PRELIMINARY GEOTECHNICAL EVALUATION AND INFILTRATION STUDY

FOR THE

PROPOSED COMMERCIAL DEVELOPMENT
DEEMARCO PROJECT
21705 CAJALCO ROAD
PERRIS AREA, RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

ALABBASI CONSTRUCTION & ENGINEERING
764 W. RAMONA EXPRESSWAY, SUITE C
PERRIS, CALIFORNIA 92571

PREPARED BY

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

PROJECT NO. 2550-CR **DECEMBER 29, 2020**

GEOTEK

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

December 29, 2020
Project No. 2550-CR

Alabbasi Construction & Engineering
764 W. Ramona Expressway, Suite C
Perris, California 92571

Attention: Ms. Corinne Mostad

Subject: Preliminary Geotechnical Evaluation and Infiltration Study
Proposed Commercial Development
Deemarco Project
21705 Cajalco Road
Perris Area, Riverside County, California

Dear Ms. Mostad:

We are pleased to provide the results of our preliminary geotechnical evaluation and infiltration study for the subject project located in the Perris area of Riverside County, California. This report presents the results of our evaluation and discussion of our findings.

Based on the results of our evaluation, development of the property appears feasible from a geotechnical viewpoint provided that the recommendations presented in this report and in future reports are incorporated into design and construction.

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

Respectfully submitted,
GeoTek, Inc.

Edward H. LaMont
CEG 1892, Exp. 07/31/22
Principal Geologist

Noelle C. Toney
PE 84700, Exp. 03/31/22
Project Engineer

Distribution: (1) Addressee via email

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions for the proposed development. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site,
- Site exploration consisting of the excavation, logging, and sampling of four exploratory hollow-stem auger borings; and logging and infiltration testing of two hollow-stem auger borings,
- Collection of relatively undisturbed and bulk samples of the on-site materials,
- Laboratory testing of the samples obtained from the site,
- Review and evaluation of site seismicity, including seismic settlement analysis, and
- Compilation of this geotechnical report which presents our findings and a general summary of pertinent geotechnical conditions relevant for site development.

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

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The rectangular-shaped site is located at and addressed as 21705 Cajalco Road in the city of Perris, Riverside County, California. The site consists of approximately 3.2-acres and is comprised of one parcel identified with Riverside County Assessor's Parcel Number (APN) 318-130-012-5. The approximate location of the site is noted on the attached Figure 1, Site Location and General Topography Map.

Based on a review of available maps, observations at the time of our site reconnaissance and field exploration, the currently proposed development will be located in the area of a currently vacant site area with some palm trees and moderate amounts of surface vegetation. Based on historical imagery of the area, the western portion of the site was previously utilized as a storage yard with some sheds and trailers, but no permanent structures were observed. The sheds and trailers were not observed on-site at the time of our site reconnaissance and field investigation. Remnants of site improvements consisting of concrete fountains and flatwork within the north-central portion of the site and a small gazebo were observed. The subject property is bounded to the south by vacant land and single-family residential homes, to the north by Cajalco Road and vacant land beyond, to the east by Carroll Street and vacant land and single-family residential homes beyond, and to the west by vacant land and a small commercial building beyond. The subject site is rural in nature and is located in an area characterized by single-family homes and commercial buildings. Site topography is relatively planar and slopes gently down to the northwest with surface drainage generally directed towards the north-northwest. Total relief across the site is on the order of nine feet.

2.2 PROPOSED DEVELOPMENT

Based on our review of the *Conceptual Site Plan*, prepared by WSCS Designs and dated September 30, 2020; and the *Conceptual Grading Plan*, prepared by KWC Engineers and with a plot date of October 21, 2020, it is our understanding that the proposed site improvements will consist of a restaurant with a footprint of approximately 7,130 square feet, a building to be utilized as a car wash (1,482 square feet), gas pumps and a canopy structure (5,162 square feet), diesel pumps and a canopy structure (932 square feet) and a convenience store/drive through restaurant (1,675 square feet). Associated parking, drive aisles, underground utilities (including gas storage tanks), concrete flatwork and landscaping are also anticipated for development. On-site water disposal consisting of underground retention/detention chambers (i.e. MC-4500 Stormtech Chamber) is planned to be located toward the center of the site. Based on the Stormtech Chamber plans provided, prepared by Advanced Drainage Systems, Inc. (undated), the chambers will encompass and area of approximately 4,100 square feet with a planned infiltration depth of 15 feet below existing grade.

Specific structural loading was not provided to us; however, it is anticipated that the structures will be single-story, of wood-framed construction, will be supported by conventional shallow foundations and will include concrete slab-on-grade floors. For the purpose of this evaluation, we have assumed maximum column and wall loads of 100 kips and 4 kips per foot, respectively.

As site development planning progresses and plans become available, the plans should be provided to GeoTek for review and comment. Additional engineering analyses may be necessary in order to provide specific earthwork recommendations and geotechnical design parameters for actual site development.

3. GEOTECHNICAL WORK

3.1 FIELD EXPLORATION

Our geotechnical field exploration was conducted on November 23, 2020. A geologist from GeoTek logged four exploratory hollow-stem auger borings excavated by a truck-mounted hollow-stem auger drill. In addition, two percolation test borings were excavated and infiltration testing subsequently performed within the test borings. The borings were located throughout the site (see Exploration Location Map, Figure 2). Logs of the exploratory borings are included in Appendix A. Samples of on-site soils encountered in the excavations were returned to the laboratory for testing and evaluation.

3.2 LABORATORY TESTING

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

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

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

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

More specific to the subject property, the site is located in an area geologically mapped to be underlain by bedrock (Dibblee, T.W. and Minch, J.A., 2003). No active faults are shown in the immediate site vicinity on the maps reviewed for the site and site area.

4.2 EARTH MATERIALS

A brief description of the earth materials encountered during our explorations is presented in the following sections.

4.2.1 Undocumented Fill

Undocumented fill was not encountered in any of the explorations excavated on-site. Due to the proximity of existing improvements on and offsite, undocumented fill may be present within areas of the site that were not explored.

4.2.2 Alluvium

Alluvial materials were encountered within the upper one foot of the borings excavated on the site. In general, the alluvial materials typically consist of sand with varying amounts of clay.

According to the results of the laboratory testing performed, the near-surface alluvial soils exhibited a "very low" expansion potential when tested in accordance with ASTM D 4829. The test results are provided in Appendix B.

4.2.3 Bedrock

Bedrock materials consisting of quartz diorite were encountered underlying alluvium in all of the borings excavated on the site. In general, the bedrock materials were observed to be slightly weathered to weathered, slightly moist to wet, and indurated at approximately 11 feet to 14 feet below ground surface (bgs).

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during our site reconnaissance or investigation. If encountered during earthwork construction, surface water on this site is the result of precipitation or possibly some minor surface run-off from immediately surrounding properties. Overall site area drainage is generally in a northerly direction, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was encountered within all of our exploratory borings at a depth of approximately 14 feet bgs. Based on a review of information contained within the State of California Geotracker website (https://geotracker.waterboards.ca.gov/), groundwater was recently reported at a depth of approximately five feet bgs at a location 0.7-mile to the west of the subject site. The depth to groundwater is expected to vary seasonally and localized perched groundwater conditions could be encountered. Groundwater is not anticipated to impact the planned development with the exception of the planned underground gas tanks and infiltration chambers.

4.4 INFILTRATION STUDY

Two percolation test borings were set up within the planned underground infiltration areas. The borings were excavated with a truck-mounted hollow-stem auger drill rig and were approximately eight inches in diameter. Since groundwater was encountered at a depth of approximately 14 feet bgs in all of our exploratory borings, infiltration testing at the requested 15 feet bgs is not considered to be feasible, in accordance with the referenced County guidelines (Riverside County, 2011). Test borings I-1 and I-2 were excavated to a depth of approximately four feet bgs in order to allow for the minimum 10 feet interval above groundwater. Since bedrock was confirmed to be present from one foot bgs to our boring termination depths, permeable soil was not determined to be present beneath the anticipated infiltration areas. A three-inch diameter perforated PVC pipe encapsulated in filter sock was inserted into each of the percolation test holes. The annular space between the test hole sidewalls and PVC pipe was filled with gravel to prevent caving. The locations of the test borings are presented on Figure 1.

The soils encountered in our borings generally consisted of weathered bedrock. The boring logs are presented in Appendix A.

As mentioned above, groundwater was encountered within all of the borings drilled at this site at a depth of approximately 14 feet bgs (see Appendix A). Shallower groundwater depths on the order of five feet bgs were recently reported on the State GeoTracker website.

Subsequent to pre-soaking the test holes in general conformance with the referenced document (Riverside County, 2011), percolation testing was performed in the bottom 24 inches in test borings I-1 and I-2 by a representative from our firm. The percolation testing was conducted in general conformance with the referenced document from Riverside County. The percolation rate was converted to an infiltration rate utilizing the Porchet Method.

The infiltration rates for the borings are presented in the following table, after the water level had stabilized.

Boring No.	Infiltration Rate (inches per hour)	Depth of Boring (feet)
Boring I-1	0.1	4.0
Boring B-2	0.3	4.0

Copies of the percolation data and infiltration conversions (Porchet Method) are included in Appendix C. The reported infiltration rates are the measured rates without any factor of safety applied. Over the lifetime of the infiltration areas, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate in designing the infiltration system.

It should be noted that the infiltration rates provided above were performed in relatively undisturbed on-site soils. Infiltration rates will vary and are mostly dependent on the underlying consistency of the site 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 facility.

4.5 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within a State of California designated *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; State of California, 1993). The nearest zoned faults are the Elsinore Fault, approximately 12 miles to the southwest, and the San Jacinto Fault, approximately 15 miles to the northeast. The project site has not been evaluated by the State of California for liquefaction or landslide potential. The County of Riverside has designated the site as "not in fault zone, "not in a fault line," and not in a liquefaction nor subsidence area.

4.5.1 Seismic Design Parameters

The site is located at approximately 33.8367□Latitude and -117.2841□Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "C" site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format.

The results, based on the 2019 CBC, are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.5g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.553g
Site Coefficient for Site Class "C," F_a	1.2
Site Coefficient for Site Class "C," F_v	1.447
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.8g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.8g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.2g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.533g
Seismic Design Category	D

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

4.5.2 Surface Fault Rupture

The site is in a seismically active region; however, no active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; State of California, 1993). The nearest known active fault is located approximately 12 miles to the southwest. Therefore, the potential for surface rupture at the site is considered to be nil.

4.5.3 Seismic Settlement Analysis

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may 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 has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures. The site is not designated as having the potential for liquefaction by the State of California nor Riverside County. Based on the borings excavated on-site and the groundwater data reviewed for locations in the site vicinity, the groundwater is expected to be at approximately 5 feet to 14 feet bgs. However, based on the presence of dense bedrock at this site, it is our opinion that the potential for liquefaction at this site is nil.

4.5.4 Other Seismic Hazards

The potential for secondary seismic hazards such as seiche and tsunami is considered to be remote due to site elevation and distance from an open body of water. Due to the absence of a nearby free-face and the low liquefaction hazard, the potential for lateral spreading is considered to be nil.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Development of the site appears feasible from a geotechnical viewpoint. Specific recommendations for site development provided in this report will need to be further evaluated when development plans are provided for our review. The following sections present general recommendations. More specific geotechnical recommendations for site development can be provided when more finalized site development plans are available for review.

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of Riverside County, the 2019 California Building Code (CBC) and recommendations contained in this report. The General Grading Guidelines included in Appendix D outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix D.

5.2.2 Site Clearing and Demolition

Site preparation should start with removal of existing deleterious materials and vegetation within the planned development areas of the site. All deleterious materials should be properly disposed of off-site.

