GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED WAREHOUSE FACILITY
534 WEST STRUCK AVENUE
ORANGE, ORANGE COUNTY, CALIFORNIA

PREPARED FOR

PROLOGIS
11777 CENTER COURT DRIVE NORTH, SUITE 100
CERRITOS, CALIFORNIA 90703

PREPARED BY

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

PROJECT NO. 2361-CR **MARCH 31, 2020**

GEOTEK

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March 31, 2020
Project No. 2361-CR

ProLogis
11777 Center Court Drive North, Suite 100
Cerritos, California 90703

Attention: Mr. John Carter

Subject: Geotechnical and Infiltration Evaluation
Proposed Warehouse Facility
534 West Struck Avenue
Orange, Orange County, California

Dear Mr. Carter:

We are pleased to provide the results of our geotechnical and infiltration evaluation for the proposed warehouse facility that will be constructed on the subject site in the city of Orange, Orange County. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for site preparation, foundation design, infiltration rates and construction.

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

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

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

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

- Research and review of available geologic data and general information pertinent to the site,
- A site reconnaissance,
- A geophysical survey, performed by SubSurface Surveys & Associates, Inc., to locate and identify the existence of any pipes, conduits, utilities and other underground obstructions within the vicinity of our 12 borings,
- Excavation and logging of eight (8) geotechnical exploratory borings,
- Logging and infiltration testing of an additional four (4) hollow stem auger borings in the vicinity of the planned storm water treatment areas,
- Collection of soil samples,
- Laboratory testing of selected soil samples,
- Review and evaluation of site seismicity, and;
- Compilation of this geotechnical report which presents our preliminary recommendations for site development.

The intent of this report is to aid in the evaluation of the site for future proposed development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report may need to be updated based upon our review of the 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 approximately 10-acre site is located on the south side of Struck Avenue near its eastern terminus in the city of Orange, Orange County. The site is currently developed with an approximate 40,000 square foot 1-story building, several canopy structures, storage areas, parking/drive areas and landscaping. Several silos exist near the southern limits of the existing 1-story building. Topographically, the site is relatively level sloping down gently to the west. The site location is presented on Figure 1.

Based on a review of aerial photographs flown between 1938 and 2016, the site was undeveloped and used for agricultural purposes until between 1966 to 1972. Since that time, the site has been developed and used for industrial purposes. As noted in the CERCLA Screening Report (Ecology and Environment, Inc., 1990), six or seven (conflicting information) underground storage tanks (USTs) were previously present on the site. The USTs appear to have ranged in storage capacity from about 3,500 to 50,000 gallons. Reportedly, six of the USTs were removed in 1983. However, the report only obtained documentation for the removal of three of the USTs.

The site is bounded by Struck Avenue to the north, a railroad right-of-way to the east and commercial/industrial properties to the west and south.

2.1 PROPOSED DEVELOPMENT

We understand that the existing improvements on the site are to be demolished and a new 216,860 square foot warehouse building and associated surface parking and drive areas are to be constructed. The new building will include 34 dock doors along the east side of the building.

The structure is anticipated to be a single-story concrete tilt-up (or similar) building and maximum column and wall loads of about 150 kips and 6 kips per foot have been estimated for the purpose of this report. Once actual structural loading information is known, that information should be provided to GeoTek to determine if modifications to the recommendations presented in this report are warranted.

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

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

Prior to our subsurface explorations, a geophysical survey was performed at the site by SurbSurface Surveys & Associates, Inc. in order to locate and identify the existence of any pipes, conduits, utilities and other underground obstructions within the vicinity of our proposed borings. A copy of the report is included in Appendix A.

Our field exploration was conducted on March 9, 2019 and consisted of eight (8) geotechnical test borings which were excavated with a hollow-stem auger drill rig to a maximum depth of 30-½ feet below ground surface (bgs). A hollow-stem auger with an outside diameter of 8 inches was utilized. The inside diameter of the auger was 4.5 inches. A geologist from GeoTek, Inc. logged the exploratory borings. The boring locations are presented on Figure 2. Logs of the exploratory borings are included in Appendix A.

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

Relatively undisturbed soil samples were recovered at various intervals in the geotechnical borings with a California sampler. The California sampler is a 3-inch outside diameter, 2.4-inch inside diameter, split barrel sampler lined with brass rings. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for testing and evaluation. The California Ring sampler data are presented on the boring logs.

Four borings were also excavated in the vicinity of the proposed stormwater infiltration areas to depths of about six to eight feet. The locations and depths of the infiltration borings were as requested by the civil engineer. Infiltration testing was conducted in these borings in general accordance with the requirements of the County of Orange The infiltration tests consisted of drilling eight-inch diameter test holes to the desired depth and installing approximately two inches of gravel in the bottom of the hole. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within the boring. Water was then placed in the borings to presoak the holes overnight and percolation testing was performed the following day.

3.2 LABORATORY TESTING

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

Included in our laboratory testing were moisture-density determinations on undisturbed samples. Gradation (percent passing #200 sieve) testing was performed on selected samples. Collapse testing was performed on representative undisturbed samples to evaluate the collapse and settlement potential of the site soils. Optimum moisture content-maximum dry density relationships were established for typical soil types so that the relative compaction of the subsoils could be determined. Direct shear testing was performed on a selected sample to help evaluate the bearing capacity of the soils. Expansion index and Atterberg Limit testing was performed on selected samples to evaluate the expansion potential and plasticity of the on-site soils. Chemical testing comprised of pH, soluble sulfate, chloride and resistivity testing was conducted on selected samples. Resistance value (R-Value) testing was performed on two selected samples to help determine a preliminary pavement design section for the project. The moisture-density, Atterberg Limits and gradation data are presented on the exploration logs in Appendix A. The maximum density, direct shear, expansion index, chemical test and R-Value data are presented in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

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

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

Fault zone trend northwest-southeast and are found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by older alluvial fan deposits (Morton, D.M. and Miller, F.K., 2006).

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the soils encountered on the site is presented in the following sections. Based on our field exploration and observations, the site is generally underlain by undocumented fill and older alluvial fan deposits.

4.2.1 Undocumented Fill

Undocumented fill soils were encountered within ten of the test borings to depths ranging from about 3 to 5 feet below existing grade. As encountered, the undocumented fill consisted of medium dense to dense silty sand, very dense/hard clayey sand to sandy clay, clayey silt and silty clay.

Expansion index testing reveals that the near-surface soils exhibit a "medium" expansion potential.

4.2.2 Older Alluvial Fan Deposits

Older alluvial fan deposits generally consisting of medium dense to very dense sand and silty and clayey sands and stiff to very stiff sandy silts, clayey silts and sandy clays were encountered below the undocumented fill and/or the existing ground surface and extended to the maximum depths explored.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

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

4.3.2 Groundwater

Groundwater was not encountered within our test borings to depths up to about 30 feet below grade. Based on a review of the Seismic Hazard Zone Report for the Orange Quadrangle, historic high groundwater at the site is estimated to be greater than 40 feet below grade.

4.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. The site is not situated within a State of California *"Alquist-Priolo"* Earthquake Fault Zone (CGS, 1980). A review of the Earthquake Zones of Required Investigation Map (Orange Quadrangle, CGS, 1998), the site is not located within a fault or liquefaction hazard area. The nearest known active faults are the San Joaquin Hills and Elsinore fault zones, located about 7.5 miles to the south and 8.3 miles to the northeast, respectively.

4.4.1 Seismic Design Parameters

The site is located at approximately 33.8052^{0} Latitude and -117.8580^{0} Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. As noted using the ASCE 7-16 option on the SEAOC/OSHPD website, the values for S_{M1} and S_{D1} are reported as "null-See Section 11.4.8 (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

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

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

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.405g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.499g
Site Coefficient for Site Class "D," F_a	1.0
Site Coefficient for Site Class "D," F_v	1.801
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.405g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.899g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.937g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.599g
PGA_M	0.649g
Seismic Design Category	D

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

4.5 LIQUEFACTION ANALYSIS

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

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

The project site is not located within an area mapped by the State of California for liquefaction potential. Based on the current map designation and the estimated depth to historic high groundwater (+40 feet) and the density of the materials encountered, it is our opinion that the site possesses a very low potential for liquefaction during a seismic event.

