UPDATED GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED TOWNHOME RESIDENTIAL DEVELOPMENT
2375 NORTH TUSTIN STREET
ORANGE, ORANGE COUNTY, CALIFORNIA

PREPARED FOR

MELIA HOMES
9860 IRVINE CENTER DRIVE
IRVINE, CALIFORNIA 92618

PREPARED BY

GEOTEK, INC.
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CORONA, CALIFORNIA 92878

PROJECT NO. 4150-CR **DECEMBER 30, 2024**

GEOTEK

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December 30, 2024
Project No. 4150-CR

Melia Homes
9860 Irvine Center Drive
Irvine, California 92618

Attention: Mr. Chad Brown,
Vice President of Planning & Development

Subject: **Updated Geotechnical and Infiltration Evaluation**
Proposed Townhome Residential Development
2375 North Tustin Street
Orange, Orange County, California

Dear Mr. Brown:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Updated Geotechnical and Infiltration Evaluation for a Proposed Townhome Residential Development that will be constructed at 2375 North Tustin Street, in the City of Orange, Orange County, California. This report presents a discussion of GeoTek's evaluation and provides preliminary geotechnical recommendations for site preparation, foundation design, preliminary infiltration rates and construction of proposed site improvements.

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 site development.

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

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the site geotechnical conditions for a proposed townhome residential development to be constructed at 2375 North Tustin Street, in the City of Orange, Orange County, California, as outlined in GeoTek's proposal P-1105024r-CR dated November 14, 2024. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data and general information pertinent to the site,
- Review of a Geotechnical Investigation previously prepared for this site by Southern California Geotechnical, Inc. (SCGI, 2005), as well as additional geotechnical reports issued by SCGI (SCGI, 2007) for the subject site. Select information contained in these reports was used in the preparation of this report,
- A site reconnaissance,
- Advancement of three (3) exploratory borings to depths ranging between about 10 to 25.2 feet below existing grades,
- Infiltration testing of two (2) test borings at depths of about ten (10) feet each within the area of the proposed project stormwater management facility (BMP) in the southwestern and southeastern portions of the site,
- Review and evaluation of site seismicity,
- Compilation of this Updated Geotechnical and Infiltration Evaluation which presents GeoTek's findings, preliminary recommendations and a general summary of pertinent geotechnical conditions relevant for site development.

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

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The irregular-shaped project site is located at 2375 North Tustin Street in a mixed-use residential/commercial area of Orange, California. The project site is comprised of one (1) parcel of land totaling approximately 4.32-acres in size, located on the east side of Tustin Street, on the south side of East Heim Avenue, in the City of Orange, Orange County, California (see Figure 1, Site Location Map). California State Highway 55 (Costa Mesa freeway) is located adjacent to the eastern side of the project but does not provide direct access as it is separated from the site by a sound wall. The site is fenced/block walled along most of the site perimeter with access only available from Tustin Street, a paved and improved roadway adjacent to the western boundary of the site.

The site is bordered to the south by an existing mobile home park, a multi-family residential development and a commercial development. A munti-family residential development and commercial development border the proejct site to the north. The site is currently occupied by an existing approximate 45,000 square foot one-story retail building (Best Buy) and adjacent parking/drive lane/hardscape/landscape improvements. Numerous utility lines, light poles, loading docks, domestic trees, and associated improvements are present on the proejct site.

The site can be considered as having a relatively flat to gently sloping topography (see Figure 1). Elevations range from approximately 271 feet above mean sea level (amsl) in the northeastern corner to about 263 feet amsl in the southwest corner. Drainage is generally directed in a southwesterly direction by sheetflow.

2.2 PROPOSED DEVELOPMENT

Based upon review of *Site Plan - A*, prepared by Summa Architecture and dated March 25, 2024 (see Figure 3, Project Plan), site development for the project will consist of 84 three-story row townhomes in multi-family structures (see Figure 3, Project Plan). The development will also include a recreation area, as well as stormwater disposal facilities, open space, perimeter/retaining walls, associated interior parking/driveways, underground utilities and other typical tract improvements.

It is anticipated that the residential structures will be of wood frame construction, supported by a post-tensioned (PT) slab foundation system. Although structural loading information was not

provided, GeoTek has assumed the structures will exert relatively light to moderate foundation loads. Maximum column and wall loads on the order of 100 kips and 3.5 kips per lineal foot, respectively, are estimated for the proposed structures. 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.

On-site storm water disposal is proposed by means of underground disposal systems to be located in the southwestern and southeastern portions of the site. Cut and fills of less than approximately 5 feet in height, not including any recommended remedial grading are anticipated for site development. Perimeter/retaining walls are anticipated. Sewage disposal is anticipated to be be provided by a public sewer system. If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 CURRENT FIELD EXPLORATION

The current field exploration by GeoTek for this evaluation was conducted on December 3, 2024. For the geotechnical portion of the investigation, one (1) exploratory boring (Boring B-1) was excavated with a hollow-stem auger drill rig to a depth of about 25.3 feet below the existing ground surface at the boring location. For the infiltration part of the study, two (2) test borings were excavated to a depth of approximately ten (10) feet each below the existing ground location within the proposed stormwater management chambers (BMP) in the southwestern and southeastern portions of the site.

A hollow-stem auger with an outside diameter of approximately 8.0 inches was utilized to conduct the borings. The inside diameter of the auger was approximately 4.5 inches. A geologist from GeoTek, Inc. logged the exploratory borings and obtained soil samples from within the borings. The boring and infiltration test locations are presented on Figure 2. The logs of the exploratory borings are included in Appendix A.

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

In the geotechnical boring, relatively undisturbed soil samples were recovered at various intervals with a California sampler. The California sampler is a 2.9-inch outside diameter, 2.5-inch inside diameter, split barrel sampler lined with brass rings. The sampler is approximately 18 inches in length. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping approximately 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.

Infiltration Testing

Two (2) borings (Borings I-1 and I-2) were excavated on the project site to evaluate preliminary infiltration rates for the proposed stormwater management facilities to be located in the southwestern and southeastern portions of the site. The boring diameter was approximately eight inches. Percolation testing was performed within the lower approximately 24 inches in the borings by a representative of GeoTek, in general conformance with the Boring Percolation Test Procedure outlined in the Technical Guidance Document Appendix VII (Orange County, 2013). As required, the percolation rates were corrected to account for discharge of water from both the sides and bottom of the borings. This correction was done using the Porchet Method, obtaining the infiltration rates tabulated below:

SUMMARY OF RESULTS		
Boring	Observed Infiltration Rate, $K_{observed}$ (inches per hour)	Measured Infiltration Rate, $K_{measured}$ (inches per hour)
I-1	7.7	3.86
I-2	1.1	0.57

As recommended by the referenced Technical Guidance Document (Orange County, 2013), a factor of safety of 2.0 was applied to the observed (field) rates to estimate measured infiltration rates. As shown on the table above, poor to favorable infiltration rates (0.57 to 3.86 inches per hour) were encountered at the site. Detailed infiltration test data is included in Appendix D. Stormwater BMP systems should be setback from foundations and improvements as required by the regulatory agency or jurisdiction.

It should be noted that the infiltration rates provided above were performed in relatively undisturbed on-site soil. 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 the 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.

Representatives of GeoTek should observe the soils exposed at the bottom of the stormwater management basin during construction/earthwork operations to confirm suitability and that the conditions exposed are as anticipated for the proposed stormwater disposal basin.

3.2 PREVIOUS FIELD EXPLORATION

Field explorations were previously performed at this site by Southern California Geotechnical, Inc. (SCGI) in February, 2005. The subsurface exploration program conducted by SCGI occurred on February 9 and 23, and included the excavation of five (5) exploratory borings (B-1 through B-5) to a maximum depth of 32 feet below existing grades. The approximate locations of the SCGI excavations for the project are presented on the *Boring Location Plan* (Plate 2) attached to the referenced SCGI report dated March 29, 2005, prepared by Southern California Geotechnical, Inc. (SCGI, 2005). This plan is included in Appendix C of this report. The logs of the exploratory borings performed by SCGI are also included in Appendix C of this report.

3.3 LABORATORY TESTING

Laboratory testing was previously performed by Southern California Geotechnical, Inc. as indicated in their report dated March 29, 2005 (SCGI, 2005). This laboratory testing included in-situ dry density and moisture content, expansion index, direct shear, consolidation, and soluble sulfate testing. The results of this laboratory testing are presented in Appendix C of this report.

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

4. PREVIOUS REPORT REVIEW

Geotechnical Investigation (Southern California Geotechnical, Inc., 2005)

SCGI issued the referenced *Geotechnical Investigation* for the project on March 29, 2005. SCGI stated that detailed site plans for the proposed development had not been prepared at the time of their field exploration. SCGI understood that the proposed project would consist of the demolition of an existing site structure (Brunswick Orange Bowl) and construction of several two and three-story structures. Field exploration was conducted by drilling and sampling five (5) hollow stem auger borings to a maximum depth of approximately 32 below existing grades. Boring B-1 was terminated at approximately 32 feet due to auger refusal on gravel and cobbles.

SCGI reported that artificial fill was encountered beneath existing pavement sections at Boring B-5 and extended to a depth of 3± feet below existing grade. SCGI stated that additional soils classified as possible fill were encountered in all of the borings and extended to depths of up to 5.5± feet. These fill soils consisted of stiff to very stiff clays with some fine to coarse gravel (CL soil type based upon the Unified Soil Classification System).

Native soils (alluvium) were reported by SCGI to underlie the artificial fills at the site. The alluvium encountered to depths of 5.5± to 12± feet generally consisted of very stiff to hard silty clays with tract to some fine gravel content (CL soil type). Beneath these materials, the borings generally encountered interbedded zones of dense to very dense silty sands and very stiff to hard silty clays (SM and CL soil types) with varying amounts of fine to coarse gravel content, extending to depths of at least 32± feet (the deepest depth explored).

SCGI stated that the site soils possess a high expansion potential (EI = 108).

SCGI stated that groundwater was not encountered in their borings to a maximum depth of approximately 32 feet below existing site grades (Boring B-1). SCGI stated that based upon the Seismic Hazard Zone Map published by the Site of California, the project is not mapped as having a potential for seismic induced liquefaction. SCGI stated that based upon the soil encountered on the site and the depth to groundwater liquefaction is not considered to be a significant design concern for the project.

SCGI stated that remedial grading should be performed within the proposed building areas in order to remove the existing fill and possible fill soils and a portion of the near-surface native soils, as well as any soils disturbed by the demolition procedures. SCGI stated that that overexcavation was to extend to a depth of at least 3 feet below proposed pad grade and to a

depth of 3 feet below existing grade or to the fill/native content, or to a depth of 3 feet below the foundation bearing grade, whichever is greater, within the building areas.

SCGI provided seismic design parameters for the project structures based on the 1997 Uniform Building Code (UBC).

SCGI stated that shrinkage of reworked materials should be in the range of 8 to 10 percent. SCGI estimated subsidence to be on the order of 0.1 feet.

SCGI provided geotechnical recommendations for shallow foundations and a conventional slab on grade system for the project structures, based on a "High" (90-120) expansion index laboratory testing. SCGI also reported the on-site soils as having a "negligible" water soluble sulfate content. No discussion regarding the potential corrosivity of the site soils and buried ferrous metals was provided.

SCGI provided geotechnical recommendations for site retaining walls for the project. Preliminary pavement design recommendations for the project were also provided and were based on an assumed R-Value of 10.

No percolation testing was conducted for the referenced report dated March 29, 2005.

Geotechnical Update and Plan Review (Southern California Geotechnical, Inc., 2005)

SCGI issued the referenced *Geotechnical Update and Plan Review* for the project on October 17, 2007. SCGI stated that the architectural site plan and conceptual grading plan indicated the proposed development will consist of a 45,750 square foot Best Buy store and a 1,7000 square foot retail outlot pad.

SCGI stated that based upon review of the submitted plans and a recent site reconnaissance, it was their opinion that the proposed development was suitable from a geotechnical standpoint, and the referenced report was still applicable.

SCGI provided additional recommendations for new asphalt concrete (AC) and Portland Cement Concrete (PCC) pavement sections for the proposed Best Buy development.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 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. Basically, it extends roughly 975 miles from the north and extends from the Transverse Ranges geomorphic province to the tip of Baja California, from north to south. 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 in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specifically to the subject property, Morton, D.M., and Miller, F.K., (2006) map the site to be underlain by old alluvial fan deposits (late to middle Pleistocene age). Additionally, the nearest known active fault to the site is the Elsinore fault (Whittier Section) located approximately 6.4 miles to the northeast.

5.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the soils encountered at the site as reported by SCGI is presented in the following sections and supplemented by the current GeoTek exploration findings. Based on the previous and current field explorations, the area investigated has artificial fill which transitions into deeper alluvium.

5.2.1 Artificial Fill

SCGI reported that artificial fill was encountered beneath existing pavement sections at Boring B-5 and extended to a depth of 3± feet below existing grade prior to development of the site. SCGI stated that additional soils classified as possible fill were encountered in all of the borings and extended to depths of up to 5.5± feet. These fill soils consisted of stiff to very stiff clays with some fine to coarse gravel (CL soil type based upon the Unified Soil Classification System). Compacted fill is anticipated to be present within the existing building area and pavement areas.

Asphalt concrete pavement sections, consisting of asphalt concrete (AC) and crushed miscellaneous base (CBB), were encountered at all of the boring locations during GeoTek's recent site exploration.

5.2.2 Alluvium

SCGI reported that mostly clayey alluvium was encountered to depths of 5.5± to 12± feet. This alluvium was stated by SCGI to generally consist of very stiff to hard silty clays with tract to some fine gravel content (CL soil type). Beneath these materials, SCGI state that the borings generally encountered interbedded zones of dense to very dense silty sands and very stiff to hard silty clays (SM and CL soil types) with varying amounts of fine to coarse gravel content, extending to depths of at least 32± feet (the deepest depth explored). SCGI stated that the site soils possess a high expansion potential (EI = 108).

