsinore Fault Zone, in Ehlig, P.L., ed, Neotectonics and Faulting in Southern California, Guidebook for the 82nd Annual Meeting of the Cordilleran Section of the Geologic Society of America.
- Morton, D.M., Miller, F.K., 2006, "Geologic Map of the San Bernardino and Santa Ana 30' by 60' Quadrangles, California," U.S. Geological Survey, Open-File Report OF-2006-1217, Scale 1:100,000.
- Peck R. B., Hanson W. H. & Thornburn T. H., 1974, "Foundation Engineering", Second edition.
- Riverside, County of, 2022, "Standard No. 114 (Sheets 1 and 2)", dated November 30.
- _____, Map My County web service, v11.5 ([Map My County v11.5 \(countyofriverside.us\)](http://mapmycounty.com/v11.5/)).
- Riverside County Flood Control and Water Conservation District, 2018, "Riverside County Santa Margarita River Watershed Region, Design Handbook for Low Impact Development, Best Management Practices," dated June.
- Roberge, P. R., 2005, "Handbook of Corrosion Engineering," 2nd Edition.
- Seismic Design Values for Buildings (<http://geohazards.usgs.gov/designmaps/us/application.php>).



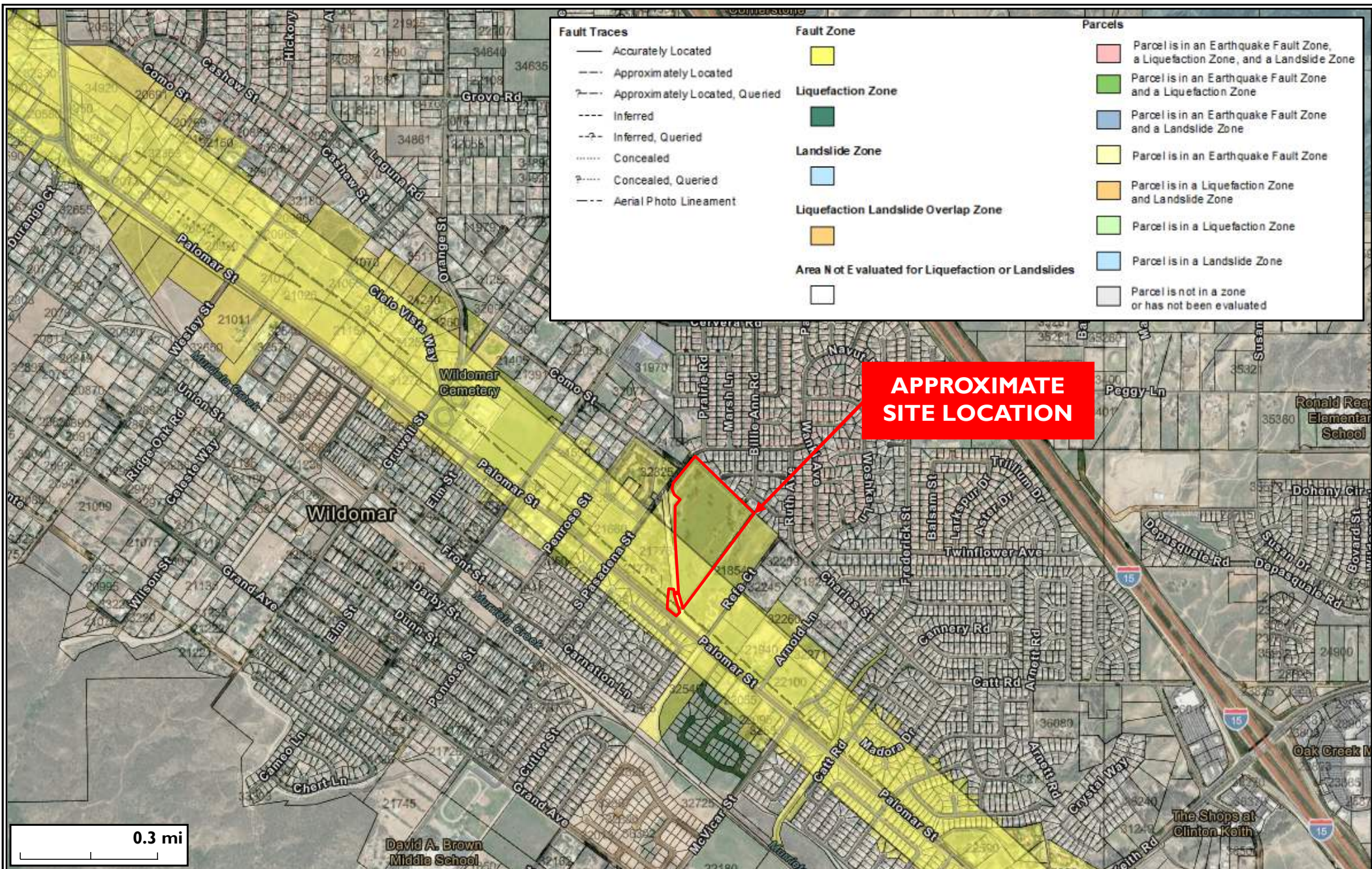
Modified from USGS Corona
South and Alberhill 7.5-minute
Topographic Map Sheet

Architerra Design Group
Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
Wildomar, Riverside County, California

GeoTek Project No 3901-CR

Figure 1
**Site Location
and Topography
Map**





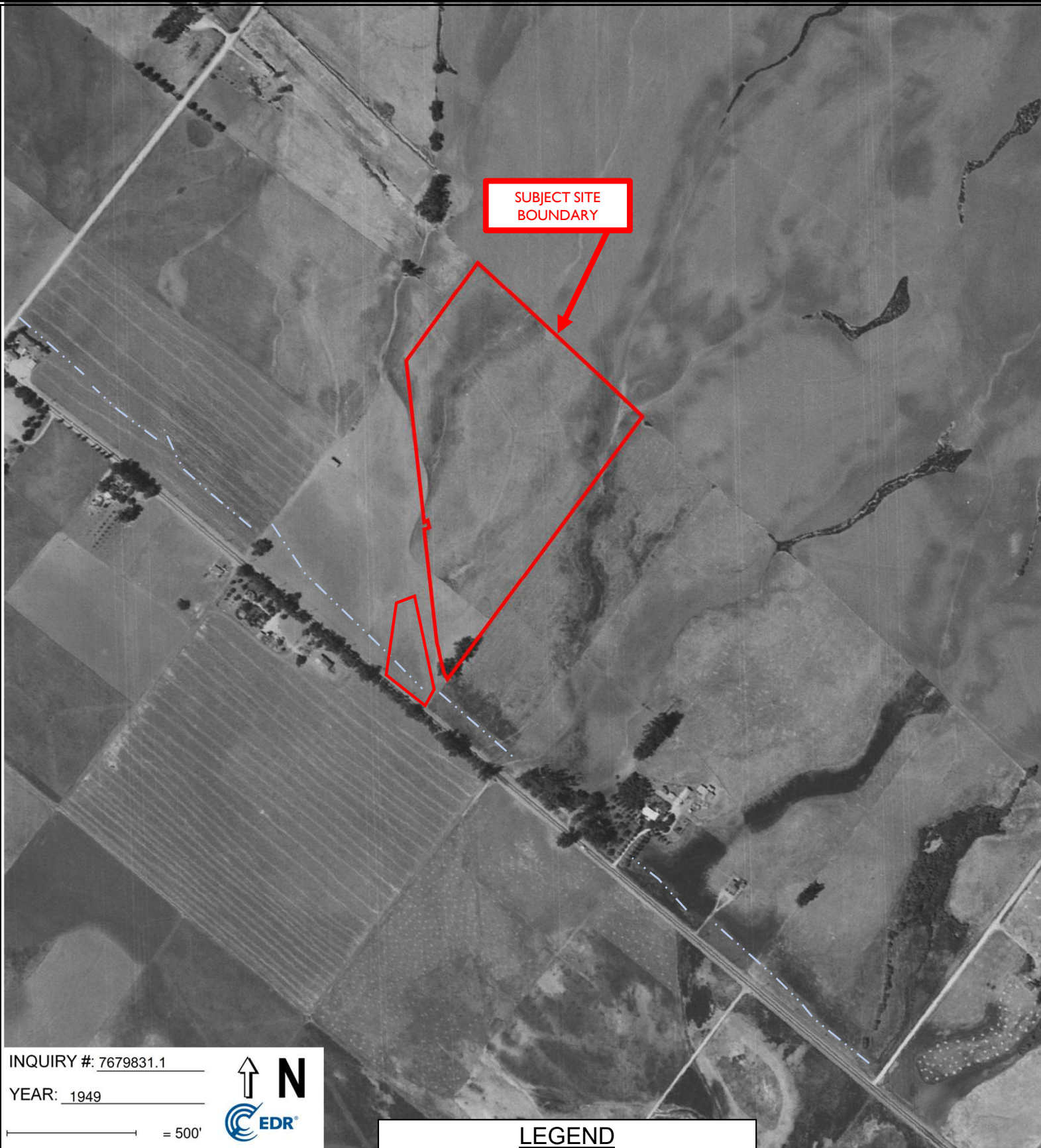
Architerra Design Group
 Proposed Wildomar 20-Acre Nature Park
 Palomar Street and South Pasadena Street
 City of Wildomar, Riverside County, California

Project No. 3901-CR

Scale As Shown

Figure 3
 California Geological Survey
 Earthquake Zones of
 Required Investigation Map





Architerra Design Group
Proposed Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
City of Wildomar, Riverside County, California

Project No. 390I-CR



Figure 4
Lineament
Air Photo
Assessment Map



APPENDIX A

LOGS OF EXPLORATORY BORINGS

**Geotechnical Evaluation
Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
City of Wildomar, Riverside County, California
Project No. 3901-CR**



A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)

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

The Standard Penetration Test (SPT) Sampler

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

Bulk Samples (Large)

