

**GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED SINGLE-FAMILY RESIDENTIAL DEVELOPMENT
APN 0143-191-59
HIGHLAND AVENUE AND MEDICAL CENTER DRIVE
SAN BERNARDINO, SAN BERNARDINO COUNTY, CALIFORNIA**

PREPARED FOR

**WARMINGTON RESIDENTIAL
3090 PULLMAN STREET
COSTA MESA, CALIFORNIA 92626**

PREPARED BY

**GEOTEK, INC.
1548 NORTH MAPLE STREET
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PROJECT No. 2849-CR

AUGUST 12, 2021





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August 12, 2021
Project No. 2849-CR

Warmington Residential
3090 Pullman Street
Costa Mesa, California 92626

Attention: Mr. Bret Ilich

Subject: Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Development
APN 0143-191-59
Highland Avenue and Medical Center Drive
San Bernardino, San Bernardino County, California

Dear Mr. Ilich:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this geotechnical and infiltration evaluation for the proposed project located in San Bernardino, San Bernardino County, California. This report presents the results of GeoTek's evaluation, discussion of findings, and provides geotechnical recommendations for foundation design and construction.

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

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

Respectfully submitted,
GeoTek, Inc.



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Appendix B – Results of Laboratory Testing

Appendix C – Percolation Data & Porchet Calculations

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I. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical engineering and geologic conditions at the project site, as outlined in GeoTek's proposal P-0604321-CR, dated June 15, 2021. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- Site exploration consisting of the excavation, logging, and sampling of five (5) exploratory test borings extending to depths ranging from 16.5 to 51.5 feet below grade,
- Excavation of two (2) additional borings to a depth of about five (5) feet below grade and performing an infiltration test in each boring,
- Laboratory testing of soil samples collected during the field investigation,
- Review and evaluation of site seismicity, and
- Preparation of this geotechnical report which presents GeoTek's findings, conclusions, and recommendations for this site.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The approximate 10-acre "L"-shaped project site is located on the south side of West Highland Avenue and Medical Center Drive, approximately 500 feet west of Medical Center Drive, in the City of San Bernardino, San Bernardino County, California (See Figure 1). Access to the site is available from West Highland Avenue, a paved improved street located adjacent to the northern boundary of the site. A dirt trail (extension of North Gardena Street) extending to West Highland Drive is present along the northwestern boundary of the site. The site is bordered to the east by a mobile home park, to the south by single-family residences, and to the west by vacant land and a commercial development.

Topographically, the site slopes gently downward to the east at an approximate two (2) percent gradient. Elevation of the western portion of the the site is approximately 1,230 feet with approximately 10 feet of elevation differential across the site.

The site was vacant land at the time of the field exploration. The site was vegetated with a light covering of weeds and grasses with a line of domestic tress trending east-west in the central portion of the site.

2.2 PROJECT DESCRIPTION

Based upon review of the Conceptual Design Plan prepared by KTG Architecture and Planning, Inc. dated June 10, 2021 (see Figure 2), GeoTek understands the subject property is to be developed with about 72 single-family residential lots and associated infrastructure improvements. Stormwater disposal is to be by means of a stormwater detention basin.

The proposed residential structures are anticipated to be of wood-frame construction, one- to two-stories in height, and incorporate conventional shallow foundations and concrete slab-on-grade floors. It is our understanding that sewage disposal will be by a public sewer. For the purposes of this report, it is assumed maximum column and wall loads will be about 50 kips and 2.5 kips per foot, respectively. Specific site development plans were not provided as of the date of this report. 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.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

The field exploration for this report was conducted on July 26, 2021 and consisted of excavating five (5) geotechnical exploratory borings with a hollow-stem drill rig to depths ranging from about 16.5 to 51.5 feet below grade. The approximate locations of the GeoTek excavations are shown on the Boring Location Map (Figure 2). A geologist from GeoTek

logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory borings are included in Appendix A.

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

Percolation Testing

In addition to the geotechnical exploratory borings, two percolation test borings (I-1 and I-2) were excavated in the area of the proposed storm water management basin to depths of about 5 feet. Infiltration/percolation testing was conducted in these borings in general accordance with the requirements of the County of San Bernardino.

The percolation tests consisted of drilling an eight-inch diameter test hole to the desired depth and installing approximately two inches of gravel in the bottom of the hole. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within the boring. Water was then placed in the borings to presoak the holes and percolation testing was performed the following the pre-soak period. Following presoaking it was determined that “sandy soil” criteria was met within both percolation borings. The percolation tests were then performed which consisted of adding water to each test hole and measuring the water drop over a 10-minute period. The water drop was recorded for eight test intervals. Water was added to the test holes after each test interval. The field percolation rates were then converted to an infiltration rate using the Porchet Method.

The results of the conversions indicate infiltration rate range from about 8.49 to 8.74 inches per hour. Copies of the percolation data sheets and the Porchet infiltration rate conversion calculations are presented in Appendix C. No factors of safety were applied to the rates provided. Over the lifetime of the infiltration areas, the infiltration rates may be affected by sediment build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate in designing the infiltration system.

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

3.2 LABORATORY TESTING

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

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

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

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

More specific to the subject property, the site is located in an area geologically mapped to be underlain by alluvium (Dibblee, T.W. and Minch, J.A., 2004). No active faults are shown in the immediate site vicinity on the maps reviewed for the area.

4.2 GENERAL SOIL CONDITIONS

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

4.2.1 Fill

Fill was encountered in most of the exploratory borings to depths ranging from approximately 2.5 to 4.5 feet below existing grade. This (undocumented) fill is likely a result of the historical use of the site for agriculture purposes and the subsequent ground disturbance/tilling that has occurred after. The fill encountered in the exploratory borings generally consisted of silty fine sand and relatively clean sands (SM and SP soil types based upon the Unified Soil Classification System). Greater depths of fill may be present within unexplored areas of the site.

4.2.2 Younger Alluvium

Younger alluvial soils were encountered in the borings beneath the fill soils and extended to the maximum depths explored (51.5 feet). As encountered in the borings, the alluvium consisted of interbedded layers of silty sands and relatively clean sands with variable amounts of gravel (SM and SP soil types based upon the Unified Soil Classification System).

Based on the laboratory test results, the near surface soils have a “very low” expansion potential (ASTM D 4829). Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). The test results are provided in Appendix B.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

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

4.3.2 Groundwater

Groundwater was not encountered within any of the exploratory borings drilled at the site to the maximum depth drilled of 51.5 below the existing ground surface. Based on a review of groundwater depths noted on the State Department of Water Resources Water Data Library website, it is estimated the historic high groundwater depth is in excess of 50 feet below

existing grade at the site. Based on the results of the field exploration, review of site area geomorphology and geology, groundwater is not anticipated to adversely affect the proposed improvements.

4.4 FAULTING AND SEISMICITY

4.4.1 Faulting

The geologic structure of the entire California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. However, the site is not situated within a State of California designated “Alquist-Priolo” Earthquake Fault Zone. The nearest known active fault is the San Andreas fault located about 4 miles to the northeast.

4.4.2 Seismic Design Parameters

The site is located at approximately 34.1347 degrees West Latitude and -117.3256 degrees North Longitude. Site spectral accelerations (S_a and S_1) for 0.2 and 1.0 second periods for a Class “D” site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. Using the ASCE 7-16 option on the SEAOC/OSHPD website results in the values for S_{M1} and S_{D1} reported as “null-See Section 11.4.8” (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

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

The results, based on the 2015 NEHRP and the 2019 CBC, are presented in the following table assuming that the exception as allowed in ASCE 7-16 is applicable. If the exception is deemed not appropriate, a site-specific ground motion analysis will be required.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	2.298g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.919g
Site Coefficient for Site Class "D", F_a	1
Site Coefficient for Site Class "D", F_v	1.7
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	2.298g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	1.561g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.532g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	1.041g
Peak Ground Acceleration (PGA_M)	1.065g
Seismic Design Category	E

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

4.5 LIQUEFACTION

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

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

The project site is not located within an area mapped by the State of California for liquefaction potential. The County of San Bernardino Land Use Services (Geologic Hazard Maps) has designated the site as not having a potential for liquefaction. Based on the current mapping and the depth to groundwater, it is GeoTek's opinion that the liquefaction potential at the site is very low.

4.6 OTHER SEISMIC HAZARDS

An assessment of the potential “dry” settlement (i.e., settlement above the water table) resulting from seismic shaking of the site was evaluated. For this analysis we used a groundwater depth of 100 feet, a ground acceleration (PGAM) of 1.07g and a mean earthquake magnitude of 7.3. The ground acceleration and earthquake magnitude were obtained from the USGS websites. The computer software program LiquefyPro and the soil profiled from Boring B-1 were used in the analysis. Based on the recommendations provided in this report, engineered fill will be incorporated within the upper six feet of pad grade; this change has been incorporated into the settlement analysis. The results of this analysis indicate a potential ground surface settlement of about 2 inches is possible. A differential seismic settlement of about 1 inch over a 40 foot span is estimated. Based on these estimated magnitudes, ground modification or special foundation design is not deemed necessary. The results of the seismic dry settlement analysis are presented in Appendix E. However, the estimated seismic settlements should be considered in structural design.

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

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

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

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the County of San Bernardino, City of San Bernardino and 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 E.

5.2.2 Site Clearing

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

5.2.3 Site Preparation

Due to the non-uniform nature and thickness of the near-surface undocumented fill and loose condition of the upper younger alluvium, it is recommended that the soils be removed beneath the planned building footprint of the proposed structure to a depth of at least 5 feet below existing natural (below existing fill) grade, or three (3) feet beneath the base of the proposed foundations, whichever is greater. Removal bottoms should be relatively uniform in soil type which is not visibly porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures. A representative of this firm should observe and approve the bottom of all remedial excavations. The lateral extent of this recommended over-excavation should extend at least 5 feet beyond the building or foundation limits.

Following site clearing operations, over-excavation and lowering of site grades, where necessary, it is recommended that the exposed subgrade soils beneath all surface improvements be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the geotechnical engineering representative. The proof rolling equipment should possess a minimum weight of 15 tons and proof rolling should include at least 4 passes, two in each perpendicular direction. All soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative. Following proof rolling and removal of any unsuitable bearing soil, the exposed subgrade should be scarified to a depth of about 12 inches, be moisture conditioned to slightly above the soil's optimum moisture content and then be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM D-1557 test procedures.

5.2.4 Engineered Fill

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris, oversized materials (6 inch diameter or greater) and other deleterious material. All areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in 6-to-8-inch loose lifts, moisture conditioned to slightly above

the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D-1557 test procedures.