GeoTek's recent exploration also encountered alluvium below the existing asphalt concrete pavement sections to the maximum depth explored of approximately 26.5 feet below existing site grades at the boring locations. The alluvium consisted of interbedded layers of clay, clayey sand, silty sands and fine to coarse gravel CL, SC, SM and GP soil types).

SCGI (2005) and GeoTek's current laboratory testing indicated that the near surface soils exhibit a "Medium" (51≤EI≤90) to "High" (91≤EI≤120) Expansion Index when tested in accordance with ASTM D 4829. Based on the SCGI and current GeoTek laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent. The results of the recent testing performed by GeoTek indicate that the soil samples were considered "highly corrosive" to buried ferrous metals, based on the guidelines provided in *Corrosion Basics: An Introduction* (Roberge, 2005).

5.3 SURFACE AND GROUNDWATER

5.3.1 Surface Water

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

5.3.2 Groundwater

Groundwater was not encountered in the previous site borings conducted by SCGI to a maximum depth of about 32 feet (Boring B-1) at the time of drilling or the current GeoTek borings to a maximum depth of about 26.5 feet. The California Water Data Library shows that the depth to groundwater in the site region is in excess of 40 feet. The depth of historic groundwater at the site is estimated to be greater than 40 feet below existing grades.

Based on the above, groundwater is not anticipated to be a factor during the site grading. However, seasonal perched groundwater may be encountered during grading within portions of the site.

5.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced liquefaction (CGS, 1997). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced landsliding. The nearest known active fault to the site is the Elsinore fault (Whittier Section) located approximately 6.4 miles to the north.

5.4.1 Seismic Design Parameters

The site is located at approximately 33.8279 degrees Latitude and -117.8340 degrees Longitude. Based on the relatively dense soil conditions encountered across the site, a Site Class "D" is considered appropriate for this project. Site spectral accelerations (Sa and S1), 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.

The following seismic design parameters, based on the 2015 National Earthquake Hazards Reduction Program (NEHRP)/ASCE 7-16/2022 CBC, are presented below:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.0508g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.534g
Site Coefficient for Site Class "D", F_a	1.0
Site Coefficient for Site Class "D", F_v	1.766**
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.508
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	1.415g*
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.005
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.944
Site Modified Peak Ground Acceleration, PGA_M	0.698g
Seismic Design Category	D

*ASCE 7-16 Supplement 3 Section 11.4.8 requires a ground motion hazard analysis for structures on Site Class "D" for values of S1 greater than or equal to 0.2g. However, a ground motion hazard analysis is not required where the values of SM1 and SD1 are increased by 50%. The SM1 and SD1 values shown above already include the 50% increase, so that exception can be obtained.

**ASCE 7-16 Supplement 3 Section 11.4.8 indicates that the value of Fv should only be used for calculation of Ts, determination of Seismic Design Category, linear interpolation for intermediate values of S1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of SD1.

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.

5.5 LIQUEFACTION AND SEISMIC SETTLEMENT

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

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

The project site is not located within an area mapped by the State of California for liquefaction potential. Due to the lack of shallow groundwater, the liquefaction hazard at the site is nil.

Due to the relatively dense/clayey nature of the subsurface soils and anticipated site grading, the seismic settlement potential of the site soils is anticipated to be minimal.

5.6 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instability at this site was not observed during the field investigation. Due to the relatively flat site topography and relatively low liquefaction hazard potential, the potential for seismic-induced lateral displacements (i.e., "lateral spread") and slope instability hazards are considered negligible.

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.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

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

The near-surface upper site soils are anticipated to contain undocumented fill. In addition, removal of the existing site improvements and utility lines is anticipated to disturb the upper soils. Therefore, the upper site soils are not considered suitable for foundation loading in their current condition. Remedial grading, consisting of overexcavation and recompaction of the upper site soils, is recommended to provide a uniform bearing for any proposed structures.

The site soils were found to exhibit a "Medium" (51≤EI≤90) to "High" (91≤EI≤120) Expansion Index when tested in accordance with ASTM D 4829. As noted in the proposed development section of this report, a post-tensioned (PT) slab foundation system will be used to support the

proposed structures. This foundation system should be designed to accommodate the site soil expansion potential and the total and differential settlement potentials provided in this report.

6.2 EARTHWORK CONSIDERATIONS

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

6.2.1 Site Clearing and Demolition

In areas of planned grading and improvements, the locations of existing utilities should be determined. Existing utilities should be relocated or abandoned. The site should be cleared of existing structures, foundations, abandoned utilities, pavements, slabs, trees, vegetation and other deleterious materials. Demolition of existing buildings/structures should include removal of all shallow foundations, floor slabs and any below-grade construction. Debris should be properly disposed of off-site. Voids resulting from site clearing should be backfilled with engineered fill.

Portland Cement Concrete (PCC) generated from the demolition of existing site improvements may be incorporated into site fills provided the following guidelines are implemented: 1) concrete should be free of rebar or other deleterious materials and should be broken down to a maximum dimension of six (6) inches; 2) concrete should not be placed within three (3) feet of finish grade in the building pad areas or within one (1) foot of subgrade elevations in the street/drive areas; 3) concrete should be distributed in the fill and should not be "nested" or placed in concentrated pockets.

Asphalt concrete (AC) should not be used in fill for building pads or landscape areas. This material may be used in pavement areas of the project or should be removed from the site.

Alternatively, existing PCC concrete, AC and aggregate base may be pulverized and reused as Crushed Miscellaneous Base (CMB) in new pavement sections. These pulverized materials should not contain any detrimental or deleterious materials (reinforcing steel, nails, metal, glass, plastic rubber, wood, organic matter, friable materials, elongated or laminated pieces, etc.). Any materials to be used as CMB should be tested to confirm conformance with appropriate Standard Specifications for Public Works Construction ("Greenbook") specifications (Section 200-2.4).

6.2.2 Site Preparation

Due to the non-uniform condition of the near-surface upper site soils, presence of undocumented fill/soils and expected disturbance form the removal of the existing site improvements, it is recommended that the soils be removed beneath the planned building footprints to a depth of at least five (5) feet below existing grades, or three (3) feet below the base of the proposed foundations, whichever is greater. It should be noted that the above recommendations are recommended throughout the majority of the site; however, deeper artificial fills may be present within the site. These artificial fills will require deeper removals accordingly. All undocumented fill soils shall be removed as part of remedial grading. The lateral extent of this recommended over-excavation should extend at least five (5) feet beyond the building limits, where obtainable.

Removal bottoms should be relatively uniform in soil type and not adversely 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. Deeper removals may be required and should be determined during site grading by a GeoTek representative.

Following site clearing operations, over-excavation and lowering of site grades, where necessary, it is recommended that the exposed subgrade soils beneath all surface improvements be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the geotechnical engineering representative. All soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative.

Following proof rolling and removal of any unsuitable bearing soil, the exposed subgrade should be scarified to a depth of about 12 inches, be moisture conditioned to at least 2 percent 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.

6.2.3 Pavement Areas

Undocumented fill should be removed below proposed pavement areas. If no undocumented fill is encountered or is relatively shallow, the natural soils should be overexcavated to a depth of 12 inches below existing grade or 12 inches below proposed finished grade, whichever is deeper. Finished grade is defined as the top of the subgrade. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least 2 percent above the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. Pavement subgrade should be firm and unyielding upon proof rolling with heavy construction equipment prior to placement of aggregate base material.

6.2.4 Hardscape Areas

Undocumented fill should be removed below hardscape areas. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least 2 percent above the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. Due to the soil expansion potential, a minimum of four inches of gravel (1-inch minus) or crushed aggregate base should be placed below all exterior hardscape improvements.

6.2.5 Preparation of Excavation Bottoms

A representative of this firm should observe the bottom of all excavations. Upon approval, the exposed soils should be scarified to a depth of approximately eight inches, moistened to at least 2 percent above the optimum moisture content and compacted to a minimum relative compaction of at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

6.2.6 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Engineered fill should be placed in loose lifts with a thickness of eight inches or less and moisture conditioned to at least two percent above the optimum moisture content.

Below and within the proposed building area, engineered fill should be moisture conditioned to at least 2 percent above optimum moisture content and be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures if a shallow foundation system is used.

Below other structural elements, such as hardscape areas and walls independent of the building, engineered fill should be moisture conditioned to at least 2 percent above optimum moisture content and be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

6.2.7 Excavation Characteristics

Excavation of the on-site alluvial soils is expected to be feasible utilizing heavy-duty grading equipment in good operating condition. All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the on-site materials should be stable at 1.5:1 (horizontal: vertical) inclinations for cuts less than five feet in height.

6.2.8 Slot-Cut Excavations

Assuming that any existing block walls along existing site boundaries are screen walls (imparting minimal loads), GeoTek evaluated the use of slot-cut excavations adjacent to these improvements in order to facilitate the required remedial grading for construction of proposed site walls. Preliminary analyses by GeoTek using a soil cohesion of about 250 psf along a friction angle of approximately 24 degrees, an assumed slot-cut height of about five feet, and an assumed footing surcharge of approximately 1,000 psf yielded an approximate slot-cut width of six feet. Wider slot cuts may be feasible if favorable conditions (i.e. denser soils with significant cohesion) are exposed in the site excavations.

Slot-cut excavations should follow an A-B-C sequence. Each "A" section should be excavated to the required depth and then backfilled with compacted soil (or wall section constructed). Following completion of the "A" sections, the "B" sections are then excavated and backfilled/constructed and then all "C" sections are completed in a similar manner. A GeoTek representative should observe that the slot-cuts are being performed in conformance with these recommendations.

6.2.9 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including orchard tree removal, shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 10 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.1 foot may be anticipated for the underlying soils.

6.2.10 Import Soils

Import soils should have a "Very Low" (0-20) to "Low" (21-50) Expansion Index. GeoTek, Inc. also recommends that the proposed import soils be tested for expansion and corrosivity potential. GeoTek, Inc. should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

6.3 DESIGN RECOMMENDATIONS

6.3.1 Foundation Design

The soils are classified as having a "Medium" (51≤EI≤90) to "High" (91≤EI≤120) Expansion Index in accordance with ASTM D 4829. However, verification testing should be performed after site remedial grading.

The foundation elements for the proposed structures should bear entirely in engineered fill soils and should be designed in accordance with the 2022 CBC. Given the site soil expansion potential at the site, residential buildings should be supported by post-tension (PT) slab foundation systems. Geotechnical parameters for post-tension slab design, in general conformance with *Design of Post-Tensioned Slabs-on-Ground*, Third Edition *with 2008 Supplement* (PTI, 2008), are provided below. These recommendations are minimal recommendations and are not intended to supersede the design by the project structural engineer.

DESIGN PARAMETERS FOR POST-TENSION SLABS	
Foundation Design Parameter	Design Value
	"High" Expansion Index (LL≤46; PI≤26; Passing #200 Sieve ≈ 60%; Clay fines ≈ 20%)
Edge Moisture Variation Distance, e_m -Edge Lift (swelling) -Center Lift (shrinkage)	 4.5 ft 8.7 ft
Soil Differential Movement, y_m -Edge Lift (swelling) -Center Lift (shrinkage)	 ≈ 1.07 in ≈ 0.39 in
Exterior Perimeter Beam Embedment	One- and Two-Story – 12 inches* Three-story – 18 inches*
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 120% to a depth of 18 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.

Based upon review, a modulus of subgrade reaction (E_I) of 75 pci may be used in the design of the post-tensioned slab foundation system. It should be noted that this value is based upon standard one foot plate load tests. Depending upon the design methodology and foundation geometry this value may need to be modified.

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

The following criteria for design of foundations are preliminary and should be re-evaluated based on the results of additional laboratory testing of samples obtained near finish pad grade. 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 12 inches wide, and pad footings 24 inches square and 12 inches deep. This allowable soil bearing capacity may be increased by 200 psf for each additional foot of footing depth and 100 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). These bearing values contain a minimum factor of safety of three (3).

The recommended allowable bearing capacity is based on an estimated maximum post-construction static settlement of approximately 1-inch. Static differential settlement of about one-half of the total settlement over a horizontal distance of 40 feet could result. Estimated seismic settlements are anticipated to be minimal. The project structural engineer, foundation engineer, and earth retention structure designer should incorporate these settlement estimates into the design, as appropriate.

The passive earth pressure may be computed as an equivalent fluid having a density of 280 psf per foot of depth, to a maximum earth pressure of 2,800 psf for footings founded on engineered fill or competent native soil. A coefficient of friction between soil and concrete of 0.30 may be used with dead load forces. Passive pressure and frictional resistance may be combined without reduction. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements.

A grade beam, a minimum of 12 inches wide and 18 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

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.

6.3.2 Moisture Vapor and Retarding System

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

The effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures. These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6-mil vapor retarder membrane, a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties such as thickness, composition, strength, and permeability to achieve the desired performance level. Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

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

In addition, the recommendations in this report and 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.

6.3.3 Miscellaneous Foundation Recommendations

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

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

6.3.4 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2022 CBC or City of Orange requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movement and/or differential settlement. If large enough, these movements can compromise the integrity of the improvements.

- The outside top edge of all footings should be set back a minimum of H/3, where H is the slope height, from the face of any descending slope. The setback should be at least five feet and need not exceed 40 feet.
- The bottom of any proposed foundations should be deepened so as to extend below a 1:1 upward projection from the bottom edge of the nearest excavation and the bottom edge of the closest footing.