5.2.3 Removals and Overexcavations

All existing undocumented fills and alluvium should be removed and replaced with engineered fill. Removals should extend down to competent alluvium or bedrock materials. Competent alluvium is defined as native materials that are visually relatively non-porous and having a relative compaction of at least 85 percent of the soil/bedrock's maximum dry density as determined per ASTM D 1557. In areas of the proposed buildings and improvements, a minimum of two feet of engineered fill below the bottom of the proposed footings and floor-slabs should be provided. A minimum of two feet of fill should be provided beneath the pavement subgrade.

In cut areas, overexcavation should extend down to a depth such that a minimum of two feet of engineered fill is provided below the bottom of the deepest proposed foundation.

In transition areas (requiring cut and fill), a minimum of two feet of engineered fill should be provided below the bottom of the deepest proposed foundation. To mitigate the potential of excessive differential settlement associated with variable depths of engineered fill, overexcavation should extend down to a depth of one-half the maximum fill depth.

As a minimum, removals should extend down and away from foundation elements at a 1:1 (h:v) projection to the recommended removal depth, or a minimum of five feet laterally.

All undocumented fill should be also removed beneath flatwork improvement areas. A minimum of 12 inches of engineered fill should be provided below asphaltic concrete pavement and Portland cement concrete hardscape areas. The horizontal extent of removals should extend at least two feet beyond the edge of hardscape.

Development plans should be reviewed by this firm when available. Depending on actual field conditions encountered during grading, locally deeper areas of removal may be recommended.

The bottom of all removals should be scarified to a minimum depth of six inches, brought to at least the optimum moisture content, and then recompacted to at least 90 percent of the soil's maximum dry density (ASTM D 1557). The bottoms of removals should be observed by a GeoTek representative prior to scarification.

5.2.4 Engineered Fill

The on-site soils are generally considered suitable for reuse as engineered fill provided that they are free from vegetation, debris and other deleterious material. The undercut 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 six-inch to eight-inch loose lifts, moisture conditioned to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D 1557.

5.2.5 Excavation Characteristics

The anticipated excavations in the on-site alluvial and bedrock materials should be readily accomplished with heavy-duty earthmoving or excavating equipment in good operating condition.

5.2.6 Trench Excavations and Backfill

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

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

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

5.2.7 Shrinkage and Subsidence

For planning purposes, a shrinkage factor from 10 to 15 percent may be considered for the alluvial soils. A bulking factor of up to 20 percent can be considered for bedrock. A subsidence value of up to 0.1 may occur in alluvial areas, if any remain after remedial removals.

Several factors will impact earthwork balancing on the site, including shrinkage, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Shrinkage and bulking are primarily dependent upon the degree of compactive effort achieved during construction, depth of fill and underlying site conditions.

Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork construction.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented in this section. These are typical design criteria and are not intended to supersede the design by the structural engineer. If desirable, preliminary design recommendations for post-tension foundation systems can be provided upon request.

Based on the results of this investigation and laboratory testing previously performed at this site, GeoTek anticipates that the majority of the on-site soils to be encountered during grading may be classified as having "very low" ($0 \leq EI \leq 20$) expansion potential per ASTM D 4829. Additional laboratory testing should be performed at the completion of site grading to verify the expansion potential of the near-surface soils.

A summary of our preliminary foundation design recommendations is presented in the table below:

GEOTECHNICAL RECOMMENDATIONS FOR FOUNDATION DESIGN

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONALLY REINFORCED FOUNDATIONS	
Design Parameter	**"Very Low" Expansion Potential**
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	One-Story – 12
Minimum Foundation Width (Inches)*	One-Story – 12
Minimum Slab Thickness (actual)	4 – Actual
Minimum Slab Reinforcing	No. 3 rebar 24 inches on-center, each way, placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

* Code minimums per Table 1809.7 of the 2019 CBC.

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

The following criteria for design of foundations are preliminary and should be re-evaluated based on the results additional laboratory testing of samples obtained at/near finish pad grade.

5.3.1.1 An allowable bearing capacity of 1,500 pounds per square foot (psf) may be used for design of continuous footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This value may be increased by 200 psf for each additional 12 inches in depth and 100 psf for each additional 12 inches in width to a maximum value of 2,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

5.3.1.2 Structural foundations should be designed in accordance with the 2019 CBC, and to withstand a total estimated static settlement of less than 1 inch and a maximum differential static settlement of one-half of the total settlement over a horizontal distance of 40 feet.

5.3.1.3 The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

5.3.1.4 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.

5.3.1.5 A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2019 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 limited 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, California Building Code and Cal Green 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 our services in general are not intended to address mold prevention; since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

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

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.

5.3.2.3 Unsuitable soil removals along the property lines will likely be restricted due to adjacent improvements. Special considerations will be required for foundation elements in these areas. Such considerations may include deepening of foundations, reduced bearing capacity, or other measures. This issue should be further evaluated once site plans become available.

5.3.3 Foundation Set Backs

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

5.3.3.2 The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least seven feet and need not exceed 40 feet.

5.3.3.3 The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall stem. This applies to the existing retaining walls along the perimeter, if they are to remain.

The bottom of any proposed foundations for structures should be deepened so as to extend below a 1:1 projection upward from the bottom of the nearest excavation.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Recommendations presented may apply to typical masonry or concrete vertical retaining walls to a maximum height of six feet. Additional review and recommendations should be requested for higher walls.

The passive earth pressure may be computed as an equivalent fluid having a density of 200 psf per foot of depth, to a maximum earth pressure of 3,000 psf. A coefficient of friction between soil and concrete of 0.30 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

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

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Select Backfill*
Level	30
2:1	45

*Select backfill should consist of imported sand other approved materials with an expansion index less than or equal to 20.

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

5.4.2 Wall Backfill and Drainage

Wall backfill should include a minimum one-foot wide section of ¾- to 1-inch clean crushed rock (or approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from the backdrain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted on-site materials. If the walls are designed using the "select" backfill design parameters, then the "select" materials shall be placed within the active zone as defined by a 1:1 (H:V) projection from the back of the retaining wall footing up to the retained surface behind the wall. The presence of other materials might necessitate revision to the parameters provided and modification of wall designs.

The backfill materials should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM D 1557. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC, or approved equivalent) in a minimum of one cubic foot per linear foot of ¾-inch or one-inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and be directed (via a solid outlet pipe) to an appropriate disposal area.

Walls from two to four feet in height may be drained using localized gravel packs behind weep holes at 10 feet maximum spacing (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

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

5.4.3 Restrained Retaining Walls

Any retaining wall that will be restrained prior to placing backfill or walls that have male or reentrant corners should be designed for at-rest soil conditions using an equivalent fluid pressure of 60 pcf (select backfill), plus any applicable surcharge loading. For areas having male or reentrant corners, the restrained wall design should extend a minimum distance equal to twice the height of the wall laterally from the corner, or as otherwise determined by the structural engineer.

5.4.3.1 Other Design Considerations

- Retaining and garden wall foundation elements should be designed in accordance with building code setback requirements. A minimum horizontal setback distance of five feet as measured from the bottom outside edge of the footing to a sloped face is recommended.
- 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 PAVEMENT DESIGN AND CONSTRUCTION

5.5.1 Asphaltic Concrete Pavement

GeoTek utilized a bulk sample obtained from the field investigation for R-Value testing. The testing (by others) indicated an R-Value of 44. The R-Value test results are included in Appendix B.

Traffic Indices (TI) of 5.5 and 7.0 were assumed for preliminary pavement design for the parking lot and drive aisles. The traffic indices selected to determine the pavement section should be reviewed by a design engineer when truck traffic loading is known. The table below provides the roadway area, TI, and two options for the recommended minimum structural pavement sections.

MINIMUM RECOMMENDED ASPHALT CONCRETE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Design R-Value	Asphaltic Concrete (inches)	Aggregate Base (inches)
Light Vehicular Traffic Areas (including parking stalls and drive aisles not subject to heavy truck traffic	5.5	44	3.0	4.5
			3.5	3.5
Heavy Truck Traffic Areas (including fire lanes, trash dumpster pads and approaches)	7.0	44	4.0	6.0
			4.5	5.0

The pavement sections recommended are subject to review by the County of Riverside. Performance of the pavement sections will ultimately be based largely on construction methods, traffic loading and subgrade performance.

Additional laboratory testing should be completed during earthwork construction when pavement subgrade elevations are reached to confirm the sections presented above.

5.5.2 Portland Cement Concrete Pavement for Heavy Truck Traffic Areas

It is anticipated that heavy truck traffic will be exerting loads within heavy truck traffic areas paved with Portland Cement Concrete (PCC) pavement.

The table below provides the street area/usage, associated TI, and the recommended minimum concrete pavement section for the subject project. An R-Value of 44 was correlated to a modulus of subgrade reaction, k-Value, of approximately 210 for design purposes.

MINIMUM RECOMMENDED CONCRETE PAVEMENT SECTIONS

Location	**Assumed Traffic Category***	**Design k-Value**	**PCC (inches)**	**Aggregate Base (inches)**
Heavy Truck Traffic Areas (including dock aprons, fire lanes, trash dumpster pads and approaches)	D	210	7.0	4.0

*Reference: *Guide for the Design and Construction of Concrete Parking Lots*, Reported by ACI Committee 330, ACI 330R-08, 2008.

The PCC pavement sections should incorporate appropriate steel reinforcement as designed by the project structural engineer. Crack control joints should be provided in the transverse direction spaced at horizontal intervals with a maximum spacing of 15 feet. The actual design should also be in accordance with design criteria specified by the governing jurisdiction.

The concrete should have a minimum modulus of rupture of 500 pounds per square inch (psi), and a minimum 28-day compressive strength of 2,500 psi. Concrete should incorporate one-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.

The concrete pavement section is subject to the review and approval by the County of Riverside. Performance of the pavement sections will ultimately be based largely on construction methods, traffic loading and subgrade performance.

5.5.3 Pavement Construction

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement of concrete and rolling of asphaltic concrete, should be done in accordance with the County of Riverside specifications and under the observation and testing of GeoTek and a County inspector where required.

The aggregate base should consist of crushed rock with an R-Value and gradation in accordance with Crushed Aggregate Base (Section 200-2 of the "Greenbook"). Asphaltic concrete materials and construction should conform to Section 203 of the Greenbook. Minimum compaction requirements should be 95 percent for the upper three feet of subgrade and 95 percent for aggregate base, as per ASTM D 1557. The upper 12 inches of subgrade should be moisture conditioned to at least the optimum moisture content. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

5.5.4 Soil Corrosivity

The soil corrosivity at this site was tested in the laboratory on one sample collected by our firm. The results of the testing indicate that the soil samples were considered "corrosive" to buried ferrous metals in accordance with current standards commonly used by corrosion engineers. Consideration should be given to consulting with a corrosion engineer. The laboratory test results are provided in Appendix B.

5.5.5 Soil Sulfate Content

The sulfate content was determined in the laboratory for one representative soil sample collected by our firm. The results indicate that the water-soluble sulfate for the tested samples was less than 0.1 percent by weight, which is considered "not applicable" (i.e. negligible) as per Table 4.2.1 of ACI 318. Based upon the test results, no special concrete mix design is required for sulfate attack resistance. The laboratory test results are provided in Appendix B.

5.5.6 Import Soils

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

5.6 CONCRETE CONSTRUCTION

5.6.1 General

Concrete construction should follow the 2019 CBC and ACI guidelines regarding design, mix placement and curing of the concrete. If desired, we could provide quality control testing of the concrete during construction.

5.6.2 Concrete Flatwork

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in industrial construction. Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. should be pre-saturated to a minimum of 110 percent of the optimum moisture content to a depth of 12 inches.

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

5.6.3 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable 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 directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered "non-structural" components. We suggest that the same standards of care be applied to these features as to the structures themselves.