4.6 OTHER SEISMIC HAZARDS

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

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

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

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

5.2 EARTHWORK CONSIDERATIONS

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

5.2.1 Site Clearing & Demolition

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

5.2.2 Remedial Grading

Subsequent to site clearing and lowering of site grades, where necessary, all existing undocumented fill and the upper 1 foot of alluvium should be removed beneath and extending at least 5 feet beyond the planned building limits. As previously noted, USTs were previously present on-site and were reportedly removed in about 1983. No documentation has been provided to indicated that the UST excavations were backfilled with compacted engineered fill. Some localized deeper over-excavations should be expected. As a minimum, all foundations should be underlain by at least 3 feet of newly placed properly compacted engineered fill.

In other areas of the site to support new improvements, we recommend that following site clearing and prior to fill placement, the exposed soil should then be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of GeoTek. The proof roll equipment should possess a minimum weight of 15 tons and proof rolling should include at least four passes, 2 in each perpendicular direction. Any soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative.

5.2.3 Preparation of Excavation Bottoms

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

5.2.4 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided that they are free from vegetation, debris and other deleterious material. Engineered fill should be placed in loose lifts with a thickness of eight inches or less, moisture conditioned to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D-1557).

5.2.5 Excavation Characteristics

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

5.2.6 Shrinkage and Subsidence

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

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

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented below. Based on laboratory test results and the recommendations provided in this report, subsequent to earthwork operations it is anticipated that the as-graded near-surface soils may have a "very low" to "medium" expansion potential.

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

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

Design Parameter	"Very Low" to "Low" Expansion Potential	"Medium" Expansion Potential
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12	18
Minimum Foundation Width (Inches)*	12	12
Minimum Slab Thickness (actual)[1]	4 – Actual	4 – Actual
Sand Blanket and Moisture Retardant Membrane Below On-Grade Building Slabs	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab or No. 3 bars at 24 inch centers	6" x 6" – W2.9/W2.9 welded wire fabric placed in middle of slab or No. 3 bars at 18 inch centers.
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom	Four No. 4 reinforcing bars, two placed near the top and two near the bottom
Effective Plasticity Index***	N/A	18
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 110% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum of 120% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

* Code minimums per Table 1809.7 of the 2019 CBC.
** Sand should have a sand equivalent of at least 30.
*** Effective plasticity index should be verified at the completion of rough grading.

1. Slab thickness and reinforcement should be determined necessary by the structural engineer.

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

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

An allowable bearing capacity of 2,800 pounds per square foot (psf) may be used for design of footings 12 inches deep and 12 inches wide. This value may be increased by 400 pounds per square foot for each additional 12 inches in depth and 150 pounds per square foot for each additional 12 inches in width to a maximum value of 3,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

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

The passive earth pressure may be computed as an equivalent fluid having a density of 210 psf per foot of depth, to a maximum earth pressure of 3,000 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.3 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

If desired, the building floor slab may be designed using an estimated subgrade modulus of 125 pci, which is based on a value typically obtained from a 1 foot by 1 foot plate bearing test. Depending on how the floor slab is loaded, the subgrade modulus may need to be geometrically modified.

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

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

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

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

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

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

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

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

5.3.2 Miscellaneous Foundation Recommendations

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

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

5.3.3 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2019 CBC or 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.

5.3.4 Soil Corrosivity

Based on the chemical test results presented in Appendix B, the corrosivity test results indicate that the on-site soils are "highly corrosive" to "corrosive" to buried ferrous metal. This corrosion classification is obtained from "Corrosion Basics: An Introduction," by Pierre R. Roberge, 2nd Edition, 2000. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer. Additional corrosion testing should be performed at the time of site grading to assess the corrosion of potential of the as-graded soils.

5.3.5 Soil Sulfate Content

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

5.3.6 Import Soils

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

5.3.7 Concrete Flatwork

5.3.7.1 Exterior Concrete Slabs, Sidewalks and Driveways

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

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

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. should be pre-saturated to a minimum of 110 percent (for "low") or 120 percent (for "medium") 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.

5.3.7.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 1/8 inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete undergoes chemical processes that are dependent on a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

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

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

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

Retaining wall foundations should be designed in accordance with *Section 5.3.1* of this report. A minimum foundation embedment of 12 inches into engineered compacted fill with "very low" to "low" expansion potential and 18 inches below grade for footings underlain by "medium" expansive soil is recommended. Structural needs may govern and should be evaluated by the project structural engineer.

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

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

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

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

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

5.4.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES		
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) Native Backfill*	Equivalent Fluid Pressure (pcf) Select Backfill**
Level	45	35
2:1	65	55

* The design pressures assume the backfill material has an expansion index less than or equal to 50. Backfill zone includes area between the back of the wall and footing to a plane (1:1 h:v) up from the bottom of the wall foundation to the ground surface.

** Select backfill to consist of soil with angle of internal friction of at least 34 degree and a very low expansion potential.

5.4.3 Restrained Retaining Walls

Retaining walls that will be restrained prior to placing and compacting backfill material, or that have reentrant or male corners, should be designed for an at-rest equivalent fluid pressure of 70 pcf, plus any applicable surcharge loading, for native backfill (EI<50) and level back slope condition. If backfilled with select soil (EI<20), the walls may be designed for an at-rest pressure of 55 pcf. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.4.4 Retaining Wall Backfill and Drainage

Retaining wall backfill should consist of materials with expansion index (EI) ≤ 20 or <50, dependent upon what design value is used, and free of deleterious and/or oversized materials. The wall backfill should also include a minimum one-foot wide section of ¾- to 1-inch clean crushed rock (or approved equivalent). The rock should be placed immediately adjacent to the back of wall and extend up from the back drain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs. The backfill materials should be placed in lifts no greater than 8-inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained. Bracing of the walls during backfilling and compaction may also be necessary.

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

Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

5.5 INFILTRATION TEST RESULTS

Field percolation testing was performed on March 10 and 11, 2020. Percolation rates obtained from the infiltration testing were converted to a field infiltration rate using the Porchet Method. The field infiltration rates calculated are indicated in the following table:

SUMMARY OF FIELD INFILTRATION RATES			
Boring/Area	Depth of Test (Feet)	Material Encountered at Depth of Test	Field Infiltration Rate (Inches per Hour)
I-1	6	Clayey Silt to Silty Clay	0.02
I-2	7	Clayey Silt	0.02
I-3	8	Clayey Silt	0.02
I-4	6	Clayey Silt	0.02

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

In addition, over the lifetime of the detention or retention basin, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rates to design the infiltration system.

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

5.6 PRELIMINARY PAVEMENT DESIGN

Asphalt Pavements

Preliminary pavement design for areas to receive new asphalt pavements was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. Two representative samples of the near surface subgrade in the future pavement improvement areas were collected and submitted to our subconsultant (LaBelle•Marvin) to determine the resistance value (R-value) of the collected samples. These tests were performed in accordance with California Test Method 301 and those results are provided in Appendix B. Assumed Traffic Indices (TIs) of 5.0, 8.0 and 10.0 were utilized for light, medium and heavy-duty pavement areas, respectively.

The R-value results indicated an R-value for the silty sand soils of 71 and an R-value for the clayey silt soils of 13. Since it is not currently known what soils will be present at the subgrade elevation following site grading, we have provided preliminary pavement thickness recommendations based on the R-value of 13. Based on these preliminary assumptions, the following preliminary sections were calculated:

GEOTECHNICAL RECOMMENDATION FOR MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0	3	8-½
8.0	5	15-½
10.0	5	22-½

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density, per ASTM D-1557.

Traffic Indices (TIs) used in our preliminary pavement design are considered reasonable values for the proposed pavement 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 preliminary 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. We recommend that final pavement design be based on R-value testing of the subgrade soils along with the assigned TI values for the planned pavement areas.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively.

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with the City of Orange specifications, and under the observation and testing of GeoTek and a City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

Concrete Pavements

We understand that Portland Cement concrete (PCC) pavements will also be used for select pavement areas for the site. For this preliminary design we have utilized a subgrade modulus of 75 kci, which is considered to be approximately equivalent to an R-value of 13. PCC design is based on equivalent 18-kip single axle loads (ESAL). In our analysis we have utilized ESAL values of 8,000, 380,000 and 2,500,000 ESAL for light duty, medium duty and heavy-duty pavement areas. The ESAL values noted are approximately equivalent to TI values of 5.0, 8.0 and 10.0, respectively. The design thicknesses presented below are also based on design procedures from the Portland Cement Association (PCA, 1966). Based on these assumptions, the following preliminary PCC pavement thickness are recommended.

