GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED SINGLE-FAMILY RESIDENTIAL DEVELOPMENT
ASSESSOR'S PARCEL NUMBER (APN) 290-190-005
23900 TEMESCAL CANYON ROAD
TEMESCAL VALLEY AREA OF RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

WARMINGTON RESIDENTIAL
3090 PULLMAN STREET
COSTA MESA, CALIFORNIA 92626

PREPARED BY

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

PROJECT NO. 4057-CR **SEPTEMBER 26, 2024**

GEOTEK

GEOTEK

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

September 26, 2024
Project No. 4057-CR

Warmington Residential
3090 Pullman Street
Costa Mesa, California 92626

Attention: Mr. Bret Ilich

Subject: **Geotechnical and Infiltration Evaluation**
Proposed Single-Family Residential Development
Assessor's Parcel Number (APN) 290-190-005
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California

Dear Mr. Ilich:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical and Infiltration Evaluation for the proposed single-family residential development to be constructed at 23900 Temescal Canyon Road, in the Temescal Valley area of Riverside County, California. This report presents the results of GeoTek's evaluation and discussion of findings.

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

GeoTek has reviewed the recommendations provided within the referenced reports prepared by Petra Geotechnical, Inc. (Petra, 2000a and 2000b) and generally concur with the conclusions and recommendations provided in their referenced reports. GeoTek accepts the previous data, findings and geotechnical recommendations provided in the referenced documents, except as amended in this report. GeoTek is now assuming the status of Geotechnical Engineer of Record for the proposed development.

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

Respectfully submitted,
GeoTek, Inc.

EDWARD H. LaMONT
No. 1892

BRUCE A. HICK
GE 2284

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

Bruce A. Hick

Bruce A. Hick
GE 3133, Exp. 12/31/24
Geotechnical Engineer

KYLE R. McHARGUE
2790

Kyle R. McHargue
CEG 2790, Exp. 02/28/26
Project Geologist

Anna M. Scott

Anna M. Scott
Project Geologist

Distribution: (1) Addressee via email

\\geotekfs1\Riverside\Projects\4051 to 4100\4057CR Warmington Residential 23900 Canyon Road\Geotechnical and Infiltration Evaluation\4057CR Geotechnical and Infiltration Evaluation Temescal Valley.doc

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ENCLOSURES

1. PURPOSE AND SCOPE OF SERVICES

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

- Research and review of available geologic and geotechnical data pertinent to the site,
- Review of the referenced *Geologic Fault Trenching and Liquefaction Study* performed for the project site by Petra Geotechnical, Inc. (Petra, 2000) and the referenced *Alquist Priolo Earthquake Fault Zoning Act Report Review* prepared by the County of Riverside Planning Department (County of Riverside, 2000),
- Perform a reconnaissance of the site,
- Site exploration consisting of the excavation, logging, and sampling of six (6) exploratory test borings extending to depths ranging from about 14.0 to 51.5 feet below grade,
- Excavation of an additional two (2) infiltration test borings in the central and southwesterly portions of the site in the currently planned locations of the on-site proposed stormwater management facilities, followed by infiltration testing in general accordance with the County of Riverside procedures within the two (2) borings at depths of about five (5) feet below existing grades each,
- Collection of bulk and in-situ samples of the onsite materials for laboratory testing,
- Laboratory testing of select samples obtained from the field investigation,
- Review and evaluation of site seismicity, and
- Compilation of this Geotechnical and Infiltration Evaluation report which presents GeoTek's findings, conclusions, and recommendations for the site development.

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

2. SITE DESCRIPTION, PROPOSED DEVELOPMENT AND PRIOR REPORT REVIEW

2.1 SITE DESCRIPTION

The project site is located at 23900 Temescal Canyon Road, adjacent to the southeast corner of Lawson Road and Temescal Canyon Road, in the Temescal Valley area of Riverside County, California (See Figure 1). The project site can also be identified as Riverside County Assessor's Parcel Number (APN) 290-190-005, which totals approximately 10-acres in size. The site currently consists of vacant, undeveloped land. The site is generally cleared of vegetation; however, a large tree is located in the north-central portion of the site and several large trees align the eastern property line, adjacent Temescal Canyon Road.

The site can be considered as having relatively flat terrain, with surface drainage generally directed to the east and north. Based on the USGS topographic map for the site vicinity, as well as Google Earth Pro, the elevation of the subject site is approximately 1,097 feet above mean sea level (AMSL) in the northwestern portion of the parcel and gently slopes down at an approximate 3 percent gradient to an elevation of about 1,062 feet amsl in the northeastern portion of the site.

The site is in an area largely characterized by vacant land and scattered residences. The site is bordered by Lawson Road along the north, followed by vacant land and a modular home; a golf course to the west; a drainage channel, followed by vacant land to the south; and by Temescal Canyon Road, followed by undeveloped land to the east.

2.2 PROPOSED DEVELOPMENT

Based upon review of a *Conceptual Density Study (Option 4)* prepared by KTGY Architecture and Planning, dated June 27, 2024, GeoTek understands the subject property is to be developed as a single-family residential development with 93 units, as well as associated open spaces, utilities, streets and typical hardscape and landscape improvements (See Figure 2). An on-site stormwater management system, currently unknown, is anticipated to manage on-site stormwater design flows.

GeoTek has assumed that the structures will be three-stories in height of wood-framed construction and that the buildings will be supported by conventional shallow spread footings which will most likely include a post-tension (PT) slab foundation system (as preferred by

Warmington Residential). Estimated maximum structural loads are 3.5 kips per linear foot for continuous foundations and 100 kips for columns. Sewage disposal is anticipated by a public sewer system. Once actual loads are known that information should be provided to GeoTek to determine if modifications to the recommendations presented in this report are warranted.

Based on the relatively flat existing topography, minimal cuts and fills of less than about five (5) feet are expected to be required to reach design grades. Interior and perimeter retaining walls are anticipated as part of the project development.

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

2.3 PRIOR REPORT REVIEW

Geologic Fault Trenching and Liquefaction Study (Petra Geotechnical, Inc., 2000)

Petra Geotechnical, Inc. (Petra) issued a *Geologic Fault Trenching and Liquefaction Study* report for the project on March 31, 2000.

Petra stated that the southwestern one-half of the site is located within an Alquist-Priolo Earthquake Fault Zone established for the Elsinore Fault. Petra stated that the report was prepared to satisfy the requirements of the California (Alquist-Priolo) Earthquake Fault Zoning Act. As such, Petra stated that the main objectives of the investigation were to assess whether active faults cross the site that could significantly impact development of the site and to determine whether liquefaction may pose a constraint regarding future development.

Petra stated that the following scope of work was performed:

- Review of stereoscopic aerial photographs dated 1948, 1962 and 1990.
- Review of geotechnical reports for nearby and adjacent properties which were also discussed with the County Geologist.
- The excavation of four (4) exploratory trenches with a heavy duty track excavator that totaled 780 lineal feet, to depths of nine (9) to 14 feet. The trenches were observed and logged under the direct supervision of an Engineering Geologist. The County

Geologist was reported to have observed the excavations as well. An additional trench was excavated and logged at the suggestion of the County Geologist.

- The excavation of two (2) hollow stem auger borings to depths of 46 to 79 feet.
- Laboratory testing included in-situ and maximum density, in-situ and optimum moisture content, grain size analysis, expansion index and consolidation.

Petra stated that they reviewed a number of geotechnical reports for adjacent properties, as well as published articles regarding nearby fault activity as part of their scope of work.

Petra stated that several sets of stereographic aerial photographs were reviewed to assess photo lineaments that might trend near or across the site. A well-defined, northwest-southeast trending photo lineament and two weak northeast-southwest trending photo lineaments were discussed in the report.

Petra stated that two (2) exploratory trenches were excavated for the study and were excavated from the west property line northeastward, across the width of the Earthquake Fault Zone which extends to near the center of the site. Trench 2 was reported to have been separated into three sections: 2A, 2B and 2C. Trenches 2A and 2C were first excavated and were later connected by Trench 2B to establish stratigraphic continuity.

Petra stated that the fault trenches were backfilled with no compactive efforts.

Petra reported that Boring B-1 was excavated near the northeast corner of the site and was advanced to a depth of 46 feet. Approximately four (4) feet of surficial soils were encountered overlying coarse-grained alluvial materials. Groundwater was encountered at a depth of 22 feet below the ground surface and bedrock of the Silverado Formation was encountered at a depth of 24 feet.

Petra reported that Boring B-2 was excavated near the westernmost portion of the site and was advanced to a total depth of 79 feet. Approximately six (6) feet of undocumented fill soils was encountered overlying topsoil and colluvial materials. Alluvial materials were first encountered at a depth of 15 feet below the ground surface and were encountered to the total depth of the boring. Groundwater was encountered 19 feet below the ground surface.

Petra reported that undocumented fill was also encountered in Trench 2A up to a maximum depth of eight (8) feet.

Petra provided the following conclusions and recommendations for the site based on their scope of work:

- No fault trace was found to cross the site. The parcel was reported to be underlain by well-defined, continuous, unbroke mid- to early-Holocene age alluvial deposits. These strata were traced in the trenches across the full width of the Earthquake Fault Zone established within the western half of the site.
- The active, Holocene-age, Glen Ivy North fault has been well-documented by others to lie at least 100 feet west of the west property line, within the Glen Ivy Marsh. Although it appeared that active faults lie well to the west of the property line, as a conservative measure, Petra recommended that no structure for human occupancy be permitted within 50 feet, measured horizontally, from the west property line.
- The building setback restriction recommended could be reduced or lifted if additional exploration beyond the west property line confirmed the absence of faults to the satisfaction of the Zoning Act. Offsite access for trenching would be required in that case.
- The above recommended setback zone, as well as the locations of the uncompacted exploratory trenches, should be recorded by the Project Civil Engineer.
- Petra stated that the report satisfied the California (Alquist-Priolo) Earthquake Fault Zoning Act. Petra concluded that the probability of surface fault rupture on the site was remote and the site was considered safe, relative to surface fault rupture.
- Petra stated that the near surface site soils generally between depths of 10 to 20 feet below the existing grade are highly susceptible to liquefaction during a design earthquake event at the nearby Elsinore-Glen Ivy fault. Petra further stated that the post-liquefaction total and maximum differential settlement are estimated to be on the order of 2 and 1 ¼ inches, respectively.

Alquist-Priolo Earthquake Fault Zoning Act Report Review (County of Riverside Transportation and Land Management Agency, 2000)

The County of Riverside Transportation and Land Management Agency (TLMA) issued an *Alquist-Priolo Earthquake Fault Zoning Act Report Review* letter for the project on August 1, 2000.

The County of Riverside TLMA stated that the report prepared by Petra (2000) was prepared in a state-of-the-art manner and satisfied the requirements of the Alquist-Priolo Earthquake Fault Zoning Act. Final approval of the report was provided by the County of Riverside.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

After completing an initial site reconnaissance, GeoTek excavated six (6) exploratory borings on September 5, 2024. The six (6) exploratory borings (Borings B-1 through B-6) were drilled with a hollow-stem drill rig to depths ranging from about 14.0 to 51.5 feet below the existing ground surface at the boring locations. The approximate locations of the GeoTek excavations are shown on the Exploration Location Map (Figure 2). A geologist from GeoTek logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory borings are included in Appendix A.

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

In Boring B-2, 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 Log of Boring B-2.