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

Bulk Samples (Small)

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

B - BORING LOG LEGEND

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

SOILS

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

GEOLOGIC

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'2.31"N, 117°16'1.79"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,252 feet amsl

LOGGED BY: JC
OPERATOR: Victor
RIG TYPE: GT 8 Track
DATE: 5/30/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-1	Laboratory Testing			
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others	
					MATERIAL DESCRIPTION AND COMMENTS				
0					Disturbed Soil:			Collapse	
		7 15	R1	CL	F sandy CLAY, olive gray, moist, very stiff, oxidation staining				
		20	R2	SC	Alluvium: Clayey f-m SAND, dark brown, moist medium dense	13.5	110.1		
		6 8 11					12.4		122.9
5		9 10 14			R3		23.5		108.1
		6 10 15	R4	CL	F-m sandy CLAY, dark brown, moist, very stiff	16.0	117.3		
10		7 10 16	R5		F-c sandy CLAY, dark brown, moist, very stiff	10.9	124.4		
15		24 50/4"	R6		Pauba Formation: Clayey/silty SANDSTONE, brown, moist				
					BORING TERMINATED AT 16.0 FEET				
					No groundwater encountered Boring backfilled with soil cuttings				
20									
25									
30									

LEGEND	Sample type:	---Ring	---SPT	---Small Bulk	---Large Bulk	---No Recovery	---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

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'2.10"N, 117°16'1.53"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,251 feet amsl

LOGGED BY: JC
OPERATOR: Victor
RIG TYPE: GT 8 Track
DATE: 5/30/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-2	Laboratory Testing		
	Sample Type	Blows / 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
0				SM/ML	Disturbed Soil: Silty f-c SAND to f-c sandy SILT, light brown, slightly moist			MD, Ei, SH Expansion Index = 23
		35 13 14	R1	SC	Alluvium: Clayey f-c SAND, brown, moist, medium dense, trace silt	9.4	118.9	
5		6 8 14	R2	SM	Silty f-c SAND, brown, moist, medium dense, trace clay	6.3	114.7	SH
		8 10 14	R3	CL	F-m sandy CLAY, dark brown, moist, very stiff, trace silt	16.2	118.9	
		11 17 23	R4		F-c sandy CLAY, dark brown, moist, very stiff	12.0	122.8	
10		7 17 23	R5			10.9	123.2	
15		14 16 15	R6		Pauba Formation: Clayey SILTSTONE, olive gray, moist	23.8	103.7	SH
20		11 17 24	R7		Trace oxidation staining, trace caliche stringers	20.9	107.3	
25		5 8 11	S1		Some caliche			
30		5 14 38	S2		Clayey SILTSTONE, brown, moist, hard, trace oxidation & mottling stains	19.1		

LEGEND	Sample type:	 ---Ring	 ---SPT	 ---Small Bulk	 ---Large Bulk	 ---No Recovery	 ---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

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'2.10"N, 117°16'1.53"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,251 feet amsl

LOGGED BY: JC
OPERATOR: Victor
RIG TYPE: GT 8 Track
DATE: 5/30/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-2 (continued)	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
35		7 14 32	S3		Clayey/silty f-m SANDSTONE, dark brown, moist, oxidation staining			
40		7 10 18	S4			14.2		
45		4 11 12	S5		F sandy/clayey SILTSTONE, dark brown, moist			
50		7 10 16	S6		Becomes dark grey-brown	22.8		
					BORING TERMINATED AT 51.5 FEET No groundwater encountered Boring backfilled with soil cuttings			

LEGEND

Sample type:

---Ring

---SPT

---Small Bulk

---Large Bulk

---No Recovery

---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

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'2.22"N, 117°16'0.57"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,252 feet amsl

LOGGED BY: JC
OPERATOR: Victor
RIG TYPE: GT 8 Track
DATE: 5/30/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-3	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
0					Disturbed Soil:			
		8 14 11	R1	ML	F-m sandy SILT, brown, moist, very stiff	4.5	118.7	SH
		8 7 8	R2	SM	Alluvium: Silty f-c SAND, brown, moist, medium dense, trace clay	8.5	123.1	
5		6 9 7	R3			7.6	117.0	
		16 30 50/6"	R4	SC/SM	Clayey/silty f-c SAND, brown, moist, very dense			
					Pauba Formation: Clayey/silty f-c SANDSTONE, brown, moist	9.5	122.7	
10		20 33 36	R5		F sandy/clayey SILTSTONE, olive gray, moist	18.9	112.6	SH
15		6 15 45	R6		Some oxidation staining	14.0	103.5	
20		10 32 32	R7		Trace caliche	18.6	112.2	
25		19 32 44	S1		Clayey/silty f-m SANDSTONE, brown, moist, oxidation staining, caliche stringers	13.5		
30		6 11 17	S2		F-m sandy SILTSTONE, olive gray, moist, caliche			

LEGEND	Sample type:	 ---Ring	 ---SPT	 ---Small Bulk	 ---Large Bulk	 ---No Recovery	 ---Water Table
	Lab testing:	AL = Atterberg Limits SR = Sulfate/Resisitivity Test		EI = Expansion Index SH = Shear Test		SA = Sieve Analysis HC= Consolidation	

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'2.22"N, 117°16'0.57"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,252 feet amsl

LOGGED BY: JC
OPERATOR: Victor
RIG TYPE: GT 8 Track
DATE: 5/30/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-3 (continued)	Laboratory Testing		
	Sample Type	Blows / 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS							
35		15 10 16	S3		Clayey/silty f-c SANDSTONE, brown, moist	15.5		
40		6 11 18	S4		F sandy/clayey SILTSTONE, dark brown, moist, some caliche			
45		13 16 21	S5		F-m sandy/clayey SILTSTONE, dark brown, moist	15.3		
50		7 11 16	S6		Clayey/silty f-c SANDSTONE, dark brown, moist			
					BORING TERMINATED AT 51.5 FEET No groundwater encountered Boring backfilled with soil cuttings			

LEGEND	Sample type:	 ---Ring	 ---SPT	 ---Small Bulk	 ---Large Bulk	 ---No Recovery	 ---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

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'5.89"N, 117°15'58.45"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,260 feet amsl

LOGGED BY: JC
OPERATOR: Victor
RIG TYPE: GT 8 Track
DATE: 5/30/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-4	Laboratory Testing		
	Sample Type	Blows / 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
0				SM	Disturbed Soil: Silty f-c SAND, light brown, slightly moist			
					Alluvium:			
		13	R1	SM	Silty f-c SAND, brown, moist	4.3	115.2	
		50/6"			Pauba Formation:			
					Silty f-c SANDSTONE, brown, moist			
		11	R2		Clayey/silty f-c SANDSTONE, brown, moist	14.6	121.1	
		50/6"						
5		15	R3			7.9	122.3	
		50/5"						
10		7	R4		F-c sandy/clayey SILTSTONE, olive gray, moist			
		23						
		34						
					BORING TERMINATED AT 11.5 FEET			
					No groundwater encountered			
					Boring backfilled with soil cuttings			

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'9.38"N, 117°16'0.53"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,262 feet amsl

LOGGED BY: JC
OPERATOR: Adrian
RIG TYPE: Simco
DATE: 6/11/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-5	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
0				SM	Disturbed Soil: Silty f-c SAND, light brown, slightly moist			
		13 29 29	R1	SM	Alluvium: Silty f-c SAND, brown, moist, dense, trace clay	3.8	124.8	
		23 30 40	R2	SC	Clayey f-m SAND, dark brown, moist, dense	3.4	129.8	
5		21 41 50/5"	R3		Pauba Formation: Clayey/silty f-c SANDSTONE, brown, moist	5.4	134.8	
		45 50/5"	R4		F-c SANDSTONE, light brown, moist	3.7	115.0	
10		21 30 50	R5		Clayey/silty f-m SANDSTONE, olive grey, moist			
		42 50/3"	R6			10.6	122.4	
					BORING TERMINATED AT 16.0 FEET No groundwater encountered Boring backfilled with soil cuttings			
20								
25								
30								

LEGEND	Sample type:	 ---Ring	 ---SPT	 ---Small Bulk	 ---Large Bulk	 ---No Recovery	 ---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

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'10.41"N, 117°15'55.72"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,276 feet amsl

LOGGED BY: JC
OPERATOR: Adrian
RIG TYPE: Simco
DATE: 6/11/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-6	Laboratory Testing		
	Sample Type	Blows / 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
0				SM	Disturbed Soil: Silty f-c SAND, light brown, slightly moist			

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'15.58"N, 117°15'59.05"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,252 feet amsl

LOGGED BY: JC
OPERATOR: Adrian
RIG TYPE: Simco
DATE: 6/11/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-7	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS			
0					Disturbed Soil:			
		7	R1	SC/SM	Silty/clayey f-m SAND, brown, slightly moist			
		15						
		20				11.8	120.0	
		21	R2	SC	Alluvium: Clayey f-m SAND, dark brown, moist medium dense			
		24						
		25			Clayey f-c SAND, brown, moist, dense	9.4	127.9	
5		23	R3		Clayey f-m SAND, dark brown, moist, very dense			
		36						
		46				9.1	126.0	
		37	R4		Pauba Formation: F sandy/clayey SILTSTONE, olive grey, moist	9.1	114.7	
		50/5"						
		50/6"	R5		F-m sandy/clayey SILTSTONE, olive-grey, moist			
10								
		50/5"	R6		Silty f SANDSTONE, light brown, moist, trace oxidation staining	6.9	120.3	
15								
		32	R7		Silty f-m SANDSTONE, light brown, moist			
		50/3"						
20					BORING TERMINATED AT 19.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND	Sample type:	 ---Ring	 ---SPT	 ---Small Bulk	 ---Large Bulk	 ---No Recovery	 ---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

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Architerra Design Group
PROJECT NAME: Wildomar Nature Park
PROJECT NO.: 3901-CR
COORDINATES: 33°36'15.59"N, 117°16'0.18"W

