

**GEOTECHNICAL AND INFILTRATION EVALUATION  
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT  
ASSESSOR'S PARCEL NUMBER 3153-034-087  
WEST LANCASTER BOULEVARD AND 30<sup>TH</sup> STREET WEST  
CITY OF LANCASTER, LOS ANGELES COUNTY, CALIFORNIA**

**PREPARED FOR**

**GUARDIAN CAPITAL  
5780 FLEET STREET  
CARLSBAD, CALIFORNIA 92008**

**PREPARED BY**

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April 19, 2023  
Project No. 3493-CR

**Guardian Capital**

5780 Fleet Street  
Carlsbad, California 92008

Attention: Mr. Glen Powles

Subject: **Geotechnical and Infiltration Evaluation**  
Proposed Multi-Family Residential Development  
Assessor's Parcel Number 3153-034-087  
West Lancaster Boulevard and 30<sup>th</sup> Street West  
City of Lancaster, Los Angeles County, California

Dear Mr. Powles:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical and Infiltration Evaluation for the proposed multi-family residential development project to be located northeast of the intersection of West Lancaster Boulevard and 30<sup>th</sup> Street West in the City of Lancaster, Los Angeles County, California. This report presents the results of GeoTek's evaluation, discussion of findings and provides geotechnical recommendations for foundation design and construction.

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

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

Respectfully submitted,  
**GeoTek, Inc.**



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Appendix A – Log of Exploratory Borings

Appendix B – Results of Laboratory Testing

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## **I. PURPOSE AND SCOPE OF SERVICES**

The purpose of this study was to evaluate the geotechnical engineering and geologic conditions at the project site, as outlined in GeoTek's proposal P-0201823-CR, dated February 2, 2023. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- Site exploration consisting of the excavation, logging, and sampling of seven (7) exploratory test borings extending to depths ranging from about 15 to 51.5 feet below grade,
- Excavation of four (4) additional borings to a depth of about five (5) feet below grade and performing an infiltration test in each boring,
- Laboratory testing of soil samples collected during the field investigation,
- Review and evaluation of site seismicity, and
- Preparation of this geotechnical report which presents GeoTek's findings, conclusions, and recommendations for this site.

## **2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT**

### **2.1 SITE DESCRIPTION**

The approximate 20.17-acre irregular-shaped project site is located northeast of the intersection of West Lancaster Boulevard and 30<sup>th</sup> Street West, in the City of Lancaster, Los Angeles County, California (see Figure 1). The site is referenced as Los Angeles County Assessor's Parcel Number (APN) 3153-034-087. Access to the site is available from West Lancaster Boulevard or 30<sup>th</sup> Street West, both paved improved streets located adjacent to the southern and western boundary of the site, respectively.

The site currently consists of vacant land; however, several small stockpiles of soil and some debris are located in the southwestern portion of the site and a large stockpile of soil is

located in the northwestern portion of the site. The site is situated within an area consisting of vacant land and residential development. Topographically, the site slopes gently downward to the northeast with less than five (5) feet of elevation differential across the site.

## **2.2 PROJECT DESCRIPTION**

Based upon review of a *Conceptual Site Plan* prepared by STK Architecture dated April 11, 2023 (see Figure 2), GeoTek understands the subject property is to be developed with 12, three-story residential apartment buildings with a total of 324 units along with a clubhouse, swimming pool, spa and sports courts area in the western portion of the project site; along with 23, three-story townhome buildings with a total of 105 units along with a clubhouse, a swimming pool, spa and playground area in the eastern portion of the site. Associated dog parks, lawn areas, putting greens, utilities, access and drive lanes, pavement, hardscape and landscape improvements are also proposed as part of the site development. Storm water disposal by means of on-site storm water disposal devices is anticipated for this project.

GeoTek has not been provided any foundation plans for the proposed development as of the date of this report. The proposed structures are anticipated to be of wood-frame or masonry construction and currently anticipated to utilize conventional shallow foundations and concrete slab-on-grade floors. For the purposes of this report, it is assumed maximum column and wall loads will be about 75 kips and 3.5 kips per foot, respectively. 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 upon past experience, grading of the site will involve cuts and fills generally less than about five (5) feet in depth, not including any recommended remedial grading. Subterranean facilities (underground parking) are not anticipated at this time. Sewage disposal is anticipated to be provided by a public sewer system. If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available.

### **3. FIELD EXPLORATION AND LABORATORY TESTING**

#### **3.1 FIELD EXPLORATION**

The field exploration for this report was conducted on March 30, 2023 and consisted of excavating seven (7) geotechnical exploratory test borings (Borings B-1 through B-7) with a truck-mounted hollow-stem auger drill rig to depths ranging from about 15 to 51.5 feet below the existing site grades. The approximate locations of the GeoTek borings are shown on the Exploration Location Map (Figure 2). An engineer 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 logs.

In Boring B-1, 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. The sampler penetration test data are presented on the Logs of Borings in Appendix A.

#### **Percolation Testing**

The four (4) infiltration borings were excavated to depths of approximately five (5) feet each. Infiltration testing was conducted in these borings in general accordance with the requirements of the County of Los Angeles LID Stormwater Infiltration Guidelines (2021).

The infiltration tests consisted of drilling an eight-inch diameter test hole to the desired depth and installing approximately two (2) 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 the boring. Water was then placed

in the borings to presoak the holes for at least one (1) hour prior to infiltration testing. Following presoaking, infiltration tests were performed which consisted of adding water to each test hole to a level of 12 inches above the bottom of the hole and measuring the water drop. Observed initial readings from the four (4) infiltration borings following the pre-soak period indicated the soils were able to infiltrate the full 12 inch column of water in greater than 10 minutes but not greater than 30 minutes, therefore a falling head test was deemed appropriate to conduct. The water drop was recorded for at least three (3) hours, in intervals dependent upon the full column of water fully infiltrating. Water was added to the test holes after each test interval up to the initial level. The stabilized rates encountered at each test location are summarized below:

<b>Boring No.</b>	<b>Measured Infiltration Rate* (inches per hour)</b>	<b>Depth of Boring (feet)</b>
Boring I-1	0.39	5.0
Boring I-2	0.04	5.0
Boring I-3	0.31	5.0
Boring I-4	0.48	5.0

\*Estimated as flow rate divided by the surface area of test section per GS200.1 Los Angeles County LID Stormwater Infiltration Guidelines (2021)

As required, reduction factors should be applied to the measured rates to account for the test method reliability (RFt), site variability (RFv), and long-term siltation (RFs). As noted in the LA County Manual (2021), an RFt of 2.0 should be utilized for the small diameter boring method. RFv and RFs should vary between 1.0 and 3.0. A RFv of 2.0 is preliminarily considered suitable, and the value to be selected for RFs should be based on the level of pre-treatment and maintenance for the proposed BMPs.

Assuming RFs of 1.0 and RFv and RFt values of 2.0, it is recommended a Total Reduction Factor of 5.0 be applied to the measured rates obtained. Detailed infiltration test data is included in Appendix C.

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 soils and relative density. Infiltration rates may be impacted by weight of equipment travelling over the soils, placement of engineered fill and other various factors. GeoTek assumes no responsibility or liability for the ultimate design or performance of the storm water facilities.

### **3.2 LABORATORY TESTING**

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

## **4. GEOLOGIC AND SOILS CONDITIONS**

### **4.1 REGIONAL SETTING**

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

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

More specific to the subject property, the site is located in an area geologically mapped to be underlain by alluvium (Hernandez, J.L., 2010). No active faults are shown in the immediate site vicinity on the maps reviewed for the area.

### **4.2 GENERAL SOIL CONDITIONS**

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

#### **4.2.1 Artificial Fill**

Artificial fill soils were encountered in all of the exploratory borings (Borings B-1 through B-7 and I-1 through I-4) and ranged in depth from approximately one (1) foot to 15.5 feet. The area of deeper artificial fill soils was encountered in the soil stockpile located in the northwest portion of the site. Obvious debris was not encountered in the two (2) borings excavated in this stockpile area. The artificial fill soils were found to consist of silt and clay (ML and CL soil types based upon the Unified Soil Classification System). The fine-grained soils (ML and CL soil types) were found to be soft to hard.