3.2 LABORATORY TESTING

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

Included in the laboratory testing were moisture-density determinations on relatively undisturbed samples. Optimum moisture content-maximum dry density relationships were established for typical soil types so that the relative compaction of the subsoils could be determined. Direct shear testing was conducted on remolded samples to estimate soil strength and lateral earth pressures. Collapse testing was performed on selected samples. Expansion index testing was performed on selected samples to evaluate the expansion potential of the on-site soils. Soil chemical testing comprised of pH, soluble sulfate, chloride and resistivity testing was conducted on selected samples. Soil classification tests consisted of gradation (percent passing No. 200 sieve) and Atterberg limits. The moisture-density, Atterberg Limits and gradation test data are presented on the Logs of Exploratory Borings in Appendix A. The maximum density, direct shear, collapse, expansion index, and chemical test data are presented in Appendix B.

3.3 INFILTRATION TESTING

In addition to the geotechnical exploratory borings, two (2) percolation test borings (Borings I-1 and I-2) were excavated in anticipated areas of the proposed storm water management facilities to depths of about five (5) feet below existing grades. Infiltration/percolation testing was conducted in these borings in general accordance with the requirements of the County of Riverside.

The percolation tests consisted of drilling an eight-inch diameter test hole to the desired depth and installing approximately two inches of gravel in the bottom of the hole. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within each boring. Water was then placed in the borings to presoak the holes and percolation testing was performed the following the pre-soak period. The percolation tests were then performed which consisted of adding water to each test hole and measuring the water drop over a 10-minute period. The water drop was recorded for a minimum of six (6) test intervals. Water was added to the test holes after each test interval. The field percolation rates were then converted to an infiltration rate using the Porchet Method. The converted infiltration rates are summarized in the table below.

Infiltration Boring	Depth (ft.)	Infiltration Rate* (in/hr)
I-1	5	8.95
I-2	5	12.57

*Porchet Method calculated unfactored infiltration rate.

The results of the conversions indicate infiltration rates at the test locations between 8.95 and 12.57 inches per hour, which indicate good infiltration rates. Copies of the percolation data sheets and the Porchet infiltration rate conversion calculations are presented in Appendix C. No factors of safety were applied to the rates provided. Over the lifetime of the infiltration areas, the infiltration rates may be affected by sediment build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate in designing the infiltration system.

It should be noted that the infiltration rates provided above were performed in relatively undisturbed on-site soils. Infiltration rates will vary and are mostly dependent on the underlying consistency of the site soil and relative density. Infiltration rates may be impacted by weight of equipment travelling over the soil, placement of engineered fill and other various factors. GeoTek assumes no responsibility or liability for the ultimate design or performance of the storm water facility.

Representatives of GeoTek should observe the soils at the bottom of all stormwater disposal facilities during construction/earthwork operations to confirm suitability and that the conditions exposed are as anticipated for the proposed stormwater disposal facilities.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province where it interacts with the Transverse Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends from the point of contact with the Transverse Ranges geomorphic province, southerly 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 mostly found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province, and the San Jacinto fault borders the province adjacent the Colorado Desert province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by young valley deposits (Morton, D.M. and Miller, F.K., 2006).

The site is partially located within an "*Alquist-Priolo*" fault hazard zone for the approximate western two-thirds 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 | Glen Ivy North Fault", as having a "Moderate/Very Low" liquefaction potential hazard and as "susceptible" with regards to subsidence.

No evidence of active faulting was identified by Petra (2000) within the limits of their Fault Trenches 1 and 2. Petra recommended a 50-foot structural setback from the western edge of the site. GeoTek concurs with the general findings, conclusions, and recommendations provided by Petra (2000).

4.2 EARTH MATERIALS

A brief description of the earth materials encountered in GeoTek's explorations is presented in the following sections.

4.2.1 Undocumented Fill Soils

Undocumented fill soils were encountered in all GeoTek's boring explorations to depths ranging from about four (4) to seven (7) feet below the existing ground surface. The undocumented fill soils encountered were observed to generally consist of sandy silts and silty sands (ML and SM soil types based on the Unified Soil Classification System). These undocumented fills soils may have been placed on the site following maintenance/clearing of the drainage channel along the southern edge of the site. These fill soils are considered uncompacted and unsuitable for support of structures or additional fill. Detailed logs of the subsurface conditions of the site are presented in Appendix A.

Petra (2000) excavated approximately 780 lineal feet of exploratory fault trench in total, to depths of about 9 to 14 feet below existing grades. These fault trenches were not backfilled with properly compacted engineered fill, and therefore the existing trench backfill materials are considered undocumented fill soils and will need to be removed and replaced with properly compacted engineered fill.

4.2.2 Alluvium

Alluvium was encountered in all GeoTek's boring explorations below the surficial undocumented fill soils to the maximum depths drilled of approximately 51.5 feet below the existing ground surface. The alluvial soils encountered were observed to generally consist of interbedded clayey silts, silty sands, clay, poorly graded sands and silts (ML, SM, CL and SP soil types based on the Unified Soil Classification System). The coarse-grained soils (SM and SP soil types) were found to be medium dense to very dense, while the fine-grained soils (ML soil type) were found to be stiff to hard. Detailed logs of the subsurface conditions of the site are presented in Appendix A.

Based on the results of laboratory testing, the surficial soils are considered to have a "Very Low" (0≤EI≤20) to "Low" (21≤EI≤50) Expansion Index (EI) value, when tested in accordance with ASTM D 4829. Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). Based on the results of the collapse testing, the native on-site soils tested were determined to have a low potential for hydroconsolidation (settlement upon wetting with or without additional loading). The laboratory test results are provided in Appendix B.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

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

4.3.2 Groundwater

Groundwater was not encountered in any of GeoTek's exploratory borings to the maximum depths explored (approximately 51.5 feet below existing grade). According to a review of historical groundwater data (California Department of Water Resources Water Data Library, the California State Water Resources Control Board groundwater well data [https://wdl.water.ca.gov/waterdatalibrary/ and http://geotracker.waterboards.ca.gov], and in-house information, groundwater is greater than 50 feet below the existing ground surface with a flow estimated to be directed to the east and north.

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

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 located within an "*Alquist-Priolo*" fault hazard zone for the approximate western two-thirds of the project site for the Elsinore Fault. 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 | Glen Ivy North Fault", as having a "Moderate/Very Low" liquefaction potential hazard and as "susceptible" with regards to subsidence.

No evidence of active faulting was identified by Petra (2000) within the limits of their Fault Trenches 1 and 2. Petra recommended a 50-foot structural setback from the western edge of the site. GeoTek concurs with this recommendation provided by Petra (2000).

4.4.1 Seismic Design Parameters

The site is located at approximately 33.7699 Latitude and -117.4894 West Longitude. Based on the relatively dense soil conditions encountered across the site, a Site Class "D" is considered appropriate for this project. 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.

The following seismic design parameters, based on the 2015 National Earthquake Hazards Reduction Program (NEHRP)/ASCE 7-16/2022 CBC, are presented below:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	2.437g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.974g
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}	2.437g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	2.484g*
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.625g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	1.656g*
Site Modified Peak Ground Acceleration, PGA_M	1.13g
Seismic Design Category	E

*ASCE 7-16 Supplement 3 Section 11.4.8 requires a ground motion hazard analysis for structures on Site Class “D” for values of S1 greater than or equal to 0.2g. However, a ground motion hazard analysis is not required where the values of SM1 and SD1 are increased by 50%. The SM1 and SD1 values shown above already include the 50% increase, so that exception can be obtained.

**ASCE 7-16 Supplement 3 Section 11.4.8 indicates that the value of Fv should only be used for calculation of Ts, determination of Seismic Design Category, linear interpolation for intermediate values of S1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of SD1.

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

4.4.2 Surface Fault Rupture

The site is partially located within an “*Alquist-Priolo*” fault hazard zone for the approximate western two-thirds of the project site for the Elsinore Fault. 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 | Glen Ivy North Fault”, as having a “Moderate/Very Low” liquefaction potential hazard and as “susceptible” with regards to subsidence.

No evidence of active faulting was identified by Petra (2000) within the limits of their Fault Trenches 1 and 2. Petra recommended a 50-foot structural setback from the western edge of the site. GeoTek concurs with the findings, conclusions, and recommendations provided by Petra (2000). Therefore, the potential for surface fault rupture at the site is considered remote.

4.4.3 Liquefaction and Seismically Induced Settlement

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

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

The project site is not located within an area mapped by the State of California for liquefaction potential. The County of Riverside has designated the site as having a “very low” to “moderate” potential for liquefaction.

Groundwater was not encountered in any of the GeoTek borings to the maximum depths explored (approximately 51.5 feet below existing grade). Petra (2000) encountered groundwater at depths of 19 to 22 feet in 2000. This water may represent perched water conditions. Historic groundwater is estimated to be greater than 50 feet below the existing ground surface.

Petra estimates total and maximum post-liquefaction settlements on the order of 2 inches total settlement and 1 ¼ inches differential settlement. Due to the relatively hard/dense nature of the subsurface soils and relatively deep estimated groundwater levels, it is GeoTek’s opinion that liquefaction at the project site is considered low.

As noted in the proposed development section of this report, a post-tensioned (PT) slab foundation system will be used to support the proposed structures. This foundation system should be designed to accommodate the total and differential settlement potentials (static and seismic) provided in this report.

Due to the relatively dense/hard nature of the alluvial materials underlying the site, seismic induced (“dry sand”) settlements are estimated to be minimal.

4.4.4 Other Seismic Hazards

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

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

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

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

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the County of Riverside, the 2022 California Building Code (CBC) and recommendations contained in this report. The General 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.

The upper site soils consisted of undocumented fill soils. In addition, the upper alluvial soils were found to be relatively loose, non-uniform, of low relative compaction and anticipated to have a low potential for hydroconsolidation (settlement upon wetting with or without additional loading). Remedial grading, consisting of overexcavation and recompaction of the upper site soils and previous fault trenches by Petra (2000), is recommended to provide a uniform bearing for the proposed structures.

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

5.2.2 Site Clearing

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

5.2.3 Remedial Grading

Due to the presence of surficial undocumented fill soils and the relatively loose condition of the upper site soils, it is recommended that the soils be removed beneath the planned building footprint to a depth of at least eight (8) feet below existing grades, or five (5) feet below the base of the proposed foundations, whichever is greater. The lateral extent of this recommended over-excavation should extend at least 10 feet beyond the building limits, where obtainable. Removal bottoms should expose native alluvial materials, be relatively uniform in soil type and not adversely porous and have an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures.

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

Petra (2000) previously excavated approximately 780 linear feet of fault trenches to depths ranging between about 9 to 14 feet below existing grades. These previous fault trenches should be completely identified, followed by the deep removal and replacement of all fault trench backfill soil as properly compacted engineered fill. The maximum anticipated depth of the trench backfill is up to about 14 feet below existing grades.

5.2.4 Pavement Areas

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

5.2.5 Hardscape Areas

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

5.2.6 Transition Lot Condition

Building pads graded with a cut/fill transition should be undercut to reduce the potential for differential settlement. The cut portion of the cut/fill transition should be undercut to a depth of at least eight (8) feet from natural grade or five (5) feet below the deepest proposed footing, whichever is deeper, and be backfilled with a properly compacted engineered fill. The bottom of the undercut should be sloped at a minimum of 1 percent toward the adjacent street area.

5.2.7 Slope Construction

Fill and cut slopes constructed at gradients of 2:1 (h:v) or flatter, in accordance with industry standards, are anticipated to be both grossly and surficially stable. Fill placed on slopes should be properly benched into competent soils per the recommendations of the geotechnical engineer. Any cut slopes (not anticipated at this site) should be observed by a geotechnical engineer/engineering geologist to approve the exposed conditions upon excavation.

Swales should be constructed at the top of all slopes to collect and divert drainage away from the slope face. Drainage should be directed to an approved drainage discharge location. Swales should be constructed with concrete, shotcrete or approved non-erosive material. Swales should be cleaned of loose soil and debris on an on-going basis.

Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slope to provide a dense, erosion resistant surface. Berms should be constructed and maintained at the top of all fill slopes to divert drainage away from the slope faces. An abatement program to control ground-burrowing rodents should be implemented and maintained. Burrowing rodents can decrease the long-term performance of slopes.

5.2.8 Preparation of Excavation Bottoms

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

5.2.9 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Engineered fill should be placed in loose lifts with a thickness of eight inches or less and moisture conditioned to at least two percent above the optimum moisture content.

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

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

5.2.10 Excavation Characteristics

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

5.2.11 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 10 to 20 percent may be considered for the materials requiring removal and/or recompaction.

Site balance areas should be available to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.2 foot may be anticipated for the underlying soils.

5.2.12 Trench Excavations and Backfill

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

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

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

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

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation and Building Slab Design Criteria

The soils are classified as having a "Very Low" ($0 \leq EI \leq 20$) to "Low" ($21 \leq EI \leq 50$) Expansion Index in accordance with ASTM D 4829. However, verification testing should be performed after site remedial grading.

The foundation elements for the proposed structures should bear entirely in engineered fill soils and should be designed in accordance with the 2022 CBC. Given the potential for liquefaction at the site, residential buildings should be supported by post-tension foundation systems. Geotechnical parameters for post-tension slab design, in general conformance with *Design of Post-Tensioned Slabs-on-Ground*, Third Edition *with 2008 Supplement* (PTI, 2008), are provided below. These recommendations are minimal recommendations and are not intended to supersede the design by the project structural engineer.

DESIGN PARAMETERS FOR POST-TENSION SLABS	
Foundation Design Parameter	Design Value "Low" Expansion Index (LL≤39; PI≤14; Passing #200 Sieve ≈ 91%; Clay fines ≈ 20%)
Edge Moisture Variation Distance, e_m -Edge Lift (swelling) -Center Lift (shrinkage)	4.3 ft 8.5 ft
Soil Differential Movement, y_m -Edge Lift (swelling) -Center Lift (shrinkage)	≈ 0.66 in ≈ 0.28 in
Exterior Perimeter Beam Embedment	One- and Two-Story – 12 inches* Three-story – 18 inches*
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 110% to a depth of 12 inches

*Required depth of perimeter beam/stiffening rib per structural calculations may govern.
The following assumptions were used to generate e_m and y_m values: Thornthwaite Moisture Index = -20; constant suction value = 3.9pF; post-equilibrium case assumed with wet (swelling) cycle going from 3.9pF to 3.0pF and drying (shrinking) cycle going from 3.9pF to 4.5pF.

Based upon review, a modulus of subgrade reaction (E_1) of 100 pci may be used in the design of the post-tensioned slab foundation. It should be noted that this value is based upon standard one foot plate load tests. Depending upon the design methodology and foundation geometry this value may need to be modified by the following:

$$E_s = E_1 \left((B+1)/2B\right)^2$$

where: Es = design modulus
B = footing width

Post-tensioned slabs should be designed in accordance with the *2022 CBC* and PTI design methodology.

An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This value may be increased by 400 psf for each additional 12 inches in depth and by 200 psf for each additional 12 inches in width to a maximum value of 3,500 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads). These allowable bearing values contain a minimum factor of safety of three (3).

The recommended allowable bearing capacity is based on an estimated maximum post-construction settlement of approximately three-quarters of an inch. Static-induced differential settlement of about one-half of the total settlement over a horizontal distance of 40 feet could result. Seismic-induced vertical settlements for the design earthquake is anticipated to be approximately 2 inches total settlement and 1 1/14 inches differential settlement. The project structural engineer, foundation engineer, and earth retention structure designer should incorporate these static and dynamic settlement estimates into the foundation design, as appropriate.

The passive earth pressure may be computed as an equivalent fluid having a density of 325 psf per foot of depth, to a maximum earth pressure of 3,250 psf for footings founded on engineered fill. The allowable passive earth pressure contains a factor of safety of 1.5. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. Passive pressure and frictional resistance may be combined without reduction. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements.

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

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

5.3.2 Miscellaneous Foundation Recommendations

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

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

5.3.3 Foundation Setbacks

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

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

5.3.4 Structural Slab Moisture and Vapor Retarding System

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2022 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

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

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

5.4 RETAINING WALL DESIGN AND SITE CONSTRUCTION

5.4.1 General Retaining Wall Design Criteria

Retaining wall foundations embedded a minimum of 12 inches below the lowest adjacent grade and should rest on at least 12 inches of compacted fill. Wall footings should be designed using an allowable bearing capacity of 2,000 psf. An increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).

The passive earth pressure may be computed as an equivalent fluid having a density of 325 psf per foot of depth, to a maximum earth pressure of 3,250 psf for footings founded on engineered fill. The allowable passive earth pressure contains a factor of safety of 1.5. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. Passive pressure and frictional resistance may be combined without reduction. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in the table below for specific slope gradients of retained materials.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Native Materials*
Level	40
2:1	60

The above equivalent fluid weights do not include superimposed loading conditions such as expansive soils, vehicular traffic, structures, seismic conditions or adverse geologic conditions.

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 $15H^2$ [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 $25H^2$ 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.

5.4.2 Restrained Retaining Walls

Any retaining wall that will be restrained prior to placing backfill or walls that have male or reentrant corners should be designed for at-rest soil conditions using an equivalent fluid pressure of 62 pcf, plus any applicable surcharge loading. For areas having male or reentrant corners, the restrained wall design should extend a minimum distance equal to twice the height of the wall laterally from the corner, or as otherwise determined by the structural engineer.

5.4.3 Wall Backfill and Drainage

Retaining wall backfill should be free of deleterious and/or oversized materials and should have an expansion index of less than 20. 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-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one-cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining wall backfill should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction as determined by ASTM D 1557 test procedures. The wall backfill should also include a minimum one-foot-wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The rock should be separated from the earth with filter fabric. The upper 24 inches should consist of compacted on-site soil.

As an alternative to the drain rock and fabric, Miradrain 2000, or approved equivalent, may be used behind the retaining wall. The Miradrain 2000 should extend from the base of the wall to within two feet of the ground surface. The subdrain should be placed at the base of the wall in direct contact with the Miradrain 2000.

The presence of other materials might necessitate revision of the parameters provided and modification of the wall designs. Proper surface drainage needs to be provided and maintained. Walls from two to four feet in height may be drained using localized gravel packs behind weep holes at eight feet maximum spacing (e.g., approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided, or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

5.4.3.1 Other 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.

5.4.4 Soil Corrosivity

The soil resistivity at this site was tested in the laboratory on two (2) samples collected during the field investigation. The results of the testing indicate that the on-site soils are considered “highly corrosive” (1,943 to 5,546 ohm-cm) (Roberge, 2000) to buried ferrous metal in accordance with current standards used by corrosion engineers. It is recommended that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site.

5.4.5 Soil Sulfate Content

The sulfate content was determined in the laboratory on two (2) samples collected during the field investigation. The results indicate that the water-soluble sulfate results are 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.

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

5.4.6 Import Soils

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

5.4.7 Concrete Flatwork

5.4.7.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated because of typical mix designs and curing practices commonly utilized in industrial construction.

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

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior flatwork with “Very Low” ($0 \leq EI \leq 20$) Expansion Index (EI) soils should be pre-saturated to a minimum of 100 percent of optimum moisture content. The subgrade soils below exterior flatwork with “Low” ($21 \leq EI \leq 50$) Expansion Index (EI) soils should be pre-saturated to a minimum of 110 percent of optimum moisture content.

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

5.4.7.2 Concrete Performance

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

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

5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

5.5.1 Asphalt Concrete (AC) Pavement

Pavement design for proposed on-site pavement (i.e., local, collector and secondary roadways) improvements was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. An R-value of 20 is being estimated for the post-graded roadway subgrade soils. For preliminary pavement design, assumed Traffic Indexes (TI) of 5.5 and 7.0 were used. Final pavement design should be based on R-value testing of the graded street subgrades and the assigned TI values. Based on the assumptions noted, the following preliminary pavement design recommendations are provided.

TI	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.5 (Local)	3.0	9.0
7.0 (Collector)	4.2	12.0

The TIs used in the pavement design are typically considered reasonable values for the proposed street/parking/drive areas 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.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper 12 inches) after final grading has been completed.

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 six (6) 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 in the design and construction of the paver system.

5.5.3 Portland Cement Concrete (PCC) Pavement

For the proposed vehicle service and access lanes, it is recommended that a minimum of six (6) 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. 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 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 County of Riverside and under the observation and testing of GeoTek and a County inspector, where required.

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

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

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 more than 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 POST CONSTRUCTION CONSIDERATIONS

5.6.1 Landscape Maintenance and Planting

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

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

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

5.6.2 Drainage

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

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

5.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

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

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

6. INTENT

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

The scope of GeoTek's evaluation is limited to the area explored that is shown on the Exploration Location Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to GeoTek by the client. Further, no evaluation of any existing site improvements is included. The scope is based on GeoTek's understanding of the project and the client's needs, GeoTek's proposal (Proposal No. P-0804224-CR) dated August 9, 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.

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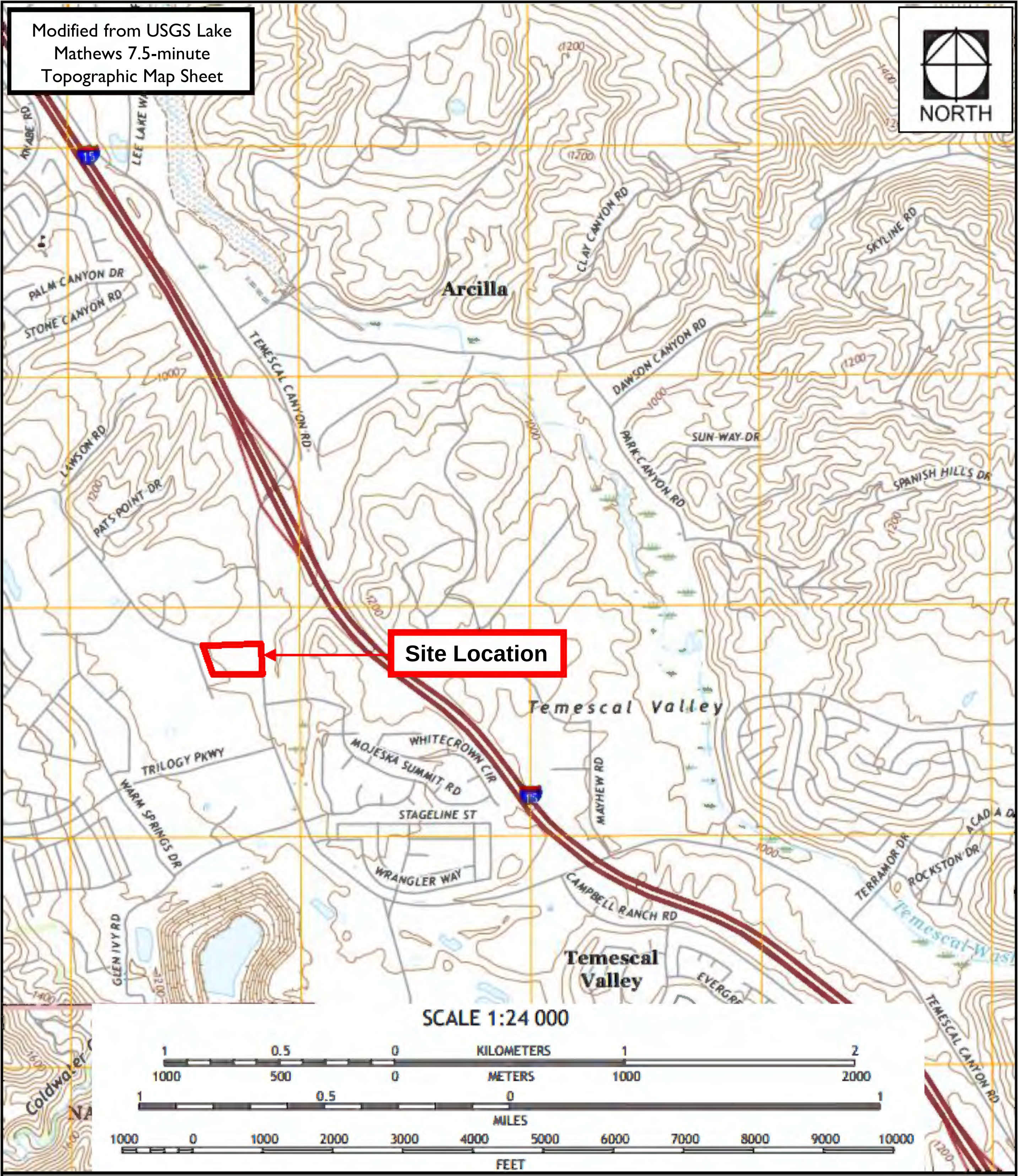