6.3.5 Soil Corrosivity

Corrosivity test results recently performed by GeoTek are presented in Appendix B. These test results indicate that the on-site soils are "highly corrosive" (1,005 ohm-cm) to buried ferrous metal. This corrosion classification is obtained from "Corrosion Basics: An Introduction," by Pierre R. Roberge, 2nd Edition, 2005. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer.

6.3.6 Soil Sulfate Content

Based on the chemical test results presented in the SCGI report (see Appendix C) and recent testing performed by GeoTek (Appendix B), the sulfate test results on soil samples obtained from the project site indicate a soluble sulfate content of less than 0.1% by weight. Soluble sulfate contents of this level would be in the range of "negligible" (i.e., "S0" exposure classification) in accordance with ACI 318-19. Based on the test results and Table 19.3.1.1 of

ACI 318-19, no special recommendations for concrete are required for this project due to soil sulfate exposure.

6.4 RETAINING AND SCREEN WALL DESIGN AND CONSTRUCTION

6.4.1 General Design Criteria

Retaining wall foundations independent of the building should be embedded a minimum of 18 inches into engineered fill. Retaining wall foundations should be designed in accordance with Section 5.3.1 of this report. Structural requirements may govern and should be evaluated by the project structural engineer. All earth retention structure plans should be reviewed by this office prior to finalization.

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 wall designer. The soil used as backfill behind retaining walls should have a "Very Low" (0-20) Expansion Index and should be compacted to at least 90 percent relative compaction (ASTM D 1557).

In general, cantilever retaining walls which are designed to yield at least 0.001H, where H is equal to the height of the wall to the base of the footing may be designed using the active condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at the 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 structure, should be considered in the design of the retaining walls. Loads applied within a 1:1 (horizontal : vertical) projection from the surcharging structure on the stem of the retaining wall should be considered in the design.

Final selection of the appropriate design parameters should be made by the retaining wall designer based upon the local practices and ordinances, expected structure response, and desired level of conservatism.

6.4.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to six (6) feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. Appropriate fluid unit weights are given below for specific

slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure* (pcf)
Level	50
2:1	85

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

For walls with a retained height greater than six (6) feet, an incremental seismic pressure should be included in the wall design. Where needed, it is recommended that an incremental seismic load for unrestrained walls with level backfill of $10H^2$ [Units: pounds per lineal foot of wall] should be included into the wall design to account for seismic loading conditions, where H is the retained height of the wall. For unrestrained walls with a retained height greater than six (6) feet with backfill of a 2:1 [horizontal: vertical] gradient, a dynamic load increment of $17H^2$ should be included in the wall design. These incremental seismic loads may be assumed to be applied at a point 1/3H above the base of the wall.

6.4.3 Retaining Wall Backfill and Drainage

Wall backfill should include a minimum one-foot-wide section of ¾- to one-inch clean crushed rock or approved equivalent. The rock should be placed immediately adjacent to the back of the wall and extend up from a backdrain to within approximately 12 inches of finish grade. The portion of the rock opposite the back of the wall adjacent to the soil backfill should be covered with a layer of filter fabric comprised of Mirafi 140N or the equivalent. The upper 12 inches of backfill should consist of compacted on-site soil. Backfill placed within the active zone as defined by a 1:1 (h:v) projection from the back of the retaining wall footing up to the retained surface behind the wall should consist of very low expansive soil. The presence of other soils placed within the 1:1 projection will necessitate revision to the parameters provided and modification of wall designs.

The backfill soil should be placed in lifts no greater than eight inches in thickness, moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining

walls. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

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

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.

Walls less than two (2) feet in height do not require backdrains. Walls from two to four feet in height may be drained using localized gravel packs (e.g., approximately 1.5 cubic feet of gravel in a woven plastic bag) behind weep holes at 10 feet maximum spacing. Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

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

6.4.4 Restrained Retaining Walls

Retaining walls that will be restrained prior to placing and compacting backfill material or that have reentrant or male corners, should be designed for an at-rest equivalent fluid pressure of 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.

6.4.5 Additional Retaining Wall Design Considerations

- Retaining and screen wall foundation elements should be designed in accordance with building code setback requirements. A minimum horizontal setback distance of five feet as measured from the top outside edge of the footing to an adjacent slope face is recommended.
- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.

- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in screen walls at horizontal distances not exceeding 20 feet.

6.5 CONCRETE FLATWORK

6.5.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices utilized in construction.

Due to the soil expansion potential, a minimum of four inches of gravel (1-inch minus) or crushed aggregate base should be placed below all exterior hardscape improvements. In addition, reinforcement of exterior slabs with at least 6" x 6" – W2.6 x W2.6 welded wire fabric placed at the middle of the slab should be considered.

Sidewalks may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria will apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs and sidewalks should be pre-saturated to a minimum of 100 percent of the optimum moisture content to a depth of 12 inches.

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

6.5.2 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, Inc. suggests that control joints be placed in two directions and located a distance approximately equal to 24 to 36 times the slab thickness. All concrete flatwork subgrade should be moisture conditioned to at least 120% of optimum moisture content down to a depth of 18" below subgrade prior to placing concrete.

Due to the soil expansion potential, a minimum of four inches of gravel (1-inch minus) or crushed aggregate base should be placed below all exterior hardscape improvements. In addition, reinforcement of exterior slabs with at least 6" x 6" – W2.6 x W2.6 welded wire fabric placed at the middle of the slab should be considered.

6.6 PRELIMINARY PAVEMENT DESIGN

6.6.1 Asphaltic Concrete Pavement

Preliminary pavement thickness design is based on the Caltrans Highway Design Manual (2018). The following preliminary pavement design recommendations are based on assumed Traffic Indexes of 5.0 for automobile parking and 5.5 for interior drives. An R-value of 10 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 section recommendations are provided for the site:

PRELIMINARY MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0 (Local Street)	3.0	9.0
5.5 (Interior Drives)	4.0 5.0	9.0 7.0

Traffic Indices (TIs) used in the pavement design are considered reasonable values for the proposed residential street areas and 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.

Pavement subgrade should be compacted to at least 90 percent relative compaction and 2 percent over optimum, and be firm and unyielding to heavy construction equipment prior to placing aggregate base materials. All base material should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D1557 test procedures. All materials and methods of construction should conform to the requirements of the City of Orange.

6.6.2 Permeable Interlocking Concrete Pavers

Interlocking concrete pavers can be used for this project. The pavers are assumed to be approximately 3.2 inches (80 millimeters) in thickness. Concrete pavers should be underlain by 1.0 inch to 1.5 inches of bedding sand overlying four inches of aggregate base founded on compacted subgrade soils (90 percent relative compaction, at least 2% over optimum moisture content). The aggregate base should be compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

Where possible, the aggregate base should extend beyond the perimeter of the pavers a minimum distance of four inches.

The bedding sand should be placed and lightly moistened and compacted. Since this compaction cannot be tested it should be observed by a representative of this firm.

Historically, paver systems have experienced failures in areas where water has degraded the support characteristics of the underlying base and/or subgrade soils. Since paver systems are permeable and allow transmission of water into the underlying materials, it may be prudent to discuss with the paver designer/manufacturer what methods may be employed to address the issue of potential water introduction into the underlying materials. Underdrain systems, local subgrade reinforcement, tilting the pavement subgrade to drain away from paver areas and not pond, or additional structural elements such as geotextiles can be considered, particularly in high traffic areas and/or low areas where water will tend to collect. The recommendations of the designer/manufacturer should then be implemented in the design and construction of the paver system.

6.6.3 Portland Cement Concrete (PCC) Pavement

For the proposed vehicle parking areas, it is recommended that a minimum of six (6.0) inches of PCC pavement over 12 inches subgrade compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures and at least 2 percent over-optimum be utilized. For vehicle service and access lanes, it is recommended that a minimum of seven (7.0) inches of PCC pavement over 12 inches subgrade compacted to at least 90 percent of maximum dry density be utilized. All pavement subgrade should be firm and unyielding to heavy construction equipment prior to base and PCC placement. This section should also be used in heavy truck traffic areas such as fire lanes, trash dumpster pads and approaches. Due to the soil expansion potential, a minimum of four inches of gravel (1-inch minus) or crushed aggregate base should be placed below all exterior pavement improvements. In addition, reinforcement of exterior slabs with at least 6" x 6" – W2.6 x W2.6 welded wire fabric placed at the middle of the slab should be considered.

Requirements of Section 90 of Caltrans Standard Specifications, and various ACI and ASTM standards regarding mixing and placing concrete should be followed. The PCC pavement should have a minimum modulus of rupture of 500 pounds per square inch, and a minimum 28-day compressive strength of 4,000 pounds per square inch. Concrete should incorporate a 1-inch maximum size aggregate and should be proportioned to achieve a maximum slump of four inches. Instead of increasing the water content, a plasticizing admixture may be utilized to increase the workability of the concrete. The concrete should be properly cured after placement. Concrete should not be placed during hot and windy weather.

Crack control joints should be provided in the transverse direction spaced at horizontal intervals ranging from 24 to 36 times the thickness of the concrete.

6.6.4 Pavement Construction

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

Any aggregate base should consist of crushed rock with an R-Value and gradation in accordance with Crushed Aggregate Base (Section 400-2.4 of the "Greenbook" Regional Supplement Amendments). Any Crushed Miscellaneous Base shall conform to Section 200-2.4 of the Green Book.

Minimum compaction requirements for aggregate base should be 95 percent of maximum dry density as determined by ASTM D 1557 test procedures for aggregate base. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern. The upper 12 inches of subgrade should be moisture-conditioned to at least 2 percent over optimum moisture and be compacted to at least 90 percent relative compaction. The top of the subgrade and aggregate base should be graded to drain to the perimeter of the pavement.

6.7 POST CONSTRUCTION CONSIDERATIONS

6.7.1 Landscape Maintenance and Planting

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

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

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

6.7.2 Drainage

Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings and floor-slabs. Pad drainage should be directed toward approved areas and not be blocked by other improvements. Roof gutters should be installed that will direct the collected water at least 20 feet from the buildings.

6.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

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

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

7. INTENT

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

The scope of this evaluation is limited to the boundaries of the subject property. This review does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope of this evaluation is based on GeoTek's understanding of the project and geotechnical engineering standards normally used on similar projects in this locality.

8. 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 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, and laboratory testing, 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.

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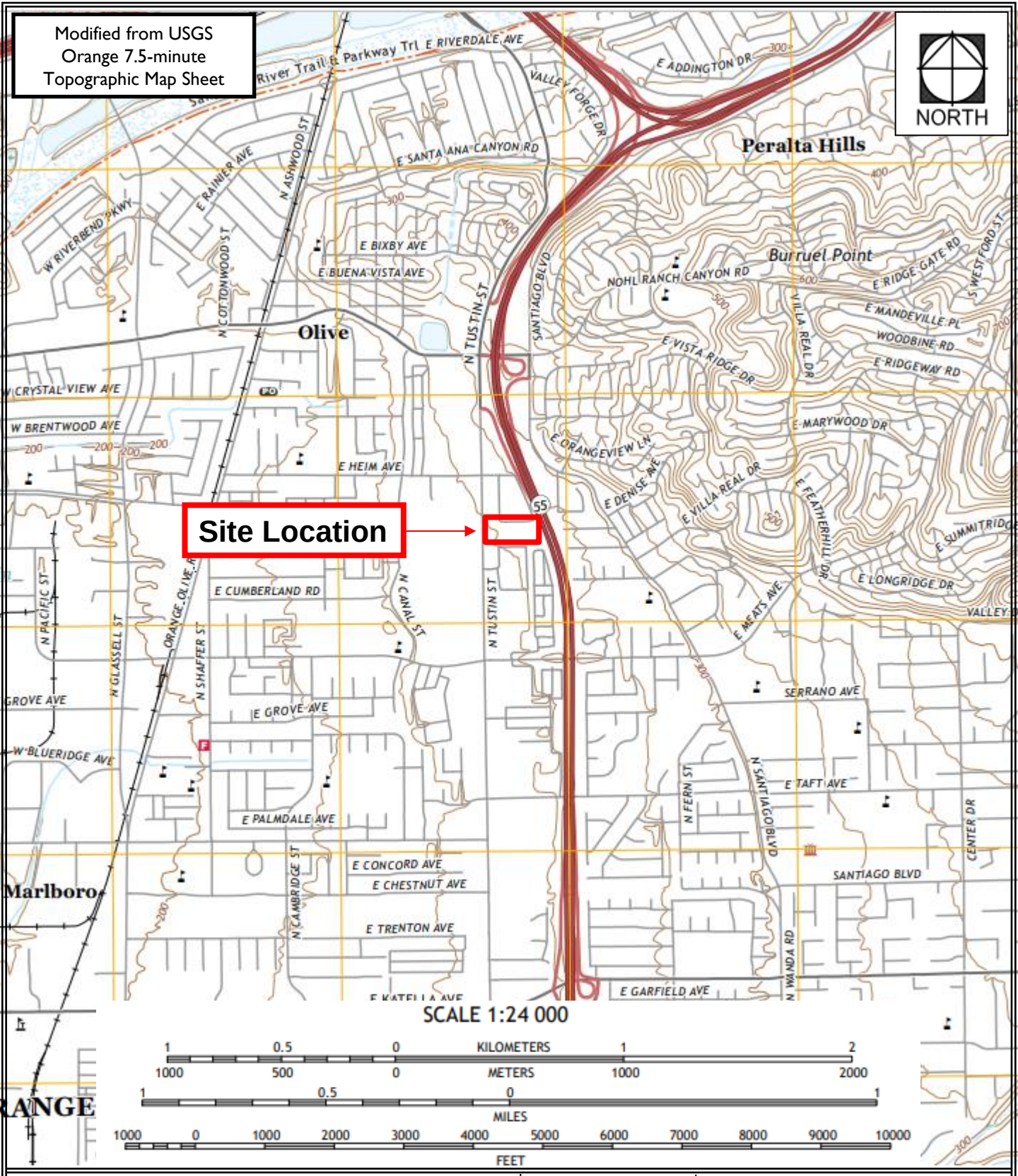


