GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED COMMERCIAL DEVELOPMENT
ASSESSOR'S PARCEL NUMBER 125-050-54-00
3233 OLD HIGHWAY 395
FALLBROOK, SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR

MR. RAFID HAMIKA
370 BRIDGETON CROSS COURT
LAS VEGAS, NEVADA 89148

PREPARED BY

GEOTEK, INC.
1548 NORTH MAPLE STREET
CORONA, CALIFORNIA 92880

PROJECT NO. 3642-SD **JUNE 2, 2020**

GEOTEK

GeoTek, Inc.
1548 North Maple Street, Corona, California 92880
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

June 2, 2020
Project No. 3642-SD

Mr. Rafid Hamika
370 Bridgeton Cross Court
Las Vegas, Nevada 89148

Subject: Geotechnical and Infiltration Evaluation
Proposed Commercial Development
Assessor's Parcel Number 125-050-54-00
3233 Old Highway 395
Fallbrook Area, San Diego County, California

Dear Mr. Hamika:

We are pleased to provide the results of our geotechnical and infiltration evaluation for the proposed commercial development that will be constructed on the subject site in the Fallbrook area of San Diego County, California. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for site preparation, foundation design, infiltration rates, and construction.

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 have any questions, please do not hesitate to contact our office.

Respectfully submitted,
GeoTek, Inc.

Gaby M. Bogdanoff
GE 3133, Exp. 06/30/22
Project Engineer

Edward H. LaMont
CEG 1892, Exp. 07/31/20
Principal Geologist

Kyle R. McHargue
PG 9790, Exp. 02/28/22
Project Geologist

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

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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 data and general information pertinent to the site,
- A site reconnaissance,
- Excavation and logging of eight (8) geotechnical exploratory borings,
- Logging and infiltration testing of an additional six (6) borings in the vicinity of the planned stormwater treatment areas,
- Collection of soil samples,
- Laboratory testing of selected soil samples,
- Review and evaluation of site seismicity, and;
- Compilation of this geotechnical and infiltration report which presents our preliminary recommendations for site development.

The intent of this report is to aid in the evaluation of the site for future proposed commercial development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report may need to be updated based upon our review of the site development plans. These plans should be provided to GeoTek, Inc. for evaluation when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The irregular shaped site is currently comprised of one parcel of land (identified as San Diego County Assessor's Parcel Number [APN] 125-050-54-00), encompassing an area of about 4.37 acres. The site can be accessed from Old Highway 395 to the east.

The site is currently occupied by a multi-use commercial building including a market (Pala Mesa Market), two eateries (Fallbrook Rib Shack and Pala Mesa Pizza) and an insurance office (Arian Knutson Insurance). In addition, there is a stand-alone restaurant (Nessy Burger), a parking lot, and a plant nursery (Pala Mesa Nursery) on the south and north portions of the site. The elevation of the subject site ranges from approximately 330 feet above mean sea level in the northwest to approximately 318 feet above mean sea level in the southeast. Surface drainage in the vicinity is interpreted to be dominantly directed across graded pads and along streets toward the southeast.

The site is in an area largely characterized by residential development, vacant land and some hotel development to the north. The site is bounded by Via Belmonte Roadway, followed by the Quality Inn Fallbrook hotel to the north; Old Highway 395, followed by Interstate 15 to the east; Via Altamira Roadway, followed by residential development to the south; and Via De Todos Santos Roadway, followed by residential development to the west. The site location is presented on Figure 1.

2.1 PROPOSED DEVELOPMENT

GeoTek has reviewed a Site Concept Plan, prepared by Excel Engineering, dated March 27, 2020. It is proposed to construct two new 6,000 square feet retail buildings, a gas station/convenience store with a fuel canopy area and a car wash, paved parking/drive areas, retaining walls and six water quality basins.

It is anticipated that the structures will be one-story, of wood-framed construction and will utilize a conventional shallow system with slab-on grade. We anticipate that the structures will transmit light to moderate structural loads to the subsoils. Once actual loads are known that information should be provided to GeoTek to determine if modifications to the recommendations presented in this report are warranted. Dependent upon the actual structural loads, additional field investigation and analysis may be warranted.

Due to the relatively level nature of the site, we estimate that the maximum depth of cut and fill will be less than about 5 feet, not including any remedial grading.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

Our field exploration was conducted on April 28, 2020 and April 29, 2020 and consisted of eight (8) geotechnical test borings which were excavated with a hollow-stem auger drill rig to a maximum depth of 51.5 feet below ground surface (bgs). A hollow-stem auger with an outside diameter of 8 inches was utilized. The inside diameter of the auger was 4.5 inches. A registered geologist from GeoTek, Inc. logged the exploratory borings. The approximate boring locations are shown on Figure 2. Logs of the exploratory borings are included in Appendix A.

The exploration logs show subsurface conditions at the dates and locations indicated and may not be representative of other locations and times. The stratification lines presented on the logs represent the approximate boundaries between soil types and the transitions may be gradual.

Relatively undisturbed soil samples were recovered at various depths/intervals in the geotechnical borings with a California sampler. The California sampler is a 3-inch outside diameter, 2.4-inch inside diameter, split barrel sampler lined with brass rings. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for testing and evaluation. Standard Penetration Tests (SPT) were performed below depths of 25 feet within Boring B-1 to assist in evaluating the seismic densification potential at the site. The SPT tests were performed in general conformance with ASTM D 1586. The California Ring sampler and SPT data are presented on the boring logs.

Six (6) borings were also excavated in the vicinity of the proposed stormwater infiltration areas to depths of approximately five feet. Preliminary infiltration testing was conducted in these borings in general accordance with the requirements of the County of San Diego. The infiltration tests consisted of drilling eight-inch diameter test holes to the desired depth and installing approximately two inches of gravel in the bottom of the hole. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within the boring. Clean water was then placed in the borings to presoak the holes overnight and percolation testing was performed the following day.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected soil samples obtained during our field exploration. The purpose of the laboratory testing was to confirm the field classification of the soils encountered and to evaluate the physical properties of the soils for use in engineering design and analysis.

Included in our laboratory testing were moisture-density determinations on undisturbed samples. Collapse potential testing was performed on selected samples. Optimum moisture content-maximum dry density relationships were established for typical soil types so that the relative compaction of the subsoils could be determined. Direct shear testing was performed on a selected sample to help evaluate the bearing capacity of the soils. Expansion index testing was performed on selected samples to evaluate the expansion potential of the on-site soils. Chemical testing comprised of pH, soluble sulfate, chloride and resistivity testing was conducted on selected samples. The moisture-density and gradation data are presented on the exploration logs in Appendix A. The maximum density, direct shear, expansion index and chemical test data are presented in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

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

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

More specific to the subject property, the site is located in an area geologically mapped to be underlain by surficial artificial fill and older alluvium (Kennedy, M.P., Tan S.S., Boulevard, K.R. Alvarez, R.M., Watson, M.J. and Gutierrez, C.I, 2007).

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the soils encountered on the site is presented in the following sections. Based on our field exploration and observations, the site is generally underlain by artificial fill and older alluvium at depth.

4.2.1 Existing Fill

Artificial fill soils were encountered within test borings B-2 and B-6 to depths ranging from about 2.5 to 3 feet below existing grade. As encountered, the fill consisted of medium dense silty sand. The materials had various shades of brown and were dry to slightly moist and contained trace amount of fine gravel.

4.2.2 Older alluvium

Older alluvium generally consisted of medium dense to very dense silty sand and poorly graded sand as well as hard sandy silt with varying amounts of gravel. The alluvial materials were encountered at the ground surface or below the existing fill and extended to a maximum explored depth of 51.5 feet. The materials were generally various shades of brown and slightly moist to moist with trace amounts of pinhole pores, oxidation staining and carbon. Faint mottling was noted locally in borings B-4, B-6 and B-8.

Expansion index test results indicate that the near-surface soils exhibit a "very low" expansion potential.

Detailed logs of the subsurface conditions of the site are presented in Appendix A.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during our site visit. 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. Surface drainage in the vicinity is interpreted to be dominantly directed across graded pads and along streets toward the southeast.

Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was not encountered within our test borings to depths up to about 51.5 feet below grade. Based on a review of the California Department of Water Resources Water Data Library, a nearby well (State Well Number 10S03W12D001S) had a depth to groundwater of about 60 feet below ground surface. The well had a ground surface elevation of 252.2 feet above mean sea level. The elevation of the subject site ranges from approximately 330 feet above mean sea level in the northwest to approximately 318 feet above mean sea level in the southeast

4.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. A review of the California Department of Conservation, Regulatory Maps database, the site is not located within a State of California *"Alquist-Priolo"* Earthquake Fault Zone or fault hazard area. The nearest known active fault is the Elsinore Fault Zone-Temecula Section, located about 8.6 miles east of the site.