#### **4.2.2 Alluvium**

Alluvium was encountered in all of the exploratory and percolation test borings below the artificial fill soils to the maximum depth explored (51.5 feet). The alluvium was found to consist of interbedded layers of silt, clay and clayey sand (ML, CL and SC soil types based upon the Unified Soil Classification System). The fine-grained alluvial soils (ML and CL soil types) were found to be stiff to hard while the coarse-grained alluvium (SC soil types) was found to be medium dense.

Based on the results of laboratory testing, the upper site soils are considered to have a “low” (21-50) expansion potential (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). The test results are provided in Appendix B.

### **4.3 SURFACE WATER AND GROUNDWATER**

#### **4.3.1 Surface Water**

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

#### **4.3.2 Groundwater**

Groundwater was not encountered at the time of drilling in any of the exploratory borings conducted on the project site to the maximum depth explored (51.5 feet). Based on a review of select literature, it is estimated the historic high groundwater depth is approximately 30 feet below existing grade at the site. Based on the results of the field exploration, review of site area geomorphology and geology, groundwater is not anticipated to adversely affect the proposed improvements.

It is possible that seasonal variations (temperature, rainfall, etc.) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered in discontinuous zones within the fill and/or native materials. The groundwater levels presented in this report are the levels that were measured at the time of the field activities or as stated in the referenced source.

## **4.4 FAULTING AND SEISMICITY**

### **4.4.1 Faulting**

The geologic structure of the entire California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. However, the site is not situated within a State of California designated “Alquist-Priolo” Earthquake Fault Zone. The nearest known active fault is the San Andreas fault zone (Mojave section) located about 7.5 miles to the southwest.

### **4.4.2 Seismic Design Parameters**

The site is located at approximately 34.6970 degrees North Latitude and -118.1834 degrees West Longitude. Due to the presence of relatively dense/stiff site soils (blow counts ( $\tilde{N}$ ) in excess of 15), 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. Using the ASCE 7-16 option on the SEAOC/OSHPD website results in the values for  $S_{M1}$  and  $S_{D1}$  reported as “null-See Section 11.4.8” (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value  $S_1$  exceeds 0.2. The value  $S_1$  for the subject site exceeds 0.2.

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

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

<b>SITE SEISMIC PARAMETERS</b>	
Mapped 0.2 sec Period Spectral Acceleration, $S_s$	1.5g
Mapped 1.0 sec Period Spectral Acceleration, $S_1$	0.6g
Site Coefficient for Site Class "D", $F_a$	1
Site Coefficient for Site Class "D", $F_v$	1.7
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, $S_{MS}$	1.5g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, $S_{M1}$	1.02g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, $S_{DS}$	1.0g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, $S_{D1}$	0.68g
Peak Ground Acceleration ( $PGA_M$ )	0.664g
Seismic Design Category	D

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

#### **4.5 LIQUEFACTION**

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless and some low-plastic silt and clay soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction occurs, the liquefied soil/water matrix 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, plasticity, 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 and some low plastic silts and clays. Clay soils of the type encountered on the project site are typically not subject to liquefaction.

Groundwater was not encountered at the time of drilling within any of the exploratory borings drilled on the site to the maximum depth explored (51.5 feet). Based on a review of select literature, it is estimated the historic groundwater depth near the site is approximately 30 feet below existing site grade. For this report, a high groundwater depth of 30 feet was used in the

site liquefaction analysis. Liquefaction is not considered to be a hazard at the subject site due to the fine-grained (clayey) nature of the subsurface soil and the lack of a shallow groundwater level within the site.

#### **4.6 OTHER SEISMIC HAZARDS**

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

## **5. CONCLUSIONS AND RECOMMENDATIONS**

### **5.1 GENERAL**

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

### **5.2 EARTHWORK CONSIDERATIONS**

#### **5.2.1 General**

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

#### **5.2.2 Site Clearing**

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

### **5.2.3 Site Preparation**

It is recommended that all the undocumented fill soils and the upper native alluvial soils be removed beneath the planned building footprint to a depth of at least five (5) feet below existing grade, or three (3) feet below the base of the proposed foundations, whichever is greater. The lateral extent of this recommended over-excavation should extend at least five (5) feet beyond the building limits, where obtainable. Removal bottoms should be relatively uniform in soil type which is not visibly porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures.

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. 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.

### **5.2.4 Engineered Fill**

The on-site soils and existing stockpiles are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris, oversized materials (~6 inches) and other deleterious material. All areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in 6-to-8-inch loose lifts, moisture conditioned to about three (3) percent above the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D-1557 test procedures.

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

### **5.2.5 Excavation Characteristics**

Excavations in the on-site soils should be readily accomplished with heavy-duty earthmoving or excavating equipment in good operating condition. All excavations should be formed in accordance with current Cal-OSHA requirements.

### **5.2.6 Trench Excavations and Backfill**

Temporary trench excavations within the on-site materials should be stable at a 1.5:1 inclination for short durations during construction and where cuts do not exceed 15 feet in height. Deeper temporary excavations should be reviewed by GeoTek prior to their planned excavation to determine if supplemental recommendations or analysis are warranted. It is anticipated that temporary cuts to a maximum height of four (4) feet can be excavated vertically.

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

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined by ASTM D-1557 test procedures). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for pavement areas, if any, should be compacted to at least 95 percent relative compaction. On-site materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than six (6) inches are removed.

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

### **5.2.7 Shrinkage and Bulking**

Several factors will impact earthwork balancing on the site, including shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Site demolition and infrastructure removal operations may impact site earthwork quantities and needs to be accounted for.

Shrinkage is primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 10 to 15 percent may be considered for the site soils that will be excavated and replaced as engineered fill. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction.

A subsidence loss of up to about 0.2 foot is estimated for the site.

### 5.2.8 Grading Plan Review

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

## 5.3 DESIGN RECOMMENDATIONS

### 5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional shallow foundation system, in general conformance with the 2022 CBC, are presented below. Based on laboratory test results, subsequent to earthwork operations it is anticipated that the near-surface soils will have a “low” (0-20) expansion index.

Additional expansion index and soluble sulfate testing of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions. A summary of the foundation design recommendations is presented in the following table:

Design Parameters	“Low” Expansion Index
Minimum Building Foundation Depth (inches below lowest adjacent grade)	18
Minimum Foundation Width (Inches)	12*
Minimum Slab Thickness (actual)	5 inches
Minimum Slab Reinforcing	No. 3 bars at 18-inch centers placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and two near the bottom of the footing
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 110 percent of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

\*Code minimums per Table 1809.7 of the 2022 CBC need to be adhered to.

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and foundation reinforcement based on actual loading conditions. The following criteria for design of foundations are preliminary and should be re-evaluated based on the results of additional laboratory testing of samples obtained near finish pad grade.

Footings should have a minimum embedment depth of 18 inches. An allowable bearing capacity of 2,500 pounds per square foot (psf) may be used for design of isolated pad (column) and continuous footings. An increase of one-third may be applied when considering short-term seismic and wind loads.

Based upon review, a modulus of subgrade reaction ( $E_1$ ) of 150 pci may be used in the design of foundations. 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:

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

where:  $E_s$  = design modulus  
B = footing width

A grade beam, a minimum of 12 inches wide and 18 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.

Structural foundations should be designed in accordance with the 2022 CBC, and to withstand a total settlement (static plus seismic) of approximately 1 inch and a maximum differential settlement (static plus seismic) of approximately 0.5 inches over a horizontal distance of 40 feet.

The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 3,000 psf for footings founded on engineered fill. The upper one foot of foundation should be neglected for passive pressure unless covered by slab or pavement. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. Passive pressure and frictional resistance can be combined without reduction.

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

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g., stake penetrations, tears, punctures from walking on the vapor retarder placed atop the underlying aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6-mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeability) to achieve the desired performance level.

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

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

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

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**

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

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

### **5.3.3 Foundation Setbacks**

Minimum setbacks for all foundations, including offset from existing structures, should comply with the 2022 CBC or City of Lancaster requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The top outside edge of all footings should be set back a minimum of  $H/3$  (where  $H$  is the slope height) from the face of any descending slope. The setback should be at least five (5) feet and need not exceed 40 feet.

### **5.3.4 Soil Corrosivity**

The soil resistivity at this site was tested in the laboratory on one (1) sample collected during the field investigation. The results of the testing indicate that the on-site soils are considered “*highly corrosive*” (1,675 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.