Modified from USGS Lake Mathews 7.5-minute Topographic Map Sheet

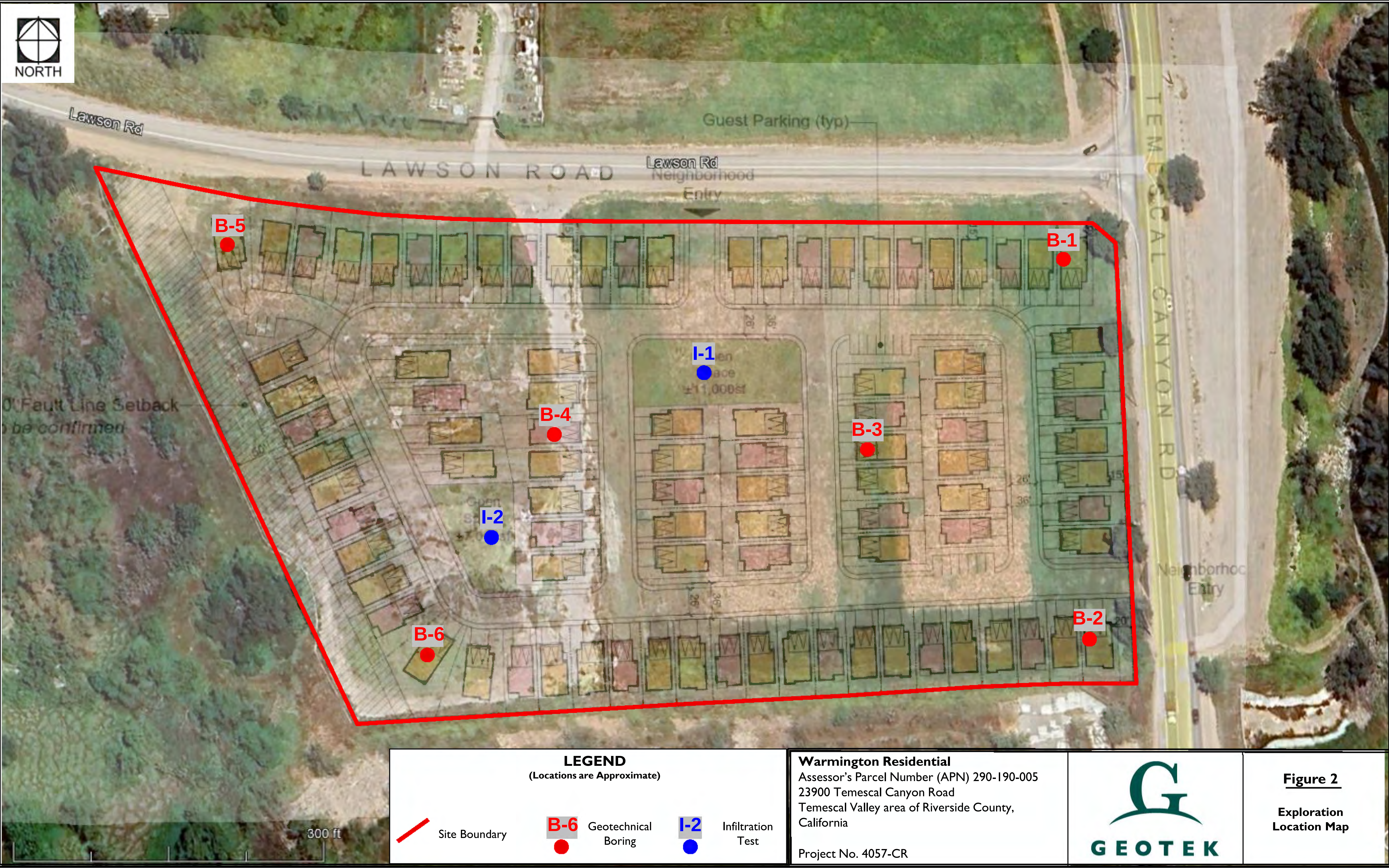
Warmington Residential
Assessor's Parcel Number (APN) 290-190-005
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California

Project No. 4057-CR

Figure 1

Site Location and Topography Map

GEOTEK



LEGEND

(Locations are Approximate)

🟥	Site Boundary
B-6 ●	Geotechnical Boring
I-2 ●	Infiltration Test

Warmington Residential
Assessor's Parcel Number (APN) 290-190-005
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California

Project No. 4057-CR

GEOTEK

Figure 2

Exploration Location Map

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Development
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California
Project No. 4057-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 logs of borings:

SOILS

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

GEOLOGIC

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Warmington	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	Temescal and Lawson	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Eddie
PROJECT NO.:	4057-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig
LOCATION:	See Boring Location Map			**DATE:**	9/5/2024

BORING NO.: B-1

Depth (ft)	SAMPLES Sample Type	 Blows/ 6 in	 Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	 Dry Density (pcf)	 Others
	⧄				**Undocumented fill:**			
	■	4 4 7		ML	F sandy SILT, dark brown, slightly moist, medium stiff, trace gravel, trace rootlets	19.1	79.6	SR, MD, AL, EI, SH % Passing #200 = 91.1 LL=39; PL=26; PI=14 EI=24
	⧄ ■ ⧄	6 7 10			Clayey SILT, dark brown, slightly moist, very stiff	18.6	92.4	
5	■	9 12 16		ML	**Alluvium:** F sandy SILT, medium brown, slightly moist, very stiff, trace caliche stringers	17.0	96.5	Collapse
	■	11 17 29			F sandy SILT, light brown, slightly moist, hard, trace gravels, oxidation stains	8.7	97.2	
10	■	24 50/5"		SM	Silty f SAND, light brown, slightly moist, hard, some gravel			
15	■	50/6"		GP	GRAVELS, gray, very dense, some cobble	0.2		disturbed sample
	▮	25 28 37		SP	Gravelly f-c SAND, gray-brown, slightly moist, very dense			
20	☐	50/6"			F-c sandy GRAVELS, gray, very dense, trace cobble			
					BORING TERMINATED AT 20.5 FEET No groundwater encountered Boring backfilled with excavated materials			
25								
30								

LEGEND

Sample type: ■ ▮ ---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:	Warmington	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	Temescal and Lawson	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Eddie
PROJECT NO.:	4057-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig
LOCATION:	See Boring Location Map			**DATE:**	9/5/2024

BORING NO.: B-2

Depth (ft)	SAMPLES Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	Dry Density (pcf)	Others
					Undocumented fill:			
	■	9 15 19		ML	Clayey SILT, dark brown, slightly moist, very stiff	17.2	94.5	
	■	15 22			Clayey SILT, dark brown, slightly moist, hard, trace caliche stringers, oxidation stains	16.4	101.1	
5		28			**Alluvium:**			
				ML	F sandy SILT, medium brown, slightly moist, hard, trace clay			
	■	16 22 27				14.5	106.0	
	■	10 15 19			F sandy SILT, light brown, slightly moist, very stiff, oxidation stains, porosity, trace gravel, trace clay	11.3	100.2	
10								
15	■	15 20 21						
	■	8 8 8		SM	Silty f-c SAND, orange-brown, slightly moist, stiff, oxidation stains, few gravels	10.6	108.5	
20	▮	4 4 4		ML	Clayey SILT, orange-brown, sightly moist, medium stiff, oxidation stains, trace gravel	22.3		
25	☐	50/1"			Silty SAND, medium brown, slightly moist, very dense, oxidation stains, some gravels			
30	▮	10 11 11		SP	F SAND, light gray, slightly moist, medium dense, trace clay	9.7		

LEGEND

Sample type: ■ ▮ ---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: Warmington
PROJECT NAME: Temescal and Lawson
PROJECT NO.: 4057-CR
LOCATION: See Boring Location Map
DRILLER: 2R Drilling Inc.
DRILL METHOD: Hollw stem Auger
HAMMER: 140lbs/30in.
LOGGED BY: KLP
OPERATOR: Eddie
RIG TYPE: Truck rig
DATE: 9/5/2024

BORING NO.: B-2 Continued

Depth (ft)	SAMPLES Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	Dry Density (pcf)	Others
35	■	6 6 10		CL	F-m sandy CLAY, brown-gray, slighly moist, very stiff			
40	■	5 7 11			F-m sandy CLAY, brown-gray, moist (water added by driller), very stiff	23.5		
45	■	6 18 15		SP	F-m SAND, gray, moist (water added by driller), dense			
50	■	6 19 14		CL	F-m sandy CLAY, medium brown, slightly moist, hard, effervescent	29.0		
55					**BORING TERMINATED AT 51.5 FEET** No groundwater encountered Boring backfilled with excavated materials			
60								

LEGEND

Sample type: ■ ■ ---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: Warmington
PROJECT NAME: Temescal and Lawson
PROJECT NO.: 4057-CR
LOCATION: See Boring Location Map
DRILLER: 2R Drilling Inc.
DRILL METHOD: Hollw stem Auger
HAMMER: 140lbs/30in.
LOGGED BY: KLP
OPERATOR: Eddie
RIG TYPE: Truck rig
DATE: 9/5/2024

BORING NO.: B-3

Depth (ft)	SAMPLES Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	Dry Density (pcf)	Others
	⧄				**Undocumented fill:**			SR
	■	9 11 16		SM	Silty f-c SAND, light brown, slightly moist, medium dense, some gravels, trace cobble	5.4	98.9	
	⧄							
	■	16 20 18				5.6	109.7	
	⧄							
5	■	14 15 16		SM	**Alluvium:** Silty f-c SAND, light brown, slightly moist, medium dense, some gravels, trace cobble, trace clay	5.8	112.8	Collapse
	☐	11 12 14			Silty f-c SAND, light brown, medium dense, some gravels, some cobbles			
10	☐	21 50/6"			Silty f-c SAND, light brown, very dense, some gravels, some cobbles			
15	■	21 25 26		SM	Silty f-c SAND, yellow-brown, slighty moist, dense, trace gravels	7.4	105.1	
					BORING TERMINATED AT 16.5 FEET No groundwater encountered Boring backfilled with excavated materials			
20								
25								
30								

LEGEND

Sample type: ■ ▮ ---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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Warmington	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	Temescal and Lawson	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Eddie
PROJECT NO.:	4057-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig
LOCATION:	See Boring Location Map			**DATE:**	9/5/2024