4.4.1 Seismic Design Parameters

The site is located at approximately 33.3326^{o} Latitude and -117.1618^{o} Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, were 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. As noted using the ASCE 7-16 option on the SEAOC/OSHPD website, the values for S_{M1} and S_{D1} are reported as "null-See Section 11.4.8 (of ASCE 7-16)". As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

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

Assuming that the Cs value calculated by and used by the structural engineer allows for the exclusion per ASCE 7-16, noted above, then a site-specific ground motion analysis is not required. For this assumption and condition, the following seismic design parameters, based on the 2015 National Earthquake Hazards Reduction Program (NEHRP), are presented on the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.083g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.392g
Site Coefficient for Site Class "D," F_a	1.067
Site Coefficient for Site Class "D," F_v	1.908
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.155g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.748g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.77g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.499g
Site Modified Peak Ground Acceleration, PGA_M	0.531g
Seismic Design Category	D

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

4.5 LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, 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, soil plasticity, 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 and some low plastic silts and clays under low confining pressures.

The project site is not located within an area mapped by the State of California as having a liquefaction potential. Based on the current map designation and the estimated depth to groundwater (greater than 51.5 feet) and the density of the materials encountered, it is our opinion that the site possesses a very low potential for liquefaction during a seismic event.

Seismically induced settlement is estimated to be on the order of up to 1-inch total and about 0.5 inches differential over a horizontal distance of 30 feet.

4.6 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instability at this site was not observed during our investigation and the project site is relatively flat. Thus, the potential for landslides is considered negligible for design purposes.

Based on the distance from the Pacific Ocean and open bodies of water compared to the site elevation, they likelihood of a seiche or tsunami is considered remote.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations, and those provided by this firm at a later date are incorporated into the design and construction phases of development. Site development and grading plans should be reviewed by GeoTek, Inc. when they become available.

The on-site soils exhibit “very low” expansion potential. Expansion index testing for near-surface soils should be conducted at the completion of earthwork operations to verify.

5.2 EARTHWORK CONSIDERATIONS

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

5.2.1 Site Clearing & Demolition

In areas of planned grading and improvements, the site should be cleared of vegetation and other deleterious materials. Demolition of existing improvements should include removal of their floor slabs, foundations, pavements and any other below-grade construction. Existing utilities should be properly capped off at the property boundaries and removed or be re-routed around the new building. All debris resulting from site clearing and demolition should be properly disposed of off-site. Voids resulting from site clearing should be backfilled with engineered fill following proper preparation as described in the following report sections.

5.2.2 Remedial Grading

All existing fill or the upper loose portions of the older alluvium should be removed from all structural areas to expose competent alluvium. Competent alluvium is defined as materials not visually porous with an in-place selective compaction of at least 85 percent of the soil's maximum dry density as defined per ASTM D 1157. Combined fill and alluvial removals on the order of 3 to 4 feet deep are anticipated to be required within building areas. However, some localized deeper removals/over-excavations may be encountered during grading operations. As a minimum, all foundations should be underlain by at least 2 feet of competent compacted engineered fill. Removals/over-excavations should extend a minimum of five feet outside the proposed building envelope.

Soils below proposed hardscape areas should be observed by a representative of this firm. All existing fill should be removed below hardscape areas. The upper 12 inches should be moistened to at least the optimum moisture content and densified to a minimum relative compaction of 90 percent (ASTM D 1557).

5.2.3 Preparation of Excavation Bottoms

A representative of this firm should observe the bottom of all excavations. Upon approval, the exposed soils and all soils in areas to receive engineered fill should be scarified to a depth of approximately 12 inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

5.2.4 Engineered Fills

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. Engineered fill should be placed in loose lifts with a thickness of eight inches or less, moisture conditioned to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

5.2.5 Excavation Characteristics

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

5.2.6 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of about 5 to 15 percent may be considered for undocumented fill materials requiring removal and recompaction. A shrinkage factor of about 0 to 10 percent may be considered for excavation and recompaction of the native older alluvium. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.10 foot may be anticipated resulting from preparation of the underlying soils.

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 below. Based on laboratory test results and the recommendations provided in this report, subsequent to earthwork operations it is anticipated that the as-graded near-surface soils may have a "very low" expansion potential.

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

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

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)	Single-Story - 12
Minimum Foundation Width (Inches)*	Single-Story - 12
Minimum Slab Thickness (actual)****	4 – Actual
Sand Blanket and Moisture Retardant Membrane Below On-Grade Building Slabs	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab or No. 3 bars at 24 inch centers
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom
Effective Plasticity Index***	N/A
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.
** Sand should have a sand equivalent of at least 30. Alternatively, CAL Green Section 4.505.2 can be adopted.
*** Effective plasticity index should be verified at the completion of rough grading.
****Slab thickness and reinforcement should be determined necessary by the structural engineer.

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 of additional laboratory testing of samples obtained near finish pad grade.

An allowable bearing capacity of 1,800 pounds per square foot (psf) may be used for design of footings 12 inches deep and 12 inches wide. This value may be increased by 400 pounds per square foot for each additional 12 inches in depth and 100 pounds per square foot for each additional 12 inches in width to a maximum value of 3,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

Structural foundations may be designed in accordance with the 2019 CBC, and to withstand a total static settlement of 1 inch and maximum differential static settlement of 0.5 inches over a horizontal distance of 30 feet. Seismically induced settlement is also anticipated to be about 1-inch total and 0.5 inches differential over a 30-foot span.

The passive earth pressure may be computed as an equivalent fluid having a density of 250 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 0.4 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

If desired, the building floor slab may be designed using an estimated subgrade modulus of 150 pci, which is based on a value typically obtained from a 1 foot by 1 foot plate bearing test. Depending on how the floor slab is loaded, the subgrade modulus may need to be geometrically modified.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 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 on 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, 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.

A two-inch layer of clean sand with a sand equivalent of at least 30 should be placed over the moisture vapor retardant membrane to promote setting of the concrete. The moisture in the sand should not exceed two percent below the optimum moisture content.

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

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

GeoTek recommends that a qualified person, such as a flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the buildings be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person 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.

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

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

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

5.3.3 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2019 CBC or County of San Diego requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movement and/or differential settlement. If large enough, these movements can compromise the integrity of the improvements.

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

- The bottom of any proposed foundations should be deepened so as to extend below a 1:1 (h:v) upward projection from the bottom edge of the nearest excavation and the bottom edge of the closest footing.

5.3.4 Soil Corrosivity

Based on the chemical test results presented in Appendix B, the minimum resistivity test results indicate that the on-site soils are "corrosive" to buried ferrous metal. This corrosion classification is obtained from "Corrosion Basics: An Introduction," by Pierre R. Roberge, 2nd Edition, 2005. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer. Additional corrosion testing should be performed at the time of site grading to assess the corrosion of potential of the as-graded soils.

5.3.5 Soil Sulfate Content

The results of chemical testing performed on a representative sample of the site soils indicate soluble sulfate contents less than 0.1 percent by weight. Soluble sulfate contents of this level would be in the range of "not applicable" per Table 4.2.1 of ACI 318. Based on the test results and Table 4.3.1 of ACI 318, special concrete mix design is not anticipated to be necessary to resist sulfate attack. Additional soluble sulfate testing should be performed during site grading to further evaluate the as-grade sulfate exposure.

5.3.6 Import Soils

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

5.3.7 Concrete Flatwork

5.3.7.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices typically utilized in 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. “Very low” expansive soils below exterior flatwork should be pre-saturated to a minimum of 100 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 Diego specifications, and under the observation and testing of GeoTek, Inc. and a City inspector, if necessary.

5.3.7.2 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, Inc. suggests that control joints be placed in two directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete vertical retaining walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations should be designed in accordance with *Section 5.3.1* of this report. A minimum foundation embedment of 12 inches into engineered compacted fill with "very low" expansion potential. Structural needs may govern and should be evaluated by the project structural engineer.

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

The backfill material placement for all earth retention structures should meet the requirement of *Section 5.4.4* in this report.

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

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

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

5.4.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES		
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) Native Backfill*	Equivalent Fluid Pressure (pcf) Imported Backfill**
Level	43	37
2:1	69	53

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

** Imported granular backfill to consist of soil with angle of internal friction of at least 34 degree and an expansion index of less than or equal to 20.

It should be noted that the 2019 CBC only requires the additional earthquake induced lateral force to be considered on retaining walls in excess of 6 feet. Additional lateral forces can be induced on retaining walls during an earthquake. For level backfill, the minimum earthquake-induced load onto retaining walls may be considered to be equivalent to a fluid pressure of 16 pcf. The seismic pressure can be assumed to be a conventional triangular distribution seismic.

5.4.3 Restrained Retaining Walls

Retaining walls that will be restrained prior to placing and compacting backfill material, or that have reentrant or male corners, should be designed for an at-rest equivalent fluid pressure of 64 pcf, plus any applicable surcharge loading, for native backfill (EI<20) and level back slope condition. If backfilled with imported granular soil (EI<20), the walls may be designed for an at-rest pressure of 53 pcf. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.4.4 Retaining Wall Backfill and Drainage

Retaining wall backfill should consist of materials with expansion index (EI) ≤ 20 and free of deleterious and/or oversized materials. The wall backfill should also 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 wall and extend up from the back drain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. 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 8-

inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained. Bracing of the walls during backfilling and compaction may also be necessary.