5.7 POST CONSTRUCTION CONSIDERATIONS

5.7.1 Landscape Maintenance and Planting

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

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

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

5.7.2 Drainage

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

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

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

5.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

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

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

6. LIMITATIONS

This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our proposal (Proposal No. P-1008220-CR) dated October 21, 2020 and geotechnical engineering standards normally used on similar projects in this region.

The materials observed on the project site appear to be representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

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

7. SELECTED REFERENCES

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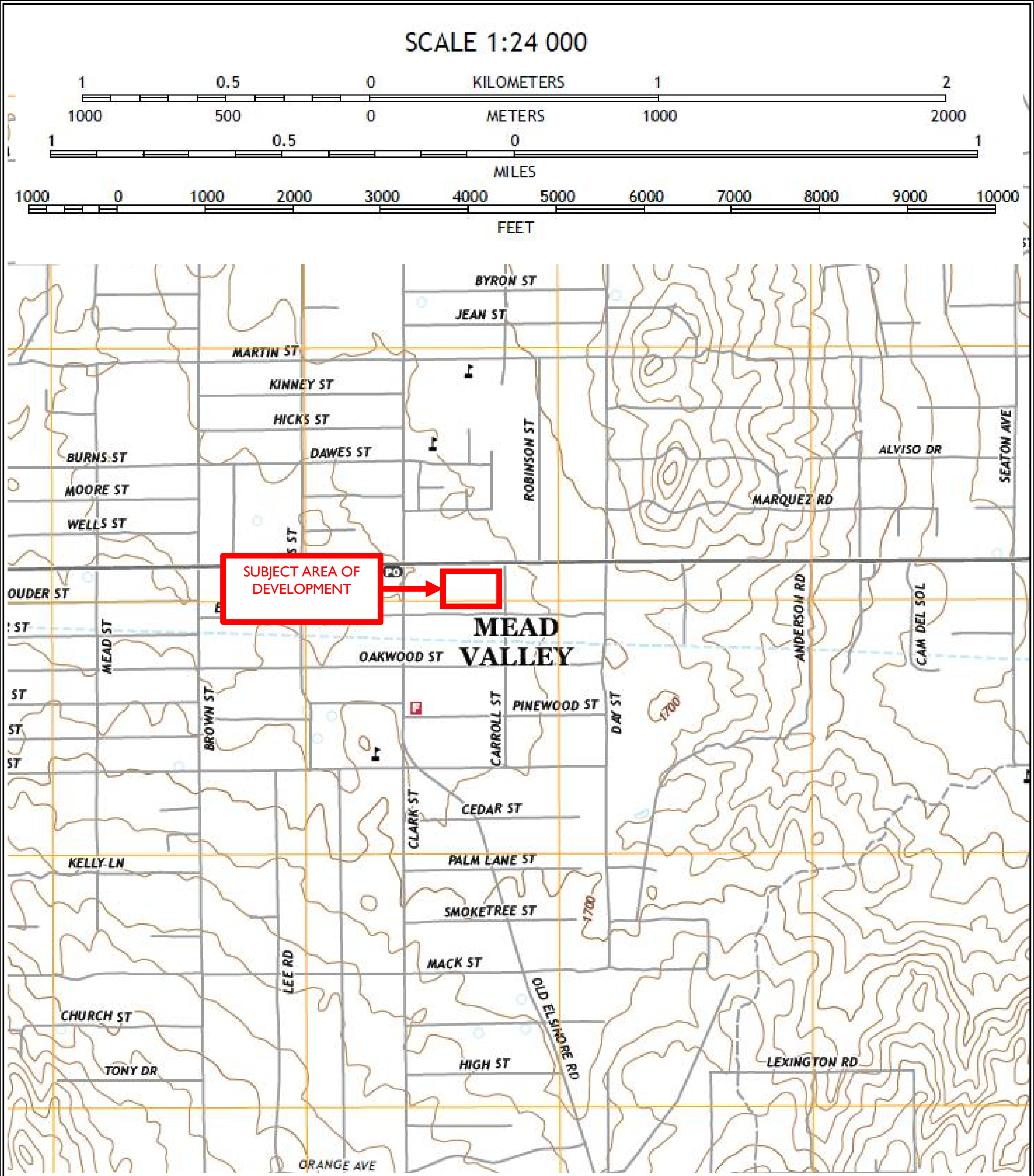


Alabbasi Construction & Engineering
Deemarco Project – Commercial Development
21705 Cajalco Road
Perris Area, Riverside County, California

GeoTek Project No. 2550-CR

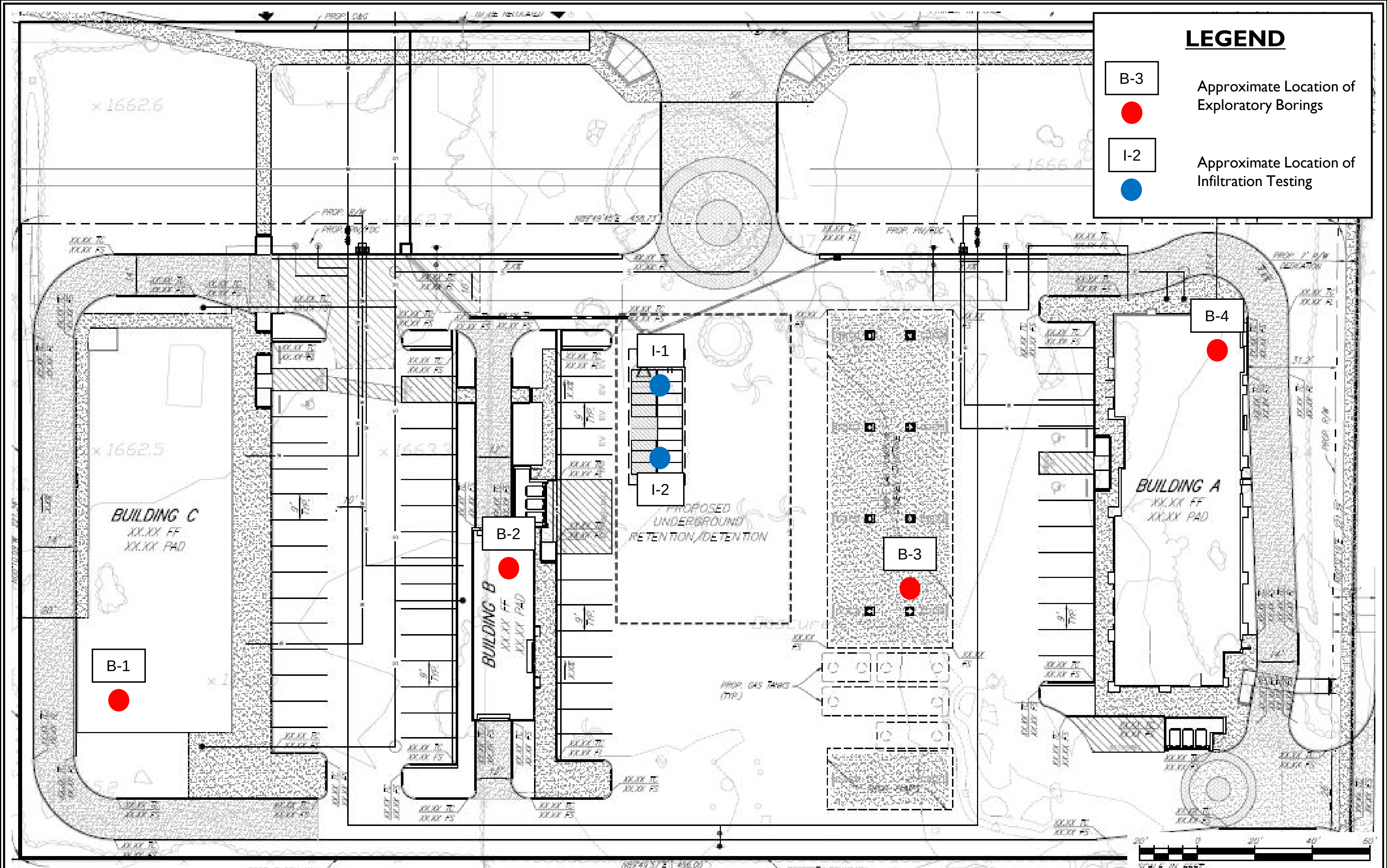
N

Modified from USGS 7.5-minute Steele-Peak Topographic Map

Figure 1

Site Location and General Topography Map

GEOTEK



Alabbasi Construction & Engineering
Deemarco Project – Commercial Development
21705 Cajalco Road
Perris, Riverside County, California

GeoTek Project No. 2550-CR

N

Figure 2

Exploration Location Map

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APPENDIX A

LOGS OF EXPLORATORY BORINGS

Commercial Development
21705 Cajalco Road
Perris Area, Riverside County, California
Project No. 2550-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)
The Ring sampler is driven into the ground in accordance with ASTM Test Method D 3550. 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.

Bulk Samples (Large)
These samples are normally large bags of representative 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 representative 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/TRENCH LOG LEGEND

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

SOILS

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

GEOLOGIC

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Alabbasi Construction & Engineering	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	21705 Cajalco Road	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Jeff
PROJECT NO.:	2550-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	11/23/2020

BORING NO.: B-1

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
	Large Bulk			SC	**Alluvium:** Clayey f-c SAND, brown, dry to slightly moist			
					Granitic Bedrock:			
	Ring	50/6"	R1 B1		Weathered granite, excavates as silty f-c SAND, orangish brown, slightly moist	4.7	109.6	MD, EI, SR, DS
5	Ring	50/5"	R2			2.5	116.5	
	Ring	50/5"	R3		Becoming slightly weathered at 7'	4.2	118.0	
	Ring	50/4"	R4			3.3	114.8	
10								
	Ring	50/5"	R5		Groundwater encountered at 14.1' Becoming indurated, gray, and moist to wet at 14'	5.9	118.1	
15								
	SPT	50/2"						
20					**BORING TERMINATED AT 19.5 FEET** Groundwater encountered at 14.1 feet Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Alabbasi Construction & Engineering	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	21705 Cajalco Road	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Jeff
PROJECT NO.:	2550-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	11/23/2020

BORING NO.: B-2

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
				SC	**Alluvium:** Clayey f-c SAND, brown, dry to slightly moist			
	Ring	50/6"	R1		**Granitic Bedrock:** Weathered granite, excavates as silty f-c SAND, orangish brown, slightly moist	2.9	121.4	
	Ring	50/6"	R2			3.2	114.2	
5								
	Ring	50/4"	R3		Becoming slightly weathered at 6'	3.5	114.5	
10								
	Ring	50/3"	R4		Becoming indurated, gray, and moist to wet at 11'	3.3	116.8	
15					Groundwater encountered at 14.7'			
	Ring	50/2"	R5			7.9	124.2	
					BORING TERMINATED AT 16.5 FEET			
					Groundwater encountered at 14.7 feet Boring backfilled with soil cuttings			
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Alabbasi Construction & Engineering	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	21705 Cajalco Road	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Jeff
PROJECT NO.:	2550-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	11/23/2020

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				SC	**Alluvium:** Clayey f-c SAND, brown, dry to slightly moist			
	Ring	50/4"	R1		**Granitic Bedrock:** Weathered granite, excavates as silty f-c SAND, orangish brown, slightly moist	3.1	122.1	
	Ring	50/4"	R2			2.3	115.9	
5								
	Ring	50/3"	R3		Becoming slightly weathered at 8'	2.8	114.7	
10								
	Ring	50/4"	R4		Becoming indurated, gray, and moist to wet at 12'	3.2	113.0	
▽					Groundwater encountered at 14'			
15								
	Ring	50/3"	R5			3.5	114.3	
					BORING TERMINATED AT 19.5 FEET			
20					Groundwater encountered at 14.0 feet Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Alabbasi Construction & Engineering	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	21705 Cajalco Road	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Jeff
PROJECT NO.:	2550-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	11/23/2020