### **5.3.5 Soil Sulfate Content**

The sulfate content was determined in the laboratory on one (1) sample collected during the field investigation. The results indicate that the water-soluble sulfate result is less than 0.1 percent by weight, which is considered “negligible” as per Table 4.2.1 of ACI 318. Based on the test results and Table 4.3.1 of ACI 318, no special recommendations for concrete are required for this project due to soil sulfate exposure.

## **5.4 RETAINING AND GARDEN WALL DESIGN AND CONSTRUCTION**

### **5.4.1 General Design Criteria**

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

Retaining wall foundations should be embedded a minimum of 18 inches into engineered fill and/or competent native soil. Retaining wall foundations should be designed in accordance with Section 5.3 of this report. Structural needs may govern and should be evaluated by the project structural engineer.

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

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of Section 5.2.4 in this report.

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

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

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

## 5.4.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to six (6) feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, or seismic events.

<b>ACTIVE EARTH PRESSURES</b>	
Surface Slope of Retained Materials (horizontal: vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	42
2:1	68

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

## 5.4.3 Retaining Wall Backfill and Drainage

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

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

Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four (4)-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one (1) cubic foot per linear foot of  $\frac{3}{4}$ - to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

#### **5.4.4 Restrained Retaining Walls**

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

#### **5.4.5 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.

### **5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS**

#### **5.5.1 Asphaltic Concrete Pavement**

Although planned final grades beneath the proposed parking, access roads and adjacent street improvements within the site are not yet known, the following preliminary pavement design recommendations are based on assumed Traffic Indexes of 5.0 for car parking areas and 6.0 for access drives. Preliminary pavement thickness design is based on the CalTrans Highway Design Manual (2018). An R-value of 15 was assumed for the determination of preliminary pavement sections for this report. Once the traffic loading information becomes more defined, revision to the pavement design recommendations may be warranted. It is recommended that the final

pavement design be based on R-value testing of the as-graded subgrade soils within the pavement areas.

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

<b>PRELIMINARY MINIMUM PAVEMENT SECTION</b>		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0	3.0	8.0
(Car Parking Areas)	3.5	7.0
6.0	3.5	10.5
(Automobile Access Lanes)	4.0	9.5

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

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 test procedures. All materials and methods of construction should conform to the requirements of the City of Lancaster.

### **5.5.2 Permeable Interlocking Concrete Pavers**

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

Where possible, the aggregate base should extend beyond the perimeter of the pavers a minimum distance of four (4) 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, 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 proposed vehicle parking and access lanes, it is recommended that a minimum of five (5) inches of PCC pavement over four (4) inches of aggregate base over 12 inches subgrade compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures be utilized. For vehicle service lanes, it is recommended that a minimum of six (6) inches of PCC pavement over four (4) inches of aggregate base over 12 inches subgrade compacted to at least 95 percent of maximum dry density be utilized. This section should also be used in heavy truck traffic areas such as fire lanes, trash dumpster pads and approaches.

Requirements of Section 90 of Caltrans Standard Specifications, and various ACI and ASTM standards regarding mixing and placing concrete should be followed. The PCC pavement should have a minimum modulus of rupture of 550 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, compaction of base material, placement and rolling of asphaltic concrete should be done in accordance with the City of Lancaster specifications, and under the observation and testing of GeoTek and a City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

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

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 test procedures. All materials and methods of construction should conform to the requirements of the City of Lancaster.

## **5.6 CONCRETE CONSTRUCTION**

### **5.6.1 General**

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

### **5.6.2 Concrete Mix Design**

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

### **5.6.3 Concrete Flatwork**

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

Flatwork should consist of a minimum four-inch (actual) thick concrete and the use of temperature and shrinkage control reinforcement is suggested. The project structural engineer should provide final design recommendations.

#### **5.6.4 Concrete Performance**

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

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

#### **5.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS**

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

- Observe site clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trench excavation backfill. Also, test the fill for density, relative compaction and moisture content.
- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

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

## **6. INTENT**

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

The scope of this 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-0201823-CR) dated February 2, 2023 and geotechnical engineering standards normally used on similar projects in this region.

## **7. LIMITATIONS**

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

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

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

## **8. SELECTED REFERENCES**

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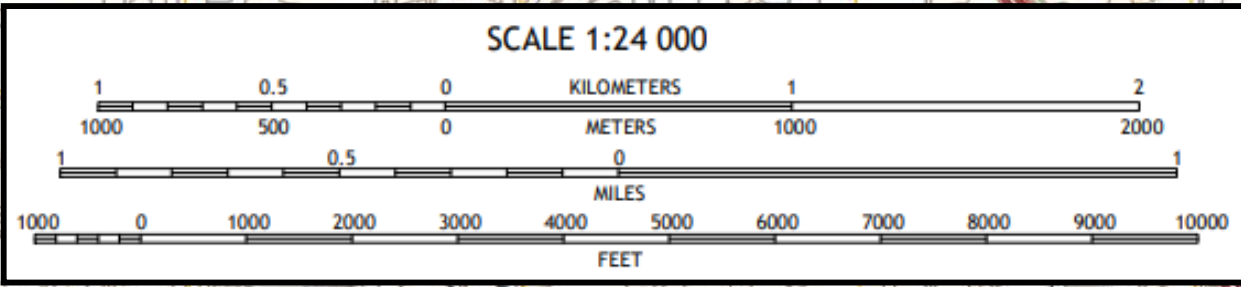
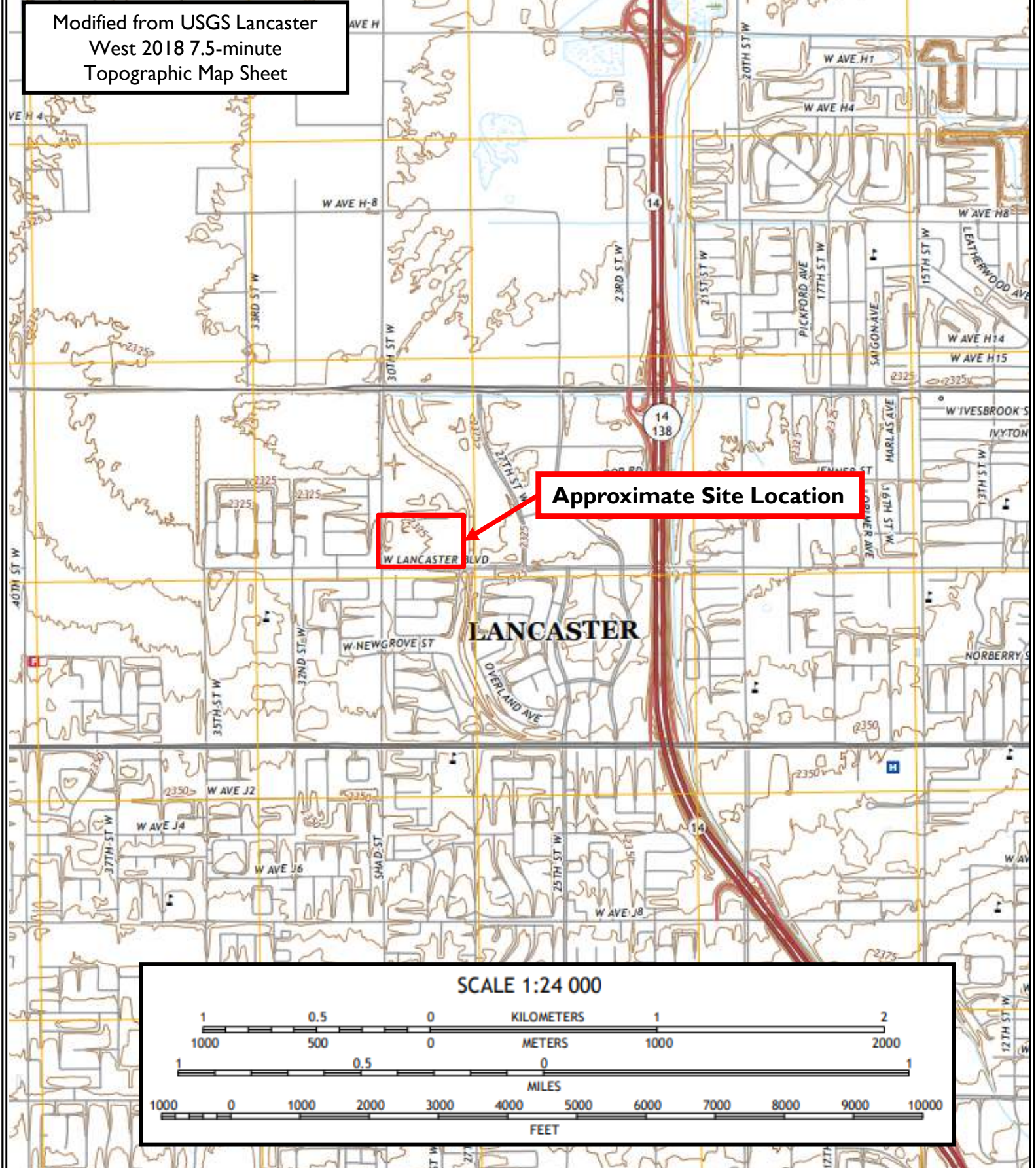
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Modified from USGS Lancaster  
West 2018 7.5-minute  
Topographic Map Sheet