All earth retention structures should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressure build up. As a minimum, backdrains should consist of a four-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one cubic foot per lineal foot of ¾- to 1-inch clean crushed rock or equivalent, wrapped in filter fabric (Mirafi 140N or approved equivalent). The drain system should be connected to a suitable outlet, as determined by the civil engineer. Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

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 wall is undesirable.

5.5 PAVEMENT DESIGN

Based on laboratory testing of onsite soils, preliminary pavement sections have been estimated for an assumed R-value of 40. In the absence of specific Traffic Indices (TI's) provided by the designer, we have provided structural sections for a range of TI's.

PRELIMINARY ASPHALT PAVEMENT STRUCTURAL SECTION		
Traffic Index	Asphaltic Concrete Thickness (inches)	Aggregate Base Thickness (inches)
6.0	3	7
6.5	3	8
7.0	3	9

Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us. Upon receiving a desired traffic index, our preliminary asphalt concrete pavement design can be updated.

The provided pavement sections are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving will result in premature pavement failure. Final pavement design should be checked by testing of soils exposed at subgrade (the upper 12 inches) after final grading has been completed.

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

5.6 INFILTRATION TEST RESULTS

Field percolation testing was performed on April 20, 2020 and April 21, 2020. Percolation rates obtained from the infiltration testing were converted to a field infiltration rate using the Porchet Method. The field infiltration rates calculated are indicated in the following table:

SUMMARY OF FIELD INFILTRATION RATES			
Boring/Area	Depth of Test (Feet)	Material Encountered at Depth of Test	Field Infiltration Rate (Inches per Hour)
I-1	5	Silty Fine Sand to Sandy Silt	0.09
I-2	5	Silty Fine Sand	0.80
I-3	5	Silty Fine Sand to Sandy Silt	0.33
I-4	5	Silty Fine Sand	0.80
I-5	5	Poorly Graded Sand	0.69
I-6	5	Silty Fine Sand to Sandy Silt	0.09

The percolation data sheets and infiltration conversion worksheets are presented in Appendix C. The field infiltration rates presented above do not incorporate a safety factor. The civil engineer should assign a suitable safety factor to these values prior to determining the design infiltration rate.

In addition, over the lifetime of the basins, 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 rates to design the infiltration system.

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

5.7 POST CONSTRUCTION CONSIDERATIONS

5.7.1 Landscape Maintenance and Planting

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

Overwatering should be avoided. An abatement program to control ground-burrowing rodents should be implemented and maintained. Burrowing rodents can decrease the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundations. This type of landscaping should be avoided.

5.7.2 Drainage

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

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

5.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that specifications and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek, Inc. representatives be present during site grading and foundation construction to observe and document proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek, Inc. representatives 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 where necessary.
- Observe the fill for uniformity during placement, including utility trench backfill. Also, perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

If requested, a construction observation and compaction report can be provided by GeoTek, Inc. 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. INTENT

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

The scope of our evaluation is limited to the boundaries of the subject property. This review does not and should in no way be construed to encompass any areas beyond the specific area of the 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 geotechnical engineering standards normally used on similar projects in this locality.

7. LIMITATIONS

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

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

Since 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 of any kind is expressed or implied. Standards of care/practice are subject to change with time.

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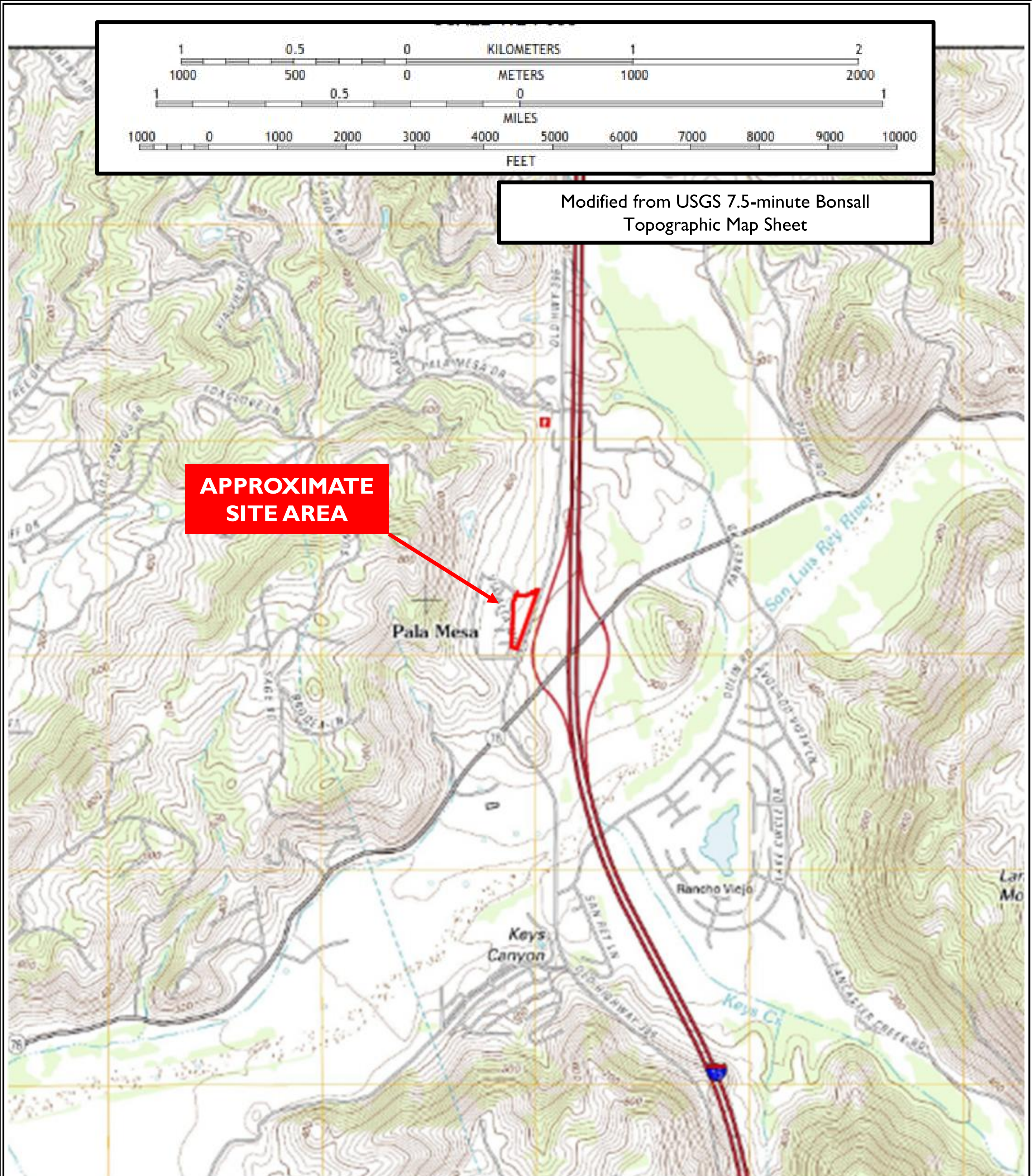