Light Duty Pavement (PCC/AB)	**Medium Duty Pavement (PCC/AB)**	**Heavy Duty Pavement (PCC/AB)**
6"/4"	7"/4"	7-½"/4"
6-½"/no base	7-½"/no base	8"/no base

PCC-Portland Cement Concrete

AB-Aggregate Base

We have based this preliminary analysis on a minimum 28-day concrete compressive strength of 4,000 psi. All base material and the upper 12 inches of subgrade should be compacted to at least 95% of the material/s maximum dry density, per ASTM D 1557.

PCC pavements can be designed as jointed plain (unreinforced), with or without dowels for load transfer, or as jointed reinforced pavement. In general, the use of reinforcement typically allows for a wider, or longer, joint spacing, but the reinforcement does not increase the structural capacity of the pavement. In accordance with AASHTO design procedures, the maximum joint spacing for unreinforced pavement is two times the thickness of the pavement (i.e. for a 6-inch thick pavement, the maximum joint spacing is 12 feet). It should be noted that a larger joint spacing is possible for concrete with maximum aggregate sizes of ¾-inch and larger and for concrete with slumps less than 4 to 6 inches. Traffic should not be allowed on the finished pavement until the concrete has attained the minimum compressive strength. Final PCC design should be provided upon completion of rough grading based on R-value tests on the as-graded soils and the assigned TI (or ESAL) values determined by others.

5.7 POST CONSTRUCTION CONSIDERATIONS

5.7.1 Landscape Maintenance and Planting

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

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

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

5.7.2 Drainage

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

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

5.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

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

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

6. INTENT

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

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

7. LIMITATIONS

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

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

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

8. SELECTED REFERENCES

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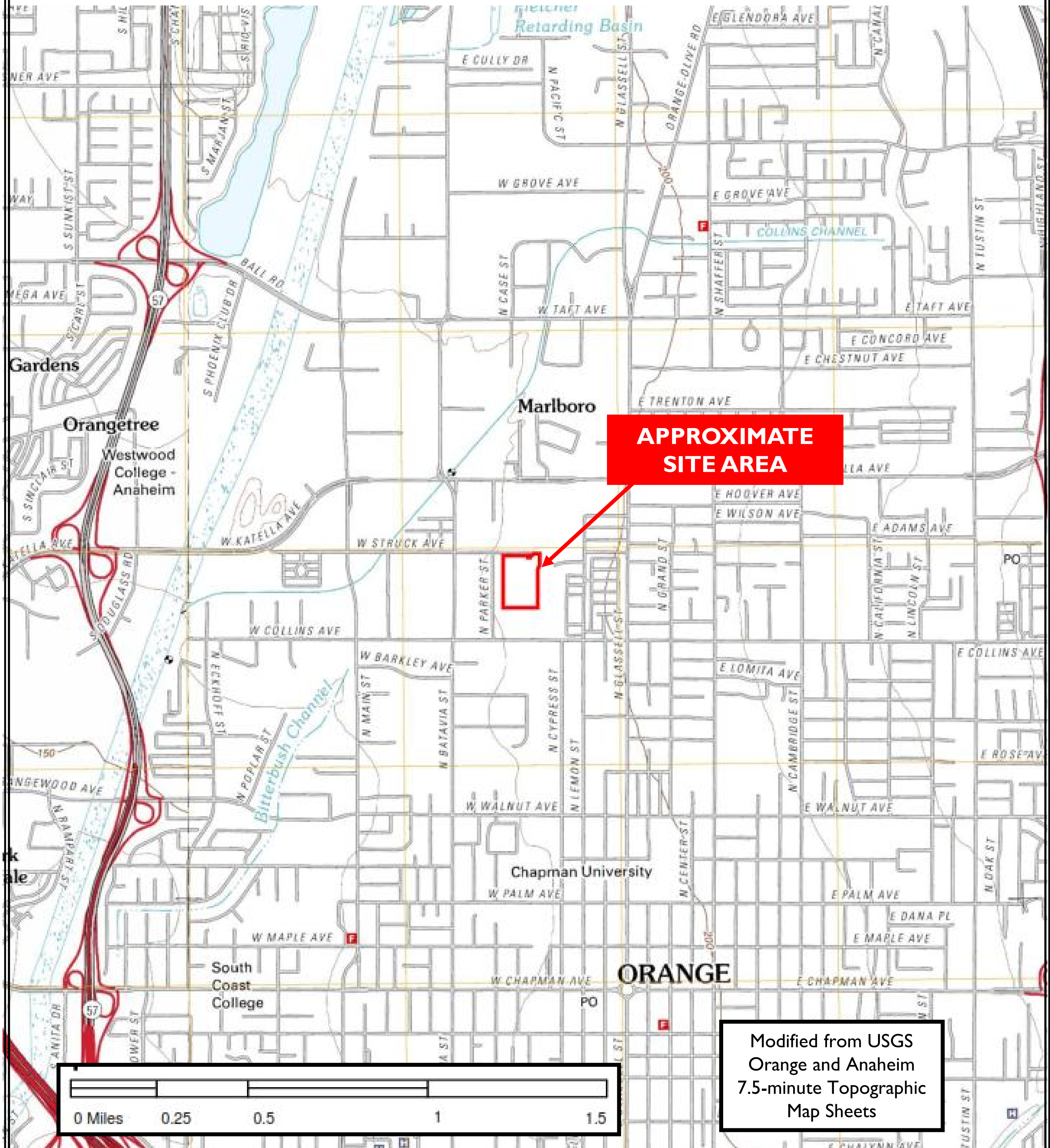
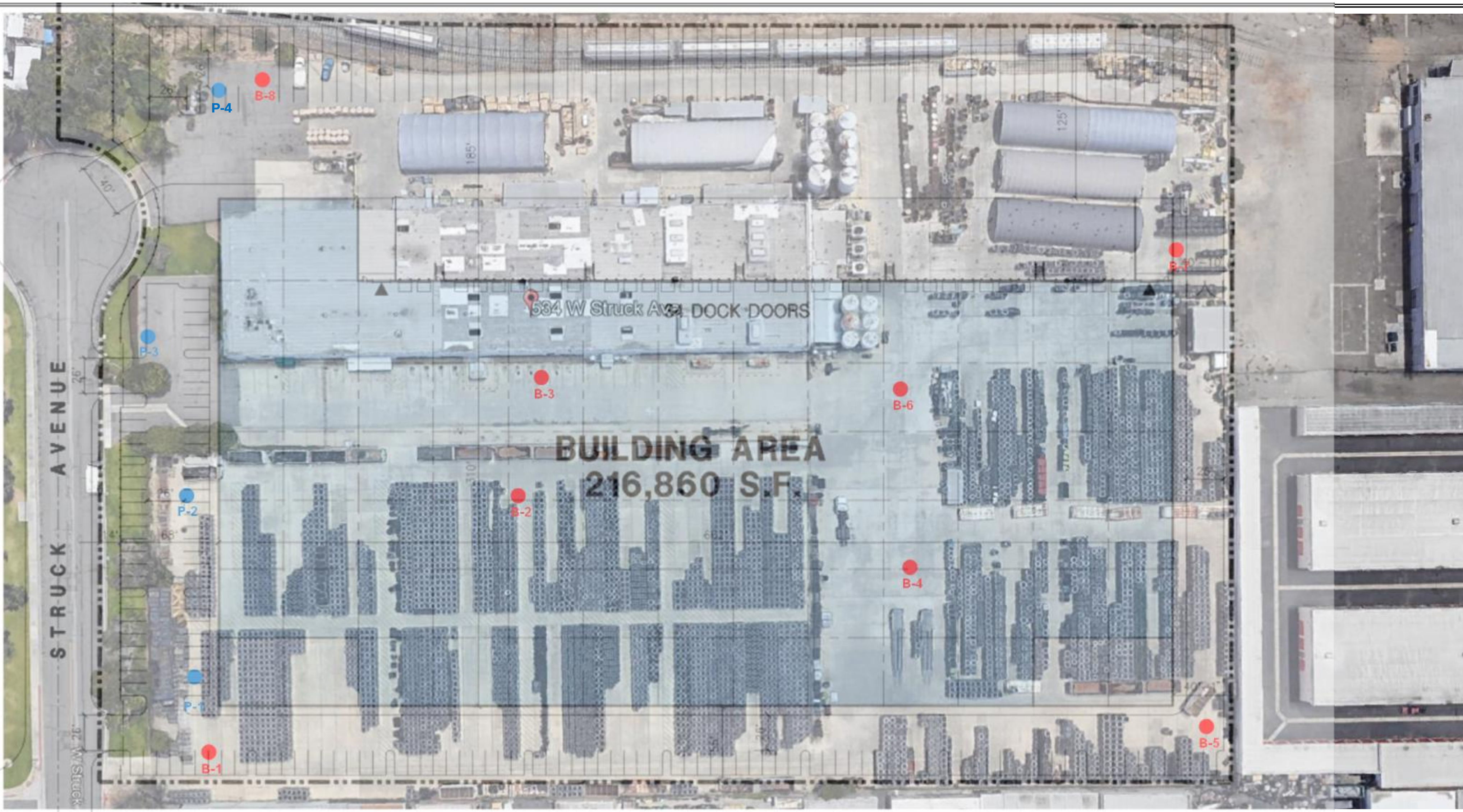