**Guardian Capital**  
Assessor's Parcel Number 3153-034-087  
West Lancaster Blvd and 30<sup>th</sup> St West  
Lancaster, Los Angeles County, California

Project No. 3493-CR



**Figure I**  
Site Location  
Map





**Guardian Capital**  
 APN 3153-034-087  
 West Lancaster Blvd and 30<sup>th</sup> St West  
 Lancaster, Los Angeles County, California



**Figure 2**  
 Exploration  
 Location Map



Project No. 3493-CR

**APPENDIX A**

**LOGS OF EXPLORATORY BORINGS**

**Proposed Multi-Family Residential Development**

**APN 3153-034-087**

**Lancaster, Los Angeles County, California**

**Project No. 3493-CR**



## **A - FIELD TESTING AND SAMPLING PROCEDURES**

### The Modified Split-Barrel Sampler (Ring)

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

### The Standard Penetration Test (SPT) Sampler

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

### Bulk Samples (Large)

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

### Bulk Samples (Small)

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

## **B - BORING LOG LEGEND**

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

### SOILS

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

### GEOLOGIC

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

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-1  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-c sandy SILT, light greyish-brown, slightly moist, few rootlets, trace gravel			
9 12 14		R1	ML/SM	F-m sandy SILT to silty f-m SAND, light greyish-brown, slightly moist, very stiff to medium dense	21.8	78.4	SH, EI, MD EI=31	
5				ML	F-m sandy SILT, light brown/white, slightly moist, very stiff, few caliche stringers	20.1		
16 24 14		R2						
27 31 28		R3			F sandy SILT, light brown/white, slightly moist, hard	13.4	104.3	
10				ML/CL	F-m sandy clayey SILT to f-m sandy silty CLAY, light brown, slightly moist, very stiff	13.3	109.0	
8 14 21		R4						
				CL	Silty CLAY, brown, moist, very stiff, trace medium sand			
15								
4 12 26		R5						
20					F sandy/silty CLAY, brown, moist, very stiff, mottled	10.4	108.4	
9 19 14		R6						
25					Silty CLAY, dark brown, moist, medium stiff to stiff, trace fine sand	22.7		
2 3 5		S1						
30					stiff			
3 4 6		S2						

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES				Boring No.: B-1 (continued)	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)
35		4 5 8	S3	CL	F sandy/silty CLAY, dark brown, moist, stiff	17.4		
40		5 12 12	S4	SC/CL	Clayey/silty f-m SAND to f-m sandy/silty CLAY, light brown, moist, medium dense to stiff			
45		4 4 6	S5	CL	F sandy/silty CLAY, dark brown, moist, stiff	27.5		
50		2 4 7	S6					
<b>BORING TERMINATED AT 51.5 FEET</b>								
No groundwater encountered Boring backfilled with soil cuttings								

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-2	Laboratory Testing			
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others	
<b>MATERIAL DESCRIPTION AND COMMENTS</b>									
0				ML	<b>Undocumented Fill:</b> F-c sandy SILT, light brown, slightly moist, few rootlets, trace gravel				
		8 16 24	R1	ML	<b>Alluvium:</b> F-m sandy/clayey SILT, light greyish-brown, slightly moist, very stiff	23.7	69.2		
		20 35 37	R2		F sandy SILT, light brown/white, slightly moist, hard, trace gravel	13.5	97.3		
		8 10 16	R3		Clayey f sandy SILT, light brown/white, slightly moist, very stiff	9.3	99.8		
		6 7	R4			19.9	104.0		
10		10		CL	F sandy/silty CLAY, brown, moist, stiff				
		7 10 11	R5						
15		6 8 11	R6			16.1	110.1		
20	<b>BORING TERMINATED AT 20.0 FEET</b>								
	No groundwater encountered Boring backfilled with soil cuttings								
25									
30									

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-3  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-c sandy SILT, light greyish-brown, slightly moist, trace rootlets			
				<b>Alluvium:</b>				
		21	R1	ML	F-m sandy/clayey SILT, light greyish-brown, slightly moist, very stiff	15.6	94.4	
		35		CL	F-m sandy/silty CLAY, light grey-brown, slightly moist, hard			
		35						
5								
		22	R2			21.5	83.4	
		44						
		50/5"						
		7	R3	ML	F sandy/clayey SILT, light brown/white, slightly moist, hard	14.1	95.6	
		14						
		50/5"						
10								
		7	R4		Becomes very stiff	9.4	94.0	
		13						
		11						
15								
		8	R5	CL/ML	F sandy/silty CLAY to f sandy clayey SILT, brown, moist, very stiff			
		12						
		16						
20								
		10	R6		Trace gravel	11.6	105.3	
		14						
		15						
					<b>BORING TERMINATED AT 21.5 FEET</b>			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-4	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0					<b>Undocumented Fill:</b>			SR
		10 16 35	R1	ML	F-m sandy/clayey SILT, light brown, slightly moist, hard, few caliche nodules	14.0	93.9	
		12 14 19	R2	ML	<b>Alluvium:</b> F-m sandy SILT, light brown/white, slightly moist, hard brown, very stiff	16.4	86.2	
		17 32	R3	CL	F-m sandy/silty CLAY, dark grey-brown, slightly moist, very stiff	10.1	97.5	
		16		ML	F sandy/clayey SILT, light brown/white, slightly moist, hard			
		5 8 11	R4	CL	F sandy/silty CLAY, greyish white-brown, moist, stiff, trace gravel	16.7	92.3	
		8 10 14	R5		F-m sandy/silty CLAY, brown, moist, very stiff, trace oxidation staining			
		7 13 18	R6		Silty CLAY, brown, moist, very stiff, mottled			
20	<b>BORING TERMINATED AT 20.0 FEET</b>							
	No groundwater encountered Boring backfilled with soil cuttings							
25								
30								

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-5  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-c sandy SILT, light greyish-brown, slightly moist, few rootlets			
		33 50/4"	R1	ML	F-m sandy clayey SILT, light brown, slightly moist, trace gravel, trace rootlets, few caliche nodules	11.1	98.4	
				ML	<b>Alluvium:</b> F-m sandy SILT, light brown/white, slightly moist, very stiff			
5		25 50/5"	R2	CL/ML	F sandy/silty CLAY to f sandy clayey SILT, brown, moist, very stiff	19.1	104.6	
		13 18 18	R3	CL	F sandy/silty CLAY, dark grey-brown, moist, very stiff	23.5	92.1	
10		20 28 20	R4	ML	Clayey SILT, light brown/white, moist, hard			
15		7 9 13	R5	CL	F sandy silty CLAY, brown, moist, stiff, trace oxidation staining	17.0	107.3	
20		5 12 24	R6		Silty CLAY, light to dark brown, moist, very stiff, trace pinhole porosity			
					<b>BORING TERMINATED AT 21.5 FEET</b>			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-6  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0					<b>Undocumented Stockpile Fill:</b>			
				CL	F-m sandy/silty CLAY, brown, moist			
5				ML	F sandy/clayey SILT, light brown, slightly moist, few gravel			
10				ML/CL	F-m sandy/clayey SILT to f-m sandy/silty CLAY, light brown, slightly moist			
15				CL	<b>Alluvium:</b> F sandy silty CLAY, brown, moist			
20					<b>BORING TERMINATED AT 20.0 FEET</b>			
25					No groundwater encountered Boring backfilled with soil cuttings			
30								