Mr. Rafid Hamika
APN: 125-050-54-00
Fallbrook, San Diego County, California

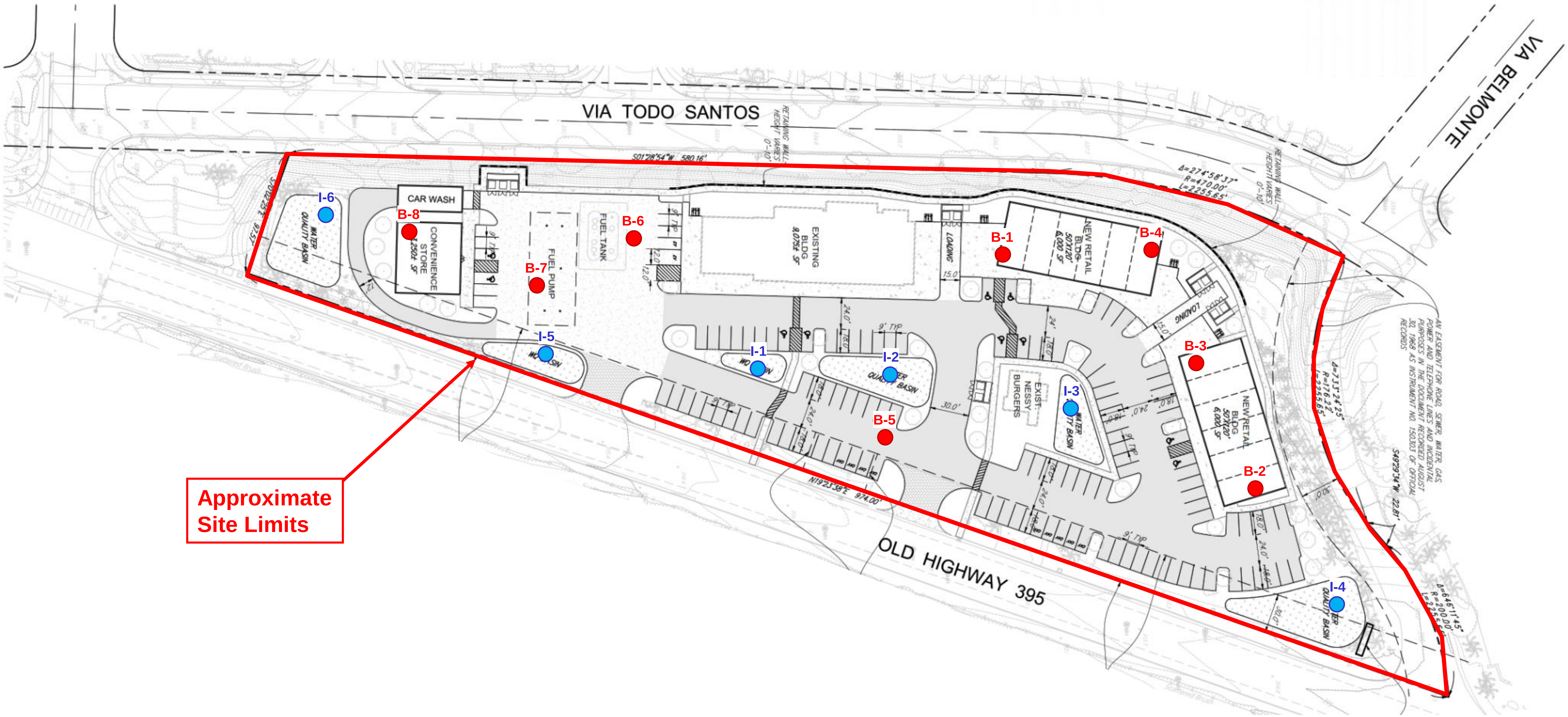
GeoTek Project No. 3642-SD

NORTH

Figure 1

Site Location and General Topography Map

GEOTEK



Legend
(Locations are approximate)

B-8 - Exploratory Boring

I-6 - Percolation Boring

Mr. Rafid Hamika
Proposed Commercial Development
APN: 125-050-54-00
3233 Old Highway 395
Fallbrook, San Diego County, California

Project No. 3642-SD

GEOTEK

Figure 2

Exploration Location Map

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Proposed Commercial Development
Fallbrook, San Diego County, California
Project No. 3642-SD

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 earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

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

B – BORING LOG LEGEND

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/28/2020

BORING NO.: B-1
Sheet 1 of 2

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium			MD, SH, EI, SR
		10 10 15	R1	SM	Silty f SAND, dark reddish brown, slightly moist, medium dense	4.0	115.8	
5		20 27 37	R2		Same, very dense, trace pinhole pores	5.8	115.5	
		17 26 29	R3	SM	Silty f-m SAND, dark reddish brown, slightly moist, very dense, slight porosity	6.1	123.3	
10		29 30 38	R4		Same, oxidation staining	6.0	125.2	
15		22 32 48	R5		Same, moist			
20		10 15 17	S1	SP	F SAND with SILT, dark reddish brown, slightly moist, dense			
					Drilling slowed			
25		9 10 15	S2		Same, oxidation staining			
					Water added downhole			
30		7 8 11	S3		Same, minor mechanical fracturing			

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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/28/2020

BORING NO.: B-1 Sheet 2 of 2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Drilling slowed			
35	SPT	8 8 10	S4		Same			
					Water added downhole			
40	SPT	11 20 11	S5	GM	Silty f-m SAND with GRAVEL, reddish brown, slightly moist, dense, gravel lens from 40-40.5'			
45	SPT	11 15 19	S6	SM	Silty f SAND, dark brown, moist, dense, trace carbon			
50	SPT	12 20 28	S7		Same, micaceous			
					BORING TERMINATED AT 51.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			

LEGEND

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

Lab testing:

AL = Atterberg Limits | 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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/28/2020

BORING NO.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
				SM	**Artificial Fill** Silty f SAND, light brown, dry, medium dense, trace asphalt and gravel			
	Ring	4 4 7	R1	SM	**Older Alluvium** Silty f SAND, reddish brown, slightly moist to moist, medium dense, roots/rootlets	6.1	116.4	
5								
	Ring	29 46 47	R2		Same, very dense, trace caliche	6.1	123.3	
10	Ring	29 39 44	R3	SM-ML	Silty f SAND/Sandy SILT, dark reddish brown, slightly moist, very dense/hard			
15	Ring	22 29 42	R4		Same, moist, trace pinhole pores	6.9	132.5	
					Drilling slowed			
20	Ring	11 16 30	R5	ML	Sandy SILT, dark reddish brown, moist, hard, slightly mottled, trace caliche			
					BORING TERMINATED AT 21.5 FEET N groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

Lab testing: AL = Atterberg Limits 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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/28/2020

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
					Older Alluvium			
		11 14 16	R1	SM	Silty f SAND, brown, slightly moist to moist, medium dense	5.4	116.6	HC
5		17 24 36	R2		Same, moist	7.1	115.4	
		21 34 43	R3	SM	Silty f-m SAND, dark reddish brown, moist, very dense	9.5	131.0	
10								
		14 36 37	R4		Same, trace caliche, massive			
15								
		22 32 35	R6	SP	F-m SAND with SILT, dark reddish brown, moist to very moist, very dense, trace f gravel	7.7	125.2	
20								
					BORING TERMINATED AT 20.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

Lab testing:
AL = Atterberg Limits | 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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/28/2020

BORING NO.: B-4

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
					Older Alluvium			
		5 4 14	R1	ML	Sandy SILT, reddish brown, slightly moist, very stiff	3.4	117.8	
5		30 50/5	R2	SM	Silty f SAND, dark reddish brown, slightly moist, very dense	5.5	111.7	
		28 38 50/5	R3		Same, trace pinhole pores	6.1	128.8	
10		28 42 50/5	R4		Same, moist, trace caliche Drilling slowed			
15		12 22 41	R5	SM-ML	Silty f SAND/Sandy SILT, dark reddish brown, moist, very dense/hard, trace pinhole pores	8.7	131.8	
20		21 34 42	R6		Same, trace mottling and oxidation			
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

Lab testing:
AL = Atterberg Limits; 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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/29/2020

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Asphaltic Concrete = 1", No Base			
					Older Alluvium			
	Ring	20 34 45	R1	SM	Silty f SAND, dark reddish brown, slightly moist, very dense	5.5	115.1	
5	Ring	20 45 50/5	R2		Same, moist	5.2	130.2	
10	Ring	48 50/4	R3	SM	Silty f-m SAND, reddish brown, slightly moist, very dense, trace caliche	5.4	124.9	
15	Ring	21 46 50/5	R4		Same, trace pinhole pores			
20	Ring	25 39 50/5	R5	SM-ML	Silty f SAND/Sandy SILT, reddish brown, slightly moist, very dense/hard, trace pinhole pores			
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

Lab testing:
AL = Atterberg Limits 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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/29/2020

BORING NO.: B-6

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Artificial Fill			
	Ring	3 6 8	R1	SM	Silty f-m SAND, light brown, dry to slightly moist, medium dense, trace f gravel	4.9	113.8	
					Older Alluvium			
5	Ring	12 22 18	R2	SM	Silty f SAND, dark reddish brown, moist, dense, trace pinhole pores	6.7	123.6	HC
	Ring	21 40 50/5	R3		Same	8.4	119.6	
10								
	Ring	21 34 45	R4	SM	Silty f-m SAND, dark reddish brown, moist, very dense			
15								
					Drilling slowed			
	Ring	21 42 50/6	R5		Same, slightly mottled	8.7	133.1	
20					**BORING TERMINATED AT 19.5 FEET**			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

Lab testing:
AL = Atterberg Limits | 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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/29/2020

BORING NO.: B-7

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
					Older Alluvium			
	Ring	8 16 24	R1	SM	Silty f SAND, dark reddish brown, moist, dense	8.7	127.1	HC
5	Ring	10 14 20	R2		Same, oxidation staining	8.6	127.4	
10	Ring	18 29 36	R3	SM	Silty f-m SAND, reddish brown, slightly moist, very dense, slightly mottled	5.0	130.8	
15	Ring	19 34 50	R4		Same, trace clay	8.7	135.1	
20	Ring	18 32 42	R5		Same, very moist			
					BORING TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

Lab testing:
AL = Atterberg Limits
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:	Mr. Rafid Hamika	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	3233 Old Highway 395	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	3642-SD	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	4/29/2020

BORING NO.: B-8

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
					Older Alluvium			
		4 6 10	R1	SM	Silty f-m SAND, dark reddish brown, very moist, medium dense	10.2	125.4	
5		10 24 34	R2		Same, very dense	10.8	128.6	
10		15 24 34	R3		Same, very moist, oxidation staining, slightly mottled, massive	9.8	130.3	
15		18 28 45	R4	SP	F-m SAND with SILT, dark reddish brown, very moist, very dense, oxidation staining, slightly mottled	9.1	129.6	
20		15 23 29	R5		Same			
25					**BORING TERMINATED AT 20.5 FEET** No groundwater encountered Boring backfilled with soil cuttings			
30								

LEGEND

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

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

APPENDIX B

LABORATORY TEST RESULTS

Proposed Commercial Development
Fallbrook, San Diego County, California
Project No. 3642-SD

GEOTEK

SUMMARY OF LABORATORY TESTING

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

In-Situ Moisture and Density
The natural water content was determined (ASTM D 2216) on samples of the materials recovered from the subsurface exploration. In addition, in-place dry density determination (ASTM D 2937) were performed on relatively undisturbed samples to measure the unity weight of the subsurface soils. Results of these tests are shown on the logs at the appropriate sample depths in Appendix A.