Melia Homes

2375 North Tustin Street

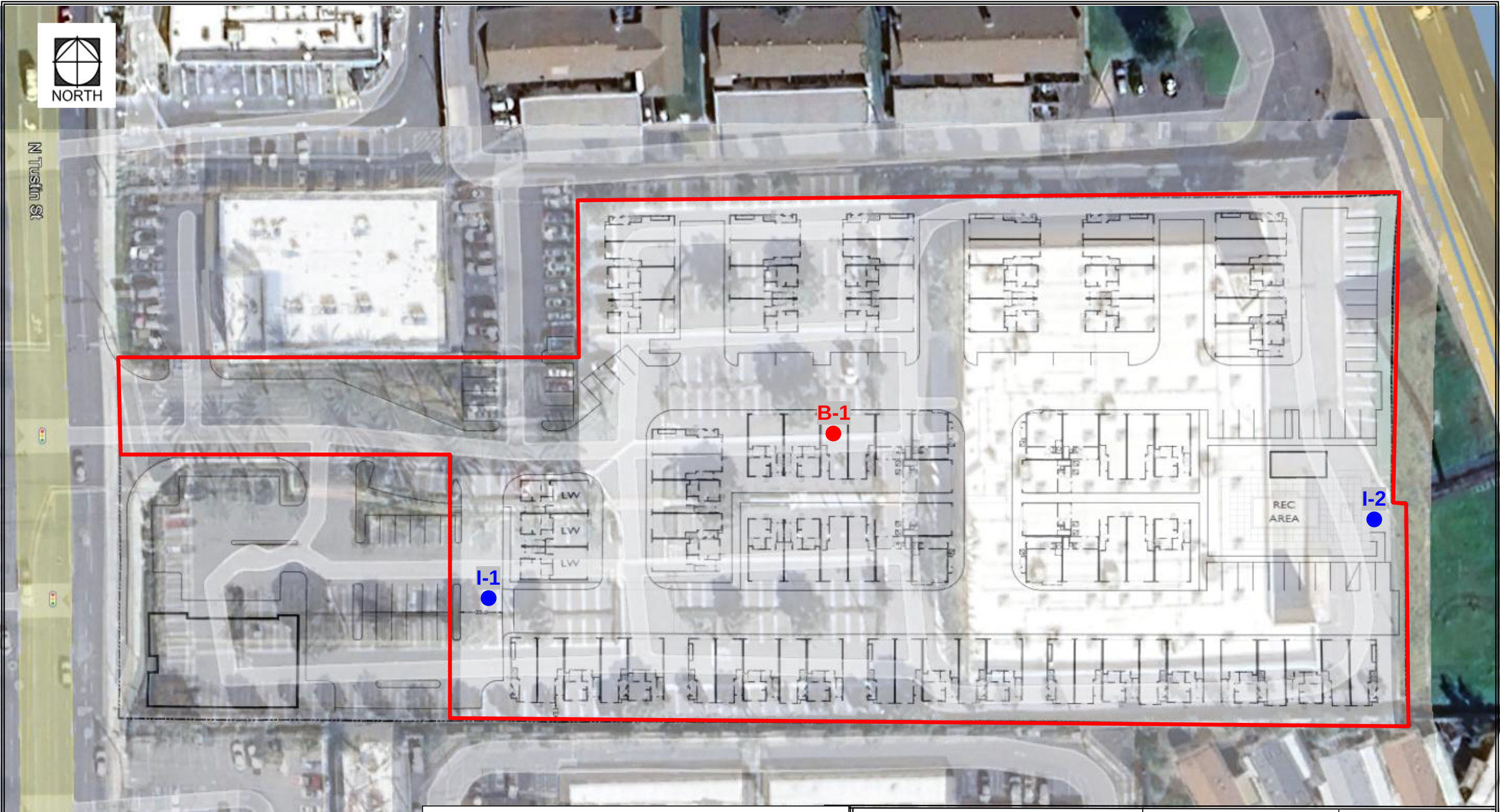
Orange, Orange County, California

Project No. 4150-CR

Figure 1

Site Location Map

GEOTEK



LEGEND
(Locations are Approximate)

Site Boundary

B-1 Geotechnical Boring

I-2 Infiltration Test

Melia Homes
2375 North Tustin Street
Orange, Orange County, California

Project No. 4150-CR

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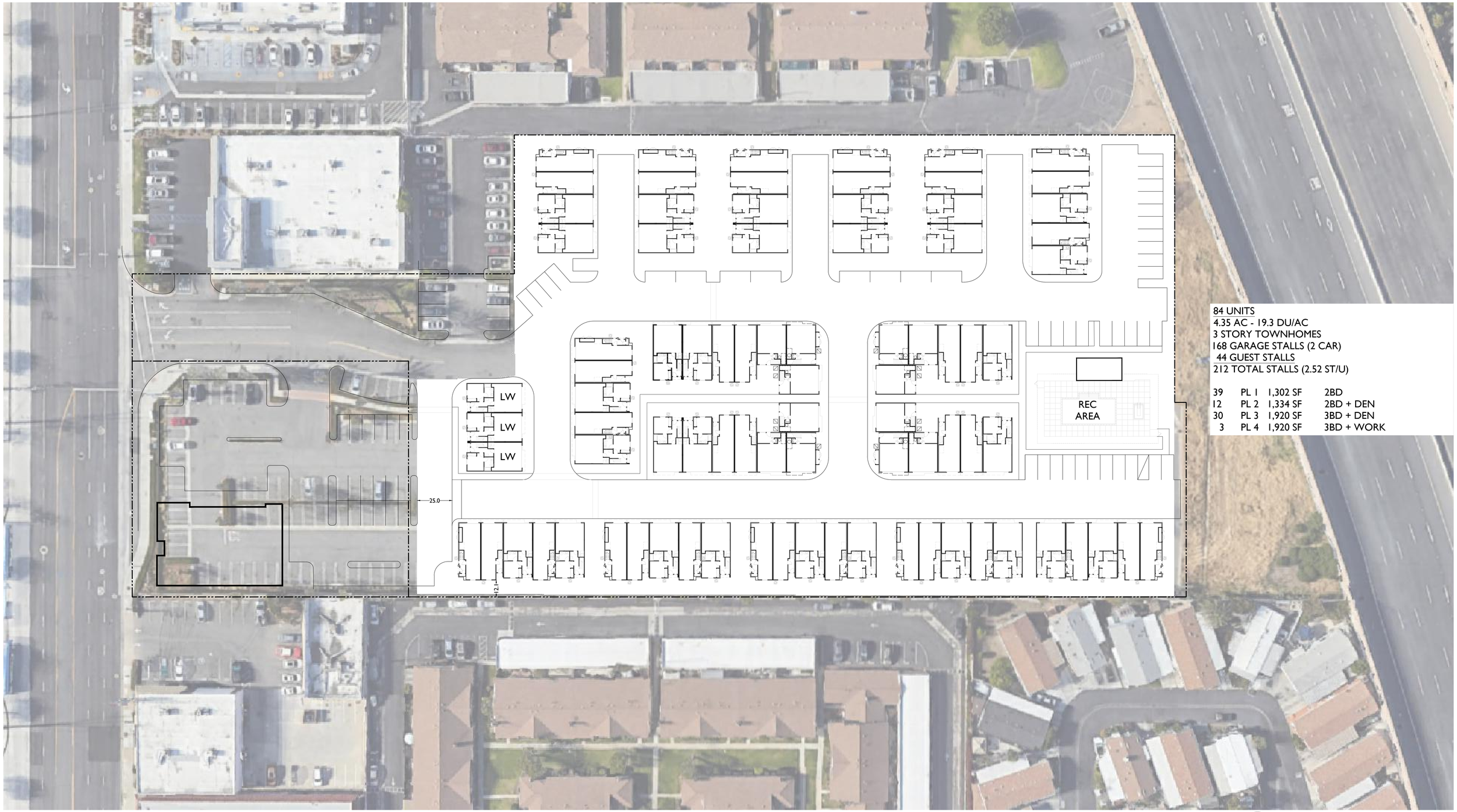
Figure 2

Exploration Location Map



84 UNITS
4.35 AC - 19.3 DU/AC
3 STORY TOWNHOMES
168 GARAGE STALLS (2 CAR)
44 GUEST STALLS
212 TOTAL STALLS (2.52 ST/U)

39	PL 1	1,302 SF	2BD
12	PL 2	1,334 SF	2BD + DEN
30	PL 3	1,920 SF	3BD + DEN
3	PL 4	1,920 SF	3BD + WORK

NORTH

ORANGE

Figure 3

Project Plan

2375 N. TUSTIN ST. - 3 STORY ROW TOWNHOMES

SITE PLAN - A

MELIA HOMES
8951 Research Dr, # 100
Irvine, CA 92618
(949) 759-4367

0 30 60 90

MAR 25, 2024

SUMMA ARCHITECTURE
5256 S. Mission Road, Ste. 404
Bonsall, CA 92003
www.summarch.com p.760.724.1198

SP

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Updated Geotechnical and Infiltration Evaluation
Proposed Townhome Residential Development
2375 North Tustin Street
Orange, Orange County, California
Project No. 4150-CR

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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/TRENCH LOG LEGEND

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

SOILS

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

GEOLOGIC

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

(Additional denotations and symbols are provided on the log of borings/trenches)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	2375 N Tustin St	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Jaime
PROJECT NO.:	4150-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig (CME 75)
LOCATION:	See Boring Location Map			**DATE:**	12/3/2024

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
					3" Asphaltic Concrete over 8" CMB			
		7 11 11		CL	**Alluvium:** Fine sandy silty CLAY, gray, slightly moist, trace asphalt pieces, stiff	16.4	107.7	MD, EI, SH, SR, AL, SA Expansion Index = 88 Liquid Limit = 46 Plastic Limit = 20 PI = 26 60% Passing #200 Sieve
		6 10 15			Silty CLAY, gray, slightly moist, very stiff	21.3	101.6	
5		11			Olive-brown, trace rootlets, hard			
		20 27		SC	Silty clayey f SAND, yellow-brown, slightly moist, dense	19.4	111.4	
		9 15 30		SM	Silty f SAND, yellow-brown, slightly moist, trace clay, trace m-c sand, dense	9.8	125.1	
10		12 14 16			Silty f SAND, light olive-brown, slightly moist, trace clay, medium dense	10.3	117.3	
15		11 17 30		GP	F-c GRAVEL, gray, slightly moist, little f-c sand, dense			
20		50/3			Very dense			
25		50/3						
					BORING TERMINATED AT 25.25 FEET No groundwater encountered Boring backfilled with excavated materials			
30								

LEGEND

Sample type: ---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 Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	2375 N Tustin St	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Jaime
PROJECT NO.:	4150-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig (CME 75)
LOCATION:	See Boring Location Map			**DATE:**	12/3/2024

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
					3" Asphaltic Concrete over 7" CMB			
				CL	**Alluvium:** Silty CLAY, gray-brown, slightly moist			
5								
				GP	F-c GRAVEL, gray, slightly moist, trace f-c sand, trace clay			
10					**BORING TERMINATED AT 10 FEET**			
					No groundwater encountered Boring backfilled with excavated materials			
15								
20								
25								
30								

LEGEND

Sample type: ---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 Inc.	**LOGGED BY:**	KLP
PROJECT NAME:	2375 N Tustin St	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Adrian
PROJECT NO.:	4150-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	Truck rig (Simco 2800)
LOCATION:	See Boring Location Map			**DATE:**	12/3/2024

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
					5.5" Asphaltic Concrete over 12" CMB			
				CL	**Alluvium:** Silty CLAY, gray-brown, slightly moist			
5								
				GP	F-c GRAVEL, gray, slightly moist, trace f-c sand, little clay			
10					**BORING TERMINATED AT 10 FEET**			
					No groundwater encountered Boring backfilled with excavated materials			
15								
20								
25								
30								

LEGEND

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

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

APPENDIX B

LABORATORY TEST RESULTS

Updated Geotechnical and Infiltration Evaluation
Proposed Townhome Residential Development
2375 North Tustin Street
Orange, Orange County, California