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES				Boring No.: B-7	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0					<b>Undocumented Stockpile Fill:</b>			
5				ML	F-m sandy SILT, light brown, slightly moist, trace gravel			
10				CL/ML	F sandy/silty CLAY to f sandy/clayey SILT, light brown, slightly moist  dry to slightly moist			
15				ML	<b>Alluvium:</b> F-m sandy SILT, light greyish-brown, slightly moist, few rootlets			
20					<b>BORING TERMINATED AT 15.0 FEET</b>			
25					No groundwater encountered Boring backfilled with soil cuttings			
30								

**LEGEND**

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

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

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: I-1  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-c sandy SILT, light greyish-brown, slightly moist			
				ML	<b>Alluvium:</b> F sandy SILT, light brown, slightly moist			
5					<b>BORING TERMINATED AT 5 FEET</b>  No groundwater encountered Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

<b>LEGEND</b>	<b>Sample type:</b>	<input type="checkbox"/>	---Ring	<input type="checkbox"/>	---SPT	<input checked="" type="checkbox"/>	---Small Bulk	<input checked="" type="checkbox"/>	---Large Bulk	<input type="checkbox"/>	---No Recovery	<input type="checkbox"/>	---Water Table
	<b>Lab testing:</b>	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test	SR = Sulfate/Resistivity Test	SH = Shear Test	HC = Consolidation	MD = Maximum Density				

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: I-2  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-m sandy SILT, light greyish-brown, slightly moist			
				ML	<b>Alluvium:</b> F sandy SILT, light brown, slightly moist			
				CL	F-m sandy/silty CLAY, brown, moist			
5	<b>BORING TERMINATED AT 5 FEET</b>							
	No groundwater encountered Boring set with pipe, sock, and gravel							
10								
15								
20								
25								
30								

<b>LEGEND</b>	<b>Sample type:</b>	<input type="checkbox"/> ---Ring	<input type="checkbox"/> ---SPT	<input checked="" type="checkbox"/> ---Small Bulk	<input checked="" type="checkbox"/> ---Large Bulk	<input type="checkbox"/> ---No Recovery	<input type="checkbox"/> ---Water Table	
	<b>Lab testing:</b>	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test	SR = Sulfate/Resistivity Test	SH = Shear Test	HC = Consolidation

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: I-3  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-m sandy SILT, light greyish-brown, slightly moist			
				ML	<b>Alluvium:</b> F sandy/clayey SILT, light brown, slightly moist			
5					F sandy SILT, light brown, slightly moist			
					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

<b>LEGEND</b>	<b>Sample type:</b>	<input type="checkbox"/> ---Ring	<input type="checkbox"/> ---SPT	<input checked="" type="checkbox"/> ---Small Bulk	<input checked="" type="checkbox"/> ---Large Bulk	<input type="checkbox"/> ---No Recovery	<input type="checkbox"/> ---Water Table	
	<b>Lab testing:</b>	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test	SR = Sulfate/Resistivity Test	SH = Shear Test	HC = Consolidation

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** Guardian Capital  
**PROJECT NAME:** APN 3153-034-087  
**PROJECT NO.:** 3493-CR  
**LOCATION:** See Boring Location Map

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** KIG  
**OPERATOR:** Ish  
**RIG TYPE:** CME 75  
**DATE:** 3/30/2023

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: I-4  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Undocumented Fill:</b> F-c sandy SILT, light greyish-brown, slightly moist			
				ML	<b>Alluvium:</b> F sandy SILT, light brown, slightly moist			
5	7 6 7	SI			trace clay			% Passing #200 = 61.3
5	<b>BORING TERMINATED AT 5 FEET</b>							
					No groundwater encountered Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

<b>LEGEND</b>	<b>Sample type:</b>	<input checked="" type="checkbox"/> ---Ring	<input type="checkbox"/> ---SPT	<input checked="" type="checkbox"/> ---Small Bulk	<input checked="" type="checkbox"/> ---Large Bulk	<input type="checkbox"/> ---No Recovery	<input type="checkbox"/> ---Water Table	
	<b>Lab testing:</b>	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test	SR = Sulfate/Resistivity Test	SH = Shear Test	HC = Consolidation

**APPENDIX B**

**RESULTS OF LABORATORY TESTING**

**Proposed Multi-Family Residential Development**

**APN 3153-034-087**

**Lancaster, Los Angeles County, California**

**Project No. 3493-CR**



## SUMMARY OF LABORATORY TESTING

### Classification

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

### Direct Shear

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

### Expansion Index

Expansion Index testing was performed on one (1) bulk soil sample obtained from the site. 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	Sandy Silt (ML)	31	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 D 1140 test procedures. The results of this testing are presented on the boring logs in Appendix A.

### Moisture-Density Relationship

Laboratory testing was performed on one (1) bulk sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with test method ASTM Test Procedure D 1557. The results 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 in general accordance with ASTM D4327 test procedures. Resistivity testing was completed by others 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.

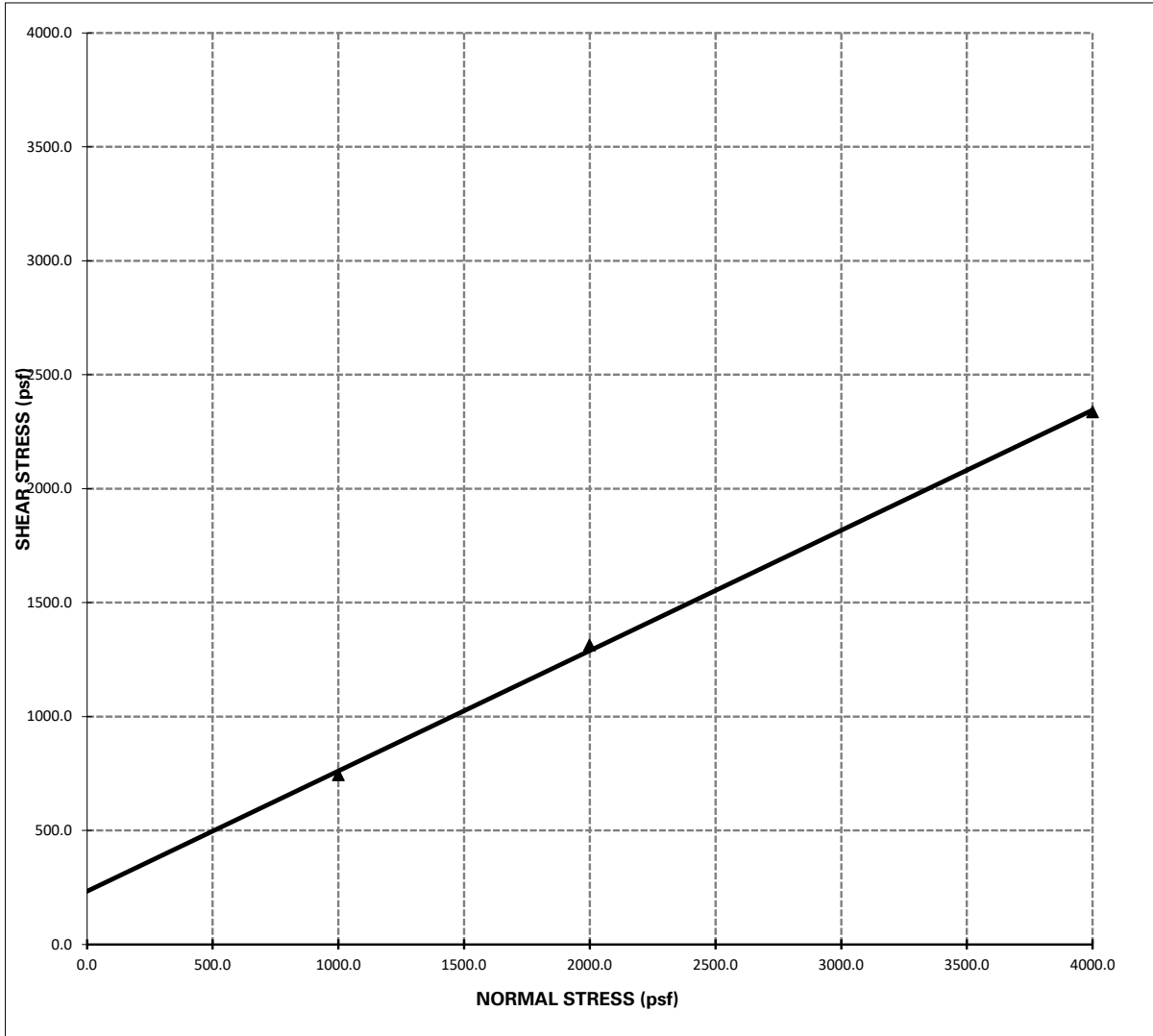
Boring No.	Depth (ft.)	pH ASTM G51	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-4	0-5	8.5	114.2	0.0290	1,675