ProLogis 534 West Struck Street Orange, Orange County, California Project No. 2361-CR	NORTH	**Figure 1** **Site Location and Topography Map**	GEOTEK



Legend
(Locations are approximate)

B-8 - Exploratory Boring

P-4 - Percolation Boring

200 FEET

NORTH

Prologis
534 West Struck Avenue
Oraneg, Orange County, California

Project No. 2361-CR

GEOTEK

Plate 1

Exploration Location Map

APPENDIX A

GEOPHYSICAL SURVEY AND LOGS OF EXPLORATORY BORINGS

Proposed Warehouse Facility

City of Orange, Orange County, California

Project No. 2361-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

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

Bulk Samples (Large)
These samples are normally large bags of 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)

SubSurface Surveys
& Associates, Inc.
An Applied Geophysical Company

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Carlsbad, CA 92011
Office: (760) 476-0492
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March 8, 2020

GeoTek, Inc.
1548 North Maple Street
Corona, CA 90501

Project No. 20-120

Attn: Kyle R. McHargue, P.G.

Re: Geophysical Investigation, Nursery Supplies Plant, 534 Struck Avenue, Orange, California

This report is to present the results of our geophysical survey carried out over the Nursery Supplies Plant property located at 534 Struck Avenue in Orange, California (Figure 1). The survey was performed on March 5, 2020, and its purpose was to locate and identify, insofar as possible, the existence of any pipes, conduits, utilities, and other underground obstructions within the vicinity of twelve (12) proposed boreholes scheduled for drilling.

A combination of electromagnetic induction (EM), magnetometry, and ground penetrating radar (GPR) were brought to the field with anticipation of use. Utility locators with line tracing capabilities were also used where applicable.

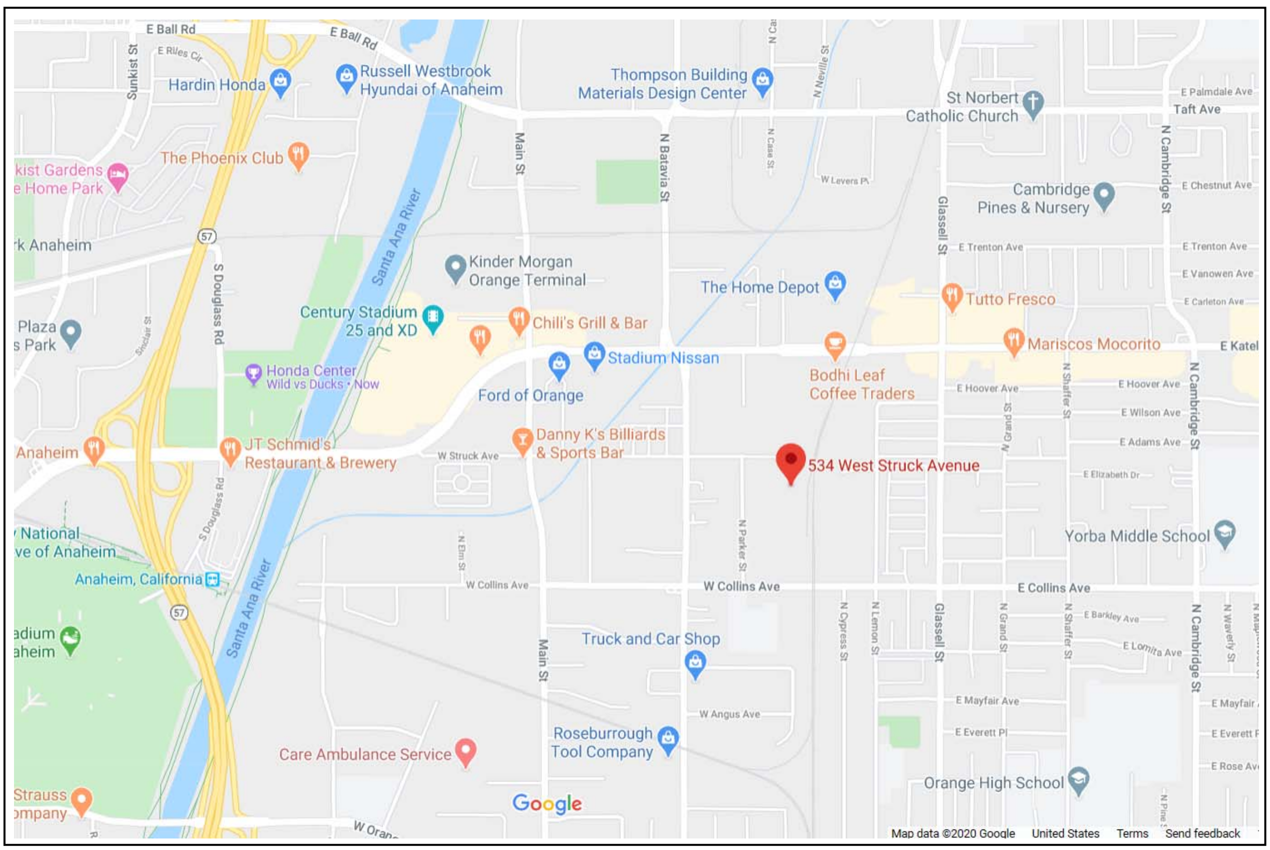


FIGURE 1 – Site location map

Survey Design – The areas to be surveyed were identified in the field by the client. It included twelve (12) proposed boreholes placed in various locations on an asphalt/concrete surfaced parking lot and landscaping.

In site situations and survey objectives such as this, the best use of time is achieved by systematically free-traversing with the instruments while monitoring them continuously to determine which responses are significant and due to true subsurface targets, and which are due to other non-target or above-ground features and must be ignored. Where applicable, the EM devices, magnetic gradiometer, and GPR were traversed systematically over the survey areas in multiple, organized directions. Other traverses were taken for detailing and confirmation where anomalous conditions were found.

In addition, the line tracers were used to impress signals onto pipes, generally through accessible risers and tracer wires when present, to delineate the lines' locations and orientations. The instruments were also used in passive mode, configured to detect 60 Hz electrical signals and other common radio-frequency signals.

Hard copy of the EM data was not acquired, that is, discrete readings on the nodes of a grid were not recorded that could be put into a contoured map format. Rather, the instruments' meters were read continuously, and in real-time, during each traverse. This free-traversing method allowed for immediate detection of anomalous objects and facilitated the opportunity to investigate them further, without the need to first download and process data in the office. The lack of hard copy for EM data sets does not degrade the quality of the survey in any way. Hard copy merely provides a basis for report documentation of these geophysical fields, if such documentation is needed.

A Fischer M-Scope was used for the EM sampling and a Sensors & Software Noggin Ground Penetrating Radar unit with a 500 MHz antenna produced the radar images. A Metrotech 9890 and RIDGID SR-60 SeekTech utility locator rounded out the tools applied.

Brief Description of the Geophysical Methods Applied – The M-Scope device energizes the ground by producing an alternating primary magnetic field with AC current in a transmitting coil. If conducting materials are within the area of influence of the primary field, AC eddy currents are induced to flow in the conductors. A receiving coil senses the secondary magnetic field produced by these eddy currents, and outputs the response as anomalous conditions. The strength of the secondary field is a function of the conductivity of the object, say a pipe, tank or cluster of drums, its size, and its depth and position relative to the instrument's two coils. Conductive objects, to a depth of approximately 7 feet below ground surface (bgs) for the M-Scope are sensed. The device is also somewhat focused; that is, it is more sensitive to conductors below the instrument than they are to conductors off to the side.