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
				SC	**Alluvium:** Clayey f-c SAND, brown, dry to slightly moist			
					Granitic Bedrock:			
		50/4"	R1 B1		Weathered granite, excavates as silty f-c SAND, orangish brown, slightly moist	2.3	115.3	RV
		50/4"	R2			3.0	111.8	
5								
		50/3"	R3		Becoming slightly weathered at 8'	3.4	113.5	
10								
		50/3"	R4		Becoming indurated, gray, and moist to wet at 13'	3.4	115.5	
					Groundwater encountered at 14.1'			
15								
		50/3"	R5			5.5	117.8	
20		50/2"						
					BORING TERMINATED AT 21.5 FEET			
					Groundwater encountered at 14.1 feet			
					Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Alabbasi Construction & Engineering	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	21705 Cajalco Road	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Jeff
PROJECT NO.:	2550-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	11/23/2020

BORING NO.: I-1

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
				SC	**Alluvium:** Clayey f-c SAND, brown, dry to slightly moist			
					Granitic Bedrock: Weathered granite, excavates as silty clayey f-c SAND, orangish brown, slightly moist			
5					**BORING TERMINATED AT 4 FEET** No groundwater encountered Boring prepared for subsequent infiltration testing (pvc pipe, gravel, filter sock)			
10								
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Alabbasi Construction & Engineering	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	21705 Cajalco Road	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Jeff
PROJECT NO.:	2550-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Exploration Location Map			**DATE:**	11/23/2020

BORING NO.: I-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				SC	**Alluvium:** Clayey f-c SAND, brown, dry to slightly moist			
					Granitic Bedrock: Weathered granite, excavates as silty f-c SAND, orangish brown, slightly moist			
5					**BORING TERMINATED AT 4 FEET** No groundwater encountered Boring prepared for subsequent infiltration testing (pvc pipe, gravel, filter sock)			
10								
15								
20								
25								
30								

LEGEND

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

Lab testing:

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

APPENDIX B

LABORATORY TEST RESULTS

Commercial Development
21705 Cajalco Road
Perris Area, Riverside County, California
Project No. 2550-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

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

In-Situ Moisture Content and Unit Weight
The field moisture content was measured in the laboratory on selected samples collected during the field investigation. The field moisture content is determined as a percentage of the dry unit weight. The dry density was measured in the laboratory on selected ring samples. The results are shown on the logs of exploratory borings in Appendix A.

Moisture-Density Relationship
Laboratory testing was performed on representative site samples collected during the recent subsurface exploration. The laboratory maximum dry density and optimum moisture content for the samples tested were determined in general accordance with test method ASTM D 1557. The results are presented below.

Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	1-5	Clayey f-c sand and weathered bedrock	133.5	8.0

Direct Shear
Direct shear testing was performed on a remolded sample of the surficial soils according to ASTM D 3080. The results of these tests are presented in Appendix B.

Expansion Index
The expansion potential of the soils was determined by performing expansion index tests on soil samples obtained from the site in general accordance with ASTM D 4829. The results of these tests are presented below.

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-1	1-5	Clayey f-c sand and weathered bedrock	9	Very Low

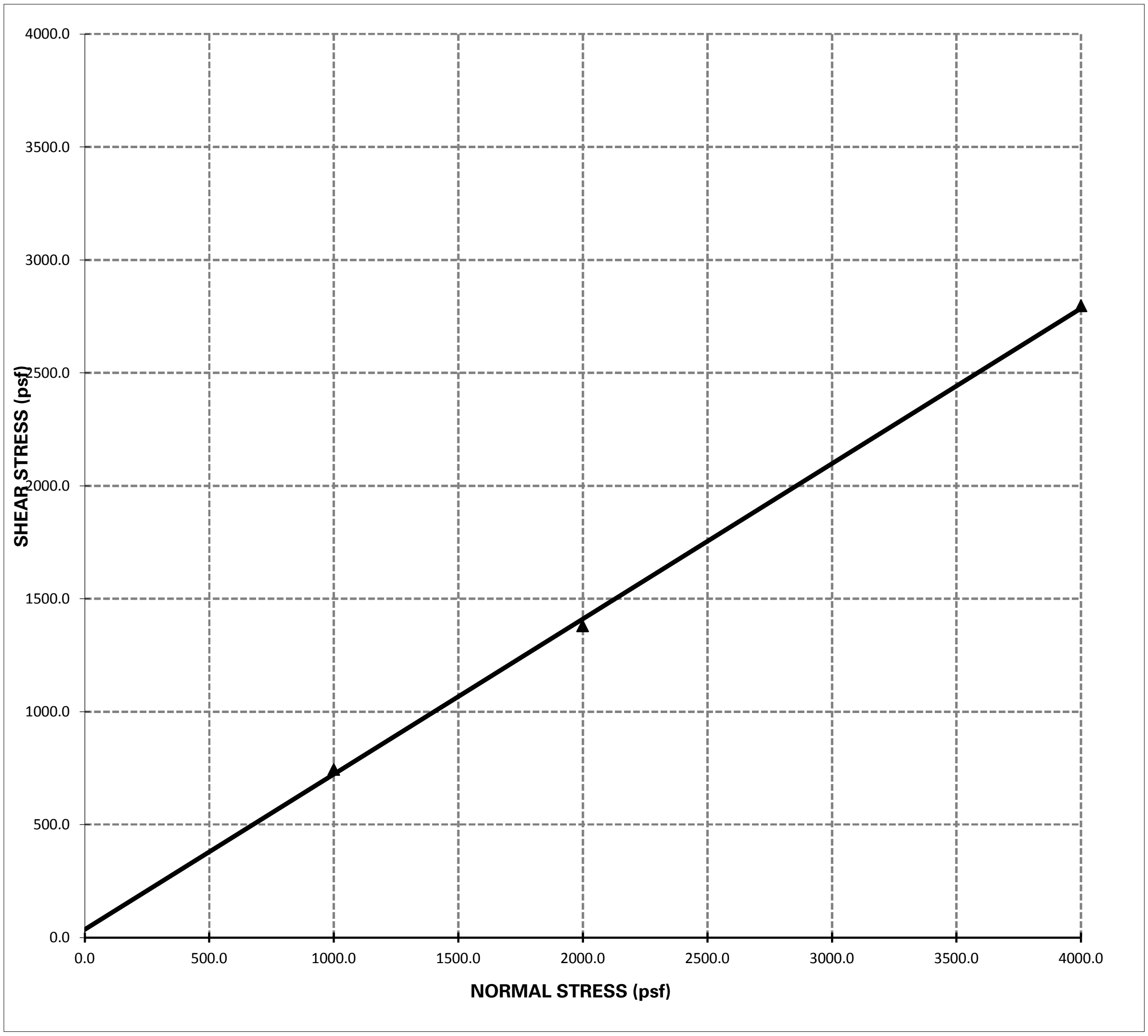
R-Value
Testing to determine the resistance value for pavement design was performed by others in accordance with California Test Method 301, on a sample collected during the subsurface exploration. The results are presented in Appendix B.

Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content was performed by others in general accordance with ASTM D4327. Resistivity testing was completed by others in general accordance with ASTM G187. Testing to determine the chloride content was performed by others in general accordance with ASTM D4327. pH testing was completed by others in general accordance with ASTM D4972. The results of the testing are provided below.

Boring No.	Depth (ft.)	pH ASTM D4972	Chloride ASTM D4327 (ppm)	Sulfates ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-1	1-5	8.8	8.9	0.0033	4,690

DIRECT SHEAR TEST

Project Name:	21705 Cajalco Road, Perris	***Sample Location:***	B-1 @ 1-5 feet
Project Number:	2550-CR	***Date Tested:***	12/23/2020



Shear Strength: $\Phi =$ **34.5°** , **C = 36.00 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 0.035 in/min.

• ANALYSIS
• DESIGN

LaBelle • Marvin

• SOILS, ASPHALT TECHNOLOGY

PROFESSIONAL PAVEMENT ENGINEERING

A CALIFORNIA CORPORATION

December 11, 2020

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

Project No. 46746

Attention Ms. Scott:
Laboratory testing of the bulk soil sample delivered to our laboratory on 12/10/2020 has been completed.

Reference:	W.O. # 2550-CR
Project:	21705 Cajalco Road, Perris, Alabbasi Construction & Engineering
Sample:	B-4 @ 1'-5'

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

Very truly yours,

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

Steven R. Marvin
RCE 30659

SRM:tw
Enclosures

R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 46746
DATE: 12/11/2020

BORING NO. B-4 @ 1'-5'
21705 Cajalco Rd., Perris, Alabbasi Construction & Engineering
W.O.# 2550-CR

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	1	2	3
Water added, grams	50	70	38
Initial Test Water, %	7.7	9.5	6.6
Compact Gage Pressure,psi	150	75	275
Exudation Pressure, psi	419	198	671
Height Sample, Inches	2.48	2.54	2.47
Gross Weight Mold, grams	3128	3133	3122
Tare Weight Mold, grams	1954	1946	1958
Sample Wet Weight, grams	1174	1187	1164
Expansion, Inches x 10exp-4	37	9	59
Stability 2,000 lbs (160psi)	20 / 41	32 / 72	17 / 32
Turns Displacement	5.01	5.30	4.68
R-Value Uncorrected	59	37	68
R-Value Corrected	59	37	68
Dry Density, pcf	133.2	129.3	133.9

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.42	0.65	0.33
G. E. by Expansion	1.23	0.30	1.97

Equilibrium R-Value	44 by EXPANSION	Examined & Checked: 12 /11/ 20
REMARKS:	Gf = 1.25 0.0% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

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

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 46746

DATE: 12 /11/ 2020

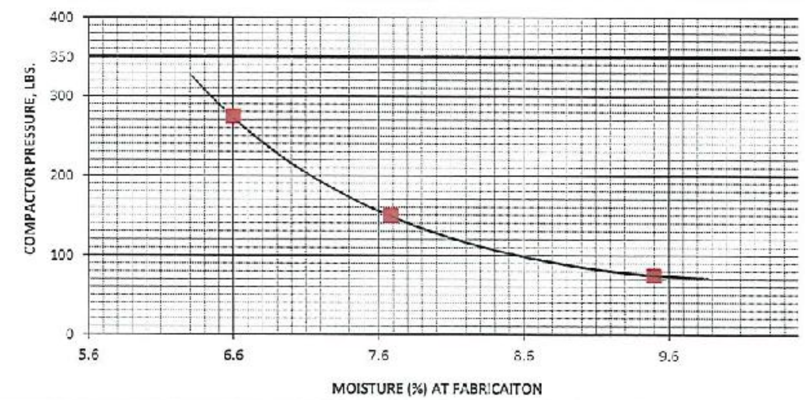
REMARKS:

BORING NO. B-4 @ 1'-5'