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

5.2.5 Transition Lot Condition

Building pads graded with a cut/fill transition should be undercut to reduce the potential for differential settlement. The cut portion of the cut/fill transition should be undercut to a depth of at least 3 feet or one (1) foot below the deepest proposed footing, whichever is deeper, and be backfilled with a properly compacted engineered fill. The bottom of the undercut should be sloped at a minimum of 1 percent toward the adjacent street/parking lot area.

5.2.6 Oversized Rock Disposal

Although unlikely, oversized cobbles, boulders and rock fragments may be encountered during rough grading and utility trench operations. If encountered, on-site disposal of oversized materials is possible, provided the oversized materials are placed as recommended on Plate 4 within Appendix E. Alternatively, over-sized materials can be exported from the site.

5.2.7 Excavation Characteristics

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

5.2.8 Trench Excavations and Backfill

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

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

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined by ASTM D-1557 test procedures). Under-slab trenches should also be

compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for road pavements should be compacted to at least 95 percent relative compaction. On-site materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than 6 inches are removed.

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

5.2.9 Shrinkage and Bulking

For planning purposes, a shrinkage loss of about 10 to 15 percent is anticipated for excavations within the undocumented fill/younger alluvium at the site. Several factors will impact earthwork balancing on the site, including shrinkage, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Shrinkage and bulking are primarily dependent upon the degree of compactive effort achieved during construction, depth of fill and underlying site conditions.

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

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

5.2.10 Grading Plan Review

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

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

The soils are classified as having a “very low” expansion potential in accordance with ASTM D 4829. GeoTek understands that post-tensioned foundations may be used for this site. Since the CBC indicates Post Tensioning Institute (PTI) design methodology is intended for expansive soils conditions, which do not apply to this project, no e_m or γ_m parameters as used in the PTI methodology are provided. The foundation elements for the proposed structures should bear entirely in engineered fill soils and should be designed in accordance with the *2019 California Building Code (CBC)*. The following design recommendations for post-tensioned foundations for this project are provided:

MINIMUM DESIGN REQUIREMENTS FOR POST-TENSIONED FOUNDATIONS	
Design Parameter	“Very Low” Expansion Potential 0 ≤ EI ≤ 20
Foundation Depth or Minimum Perimeter Beam Depth (inches below the lowest adjacent grade)	One- and Two-Stories – 12
Minimum Foundation Width (inches)*	One- to Two-Stories – 12
Minimum Slab Thickness (inches)	4 – Actual
Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)	Minimum of 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

*Greater depths and widths may be required per the structural design. Interior footing depths should be at least 12 inches below interior finished grade for 1-2 story buildings. Interior pad footings should possess a minimum width of 24 inches.

An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of building wall footings. This value may be increased by 400 psf for each additional 12 inches of embedment depth and by 200 psf for each additional 12 inches in width to a maximum of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering short-term wind and/or seismic loads.

Based upon review, a modulus of subgrade reaction (E_1) of 250 pci may be used in the design of the post-tensioned slab foundation. It should be noted that this value is based upon standard one foot plate load tests. Depending upon the design methodology and foundation geometry this value may need to be modified by the following:

$$E_s = E_1 \left(\frac{B+1}{2B} \right)^2$$

where: E_s = design modulus
 B = footing width

Based on the expansion index testing performed for this report and visual examination of the site soils, site soils possess a “very low” (0-20) expansion potential (ASTM D4829). Therefore, it is GeoTek’s opinion that conventional foundations supported by engineered fill may be used for this site. Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer. A summary of GeoTek’s preliminary foundation design recommendations for conventional foundations is presented in the table below:



Design Parameter	“Very Low” Expansion Potential ($0 \leq EI \leq 20$)
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12 - One- and -two Stories
Minimum Foundation Width (Inches)*	12
Minimum Slab Thickness (actual)	4 inches
Minimum Slab Reinforcing	6" x 6" – W2.9/W2.9 welded wire fabric placed in middle of slab or No. 3 bars at 18-inch centers.
Minimum Footing Reinforcement	Two No. 4 Reinforcing Bars, one top and one bottom
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 100% to a depth of 12 inches prior to placement of concrete

*Code minimums per Table 1809.7 of the 2019 CBC.

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

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

- 5.3.1.1 An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This allowable soil bearing capacity may be increased by 300 psf for each additional foot of footing depth and 300 psf for each additional foot of footing width to a maximum value of 4,000 psf. An increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).
- 5.3.1.2 Structural foundations should be designed in accordance with the 2019 CBC, and to withstand a total static settlement of 1 inch and maximum differential static settlement of one-half of the total settlement over a horizontal distance of 40 feet.
- 5.3.1.3 The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill or competent native soil. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements.

- 5.3.1.4 A grade beam, a minimum of 12 inches wide and 12 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.
- 5.3.1.5 A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2019 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

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

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

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

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

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

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

5.3.2 Miscellaneous Foundation Recommendations

- 5.3.2.1 To reduce moisture penetration beneath the slab on grade areas, utility trench excavations should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.
- 5.3.2.2 Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

5.3.3 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2019 CBC or City of San Bernardino requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The top outside edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least five feet and need not exceed 40 feet.

5.3.4 Soil Corrosivity

The soil resistivity at this site was tested in the laboratory on a sample collected during the field investigation. The results of the testing indicate that the on-site soils are considered “mildly corrosive” (14,070 ohm-cm) (Roberge, 2000) to buried ferrous metal in accordance with current standards used by corrosion engineers. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer. Additional corrosion testing should be performed at the time of site grading to assess the corrosion of potential of the as-graded soils.

5.3.5 Soil Sulfate Content

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

5.4 RETAINING AND GARDEN WALL DESIGN AND CONSTRUCTION

5.4.1.1 General Design Criteria

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

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

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

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

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

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

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

5.4.1.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (horizontal:vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	35
2:1	60

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

For walls with a retained height greater than 6 feet, an incremental seismic pressure should be included into the wall design. Where needed, it is recommended that an equivalent fluid



pressure of 16 pcf be included into the wall design to account for seismic loading conditions. This pressure may be applied as an inverted triangular distribution.

5.4.1.3 Retaining Wall Backfill and Drainage

The wall backfill should also include a minimum one (1) foot wide section of $\frac{3}{4}$ - to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The upper 24 inches should consist of compacted on-site materials. The rock should be separated from the earth with filter fabric. The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. The backfill materials should be placed in lifts no greater than eight (8) inches in thickness and compacted to a minimum of 90% relative compaction as determined by ASTM D 1557 test procedures. Proper surface drainage needs to be provided and maintained.

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

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

5.4.1.4 Restrained Retaining Walls

Retaining walls that will be restrained at the top that support level backfill or that have reentrant or male corners, should be designed for an equivalent at-rest fluid pressure of 55 pcf, plus any applicable surcharge loading. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.4.1.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

Although planned final grades beneath the street improvements within the site are not yet known, the following preliminary pavement design recommendations are based on Traffic Indexes of 5.5 for interior streets and 10.0 for Highland Avenue as designated by the City of San Bernardino. Preliminary pavement thickness design is based on the CalTrans Highway Design Manual (2018). An R-value of 50 has been assumed for the preliminary design of the project pavement sections. Once the traffic loading information becomes more defined, revision to the pavement design recommendations may be warranted. It is recommended that the final pavement design be based on R-value testing of the as-graded subgrade soils within the pavement areas.

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

PRELIMINARY MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.5 (Interior Streets)	3.0	4
10.0 (Highland Avenue)	4.0 5.0	11 9.5

Traffic Indices (TIs) used in the pavement design should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure. Traffic parameters used for design were

selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

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

5.6 CONCRETE CONSTRUCTION

5.6.1 General

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

5.6.2 Concrete Mix Design

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

5.6.3 Concrete Flatwork

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

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

5.6.4 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are hairline to more than 1/8 inch in width. Most cracks in concrete while unsightly do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete undergoes chemical

processes that are dependent on a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

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

5.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

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

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

6. INTENT

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

The scope of GeoTek's evaluation is limited to the area explored that is shown on the Boring Location Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to GeoTek by the client. Further, no evaluation of any existing site improvements is included. The scope is based on GeoTek's understanding of the project and the client's needs, GeoTek's proposal (Proposal No. P-0604321-CR) dated June 15, 2021 and geotechnical engineering standards normally used on similar projects in this region.

7. LIMITATIONS

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

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

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

8. SELECTED REFERENCES

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**APPROXIMATE
SITE AREA**

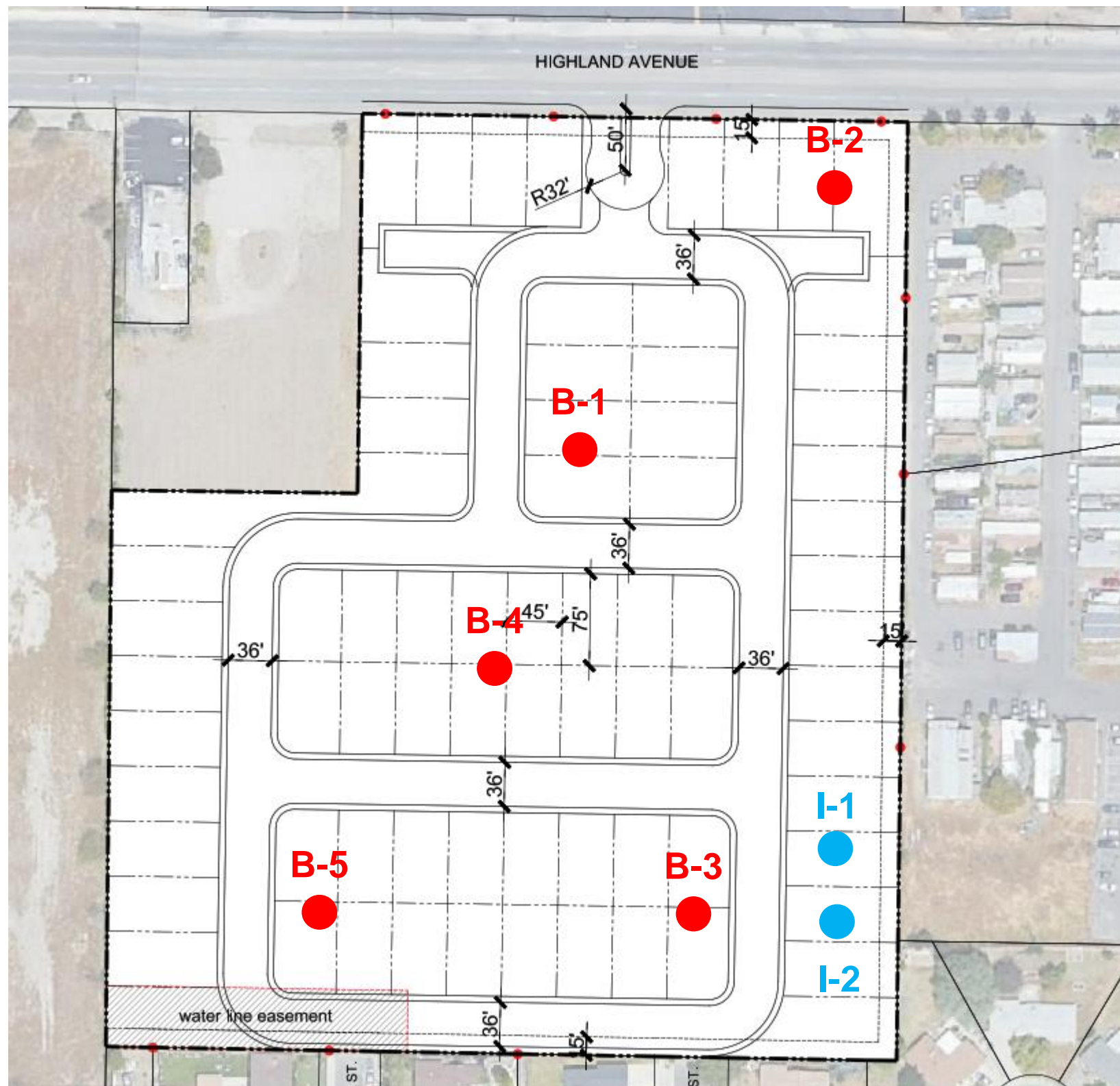
Warmington Residential
 Highland and Medical Center Drive
 APN 0143-191-59
 San Bernardino, San Bernardino County,
 California