The line locator is used to passively detect energized high voltage electric lines and electrical conduit (50-60 Hz), VLF signals (14-22 kHz), as well as to actively trace other utilities. Where risers are present, the utility locator transmitter can be connected directly to the object, and a signal (9.8-82 kHz) is sent traveling along the conductor, pipe, conduit, etc. In the absence of a riser, the transmitter can be used to impress an input signal on the utility by induction. In either case, the receiver unit is tuned to the input signal, and is used to actively trace the signal along the pipe's surface projection.

The GPR instrument beams energy into the ground from its transducer/antenna, in the form of electromagnetic waves. A portion of this energy is reflected back to the antenna at a boundary in the subsurface across which there is an electrical contrast. The instrument produces a continuous record of the reflected energy as the antenna is traversed across the ground surface. The greater the electrical contrast, the higher the amplitude of the returned energy. The radar wave travels at a velocity unique to the material properties of the ground being investigated, and when these velocities are known, the two-way travel times can be converted to depth. The depth of penetration and image resolution produced are a function of ground electrical conductivity and dielectric constant.

Interpretation and Conclusions - The interpretation took place in real time as the survey progressed, and accordingly, the findings of our investigation were verbally relayed to the client, and further documented with site photographs (Figures 2-13).

Utilities detected were marked out in chalk spray paint using red for electric and white for rebar and unknown piping.

Once completed, the proposed boreholes were spray painted white with a white circle and a yellow "SSS" to indicate they had been investigated by Subsurface Surveys personnel. Please refer to the attached photos for location and orientation of items detected in the survey.

Limitations and Further Recommendations - It should be understood that limitations inherent in geophysical instruments and/or surveying techniques exist at all sites, and nearly all sites exhibit conditions under which such might not perform optimally. Consequently, the detection of buried objects in all circumstances **cannot be guaranteed**. Such limitations are numerous and include, but are not limited to, rebar-reinforced ground cover, abrupt changes in ground cover type, above-ground obstacles preventing full traverses or traverses in one direction only, above-ground conductive objects interfering with instrument signal, nearby power lines or EM transmitters, highly conductive background soil conditions, limited GPR penetration, non-metallic targets, shallower or larger objects shielding deeper or smaller targets, tracing signal jumping from one line to another, and inaccessible risers, cleanouts, valve boxes, and manholes. If one or more geophysical instrument is rendered ineffective and cannot be utilized, the quality of the survey can be somewhat degraded.

For the above reasons, and in the interest of maximum safety, we encourage our clients to take advantage of Underground Service Alert (USA), Dig Alert, or other similar services, when possible. Furthermore, we recommend hand auguring and the use of a drilling method known as air knifing or vacuum extraction, when feasible or if applicable to this project. These methods may significantly limit damage to underground pipes, conduits, and utilities that might not have been detectable during the course of this survey. Please bear in mind, that geophysical surveying is only one of several levels of protection that is available to our clients.

SubSurface Surveys may include maps in some reports. While they are an accurate general representation of the site and our findings, they are not of engineering quality (i.e., measured and mapped by a licensed land surveyor).

SubSurface Surveys and Associates makes no guarantee either expressed or implied regarding the accuracy of the findings and interpretations present. And, in no event will SubSurface Surveys and Associates be liable for any direct, indirect, special, incidental, or consequential damages resulting from interpretations and opinions presented herewith.

All data generated on this project are in confidential file in this office and are available for review by authorized persons at any time. The opportunity to participate in this investigation is very much appreciated. Please call, if there are questions.

Daniel L. Matticks, MS	Travis Crosby, GP# 1044
Staff Geophysicist	Senior Geophysicist

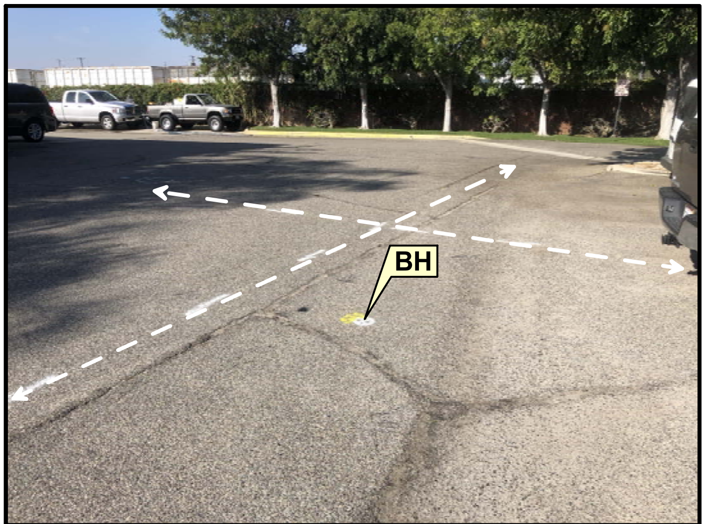


Figure 2

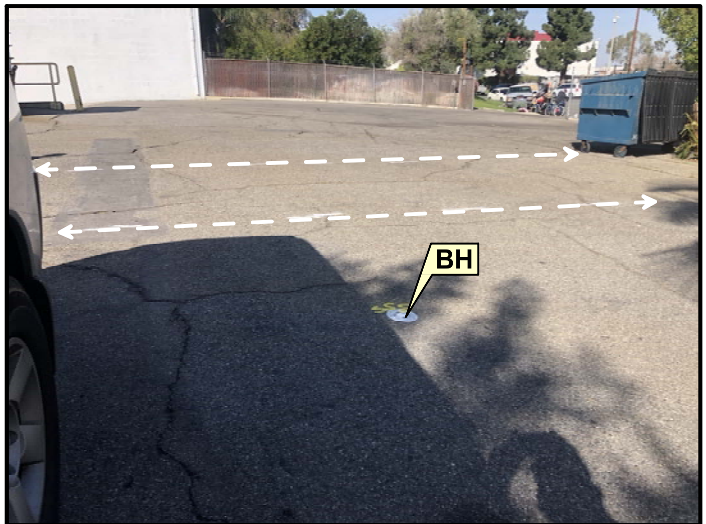


Figure 3

Figure 4

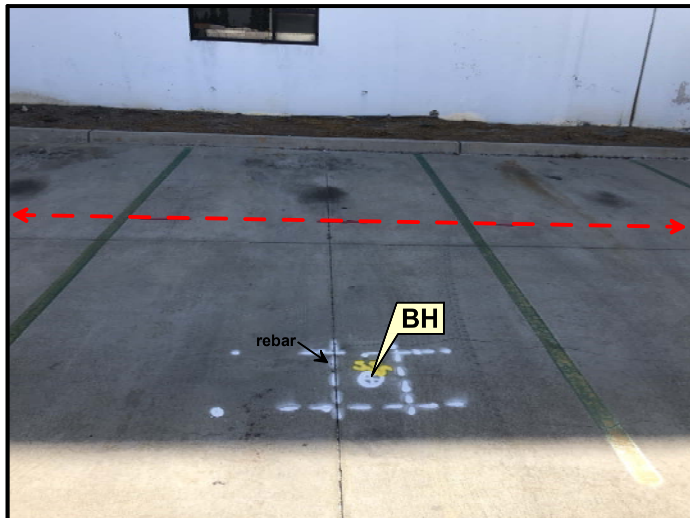


Figure 5

Figure 6

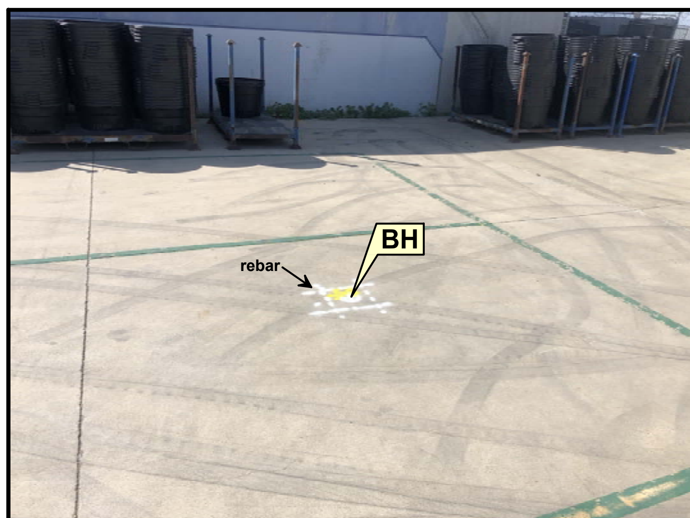


Figure 7

SITE: **Nursery Supplies Plant** **534 Struck Avenue** **Orange, California**	TITLE: **Borehole Photographs**	SURVEY DATE: **March 5, 2020**
	PREPARED FOR: **GeoTek, Inc**	SSS PROJECT NO: **20-120**