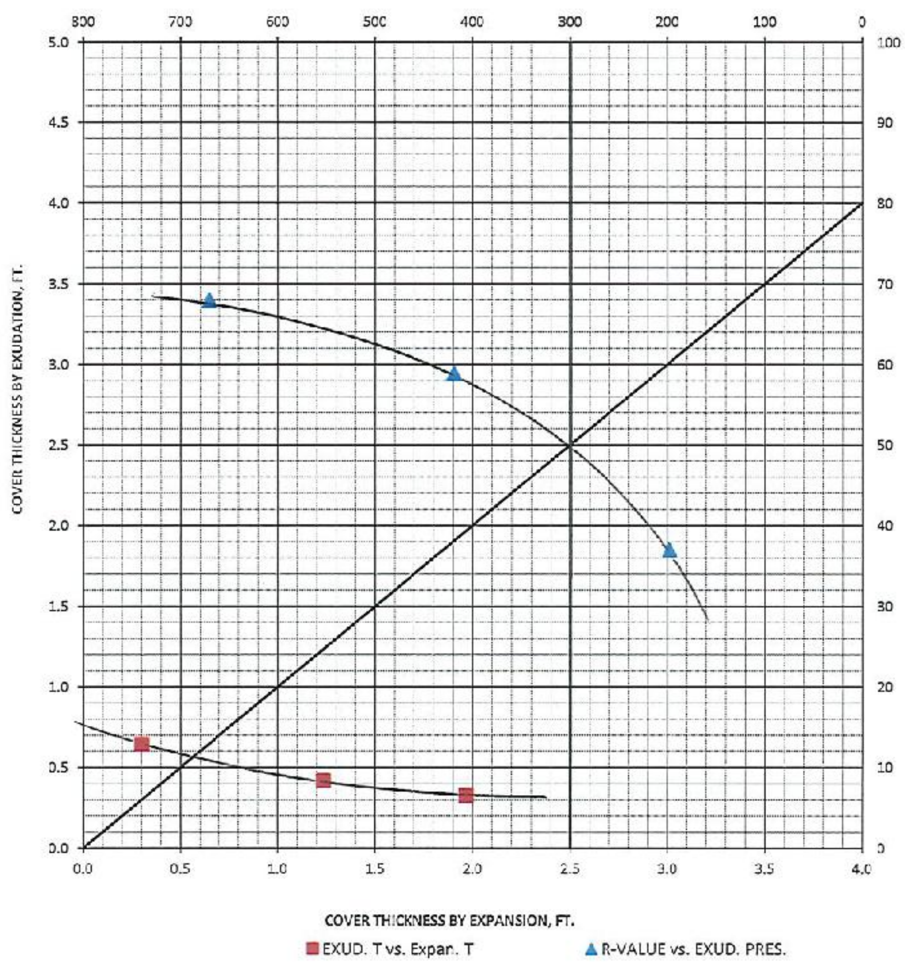
21705 Cajalco Rd., Perris, Alabbasi Construction & Engineering

W.O.# 2550-CR

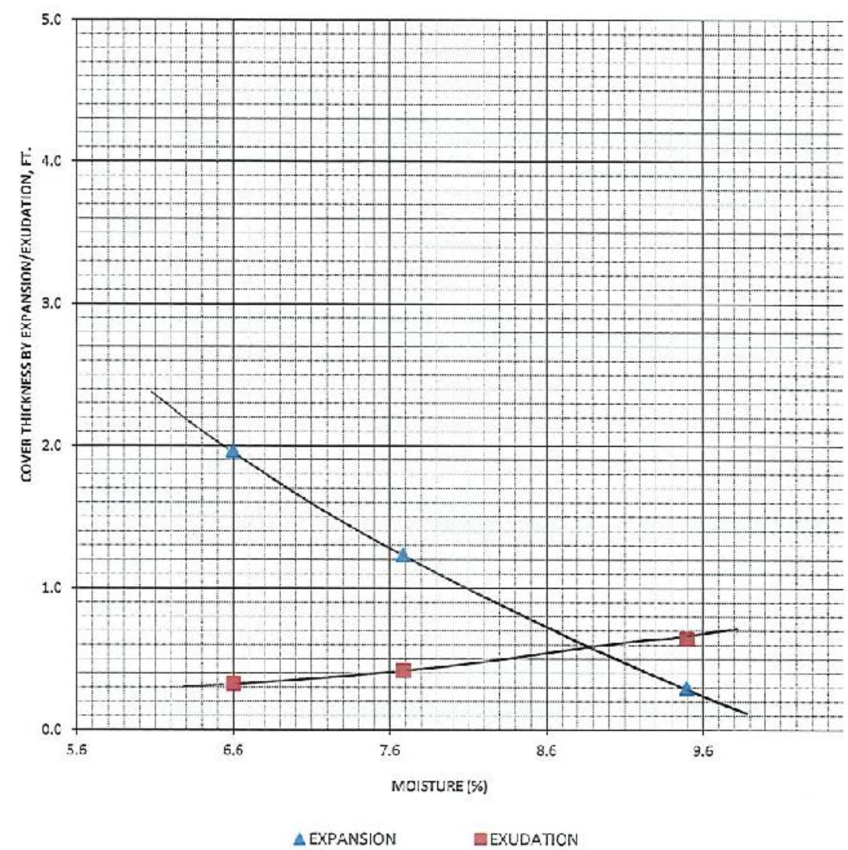
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



APPENDIX C

INFILTRATION TEST DATA

Commercial Development
21705 Cajalco Road
Perris Area, Riverside County, California
Project No. 2550-CR

GEOTEK

GeoTek, Inc.
PERCOLATION TESTING

Shallow Percolation Test (<10 ft)

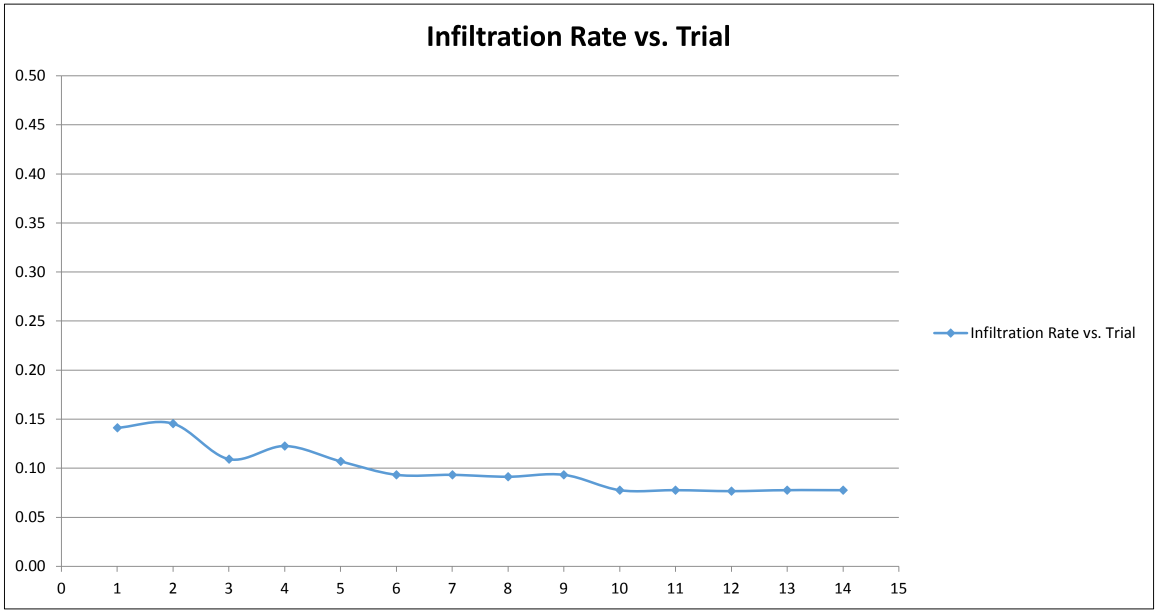
Test No. I-1

2550-CR

Depth of Hole (D_T) in.	48
Boring Radius, in.	4

Trial No.	Time Interval (ΔT) Min.	Initial Depth (D_0) in.	Final Depth (D_f) in.	Change In Level (ΔD) in.	Perc Rate (min/in)	Infiltration Rate (in/hr)
Non-Sandy Soil Trial 1	25	22.40	23.20	0.80	31.25	0.14
Non-Sandy Soil Trial 2	25	23.20	24.00	0.80	31.25	0.15
1	30	24.00	24.70	0.70	42.86	0.11
2	30	23.50	24.30	0.80	37.50	0.12
3	30	23.50	24.20	0.70	42.86	0.11
4	30	24.00	24.60	0.60	50.00	0.09
5	30	24.00	24.60	0.60	50.00	0.09
6	30	23.40	24.00	0.60	50.00	0.09
7	30	24.00	24.60	0.60	50.00	0.09
8	30	24.00	24.50	0.50	60.00	0.08
9	30	24.00	24.50	0.50	60.00	0.08
10	30	23.70	24.20	0.50	60.00	0.08
11	30	24.00	24.50	0.50	60.00	0.08
12	30	24.00	24.50	0.50	60.00	0.08

Initial Height (H_0)	Final Height (H_f)	Height Change (ΔH)	Height Average (H_{avg})
25.60	24.80	0.80	25.20
24.80	24.00	0.80	24.40
24.00	23.30	0.70	23.65
24.50	23.70	0.80	24.10
24.50	23.80	0.70	24.15
24.00	23.40	0.60	23.70
24.00	23.40	0.60	23.70
24.60	24.00	0.60	24.30
24.00	23.40	0.60	23.70
24.00	23.50	0.50	23.75
24.00	23.50	0.50	23.75
24.30	23.80	0.50	24.05
24.00	23.50	0.50	23.75
24.00	23.50	0.50	23.75



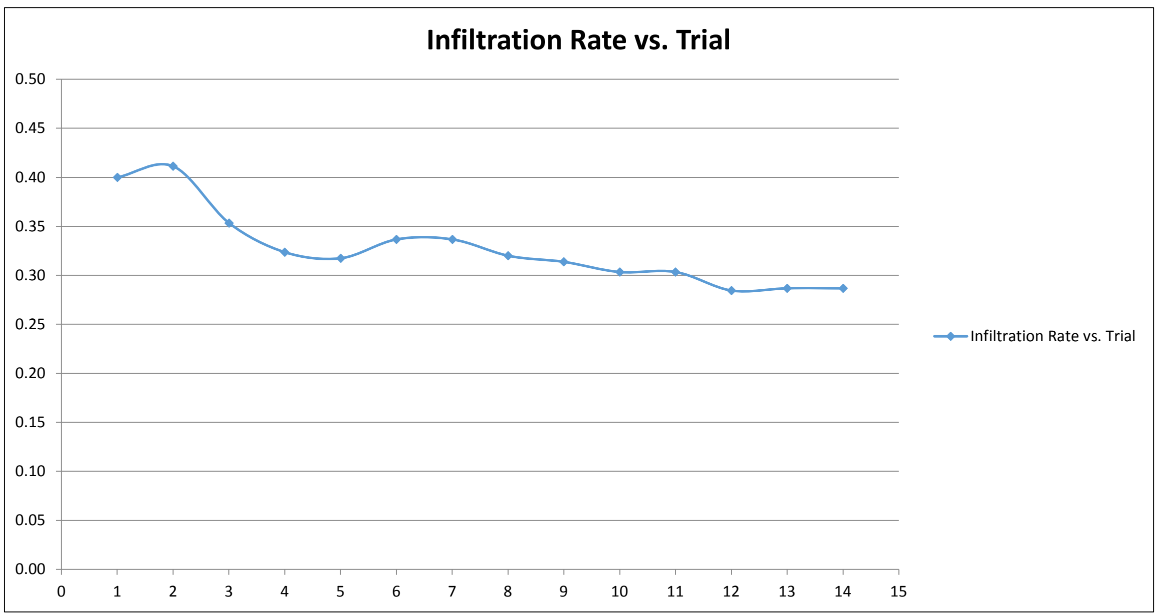
GeoTek, Inc.
PERCOLATION TESTING

Shallow Percolation Test (<10 ft) | **Test No. I-2**
2550-CR

Depth of Hole (D_T) in.	48
Boring Radius, in.	4

Trial No.	Time Interval (ΔT) Min.	Initial Depth (D_0) in.	Final Depth (D_f) in.	Change In Level (ΔD) in.	Perc Rate (min/in)	Infiltration Rate (in/hr)
Non-Sandy Soil Trial 1	25	20.00	22.40	2.40	10.42	0.40
Non-Sandy Soil Trial 2	25	22.00	24.30	2.30	10.87	0.41
1	30	24.00	26.20	2.20	13.64	0.35
2	30	23.00	25.10	2.10	14.29	0.32
3	30	23.80	25.80	2.00	15.00	0.32
4	30	24.00	26.10	2.10	14.29	0.34
5	30	24.00	26.10	2.10	14.29	0.34
6	30	24.00	26.00	2.00	15.00	0.32
7	30	23.50	25.50	2.00	15.00	0.31
8	30	24.00	25.90	1.90	15.79	0.30
9	30	24.00	25.90	1.90	15.79	0.30
10	30	23.80	25.60	1.80	16.67	0.28
11	30	24.00	25.80	1.80	16.67	0.29
12	30	24.00	25.80	1.80	16.67	0.29

Initial Height (H_0)	Final Height (H_f)	Height Change (ΔH)	Height Average (H_{avg})
28.00	25.60	2.40	26.80
26.00	23.70	2.30	24.85
24.00	21.80	2.20	22.90
25.00	22.90	2.10	23.95
24.20	22.20	2.00	23.20
24.00	21.90	2.10	22.95
24.00	21.90	2.10	22.95
24.00	22.00	2.00	23.00
24.50	22.50	2.00	23.50
24.00	22.10	1.90	23.05
24.00	22.10	1.90	23.05
24.20	22.40	1.80	23.30
24.00	22.20	1.80	23.10
24.00	22.20	1.80	23.10



APPENDIX D

GENERAL GRADING GUIDELINES

Commercial Development
21705 Cajalco Road
Perris Area, Riverside County, California
Project No. 2550-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 California Building Code, CBC (2019) and the guidelines presented below.