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"
ELEVATION: 1,280 feet amsl

LOGGED BY: JC
OPERATOR: Adrian
RIG TYPE: Simco
DATE: 6/11/2024

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-8	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				SC	Disturbed Soil: Clayey f-c SAND, light brown, slightly moist			R-Value = 12
		11 18 26	R1	SC/CL	Alluvium: Clayey/silty f-c SAND to f-c sandy/silty CLAY, brown, moist, medium dense			
						6.2	127.4	
5		22 38 41	R2	SC	Clayey f-c SAND, dark brown, moist, very dense	10.9	113.1	
		19 31 44	R3		Clayey f-c SAND, brown, moist, dense	11.6	115.9	
		32 28 44	R4		Pauba Formation: F-m sandy/clayey SILTSTONE, olive-grey, moist	19.7	109.1	
10		16 25 29	R5		F-m sandy/clayey SILTSTONE, olive-grey, moist, some oxidation staining			
15		31 35 50/5"	R6		F sandy/clayey SILTSTONE, olive-grey, moist, trace caliche	23.8	111.7	
BORING TERMINATED AT 16.5 FEET								
No groundwater encountered Boring backfilled with soil cuttings								
20								
25								
30								

LEGEND	Sample type:	 ---Ring	 ---SPT	 ---Small Bulk	 ---Large Bulk	 ---No Recovery	 ---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

LEGEND

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

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

APPENDIX B

LABORATORY TEST RESULTS

**Geotechnical Evaluation
Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
City of Wildomar, Riverside County, California
Project No. 3901-CR**



SUMMARY OF LABORATORY TESTING

Classification

Soil and 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 Borings in Appendix A.

Collapse Test

Collapse tests were performed on selected samples of the site materials in general accordance with ASTM D 5333 test procedures. The results of these tests are presented graphically in Appendix B.

Direct Shear

Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM D 3080 test procedures. The rate of deformation was approximately 0.035 inch per minute. The sample was sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The tests were performed on a soil sample remolded to approximately 90 percent of maximum dry density as determined by ASTM D 1557 test procedures and in-situ soil samples. 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
B-2	0-5	Sandy/Clayey Silt (ML) to Clayey Sand (SC)	23	Low

In-Situ Moisture and Density

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

Moisture-Density Relationship

Laboratory testing was performed on one (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.

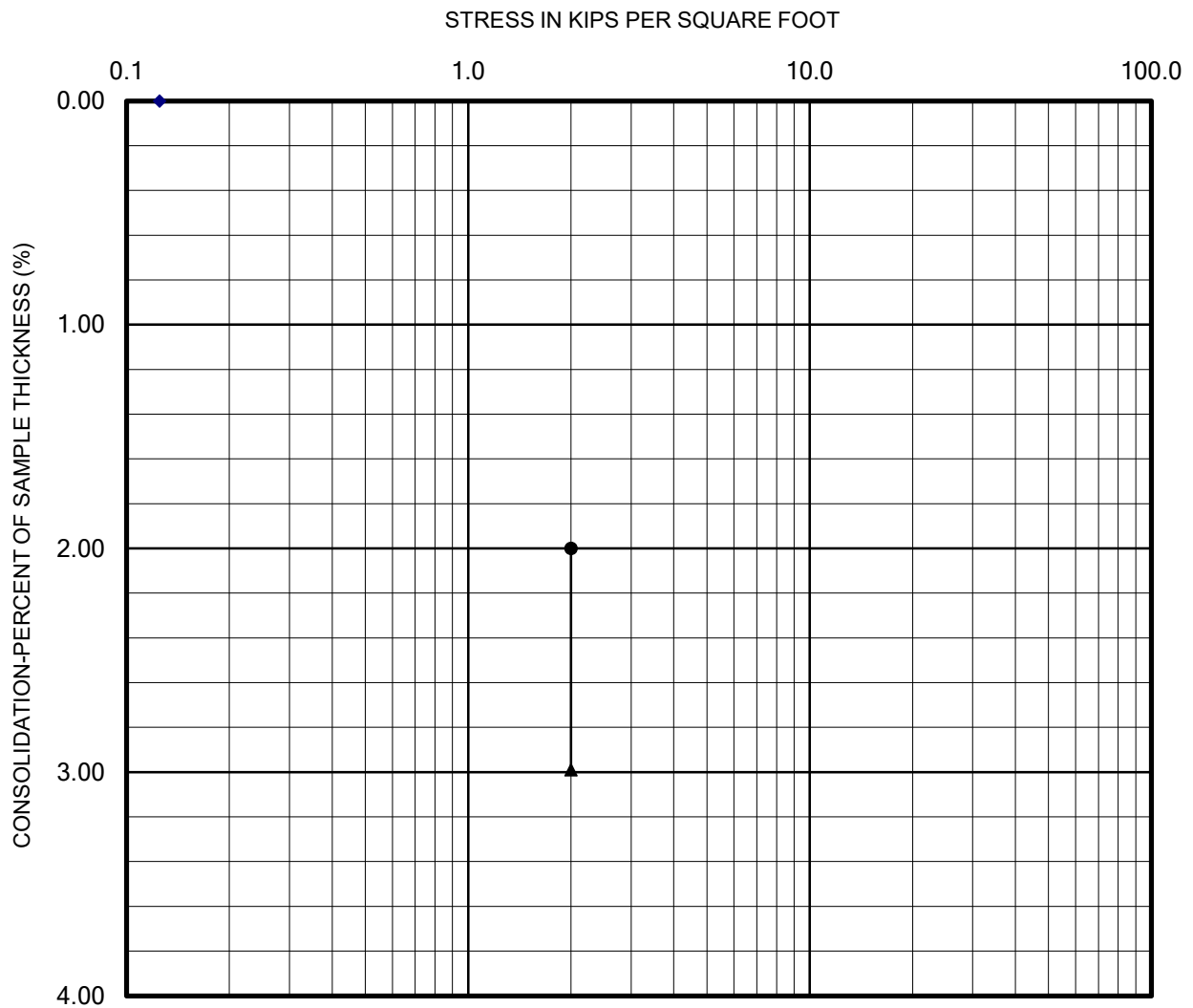
R-Value

Resistance “R”-Value testing was performed on one (1) disturbed bulk sample representative of the anticipated subgrade soils for paving areas of the project site. Testing was performed in accordance with ASTM D 2844. Test 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)
B-4	0-5	6.7	77.9	0.0096	5,226



- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



COLLAPSE REPORT

Sample: B-1 @ 3 Feet

**Architerra Design Group
Wildomar 20-Acre Park**

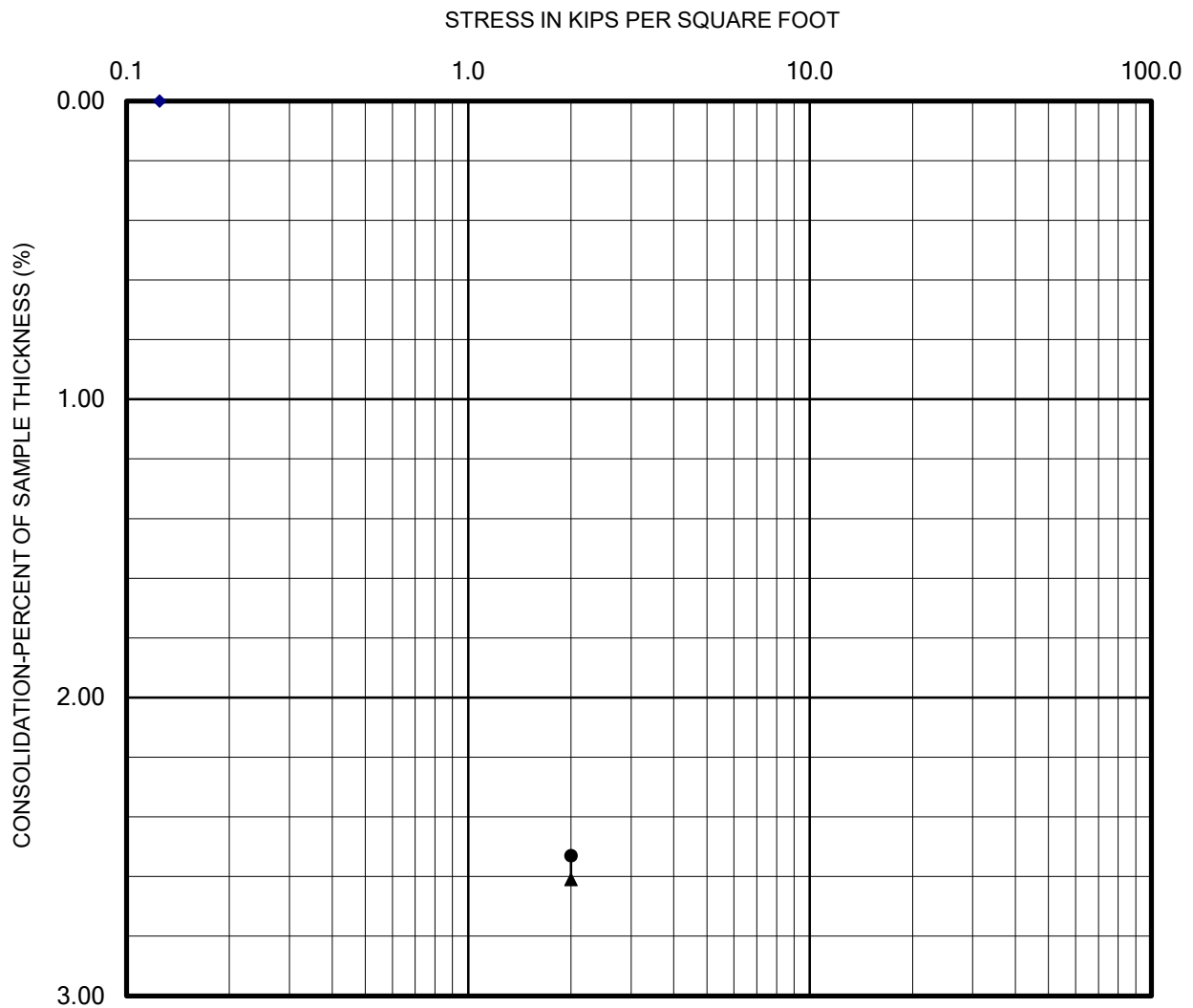
Plate B-1

CHECKED BY: EC

Lab: Corona

PROJECT NO.: 3901-CR

Date: 6/5/2024



- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



COLLAPSE REPORT

Sample: B-6 @ 4 Feet

**Architerra Design Group
Wildomar 20-Acre Park**

Plate B-2

CHECKED BY: EC

Lab: Corona

PROJECT NO.: 3901-CR

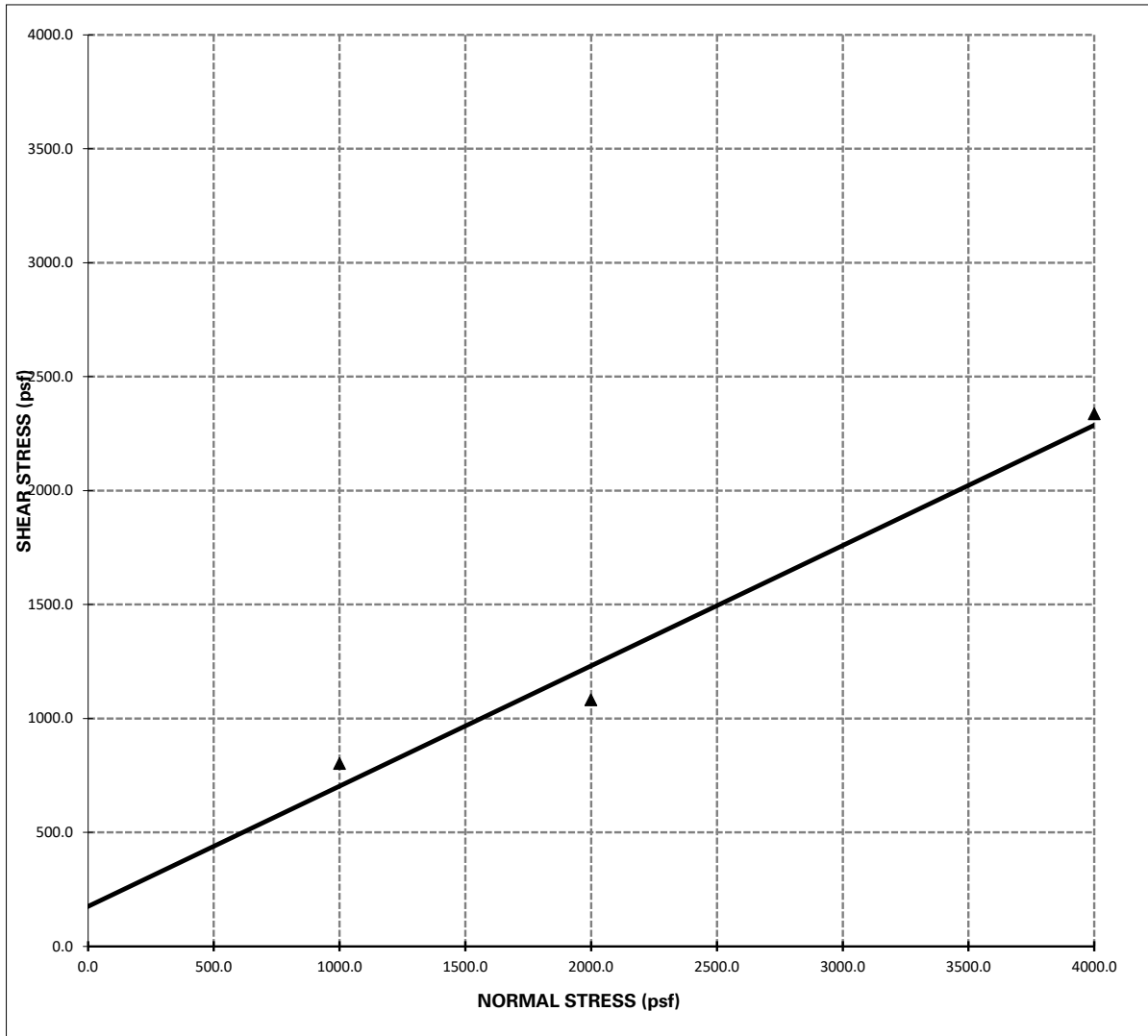
Date: 6/17/2024



DIRECT SHEAR TEST

Project Name: Wildomar 20-Acre Nature Park
Project Number: 3901-CR

Sample Location: B-2 @ 15 Feet
Date Tested: 6/6/2024



Shear Strength: $\Phi = 28^{\circ}$; $C = 175 \text{ psf}$

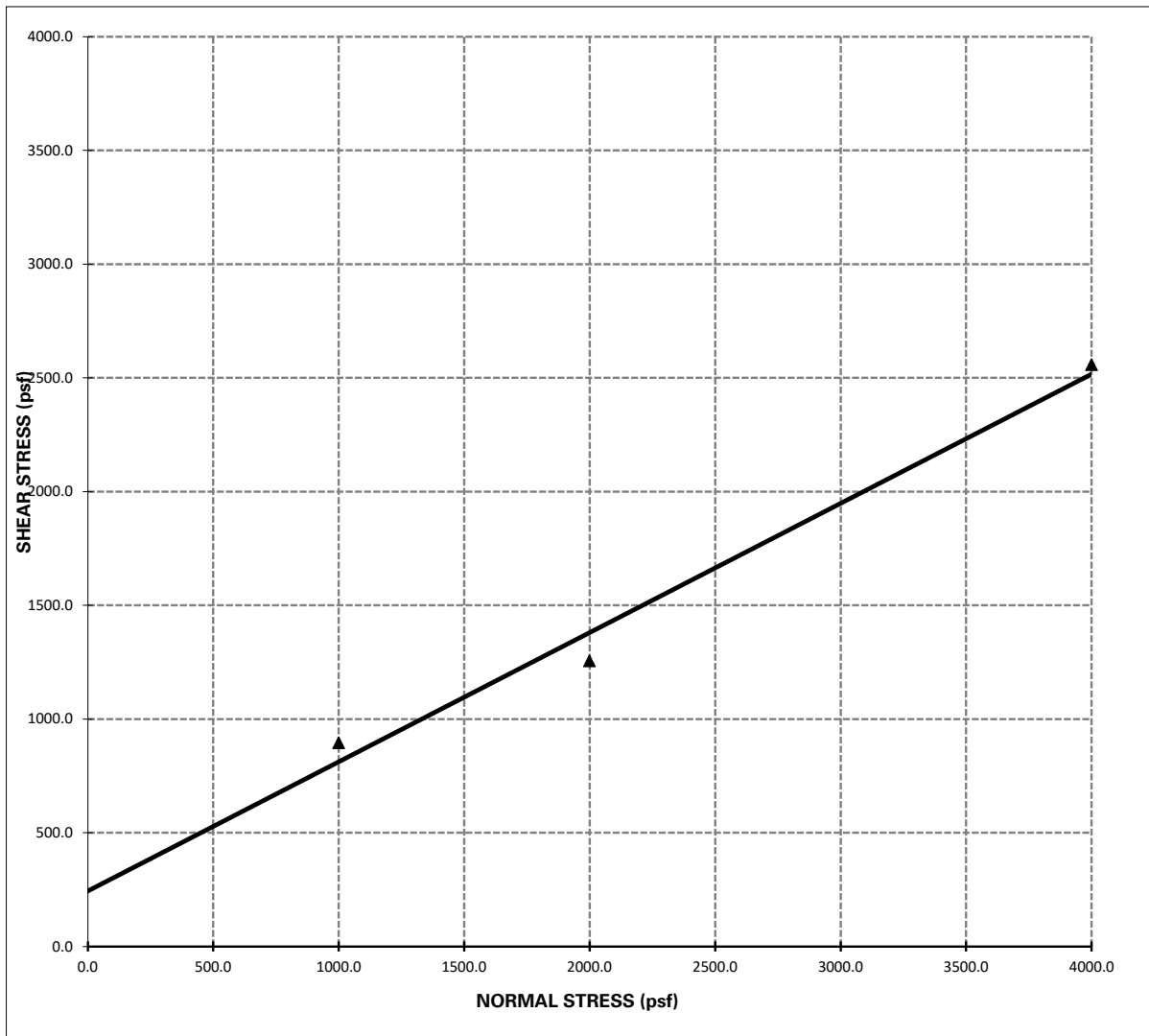
- Notes:**
- 1 - The soil specimens sheared were "undisturbed" ring samples.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.01 in/min.