Project No. 2849-CR



Figure 1
Site Location
Map





LEGEND

B-5 Approximate Location of Boring

I-2 Approximate Location of Infiltration Boring



Warmington Residential
 APN 0143-191-59
 San Bernardino, San Bernardino County,
 California

GeoTek Project No. 2842-CR



Figure 2
Boring Location Map

APPENDIX A

LOG OF EXPLORATORY BORINGS

**Proposed Single-Family Residential Development
Highland Avenue and Medical Center Drive
San Bernardino, San Bernardino County, California
Project No. 2849-CR**



A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)

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

Bulk Samples (Large)

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

Bulk Samples (Small)

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

B - BORING LOG LEGEND

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

SOILS

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

GEOLOGIC

B: Attitudes Bedding: strike/dip

J: Attitudes Joint: strike/dip

C: Contact line

.....	Dashed line denotes USCS material change
_____	Solid Line denotes unit / formational change
————	Thick solid line denotes end of boring

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING



CLIENT: Warmington Residential
PROJECT NAME: APN 0143-191-59
PROJECT NO.: 2849-CR
LOCATION: San Bernardino, CA

DRILLER: 2R Drilling
DRILL METHOD: HSA
HAMMER: 140 lbs - 30 in

LOGGED BY: JD
OPERATOR: Nick
RIG TYPE: CME 75
DATE: 7/26/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: B-1	Laboratory Testing		
	Sample Type	Blows/6 in.	Sample Number			Water Content (%)	Dry Density (pcf)	Others
MATERIAL DESCRIPTION AND COMMENTS								
					Undocumented Fill:			
4		4	R1	SM	Silty f-c SAND, light brown, slightly moist, loose	2.1	101.8	Maximum Density = 118.5 pcf Optimum Moisture = 10.5% Expansion Index = 0 Direct Shear Test
4		4						
4		4						
5		4	R2	SP	Alluvium: F-c SAND, brown, moist, medium dense			
5		5						
5		7						
5		4	R3		same as above	1.7	104.8	
5		10						
5		15						
10		16	R4		same as above, dense	1.3		
10		26						
10		30						
10		20	R5		same as above, some gravel	0.5	112.8	
10		22						
10		24						
15		16	R6	SM	Silty f-c SAND, brown, moist, medium dense			
15		19						
15		21						
20		20	R7		same as above, dense	2.8	113.3	
20		27						
20		29						
25		14	S1		same as above, medium dense			
25		18						
25		19						
30		18	S2	SM/SP	Silty f-c SAND to f-c SAND, light brown, slightly moist, dense	0.8		
30		24						
30		30						

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING



CLIENT: Warmington Residential
PROJECT NAME: APN 0143-191-59
PROJECT NO.: 2849-CR
LOCATION: San Bernardino, CA

DRILLER: 2R Drilling
DRILL METHOD: HSA
HAMMER: 140 lbs - 30 in

LOGGED BY: JD
OPERATOR: Nick
RIG TYPE: CME 75
DATE: 7/26/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: B-1 (cont.)	Laboratory Testing		
	Sample Type	Blows/6 in.	Sample Number			Water Content (%)	Dry Density (pcf)	Others
MATERIAL DESCRIPTION AND COMMENTS								
Alluvium (cont.):								
35		11 14 28	S3	SM	Silty f-c SAND, trace gravel, grey, moist, dense			
40		19 20 24	S4		same as above, dark grey	1.9		
45		10 27 27	S5		same as above, very dense	4.6		
50		16 20 28	S6	SM/SW	Silty f SAND to f SAND, brown, moist, dense			
BORING TERMINATED AT 51.5 FEET								
55					No groundwater encountered Boring backfilled with soil cuttings			
60								

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING



CLIENT: Warmington Residential
PROJECT NAME: APN 0143-191-59
PROJECT NO.: 2849-CR
LOCATION: San Bernardino, CA

DRILLER: 2R Drilling
DRILL METHOD: HSA
HAMMER: 140 lbs - 30 in

LOGGED BY: JD
OPERATOR: Nick
RIG TYPE: CME 75
DATE: 7/26/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: B-3	Laboratory Testing		
	Sample Type	Blows/6 in.	Sample Number			Water Content (%)	Dry Density (pcf)	Others
MATERIAL DESCRIPTION AND COMMENTS								
					Undocumented Fill:			
		4	R1	SM	Silty f-c SAND, brown, slightly moist, loose	0.9		
		5						
		4						
		5	R2		same as above, medium dense to dense			
		10						
		19						
5		9	R3	SM	Alluvium: Silty f-c SAND, trace gravel, grey, slightly moist, medium dense	1.4		
		12						
		9						
		7	R4	SP	F-c SAND, brown, moist, medium dense	1.4		
		8						
		11						
		6	R5	SM	Silty f-c SAND, brown, moist, medium dense	5.5	107.1	Collapse Test
		8						
		9						
10								
		6	R6	SM/SP	Silty f-c SAND to f-c SAND, brown, moist, medium dense	5.4	109.6	
		9						
		17						
15								
					BORING TERMINATED AT 16.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
20								
25								
30								

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING



CLIENT: Warmington Residential
PROJECT NAME: APN 0143-191-59
PROJECT NO.: 2849-CR
LOCATION: San Bernardino, CA

DRILLER: 2R Drilling
DRILL METHOD: HSA
HAMMER: 140 lbs - 30 in

LOGGED BY: JD
OPERATOR: Nick
RIG TYPE: CME 75
DATE: 7/26/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: B-4 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/6 in.	Sample Number			Water Content (%)	Dry Density (pcf)	Others
5					Undocumented Fill:			
4		4	R1	SP	F-c SAND, light brown, slightly moist, loose	0.8		
5		5						
6		6						
10		5	R2	SP	Alluvium: F-c SAND, some gravel, light brown, moist, medium dense			
10		10						
12		12						
15		5	R3		same as above	1.4		
15		7						
15		7						
15		7						
20		7	R4	SM/SP	Silty f-c SAND to f-c SAND, brown, moist, medium dense	4.8	114.1	
20		8						
20		8						
20		8						
25		5	R5	SM	Silty f-c SAND, brown, moist, medium dense			
25		6						
25		6						
25		6						
30		4	R6		same as above	4.7	108.4	
30		7						
30		10						
BORING TERMINATED AT 16.5 FEET								
No groundwater encountered Boring backfilled with soil cuttings								

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING



CLIENT: Warmington Residential
PROJECT NAME: APN 0143-191-59
PROJECT NO.: 2849-CR
LOCATION: San Bernardino, CA

DRILLER: 2R Drilling
DRILL METHOD: HSA
HAMMER: 140 lbs - 30 in

LOGGED BY: JD
OPERATOR: Nick
RIG TYPE: CME 75
DATE: 7/26/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: I-1 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/6 in.	Sample Number			Water Content (%)	Dry Density (pcf)	Others
5				SM	<p>Undocumented Fill:</p> <p>Silty f-c SAND, brown, slightly moist</p>			
10					<p>BORING TERMINATED AT 5 FEET</p> <p>No groundwater encountered Boring set with pipe, filter sock, and gravel</p>			
15								
20								
25								
30								

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING



CLIENT: Warmington Residential
PROJECT NAME: APN 0143-191-59
PROJECT NO.: 2849-CR
LOCATION: San Bernardino, CA

DRILLER: 2R Drilling
DRILL METHOD: HSA
HAMMER: 140 lbs - 30 in

LOGGED BY: JD
OPERATOR: Nick
RIG TYPE: CME 75
DATE: 7/26/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: I-2 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/6 in.	Sample Number			Water Content (%)	Dry Density (pcf)	Others
5				SM	<p><u>Undocumented Fill:</u></p> <p>Silty f-c SAND, brown, slightly moist</p>			
10					<p>BORING TERMINATED AT 5 FEET</p> <p>No groundwater encountered Boring set with pipe, filter sock, and gravel</p>			
15								
20								
25								
30								

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

APPENDIX B

RESULTS OF LABORATORY TESTING

**Proposed Single-Family Residential Development
Highland Avenue and Medical Center Drive
San Bernardino, San Bernardino County, California
Project No. 2849-CR**



SUMMARY OF LABORATORY TESTING

Classification

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

Collapse Test

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

Direct Shear

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

Expansion Index

Expansion Index testing was performed on one soil samples. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below.