GEOTEK

SUMMARY OF LABORATORY TESTING

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

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

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

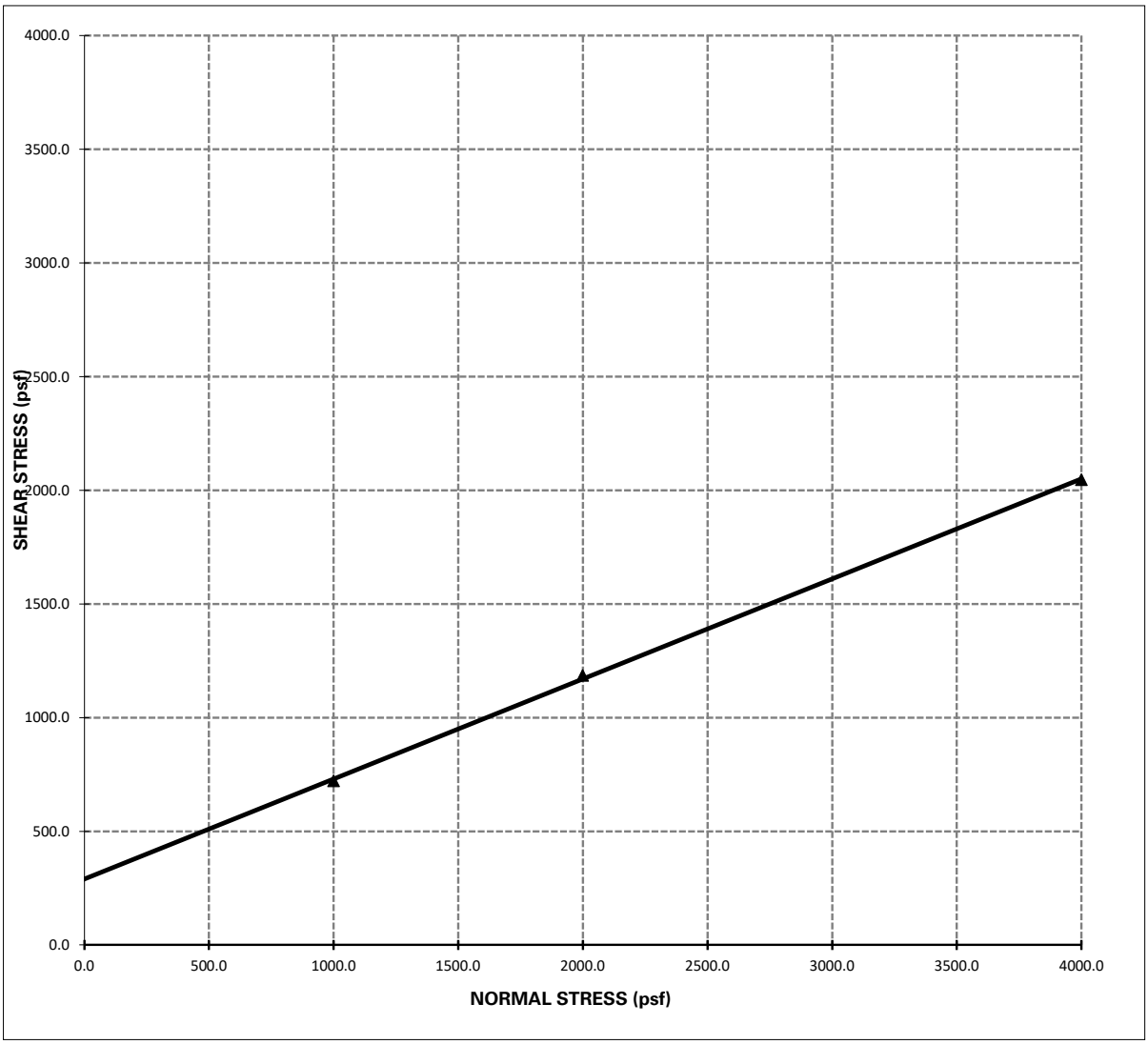
In Situ Moisture Content and Unit Weight
The field moisture content was measured in the laboratory on selected samples collected during the field investigation. The field moisture content is determined as a percentage of the dry unit weight. The dry density was measured in the laboratory on selected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

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

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

DIRECT SHEAR TEST

Project Name:	2375 N. Tustin St.	***Sample Location:***	B-1 @ 0-5 Feet
Project Number:	4150-CR	***Date Tested:***	12/12/2024



Shear Strength: Φ = **24 °** , **C = 291 psf**

Notes:
1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

GeoTek - Corona
1548 N. Maple St.
Corona, CA 92880
Phone: (951) 710-1160
Fax: (951) 710-1167

Report No: PTR:24-00241-S01

Proctor Report

Client: Melia Homes
9860 Irvine Center Drive
Irvine CA 92618

CC:

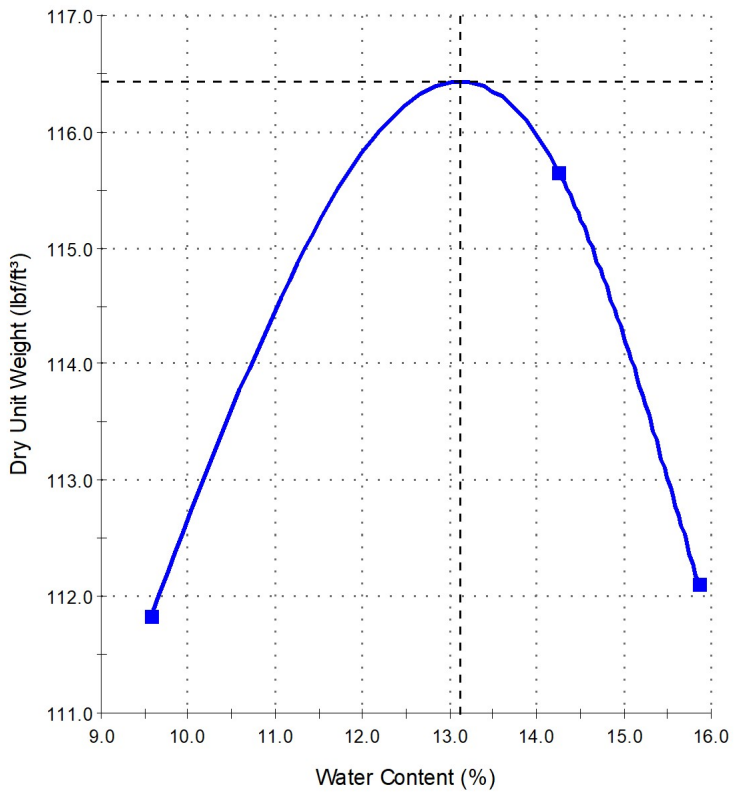
Project: 4150-CR
Multi-Family Residential Development - Updated Geotechnical & Infiltration Eval

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID: 24-00241-S01
Date Sampled: 12/3/2024
Sampled By:
Material: CLAY
Location: B-1 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lbf/ft³):	**116.4**
Optimum Water Content (%):	**13.1**
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	5
Passing Sieve No 4 (4.75mm) (%):	95
Tested By:	Eduardo Cuevas
Date Tested:	12/9/2024

ASTM D 4718

Corrected Maximum Dry Unit Weight (lbf/ft³):	**116.4**
Corrected Optimum Water Content (%):	**13.1**
Sieve Size (Oversize):	No 4
Oversize Particles (%):	5

Comments

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EXPANSION INDEX TEST

(ASTM D4829)

Client:	Melia Homes	***Tested/ Checked By:***	JZ Lab No Corona
Project Number:	4150-CR	***Date Tested:***	12/10/2024
Project Location:	2375 N. Tustin St.	***Sample Source:***	B-1 @ 0-5 Feet
		Sample Description:	

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	744.2
Weight of ring (gm)	366.2
Net weight of sample (gm)	**378.0**
Wet Density, lb / ft3 (C*0.3016)	**114.0**
Dry Density, lb / ft3 (D/1.F)	**101.8**

SATURATION DETERMINATION

Moisture Content, %	12.0
Specific Gravity, assumed	**2.70**
Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
% Saturation	**49.5**

READINGS

DATE	TIME	READING	
12/10/2024		891.0000	Initial
12/10/2024		0.8900	10 min/Dry
12/11/2024		0.9780	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
785.6	**23.0**

EXPANSION INDEX = 88

Results Only Soil Testing for 2375 N. Tustin Street

December 6, 2024

Prepared for:

Eddy Cuevas
GeoTek USA
1548 N. Maple Avenue
Corona, CA 92878
Ecuevas@geotekusa.com, jbrucelas@geotekusa.com

Project X Job#: S241205B
Client Job or PO#: 4150-CR

Prepared by:
D. Bobrova

Respectfully Submitted,

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

Soil Analysis Lab Results

Client: GeoTek USA
Job Name: 2375 N. Tustin Street
Client Job Number: 4150-CR
Project X Job Number: S241205B
December 6, 2024

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_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^{2-}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ω-cm)	(Ω-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B1	0-5	144.8	0.0145	18.6	0.0019	1,072	1,005	8.2	123	4.0	6.0	22.1	ND	257.7	49.3	117.7	637.0	8.1	0.5

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

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops. This is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

Try out our new online forms: SOIL CORROSIVITY & THERMAL RESISTIVITY LAB REQUEST FORM & IN-SITU WENNER 4 PIN QUOTE REQUEST FORM

APPENDIX C

EXPLORATION LOCATION MAP, LOGS OF EXPLORATORY BORINGS AND LABORATORY TEST RESULTS (SCGI, 2005)

Updated Geotechnical and Infiltration Evaluation

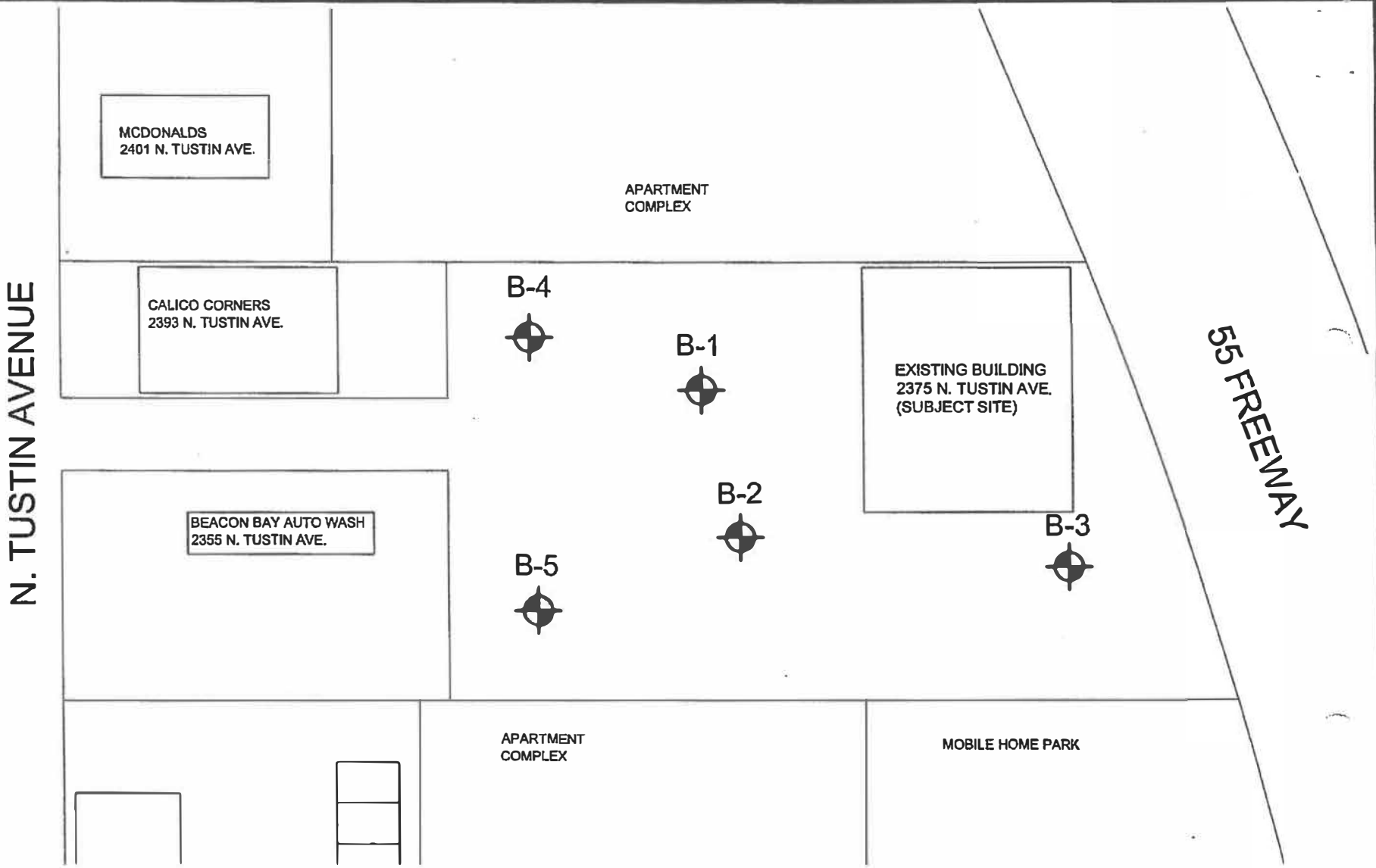
Proposed Townhome Residential Development

2375 North Tustin Street

Orange, Orange County, California

Project No. 4150-CR

GEOTEK



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: BASE MAP PROVIDED BY H & A ARCHITECTS

NORTH

BORING LOCATION PLAN

PROPOSED RESIDENTIAL DEVELOPMENT

ORANGE, CALIFORNIA

SCALE 1" = 100'	Southern California Geotechnical INC.
DRAWN: DRK CHKD: JAS	1260 North Hancock Street, Suite 101 Anaheim, California 92807
SCG PROJECT 05G124-1	Phone: (714) 777-0333 Fax: (714) 777-0398
PLATE 2	

APPENDIX B

BORING LOGS

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVER: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHEBLY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blow required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Southern California Geotechnical INC.

BORING NO. B-1

JOB NO.: 05G124
PROJECT: Proposed Residential Development
LOCATION: Orange, California

DRILLING DATE: 2/9/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Mike Adams

WATER DEPTH: Dry
CAVE DEPTH: 26 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4½ inches Asphaltic Concrete over 6 to 8 inches Aggregate Base							
		37	4.5+		POSSIBLE FILL: Dark Brown Silty Clay, some fine Sand, roots to ¼- inch thick, very stiff-damp	104	10					
		30	4.5+		ALLUVIUM: Dark Brown Silty Clay, trace fine Sand, trace calcareous veining, very stiff-moist	110	14					
5		39	4.5+		@ 5 to 6 feet, Light Brown, little calcareous nodules	118	14					
		46			Brown Silty fine Sand, some fine to coarse Gravel, dense-damp	115	6					
10		30			Brown Silty fine Sand, medium dense-moist	102	8					
15		37	4.5+		Brown Silty Clay, trace fine Sand, hard-moist	112	16					
20		50/5"			Brown Silty fine to medium Sand, some fine to coarse Gravel, very dense-damp		4					
25		50/5"										No Sample Recovered
30		50/4"					5					
					Boring Terminated at 32' due to Auger Refusal on Gravel and Cobbles							

TBL 05G124.GPJ SOCALGEO.GDT 3/30/05

TEST BORING LOG

PLATE B-1

Southern California Geotechnical INC.