Sub Surface Surveys

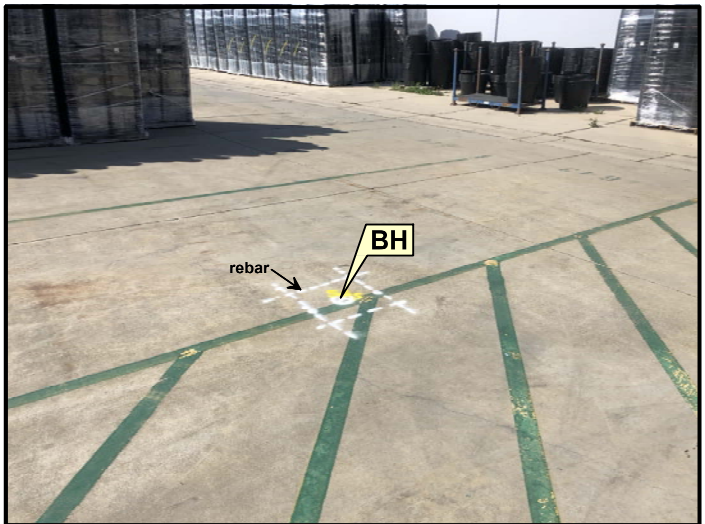


Figure 8

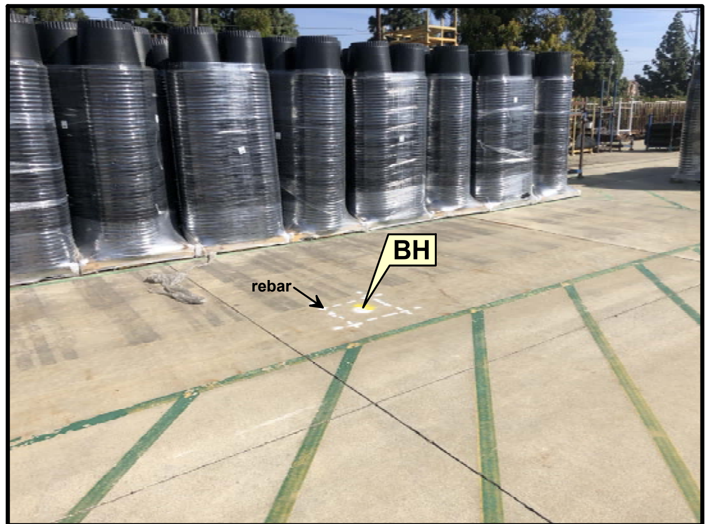


Figure 9

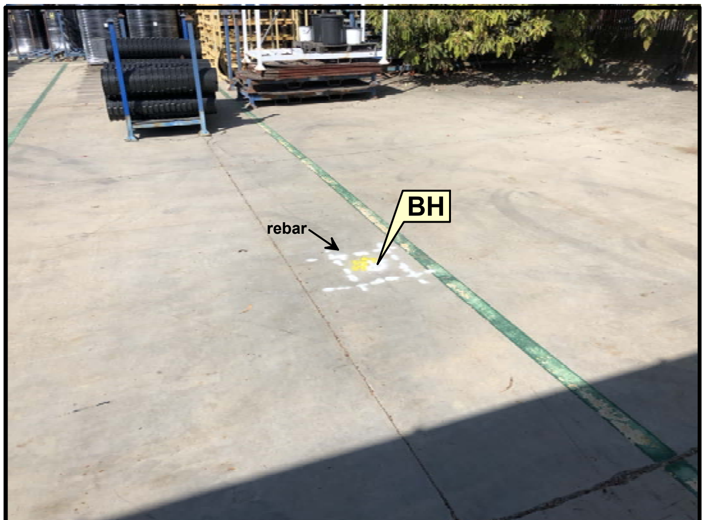


Figure 10

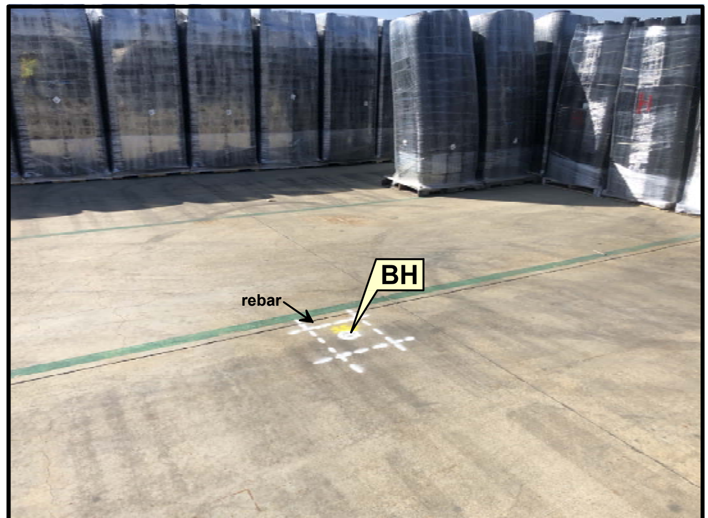


Figure 11

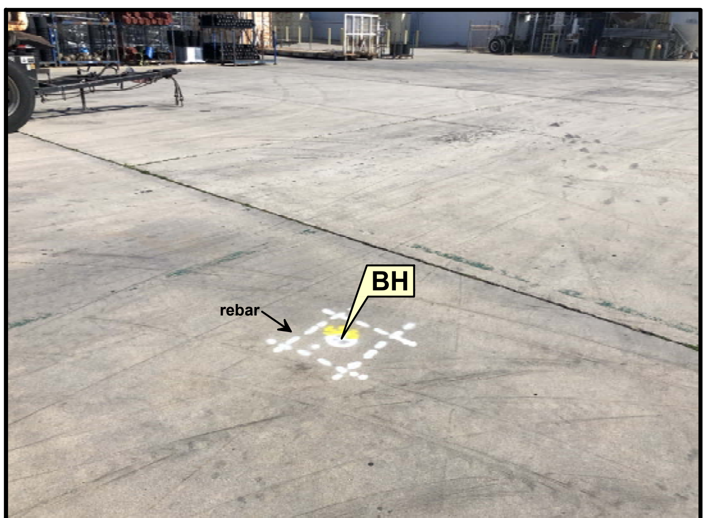


Figure 12

Figure 13

SITE: **Nursery Supplies Plant** **534 Struck Avenue** **Orange, California**	TITLE: **Borehole Photographs**	SURVEY DATE: **March 5, 2020**
	PREPARED FOR: **GeoTek, Inc**	SSS PROJECT NO: **20-120**

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

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
					4" Concrete over 4" CAB			
	Large Bulk				**Undocumented fill:**			
				ML	Clayey SILT, dark brown, moist, trace fine to coarse grained sand			SH, EI, MD, SR EI = 59
	Ring	5 12 19				16.0	119.0	
					Older Alluvial Fan Deposits:			
				CL/ML	Silty CLAY to clayey SILT, light reddish brown, moist, stiff			
5	Ring	11 15 16		CL	F sandy CLAY, light brown, moist, very stiff, trace coarse grained sand	13.4	124.8	Collapse
	Ring	7 11 12			Sandy CLAY, light reddish brown, slightly moist to moist, stiff			
10	Ring	12 17 20			Same as above, very stiff	13.3	121.4	AL LL=35; PL=17; PI=18
15	Ring	7 9 8			Clayey SILT, light brown, moist, stiff, trace very fine to fine grained sand			
					Silty CLAY, brown, moist to very moist, very stiff			
20	SPT	50/6"		SC	Clayey f-c SAND, grayish brown, moist, very dense, few gravel			
					BORING TERMINATED AT 20.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					4" Concrete over 2" CAB			
					Undocumented fill:			
	SPT	5 10 20		SC	Clayey f-c SAND, dark brown, moist, medium dense, few gravel	8.1	121.6	
					Older Alluvial Fan Deposits:			
5	SPT	22 50/5"		SP	Gravelly f-c SAND, dark reddish brown, moist, very dense, trace clay	8.0		
	SPT	35 50/6"			Same as above	5.2		
10	SPT	40 50/2"		SP/GP	Gravelly f-c SAND to f-c sandy GRAVEL, grayish brown, slightly moist to moist, very dense, some cobbles			
					BORING TERMINATED AT 11 FEET (REFUSAL)			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-3