DIRECT SHEAR TEST

Project Name: Wildomar 20-Acre Nature Park
Project Number: 3901-CR

Sample Location: B-2 @ 6 Feet
Date Tested: 6/6/2024



Shear Strength: $\Phi = 30^{\circ}$; **C = 244 psf**

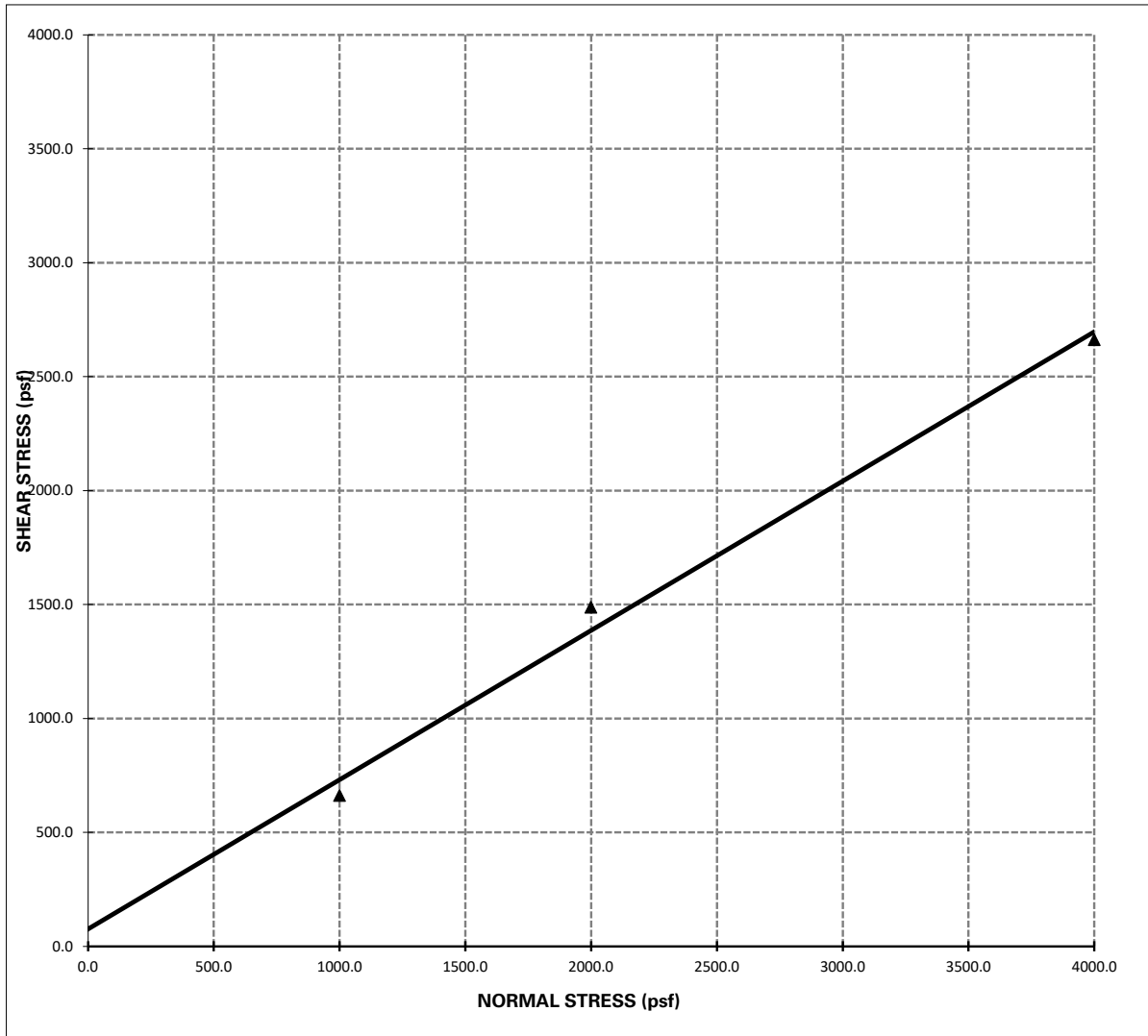
- Notes:**
- 1 - The soil specimens sheared were "undisturbed" ring samples.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.01 in/min.



DIRECT SHEAR TEST

Project Name: Wildomar 20-Acre Nature Park
Project Number: 3901-CR

Sample Location: B-3 @ 20 Feet
Date Tested: 6/3/2024



Shear Strength: $\Phi = 33^{\circ}$; **C = 75 psf**

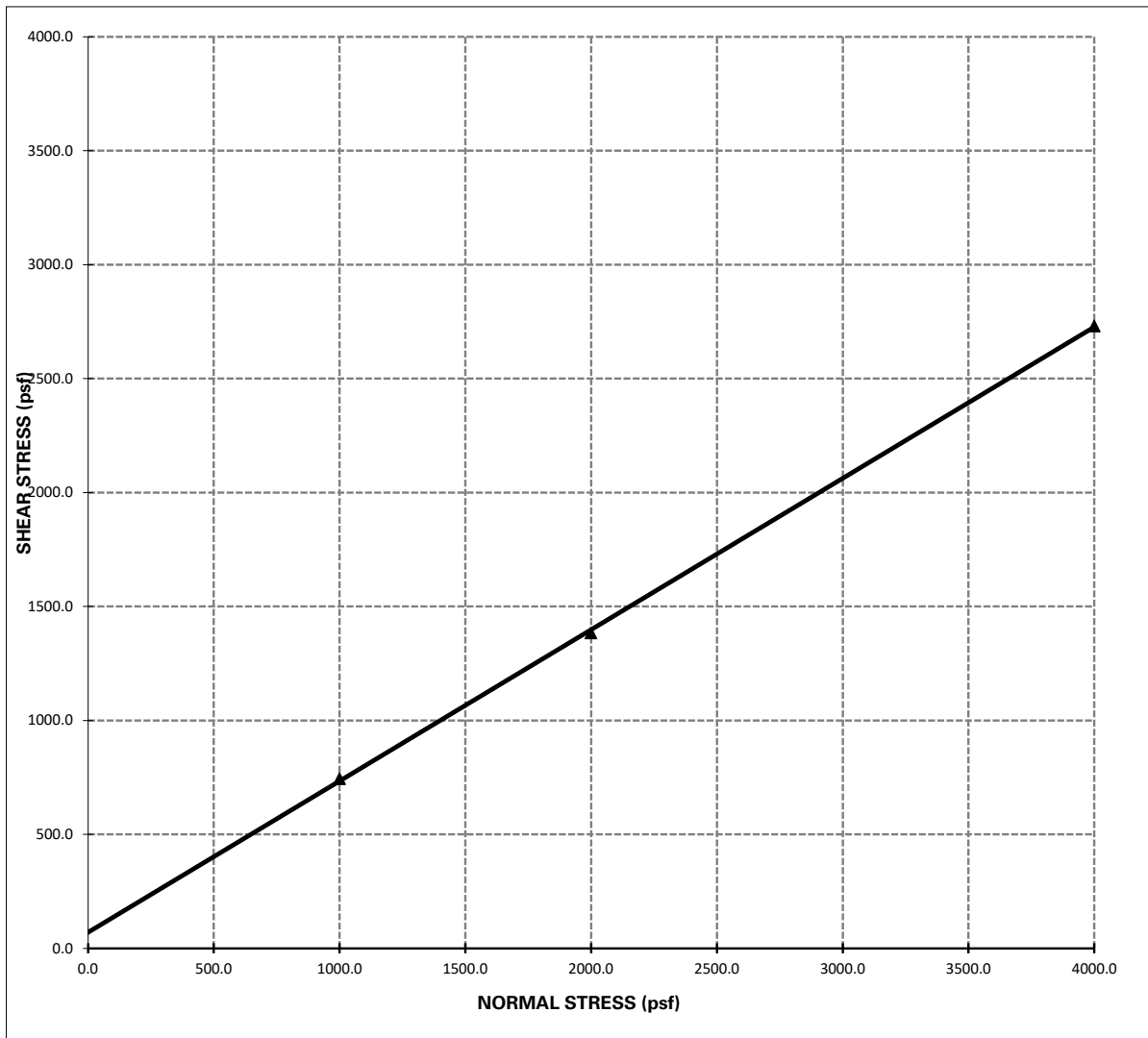
- Notes:**
- 1 - The soil specimens sheared were "undisturbed" ring samples.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.01 in/min.



DIRECT SHEAR TEST

Project Name: Wildomar 20-Acre Nature Park
Project Number: 3901-CR

Sample Location: B-3 @ 3 Feet
Date Tested: 6/3/2024



Shear Strength: $\Phi = 34^{\circ}$; **C = 70 psf**

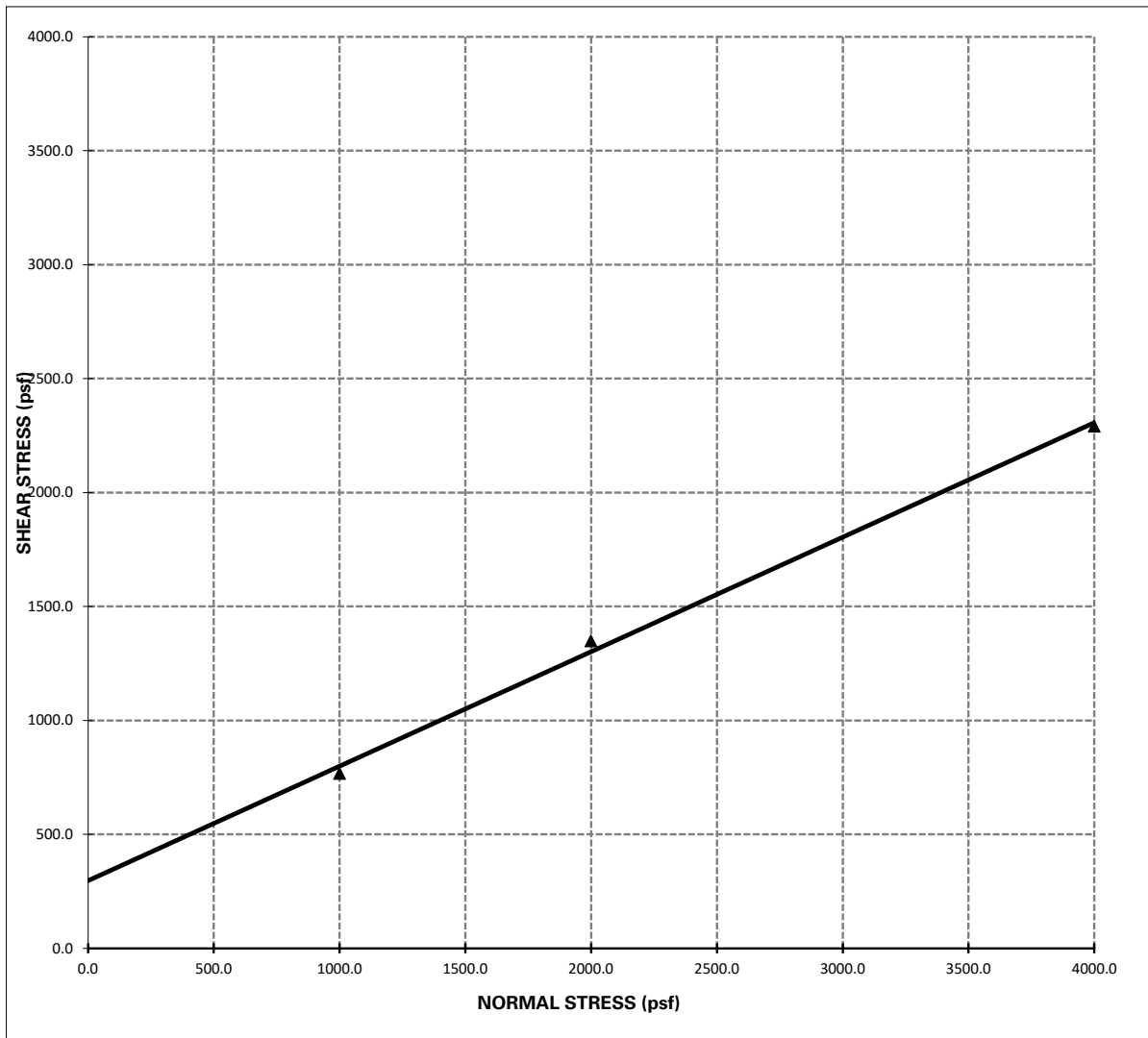
- Notes:**
- 1 - The soil specimens sheared were "undisturbed" ring samples.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.01 in/min.