Boring No.	Depth (ft.)	Description	Expansion Index	Classification
B-5	0-5	Silty Sand	0	Very Low

In-Situ Moisture and Density

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

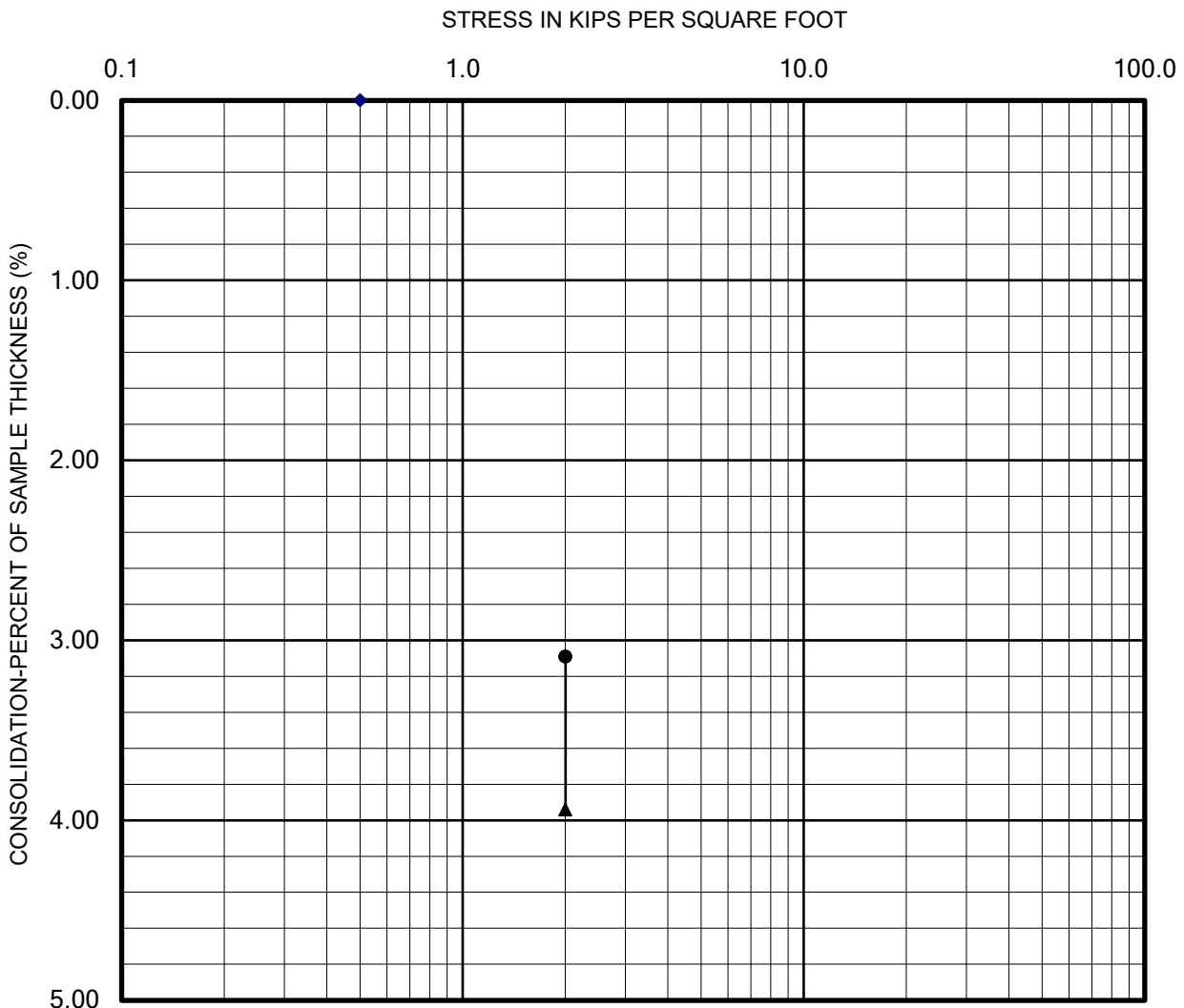
Moisture-Density Relationship

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

Sulfate Content, Resistivity and Chloride Content

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

Boring #	Depth (ft.)	pH ASTM D4972	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-5	0-5	8.4	6.0	0.0021	14,070