BORING NO.: B-4

Depth (ft)	SAMPLES Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	Dry Density (pcf)	Others
					Undocumented fill:			
	■	11 31 27		SM	Silty f-c SAND, light brown, slightly moist, dense, some gravels, trace cobble	7.0		
5	■	15 50/6"			Very dense	4.7		disturbed sample
	■	16 31 36		SM	**Alluvium:** Silty f-c SAND, orange-brown, slightly moist, dense, some gravels, trace cobble	2.5		disturbed sample
	■	14 26 29			Silty f-c SAND, orange-brown, slightly moist, dense, some gravels, oxidation stains	4.4		disturbed sample
10								
		50/6"			F-c GRAVELS, light brown, very dense, trace cobble			
15					**BORING TERMINATED AT 14 FEET** No groundwater encountered Boring backfilled with excavated materials			
20								
25								
30								

LEGEND

Sample type: ■ ■ ---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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Warmington
PROJECT NAME: Temescal and Lawson
PROJECT NO.: 4057-CR
LOCATION: See Boring Location Map
DRILLER: 2R Drilling Inc.
DRILL METHOD: Hollw stem Auger
HAMMER: 140lbs/30in.
LOGGED BY: KLP
OPERATOR: Eddie
RIG TYPE: Truck rig
DATE: 9/5/2024

BORING NO.: B-5

Depth (ft)	SAMPLES Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	Dry Density (pcf)	Others
					Undocumented fill:			
	■	9 7 10		SM	Silty f-c SAND, medium brown, slightly moist, medium dense, some gravel	6.5	89.4	
	☐	6 6 7			F-c sandy GRAVELS, light brown, loose, trace cobble			
5	■	10 11 21			Silty f-c SAND, medium brown, slightly moist, medium dense, some gravel, trace cobble	9.5		disturbed sample
	■	8 8 9		SM	**Alluvium:** Silty f-c SAND, medium brown, slightly moist, medium dense, some gravel, trace cobble, trace clay	10.2	114.1	Collapse
10	■	9 15 16			Silty f-c SAND, medium brown, slightly moist, medium dense, some gravel, trace cobble, trace clay, oxidation stains	8.0	116.2	
15	■	11 18 38			Silty f-m SAND, orange-brown, slightly moist, dense, some gravel, oxidation stains	10.8	115.0	
					BORING TERMINATED AT 16.5 FEET No groundwater encountered Boring backfilled with excavated materials			
20								
25								
30								

LEGEND

Sample type: ■ ▪ ---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:	Warmington	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	Temescal and Lawson	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Eddie
PROJECT NO.:	4057-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig
LOCATION:	See Boring Location Map			**DATE:**	9/5/2024

Depth (ft)	SAMPLES Sample Type	 Blows/ 6 in	 Sample Number	USCS Symbol	**BORING NO.: B-6** **MATERIAL DESCRIPTION AND COMMENTS**	Laboratory Testing Water Content (%)	 Dry Density (pcf)	 Others
	☒				**Undocumented fill:**			MD, SH, EI EI=7
	■	9 11 14		ML	F sandy SILT, dark brown, slightly moist, very stiff, trace clay	15.4	97.0	
5	■	10 12 16		ML	**Alluvium:** Clayey SILT, medium brown, slightly moist, very stiff, trace gravel, caliche stringers	14.9	108.5	
	■	5 6 8			Clayey SILT, medium brown, slightly moist, stiff, trace gravel	15.3	92.5	
	■	5 11 14			Very stiff	13.3	91.2	
10								
15	■	11 21 27		SM	Silty f-c SAND, medium brown, slightly moist, dense, some gravel, trace cobble, trace clay	15.5	113.8	
					BORING TERMINATED AT 15 FEET No groundwater encountered Boring backfilled with excavated materials			
20								
25								
30								

LEGEND

Sample type: ■ ■ ---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:	Warmington	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	Temescal and Lawson	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Eddie
PROJECT NO.:	4057-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig
LOCATION:	See Boring Location Map			**DATE:**	9/5/2024

BORING NO.: I-1

	SAMPLES					Laboratory Testing		
Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	**MATERIAL DESCRIPTION AND COMMENTS**	Water Content (%)	Dry Density (pcf)	Others
				ML	**Undocumented fill:** F sandy SILT, medium brown, slighty moist			
				GP	**Alluvium:** GRAVELS, medium brown			
5					**BORING TERMINATED AT 5 FEET** No groundwater encountered Boring backfilled with excavated materials			
10								
15								
20								
25								
30								

LEGEND

Sample type: ■ ▪ ---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:	Warmington	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	Temescal and Lawson	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Eddie
PROJECT NO.:	4057-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig
LOCATION:	See Boring Location Map			**DATE:**	9/5/2024

BORING NO.: I-2

Depth (ft)	SAMPLES Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing Water Content (%)	Dry Density (pcf)	Others
				ML	**Undocumented fill:** F sandy SILT, medium brown, slighty moist to moist			
5				GP	**Alluvium:** GRAVELS, medium brown, trace cobble			
					BORING TERMINATED AT 5 FEET No groundwater encountered Boring backfilled with excavated materials			
10								
15								
20								
25								
30								

LEGEND

Sample type: ■ ■ ---SPT ⧄ ---Small Bulk ☒ ---Large Bulk ☐ ---No Recovery ▽ ---Water Table

Lab testing:

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

APPENDIX B

LABORATORY TEST RESULTS

Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Development
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California
Project No. 4057-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Atterberg Limits

Atterberg limits testing was performed on a select fine-grained sample collected from the site. The test was performed in general accordance with ASTM Test Method D 4318 test procedures. The test results are presented on the boring logs included within Appendix A and graphically in Appendix B.

Classification

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

Collapse Test

Collapse tests were performed on selected samples of the site soils 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 two (2) soil samples remolded to approximately 90 percent of maximum dry density as determined by ASTM D 1557 test procedures in addition to in-situ samples obtained from the ring sampler. The shear test results are presented graphically in Appendix B.

Expansion Index

Expansion Index testing was performed on two (2) samples collected during the subsurface exploration. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	Description	Expansion Index	Classification
B-1	0-5	Clayey Silt (ML)	24	Low
B-6	0-5	Silty Sand (SM)	7	Very Low

In-Situ Moisture and Density

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

Materials Finer Than the No. 200 Sieve

A #200 sieve wash was performed on a selected sample of the site soils in general accordance with ASTM Test Method D 1140 test procedures. The results of this testing are presented on the boring logs in Appendix A and graphically in Appendix B.

Moisture-Density Relationship

Laboratory testing was performed on two (2) samples collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with ASTM Test D 1557 test procedures. The results of the testing are presented graphically in Appendix B.

Sulfate Content, Resistivity and Chloride Content

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

Boring No.	Depth (ft.)	pH ASTM D4972	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-1	0-5	6.9	19.2	0.0023	2,546
B-3	0-5	6.5	43.6	0.0050	1,943

GEOTEK

ATTERBERG LIMITS DATA

Field Classification ______

Sample Number ______

Sample Type ______

Location B-1 @ 0-5 Feet

Tested by: AH

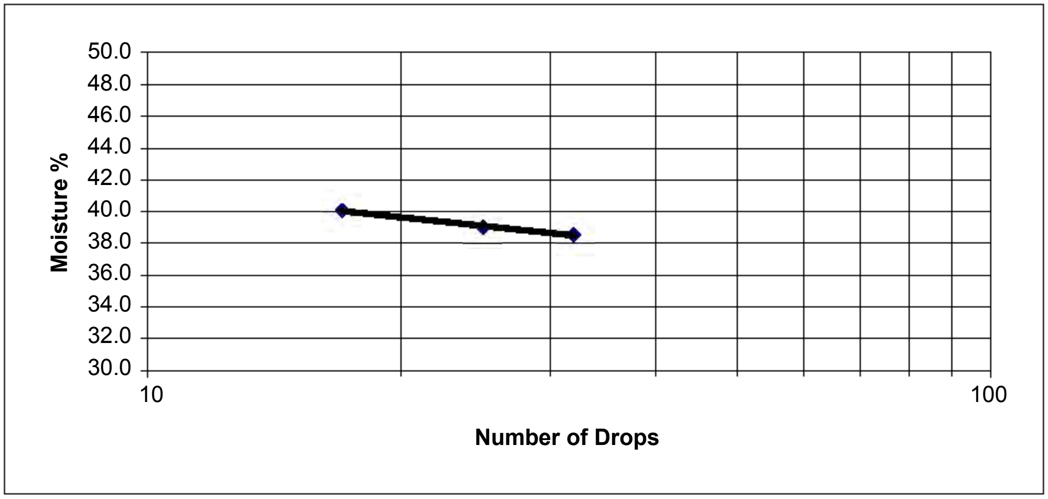
Job No. 4057-CR

Client Warmington Residential

Project 23900 Temescal Canyon Road

	Plastic Limit			Liquid Limit		
Number of Blows	17	53		17	25	32
Wt. of Dish + Wet Soil	36.77	36.84		17.64	17.06	17.10
Wt. of Dish + Dry Soil	35.53	35.57		14.39	14.04	14.10
Wt. of Moisture	1.24	1.27		3.25	3.02	3.00
Wt. of Dish	30.71	30.57		6.28	6.30	6.32
Wt. of Dry Soil	4.82	5.00		8.11	7.74	7.78
Moisture Content %	25.7	25.4		40.1	39.0	38.6