DIRECT SHEAR TEST

Project Name: Wildomar 20-Acre Nature Park
Project Number: 3901-CR

Sample Location: B-2 @ 0-5 Feet
Date Tested: 6/7/2024



Shear Strength: $\Phi = 27^{\circ}$; **C = 296 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.

Report No: PTR:24-00107-S01

Proctor Report

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

CC:

Project: 3901-CR
Wildomar 20-Acre Nature Park, Wildomar

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID: 24-00107-S01

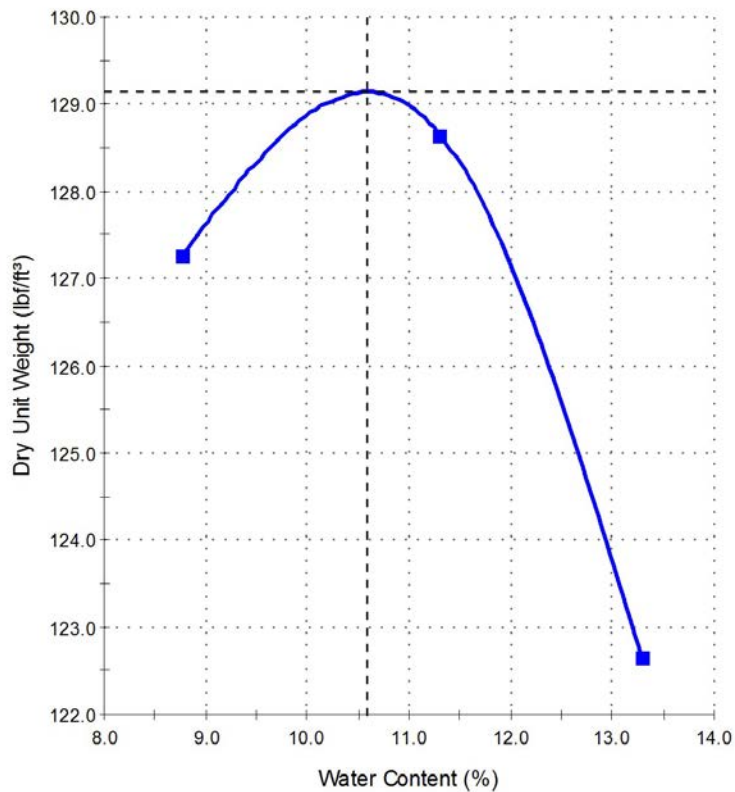
Date Sampled: 5/30/2024

Sampled By:

Material: Clayey Fine to Course SAND

Location: B-2 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lb/ft³): 129.1

Optimum Water Content (%): 10.6

Method: A

Preparation Method: Moist

Retained Sieve No 4 (4.75mm) (%): 2

Passing Sieve No 4 (4.75mm) (%): 98

Tested By: Eduardo Cuevas

Date Tested: 6/4/2024

Comments



EXPANSION INDEX TEST

(ASTM D4829)

Client: Architerra Design Group
Project Number: 3901-CR
Project Location: Wildomar 20-Acre Nature Park

Tested/ Checked By: RY Lab No Corona
Date Tested: 6/4/2024
Sample Source: B-2 @ 0-5 Feet
Sample Description:

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	767.4
Weight of ring (gm)	369.1
Net weight of sample (gm)	398.3
Wet Density, lb / ft ³ (C*0.3016)	120.1
Dry Density, lb / ft ³ (D/1.F)	109.2

SATURATION DETERMINATION

Moisture Content, %	10.0
Specific Gravity, assumed	2.70
Unit Wt. of Water @ 20 °C, (pcf)	62.4
% Saturation	49.7

READINGS

DATE	TIME	READING	
6/4/2024		0.1460	Initial
6/4/2024		0.1450	10 min/Dry
6/5/2024		0.1680	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
800.3	18.3

EXPANSION INDEX = 23

Report No: RV:24-00107-S02

R Value Report

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

CC:

Project: 3901-CR
Wildomar 20-Acre Nature Park, Wildomar

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID: 24-00107-S02

Date Sampled: 5/30/2024

Sampling Method:

Source:

Material: Sandy CLAY

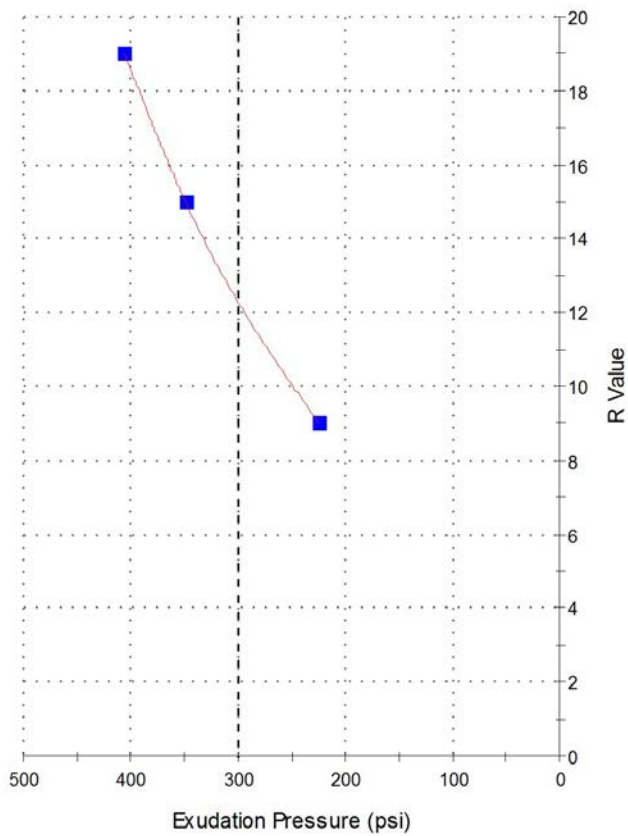
Specification:

Location: B-8 @ 0-5 Feet

Tested By: Eduardo Cuevas

Date Tested: 6/14/2024

R Value



Test Results

ASTM D 2844

R Value at 300 psi Exudation: 12

Specimen Results

Moisture Content (%)	11.4	13.2	14.1
Dry Density (lb/ft³)	113.4	108.1	107.3
Exudation Pressure (psi)	349	224	406
R Value	15	9	19
Expansion Pressure (psi)	0.9	0.7	1.0

Comments



Results Only Soil Testing for Wildomar 20 acre Nature Park

June 13, 2024

Prepared for:

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

**Project X Job#: S240612F
Client Job or PO#: 3901-CR Architerra Design**

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: Wildomar 20 acre Nature Park

Client Job Number: 3901-CR Architerra Design

Project X Job Number: S240612F

June 13, 2024

Method	ASTM D4327	ASTM D4327	ASTM G187	ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates SO ₄ ²⁻	Chlorides Cl ⁻	Resistivity As Rec'd Minimum	pH	Redox	Sulfide S ²⁻	Nitrate NO ₃ ⁻	Ammonium NH ₄ ⁺	Lithium Li ⁺	Sodium Na ⁺	Potassium K ⁺	Magnesium Mg ²⁺	Calcium Ca ²⁺	Fluoride F ₂ ²⁻	Phosphate PO ₄ ³⁻
	(ft)	(mg/kg) (wt%)	(mg/kg) (wt%)	(Ω-cm) (Ω-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-4	0-5	95.9 0.0096	77.9 0.0078	154,100 5,226	6.7	133	2.0	4.2	10.4	0.01	81.5	7.2	39.0	130.7	3.7	8.9

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
HISTORIC AERIAL PHOTOGRAPHS

**Geotechnical Evaluation
Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
City of Wildomar, Riverside County, California
Project No. 3901-CR**



Wildomar 20-Acre Nature Park

Not Reported

WILDOMAR, CA 92595

Inquiry Number: 7679831.1

June 12, 2024

The EDR Aerial Photo Decade Package



6 Armstrong Road, 4th floor
Shelton, CT 06484
Toll Free: 800.352.0050
www.edrnet.com

EDR Aerial Photo Decade Package

06/12/24

Site Name:

Wildomar 20-Acre Nature Park
Not Reported
WILDOMAR, CA 92595
EDR Inquiry # 7679831.1

Client Name:

Geotek
1548 North Maple Street
Corona, CA 92880
Contact: Kyle Richard Mchargue



Environmental Data Resources, Inc. (EDR) Aerial Photo Decade Package is a screening tool designed to assist environmental professionals in evaluating potential liability on a target property resulting from past activities. EDR's professional researchers provide digitally reproduced historical aerial photographs, and when available, provide one photo per decade.