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

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



COLLAPSE REPORT

Sample: B-2 @ 4 feet

**Medical Center Drive
San Bernardino, California**

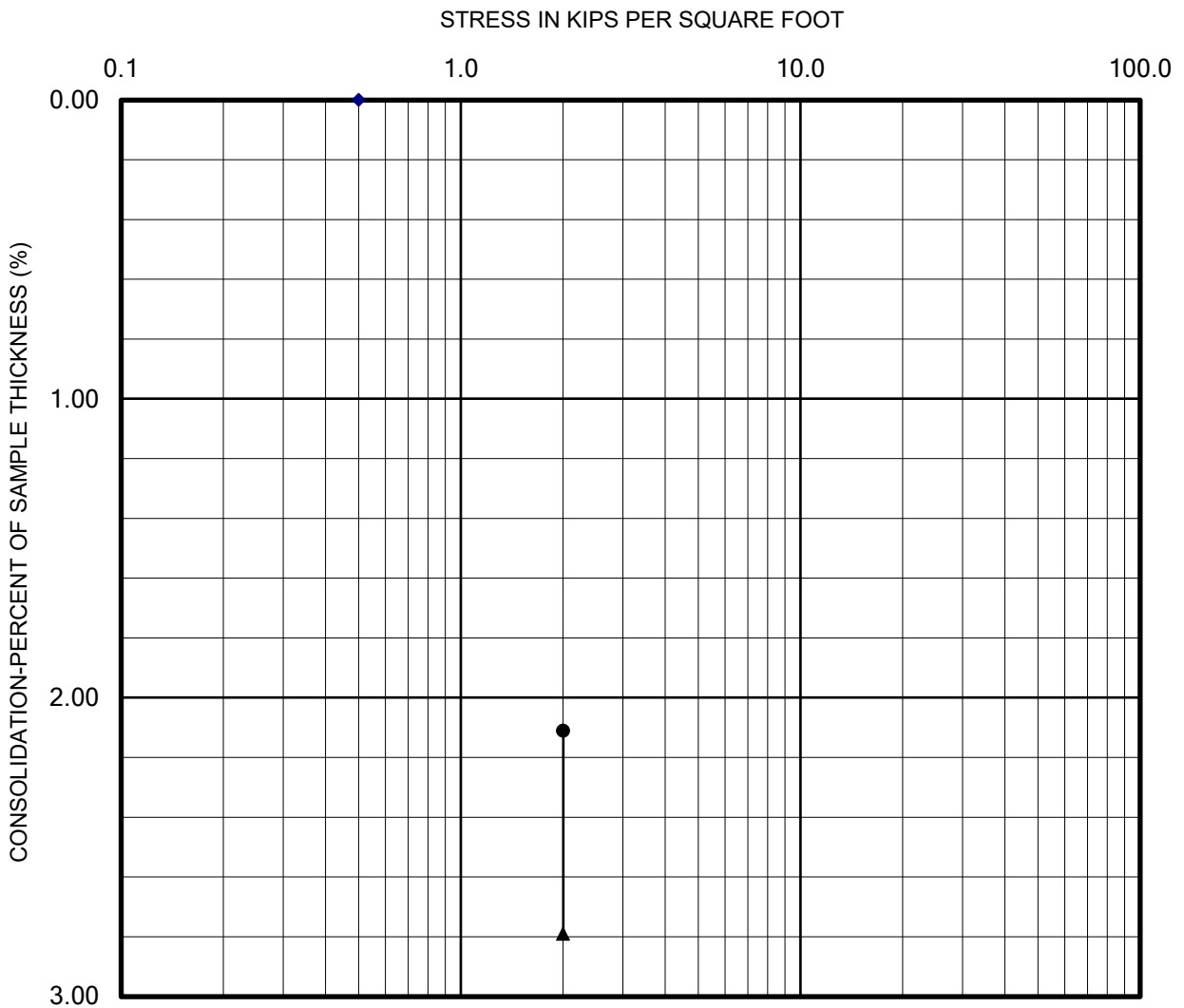
Plate B-1

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2849-CR

Date: 8-4-21



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

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



COLLAPSE REPORT

Sample: B-3 @ 9 feet

**Medical Center Drive
San Bernardino, California**

Plate B-2

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2849-CR

Date: 8-4-21

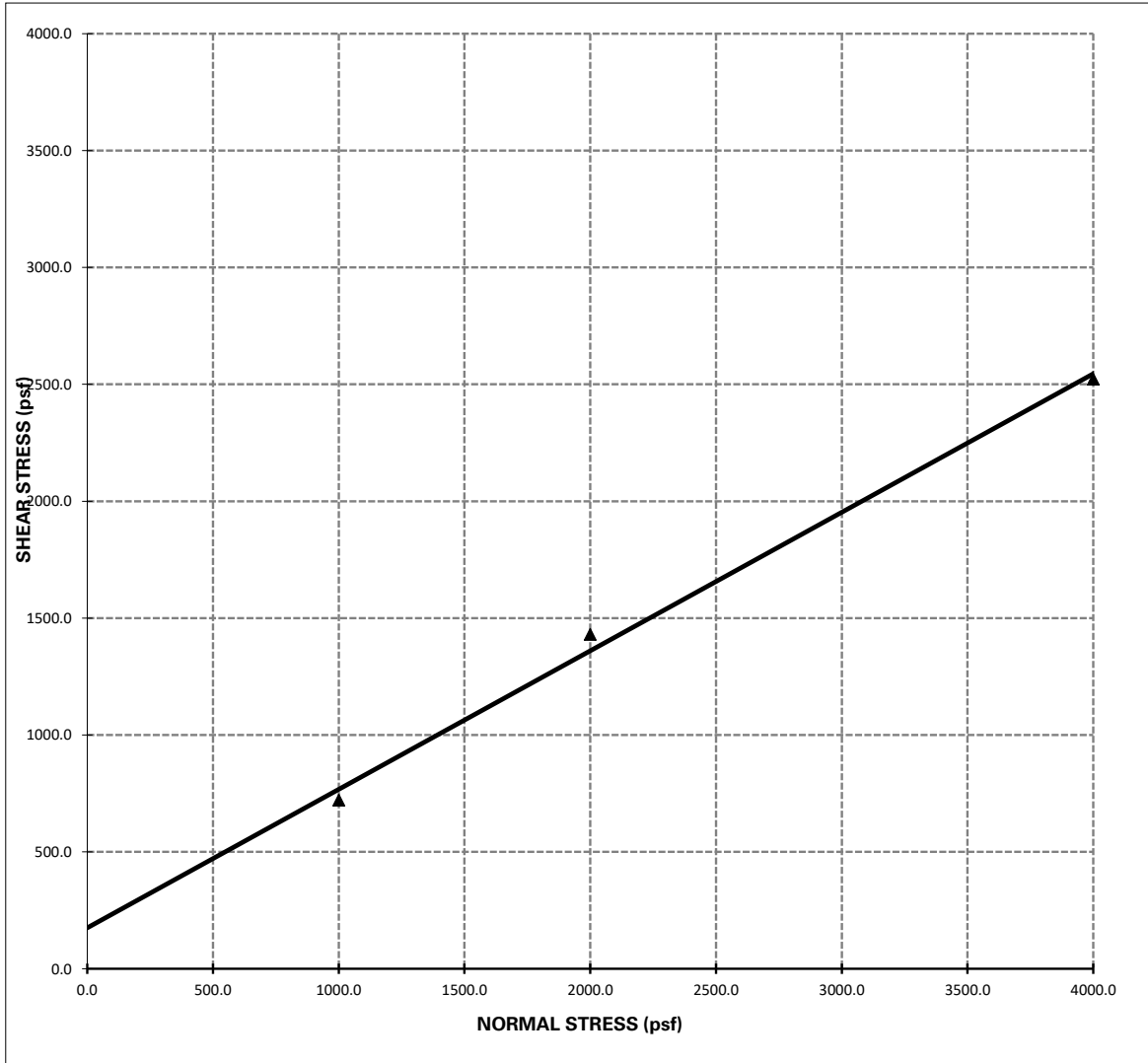


DIRECT SHEAR TEST

Project Name: Warmington Residential
Project Number: 2849-CR

Sample Location: B1 @ 0-5'
Date Tested: 8/10/2021

PEAK VALUE



Shear Strength: $\Phi = 31^\circ$; **C = 175 psf**

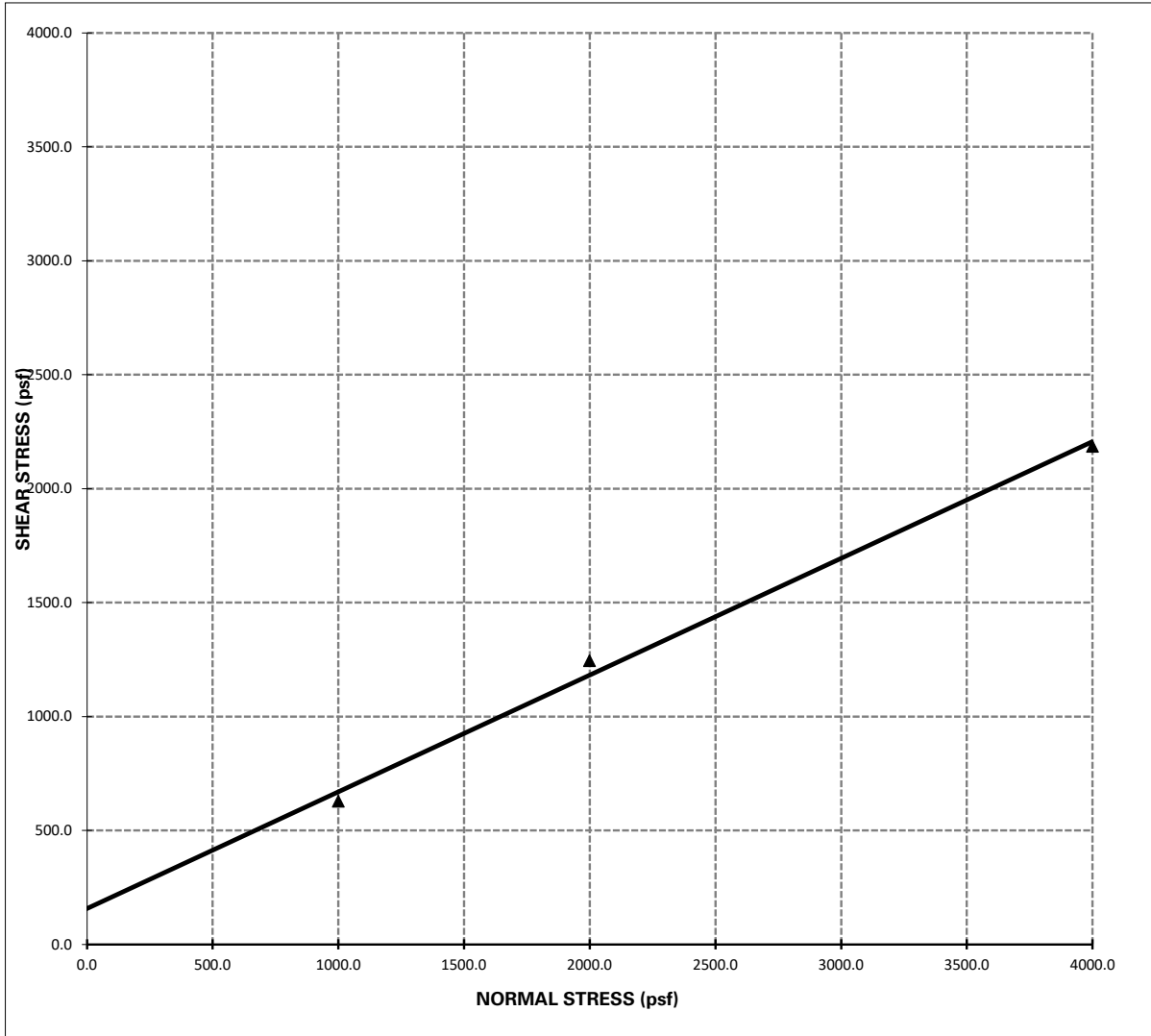
- Notes:**
- 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.035 in/min.



DIRECT SHEAR TEST

Project Name: Warmington Residential
Project Number: 2849-CR

Sample Location: BI @ 0-5'
Date Tested: 8/10/2021



Shear Strength: $\Phi = 27^{\circ}$; **C = 158 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: Warrington Residential
Project Number: 2849-CR
Project Location: SWC Highland Ave. & Medical Ctr Dr. San Bernardino

Tested/Checked By: CD Lab No Corona
Date Tested: 8/6/2021
Sample Source: B1 @ 0-5'
Sample Description:

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	747.1
B	Weight of ring (gm)	363.0
C	Net weight of sample (gm)	384.1
D	Wet Density, lb / ft ³ (C*0.3016)	115.8
E	Dry Density, lb / ft ³ (D/1.F)	104.4

SATURATION DETERMINATION

F	Moisture Content, %	11.0
G	Specific Gravity, assumed	2.70
H	Unit Wt. of Water @ 20°C, (pcf)	62.4
I	% Saturation	48.3

READINGS		
DATE	TIME	READING
8/6/2021		0.1340
8/6/2021		0.1330
8/7/2021		0.1330

Initial
10 min/Dry

Final

FINAL MOISTURE	
Final Weight of wet sample & tare	
770.2	% Moisture
	17.0

EXPANSION INDEX = 0



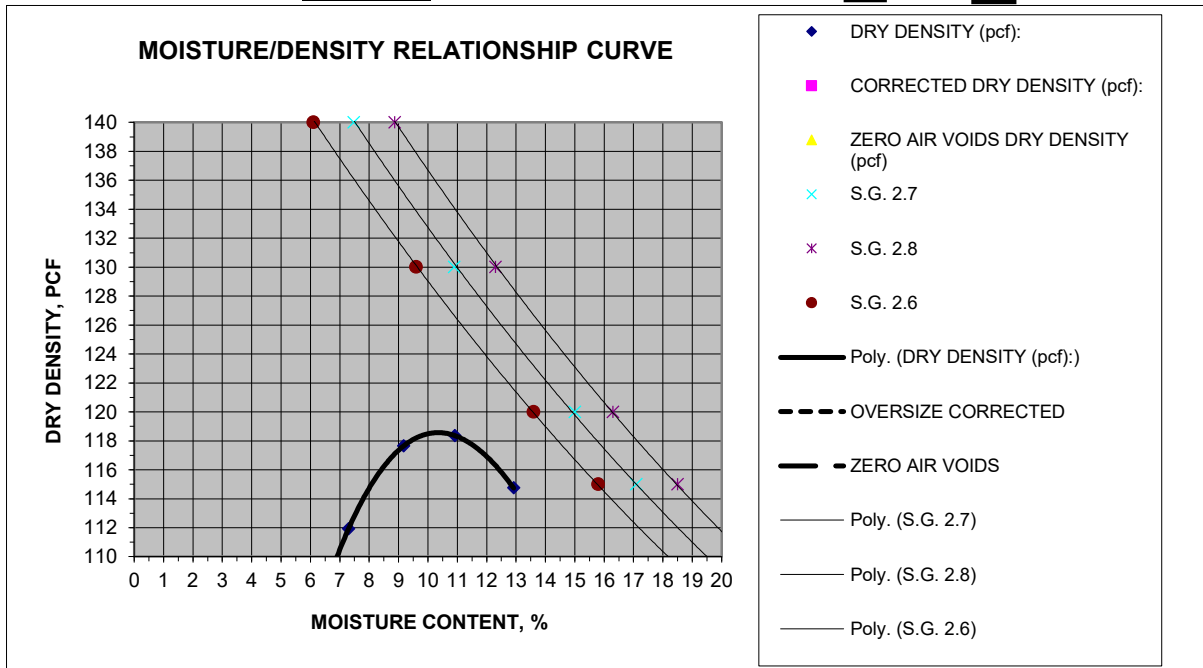
MOISTURE/DENSITY RELATIONSHIP

Client: Warmington Residential
Project: APN 0143-191-59
Location: San Bernardino
Material Type: Sand Trace Silt/ F-M Sand
Material Supplier: -
Material Source: -
Sample Location: B1 @ 0-5'
 -
Sampled By: 0
Received By: RJ
Tested By: RL
Reviewed By: RJ

Job No.: 2849-CR
Lab No.: Corona

Date Sampled: 7/27/2021
Date Received: 7/27/2021
Date Tested: 8/9/2021
Date Reviewed: 8/10/2021

Test Procedure: ASTM D1557 **Method:** A
Oversized Material (%): 0.7 **Correction Required:** yes no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf **@ Optimum Moisture, %**
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: _____



Results Only Soil Testing for SWC Highland Ave Medical Center Or, San Bernardino

August 2, 2021

Prepared for:
Anna Scott
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92280
ascott@geotekusa.com

Project X Job#: S210729D
Client Job or PO#: 2849-CR Warmington Residential

Respectfully Submitted,

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





Soil Analysis Lab Results

Client: GeoTek, Inc.

