APPENDIX C

GEOTECHNICAL AND INFILTRATION EVALUATION

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DEPARTMENT OF PUBLIC WORKS
SUBDIVISIONS
APPROVED
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GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED TOWNHOMES PROJECT
2219 WEST ORANGE AVENUE
ANAHEIM, ORANGE COUNTY, CALIFORNIA

PREPARED FOR

MELIA HOMES
8951 RESEARCH DRIVE
IRVINE, CALIFORNIA 92618

PREPARED BY

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

PROJECT NO. 2883-CR **SEPTEMBER 21, 2021**

GEOTEK

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September 21, 2021
Project No. 2883-CR

Melia Homes
8951 Research Drive
Irvine, California 92618

Attention: Mr. Chad Brown

Subject: Geotechnical and Infiltration Evaluation
Proposed Townhomes Project
2219 West Orange Avenue
Anaheim, Orange County, California

Dear Mr. Brown:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this geotechnical and infiltration evaluation for the proposed project located in Anaheim, Orange 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 our office.

Respectfully submitted,
GeoTek, Inc.

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

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical engineering and geologic conditions at the project site, as outlined in GeoTek's proposal P-0700421r2-CR, revised August 19, 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 about 21.5 to 51.5 feet below grade,
- Excavation of two (2) additional borings to depths of about 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 subject project site is addressed as 2219 West Orange Avenue, in the City of Anaheim, Orange County, California (see Figure 1). The approximate 1.28-acre rectangular-shaped project site is bounded to the south by Orange Avenue, to the west by an existing church facility, to the north by single-family residences and a commercial development and to the east by existing commercial businesses. The proposed building site is currently occupied by a school facility which includes various one-story buildings, out-structures, play courts, parking/drive areas, hardscaping, as well as landscaping and pavement improvements.

The site has generally flat topography with a gently fall of about four to five feet to the southwest. Surface drainage is by sheetflow to the south towards Orange Avenue.

2.2 PROJECT DESCRIPTION

According to the Architecture Site Plan, prepared by Summa Architects, dated May 10, 2021, the property will be developed with 24 townhomes with attached garages and related improvements. The structures are anticipated to be up to three stories in height, of wood-framed construction, and will utilize concrete slab-on-grade floors and shallow foundations. For the purposes of this report, it is assumed maximum column and wall loads of about 100 kips and 5 kips per foot, respectively. Once actual loads are known that information should be provided to GeoTek to determine if modifications to the recommendations presented in this report are warranted.

Based upon past experience, grading of the site will involve cuts and fills generally less than about 5 feet in height, not including any recommended remedial grading. Sewage disposal is anticipated to be be provided by a public sewer system. Stormwater at the site may be managed via relatively shallow infiltration systems or a "drywell" system. Specific location and depth of these systems are unknown currently. 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 September 1, 2021 and consisted of excavating five (5) geotechnical exploratory test borings (Borings B-1 through B-5) with a hollow-stem drill rig to depths ranging from about 21.5 to 51.5 feet below grade. The approximate locations of the GeoTek borings are shown on the Exploration Location Map (Figure 2). A geologist from GeoTek logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory borings are included in Appendix A.

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

In Boring B-1 standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The sampler penetration test data are presented on the Log for Boring for Boring B-1.

Percolation Testing

In addition to the geotechnical exploratory borings, two (2) borings (I-1 and I-2) were excavated in the area of the anticipated storm water control systems as designated by the project developer. In addition to borings I-1 and I-2, boring B-1 was converted to an infiltration boring by partial backfill of the boring to a depth of about 40 feet, to a depths below the contact between the fine-grained alluvium (ML soil type) and the coarse-grained alluvium (SM soil type). Infiltration/percolation testing was conducted in these borings in general accordance with the requirements of the County of Orange.

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, the percolation tests were performed which consisted of adding water to each test hole and measuring the water drop over a 30-minute period. The water drop was recorded for twelve 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 infiltration rates calculated using the Porchet Method are presented in the following table:

SUMMARY OF INFILTRATION RATES		
Boring	Depth of Test (Feet)	Infiltration Rate (Inches per hour)
I-1	5.0	0.38
I-2	5.0	4.16
B-1	40.0	1.64

The results of the conversions indicate infiltration rates of 0.38 to 4.16 inch per hour, which indicate highly variable infiltration rates based upon depth and location. 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.

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

4.2 GENERAL SOIL CONDITIONS

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

4.2.1 Artificial Fill

Artificial fill (asphalt concrete pavement sections consisting of asphalt concrete and aggregate base) was encountered in Borings B-1, B-2 and B-3. Borings B-4 and B-5 were conducted within landscape (lawn) areas of the site.

4.2.2 Alluvium

Alluvium was encountered beneath the fill/lawn in all the exploratory borings. The alluvium was found to consist of interbedded layers of silty and sandy clay, sandy and clayey silts, silty sands, and relatively clean sands (CL, ML, and SM soil types based upon the Unified Soil Classification System). The fine-grained alluvial soils (CL and ML soil types) were found to be medium stiff to hard while the coarse-grained alluvium was found to be medium dense to very dense.

Based on the results of laboratory testing, the upper site soils are considered to have a "low" (21-50) expansion potential (ASTM D 4829). Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). The test results are provided in Appendix B.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

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

4.3.2 Groundwater

Groundwater was not encountered within any of the test borings drilled at the site. Based on review of available data, it is estimated that the depth to high groundwater at the site is greater than about 50 feet below grade. 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 Newport-Inglewood fault zone located about 5 miles to the southwest.

4.4.2 Seismic Design Parameters

The site is located at approximately 33.8252 degrees West Latitude and -117.9605 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, Cs, is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of T≤1.5Ts and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for T_L≥T>1.5Ts 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 as it is assumed that the exception as allowed in ASCE 7-16 is applicable. If the exception is deemed not appropriate, a site-specific ground motion analysis will be required.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.458g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.514g
Site Coefficient for Site Class "D", F_a	1
Site Coefficient for Site Class "D", F_v	1.786
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.458g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.918g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.972
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.612
Peak Ground Acceleration (PGA_M)	0.682g
Seismic Design Category	D

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

4.5 LIQUEFACTION

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

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

Based on a review of the Orange County Parcel Report, the site is not located within an area mapped as being susceptible to liquefaction.