BORING NO. B-2

JOB NO.: 05G124
PROJECT: Proposed Residential Development
LOCATION: Orange, California

DRILLING DATE: 2/9/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Mike Adams

WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4 inches Asphaltic Concrete over 7 to 8 inches Aggregate Base							
	X	40	4.5+		POSSIBLE FILL: Dark Brown Silty Clay, some fine Sand, very stiff-damp to moist	109	12					
	X	50	4.5+		ALLUVIUM: Dark Brown Silty Clay, trace fine Sand, hard-moist	113	15					
5	X	47	4.5+		@ 5 to 6 feet, trace calcareous veining	113	13					
	X	50/5"	4.5+		Brown Silty Clay, trace fine Sand, very stiff to hard-moist	114	12					
10	X	50/4"			Brown Silty fine to medium Sand, trace Clay, some fine to coarse Gravel, very dense-damp		5					
15	X	50/5"			Brown Silty Clay, some fine to coarse Gravel, hard-damp to moist		12					
20	X	50/1"										No Sample Recovered
					Boring Terminated at 21' due to Auger Refusal on Gravel and Cobbles							

TBL 05G124.GPJ SOCALGEO.GDT 3/30/05

TEST BORING LOG

PLATE B-2

Southern California Geotechnical INC.

BORING NO. B-3

JOB NO.: 05G124
PROJECT: Proposed Residential Development
LOCATION: Orange, California

DRILLING DATE: 2/9/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Mike Adams

WATER DEPTH: Dry
CAVE DEPTH: 9 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3½ inches Asphaltic Concrete over 8 inches Aggregate Base							
	X	25	1.75		POSSIBLE FILL: Dark Brown Silty Clay, trace fine Gravel, stiff to very stiff-moist	103	21					EI = 108 @ 0 to 5'
	X	29	3.0		ALLUVIUM: Dark Brown Silty Clay, trace calcareous veining, very stiff-moist	105	21					
5	X	37	4.5+			107	18					
	X	48	4.5+		Brown Silty Clay, trace fine Sand, hard-moist	111	17					
	X	50/1"			Silty fine to coarse Sand, trace Clay and fine Gravel, dense-moist		8					Disturbed Sample
10					Boring Terminated at 10'							

TBL 05G124.GPJ SOCALGEO.GDT 3/30/05

TEST BORING LOG

Southern California Geotechnical INC.

BORING NO. B-4

JOB NO.: 05G124
PROJECT: Proposed Residential Development
LOCATION: Orange, California

DRILLING DATE: 2/23/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Mike Adams

WATER DEPTH: Dry
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3½ Inches Asphaltic Concrete over 8 Inches Aggregate Base							
	X	42	4.5+		POSSIBLE FILL: Dark Brown Silty Clay, trace fine Gravel, very stiff to hard-moist	112	18					
	X	42	4.5+		ALLUVIUM: Dark Brown Silty Clay, very stiff-moist	117	15					
5	X	32	4.5+		Brown Silty Clay, trace to little fine Sand, very stiff-moist to very moist	107	20					
	X	37	4.5+		Brown fine Sandy Clay to Silty fine Sand, very stiff-moist to very moist	108	19					
10	X	44	3.75			102	25					
	X	50/5"			Brown Silty fine to medium Sand, some fine to coarse Gravel, very dense-damp		4					
15					Boring Terminated at 15'							

TBL 05G124.GPJ SOCALGEO.GDT 3/30/05

TEST BORING LOG

Southern California Geotechnical INC.

BORING NO. B-5

JOB NO.: 05G124
PROJECT: Proposed Residential Development
LOCATION: Orange, California

DRILLING DATE: 2/23/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Mike Adams

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4½ inches Asphaltic Concrete over 10 inches Aggregate Base							
	X	24	4.5+		FILL: Brown Silty Clay, trace fine Sand, some fine to coarse Gravel, very stiff-damp to moist		13					
	X	39	4.5+		POSSIBLE FILL: Brown Silty Clay, trace fine Sand and some fine Gravel, hard-damp to moist		13					
5												
	X	81			ALLUVIUM: Brown Silty fine to medium Sand, some fine to coarse Gravel, very dense-damp		5					
	X	50/2"										No Sample Recovered
10												
					Brown Silty fine Sand, some fine to coarse Gravel, dense-moist							
	X	35					16					
15												
					Boring Terminated at 15'							

TBL 05G124.GPJ SOCALGEO.GDT 3/30/05

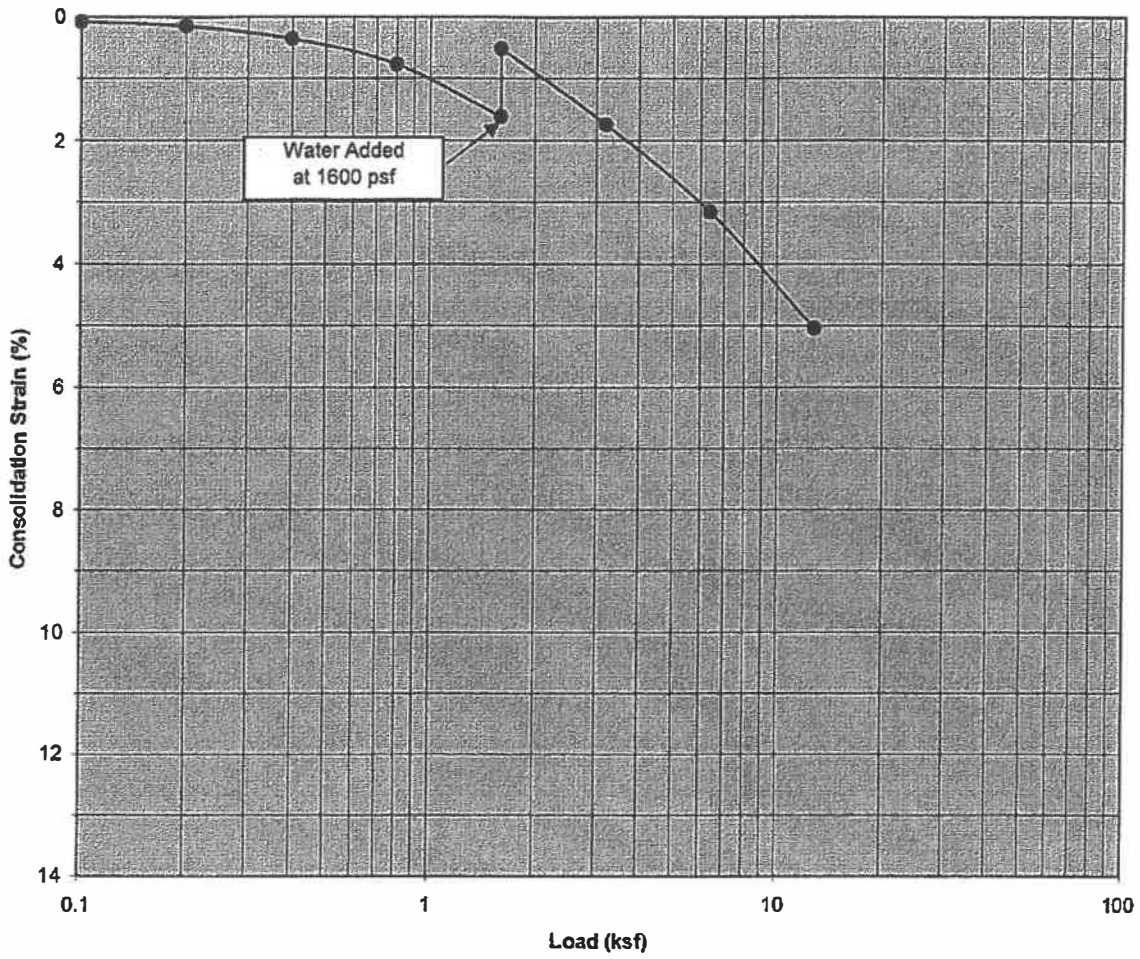
TEST BORING LOG

PLATE B-5

APPENDIX C

LABORATORY TESTING

Consolidation/Collapse Test Results



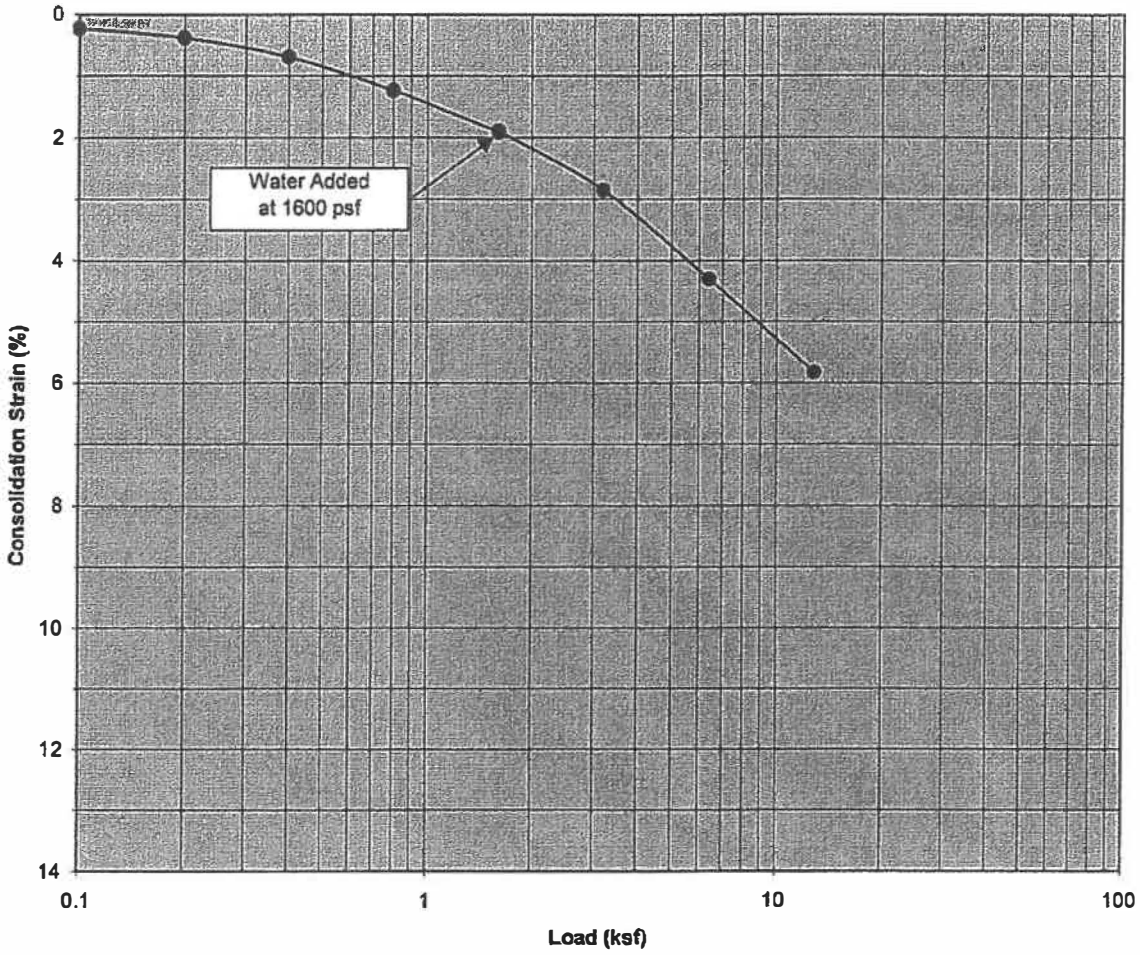
Classification: POSSIBLE FILL: Dark Brown Silty Clay, some fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	1 to 2	Initial Dry Density (pcf)	103.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	-1.10