Preconstruction Meeting

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

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor’s overall level of efforts during grading. The contractor’s personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor’s responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.

6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

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

Treatment of Existing Ground

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

Fill Placement

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

2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
    a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
    b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
    a) They are not placed in concentrated pockets;
    b) There is a sufficient percentage of fine-grained material to surround the rocks;
    c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

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

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

JOB SAFETY

General

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

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

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

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

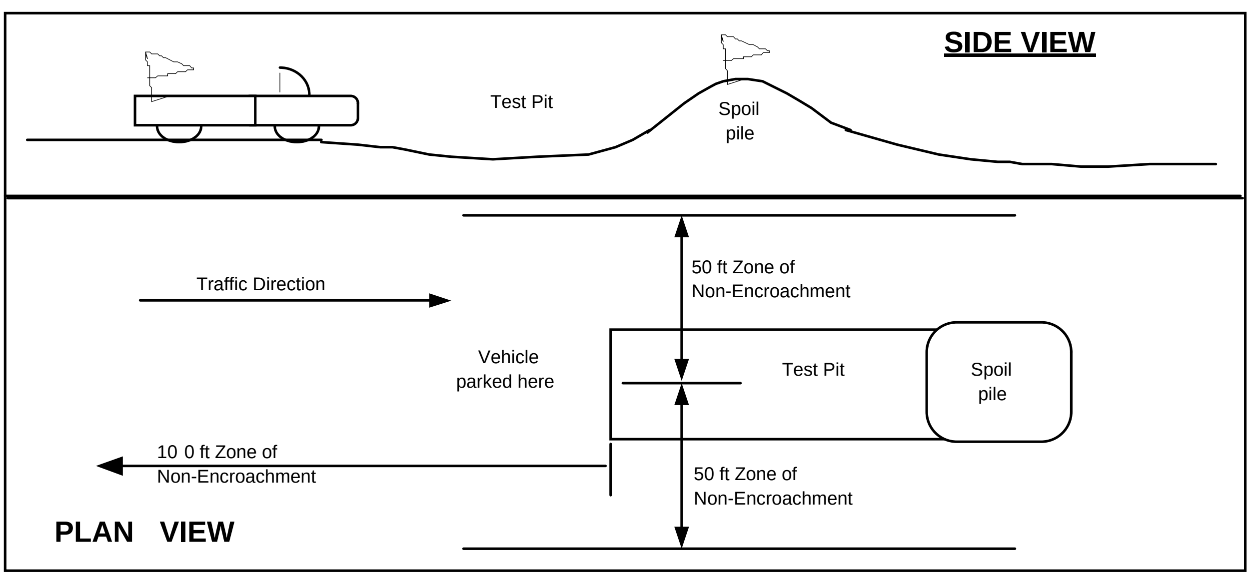
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

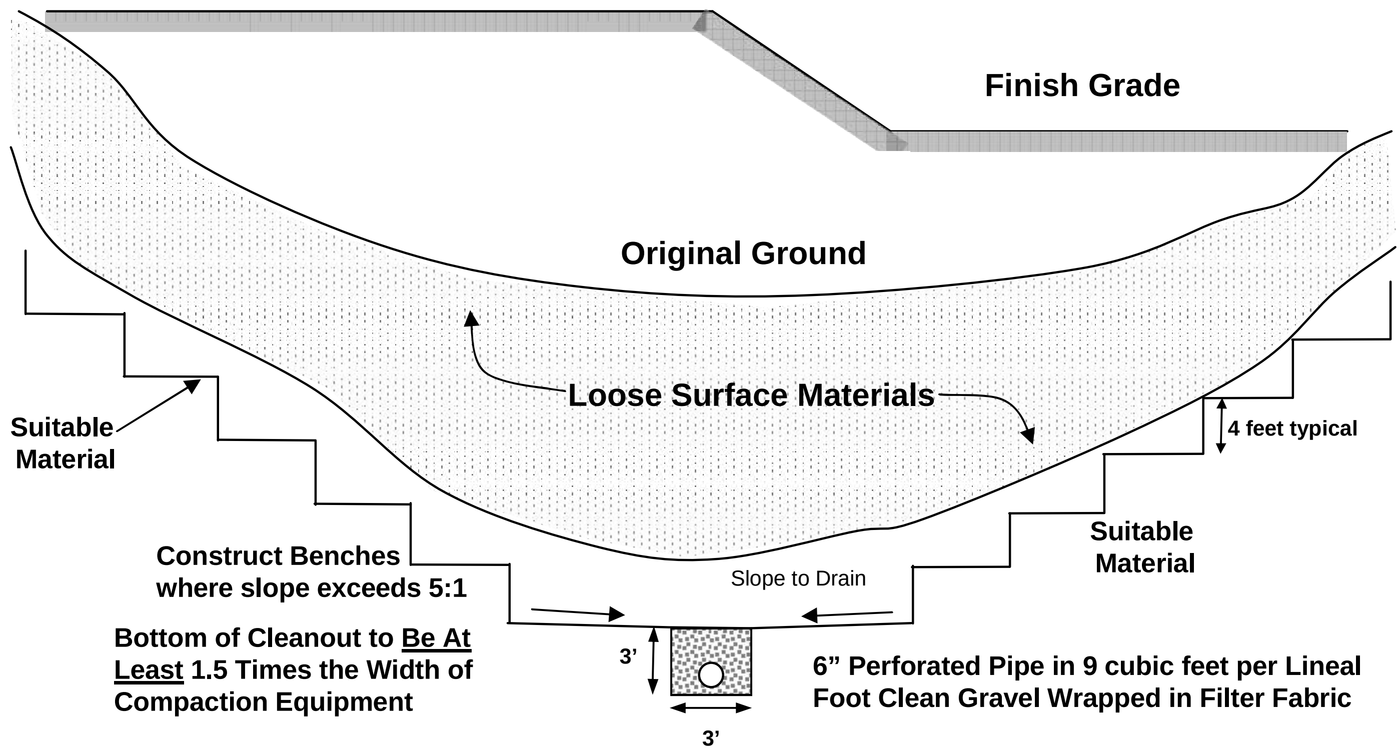
In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by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

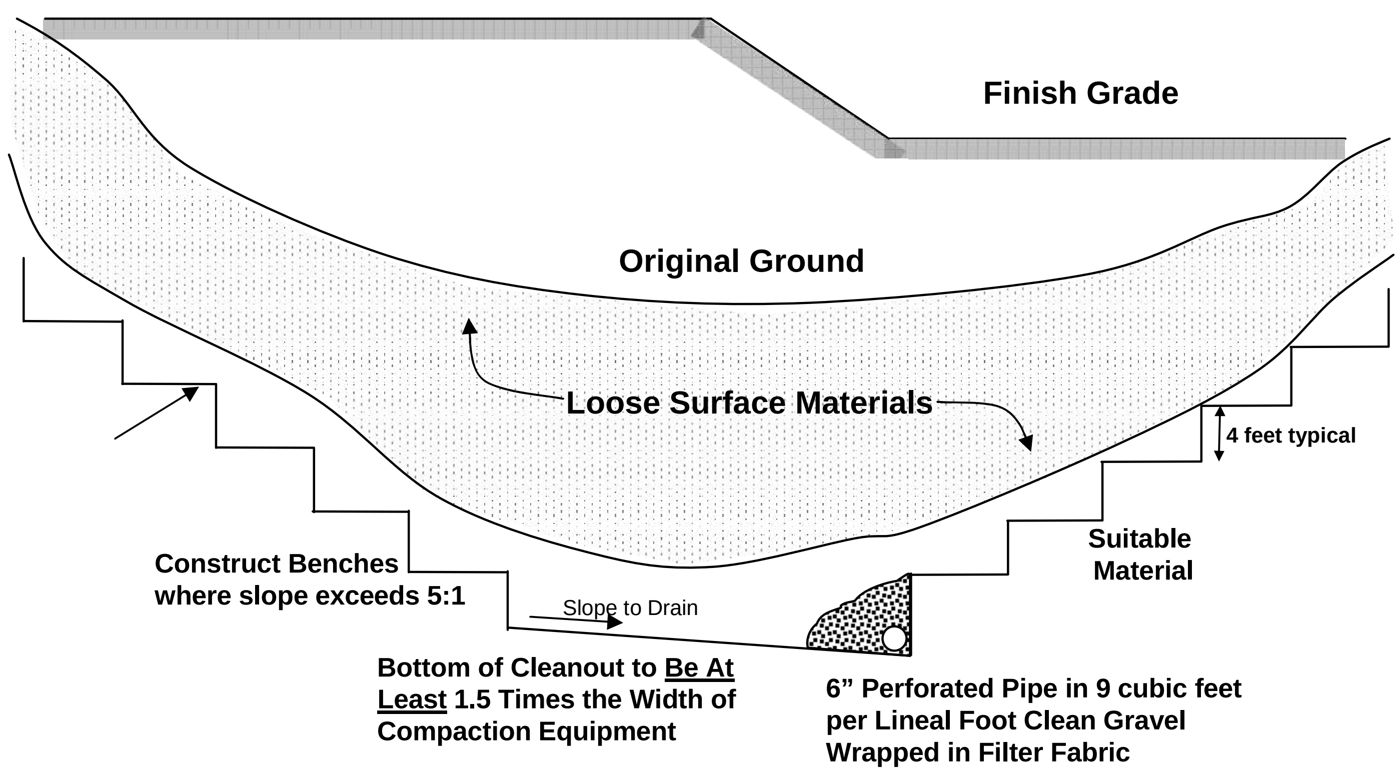
In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and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