Based on the current mapping by Orange County and the lack of shallow groundwater, it is GeoTek's opinion that the site is not susceptible to liquefaction during a seismic event. Due to the fine-grained nature of the upper site soils and the dense/stiff nature of the underlying alluvium, seismic induced ("dry sand") settlements are estimated to be minimal.

4.6 OTHER SEISMIC HAZARDS

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

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

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

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

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

5.2.2 Site Clearing

Initial site preparation should commence with removal of debris, existing structures, pavements, underground utilities, foundations, slabs-on-grade, 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

Demolition and removal of the existing on-site structure foundations, slabs and utility lines is anticipated to disturb the upper site soils. Following site demolition it is recommended that the soils be removed beneath the planned building footprint to a depth of at least five (5) feet below existing grade, or three (3) feet below the base of the proposed foundations, whichever is greater. The lateral extent of this recommended over-excavation should extend at least 5 feet beyond the building limits. Removal bottoms should be relatively uniform in soil type which is not visibly porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures.

Following site clearing operations, over-excavation and lowering of site grades, where necessary, it is recommended that the exposed subgrade soils beneath all surface improvements be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the geotechnical engineering representative. 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 inches) and other deleterious material. All areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in 6-to-8-inch loose lifts, moisture conditioned to about three percent above the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D-1557 test procedures.

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

5.2.5 Oversized Rock Disposal

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

5.2.6 Excavation Characteristics

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

5.2.7 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.

5.2.8 Shrinkage and Bulking

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

Shrinkage is primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 0 to 5 percent may be considered for the surficial soils. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction.

A subsidence loss of up to about 0.1 foot is estimated for the site.

5.2.9 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

Presented below are post-tensioned foundation design parameters for the proposed structure at the site. These parameters are in general conformance with *Design of Post-Tensioned Slabs-on-Ground*, Third Edition *with 2008 Supplement* (PTI, 2008). These recommendations are minimal recommendations and are not intended to supersede the design by the project structural engineer.

Expansion Index, Atterberg Limits (including plasticity index), soil particle size analysis (including percent passing #200 sieve and clay percentage) and soluble sulfate evaluation of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions.

DESIGN PARAMTERS FOR POST-TENSIONED SLABS	
Foundation Design Parameter	Design Value "Low" Expansion Potential (LL≤27; PI≤10; Passing #200 Sieve ≈ 60%; Clay fines ≈ 28%)
Edge Moisture Variation Distance, e_m -Edge Lift (swelling) -Center Lift (shrinkage)	 5.0 ft 9.0 ft
Soil Differential Movement, y_m -Edge Lift (swelling) -Center Lift (shrinkage)	 ≈1.48 in ≈-1.03 in
Exterior Perimeter Beam Embedment	Three-Story – 18 inches*
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 11% to a depth of 12 inches

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

Post-tensioned slabs should be designed in accordance with the *2019 CBC* and PTI design methodology.

The bottom of the perimeter edge beam/deepened footing should be designed to resist tension forces using either cable or conventional reinforcement, per the structural engineer.

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 conventional foundation design recommendations is presented in the table below:

Design Parameter	"Low" Expansion Potential (21≤EI≤50)
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	18 – Three-Story
Minimum Foundation Width (Inches)*	18 – Three-Story
Minimum Slab Thickness (actual)	4 inches (actual)
Minimum Slab Reinforcing	6" x 6" – W2.9/W2.9 welded wire fabric or No. 3 reinforcing bars at 18 inches on center each way placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 Reinforcing Bars, one top and one bottom
Effective Plasticity Index**	35
Presaturation of Subgrade Soil (Percent of Optimum/Depth in inches)	Minimum 110% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

*Code minimums per Table 1809.7 of the 2019 CBC should be complied with.
**Effective Plasticity Index should be verified at the completion of the rough grading

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 18 inches deep and 18 inches wide, and pad footings 24 inches square and 24 inches deep. This allowable soil bearing capacity may be increased by 300 psf for each additional foot of footing depth and 150 psf for each additional foot of footing width to a maximum value of 3,000 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 200 psf per foot of depth, to a maximum earth pressure of 2,000 psf for footings founded on engineered fill or competent native soil. A coefficient of friction between soil and concrete of 0.3 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 18 inches wide and 18 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

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

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

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

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

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

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

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

5.3.2 Miscellaneous Foundation Recommendations

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

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

5.3.3 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2019 CBC or County of Orange 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 two (2) samples collected during the field investigation. The results of the testing indicate that the on-site soils are considered "corrosive" (3,350 to 5,360 ohm-cm) (Roberge, 2000) to buried ferrous metal in accordance with current standards used by corrosion engineers. It is recommended that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site.

5.3.5 Soil Sulfate Content

The sulfate content was determined in the laboratory on two (2) samples 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 and/or competent native soil. Retaining wall foundations should be designed in accordance with Section 5.3 of this report. Structural needs may govern and should be evaluated by the project structural engineer.

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

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

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

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

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

5.4.1.2 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, or seismic events.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (horizontal : vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	42
2:1	65

*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 18 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 ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The upper 24 inches should consist of compacted on-site materials. The rock should be separated from the earth with filter fabric. The presence of other materials might necessitate revision 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 ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the 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 65 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 proposed parking, access roads and adjacent street improvements within the site are not yet known, the following preliminary pavement design recommendations are based on assumed Traffic Indexes of 5.0 for car parking areas and 6.0 for access drives. Preliminary pavement thickness design is based on the CalTrans Highway Design Manual (2018). An R-value of 25 was assumed for the determination of preliminary pavement sections for this report. Once the traffic loading information becomes more defined, revision to the pavement design recommendations may be warranted. It is recommended that the final pavement design be based on R-value testing of the as-graded subgrade soils within the pavement areas.

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

PRELIMINARY MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0 (Car Parking Areas)	3	8
6.0 (Automobile Access Lanes)	4	8

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 Anaheim.