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 1

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



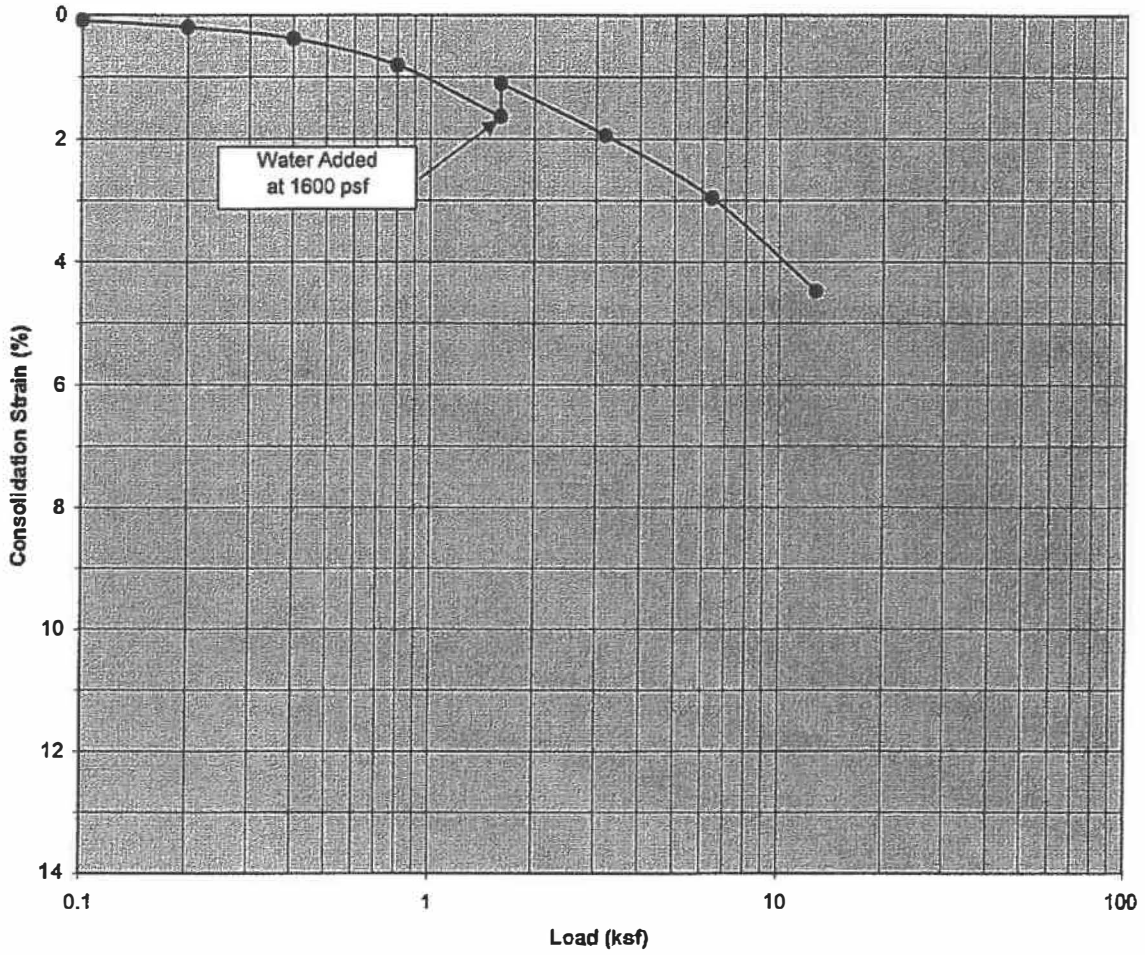
Classification: ALLUVIUM: Dark Brown Silty Clay, trace fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.02

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 2

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



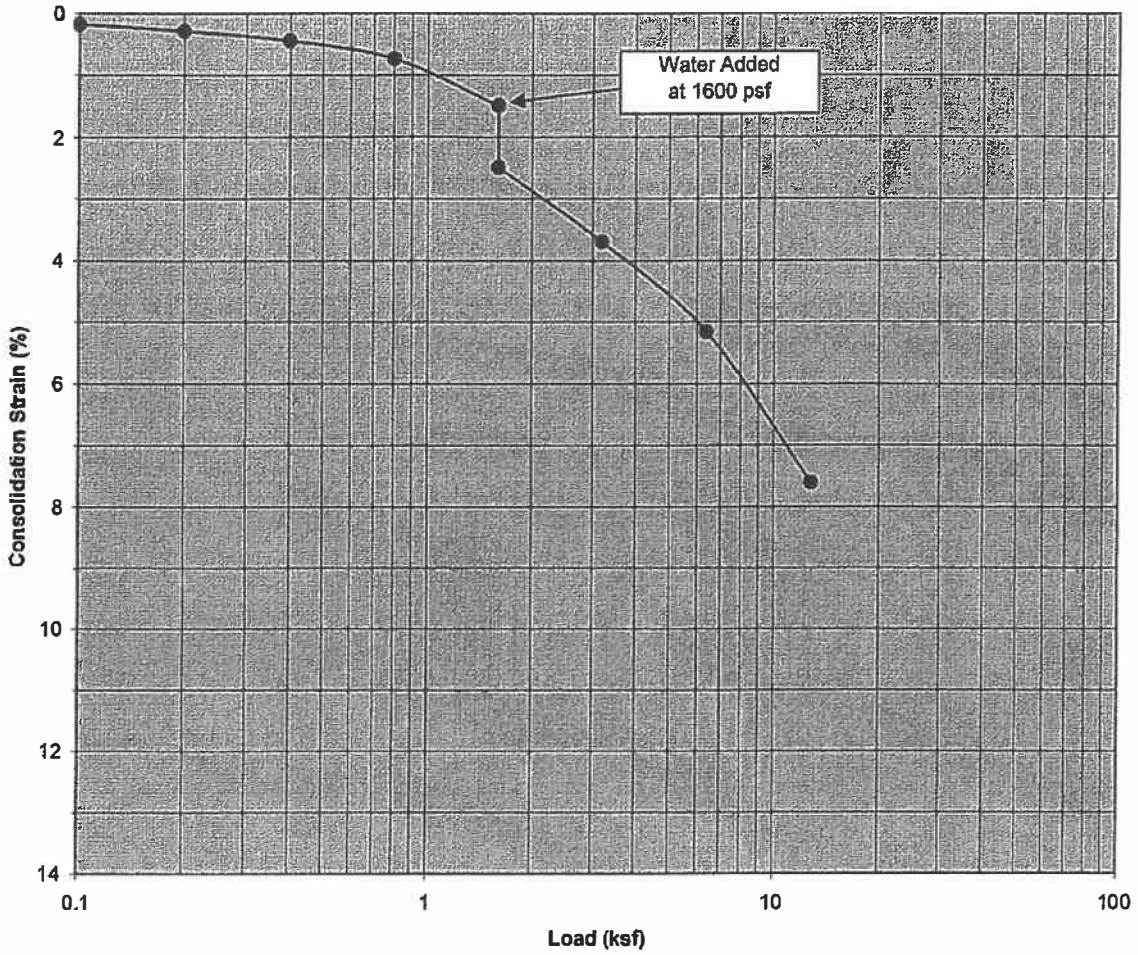
Classification: ALLUVIUM: Light Brown Silty Clay, trace fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	5 to 6	Initial Dry Density (pcf)	108.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.54

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 3

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



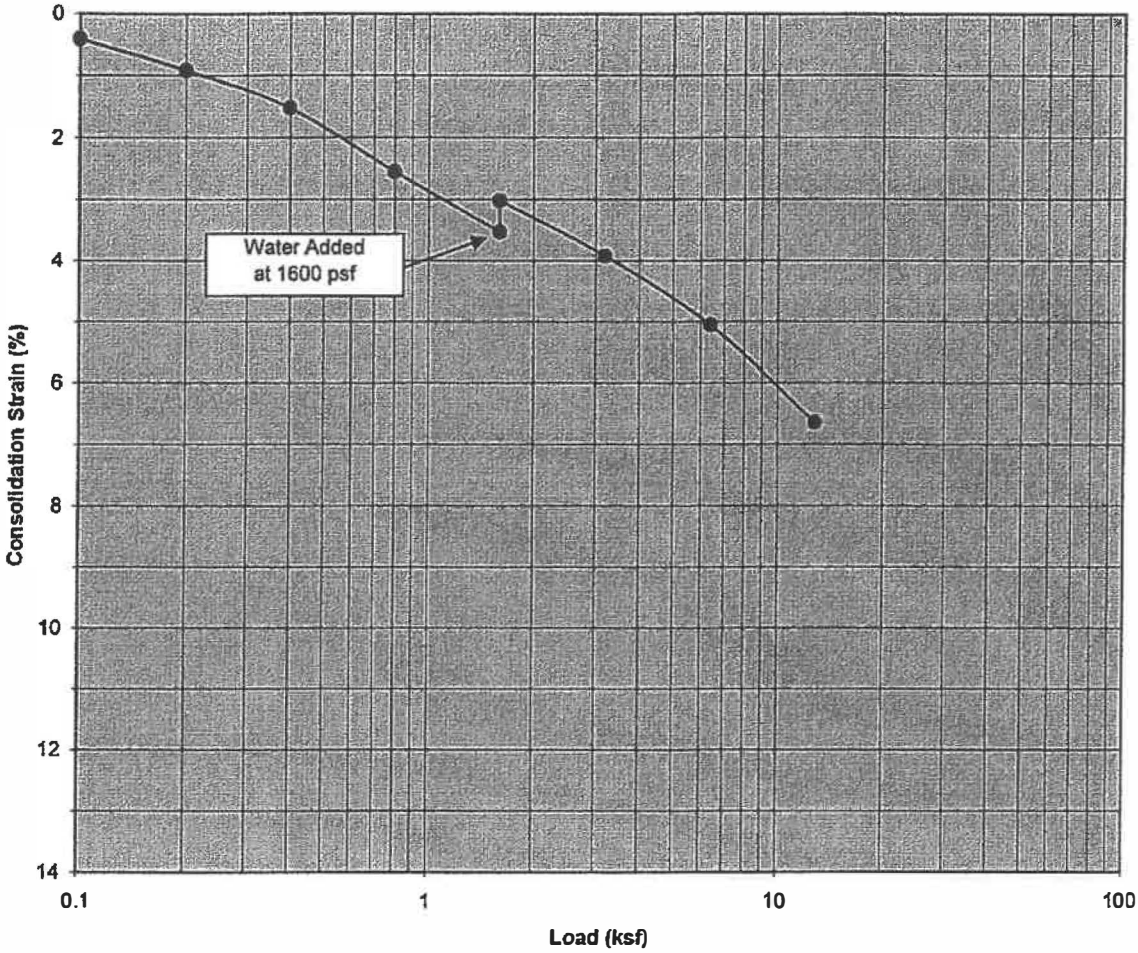
Classification: Brown Silty fine Sand, some fine to coarse Gravel

Boring Number:	B-1	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	7 to 8	Initial Dry Density (pcf)	115.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.00

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 4

Southern California Geotechnical
INC.
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0388

Consolidation/Collapse Test Results



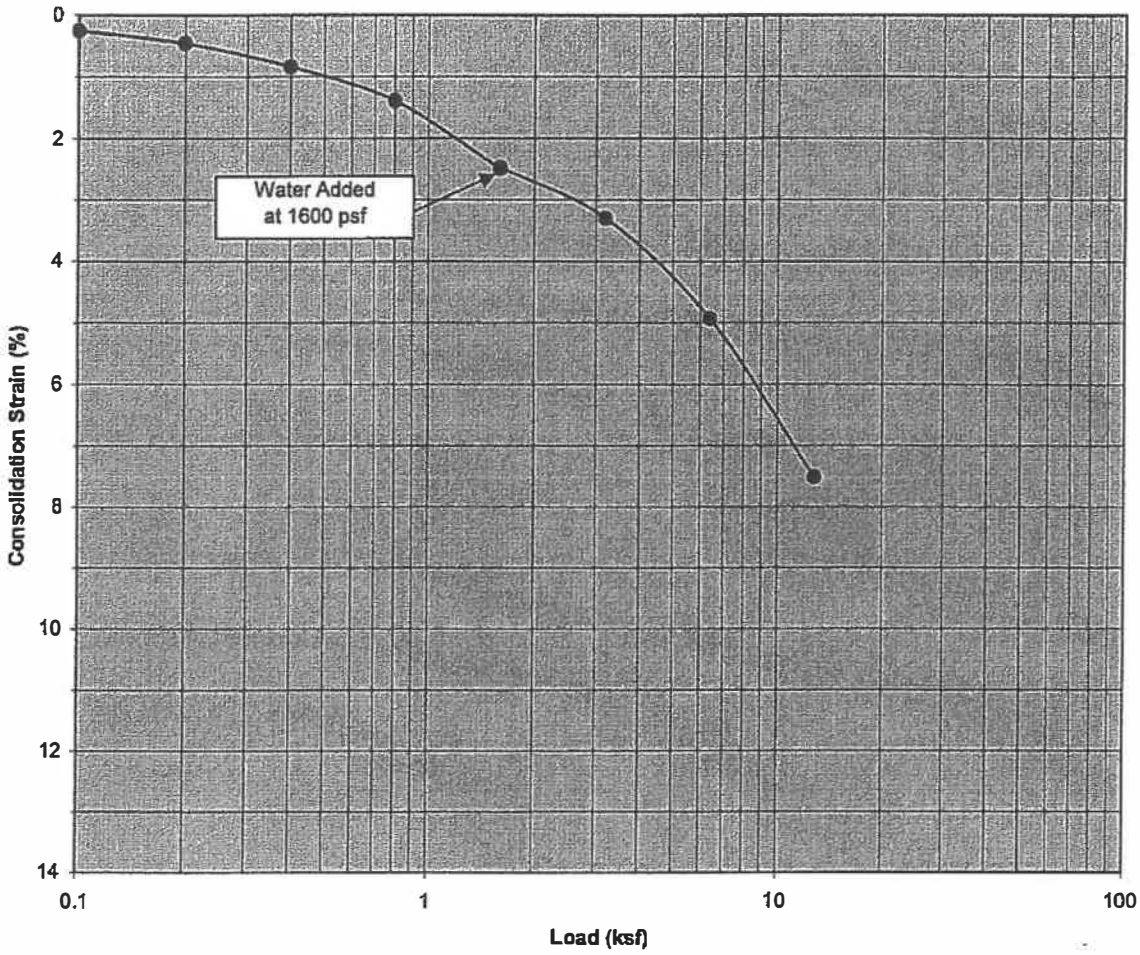
Classification: POSSIBLE FILL: Dark Brown Silty Clay, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	21
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	1 to 2	Initial Dry Density (pcf)	103.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.51