Job Name: SWC Highland Ave Medical Center Or, San Bernardino

Client Job Number: 2849-CR Warmington Residential

Project X Job Number: S210729D

August 2, 2021

Method	ASTM D4327	ASTM D4327	ASTM G187	ASTM D4972	ASTM G200	ASTM D4658	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Sulfates	Chlorides	Resistivity	pH	Redox	Sulfide	Nitrate	Ammonium	Lithium	Sodium	Potassium	Magnesium	Calcium	Fluoride	Phosphate
	SO ₄ ²⁻	Cl ⁻	As Rec'd Minimum (Ohm-cm)		(mV)	S ²⁻	NO ₃ ⁻	NH ₄ ⁺	Li ⁺	Na ⁺	K ⁺	Mg ²⁺	Ca ²⁺	F ⁻	PO ₄ ³⁻
2849-CR B5 @	(mg/kg)	(mg/kg)	(Ohm-cm)			(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
	20.6	6.0	241,200	8.4	132	<0.01	0.2	3.0	0.03	16.3	4.9	33.7	210.3	3.1	2.7

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography

mg/kg = milligrams per kilogram (parts per million) of dry soil weight

ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown

Chemical Analysis performed on 1:3 Soil-To-Water extract

PPM = mg/kg (soil) = mg/L (Liquid)

APPENDIX C

PERCOLATION DATA SHEETS & PORCHET CALCULATIONS

**Proposed Single-Family Residential Development
Highland Avenue and Medical Center Drive
San Bernardino, San Bernardino County, California
Project No. 2849-CR**



Client: Warmington Residential
Project: APN 0143-191-59
Project No: 2849-CR
Date: 7/27/2021

Boring No. I-1

Infiltration Rate (Porchet Method)

Time Interval, $\Delta t =$ 10
 Final Depth to Water, $D_F =$ 51.75
 Test Hole Radius, $r =$ 4
 Initial Depth to Water, $D_O =$ 40
 Total Test Hole Depth, $D_T =$ 60

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

$H_O = D_T - D_O =$ 20
 $H_F = D_T - D_F =$ 8.25
 $\Delta H = \Delta D = H_O - H_F =$ 11.75
 $H_{avg} = (H_O + H_F) / 2 =$ 14.125

$I_t =$ 8.74 Inches per



Client: Warmington Residential
Project: APN 0143-191-59
Project No: 2849-CR
Date: 7/27/2021

Boring No. I-2

Infiltration Rate (Porchet Method)

Time Interval, $\Delta t =$ 10
 Final Depth to Water, $D_F =$ 51.5
 Test Hole Radius, $r =$ 4
 Initial Depth to Water, $D_O =$ 40
 Total Test Hole Depth, $D_T =$ 60

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

$H_O = D_T - D_O =$ 20
 $H_F = D_T - D_F =$ 8.5
 $\Delta H = \Delta D = H_O - H_F =$ 11.5
 $H_{avg} = (H_O + H_F) / 2 =$ 14.25

$I_t =$ 8.49 Inches per



APPENDIX D

LIQUEFACTION ANALYSIS

**Proposed Single-Family Residential Development
Highland Avenue and Medical Center Drive
San Bernardino, San Bernardino County, California
Project No. 2849-CR**



LIQUEFACTION ANALYSIS CALCULATION DETAILS

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Input File Name: UNTITLED
Title: APN 0143-191-59
Subtitle: Seismic Settlement

Input Data:

Surface Elev.=1235
Hole No.=B-
Depth of Hole=50.00 ft
Water Table during Earthquake= 100.00 ft
Water Table during In-Situ Testing= 100.00 ft
Max. Acceleration=1.07 g
Earthquake Magnitude=7.30
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1
Plot one CSR curve (fs1=User)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
1.00	50.00	115.00	20.00
3.00	50.00	115.00	20.00
5.00	50.00	115.00	20.00
7.50	35.00	106.00	10.00
10.00	29.00	113.00	10.00
15.00	25.00	113.00	20.00
20.00	35.00	116.00	20.00

0.67	15.00	113.00	0.799	113.00	0.799	0.97	0.000	1.070	0.67	1.00
0.67	16.00	113.60	0.852	113.60	0.852	0.96	0.000	1.070	0.67	1.00
0.67	17.00	114.20	0.906	114.20	0.906	0.96	0.000	1.070	0.67	1.00
0.67	18.00	114.80	0.960	114.80	0.960	0.96	0.000	1.070	0.67	1.00
0.66	19.00	115.40	1.014	115.40	1.014	0.96	0.000	1.070	0.66	1.00
0.66	20.00	116.00	1.069	116.00	1.069	0.95	0.000	1.070	0.66	1.00
0.66	21.00	116.00	1.124	116.00	1.124	0.95	0.000	1.070	0.66	1.00
0.66	22.00	116.00	1.179	116.00	1.179	0.95	0.000	1.070	0.66	1.00
0.66	23.00	116.00	1.234	116.00	1.234	0.95	0.000	1.070	0.66	1.00
0.66	24.00	116.00	1.288	116.00	1.288	0.94	0.000	1.070	0.66	1.00
0.65	25.00	116.00	1.343	116.00	1.343	0.94	0.000	1.070	0.65	1.00
0.65	26.00	116.00	1.398	116.00	1.398	0.94	0.000	1.070	0.65	1.00
0.65	27.00	116.00	1.453	116.00	1.453	0.94	0.000	1.070	0.65	1.00
0.65	28.00	116.00	1.508	116.00	1.508	0.93	0.000	1.070	0.65	1.00
0.65	29.00	116.00	1.562	116.00	1.562	0.93	0.000	1.070	0.65	1.00
0.65	30.00	116.00	1.617	116.00	1.617	0.93	0.000	1.070	0.65	1.00
0.64	31.00	116.00	1.672	116.00	1.672	0.92	0.000	1.070	0.64	1.00
0.64	32.00	116.00	1.727	116.00	1.727	0.91	0.000	1.070	0.64	1.00
0.63	33.00	116.00	1.782	116.00	1.782	0.91	0.000	1.070	0.63	1.00
0.62	34.00	116.00	1.837	116.00	1.837	0.90	0.000	1.070	0.62	1.00
0.62	35.00	116.00	1.891	116.00	1.891	0.89	0.000	1.070	0.62	1.00
0.61	36.00	116.00	1.946	116.00	1.946	0.88	0.000	1.070	0.61	1.00
0.61	37.00	116.00	2.001	116.00	2.001	0.87	0.000	1.070	0.61	1.00
0.60	38.00	116.00	2.056	116.00	2.056	0.86	0.000	1.070	0.60	1.00
0.60	39.00	116.00	2.111	116.00	2.111	0.86	0.000	1.070	0.60	1.00

50.12	9.00 0.50	31.40	1.25	0.85	0.479	1.44	48.21	10.00	1.91
44.04	10.00 0.50	29.00	1.25	0.85	0.532	1.37	42.26	10.00	1.78
41.96	11.00 0.50	28.20	1.25	0.85	0.585	1.31	39.17	12.00	2.79
40.18	12.00 0.50	27.40	1.25	0.85	0.638	1.25	36.44	14.00	3.75
38.58	13.00 0.50	26.60	1.25	0.85	0.692	1.20	33.98	16.00	4.60
37.10	14.00 0.50	25.80	1.25	0.85	0.745	1.16	31.75	18.00	5.34
39.47	15.00 0.50	25.00	1.25	0.95	0.799	1.12	33.22	20.00	6.25
41.11	16.00 0.50	27.00	1.25	0.95	0.852	1.08	34.73	20.00	6.37
42.67	17.00 0.50	29.00	1.25	0.95	0.906	1.05	36.18	20.00	6.49
44.17	18.00 0.50	31.00	1.25	0.95	0.960	1.02	37.57	20.00	6.60
45.61	19.00 0.50	33.00	1.25	0.95	1.014	0.99	38.91	20.00	6.71
47.00	20.00 0.50	35.00	1.25	0.95	1.069	0.97	40.20	20.00	6.81
46.85	21.00 0.50	35.40	1.25	0.95	1.124	0.94	39.65	21.00	7.20
46.73	22.00 0.50	35.80	1.25	0.95	1.179	0.92	39.16	22.00	7.57
46.65	23.00 0.50	36.20	1.25	0.95	1.234	0.90	38.70	23.00	7.94
46.59	24.00 0.50	36.60	1.25	0.95	1.288	0.88	38.29	24.00	8.30
46.56	25.00 0.50	37.00	1.25	0.95	1.343	0.86	37.91	25.00	8.65
48.45	26.00 0.50	40.40	1.25	0.95	1.398	0.85	40.57	22.40	7.88
50.10	27.00 0.50	43.80	1.25	0.95	1.453	0.83	43.15	19.80	6.95
54.06	28.00 0.50	47.20	1.25	1.00	1.508	0.81	48.05	17.20	6.01
55.30	29.00 0.50	50.60	1.25	1.00	1.562	0.80	50.60	14.60	4.70
56.31	30.00 0.50	54.00	1.25	1.00	1.617	0.79	53.08	12.00	3.23
53.96	31.00 0.50	51.60	1.25	1.00	1.672	0.77	49.88	13.60	4.08
51.66	32.00 0.50	49.20	1.25	1.00	1.727	0.76	46.80	15.20	4.86
49.37	33.00 0.50	46.80	1.25	1.00	1.782	0.75	43.83	16.80	5.54

47.09	34.00	44.40	1.25	1.00	1.837	0.74	40.95	18.40	6.14
	0.50								
44.82	35.00	42.00	1.25	1.00	1.891	0.73	38.18	20.00	6.65
	0.50								
44.62	36.00	42.40	1.25	1.00	1.946	0.72	37.99	20.00	6.63
	0.50								
44.44	37.00	42.80	1.25	1.00	2.001	0.71	37.82	20.00	6.62
	0.50								
44.27	38.00	43.20	1.25	1.00	2.056	0.70	37.66	20.00	6.61
	0.50								
44.11	39.00	43.60	1.25	1.00	2.111	0.69	37.51	20.00	6.59
	0.50								
43.96	40.00	44.00	1.25	1.00	2.165	0.68	37.38	20.00	6.58
	0.50								
45.27	41.00	46.00	1.25	1.00	2.220	0.67	38.59	20.00	6.68
	0.50								
46.55	42.00	48.00	1.25	1.00	2.275	0.66	39.78	20.00	6.77
	0.50								
47.81	43.00	50.00	1.25	1.00	2.330	0.66	40.95	20.00	6.87
	0.50								
49.05	44.00	52.00	1.25	1.00	2.385	0.65	42.09	20.00	6.96
	0.50								
50.26	45.00	54.00	1.25	1.00	2.440	0.64	43.22	20.00	7.05
	0.50								
48.72	46.00	52.80	1.25	1.00	2.494	0.63	41.79	20.00	6.93
	0.50								
47.22	47.00	51.60	1.25	1.00	2.549	0.63	40.40	20.00	6.82
	0.50								
45.76	48.00	50.40	1.25	1.00	2.604	0.62	39.04	20.00	6.72
	0.50								
44.33	49.00	49.20	1.25	1.00	2.659	0.61	37.72	20.00	6.61
	0.50								
42.93	50.00	48.00	1.25	1.00	2.714	0.61	36.42	20.00	6.51
	0.50								