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 our evaluation is limited to the area explored that is shown on the Exploration Location Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to GeoTek by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, GeoTek's proposal (Proposal No. P-0700421r2-CR) dated August 19, 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

American Concrete Institute (ACI), 2006, Publication 302.2R-06, Guide for Concrete Slabs That Receive Moisture Sensitive Flooring Materials.

______, 2010, Publications 360R-10, Guide to Design of Slabs-On-Ground.

Bowles, J. E., 1977, "Foundation Analysis and Design", Second Edition.

Bryant, W.A., and Hart, E.W., 2007, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps, California Geological Survey: Special Publication 42.

California Code of Regulations, Title 24 Part 2, 2019 "California Building Code," 2 volumes.

California Department of Transportation, 2018, "Highway Design Manual", 6th Edition.

Department of the Navy, 1982, "Foundations and Earth Structures", Design Manual 7.2.

GeoTek, Inc., In-house proprietary information.

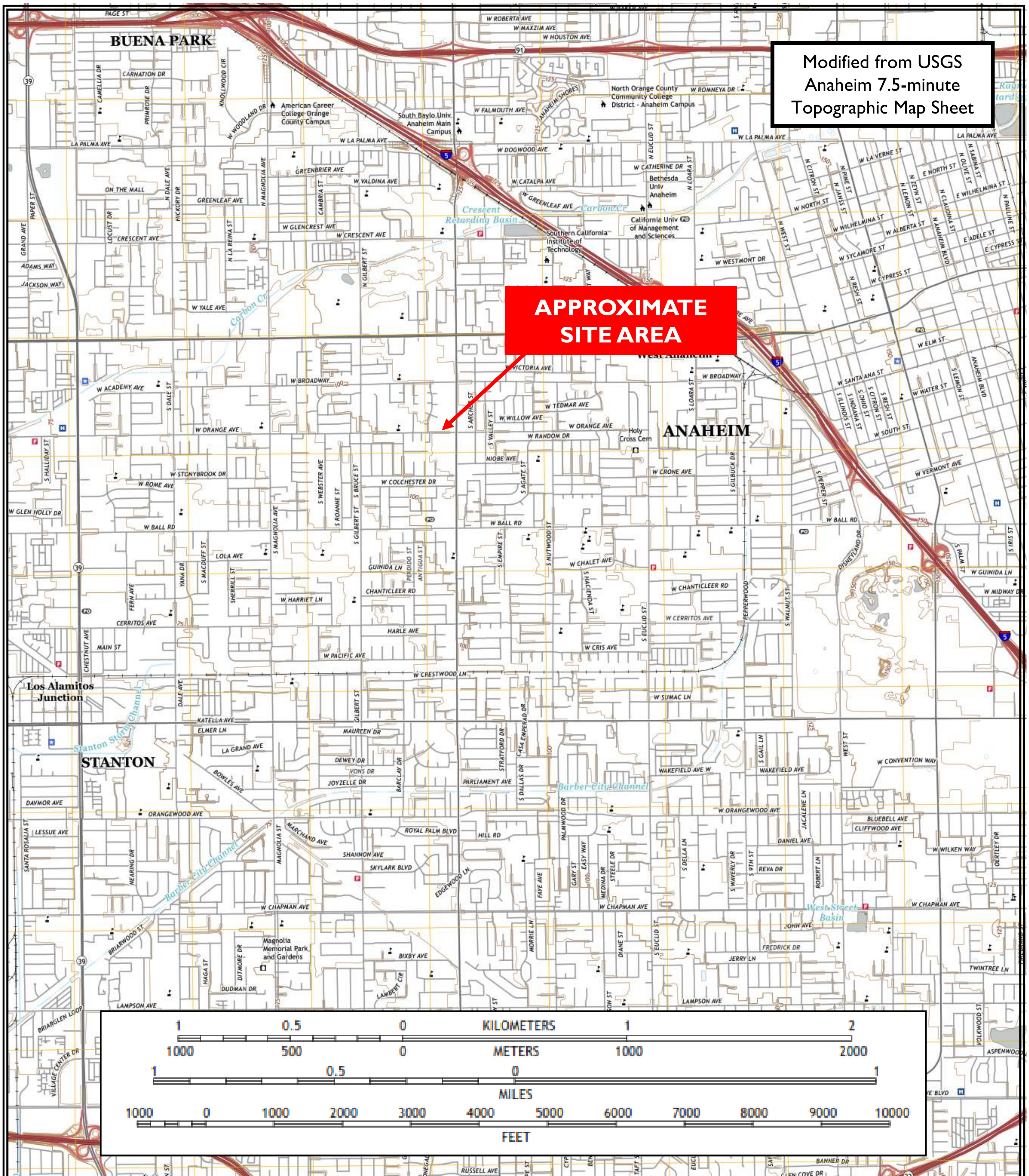
Peck R. B., Hanson W. H. & Thornburn T. H., 1974, "Foundation Engineering", Second edition.

Roberge, P. R., 2000, "Handbook of Corrosion Engineering".

SEA/OSHPD web service, "Seismic Design Maps" (https://seismicmaps.org)

Terzaghi, K. and Peck, R. B., 1967, "Soil Mechanics in Engineering Practice", Second Edition.

USGS, 2003, "Preliminary Geologic Map of the Elsinore 7.5' Quadrangle, Orange County, California", Morton, D. M. and Weber, F. H., USGS OF-2003-281.