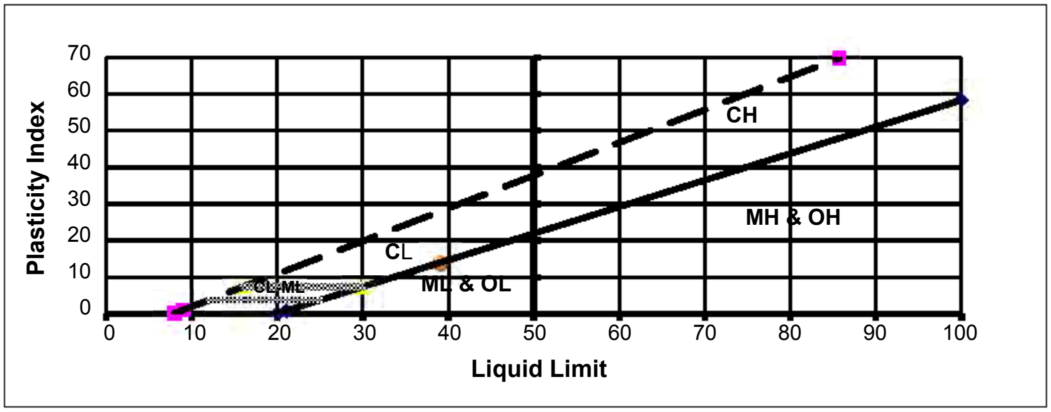
Liquid Limit Graph

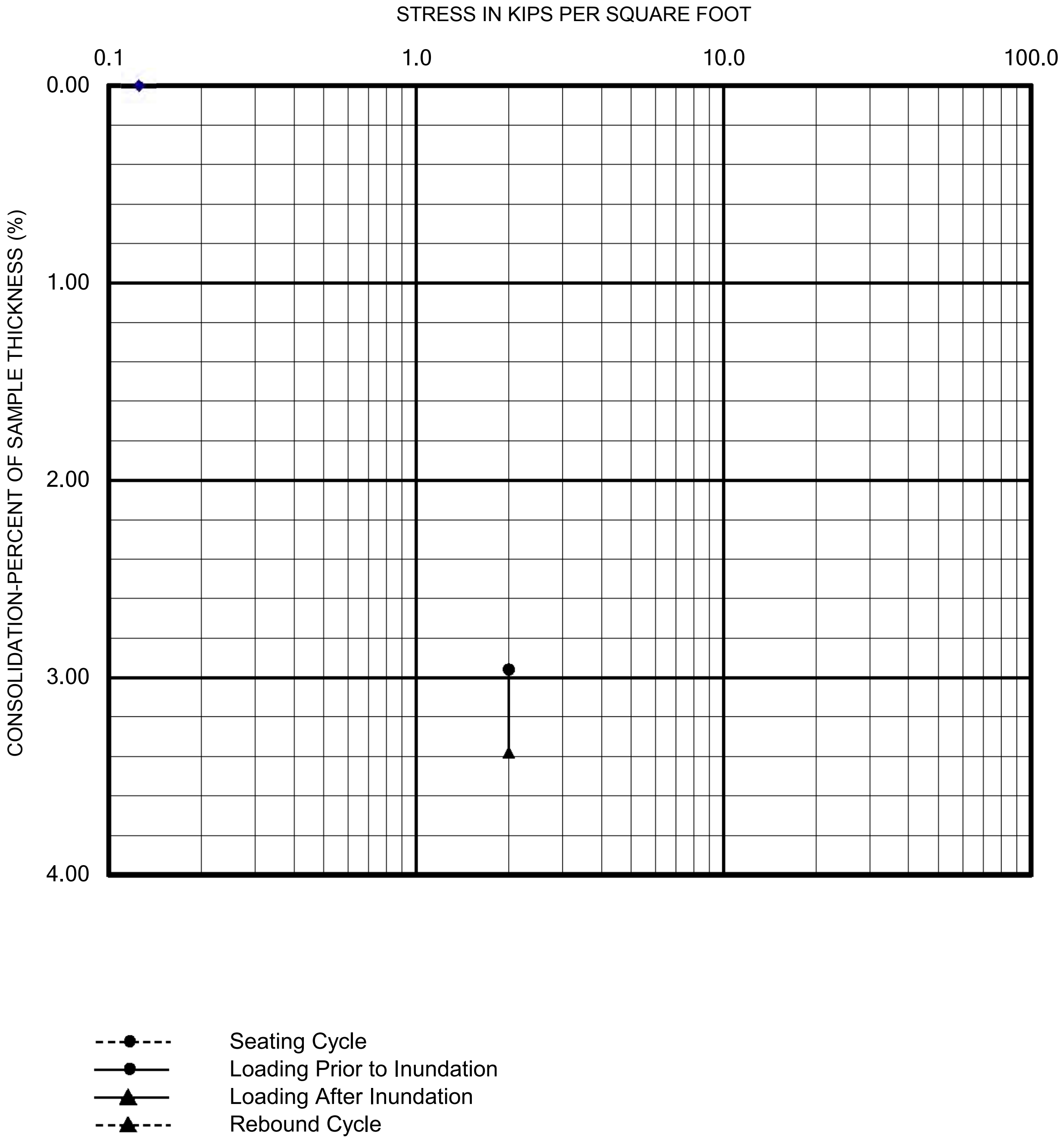


Liquid Limit
39

Plastic Limit
26

Plasticity Index
14





PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

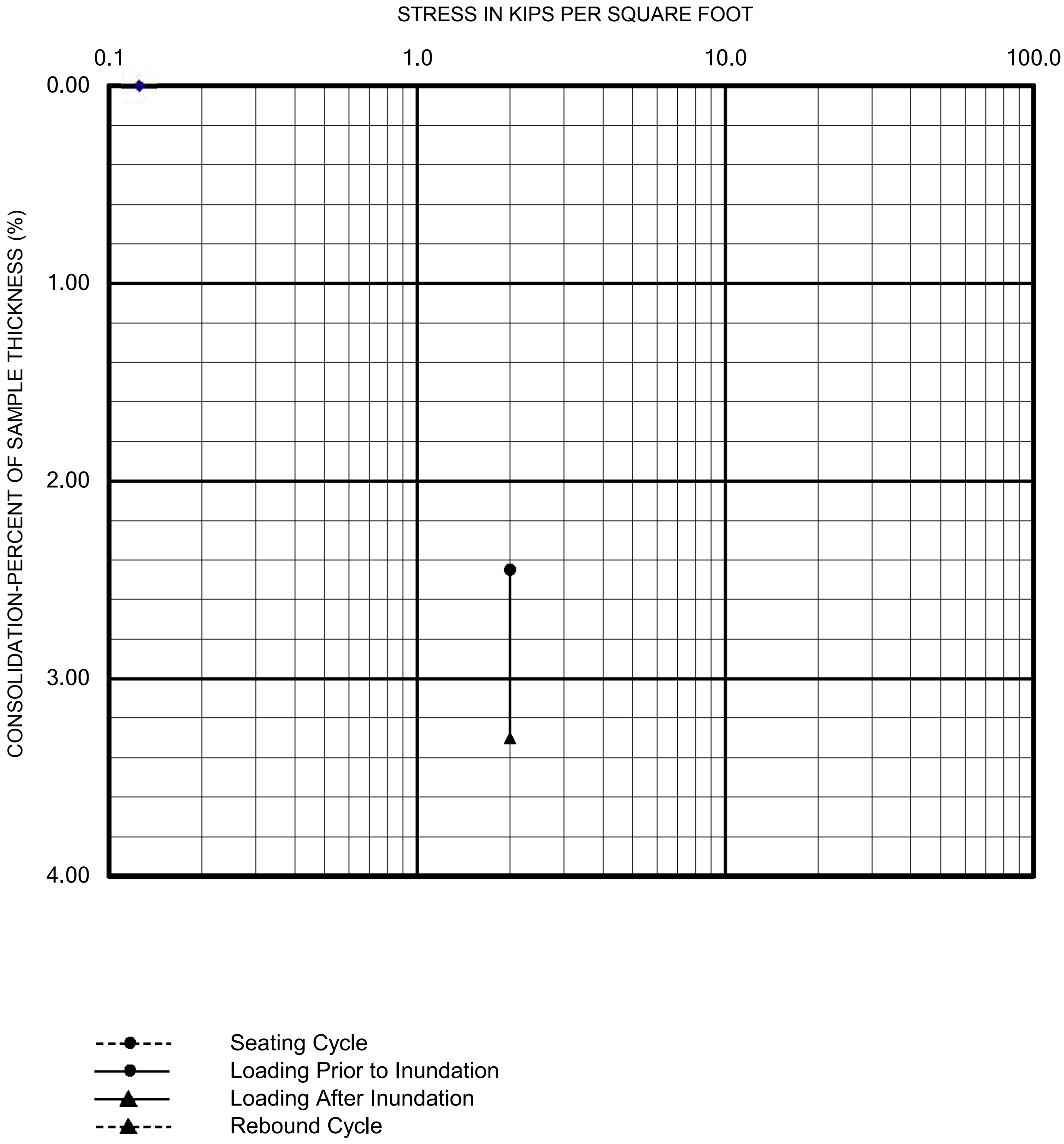
COLLAPSE REPORT

Sample: B-1 @ 5 Feet

23900 Temescal Canyon Road

Plate B-1

CHECKED BY: EC	Lab: Corona
PROJECT NO.: 4057-CR	Date: 9/16/2024



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

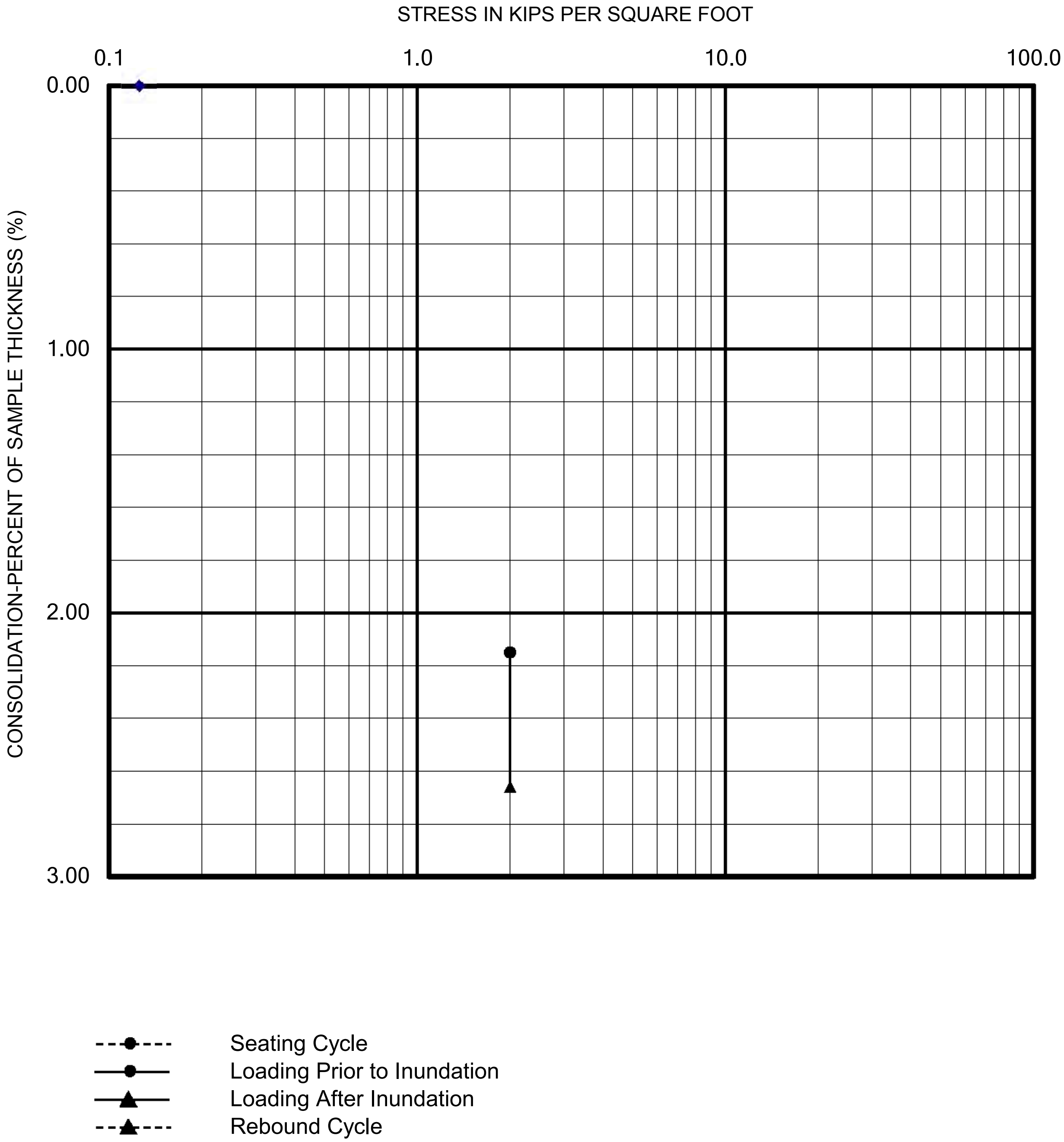
COLLAPSE REPORT

Sample: B-3 @ 5 Feet

23900 Temescal Canyon Road

Plate B-2

CHECKED BY: EC	Lab: Corona
PROJECT NO.: 4057-CR	Date: 9/16/2024



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample: B-5 @ 7 Feet

23900 Temescal Canyon Road

Plate B-3

CHECKED BY: EC

Lab: Corona

PROJECT NO.: 4057-CR

Date: 9/16/2024

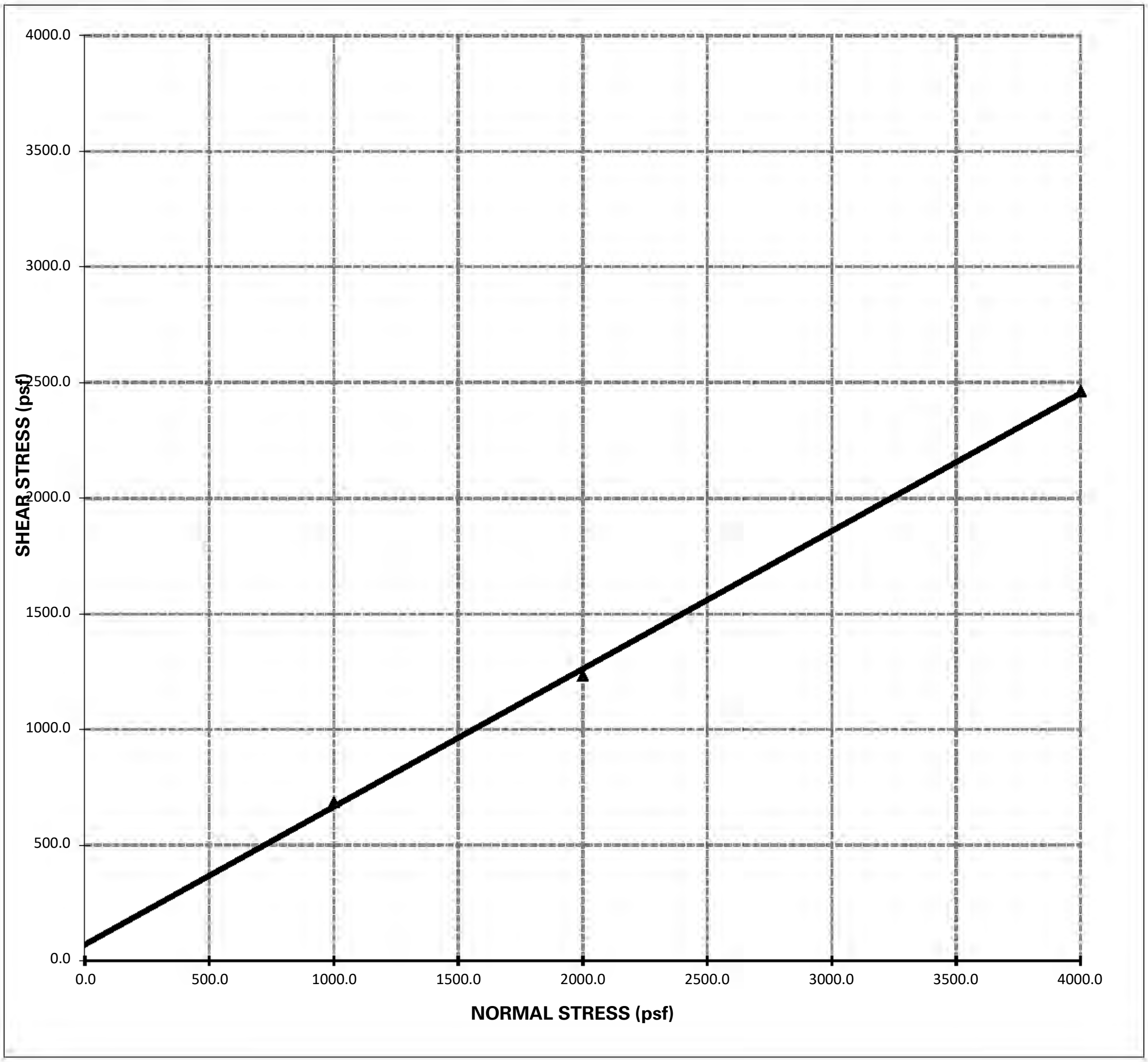
DIRECT SHEAR TEST

Project Name: 23900 Temescal Canyon Road

Project Number: 4057-CR

Sample Location: B-1 @ 0-5 Feet

Date Tested: 9/24/2024



Shear Strength: Φ = 31 ° C = 70 psf

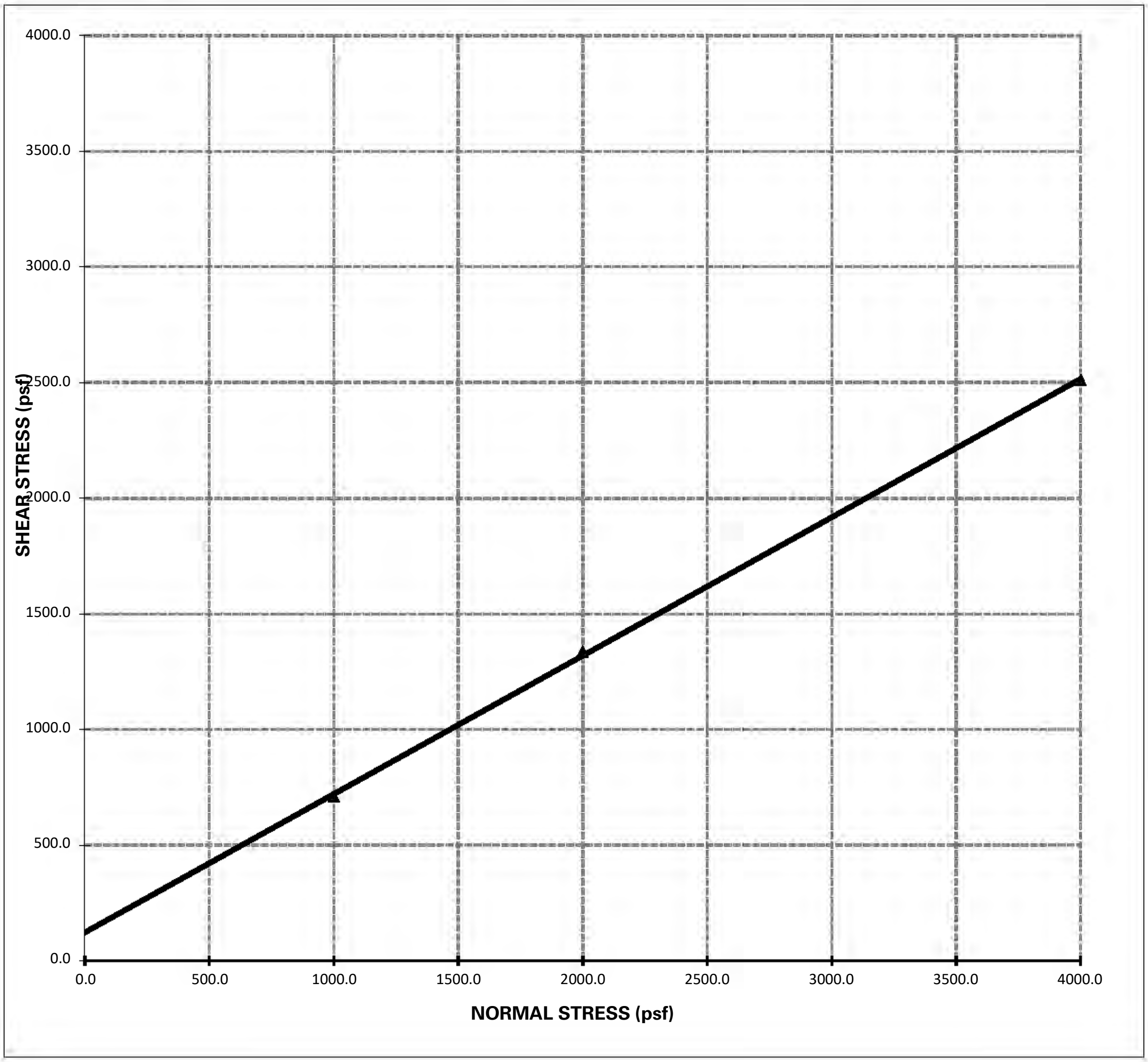
Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

Project Name: 23900 Temescal Canyon Road
Project Number: 4057-CR
Sample Location: B-6 @ 0-5 Feet
Date Tested: 9/18/2024



Shear Strength: Φ = 31 ° C = 123 psf

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

EXPANSION INDEX TEST

(ASTM D4829)

Client: Warmington Residential

Project Number: 4057-CR

Project Location: 23900 Temescal Canyon Road

Tested/ Checked By: Lyn Lab No Corona

Date Tested: 9/18/2024

Sample Source: B-1 @ 0-5 Feet

Sample Description: ______

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	709.8
Weight of ring (gm)	365.9
Net weight of sample (gm)	**343.9**
Wet Density, lb / ft3 (C*0.3016)	**103.7**
Dry Density, lb / ft3 (D/1.F)	**88.4**