CRR is based on water table at 100.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.30:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
1.00	0.04	0.50	1.00	0.50	1.07	0.54	0.69	5.00
2.00	0.07	0.50	1.00	0.50	1.07	0.54	0.69	5.00
3.00	0.11	0.50	1.00	0.50	1.07	0.54	0.69	5.00
4.00	0.14	0.50	1.00	0.50	1.07	0.54	0.69	5.00
5.00	0.18	0.50	1.00	0.50	1.07	0.54	0.69	5.00
6.00	0.21	0.50	1.00	0.50	1.07	0.54	0.69	5.00

7.00	0.25	0.50	1.00	0.50	1.07	0.54	0.68	5.00
8.00	0.28	0.50	1.00	0.50	1.07	0.54	0.68	5.00
9.00	0.31	0.50	1.00	0.50	1.07	0.54	0.68	5.00
10.00	0.35	0.50	1.00	0.50	1.07	0.54	0.68	5.00
11.00	0.38	0.50	1.00	0.50	1.07	0.54	0.68	5.00
12.00	0.41	0.50	1.00	0.50	1.07	0.54	0.68	5.00
13.00	0.45	0.50	1.00	0.50	1.07	0.54	0.67	5.00
14.00	0.48	0.50	1.00	0.50	1.07	0.54	0.67	5.00
15.00	0.52	0.50	1.00	0.50	1.07	0.54	0.67	5.00
16.00	0.55	0.50	1.00	0.50	1.07	0.54	0.67	5.00
17.00	0.59	0.50	1.00	0.50	1.07	0.54	0.67	5.00
18.00	0.62	0.50	1.00	0.50	1.07	0.54	0.67	5.00
19.00	0.66	0.50	1.00	0.50	1.07	0.54	0.66	5.00
20.00	0.69	0.50	1.00	0.50	1.07	0.54	0.66	5.00
21.00	0.73	0.50	1.00	0.50	1.07	0.54	0.66	5.00
22.00	0.77	0.50	1.00	0.50	1.07	0.54	0.66	5.00
23.00	0.80	0.50	1.00	0.50	1.07	0.54	0.66	5.00
24.00	0.84	0.50	1.00	0.50	1.07	0.54	0.66	5.00
25.00	0.87	0.50	1.00	0.50	1.07	0.54	0.65	5.00
26.00	0.91	0.50	1.00	0.50	1.07	0.54	0.65	5.00
27.00	0.94	0.50	1.00	0.50	1.07	0.54	0.65	5.00
28.00	0.98	0.50	1.00	0.50	1.07	0.54	0.65	5.00
29.00	1.02	0.50	1.00	0.50	1.07	0.54	0.65	5.00
30.00	1.05	0.50	1.00	0.50	1.07	0.53	0.65	5.00
31.00	1.09	0.50	0.99	0.50	1.07	0.53	0.64	5.00
32.00	1.12	0.50	0.99	0.49	1.07	0.53	0.64	5.00
33.00	1.16	0.50	0.98	0.49	1.07	0.53	0.63	5.00
34.00	1.19	0.50	0.98	0.49	1.07	0.52	0.62	5.00
35.00	1.23	0.50	0.97	0.49	1.07	0.52	0.62	5.00
36.00	1.27	0.50	0.97	0.48	1.07	0.52	0.61	5.00
37.00	1.30	0.50	0.96	0.48	1.07	0.51	0.61	5.00
38.00	1.34	0.50	0.96	0.48	1.07	0.51	0.60	5.00
39.00	1.37	0.50	0.95	0.48	1.07	0.51	0.60	5.00
40.00	1.41	0.50	0.95	0.47	1.07	0.51	0.59	5.00
41.00	1.44	0.50	0.94	0.47	1.07	0.50	0.58	5.00
42.00	1.48	0.50	0.94	0.47	1.07	0.50	0.58	5.00
43.00	1.51	0.50	0.93	0.47	1.07	0.50	0.57	5.00
44.00	1.55	0.50	0.93	0.46	1.07	0.50	0.57	5.00
45.00	1.59	0.50	0.92	0.46	1.07	0.49	0.56	5.00
46.00	1.62	0.50	0.92	0.46	1.07	0.49	0.56	5.00
47.00	1.66	0.50	0.91	0.46	1.07	0.49	0.55	5.00
48.00	1.69	0.50	0.91	0.45	1.07	0.49	0.54	5.00
49.00	1.73	0.50	0.90	0.45	1.07	0.48	0.54	5.00
50.00	1.76	0.50	0.90	0.45	1.07	0.48	0.53	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)

^ No-liquefiable Soils or above Water Table.

(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:

Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
1.00	-	-	-	89.63	20.00	0.00	89.63
2.00	-	-	-	89.63	20.00	0.00	89.63
3.00	-	-	-	89.63	20.00	0.00	89.63
4.00	-	-	-	89.63	20.00	0.00	89.63
5.00	-	-	-	89.63	20.00	0.00	89.63
6.00	-	-	-	76.68	16.00	0.00	76.68
7.00	-	-	-	61.40	12.00	0.00	61.40
8.00	-	-	-	50.38	10.00	0.00	50.38
9.00	-	-	-	50.12	10.00	0.00	50.12
10.00	-	-	-	44.04	10.00	0.00	44.04
11.00	-	-	-	41.96	12.00	0.00	41.96
12.00	-	-	-	40.18	14.00	0.00	40.18
13.00	-	-	-	38.58	16.00	0.00	38.58
14.00	-	-	-	37.10	18.00	0.00	37.10
15.00	-	-	-	39.47	20.00	0.00	39.47
16.00	-	-	-	41.11	20.00	0.00	41.11
17.00	-	-	-	42.67	20.00	0.00	42.67
18.00	-	-	-	44.17	20.00	0.00	44.17
19.00	-	-	-	45.61	20.00	0.00	45.61
20.00	-	-	-	47.00	20.00	0.00	47.00
21.00	-	-	-	46.85	21.00	0.00	46.85
22.00	-	-	-	46.73	22.00	0.00	46.73
23.00	-	-	-	46.65	23.00	0.00	46.65
24.00	-	-	-	46.59	24.00	0.00	46.59
25.00	-	-	-	46.56	25.00	0.00	46.56
26.00	-	-	-	48.45	22.40	0.00	48.45
27.00	-	-	-	50.10	19.80	0.00	50.10
28.00	-	-	-	54.06	17.20	0.00	54.06
29.00	-	-	-	55.30	14.60	0.00	55.30
30.00	-	-	-	56.31	12.00	0.00	56.31
31.00	-	-	-	53.96	13.60	0.00	53.96
32.00	-	-	-	51.66	15.20	0.00	51.66
33.00	-	-	-	49.37	16.80	0.00	49.37
34.00	-	-	-	47.09	18.40	0.00	47.09
35.00	-	-	-	44.82	20.00	0.00	44.82
36.00	-	-	-	44.62	20.00	0.00	44.62
37.00	-	-	-	44.44	20.00	0.00	44.44
38.00	-	-	-	44.27	20.00	0.00	44.27
39.00	-	-	-	44.11	20.00	0.00	44.11
40.00	-	-	-	43.96	20.00	0.00	43.96
41.00	-	-	-	45.27	20.00	0.00	45.27
42.00	-	-	-	46.55	20.00	0.00	46.55
43.00	-	-	-	47.81	20.00	0.00	47.81
44.00	-	-	-	49.05	20.00	0.00	49.05
45.00	-	-	-	50.26	20.00	0.00	50.26

46.00	-	-	-	48.72	20.00	0.00	48.72
47.00	-	-	-	47.22	20.00	0.00	47.22
48.00	-	-	-	45.76	20.00	0.00	45.76
49.00	-	-	-	44.33	20.00	0.00	44.33
50.00	-	-	-	42.93	20.00	0.00	42.93

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.

Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:

Settlement Analysis Method: Ishihara / Yoshimine

dsp	Depth	CSRsf	/ MSF*	=CSRm	F.S.	Fines	(N1)60s	Dr	ec	dsz
	S									
	ft					%		%	%	in.
in.	in.									

No Settlement of Saturated Sands

Settlement of Saturated Sands=0.000 in.

qc1 and (N1)60 is after fines correction in liquefaction analysis

dsz is per each segment, dz=0.05 ft

dsp is per each print interval, dp=1.00 ft

S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

ec	Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec
	dsz	dsp	S							
	ft	atm	atm			atm			%	
%	in.	in.	in.							

	49.95	2.71	1.76	43.00	0.53	2077.18	7.0E-4	0.3220	0.1018	1.01
0.1027	1.23E-3	0.001	0.001							
	49.00	2.66	1.73	44.33	0.54	2078.06	6.9E-4	0.3122	0.0987	1.01
0.0996	1.19E-3	0.023	0.024							
	48.00	2.60	1.69	45.76	0.54	2078.38	6.8E-4	0.3020	0.0955	1.01
0.0963	1.16E-3	0.023	0.048							
	47.00	2.55	1.66	47.22	0.55	2078.07	6.8E-4	0.2921	0.0924	1.01
0.0932	1.12E-3	0.023	0.070							
	46.00	2.49	1.62	48.72	0.56	2077.15	6.7E-4	0.2823	0.0893	1.01
0.0900	1.08E-3	0.022	0.092							
	45.00	2.44	1.59	50.26	0.56	2075.59	6.6E-4	0.2727	0.0862	1.01
0.0870	1.04E-3	0.021	0.114							