Melia Homes
2219 West Orange Avenue
Anaheim, Orange County, California

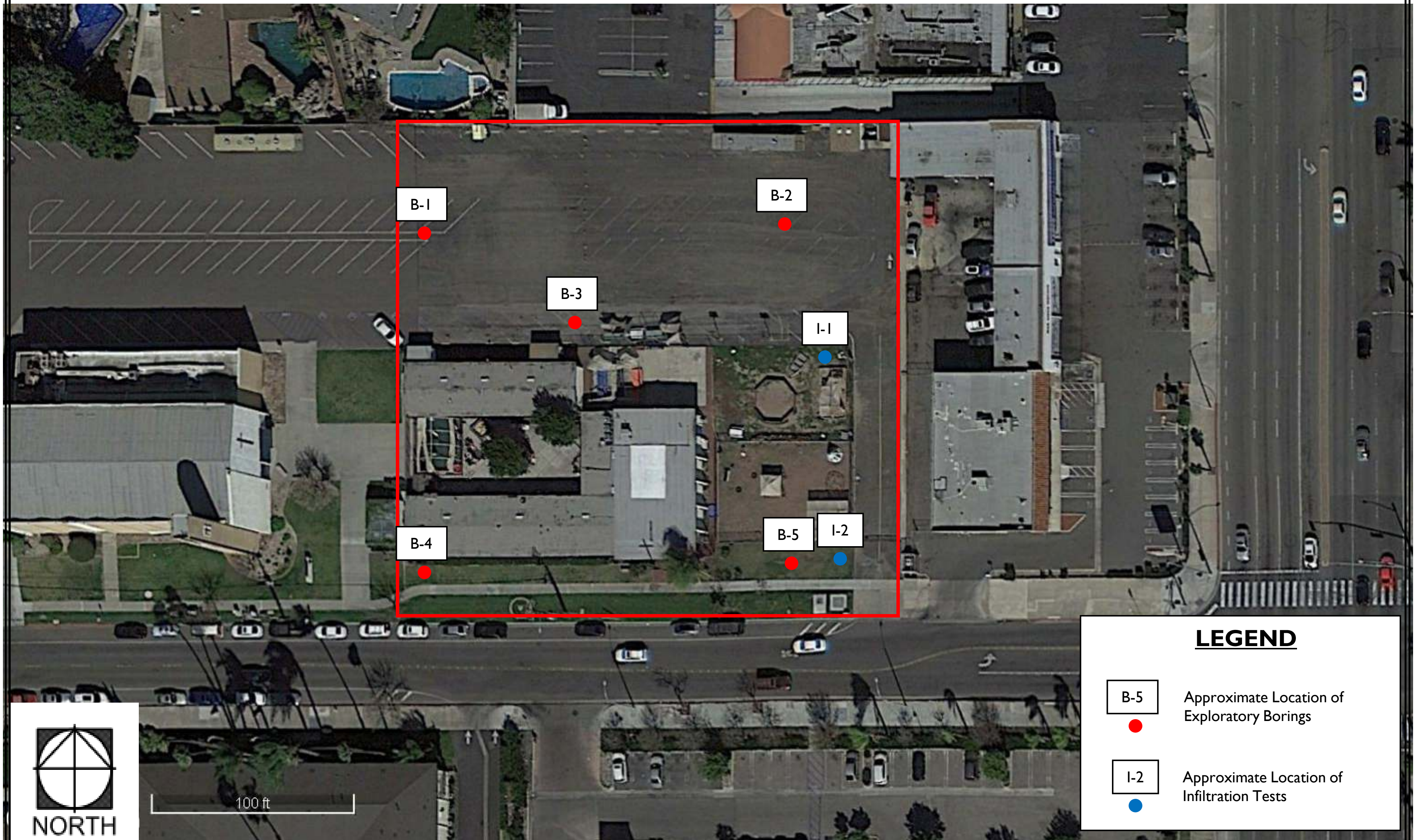
Project No. 2883-CR

NORTH

Figure 1

Site Location Map

GEOTEK



Melia Homes
Proposed Townhomes Project
2219 West Orange Avenue
Anaheim, Orange County, California

GeoTek Project No. 2883-CR

Scale As Shown

Figure 2

Exploration Location Map

GEOTEK

APPENDIX A

LOG OF EXPLORATORY BORINGS

Proposed Townhomes Project

2219 West Orange Avenue

Anaheim, Orange County, California

Project No. 2883-CR

GEOTEK

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 log)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Boring No.: B-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0	Large Bulk				2 in. asphalt over no base **Alluvium:**			Expansion Index = 37 60% passing #200 Sieve 28% Clay Liquid Limit = 27 Plastic Limit = 18 Plasticity Index = 9 Corrosion Test
	Ring	3 5 8		CL	Silty CLAY, brown, slightly moist, medium stiff	17.4	106.3	
	Ring	3 7 12			Fine sandy CLAY, brown, slightly moist, stiff			
5	Ring	3 9 6			Silty CLAY, brown, slightly moist, medium siff	8.3	115.6	
	Ring	3 7 13			Silty CLAY, brown, slightly moist, stiff	11.5	113.7	
10	Ring	6 10 11		ML	Fine sandy SILT with clay, brown, slightly moist, stiff	11.7	112.9	
15	Ring	11 25 32		SM	Silty f SAND, light brown/gray, slightly moist, dense			
20	Ring	11 25 28			Silty f SAND, lightly brown/gray, slightly moist, dense	5.9	111.9	
25	Ring	6 20 31		ML	Fine sandy SILT with clay, brown, slightly moist, hard	16.7	112.7	
30	SPT	11 23 26			Same as above	16.0		

LEGEND

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

Lab testing:

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Boring No.: B-1 (cont.)

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
35	SPT	17 19 24		ML	Fine sandy SILT, light brown, slightly moist, hard			
				SM				
40	SPT	17 17 25			Silty f-m SAND, lightly brown, moist, medium dense	13.8		26% Passing #200 Sieve
45	SPT	18 25 25		SM	Silty fine SAND, light brown, moist, dense	12.4		22% Passing #200 Sieve
50	SPT	16 22 36			Silty f-m SAND, lightly brown, very moist, dense	13.2		14% Passing #200 Sieve
					BORING TERMINATED AT 51.5 FEET No Groundwater Encountered Spoils backfilled 10 feet and boring prepped for infiltration testing at 40 feet			
55								
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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Boring No.: B-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					2 inches asphalt over 4 inches base			
					Alluvium:			
				ML	Clayey SILT, light brown, slightly moist, medium stiff			
	Ring	7 7 8				11.5	109.7	
5	Ring	8 15 15			Clayey SILT with trace sand, brown, slightly moist, stiff			
	Ring	2 5 8			Clayey SILT, brown, slightly moist, medium stiff	24.5	101.6	
10	Ring	5 8 12			Sandy SILT with clay, brown, slightly moist, stiff	16.3	115	
15	Ring	10 26 32		SM	Silty fine SAND, light brown/gray, slightly moist, dense	4.5	121.3	
20	Ring	13 27 24			Silty fine SAND, light brown/gray, slightly moist, medium dense	12.4	112.2	
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Spoils backfilled and surface patched with asphalt concrete			
25								
30								