# DIRECT SHEAR TEST

**Project Name:** APN 3153-034-087, Lancaster  
**Project Number:** 3493-CR

**Sample Location:** B-1 @ 0-5 feet  
**Date Tested:** 4/14/2023



**Shear Strength:**  $\Phi = 28^\circ$ , **C = 233 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:** Guardian Capital  
**Project Number:** 3493-CR  
**Project Location:** APN 3153-034-087, Lancaster

**Tested/ Checked By:** BC Lab No Corona  
**Date Tested:** 4/10/2023  
**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)	723.6
Weight of ring (gm)	368.6
Net weight of sample (gm)	<b>355.0</b>
Wet Density, lb / ft3 (C*0.3016)	<b>107.1</b>
Dry Density, lb / ft3 (D/1.F)	<b>93.1</b>

### SATURATION DETERMINATION

Moisture Content, %	15.0
Specific Gravity, assumed	<b>2.70</b>
Unit Wt. of Water @ 20 °C, (pcf)	<b>62.4</b>
% Saturation	<b>50.0</b>

READINGS		
DATE	TIME	READING
4/10/2023		0.2570
4/10/2023		0.2560
4/11/2023		0.2870

Initial  
10 min/Dry  
  
  
  
Final

### FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
780.9	<b>31.1</b>

**EXPANSION INDEX = 31**



**Report No: PTR:23-00069-S01**

# Proctor Report

**Client:** Guardian Capital  
 ATTN: Alex Glick  
 Carlsbad CA 92008

**CC:**

**Project:** 3493-CR  
 APN 3153-034-087

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

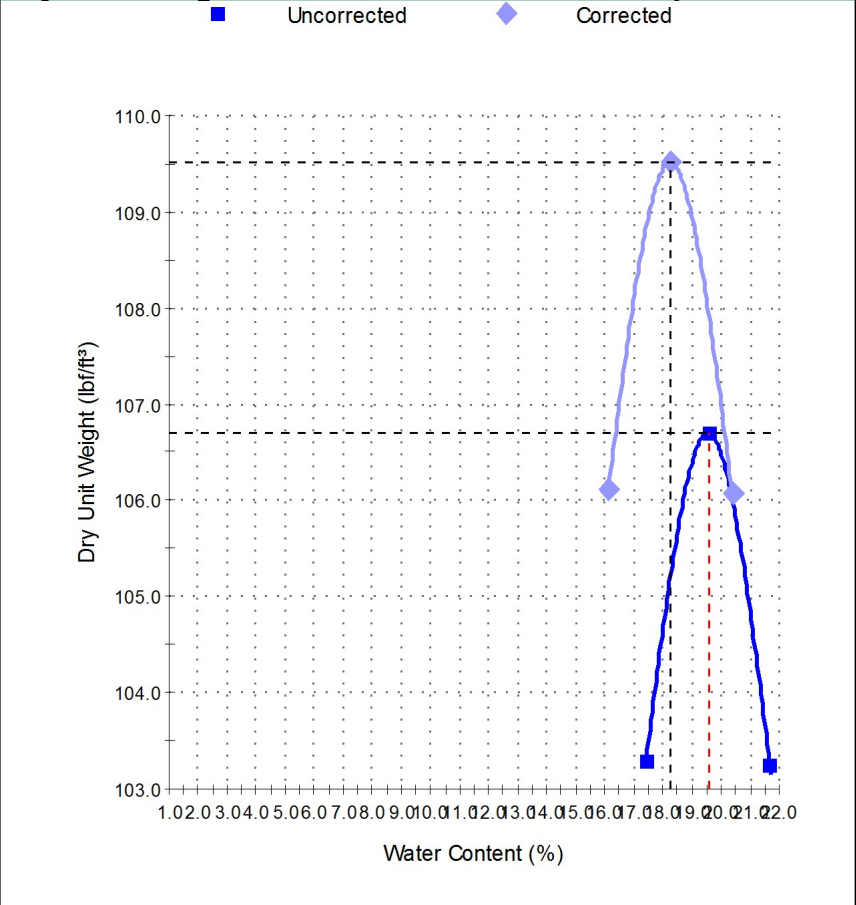
## Sample Details

**Sample ID:** 23-00069-S01      **Date Sampled:** 3/30/2023

**Sampled By:** Kase Gebbie

**Location:** B-1 @ 0-5 feet / Sandy silt

## Dry Unit Weight - Water Content Relationship



## Test Results

ASTM D 1557	
<b>Maximum Dry Unit Weight (lb/ft³):</b>	<b>106.7</b>
<b>Optimum Water Content (%):</b>	<b>19.6</b>
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	7
Passing Sieve No 4 (4.75mm) (%):	93
Tested By:	Mycheal Phillips
Date Tested:	4/10/2023
ASTM D 4718	
<b>Corrected Maximum Dry Unit Weight (lb/ft³):</b>	<b>109.5</b>
<b>Corrected Optimum Water Content (%):</b>	<b>18.3</b>
Specific Gravity (Oversize):	2.70
Sieve Size (Oversize):	No 4
Oversize Particles (%):	7

## Comments



# **Results Only Soil Testing for APN 3153-034-087 West Lancaster Blvd & 30th St W**

**April 5, 2023**

**Prepared for:**

**Bruce Hick  
Geotek USA  
1548 North Maple Street  
Corona, CA 92280  
bhick@geotekusa.com**

**Project X Job#: S230403G  
Client Job or PO#: 3493-CR Guardian Capital**

Respectfully Submitted,

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





## Soil Analysis Lab Results

Client: Geotek USA  
 Job Name: APN 3153-034-087 West Lancaster Blvd & 30th St W  
 Client Job Number: 3493-CR Guardian Capital  
 Project X Job Number: S230403G  
 April 5, 2023

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 <sub>4</sub> <sup>2-</sup>		Chlorides Cl <sup>-</sup>		Resistivity As Rec'd   Minimum		pH	Redox	Sulfide S <sup>2-</sup>	Nitrate NO <sub>3</sub> <sup>-</sup>	Ammonium NH <sub>4</sub> <sup>+</sup>	Lithium Li <sup>+</sup>	Sodium Na <sup>+</sup>	Potassium K <sup>+</sup>	Magnesium Mg <sup>2+</sup>	Calcium Ca <sup>2+</sup>	Fluoride F <sub>2</sub> <sup>-</sup>	Phosphate PO <sub>4</sub> <sup>3-</sup>
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-4	0-5	289.7	0.0290	114.2	0.0114	15,410	1,675	8.5	106	8.4	71.2	4.0	ND	872.1	678.0	192.7	1,252.1	14.1	4.1

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 which 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.

**APPENDIX C**

**PERCOLATION TEST DATA**

**Proposed Multi-Family Residential Development**

**APN 3153-034-087**

**Lancaster, Los Angeles County, California**

**Project No. 3493-CR**



**GeoTek, Inc.**  
**INFILTRATION TESTING**

Depth of Hole (D<sub>r</sub>) in.

60

Boring Radius, in.

4

**Test No. I-I**

**3493-CR**

<b>Trial No.</b>	<b>Time Interval (ΔT) Min.</b>	<b>Initial Depth (D<sub>0</sub>) in.</b>	<b>Final Depth (D<sub>r</sub>) in.</b>	<b>Change In Level (ΔD) in.</b>	<b>Flow Rate (in<sup>3</sup>/hr)</b>	<b>Surface Area of Test Section (in<sup>2</sup>)</b>	<b>Field Infiltration Rate (in/hr)</b>
I+ Hour Presoak	60+	48.00	60.000	12.00			
30 min test	30	48.00	56.250	8.25			
1	30	48.00	52.375	4.38	439.82	241.90	1.82
2	30	48.00	51.125	3.13	314.16	273.32	1.15
3	30	48.00	49.750	1.75	175.93	307.88	0.57
4	30	48.00	49.250	1.25	125.66	320.44	0.39
5	30	48.00	49.250	1.25	125.66	320.44	0.39
6	30	48.00	49.250	1.25	125.66	320.44	0.39

**GeoTek, Inc.**  
**INFILTRATION TESTING**

Depth of Hole (D<sub>r</sub>) in.