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

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

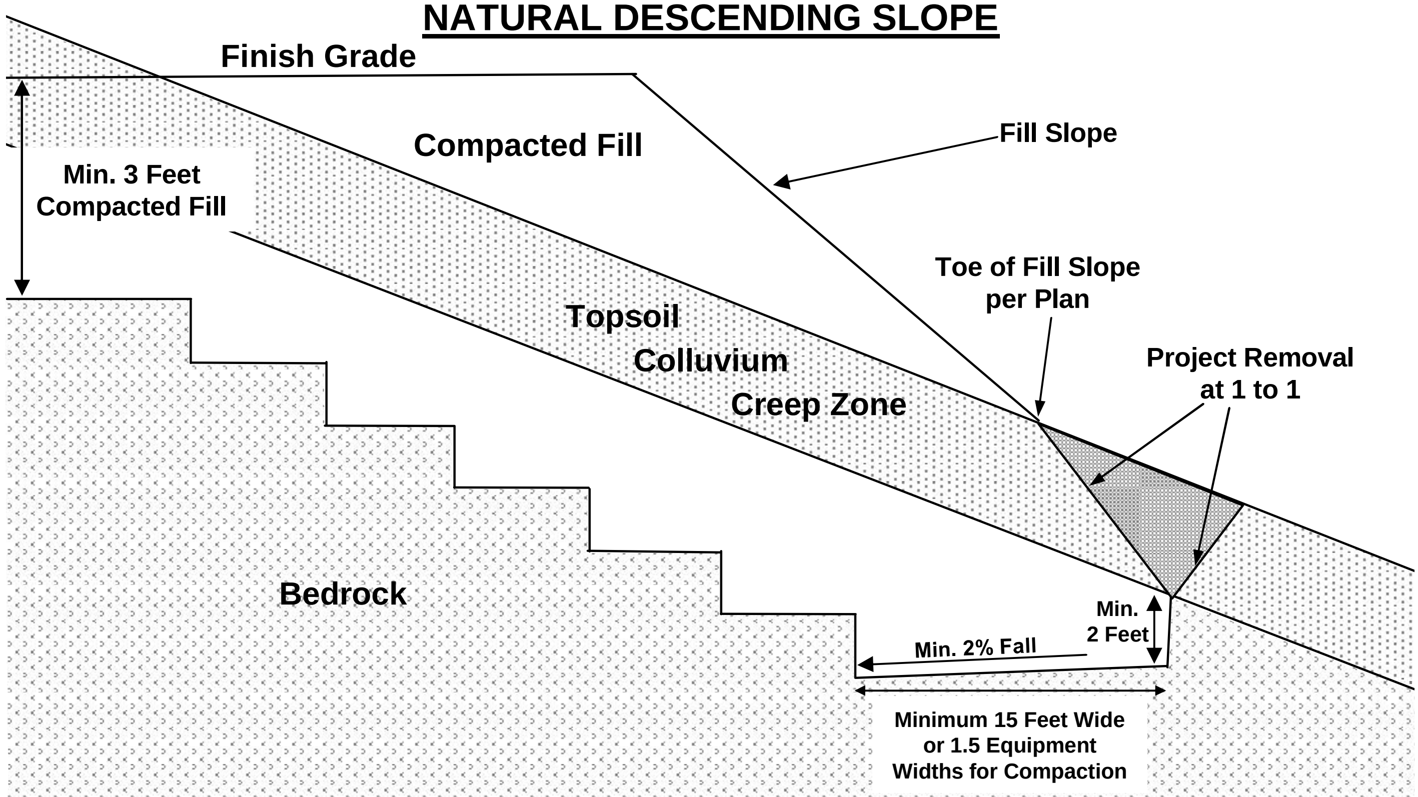
ALTERNATES



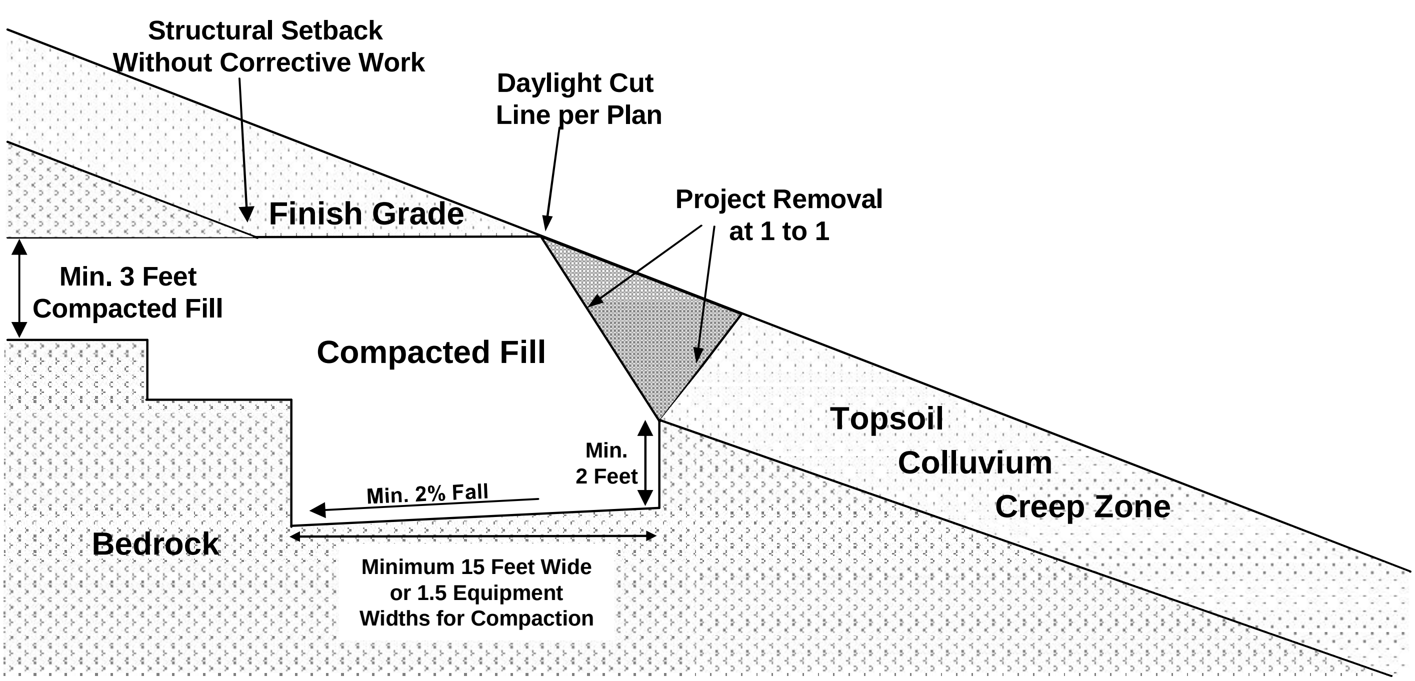


GEOTEK	1548 North Maple Street Corona, California 92880	TYPICAL CANYON CLEANOUT	STANDARD GRADING GUIDELINES PLATE G-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



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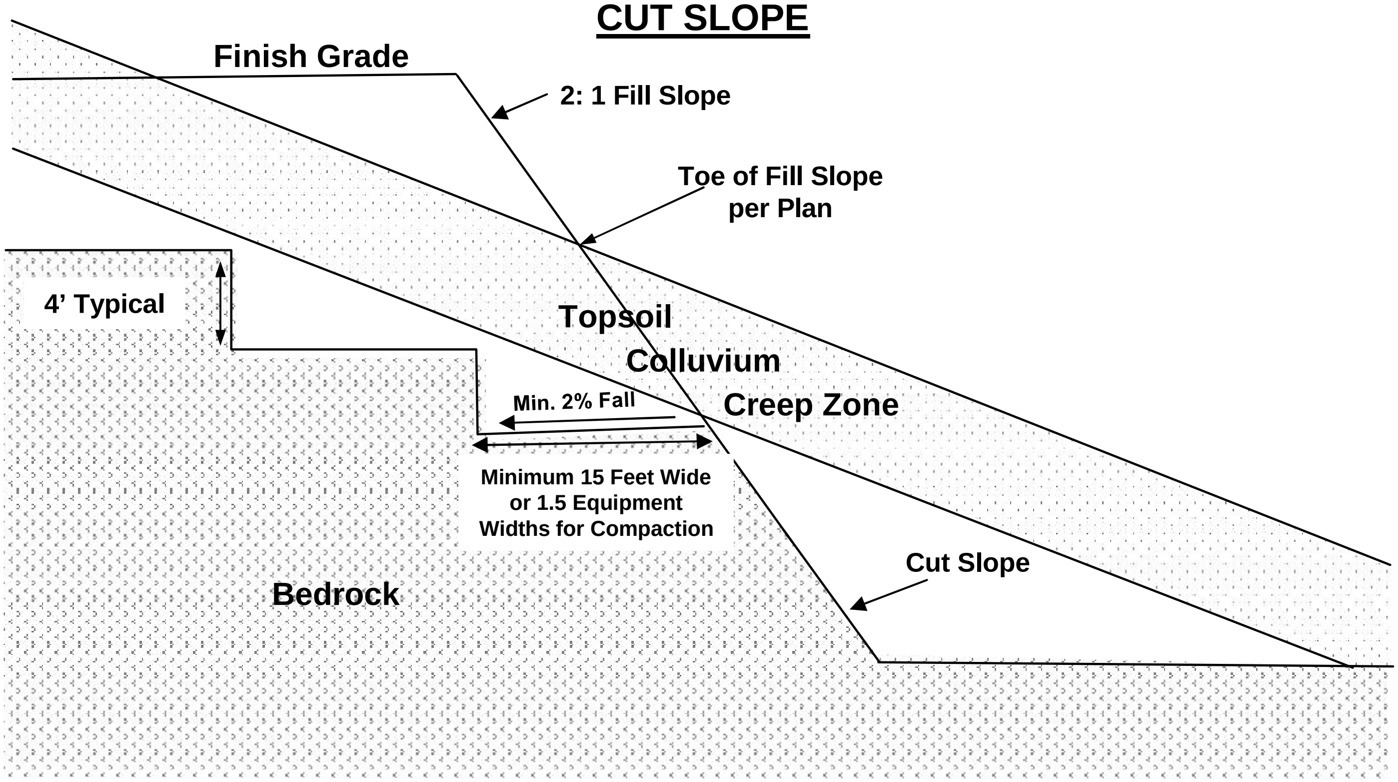
TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE G-2