LEGEND

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

Lab testing:

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Boring No.: B-3

Depth (ft)	Sample Type (SAMPLES)	Blows/ 6 in (SAMPLES)	Sample Number (SAMPLES)	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%) (Laboratory Testing)	Dry Density (pcf) (Laboratory Testing)	Others (Laboratory Testing)
0					2 inches aspalt over no base			
					Alluvium:			
	Ring	4 4 5		ML	Clayey SILT, dark brown, slightly moist, soft	22.5	102.4	
	Ring	4 6 10			Clayey SILT with trace sand, dark brown, slightly moist, medium stiff			
5	Ring	4 5 6			Clayey SILT, dark brown, slightly moist, medium stiff			
10	Ring	5 11 17		SM	Silty f SAND, light brown, slightly moist, medium dense	16.3	113	
15	Ring	12 21 30			Silty f-m SAND, yellow/brown, slightly moist, medium dense	3.3	115.5	
20	Ring	13 11 8		ML	Fine sandy SILT, brown, slightly moist, loose	5.2	107.2	
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Spoils backfilled and surface patched with asphalt concrete			
25								
30								

LEGEND

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

Lab testing:

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Boring No.: B-4

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0					**Alluvium:**			
				ML	Sandy SILT, light brown, dry, very stiff, trace organics			
	Ring	14 27 39				6.0	113.6	
	Ring	18 19 13		SM	Silty f SAND, light brown, slightly moist, medium dense	10.4	109.9	
5	Ring	3 6 9		CL	Silty CLAY, brown/gray, slightly moist, medium stiff	19.3	103.7	
10	Ring	5 5 12		ML	Sandy SILT, light brown, slightly moist, medium stiff			
15	Ring	7 13 13		ML	Sandy SILT w/ trace clay, light brown, slightly moist, stiff	14.7	112.9	
20	Ring	8 15 16		CL	Silty CLAY, brown, slightly moist, very stiff	16.9	106.9	
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Spoils backfilled			
25								
30								

LEGEND

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

Lab testing:

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Melia Homes	**DRILLER:** 2R Drilling	**LOGGED BY:** GP/CD	
PROJECT NAME: 2219 W Orange Ave	**DRILL METHOD:** Hollow Stem	**OPERATOR:** Ish/Antonio	
PROJECT NO.: 2883-CR	**HAMMER:** 140#/30"	**RIG TYPE:** CME 75	
LOCATION: Anaheim, CA		**DATE:** 9/1/2021	

Boring No.: B-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0	Large Bulk				**Alluvium:**			
				ML	Sandy SILT, light brown, slightly moist, very stiff			Expansion Index = 30 Maximum Density Test Remolded Direct Shear Corrosion Test
	Ring	10 29 32						
5	Ring	6 10 12			Clayey SILT, brown, slightly moist, stiff	10.7	108.1	
	Ring	6 10 16			Sandy SILT, brown, slightly moist, stiff			
10	Ring	7 8 10			Sandy SILT, brown, slightly moist, medium stiff	6.6	111	
15	Ring	17 22 24		SM	Silty f-m SAND, light brown, slightly moist, medium dense	2.4	109.8	
20	Ring	12 14 17		CL	Silty CLAY, brown, slightly moist, medium stiff	13.6	95.6	
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Spoils backfilled			
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	**Boring No.: I-1** — **MATERIAL DESCRIPTION AND COMMENTS**	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					2 inches asphalt over no base			
					Alluvium:			
				ML	Sandy SILT w/ trace clay, brown, slightly moist			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	GP/CD
PROJECT NAME:	2219 W Orange Ave	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Ish/Antonio
PROJECT NO.:	2883-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Anaheim, CA			**DATE:**	9/1/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	**Boring No.: I-2** **MATERIAL DESCRIPTION AND COMMENTS**	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					**Alluvium:**			
				ML	Sandy SILT, light brown, slightly moist			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30								

LEGEND

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

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

APPENDIX B

RESULTS OF LABORATORY TESTING

Proposed Townhomes Project

2219 West Orange Avenue

Anaheim, Orange County, California

Project No. 2883-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Atterberg Limits
Atterberg limits testing were performed on a bulk sample collected from the site. The tests were performed in general accordance with ASTM D 4318. Results of these tests are shown on the boring logs at the appropriate sample depths in Appendix A.

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

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

Expansion Index
Expansion Index testing was performed on two (2) bulk soil samples obtained from the site. 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-1	0-5	Silty Clay	37	Low
B-5	0-5	Sand Silt	30	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 one bulk sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with test method ASTM Test Procedure D 1557. The results of the testing are provided in Appendix B.