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 5

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



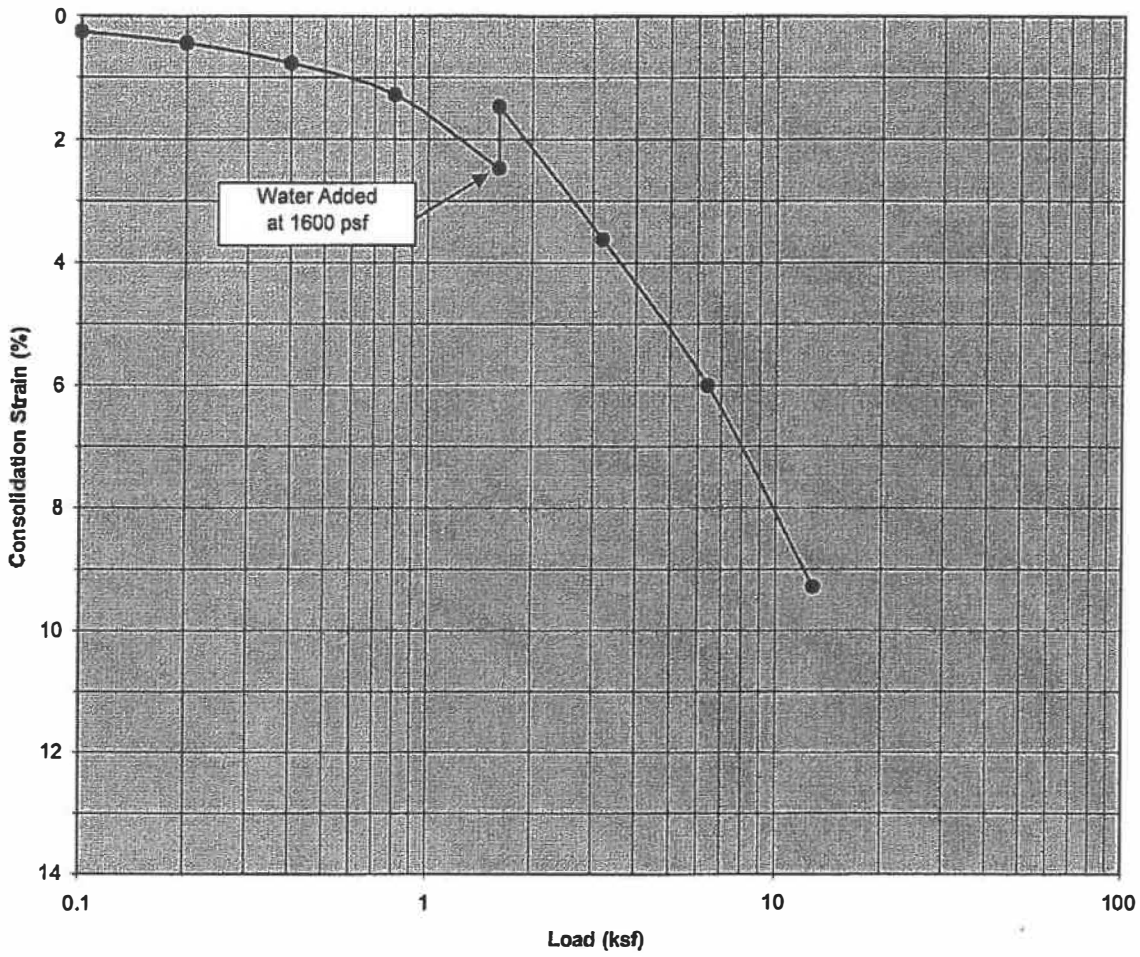
Classification: ALLUVIUM: Dark Brown Silty Clay

Boring Number:	B-3	Initial Moisture Content (%)	20
Sample Number:	—	Final Moisture Content (%)	22
Depth (ft)	3 to 4	Initial Dry Density (pcf)	104.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.02

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 6

Southern California Geotechnical
INC.
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



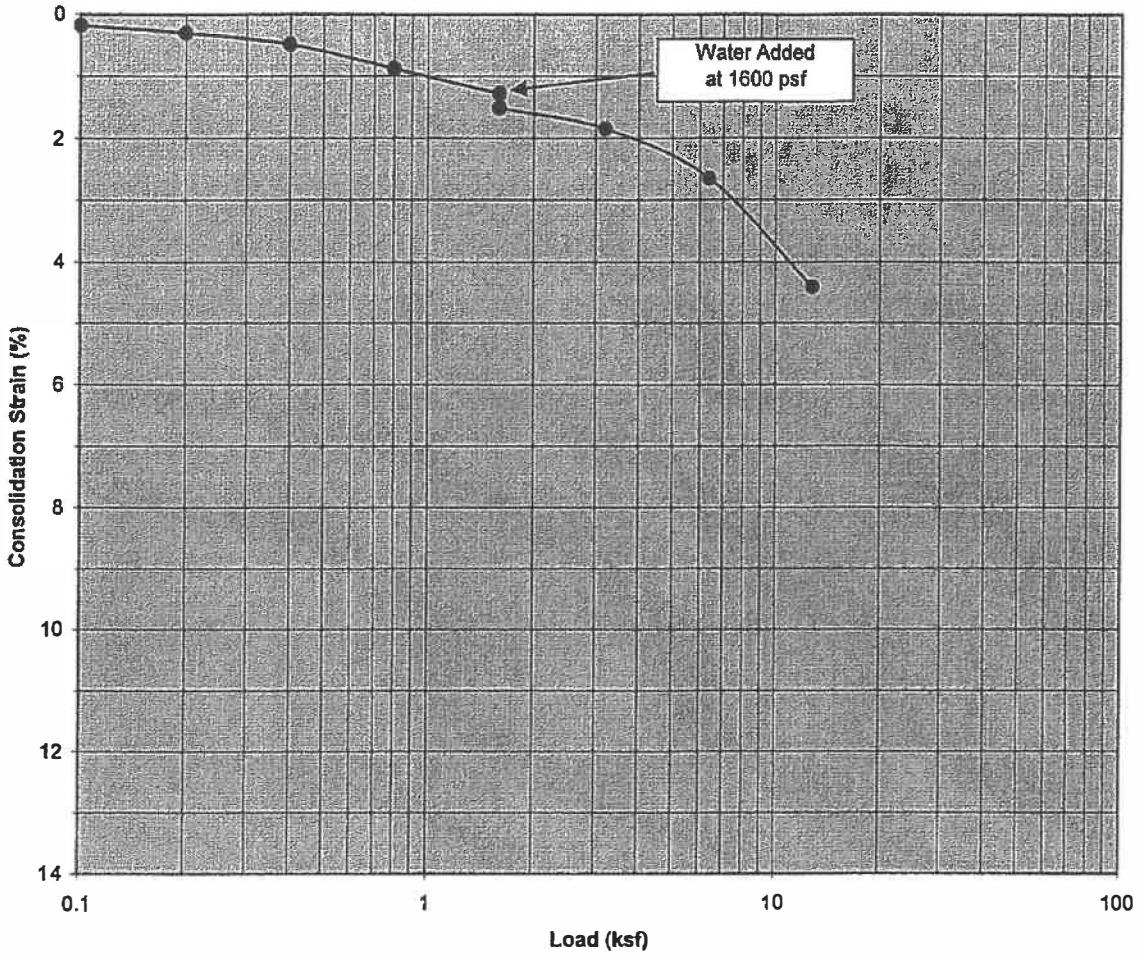
Classification: ALLUVIUM: Dark Brown Silty Clay

Boring Number:	B-3	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	103.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	-1.00

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 7

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



Classification: Brown Silty Sand, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	7 to 8	Initial Dry Density (pcf)	110.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.23

Proposed Residential Development
Orange, California
Project No. 05G124
PLATE C- 8

Southern California Geotechnical
INC.
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

In-situ Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report.

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the

soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	UBC Classification
B-1 @ 0 to 5 feet	0.037	Negligible
B-3 @ 0 to 5 feet	0.070	Negligible

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with Uniform Building Code (UBC) Standard 18-2. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50 ± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	Expansion Index	Expansive Potential
B-3 @ 0 to 5 feet	108	High

APPENDIX D

INFILTRATION TEST DATA

Updated Geotechnical and Infiltration Evaluation
Proposed Townhome Residential Development
2375 North Tustin Street
Orange, Orange County, California
Project No. 4150-CR

GEOTEK

PERCOLATION DATA SHEET

Project: 2375 N Tustin St **Job No.:** 4150-CR

Test Hole No.: I-I **Tested By:** KLP **Date:** 12/4/2024

Depth of Hole As Drilled: 120" **Before Test:** 120" **After Test:** 120"

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
1	9:00 AM		120	24				presoaked
	9:25 AM	25			7 4/5	16 1/5	1.5	
2	9:26 AM		120	24				
	9:51 AM	25			8 2/5	15 3/5	1.6	
1	9:52 AM		120	24				
	10:02 AM	10			15 3/5	8 2/5	1.2	
2	10:03 AM		120	24				
	10:13 AM	10			15 3/5	8 2/5	1.2	
3	10:14 AM		120	24				
	10:24 AM	10			16 1/5	7 4/5	1.3	
4	10:25 AM		120	24				
	10:35 AM	10			16 1/5	7 4/5	1.3	
5	10:36 AM		120	24				
	10:46 AM	10			16 4/5	7 1/5	1.4	
6	10:47 AM		120	24				
	10:57 AM	10			16 4/5	7 1/5	1.4	

GEOTEK

PERCOLATION DATA SHEET

Project: 2375 N Tustin St **Job No.:** 4150-CR

Test Hole No.: I-2 **Tested By:** KLP **Date:** 12/4/2024

Depth of Hole As Drilled: 120" **Before Test:** 120" **After Test:** 120"

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
1	7:00 AM		120	24				presoaked
	7:25 AM	25			18	6	4.2	
2	7:26 AM		120	24				
	7:51 AM	25			18	6	4.2	
1	7:52 AM		120	24				
	8:02 AM	10			21 3/5	2 2/5	4.2	
2	8:03 AM		120	24				
	8:13 AM	10			22	2	5.0	
3	8:14 AM		120	24				
	8:24 AM	10			22 1/5	1 4/5	5.6	
4	8:25 AM		120	24				
	8:35 AM	10			22 1/5	1 4/5	5.6	
5	8:36 AM		120	24				
	8:46 AM	10			22 4/5	1 1/5	8.3	
6	8:47 AM		120	24				
	8:57 AM	10			22 4/5	1 1/5	8.3	

GEOTEK

Client:	**Melia Homes**
Project:	**2375 N Tustin St**
Project No:	**4150-CR**
Date:	**12/4/2024**

Boring No. **I-1**

Percolation to Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	103.2
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	96
Total Test Hole Depth, D_T =	120

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

$H_O = D_T - D_O$ =	24
$H_F = D_T - D_F$ =	16.8
$\Delta H = \Delta D = H_O - H_F$ =	7.2
$Havg = (H_O+H_F)/2$ =	20.4

I_t = 3.86 **Inches per Hour**

GEOTEK

Client:	**Melia Homes**
Project:	**2375 N Tustin St**
Project No:	**4150-CR**
Date:	**12/4/2024**

Boring No. **I-2**

Percolation to Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	97.2
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	96
Total Test Hole Depth, D_T =	120

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

$H_O = D_T - D_O$ =	24
$H_F = D_T - D_F$ =	22.8
$\Delta H = \Delta D = H_O - H_F$ =	1.2
$H_{avg} = (H_O+H_F)/2$ =	23.4

I_t = 0.57 **Inches per Hour**

GEOTEK

APPENDIX E

GENERAL EARTHWORK GRADING GUIDELINES

Updated Geotechnical and Infiltration Evaluation
Proposed Townhome Residential Development
2375 North Tustin Street
Orange, Orange County, California
Project No. 4150-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 (2022) 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 aspects 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 done 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 delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

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

Treatment of Existing Ground

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

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by 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) The 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 contractor's responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

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

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

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the 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 should be brought to the contractor's attention.

JOB SAFETY

General

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

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

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

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

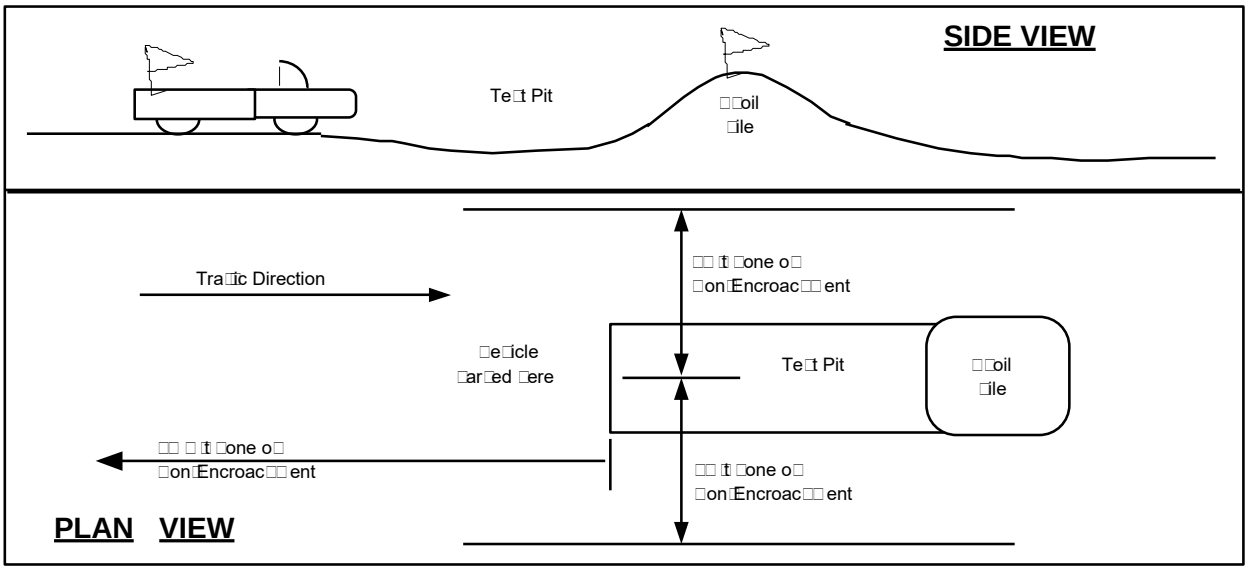
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractor's representative will then be contacted in an effort to 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 affect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

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

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

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