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					4" Concrete over 3" CAB			
					Undocumented fill:			MD, SR
	Large Bulk	10 14 19		SC	Clayey f-c SAND, dark brown, moist, medium dense, some gravel, trace plastic debris			
					Older Alluvial Fan Deposits:			
5	Ring	20 30 40		SP	Gravelly f-c SAND, brown to grayish brown, moist, dense, trace clay	3.7	129.8	
	Ring	26 38 50		GC	Clayey sandy GRAVEL, brown, moist, very dense	4.2	114.8	
10	Ring	38 50/5"		SP	Gravelly f-c SAND, grayish brown, moist, very dense, trace clay			
15	Ring	33 50/5"		GP	F-c sandy GRAVEL, grayish brown, moist, very dense			
20	Ring	7 15 22		CL	Silty CLAY, dark reddish brown, moist, very stiff			
				GC	Clayey sandy GRAVEL, grayish brown, moist			
25	No Recovery	50/2"		GP	F-c sandy GRAVEL, grayish brown, slightly moist to moist, very dense, some cobbles			
30	Ring	50/4"		SC	Clayey f-c SAND, brown, moist, very dense, some gravel, few cobbles			
					BORING TERMINATED AT 30.5 FEET No groundwater encountered Boring backfilled with soil cuttings			

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					5.5" Concrete over 2" CAB			
		3 10 17		SM	**Undocumented Fill:** Silty clayey f-c SAND, reddish brown, moist, medium dense, some gravel, trace cobbles			
					Older Alluvial Fan Deposits:			
5	Ring	19 40 23		SM	Gravelly silty f-c SAND, grayish brown, moist, dense, some cobbles	3.2	125.5	SA % Passing #200 = 14.9
	Ring	16 20 21			Gravelly silty f-c SAND, grayish brown, moist, dense, some cobbles	5.3	125.0	
10	Ring	16 36 38		SC	Clayey f-c SAND, brown, moist to very moist, dense, some gravel, trace cobble			
15	Ring	24 34 40		SP	Gravelly f-c SAND, brown to orangish brown, moist, dense			
					BORING TERMINATED AT 16 FEET (REFUSAL) No groundwater encountered Boring backfilled with soil cuttings			
20								
25								
30								

LEGEND

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

Lab testing: AL = Atterberg Limits; 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:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					4" Concrete over 2" CAB			
	Large Bulk				**Undocumented fill:**			RV
	Ring	2 2 4		ML	Clayey SILT, dark brown, moist, soft, some gravel	10.5	125.3	
					Older Alluvial Fan Deposits:			
5	Ring	17 18 27		SP	Gravelly f-c SAND, orangish brown, moist, medium dense	5.5		
	Ring	32 38 40			Same as above, dense	7.3		
10	Ring	23 35 50/5"		GP	F-c sandy GRAVEL, grayish brown, slightly moist, very dense, some cobbles			
15					**BORING TERMINATED AT 14 FEET (REFUSAL)** No groundwater encountered Boring backfilled with soil cuttings			
20								
25								
30								

LEGEND

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

Lab testing: AL = Atterberg Limits; 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:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-6

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					3" Concrete over 2" CAB			
					Undocumented fill:			
				SM	Silty f-c SAND, dark brown, moist, trace gravel			
	Ring	6 10 10		ML	Clayey SILT, light reddish brown, moist, stiff, trace fine to coarse grained sand, trace gravel	11.4	123.7	
					Older Alluvial Fan Deposits:			
5	Ring	27 36 37		SM-SP	Gravelly silty f-c SAND, grayish brown, moist, dense	18.4		SA % Passing #200 = 11.9
	Ring	20 38 35			Same as above	5.7	130.8	
10	Ring	18 25 30		SC	Clayey gravelly f-c SAND, brown, moist, dense, trace cobbles			
15	Ring	26 50/6"			Same as above, very dense			
					More cobbles become apparent			
					BORING TERMINATED AT 18.5 FEET (REFUSAL)			
20					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-7

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					3" Concrete over 2" CAB			
					Undocumented fill:			
	Ring	10 14 16		SC	Clayey f-c SAND, dark brown, moist, medium dense, some gravel	9.3	128.5	
5	Ring	15 16 17		SC	Clayey gravelly f-c SAND, reddish brown, moist, medium dense	5.8		
					Older Alluvial Fan Deposits:			
				SP	Gravelly f-c SAND, brown, moist, dense, trace clay			
	Ring	25 23 25			Same as above	4.8	114.9	
10	Ring	50/4"		GP/SP	F-c sandy GRAVEL to gravelly f-c SAND, grayish brown, moist, very dense			
					BORING TERMINATED AT 10.5 FEET (REFUSAL)			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: B-8

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					5" of asphaltic concrete			
					Older Alluvial Fan Deposits:			
	Large Bulk							RV
	Ring	6 10 11		ML	Clayey SILT, orangish brown, moist, stiff	14.8	119.5	Collapse
5	Ring	6 8 11			Same as above	13.9	119.3	
	Ring	8 13 18		ML/CL	Clayey SILT to silty CLAY, orangish brown, moist, very stiff	15.3	119.2	
10	Ring	7 9 9		ML	Clayey SILT, light brown, moist, stiff			
15	Ring	6 10 13		CL	Silty CLAY, reddish brown, moist to very moist, stiff, trace gravel			
20	Ring	36 50/6"		SP	Gravelly f-c SAND, grayish brown, moist, very dense, trace clay			
					BORING TERMINATED AT 21 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: I-1

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					5" Concrete over 2" CAB			
				ML	**Undocumented fill:** Clayey SILT, dark brown, moist, trace fine to coarse grained sand			
				ML	**Older Alluvial Fan Deposits:** Clayey SILT, reddish brown, moist			
5				ML/CL	Clayey SILT to silty CLAY, brown, moist			
					BORING TERMINATED AT 6 FEET			
					No groundwater encountered			
10								
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: I-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					4" Concrete over 3" CAB			
				ML	**Undocumented fill:** Clayey SILT, dark brown, moist, trace fine grained sand			
5				ML	**Older Alluvial Fan Deposits:** Clayey SILT, reddish brown, moist Same as above			
10					**BORING TERMINATED AT 7 FEET** No groundwater encountered			
15								
20								
25								
30								

LEGEND

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

Lab testing:

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: I-3

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					7" of asphaltic concrete			
				ML	**Undocumented fill:** Clayey SILT, orangish brown, slightly moist to moist, trace fine gravel			
				CL	**Older Alluvial Fan Deposits:** Silty CLAY, orangish brown to brown, moist			
5				ML	Clayey SILT, brown to reddish brown, moist, some gravel			
					Clayey f-m sandy SILT, light orangish brown, moist, some gravel			
					BORING TERMINATED AT 8 FEET			
					No groundwater encountered			
10								
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	ProLogis	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	534 Struck Avenue	**DRILL METHOD:**	Hollw stem Auger	**OPERATOR:**	Miguel
PROJECT NO.:	2361-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	3/9/2020

BORING NO.: I-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					5" of asphaltic concrete			
					Older Alluvial Fan Deposits:			
				ML	Clayey SILT, orrangish brown, moist, stiff			
5					Same as above			
					BORING TERMINATED AT 6 FEET			
					No groundwater encountered			
10								
15								
20								
25								
30								

LEGEND

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

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

APPENDIX B

LABORATORY TEST RESULTS

Proposed Warehouse Facility
City of Orange, Orange County, California
Project No. 2361-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Atterberg Limits
Atterberg limits testing were performed on a fine-grained sample collected from the site. The test was performed in general accordance with ASTM D 4318. The test results are presented on the log of borings in Appendix A.

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

Consolidation
Consolidation/collapse testing was performed on selected samples of the site soils according to ASTM Test Method D 2435. The results of this testing are presented in Appendix B.

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

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

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-1	1-5	Silty Clay to Clayey Silt	59	Medium

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

Percent of Soil Finer than No. 200 Sieve
Tests to determine the percent of soil finer than No. 200 sieve were performed on selected samples obtained from the property. The tests were conducted in general accordance with ASTM D1140. The results of these tests are presented on the log of borings in Appendix A.