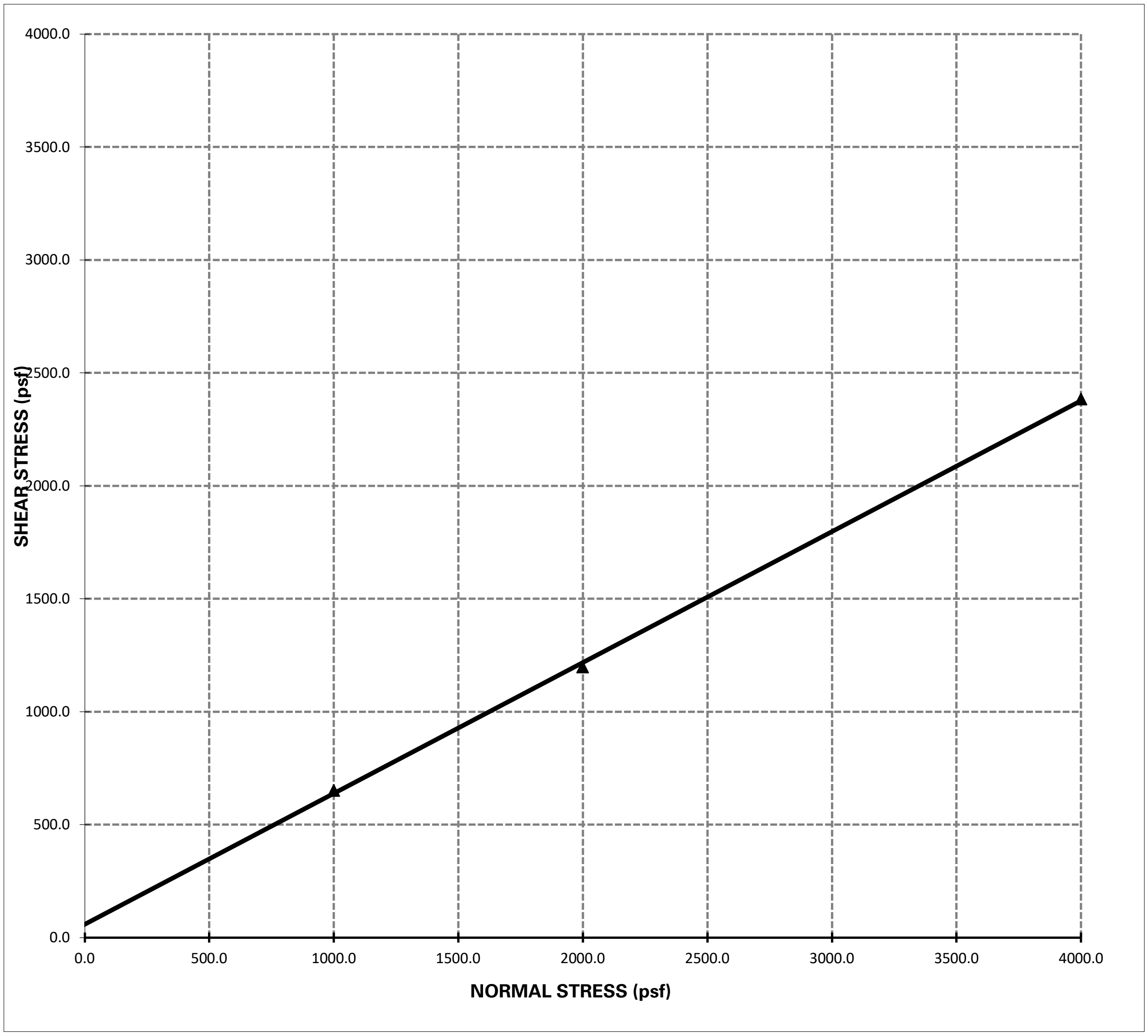
Sieve/Hydrometer
Sieve/hydrometer testing was performed on samples collected from the site. The tests were performed in general accordance with ASTM D 6913 and D 7928. The test results are presented Appendix B and on the boring logs at the appropriate sample depths in Appendix A.

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

Boring No.	Depth (ft.)	pH ASTM D4972	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-1	0-5	9.3	4.8	0.0040	5,360
B-5	0-5	8.4	33.0	0.0076	3,350

DIRECT SHEAR TEST

Project Name:	Melia Homes	***Sample Location:***	B5 @ 0-5'
Project Number:	2883-CR	***Date Tested:***	9/20/2021



Shear Strength: Φ = **30 °** , **C = 58 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.

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EXPANSION INDEX TEST

(ASTM D4829)

Client: Melia Homes

Project Number: 2883-CR

Project Location: West Orange Ave. Townhomes Anaheim

Tested/ Checked By: RL Lab No Corona

Date Tested: 9/13/2021

Sample Source: B1 @ 0-5'

Sample Description:

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	753.1
B	Weight of ring (gm)	366.7
C	Net weight of sample (gm)	**386.4**
D	Wet Density, lb / ft3 (C*0.3016)	**116.5**
E	Dry Density, lb / ft3 (D/1.F)	**105.3**

SATURATION DETERMINATION

F	Moisture Content, %	10.7
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**48.1**

READINGS			
DATE	TIME	READING	
9/13/2021		0.1230	Initial
9/13/2021		0.1230	10 min/Dry
9/14/2021		0.1600	Final

FINAL MOISTURE	
Final Weight of wet sample & tare	% Moisture
786.5	**19.3**

EXPANSION INDEX = 37

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: Melia Homes
Project Number: 2883-CR
Project Location: West Orange Ave. Townhomes Anaheim

Tested/ Checked By: RL Lab No Corona
Date Tested: 9/13/2021
Sample Source: B5 @ 0-5'
Sample Description:

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	771.4
B	Weight of ring (gm)	365.4
C	Net weight of sample (gm)	**406.0**
D	Wet Density, lb / ft3 (C*0.3016)	**122.4**
E	Dry Density, lb / ft3 (D/1.F)	**111.8**

SATURATION DETERMINATION

F	Moisture Content, %	9.5
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.6**

READINGS

DATE	TIME	READING	
9/13/2021		0.7320	Initial
9/13/2021		0.7320	10 min/Dry
9/14/2021		0.7620	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
789.9	**14.1**