Moisture-Density Relationship
Laboratory testing was performed on a sample obtained during the subsurface exploration. The laboratory maximum dry density and optimum moisture content was determined in general accordance with ASTM D 1557. The test results are provided herein.

Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080. The rate of deformation is approximately 0.035 inch per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The results of the testing are provided herein.

Expansion Index
The expansion potential of the soils was determined by performing expansion index testing on a representative sample in general accordance with ASTM D 4829. The results of the testing are provided herein.

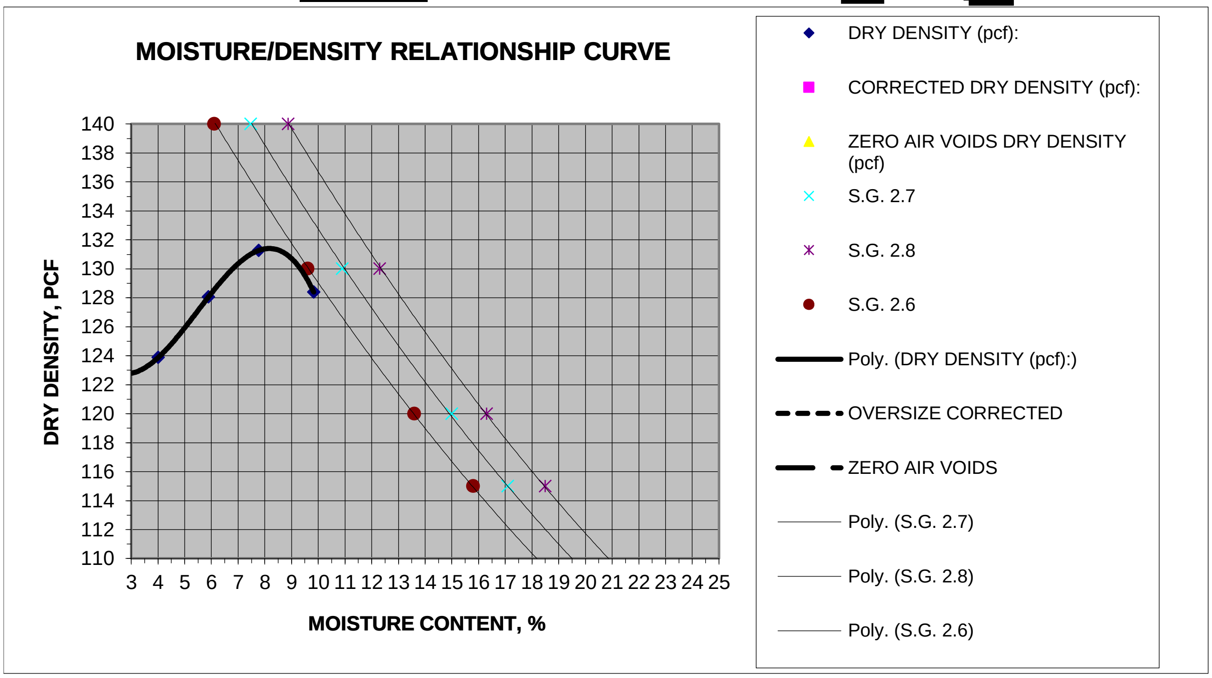
Hydro-collapse
Selected samples obtained from the property were tested to estimate their potential for hydro-collapse. The tests were conducted in general accordance with ASTM D4546. The results of these tests are presented herein.

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. The results of the testing are provided herein.

MOISTURE/DENSITY RELATIONSHIP

Client: Rafid Hamika
Job No.: 3642-SD
Project: 3233 Old HWY 395
Lab No.: Corona
Location:
Material Type: Red Brown Silty Fine Sand
Material Supplier: -
Material Source: -
Sample Location: B-1 @ 0 - 5 ft
-
Sampled By: KM
Date Sampled: 4/30/2020
Received By: DLI
Date Received: 5/4/2020
Tested By: DA
Date Tested: 5/18/2020
Reviewed By: -
Date Reviewed: -

Test Procedure: ASTM D1557
Method: A
Oversized Material (%): 1.1
Correction Required: ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	131.5	**@ Optimum Moisture, %**	8.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

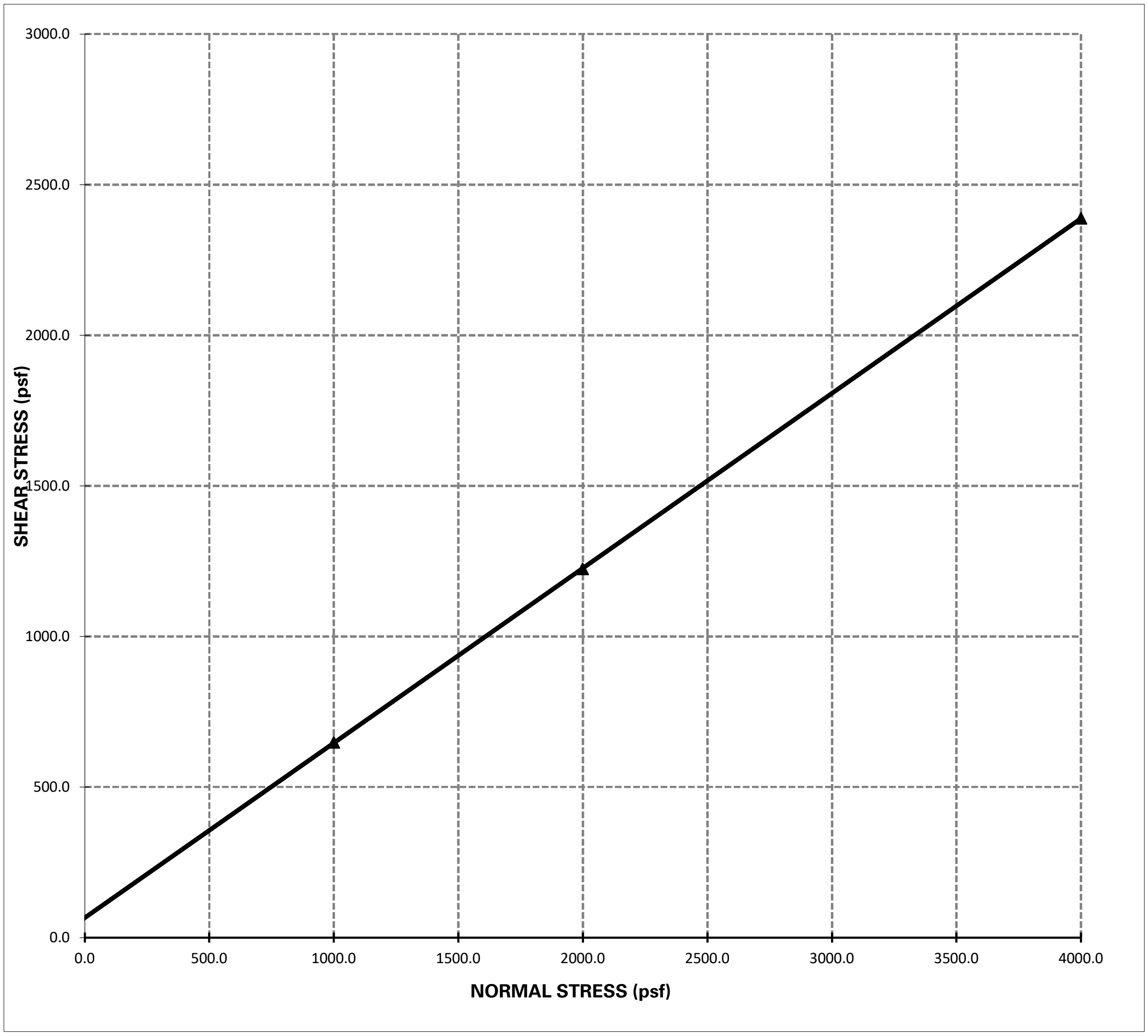
Classification:

Unified Soils Classification:

AASHTO Soils Classification:

DIRECT SHEAR TEST

Project Name:	3233 Old HWY 395	***Sample Location:***	B-1 @ 0 - 5 ft
Project Number:	3642-SD	***Date Tested:***	6/1/2020



Shear Strength: $\Phi =$ **30.1** $^{\circ}$, **C = 66.00 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.