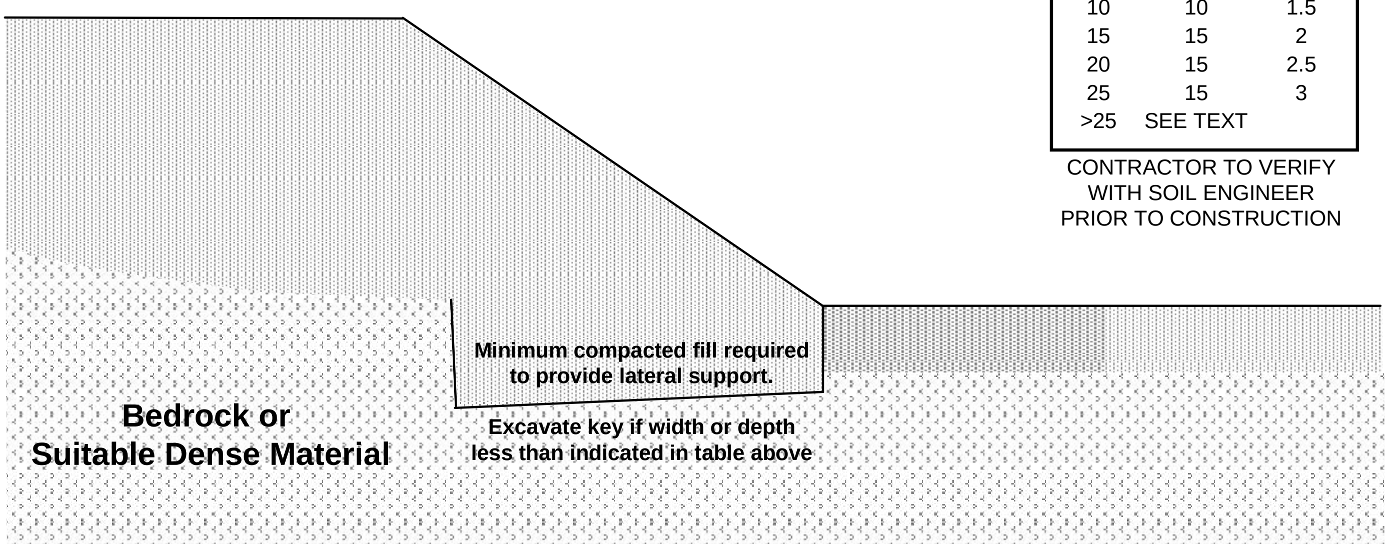
TYPICAL FILL SLOPE OVER CUT SLOPE



TYPICAL FILL SLOPE

SLOPE HEIGHT	MIN. KEY WIDTH	MIN. KEY DEPTH
5	7	1
10	10	1.5
15	15	2
20	15	2.5
25	15	3
>25	SEE TEXT	

CONTRACTOR TO VERIFY WITH SOIL ENGINEER PRIOR TO CONSTRUCTION



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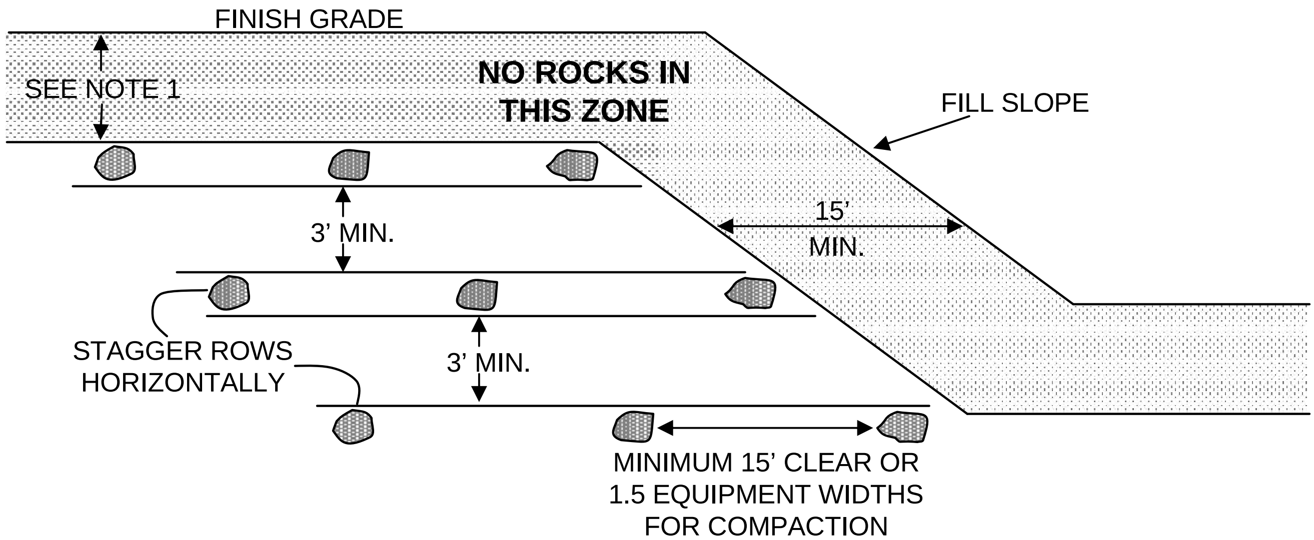
COMMON FILL SLOPE KEYS

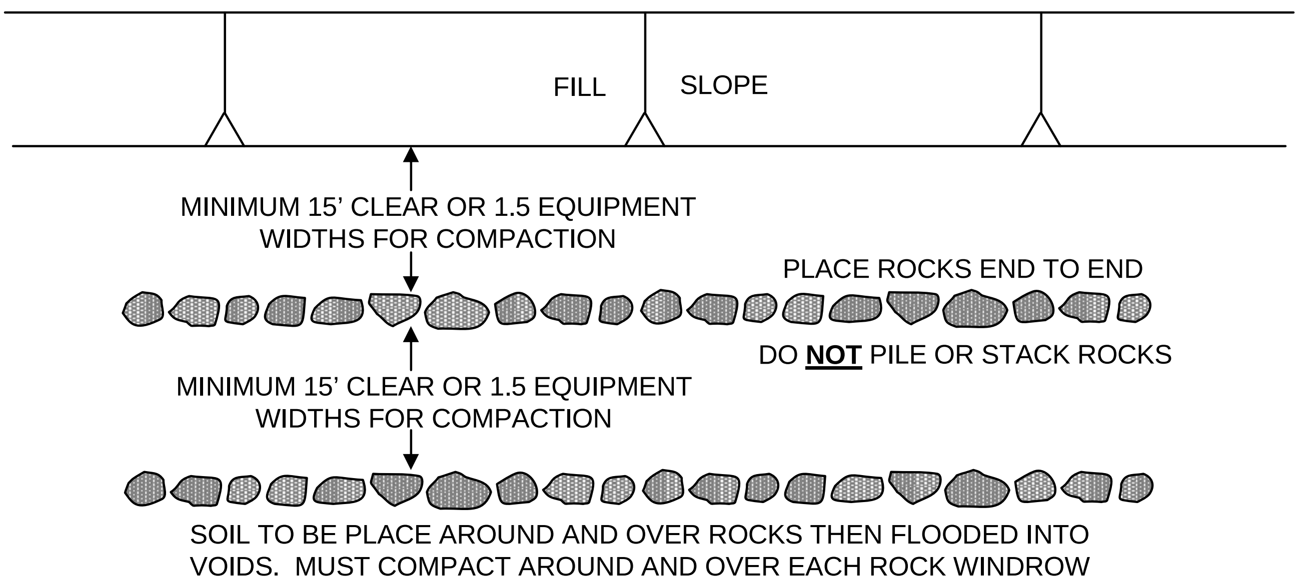
STANDARD GRADING GUIDELINES

PLATE G-3

CROSS SECTIONAL VIEW



PLAN VIEW



NOTES:

1) SOIL FILL OVER WINDROW SHOULD BE 7 FEET OR PER JURISDUICTIONAL STANDARDS AND SUFFICIENT FOR FUTURE EXCAVATIONS TO AVOID ROCKS
2) MAXIMUM ROCK SIZE IN WINDROWS IS 4 FEET IN DIAMETER
3) SOIL AROUND WINDROWS TO BE SANDY MATERIAL SUBJECT TO SOIL ENGINEER ACCEPTANCE
4) SPACING AND CLEARANCES MUST BE SUFFICIENT TO ALLOW FOR PROPER COMPACTION
5) INDIVDUAL LARGE ROCKS MAY BE BURIED IN PITS.

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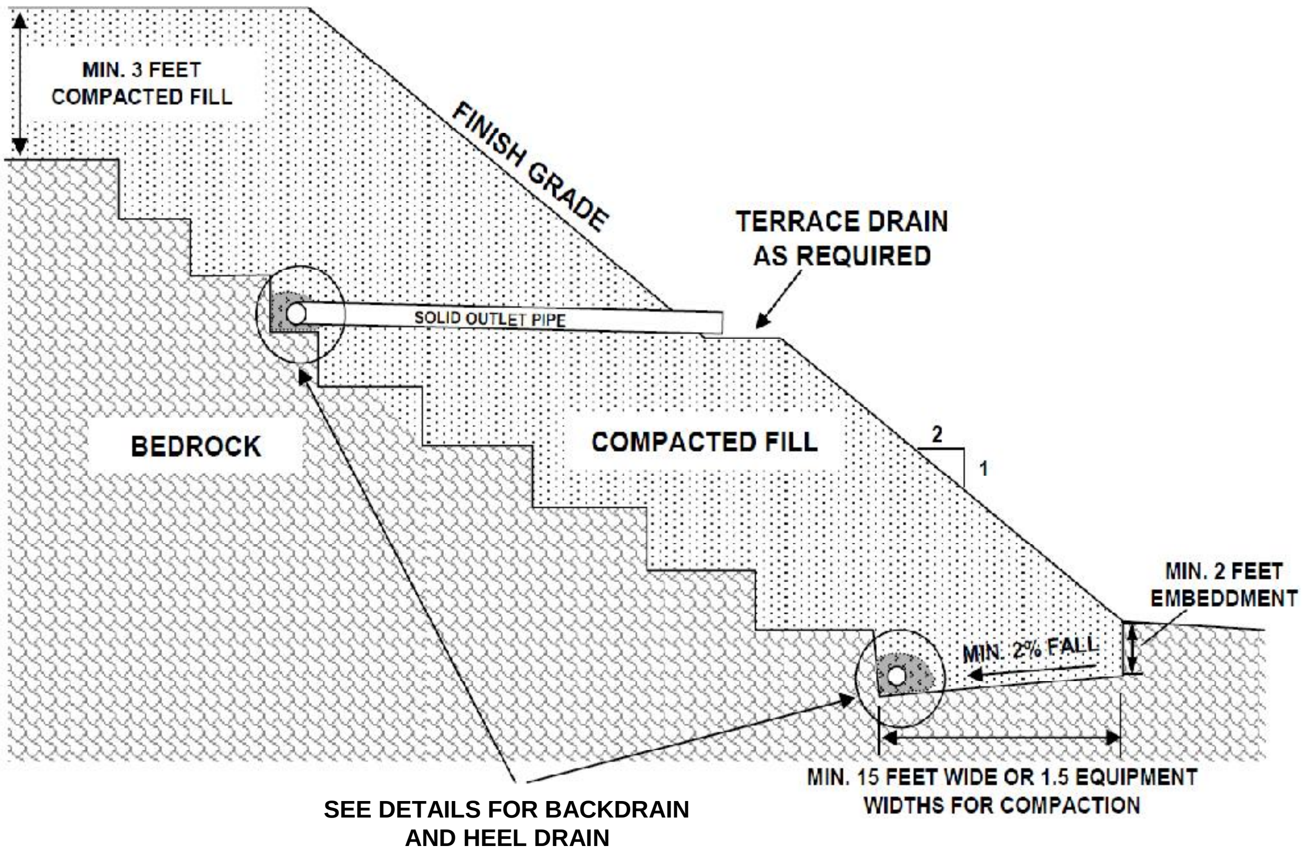
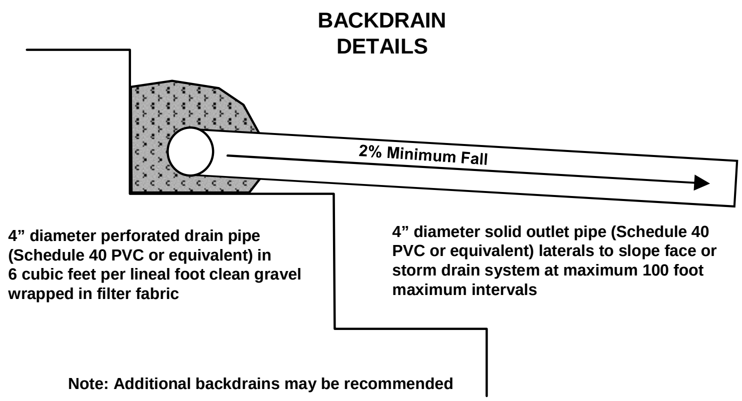
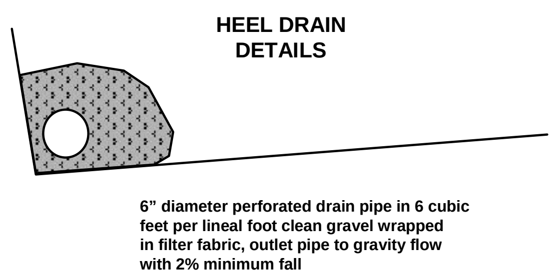
ROCK BURIAL DETAILS

STANDARD GRADING GUIDELINES

PLATE G-4







GEOTEK	1548 North Maple Street Corona, California 92880	TYPICAL BUTTRESS AND STABILIZATION FILL	STANDARD GRADING GUIDELINES PLATE G-5