60

Boring Radius, in.

4

**Test No. I-2**

**3493-CR**

<b>Trial No.</b>	<b>Time Interval (ΔT) Min.</b>	<b>Initial Depth (D<sub>0</sub>) in.</b>	<b>Final Depth (D<sub>r</sub>) in.</b>	<b>Change In Level (ΔD) in.</b>	<b>Flow Rate (in<sup>3</sup>/hr)</b>	<b>Surface Area of Test Section (in<sup>2</sup>)</b>	<b>Field Infiltration Rate (in/hr)</b>
I+ Hour Presoak	60+	48.00	48.625	0.63			
30 min test	30	48.00	48.250	0.25			
1	30	48.00	48.250	0.25	25.13	345.57	0.07
2	30	48.00	48.250	0.25	25.13	345.57	0.07
3	30	48.00	48.125	0.13	12.57	348.72	0.04
4	30	48.00	48.125	0.13	12.57	348.72	0.04
5	30	48.00	48.125	0.13	12.57	348.72	0.04
6	30	48.00	48.125	0.13	12.57	348.72	0.04

**GeoTek, Inc.**  
**INFILTRATION TESTING**

Depth of Hole (D<sub>r</sub>) in.

60

Boring Radius, in.

4

**Test No. I-3**

**3493-CR**

<b>Trial No.</b>	<b>Time Interval (ΔT) Min.</b>	<b>Initial Depth (D<sub>0</sub>) in.</b>	<b>Final Depth (D<sub>r</sub>) in.</b>	<b>Change In Level (ΔD) in.</b>	<b>Flow Rate (in<sup>3</sup>/hr)</b>	<b>Surface Area of Test Section (in<sup>2</sup>)</b>	<b>Field Infiltration Rate (in/hr)</b>
1+ Hour Presoak	60+	48.00	60.000	12.00			
30 min test	30	48.00	54.375	6.38			
1	30	48.00	50.125	2.13	213.63	298.45	0.72
2	30	48.00	49.500	1.50	150.80	314.16	0.48
3	30	48.00	49.125	1.13	113.10	323.58	0.35
4	30	48.00	49.000	1.00	100.53	326.73	0.31
5	30	48.00	49.000	1.00	100.53	326.73	0.31
6	30	48.00	49.000	1.00	100.53	326.73	0.31

**GeoTek, Inc.**  
**INFILTRATION TESTING**

Depth of Hole (D<sub>r</sub>) in.

60

Boring Radius, in.

4

**Test No. I-4**

**3493-CR**

<b>Trial No.</b>	<b>Time Interval (ΔT) Min.</b>	<b>Initial Depth (D<sub>0</sub>) in.</b>	<b>Final Depth (D<sub>r</sub>) in.</b>	<b>Change In Level (ΔD) in.</b>	<b>Flow Rate (in<sup>3</sup>/hr)</b>	<b>Surface Area of Test Section (in<sup>2</sup>)</b>	<b>Field Infiltration Rate (in/hr)</b>
I+ Hour Presoak	60+	48.00	60.000	12.00			
30 min test	30	48.00	55.500	7.50			
1	30	48.00	52.750	4.75	477.52	232.48	2.05
2	30	48.00	51.250	3.25	326.73	270.18	1.21
3	30	48.00	51.000	3.00	301.59	276.46	1.09
4	30	48.00	49.625	1.63	163.36	311.02	0.53
5	30	48.00	49.500	1.50	150.80	314.16	0.48
6	30	48.00	49.500	1.50	150.80	314.16	0.48

**APPENDIX D**

**GENERAL GRADING GUIDELINES**

**Proposed Multi-Family Residential Development**

**APN 3153-034-087**

**Lancaster, Los Angeles County, California**

**Project No. 3493-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 and the guidelines presented below.

### **Preconstruction Meeting**

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

### **Grading Observation and Testing**

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

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

### Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site, it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative. Typical procedures are similar to those indicated on Plate G-4.

**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 (see Plates F-1, F-2 and F-3) 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.

**Subdrainage**

1. Subdrainage systems should be provided in canyon bottoms prior to placing fill, and behind buttress and stabilization fills and in other areas indicated in the report. Subdrains should conform to Plates F-1 and F-5, and be acceptable to our representative.
2. For canyon subdrains, runs less than 500 feet may use six-inch pipe. Typically, runs in excess of 500 feet should have the lower end as eight-inch minimum.
3. Filter material should be clean, 1/2 to 1-inch gravel wrapped in a suitable filter fabric. Class 2 permeable filter material per California Department of Transportation Standards tested by this office to verify its suitability, may be used without filter fabric. A sample of the material should be provided to the Soils Engineer by the contractor at least two working days before it is delivered to the site. The filter should be clean with a wide range of sizes.
4. Approximate delineation of anticipated subdrain locations may be offered at 40-scale plan review stage. During grading, this office would evaluate the necessity of placing additional drains.
5. All subdrainage systems should be observed by our representative during construction and prior to covering with compacted fill.
6. Subdrains should outlet into storm drains where possible. Outlets should be located and protected. The need for backflow preventers should be assessed during construction.
7. Consideration should be given to having subdrains located by the project surveyors.

**Fill Placement**

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).

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

### **Keyways, Buttress and Stabilization Fills**

Keyways are needed to provide support for fill slope and various corrective procedures.

1. Side-hill fills should have an equipment-width key at their toe excavated through all surficial soil and into competent material and tilted back into the hill (Plates F-2, F-3). As the fill is elevated, it should be benched through surficial soil and slopewash, and into competent bedrock or other material deemed suitable by our representatives (See Plates F-1, F-2, and F-3).
2. Fill over cut slopes should be constructed in the following manner:
  - a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
  - b) A key at least one and one-half (1.5) equipment width wide (or as needed for compaction) and tipped at least one (1) foot into slope, should be excavated into competent materials and observed by our representative.
  - c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary. The contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation. (see Plate F-3 for schematic details.)
3. Daylight cut lots above descending natural slopes may require removal and replacement of the outer portion of the lot. A schematic diagram for this condition is presented on Plate F-2.
4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate F-2.
5. All fill slopes should be provided with a key unless within the body of a larger overall fill mass. Please refer to Plate F-3 for specific guidelines.

Anticipated buttress and stabilization fills are discussed in the text of the report. The need to stabilize other proposed cut slopes will be evaluated during construction. Plate F-5 shows a schematic of buttress construction.

1. All backcuts should be excavated at gradients of 1:1 or flatter. The backcut configuration should be determined based on the design, exposed conditions, and need to maintain a minimum fill width and provide working room for the equipment.
2. On longer slopes, backcuts and keyways should be excavated in maximum 250 feet long segments. The specific configurations will be determined during construction.
3. All keys should be a minimum of two (2) feet deep at the toe and slope toward the heel at least one foot or two (2%) percent, whichever is greater.
4. Subdrains are to be placed for all stabilization slopes exceeding 10 feet in height. Lower slopes are subject to review. Drains may be required. Guidelines for subdrains are presented on Plate E-5.
5. Benching of backcuts during fill placement is required.

**Lot Capping**

1. When practical, the upper three (3) feet of material placed below finish grade should be comprised of the least expansive material available. Preferably, highly and very highly expansive materials should not be used. We will attempt to offer advice based on visual evaluations of the materials during grading, but it must be realized that laboratory testing is needed to evaluate the expansive potential of soil. Minimally, this testing takes two (2) to four (4) days to complete.
2. Transition lots (cut and fill) both per plan and those created by remedial grading (e.g., lots above stabilization fills, along daylight lines, above natural slopes, etc.) should be capped with a minimum three foot thick compacted fill blanket.
3. Cut pads should be observed by our representative(s) to evaluate the need for overexcavation and replacement with fill. This may be necessary to reduce water infiltration into highly fractured bedrock or other permeable zones, and/or due to differing expansive potential of materials beneath a structure. The overexcavation should be at least three feet. Deeper overexcavation may be recommended in some cases.