	44.00	2.38	1.55	49.05	0.57	2035.49	6.6E-4	0.2784	0.0881	1.01
0.0888	1.07E-3	0.021	0.135							
	43.00	2.33	1.51	47.81	0.57	1994.93	6.7E-4	0.2843	0.0899	1.01
0.0907	1.09E-3	0.022	0.156							
	42.00	2.28	1.48	46.55	0.58	1953.87	6.7E-4	0.8230	0.2603	1.01
0.2625	3.15E-3	0.046	0.202							
	41.00	2.22	1.44	45.27	0.58	1912.29	6.8E-4	0.8563	0.2708	1.01
0.2731	3.28E-3	0.064	0.267							
	40.00	2.17	1.41	43.96	0.59	1870.17	6.8E-4	0.8916	0.2820	1.01
0.2844	3.41E-3	0.067	0.334							
	39.00	2.11	1.37	44.11	0.60	1848.43	6.8E-4	0.8689	0.2748	1.01
0.2771	3.33E-3	0.067	0.401							
	38.00	2.06	1.34	44.27	0.60	1826.46	6.8E-4	0.8444	0.2670	1.01
0.2693	3.23E-3	0.066	0.467							
	37.00	2.00	1.30	44.44	0.61	1804.27	6.7E-4	0.8184	0.2588	1.01
0.2610	3.13E-3	0.064	0.530							
	36.00	1.95	1.27	44.62	0.61	1781.84	6.7E-4	0.7910	0.2501	1.01
0.2523	3.03E-3	0.062	0.592							
	35.00	1.89	1.23	44.82	0.62	1759.16	6.6E-4	0.7623	0.2411	1.01
0.2431	2.92E-3	0.059	0.651							
	34.00	1.84	1.19	47.09	0.62	1762.24	6.5E-4	0.6757	0.2137	1.01
0.2155	2.59E-3	0.055	0.706							
	33.00	1.78	1.16	49.37	0.63	1763.27	6.4E-4	0.6026	0.1906	1.01
0.1922	2.31E-3	0.049	0.755							
	32.00	1.73	1.12	51.66	0.64	1762.31	6.2E-4	0.5401	0.1708	1.01
0.1722	2.07E-3	0.044	0.798							
	31.00	1.67	1.09	53.96	0.64	1759.51	6.1E-4	0.4861	0.1537	1.01
0.1550	1.86E-3	0.039	0.837							
	30.00	1.62	1.05	56.31	0.65	1755.09	6.0E-4	0.4392	0.1389	1.01
0.1401	1.68E-3	0.035	0.873							
	29.00	1.56	1.02	55.30	0.65	1714.77	5.9E-4	0.4222	0.1335	1.01
0.1347	1.62E-3	0.033	0.905							
	28.00	1.51	0.98	54.06	0.65	1671.70	5.9E-4	0.4079	0.1290	1.01
0.1301	1.56E-3	0.032	0.937							
	27.00	1.45	0.94	50.10	0.65	1600.04	5.9E-4	0.4251	0.1344	1.01
0.1356	1.63E-3	0.033	0.970							
	26.00	1.40	0.91	48.45	0.65	1552.15	5.9E-4	0.4146	0.1311	1.01
0.1322	1.59E-3	0.032	1.002							
	25.00	1.34	0.87	46.56	0.65	1501.37	5.9E-4	0.4068	0.1287	1.01
0.1297	1.56E-3	0.031	1.033							
	24.00	1.29	0.84	46.59	0.66	1470.73	5.8E-4	0.3753	0.1187	1.01
0.1197	1.44E-3	0.030	1.063							
	23.00	1.23	0.80	46.65	0.66	1439.68	5.6E-4	0.3454	0.1092	1.01
0.1102	1.32E-3	0.028	1.091							
	22.00	1.18	0.77	46.73	0.66	1408.19	5.5E-4	0.3173	0.1003	1.01
0.1012	1.21E-3	0.025	1.116							
	21.00	1.12	0.73	46.85	0.66	1376.22	5.4E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.053	1.169							
	20.00	1.07	0.69	47.00	0.66	1343.72	5.3E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	1.245							

	19.00	1.01	0.66	45.61	0.66	1295.88	5.2E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	1.322							
	18.00	0.96	0.62	44.17	0.67	1247.24	5.1E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	1.399							
	17.00	0.91	0.59	42.67	0.67	1197.73	5.1E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	1.475							
	16.00	0.85	0.55	41.11	0.67	1147.27	5.0E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	1.552							
	15.00	0.80	0.52	39.47	0.67	1095.76	4.9E-4	0.8965	0.2936	1.01
0.2961	3.55E-3	0.073	1.624							
	14.00	0.75	0.48	37.10	0.67	1036.81	4.8E-4	0.8316	0.3194	1.01
0.3221	3.87E-3	0.085	1.709							
	13.00	0.69	0.45	38.58	0.67	1012.12	4.6E-4	0.6157	0.2146	1.01
0.2164	2.60E-3	0.063	1.773							
	12.00	0.64	0.41	40.18	0.68	985.55	4.4E-4	0.4556	0.1441	1.01
0.1453	1.74E-3	0.042	1.815							
	11.00	0.59	0.38	41.96	0.68	957.15	4.1E-4	0.3364	0.1064	1.01
0.1073	1.29E-3	0.030	1.845							
	10.00	0.53	0.35	44.04	0.68	927.23	3.9E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.031	1.875							
	9.00	0.48	0.31	50.12	0.68	918.78	3.5E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	1.952							
	8.00	0.43	0.28	50.38	0.68	869.59	3.4E-4	1.0000	0.3162	1.01
0.3189	3.83E-3	0.077	2.028							
	7.00	0.38	0.25	61.40	0.68	872.26	3.0E-4	0.2973	0.0940	1.01
0.0948	1.14E-3	0.056	2.084							
	6.00	0.33	0.21	76.68	0.69	872.32	2.6E-4	0.0851	0.0269	1.01
0.0271	3.26E-4	0.011	2.096							
	5.00	0.27	0.18	89.63	0.69	839.84	2.2E-4	0.0478	0.0151	1.01
0.0153	1.83E-4	0.005	2.101							
	4.00	0.22	0.14	89.63	0.69	751.18	2.0E-4	0.0781	0.0247	1.01
0.0249	2.99E-4	0.004	2.105							
	3.00	0.16	0.11	89.63	0.69	650.54	1.7E-4	0.0389	0.0123	1.01
0.0124	1.49E-4	0.004	2.108							
	2.00	0.11	0.07	89.63	0.69	531.16	1.4E-4	0.0268	0.0085	1.01
0.0086	1.03E-4	0.002	2.111							
	1.00	0.05	0.04	89.63	0.69	375.59	1.0E-4	0.0201	0.0063	1.01
0.0064	7.68E-5	0.002	2.113							

Settlement of Unsaturated Sands=2.113 in.
 dsz is per each segment, dz=0.05 ft
 dsp is per each print interval, dp=1.00 ft
 S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=2.113 in.
 Differential Settlement=1.056 to 1.394 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =

pcf; Depth = ft; Settlement = in.

1 atm (atmosphere)	= 1.0581 tsf(1 tsf = 1 ton/ft ² = 2 kip/ft ²)
1 atm (atmosphere)	= 101.325 kPa(1 kPa = 1 kN/m ² = 0.001 Mpa)
SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction
F.S.=CRRm/CSRfsf	
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement

calculation $CSR_m = CSR_{sf} / MSF^*$
 inputed fs Cyclic stress ratio induced by earthquake with user
 MSF* Scaling factor from CSR, $MSF^*=1$, based on Item 2 of
 Page C.

ec Volumetric strain for saturated sands
 dz Calculation segment, $dz=0.050$ ft
 dsz Settlement in each segment, dz
 dp User defined print interval
 dsp Settlement in each print interval, dp
 Gmax Shear Modulus at low strain
 g_eff gamma_eff, Effective shear Strain
 g*Ge/Gm gamma_eff * G_eff/G_max, Strain-modulus ratio
 ec7.5 Volumetric Strain for magnitude=7.5
 Cec Magnitude correction factor for any magnitude
 ec Volumetric strain for unsaturated sands, $ec=Cec * ec7.5$
 NoLiq No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
 SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

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Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

APPENDIX E

GENERAL EARTHWORK GRADING GUIDELINES

**Proposed Single-Family Residential Development
Highland Avenue and Medical Center Drive
San Bernardino, San Bernardino County, California
Project No. 2849-CR**



GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is GeoTek's hope that these will assist the contractor to complete the project more efficiently by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code, CBC (2019) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding GeoTek's 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 GeoTek's 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 GeoTek's 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 GeoTek's representative does not provide the contractor with these reports, GeoTek's 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; GeoTek's 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 GeoTek's representative prior to placing any fill. It will be the contractor's responsibility to notify GeoTek's 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 GeoTek's 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, GeoTek's 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 GeoTek's representative.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep affected 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 GeoTek's representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory backhoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by GeoTek's 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, GeoTek's representative.

5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all GeoTek 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 GeoTek's field personnel on grading and construction projects.

1. **Safety Meetings:** GeoTek's 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 GeoTek's personnel while on the job site.
3. **Safety Flags:** Safety flags are provided to GeoTek's 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 GeoTek's personnel not following the above, we request that it be brought to the attention of GeoTek's office.

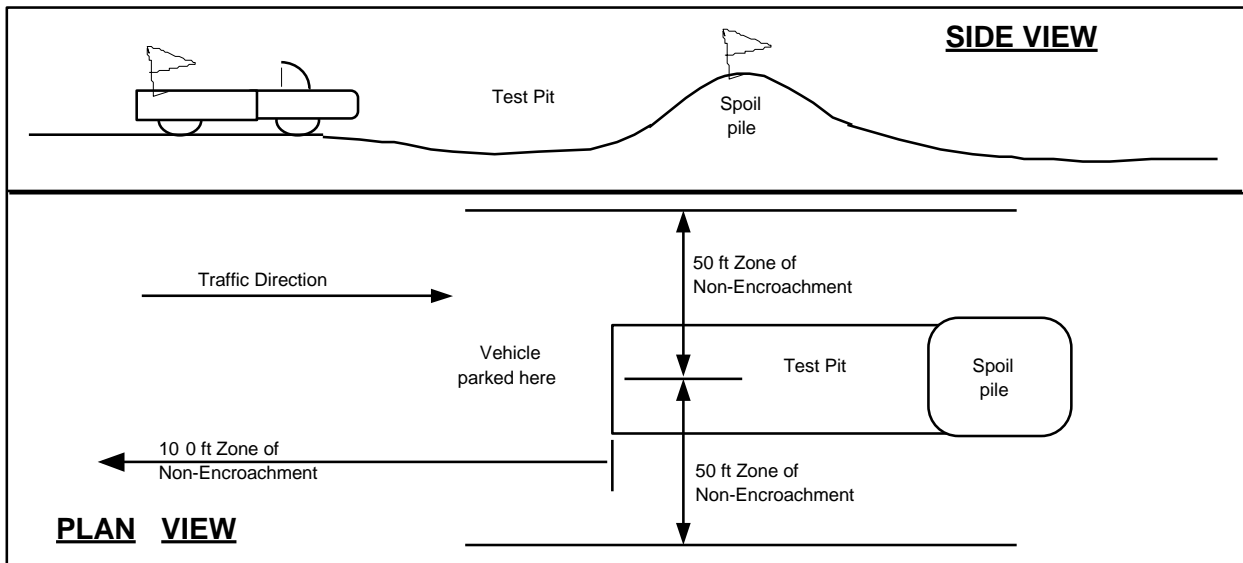
Test Pits Location, Orientation and Clearance

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. GeoTek personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

GeoTek 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, GeoTek company policy requires that the soil technician withdraws and notifies their supervisor. The contractor's representative will then be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

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

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

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

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