EXPANSION INDEX = 30

GEOTEK

MOISTURE/DENSITY RELATIONSHIP

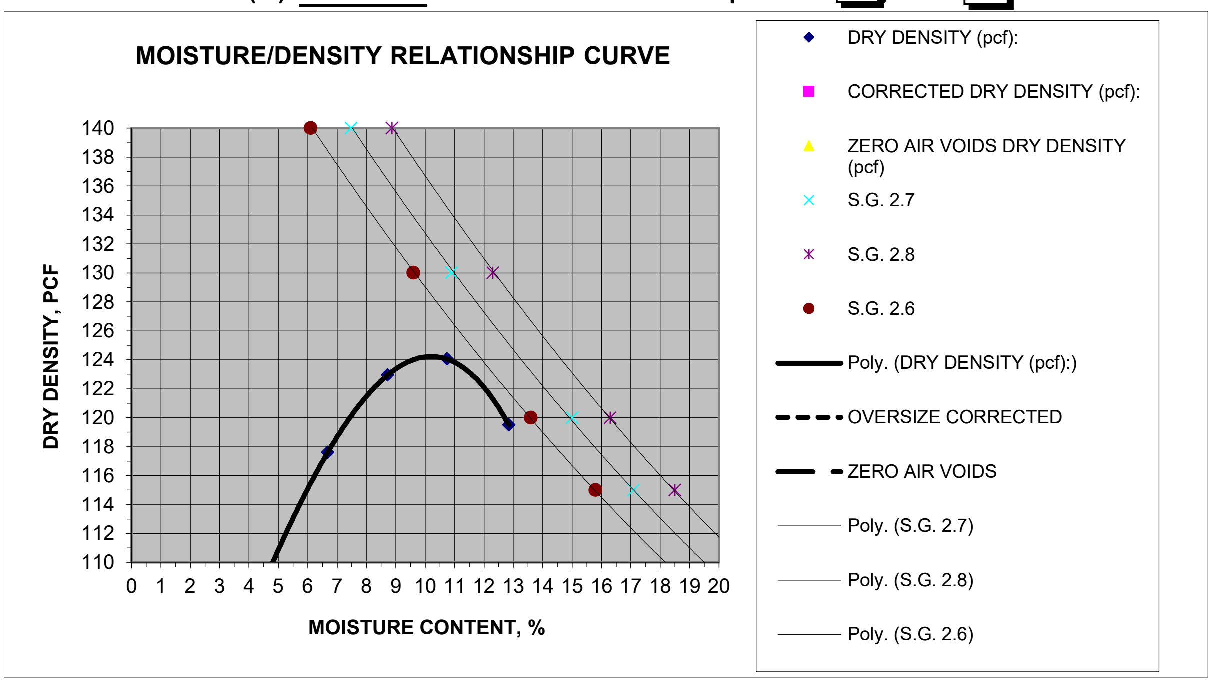
Client: Melia Homes
Project: 2219 W. Orange
Location: Anaheim
Material Type: Brown Silty Sand
Material Supplier: -
Material Source: -
Sample Location: B5 @ 0-5'
-

Job No.: 2883-CR
Lab No.: Corona

Sampled By: GP/CD
Received By: RJ
Tested By: AD
Reviewed By: RJ

Date Sampled: 9/7/2021
Date Received: 9/7/2021
Date Tested: 9/16/2021
Date Reviewed: 9/17/2021

Test Procedure: ASTM D1557 **Method:** A
Oversized Material (%): 0.3 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	123.0	**@ Optimum Moisture, %**	10.5
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 2219 W. Orange Ave - Townhomes, Anaheim

September 9, 2021

Prepared for:

Ed Lamont
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92280
Elamont@geotekusa.com

Project X Job#: S210908F
Client Job or PO#: 2883-CR Melia Homes

Respectfully Submitted,

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

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: 2219 W. Orange Ave - Townhomes, Anaheim
Client Job Number: 2883-CR Melia Homes
Project X Job Number: S210908F
September 9, 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	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
2883-CR B1	0-5	39.8	0.0040	4.8	0.0005	8,710	5,360	9.3	87	0.02	10.2	5.3	0.03	136.4	47.8	77.1	335.3	8.2	19.9

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)

Results Only Soil Testing for 2219 W. Orange Ave, Townhomes Anaheim

September 15, 2021

Prepared for:

Ed Lamont
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Corona, CA 92280
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Project X Job#: S2109010A
Client Job or PO#: 2883-CR Melia Homes

Respectfully Submitted,

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NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
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REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: 2219 W. Orange Ave, Townhomes Anaheim
Client Job Number: 2883-CR Melia Homes
Project X Job Number: S2109010A
September 15, 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	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
2883-CR B5	0-5	75.5	0.0076	33.0	0.0033	33,500	3,350	8.4	153	<0.01	2.3	3.0	ND	56.4	1.6	21.4	147.8	13.2	1.1

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

APPENDIX C

PERCOLATION DATA SHEETS & PORCHET CALCULATIONS

Proposed Townhomes Project

2219 West Orange Avenue

Anaheim, Orange County, California

Project No. 2883-CR

GEOTEK

Client: Melina Homes
Project: 2219 West Orange Avenue
Project No: 2883-CR
Date: 9/1/2021

Boring No. I-1

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	42
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
$H_F = D_T - D_F$ =	18
$\Delta H = \Delta D = H_O - H_F$ =	2
$H_{avg} = (H_O+H_F)/2$ =	19

I_t = 0.38 **Inches per Hour**

GEOTEK

Client:	**Melia Homes**
Project:	**2219 West Orange Avenue**
Project No:	**2883-CR**
Date:	**9/1/2021**

Boring No. **I-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	46.5
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
$H_F = D_T - D_F$ =	13.5
$\Delta H = \Delta D = H_O - H_F$ =	6.5
$Havg = (H_O+H_F)/2$ =	16.75

I_t = 4.16 **Inches per Hour**

GEOTEK

Client:	**Melina Homes**
Project:	**2219 West Orange Avenue**
Project No:	**2883-CR**
Date:	**9/1/2021**

Boring No. **B-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	467.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	460
Total Test Hole Depth, D_T =	480

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	12.5
$\Delta H = \Delta D = H_O - H_F$ =	7.5
$Havg = (H_O+H_F)/2$ =	16.25

I_t = 1.64 **Inches per Hour**

GEOTEK

PERCOLATION DATA SHEET

Project: 2219 ORANGE AVENUE ANAHEIM, Job No.: 2883-CR.

Test Hole No.: I-1 Tested By: DVG, Date: 9/1,2/2021.

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

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
								PRESOAK 5 GAL 9/1/2021
	849 914	25	60	20	17	3		BEGIN 9/2/2021 1ST 25 MIN.
	916 941	25	60	20	17 1/4	2 3/4		2ND 25 MIN.
	943 1013	30	60	20	16	3 1/4		1ST 30 MIN.
	1015 1045	30	60	20	17 1/4	2 3/4		2ND 30 MIN.
	1047 1117	30	60	20	17 1/4	2 3/4		3RD 30 MIN.
	1119 1149	30	60	20	17 1/2	2 1/2		4TH 30 MIN.
	1151 1221	30	60	20	17 1/2	2 1/2		5TH 30 MIN.
	1223 1253	30	60	20	17 1/2	2 1/2		6TH 30 MIN.
	1255 125	30	60	20	17 3/4	2 1/4		7TH 30 MIN.

PERCOLATION DATA SHEET

Project: 2219 ORANGE AVENUE ANAHEIM, Job No.: 2883-CR.

Test Hole No.: I-1 Tested By: DVG, Date: 9/1,2/2021.