**ROCK PLACEMENT AND ROCK FILL GUIDELINES**

It is anticipated that large quantities of oversize material would be generated during grading. It's likely that such materials may require special handling for burial. Although alternatives may be developed in the field, the following methods of rock disposal are recommended on a preliminary basis.

**Limited Larger Rock**

When materials encountered are principally soil with limited quantities of larger rock fragments or boulders, placement in windrows is recommended. The following procedures should be applied:

1. Oversize rock (greater than 8 inches) should be placed in windrows.
  - a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.

- b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).
  - c) The maximum rock size allowed in windrows is four feet
2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate E-4).
3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.
4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.
  - a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.
  - b) The over size rock trenches should be no closer together than 15 feet from any slope face.
  - c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.
  - d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

### Structural Rock Fills

If the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inches in one dimension, then placement using conventional soil fill methods with isolated windrows would not be feasible. In such cases the following could be considered:

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within seven (7) feet of finished grade, they may affect foundation design.
2. Rock fills are required to be placed in horizontal layers that should **not exceed two feet in thickness, or the maximum rock size present, which ever is less**. All rocks exceeding two feet should be broken down to a smaller size, windrowed (see above), or disposed of in non-structural fill areas. Localized larger rock up to 3 feet in largest dimension may be placed in rock fill as follows:

- a) individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill,
  - b) loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer,
  - c) the portion of the rock above grade is covered with a second lift.
3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

### **Compaction Procedures**

Compaction of rock fills is largely procedural. The following procedures have been found to generally produce satisfactory compaction.

1. Provisions for routing of construction traffic over the fill should be implemented.
  - a) Placement should be by rock trucks crossing the lift being placed and dumping at its edge.
  - b) The trucks should be routed so that each pass across the fill is via a different path and that all areas are uniformly traversed.
  - c) The dumped piles should be knocked down and spread by a large dozer (D-8 or larger suggested). (Water should be applied before and during spreading.)
2. Rock fill should be generously watered (sluiced)
  - a) Water should be applied by water trucks to the:
    - i) dump piles,
    - ii) front face of the lift being placed and,
    - iii) surface of the fill prior to compaction.
  - b) No material should be placed without adequate water.
  - c) The number of water trucks and water supply should be sufficient to provide constant water.
  - d) Rock fill placement should be suspended when water trucks are unavailable:
    - i) for more than 5 minutes straight, or,
    - ii) for more than 10 minutes/hour.
3. In addition to the truck pattern and at the discretion of the soil engineer, large, rubber-tired compactors may be required.
  - a) The need for this equipment will depend largely on the ability of the operators to provide complete and uniform coverage by wheel rolling with the trucks.
  - b) Other large compactors will also be considered by the soil engineer provided that required compaction is achieved.
4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:

- a) the general segregation of rock size,
  - b) for any unfilled spaces between the large blocks, and
  - c) the matrix compaction and moisture content.
5. Test fills may be required to evaluate relative compaction of finer grained zones or as deemed appropriate by the soil engineer.
    - a) A lift should be constructed by the methods proposed, as proposed
  6. Frequency of the test trenching is to be at the discretion of the soil engineer. Control areas may be used to evaluate the contractor's procedures.
  7. A minimum horizontal distance of 15 feet should be maintained from the face of the rock fill and any finish slope face. At least the outer 15 feet should be built of conventional fill materials.

### **Piping Potential and Filter Blankets**

Where conventional fill is placed over rock fill, the potential for piping (migration) of the fine-grained material from the conventional fill into rock fills will need to be addressed.

The potential for particle migration is related to the grain size comparisons of the materials present and in contact with each other. Provided that 15 percent of the finer soil is larger than the effective pore size of the coarse soil, then particle migration is substantially mitigated. This can be accomplished with a well-graded matrix material for the rock fill and a zone of fill similar to the matrix above it. The specific gradation of the fill materials placed during grading must be known to evaluate the need for any type of filter that may be necessary to cap the rock fills. This, unfortunately, can only be accurately determined during construction.

In the event that poorly graded matrix is used in the rock fills, properly graded filter blankets 2 to 3 feet thick separating rock fills and conventional fill may be needed. As an alternative, use of two layers of filter fabric (Mirafi 700 x or equivalent) could be employed on top of the rock fill. In order to mitigate excess puncturing, the surface of the rock fill should be well broken down and smoothed prior to placing the filter fabric. The first layer of the fabric may then be placed and covered with relatively permeable fill material (with respect to overlying material) 1 to 2 feet thick. The relative permeable material should be compacted to fill standards. The second layer of fabric should be placed and conventional fill placement continued.

### **Subdrainage**

Rock fill areas should be tied to a subdrainage system. If conventional fill is placed that separates the rock from the main canyon subdrain, then a secondary system should be installed. A system consisting of an adequately graded base (3 to 4 percent to the lower side) with a collector system and outlets may suffice.

Additionally, at approximately every 25 foot vertical interval, a collector system with outlets should be placed at the interface of the rock fill and the conventional fill blanketing a fill slope

### **Monitoring**

Depending upon the depth of the rock fill and other factors, monitoring for settlement of the fill areas may be needed following completion of grading. Typically, if rock fill depths exceed 40 feet, monitoring would be recommended prior to construction of any settlement sensitive improvements. Delays of 3 to 6 months or longer can be expected prior to the start of construction.

### **UTILITY TRENCH CONSTRUCTION AND BACKFILL**

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

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

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

The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.

3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.

4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractor's procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractor's attention.

### **JOB SAFETY**

#### **General**

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

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

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

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

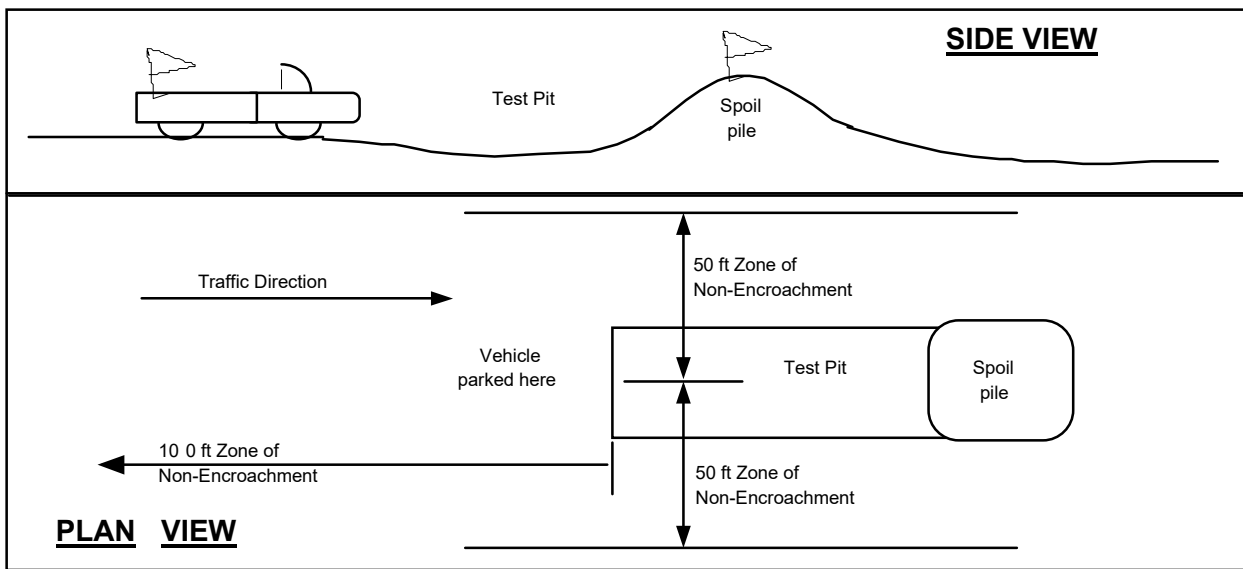
#### **Test Pits Location, Orientation and Clearance**

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

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

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

**TEST PIT SAFETY PLAN**



**Slope Tests**

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

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

**Trench Safety**

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

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

Our personnel are directed not to enter any excavation which;

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

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractor's representative will then be contacted in an effort to affect 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 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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