Search Results:

<u>Year</u>	<u>Scale</u>	<u>Details</u>	<u>Source</u>
2020	1"=500'	Flight Year: 2020	USDA/NAIP
2016	1"=500'	Flight Year: 2016	USDA/NAIP
2012	1"=500'	Flight Year: 2012	USDA/NAIP
2009	1"=500'	Flight Year: 2009	USDA/NAIP
2005	1"=500'	Flight Year: 2005	USDA/NAIP
2002	1"=500'	Acquisition Date: January 01, 2002	USGS/DOQQ
1997	1"=500'	Acquisition Date: October 16, 1997	USGS/DOQQ
1994	1"=500'	Acquisition Date: January 01, 1994	USGS/DOQQ
1989	1"=500'	Flight Date: August 15, 1989	USDA
1985	1"=500'	Flight Date: February 24, 1985	USDA
1978	1"=500'	Flight Date: September 20, 1978	USDA
1967	1"=500'	Flight Date: May 15, 1967	USDA
1961	1"=500'	Flight Date: June 17, 1961	USDA
1953	1"=500'	Flight Date: September 22, 1953	USDA
1949	1"=500'	Flight Date: May 06, 1949	USDA
1938	1"=500'	Flight Date: June 14, 1938	USDA

When delivered electronically by EDR, the aerial photo images included with this report are for ONE TIME USE ONLY. Further reproduction of these aerial photo images is prohibited without permission from EDR. For more information contact your EDR Account Executive.

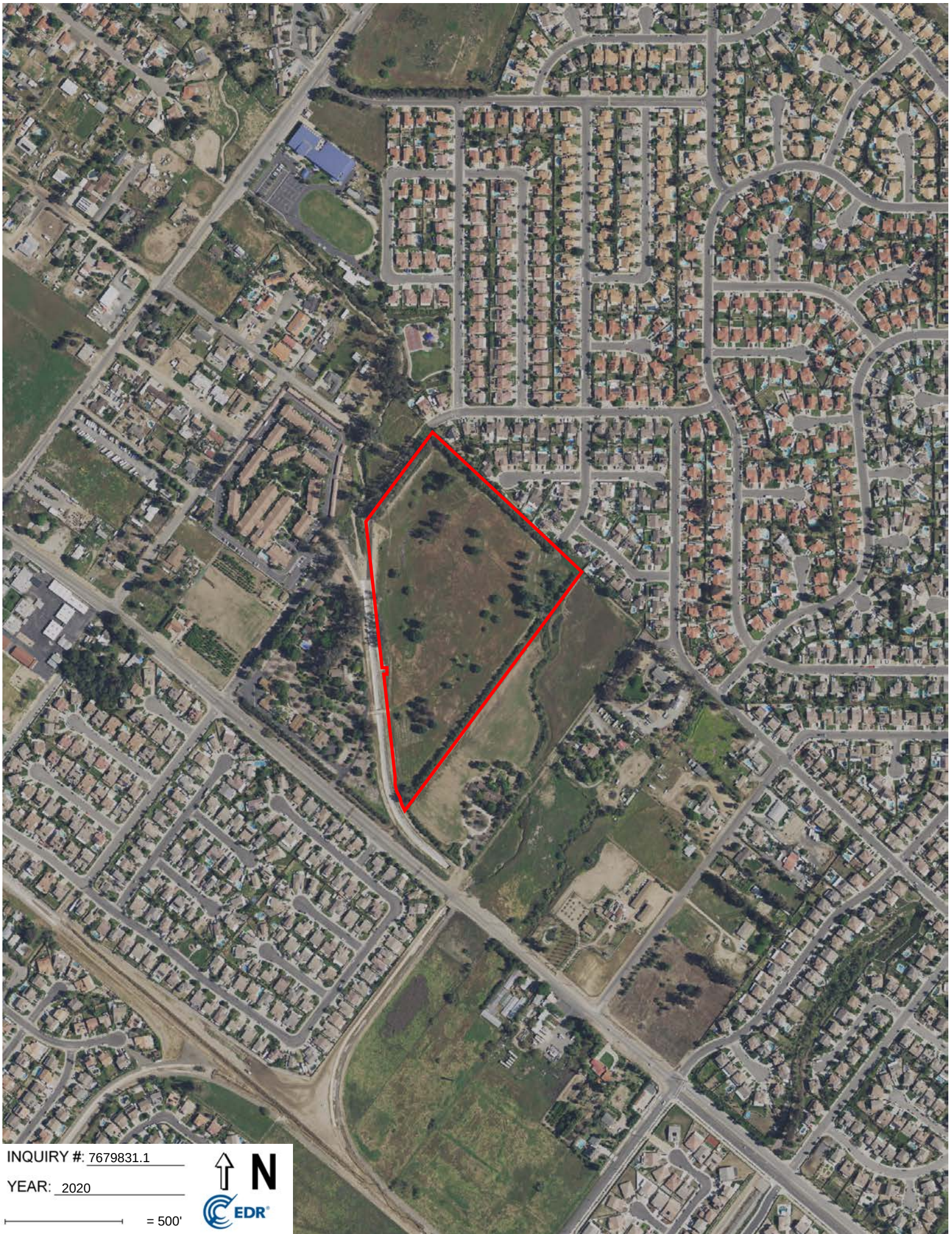
Disclaimer - Copyright and Trademark Notice

This Report contains certain information obtained from a variety of public and other sources reasonably available to Environmental Data Resources, LLC. It cannot be concluded from this Report that coverage information for the target and surrounding properties does not exist from other sources. This Report is provided on an "AS IS", "AS AVAILABLE" basis. NO WARRANTY EXPRESS OR IMPLIED IS MADE WHATSOEVER IN CONNECTION WITH THIS REPORT.

ENVIRONMENTAL DATA RESOURCES, LLC AND ITS SUBSIDIARIES, AFFILIATES AND THIRD PARTY SUPPLIERS DISCLAIM ALL WARRANTIES, OF ANY KIND OR NATURE, EXPRESS OR IMPLIED, ARISING OUT OF OR RELATED TO THIS REPORT OR ANY OF THE DATA AND INFORMATION PROVIDED IN THIS REPORT, INCLUDING WITHOUT LIMITATION, ANY WARRANTIES REGARDING ACCURACY, QUALITY, CORRECTNESS, COMPLETENESS, COMPREHENSIVENESS, SUITABILITY, MERCHANTABILITY, FITNESS FOR A PARTICULAR PURPOSE, TITLE, NON-INFRINGEMENT, MISAPPROPRIATION, OR OTHERWISE. ALL RISK IS ASSUMED BY THE USER. IN NO EVENT SHALL ENVIRONMENTAL DATA RESOURCES, LLC OR ITS SUBSIDIARIES, AFFILIATES OR THIRD PARTY SUPPLIERS BE LIABLE TO ANYONE FOR ANY DIRECT, INCIDENTAL, INDIRECT, SPECIAL, CONSEQUENTIAL OR OTHER DAMAGES OF ANY TYPE OR KIND (INCLUDING BUT NOT LIMITED TO LOSS OF PROFITS, LOSS OF USE, OR LOSS OF DATA), ARISING OUT OF OR IN ANY WAY CONNECTED WITH THIS REPORT OR ANY OF THE DATA AND INFORMATION PROVIDED IN THIS REPORT. Any analyses, estimates, ratings, environmental risk levels, or risk codes provided in this Report are provided for illustrative purposes only, and are not intended to provide, nor should they be interpreted as providing any facts regarding, or prediction or forecast of, any environmental risk for any property. Only an assessment performed by a qualified environmental professional can provide findings, opinions or conclusions regarding the environmental risk or conditions in, on or at any property.

Copyright 2024 by Environmental Data Resources, Inc. All rights reserved. Reproduction in any media or format, in whole or in part, of any report or map of Environmental Data Resources, Inc., or its affiliates, is prohibited without prior written permission.

EDR and its logos (including Sanborn and Sanborn Map) are trademarks of Environmental Data Resources, LLC or its affiliates. All other trademarks used herein are the property of their respective owners.



INQUIRY #: 7679831.1

YEAR: 2020

— = 500'





INQUIRY #: 7679831.1

YEAR: 2016

— = 500'





INQUIRY #: 7679831.1

YEAR: 2012

— = 500'





INQUIRY #: 7679831.1

YEAR: 2009

— = 500'





INQUIRY #: 7679831.1

YEAR: 2005

— = 500'





INQUIRY #: 7679831.1

YEAR: 2002

— = 500'





INQUIRY #: 7679831.1

YEAR: 1997

— = 500'





INQUIRY #: 7679831.1

YEAR: 1994

— = 500'





INQUIRY #: 7679831.1

YEAR: 1989

— = 500'





INQUIRY #: 7679831.1

YEAR: 1985

500'





INQUIRY #: 7679831.1

YEAR: 1978

— = 500'