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

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
	127 157	30	60	20	17 3/4	2 1/4		8TH 30 MIN.
	159 229	30	60	20	17 3/4	2 1/4		9TH 30 MIN.
	231 301	30	60	20	18	2		10TH 30 MIN.
	303 333	30	60	20	18	2		11TH 30 MIN.
	335 405	30	60	20	18	2		12TH 30 MIN.

PERCOLATION DATA SHEET

Project: 2219 ORANGE AVENUE ANAHEIM, Job No.: 2883-CR.

Test Hole No.: I-2 Tested By: DVG, Date: 9/1, 2/2021.

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

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
								PRESOAK 5 GAL 9/1/2021
	651		60	20				BEGIN 9/2/2021
	701	10			12 1/2	7 1/2		1ST 25 MIN.
	703		60	20				
	713	10			13	7		2ND 25 MIN.
	715		60	20				
	725	10			13	7		1ST 10 MIN.
	727		60	20				
	737	10			13	7		2ND 10 MIN.
	739		60	20				
	749	10			13	7		3RD 10 MIN.
	751		60	20				
	801	10			13 1/4	6 3/4		4TH 10 MIN.
	803		60	20				
	813	10			13 1/2	6 1/2		5TH 10 MIN.
	815		60	20				
	825	10			13 1/2	6 1/2		6TH 10 MIN.

PERCOLATION DATA SHEET

Project: 2219 ORANGE AVENUE ANAHEIM **Job No.:** 2883-CR

Test Hole No.: B-1 **Tested By:** DVG **Date:** 9/1,2/2021

Depth of Hole As Drilled: 480" **Before Test:** 480" **After Test:** 480"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
								PRESOAK 5 GAL. 9/1/2021
	835 900	25	480	20	11 1/4	8 3/4		BEGIN 9/2/2021 1ST 25 MIN.
	902 927	25	480	20	11 3/4	8 1/4		2ND 25 MIN.
	929 959	30	480	20	10 1/2	9 1/2		1ST 30 MIN.
	1001 1031	30	480	20	10 1/2	9 1/2		2ND 30 MIN.
	1033 1103	30	480	20	10 3/4	9 1/4		3RD 30 MIN.
	1105 1135	30	480	20	10 3/4	9 1/4		4TH 30 MIN.
	1137 1207	30	480	20	11	9		5TH 30 MIN.
	1209 1239	30	480	20	11	9		6TH 30 MIN.
	1241 111	30	480	20	11 1/4	8 3/4		7TH 30 MIN.

PERCOLATION DATA SHEET

Project: 2219 ORANGE AVENUE ANAHEIM, **Job No.:** 2883 - CR.

Test Hole No.: B-1 **Tested By:** DVG, **Date:** 9/1,2/2021.

Depth of Hole As Drilled: 480" **Before Test:** 480" **After Test:** 480".

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
	113		480	20				
	143	30			11 1/2	8 1/2		8TH 30 MIN.
	145		480	20				
	215	30			11 3/4	8 1/4		9TH 30 MIN.
	217		480	20				
	247	30			12	8		10TH 30 MIN.
	249		480	20				
	319	30			12 1/4	7 3/4		11 TH 30 MIN.
	321		480	20				
	351	30			12 1/2	7 1/2		12TH 30 MIN.

APPENDIX D

GENERAL GRADING GUIDELINES

Proposed Townhomes Project

2219 West Orange Avenue

Anaheim, Orange County, California

Project No. 2883-CR

GEOTEK

GENERAL GRADING GUIDELINES

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

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the 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 our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

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

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

Site Clearing

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

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and artificial should be removed unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.

Fill Placement

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

are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.

6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

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

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

JOB SAFETY

General

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

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

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

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

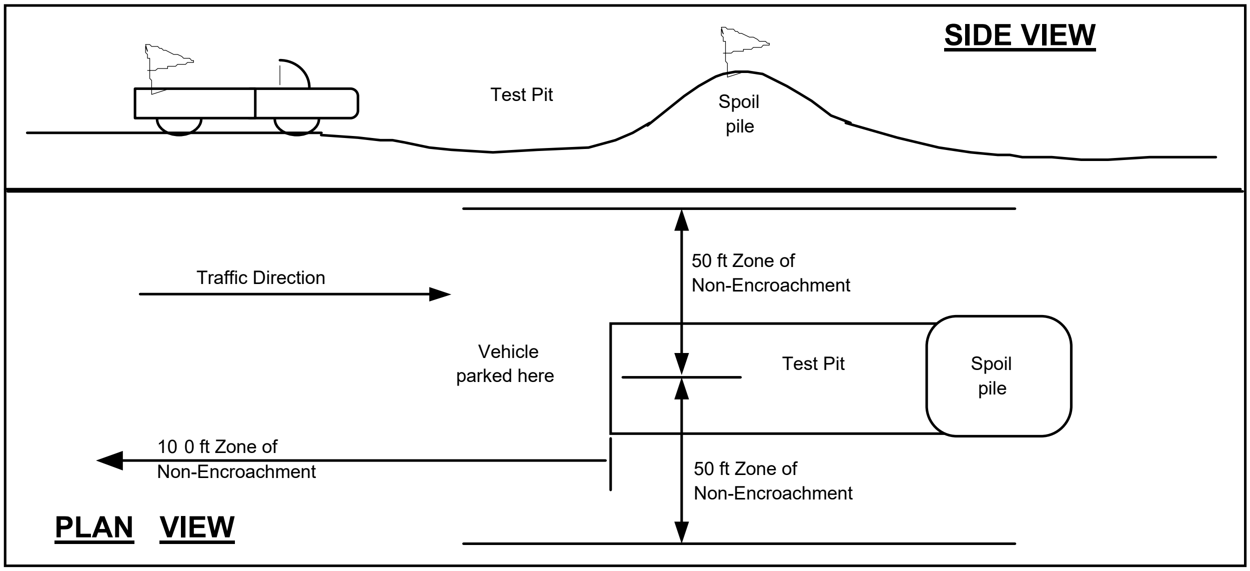
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

4. displays any other evidence of any unsafe conditions regardless of depth.

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

Procedures

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

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

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

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