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Rafid Hamika	***Tested/ Checked By:***	DA Lab No Corona
Project Number:	3642-SD	***Date Tested:***	5/20/2020
Project Location:	3233 Old HWY 395	***Sample Source:***	B-1 @ 0 - 5 ft
		Sample Description:	

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	785.7
B	Weight of ring (gm)	364.2
C	Net weight of sample (gm)	**421.5**
D	Wet Density, lb / ft3 (C*0.3016)	**127.1**
E	Dry Density, lb / ft3 (D/1.F)	**117.7**

SATURATION DETERMINATION

F	Moisture Content, %	8.0
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.1**

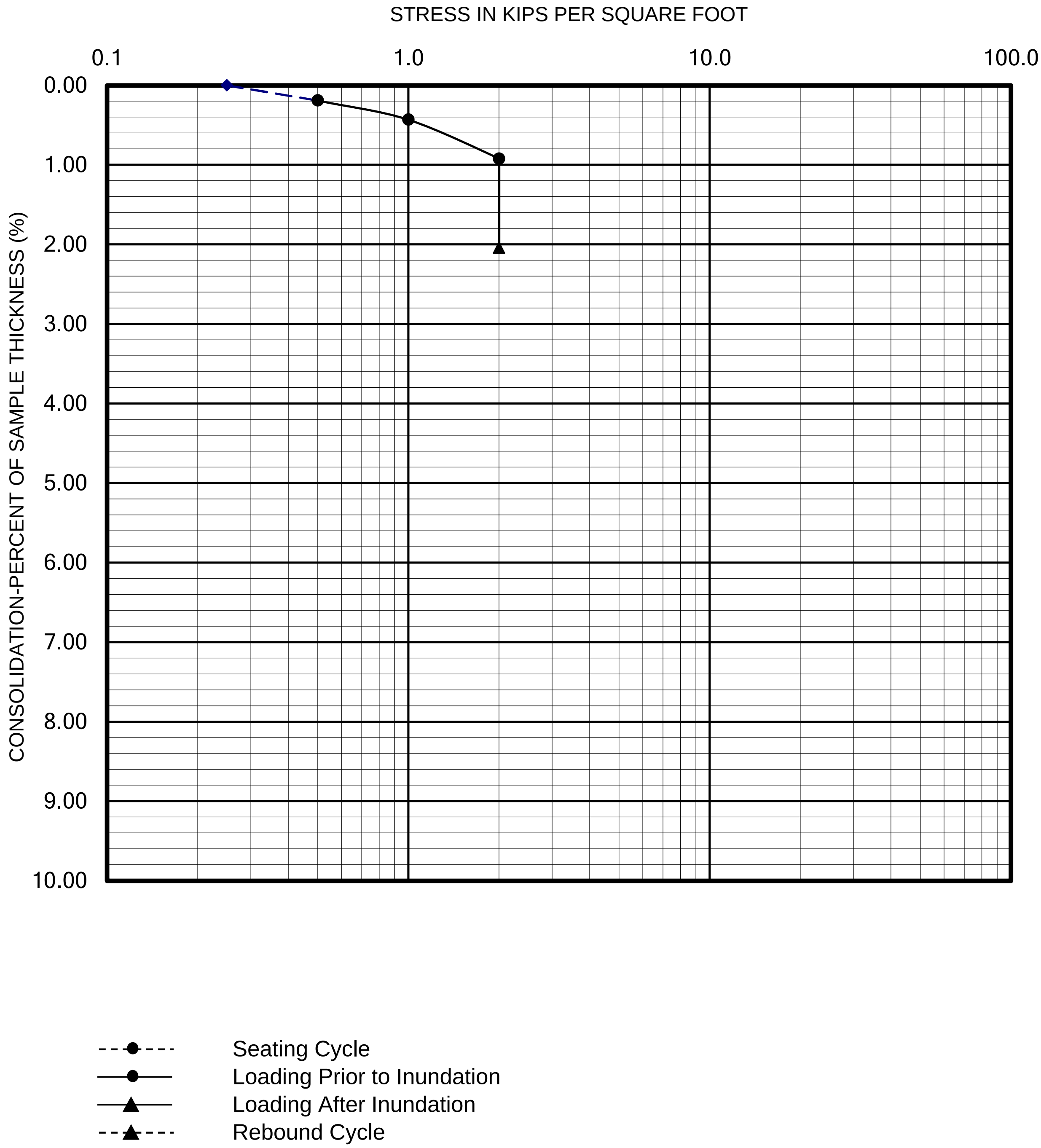
READINGS

DATE	TIME	READING	
5/20/2020	9:07	0.3900	Initial
	9:17	0.3900	10 min/Dry
5/21/2020	9:17	0.3900	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
807.4	**13.1**

EXPANSION INDEX = 0



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

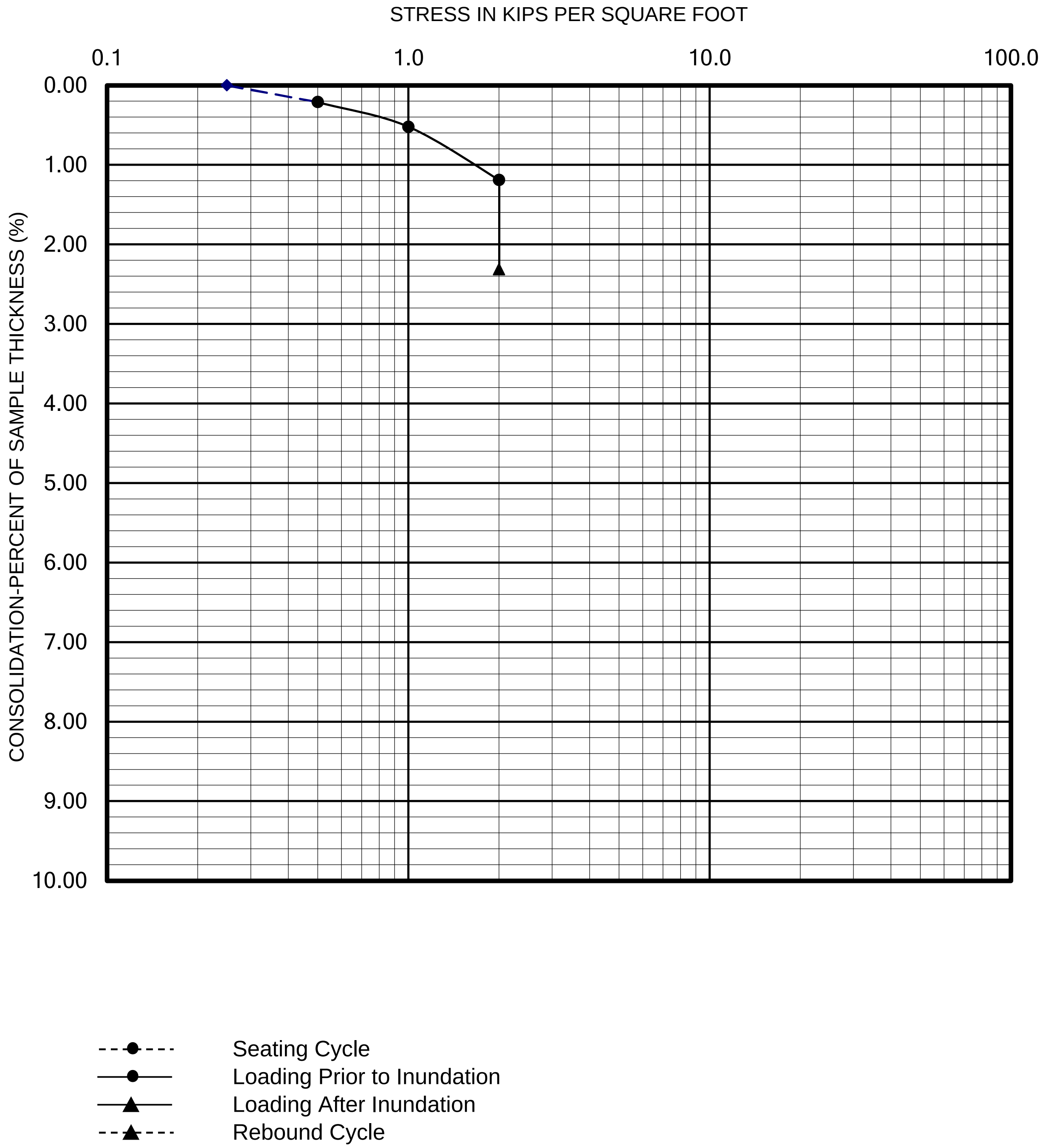
GEOTEK

COLLAPSE REPORT

Sample:

B-3 @ 2 ft

CHECKED BY:	Lab: DI
PROJECT NO.: 3642-SD	Date:



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

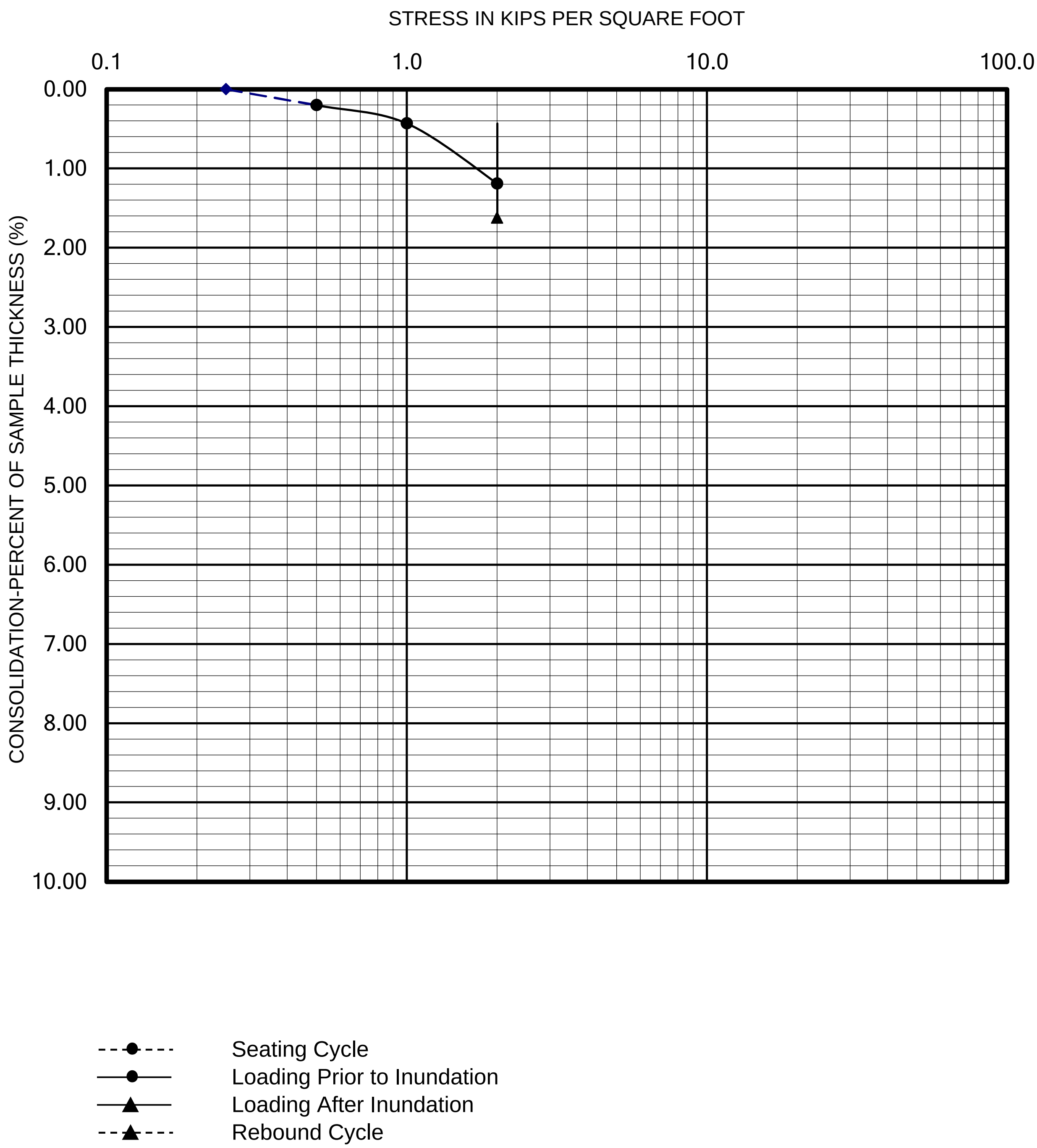
GEOTEK

COLLAPSE REPORT

Sample:

B-6 @ 5 ft

CHECKED BY:	Lab: DI
PROJECT NO.: 3642-SD	Date:



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample:

B-7 @ 2 ft

CHECKED BY:	Lab: DI
PROJECT NO.: 3642-SD	Date:

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: 3233 Old Highway 395, Fallbrook
Client Job Number: 3642-SD
Project X Job Number: S200514B
May 18, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-1	0-5	95.7	0.0096	52.3	0.0052	24,120	3,886	7.00	274	0.33	134.9	ND	0.7	90.7	ND	17.2	66.6	0.7	ND

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

APPENDIX C

INFILTRATION TEST DATA

Proposed Commercial Development
Fallbrook, San Diego County, California
Project No. 3642-SD

GEOTEK

Client:	**Mr. Rafid Hamika**
Project:	**3233 Old Highway 395**
Project No:	**3642-SD**
Date:	**6/2/2020**

Boring No. **I-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	40.50	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	40.00	inches
Total Test Hole Depth, D_T =	60.00	inches

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

$H_O = D_T - D_O$ =	20.00	inches
$H_F = D_T - D_F$ =	19.50	inches
$\Delta H = \Delta D = H_O - H_F$ =	0.50	inches
$H_{avg} = (H_O+H_F)/2$ =	19.75	inches

I_t = 0.09 **Inches per Hour**

GEOTEK

Client:	**Mr. Rafid Hamika**
Project:	**3233 Old Highway 395**
Project No:	**3642-SD**
Date:	**6/2/2020**

Boring No. **I-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	44.00	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	40.00	inches
Total Test Hole Depth, D_T =	60.00	inches

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

$H_O = D_T - D_O$ =	20.00	inches
$H_F = D_T - D_F$ =	16.00	inches
$\Delta H = \Delta D = H_O - H_F$ =	4.00	inches
$Havg = (H_O+H_F)/2$ =	18.00	inches

I_t = 0.80 **Inches per Hour**

GEOTEK

Client:	**Mr. Rafid Hamika**
Project:	**3233 Old Highway 395**
Project No:	**3642-SD**
Date:	**6/2/2020**

Boring No. **I-3**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	41.75	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	40.00	inches
Total Test Hole Depth, D_T =	60.00	inches

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

$H_O = D_T - D_O$ =	20.00	inches
$H_F = D_T - D_F$ =	18.25	inches
$\Delta H = \Delta D = H_O - H_F$ =	1.75	inches
Havg = $(H_O+H_F)/2$ =	19.13	inches

I_t = 0.33 **Inches per Hour**

GEOTEK

Client:	**Mr. Rafid Hamika**
Project:	**3233 Old Highway 395**
Project No:	**3642-SD**
Date:	**6/2/2020**

Boring No. **I-4**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	44.00	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	40.00	inches
Total Test Hole Depth, D_T =	60.00	inches

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

$H_O = D_T - D_O$ =	20.00	inches
$H_F = D_T - D_F$ =	16.00	inches
$\Delta H = \Delta D = H_O - H_F$ =	4.00	inches
$Havg = (H_O+H_F)/2$ =	18.00	inches

I_t = 0.80 **Inches per Hour**

GEOTEK

Client:	**Mr. Rafid Hamika**
Project:	**3233 Old Highway 395**
Project No:	**3642-SD**
Date:	**6/2/2020**

Boring No. **I-5**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	43.50	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	40.00	inches
Total Test Hole Depth, D_T =	60.00	inches

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

$H_O = D_T - D_O$ =	20.00	inches
$H_F = D_T - D_F$ =	16.50	inches
$\Delta H = \Delta D = H_O - H_F$ =	3.50	inches
$H_{avg} = (H_O+H_F)/2$ =	18.25	inches

I_t = 0.69 **Inches per Hour**

GEOTEK

Client:	**Mr. Rafid Hamika**
Project:	**3233 Old Highway 395**
Project No:	**3642-SD**
Date:	**6/2/2020**

Boring No. **I-6**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	40.50	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	40.00	inches
Total Test Hole Depth, D_T =	60.00	inches

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

$H_O = D_T - D_O$ =	20.00	inches
$H_F = D_T - D_F$ =	19.50	inches
$\Delta H = \Delta D = H_O - H_F$ =	0.50	inches
$H_{avg} = (H_O+H_F)/2$ =	19.75	inches

I_t = 0.09 **Inches per Hour**

GEOTEK

Percolation Test Data Sheet

Project: 3233 Old Highway 395 Job No.: 3642-SD

Test Hole No.: I-1 Date Excavated: 4/30

Depth of Test Hole (ft): 8 Soil Description: SM

Percolation Test By: KRM Date: 5/1 Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Depth, from top			Percolation Rate (min/inch)
			Initial (inch)	Final (inch)	Δ (inch)	
9:04 9:29	25	25	20	19	1	
9:30 9:55	25	51	20	19	1	
9:56 10:26	30	82	20	19.25	0.75	
10:27 10:57	30	113	20	19.25	0.75	
10:58 11:28	30	144	20	19.25	0.75	
11:29 11:59	30	175	20	19.5	0.5	
12:00 12:30	30	206	20	19.5	0.5	
12:31 1:01	30	237	20	19.5	0.5	
1:02 1:32	30	268	20	19.5	0.5	
1:33 2:03	30	299	20	19.5	0.5	
2:04 2:34	30	330	20	19.5	0.5	
2:35 3:05	30	361	20	19.5	0.5	
3:06 3:36	30	392	20	19.5	0.5	
3:37 4:07	30	423	20	19.5	0.5	