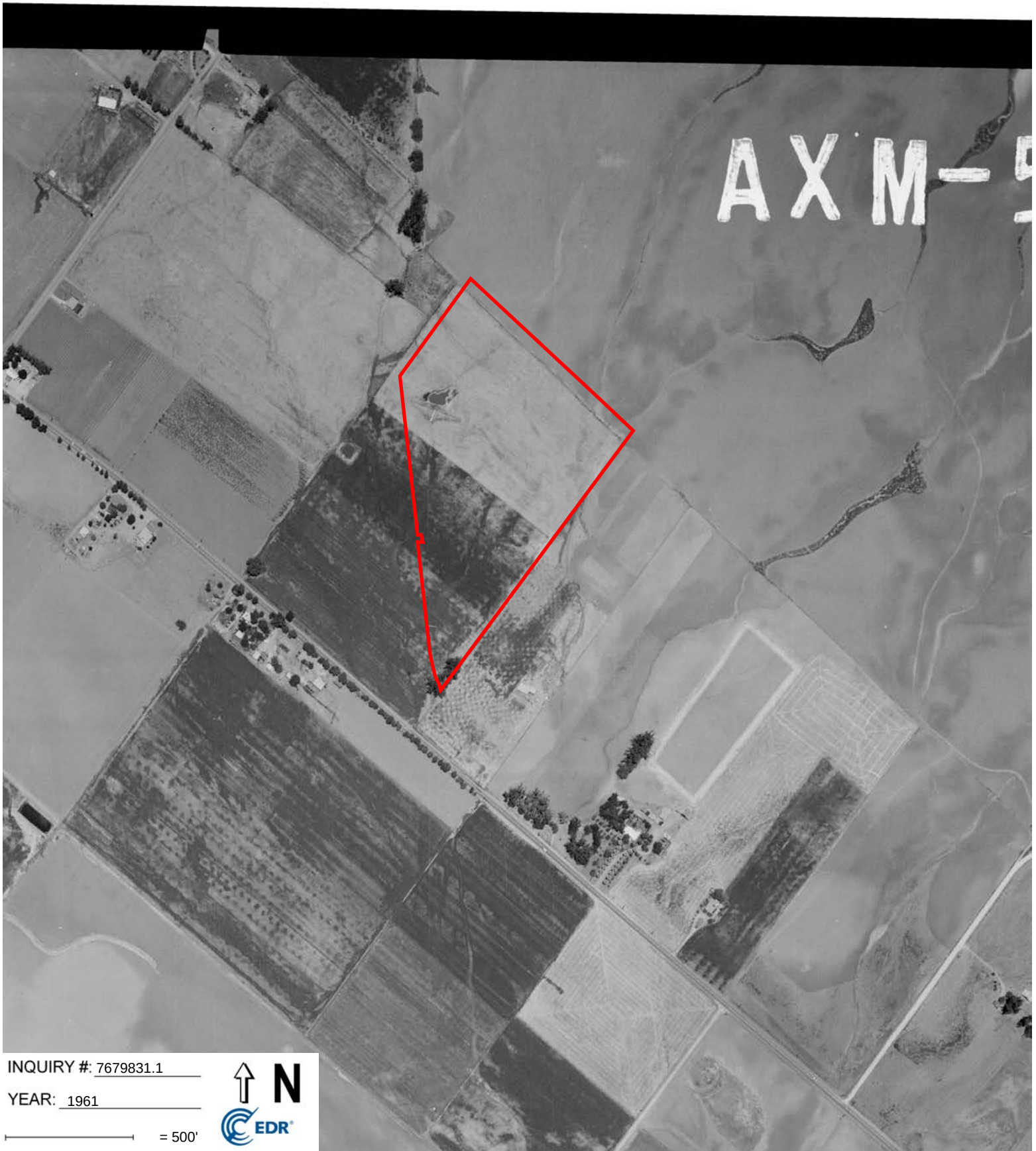


INQUIRY #: 7679831.1

YEAR: 1967

— = 500'





INQUIRY #: 7679831.1

YEAR: 1961

— = 500'



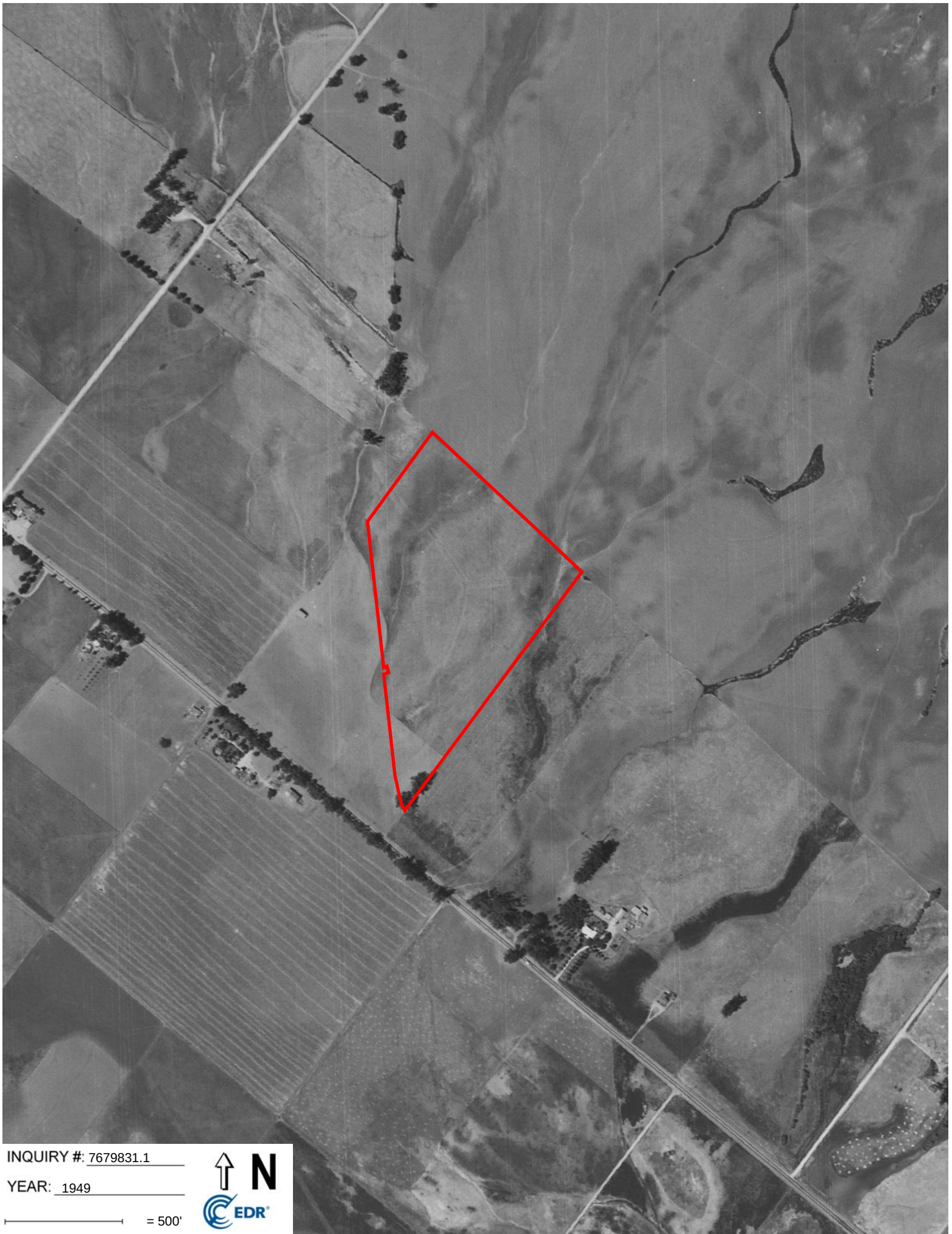


INQUIRY #: 7679831.1

YEAR: 1953

— = 500'





INQUIRY #: 7679831.1

YEAR: 1949

— = 500'





INQUIRY #: 7679831.1

YEAR: 1938

— = 500'



APPENDIX D
GENERAL EARTHWORK GRADING GUIDELINES

**Geotechnical Evaluation
Wildomar 20-Acre Nature Park
Palomar Street and South Pasadena Street
City of Wildomar, Riverside County, California
Project No. 3901-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.

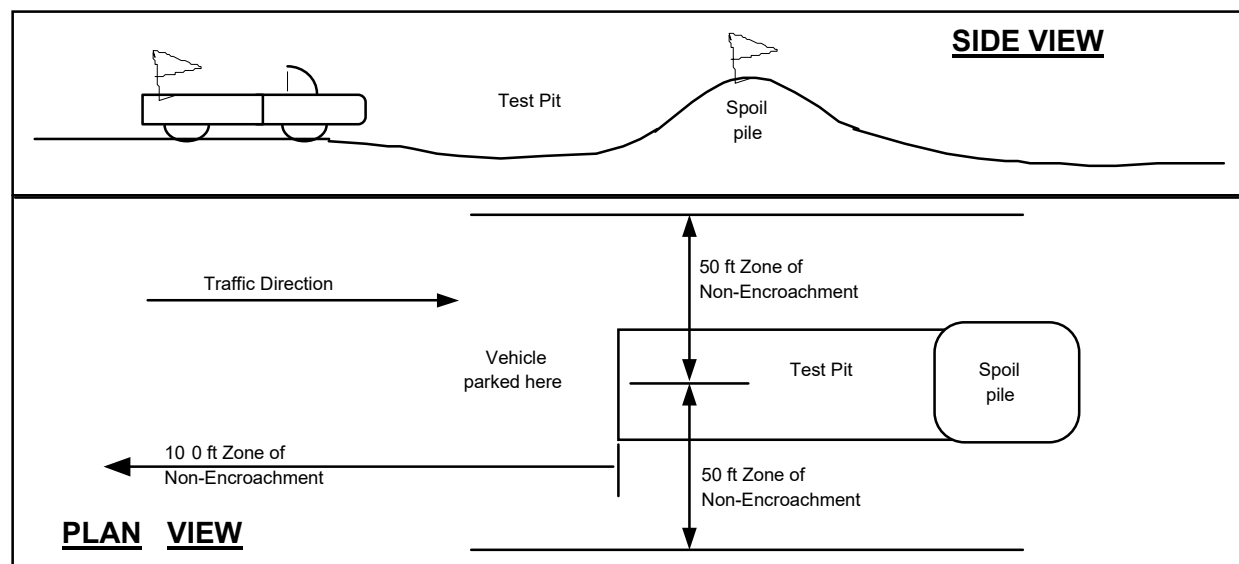
Test Pits Location, Orientation and Clearance

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.

This page intentionally left blank.