SATURATION DETERMINATION

Moisture Content, %	17.3
Specific Gravity, assumed	**2.70**
Unit Wt. of Water @ 20°C, (pcf)	**62.4**
% Saturation	**51.6**

READINGS

DATE	TIME	READING	
9/18/2024		0.4510	Initial
9/18/2024		0.4560	10 min/Dry
9/19/2024		0.4800	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
812.4	**47.1**

EXPANSION INDEX = 24

EXPANSION INDEX TEST

(ASTM D4829)

Client: Warmington Residential

Project Number: 4057-CR

Project Location: 23900 Temescal Canyon Road

Tested/ Checked By: Lyn Lab No Corona

Date Tested: 9/18/2024

Sample Source: B-6 @ 0-5 Feet

Sample Description: __________

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	731.5
Weight of ring (gm)	371.2
Net weight of sample (gm)	**360.3**
Wet Density, lb / ft3 (C*0.3016)	**108.7**
Dry Density, lb / ft3 (D/1.F)	**94.5**

SATURATION DETERMINATION

Moisture Content, %	15.0
Specific Gravity, assumed	**2.70**
Unit Wt. of Water @ 20°C, (pcf)	**62.4**
% Saturation	**51.7**

READINGS			
DATE	TIME	READING	
9/18/2024		0.8060	Initial
9/18/2024		0.8050	10 min/Dry
9/19/2024		0.8120	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
795.9	**32.9**

EXPANSION INDEX = 7

GEOTEK

-200 WASH

Date: 9/24/2024

W.O.: 4057-CR sample ID B-1

Client: Warmington Residential depth 0-5 Feet

Project: 23900 Temescal Canyon Road

Sieve Size	Particle Diameter		Wt. Retained	Wt. Passing	% Passing	Specs
	in.	mm.				
#200	0.0029	0.074	25.3	259.1	91.1%	

Dry Weight 284.4

Soak Time 1440 Minutes

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

Report No: PTR:24-00189-S01

Proctor Report

Client: Warmington Residential
3090 Pullman Street
Costa Mesa CA 92626

CC:

Project: 4057-CR
23900 Temescal Canyon Road

Sample Details

Sample ID: 24-00189-S01

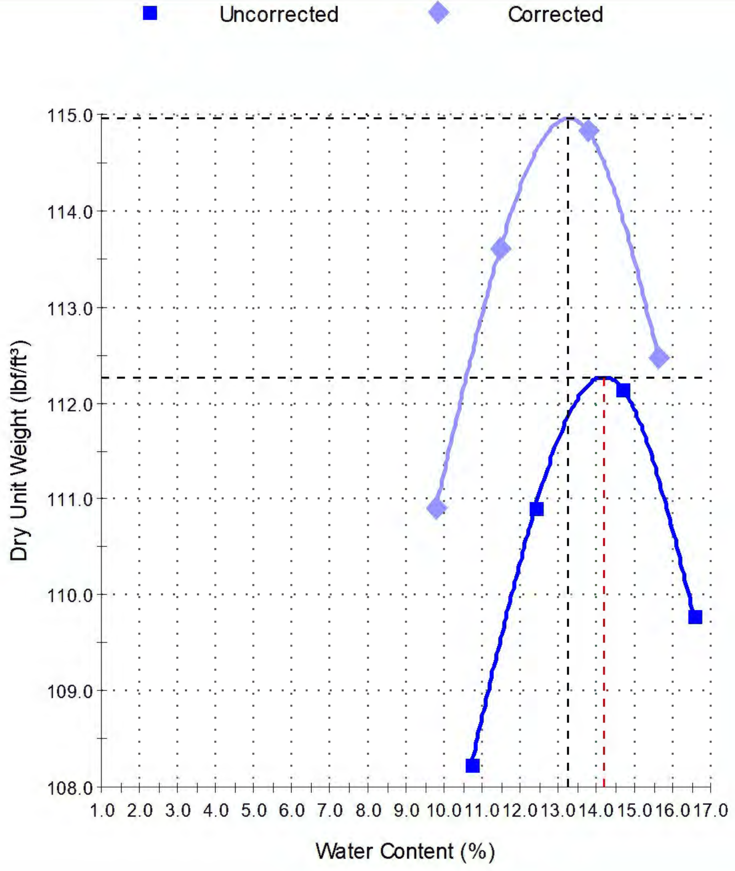
Date Sampled: 9/3/2024

Sampled By:

Material: Fine Sandy SILT

Location: B-6 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lbf/ft³):	**112.3**
Optimum Water Content (%):	**14.2**
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	7
Passing Sieve No 4 (4.75mm) (%):	93
Tested By:	Eduardo Cuevas
Date Tested:	9/13/2024

ASTM D 4718

Corrected Maximum Dry Unit Weight (lbf/ft³):	**115.0**
Corrected Optimum Water Content (%):	**13.3**
Specific Gravity (Oversize):	2.70
Sieve Size (Oversize):	No 4
Oversize Particles (%):	7

Comments

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

Report No: PTR:24-00189-S02

Proctor Report

Client: Warmington Residential
3090 Pullman Street
Costa Mesa CA 92626

CC:

Project: 4057-CR
23900 Temescal Canyon Road

Sample Details

Sample ID: 24-00189-S02

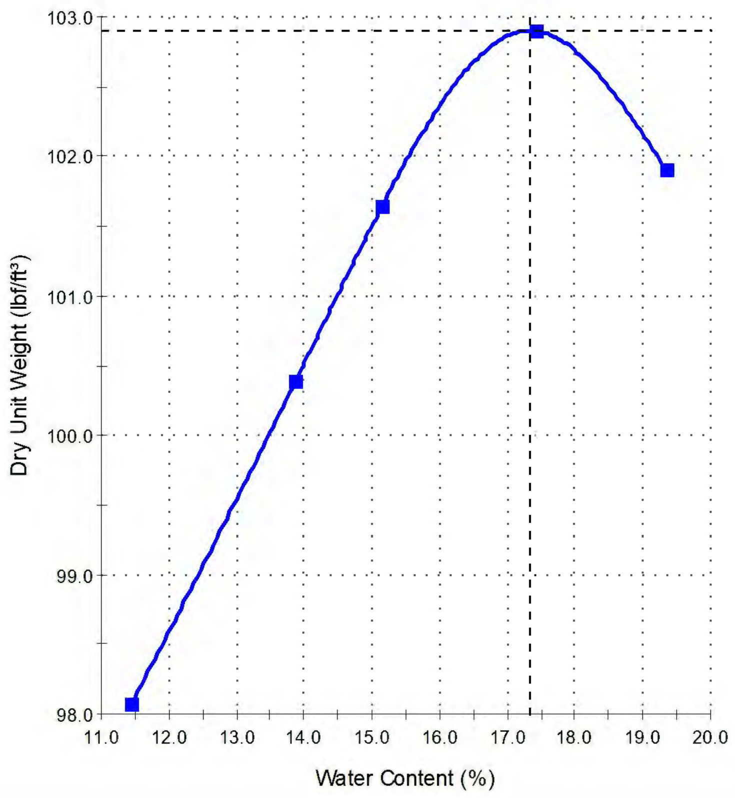
Date Sampled: 9/3/2024

Sampled By:

Material: Fine to Coarse Sandy SILT

Location: B-1 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lbf/ft³):	**102.9**
Optimum Water Content (%):	**17.3**
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	1
Passing Sieve No 4 (4.75mm) (%):	99
Tested By:	Eduardo Cuevas
Date Tested:	9/16/2024

Comments

Project X
Corrosion Engineering
Corrosion Control – Soil, Water, Metallurgy Testing Lab

Results Only Soil Testing
for
Warmington Residential

September 12, 2024

Prepared for:

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

Project X Job#: S240911G
Client Job or PO#: 4057-CR

Prepared by:
D. Bobrova

Respectfully Submitted,

EDUARDO HERNANDEZ
No. M37102

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: Warmington Residential
Client Job Number: 4057-CR
Project X Job Number: S240911G
September 12, 2024

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** **As Rec'd**	**Minimum**	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** $K^{\pm}$	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{2-}	**Phosphate** PO_4^{3-}
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ω-cm)**	**(Ω-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B1	0-5	23.3	0.0023	19.2	0.0019	8,710	2,546	6.9	163	0.5	29.0	10.2	ND	28.6	6.7	23.4	67.4	12.0	2.2
B3	0-5	49.8	0.0050	43.6	0.0044	11,390	1,943	6.5	175	0.1	188.1	1.0	ND	35.7	5.4	28.7	80.8	4.7	2.6

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

INFILTRATION TEST DATA AND PORCHET CALCULATIONS

Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Development
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California
Project No. 4057-CR

PERCOLATION DATA SHEET

Project: 23900 Temescal Canyon Road **Job No.:** 4057-CR

Test Hole No.: I-1 **Tested By:** JR **Date:** 9/5/2024

Depth of Hole As Drilled: 60" **Before Test:** 60" **After Test:** 60"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Rate (Minutes per Inch)	Comments
1	11:05 AM		60	24			1.4	presoaked
	11:30 AM	25			5 5/8	18 3/8		
2	11:31 AM		60	24			1.4	
	11:56 AM	25			6 3/4	17 1/4		
1	11:57 AM		60	24			0.9	
	12:07 PM	10			12 7/8	11 1/8		
2	12:08 PM		60	24			0.9	
	12:18 PM	10			13 3/8	10 5/8		
3	12:19 PM		60	24			1.0	
	12:29 PM	10			13 5/8	10 3/8		
4	12:30 PM		60	24			1.0	
	12:40 PM	10			13 7/8	10 1/8		
5	12:41 PM		60	24			1.0	
	12:51 PM	10			14	10		
6	12:52 PM		60	24			1.0	
	1:02 PM	10			14 1/8	9 7/8		

PERCOLATION DATA SHEET

Project: 23900 Temescal Canyon Road **Job No.:** 4057-CR

Test Hole No.: I-2 **Tested By:** JR **Date:** 9/5/2024

Depth of Hole As Drilled: 60" **Before Test:** 60" **After Test:** 60"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Rate (Minutes per Inch)	Comments
1	12:34 PM		60	24				presoaked
	12:59 PM	25			8 7/8	15 1/8	1.7	
2	1:00 PM		60	24				
	1:25 PM	25			10 7/8	13 1/8	1.9	
1	1:26 PM		60	24				
	1:36 PM	10			16 3/8	7 5/8	1.3	
2	1:37 PM		60	24				
	1:47 PM	10			17	7	1.4	
3	1:48 PM		60	24				
	1:58 PM	10			17 1/4	6 3/4	1.5	
4	1:59 PM		60	24				
	2:09 PM	10			17 1/2	6 1/2	1.5	
5	2:10 PM		60	24				
	2:20 PM	10			17 5/8	6 3/8	1.6	
6	2:21 PM		60	24				
	2:31 PM	10			17 7/8	6 1/8	1.6	

Client: **Warmington Residential**
Project: **23900 Temescal Canyon Road**
Project No: **4057-CR**
Date: **9/5/2024**

Boring No. **I-1**

Percolation to Infiltration Rate (Porchet Method)

Time Interval, Δt = 10
Final Depth to Water, D_F = 50.125
Test Hole Radius, r = 4
Initial Depth to Water, D_O = 36
Total Test Hole Depth, D_T = 60

Equation -

$$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$$

$H_O = D_T - D_O =$ 24
$H_F = D_T - D_F =$ 9.875
$\Delta H = \Delta D = H_O - H_F =$ 14.125
$Havg = (H_O+H_F)/2 =$ 16.9375

I_t = 8.95 **Inches per Hour**

GEOTEK

Client: **Warmington Residential**
Project: **23900 Temescal Canyon Road**
Project No: **4057-CR**
Date: **9/5/2024**

Boring No. **I-2**

Percolation to Infiltration Rate (Porchet Method)

Time Interval, Δt = 10
Final Depth to Water, D_F = 53.875
Test Hole Radius, r = 4
Initial Depth to Water, D_O = 36
Total Test Hole Depth, D_T = 60

Equation -

$$I_t = \frac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$$

$H_O = D_T - D_O =$ 24
$H_F = D_T - D_F =$ 6.125
$\Delta H = \Delta D = H_O - H_F =$ 17.875
$Havg = (H_O+H_F)/2 =$ 15.0625

I_t = 12.57 **Inches per Hour**

APPENDIX D

GENERAL GRADING GUIDELINES

Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Development
23900 Temescal Canyon Road
Temescal Valley area of Riverside County, California
Project No. 4057-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 aspects 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 done 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 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 backhoe 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) The 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 contractors' procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this should 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 the 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.

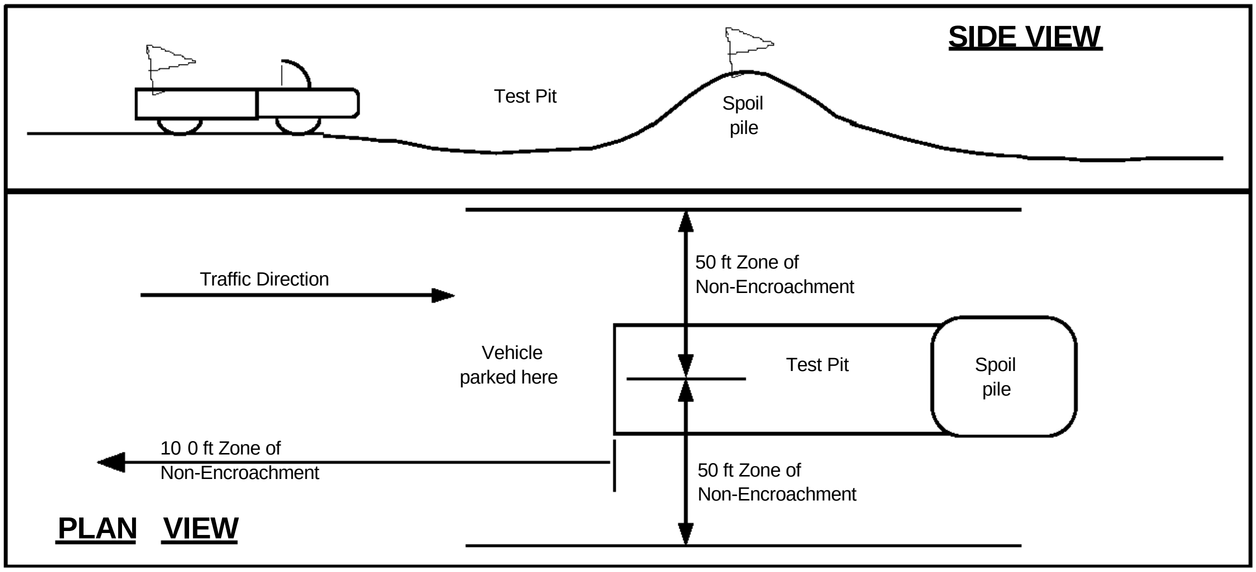
Test Pits Location, Orientation and Clearance

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 affect 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 the technician's 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.

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