Design Percolation Rate: min/inch

Percolation Test Data Sheet

Project: 3233 Old Highway 395 Job No.: 3642-SD

Test Hole No.: I-2 Date Excavated:

Depth of Test Hole (ft): 8 Soil Description: SM

Percolation Test By: KRM Date: Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Depth, from top Initial (inch)	Final (inch)	Δ (inch)	Percolation Rate (min/inch)
9:00 9:25	25	25	20	15	5	
9:26 9:51	25	51	20	15.5	4.5	
9:52 10:22	30	82	20	15.5	4.5	
10:23 10:53	30	113	20	15.75	4.25	
10:54 11:24	30	144	20	15.5	4.5	
11:25 11:55	30	175	20	15.5	4.5	
11:56 12:26	30	206	20	15.75	4.25	
12:27 12:57	30	237	20	15.75	4.25	
12:58 1:28	30	268	20	15.75	4.25	
1:29 1:59	30	299	20	16	4	
2:00 2:30	30	330	20	16	4	
2:31 3:01	30	361	20	16	4	
3:02 3:32	30	392	20	16	4	
3:33 4:03	30	423	20	16	4	

Design Percolation Rate: min/inch

Percolation Test Data Sheet

Project: 3233 Old Highway 395 Job No.: 3642-SD
Test Hole No.: I-3 Date Excavated: 4/30/20
Depth of Test Hole (ft): 8 Soil Description: SM
Percolation Test By: KRM Date: 5/1/20 Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Depth, from top Initial (inch)	Water Depth, from top Final (inch)	Water Depth, from top Δ (inch)	Percolation Rate (min/inch)
8:52 9:17	25	25	20	17	3	
9:18 9:43	25	51	20	18	2	
9:44 10:14	30	82	20	17.75	2.25	
10:15 10:45	30	113	20	18	2	
10:46 11:16	30	144	20	18	2	
11:17 11:47	30	175	20	18	2	
11:48 12:18	30	206	20	18	2	
12:19 12:49	30	237	20	18	2	
12:50 1:20	30	268	20	18.25	1.75	
1:21 1:51	30	299	20	18.25	1.75	
1:52 2:22	30	330	20	18.25	1.75	
2:23 2:53	30	361	20	18.25	1.75	
2:54 3:24	30	392	20	18.25	1.75	
3:25 3:55	30	423	20	18.25	1.75	

Design Percolation Rate: min/inch

Percolation Test Data Sheet

Project: 3233 Old Highway 395 | Job No.: 3642-SD

Test Hole No.: I-4 | Date Excavated: 4/30/20

Depth of Test Hole (ft): 8 | Soil Description: SM

Percolation Test By: KRM | Date: 5/1/20 | Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Depth, from top			Percolation Rate (min/inch)
			Initial (inch)	Final (inch)	Δ (inch)	
8:47 9:12	25	25	20	16	4	
9:13 9:38	25	51	20	17	3	
9:39 10:09	30	82	20	16.5	3.5	
10:10 10:40	30	113	20	16	4	
10:41 11:11	30	144	20	16	4	
11:12 11:42	30	175	20	16.25	3.75	
11:43 12:13	30	206	20	16.25	3.75	
12:14 12:44	30	237	20	16	4	
12:45 1:15	30	268	20	16	4	
1:16 1:46	30	299	20	16	4	
1:47 2:17	30	330	20	16	4	
2:18 2:48	30	361	20	16	4	
2:49 3:19	30	392	20	16	4	
3:20 3:50	30	423	20	16	4	

Design Percolation Rate: **min/inch**

Percolation Test Data Sheet

Project: 3233 Old Highway 395 Job No.: 3642-SD
Test Hole No.: I-5 Date Excavated: 4/30/20
Depth of Test Hole (ft): 8 Soil Description: SM
Percolation Test By: KRM Date: 5/7/20 Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Depth, from top Initial (inch)	Final (inch)	Δ (inch)	Percolation Rate (min/inch)
8:45 9:10	25	25	20	16	4	
9:11 9:36	25	51	20	16.5	3.5	
9:37 10:07	30	82	20	16	4	
10:08 10:38	30	113	20	16.5	3.5	
10:39 11:09	30	144	20	16.5	3.5	
11:10 11:40	30	175	20	16.5	3.5	
11:41 12:11	30	206	20	16.5	3.5	
12:12 12:42	30	237	20	16.5	3.5	
12:43 1:13	30	268	20	16.5	3.5	
1:14 1:44	30	299	20	16.5	3.5	
1:45 2:15	30	330	20	16.5	3.5	
2:16 2:46	30	361	20	16.5	3.5	
2:47 3:17	30	392	20	16.5	3.5	
3:18 3:48	30	423	20	16.5	3.5	

Design Percolation Rate: min/inch

Percolation Test Data Sheet

Project: 3233 Old Highway 395 Job No.: 3642-SD
Test Hole No.: I-6 Date Excavated:
Depth of Test Hole (ft): 8 Soil Description: SM
Percolation Test By: KRM Date: Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Depth, from top: Initial (inch)	Final (inch)	Δ (inch)	Percolation Rate (min/inch)
8:48 9:13	25	25	20	19.25	0.75	
9:14 9:39	25	51	20	19.25	0.75	
9:40 10:10	30	82	20	19.5	0.5	
10:11 10:41	30	113	20	19.25	0.75	
10:42 11:12	30	144	20	19.5	0.5	
11:13 11:43	30	175	20	19.5	0.5	
11:44 12:14	30	206	20	19.5	0.5	
12:15 12:45	30	237	20	19.5	0.5	
12:46 1:16	30	268	20	19.5	0.5	
1:17 1:47	30	299	20	19.5	0.5	
1:48 2:18	30	330	20	19.5	0.5	
2:19 2:49	30	361	20	19.5	0.5	
2:50 3:20	30	392	20	19.5	0.5	
3:21 3:51	30	423	20	19.5	0.5	

Design Percolation Rate: min/inch

APPENDIX D

GENERAL GRADING GUIDELINES

Proposed Commercial Development
Fallbrook, San Diego County, California
Project No. 3642-SD

GEOTEK

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.

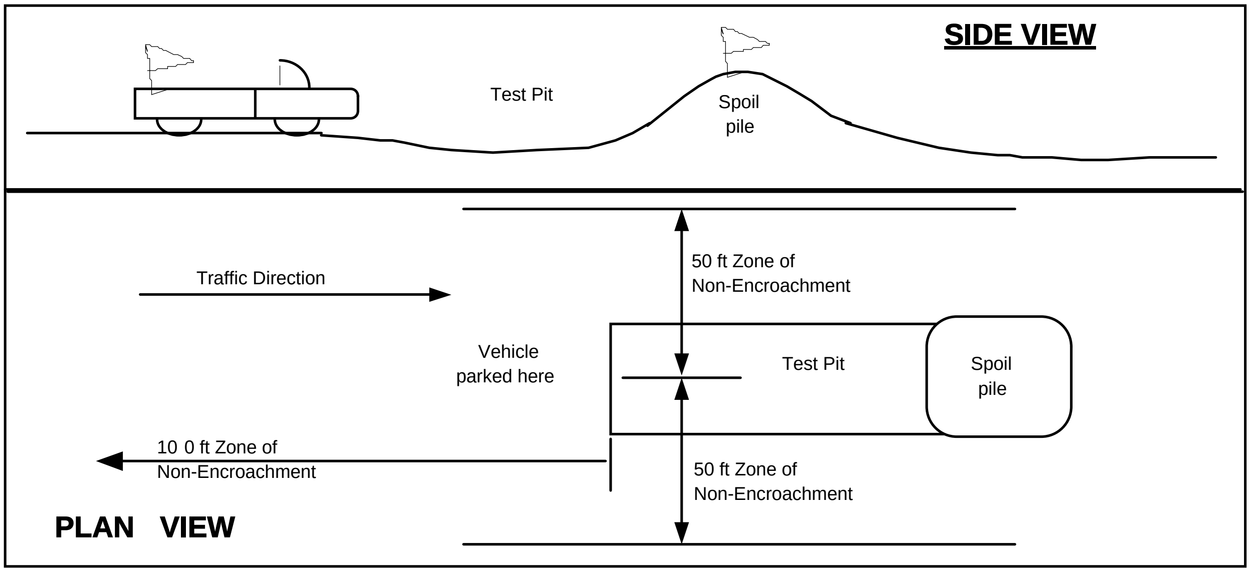
Test Pits Location, Orientation and Clearance

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.