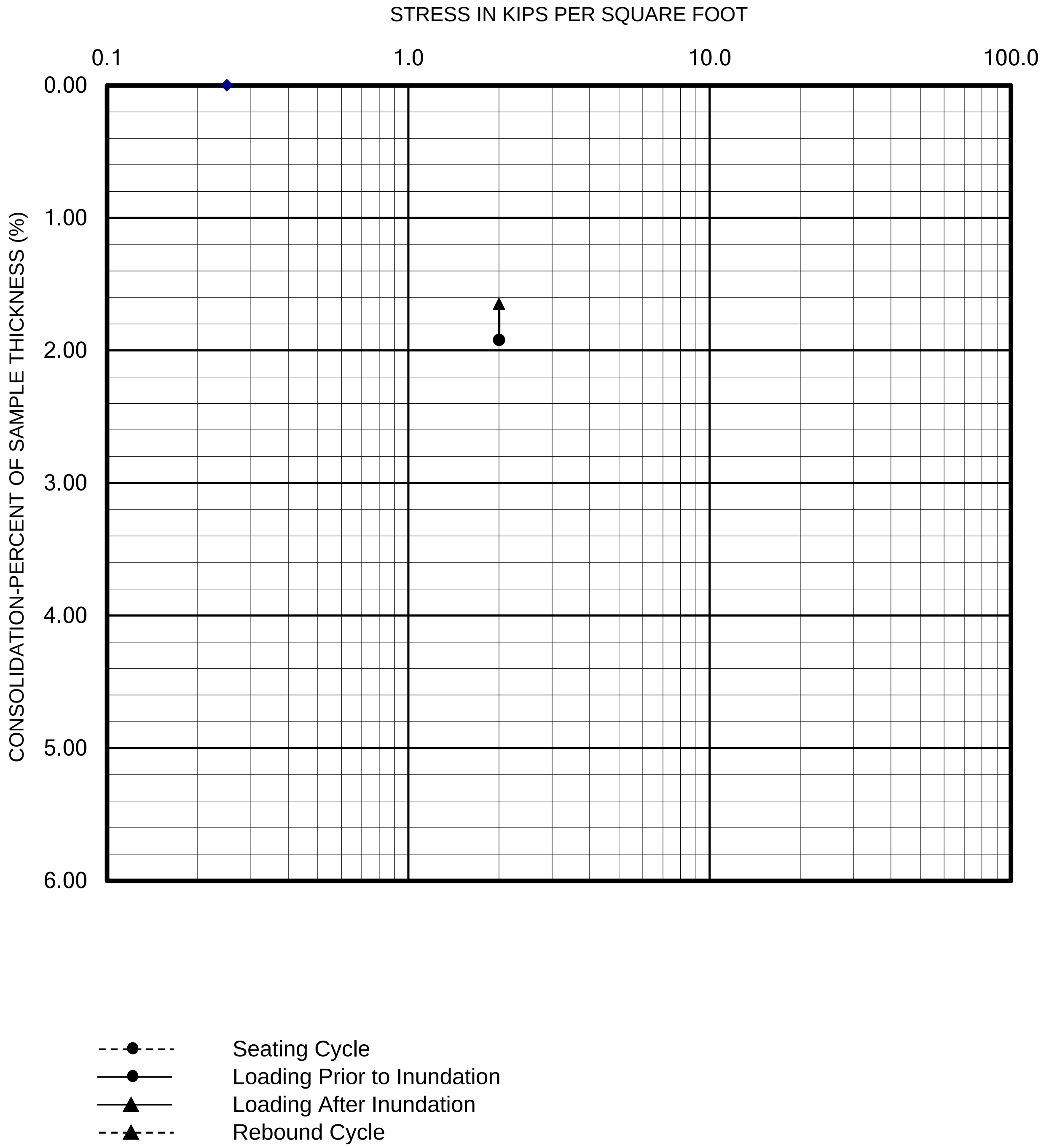
Moisture-Density Relationship
Laboratory testing was performed on a sample obtained during the subsurface exploration. The laboratory maximum dry density and optimum moisture content was determined in general accordance with ASTM D 1557. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	1-5	Silty Clay to Clayey Silt	124.0	11.0

R-Value
R-Value (i.e. resistance value) testing was performed on two samples collected from the site. The tests were performed in general accordance with California Test Method No. 301. The test results are presented in Appendix B.

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

Boring No.	Depth (ft.)	pH G51	Chloride ASTM D4327 (ppm)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-1	1-5	8.6	7.6	0.0309	1,340
B-3	1-5	8.5	4.7	0.0110	4,020



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

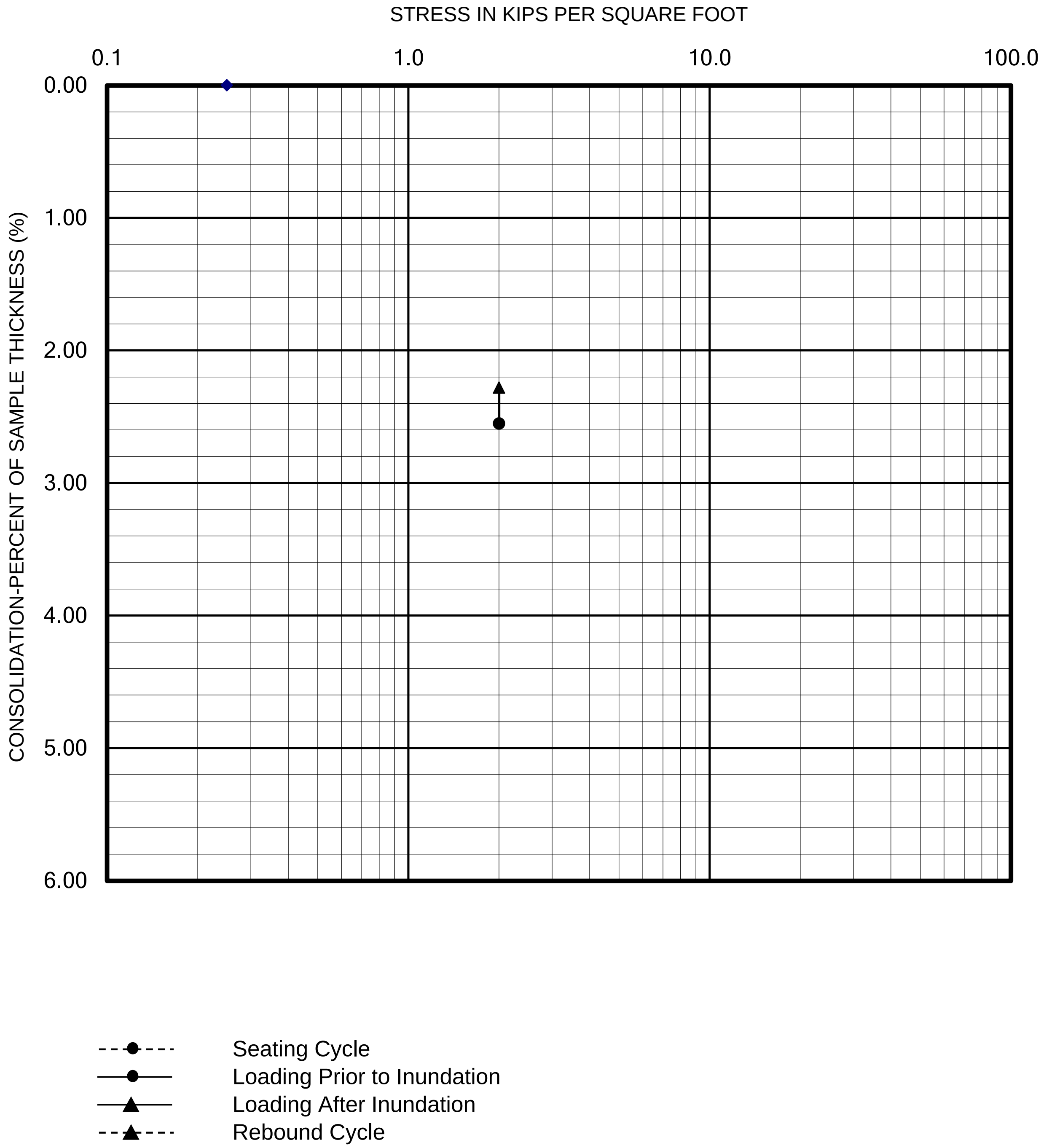
GEOTEK

COLLAPSE REPORT

Sample: B-1 @ 5'

534 West Struck Avenue
Orange, California

CHECKED BY: RRR	Lab: DI
PROJECT NO.: 2361-CR	Date: 03/2020



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

CHECKED BY: RRR	Lab: DI
PROJECT NO.: 2361-CR	Date: 03/2020

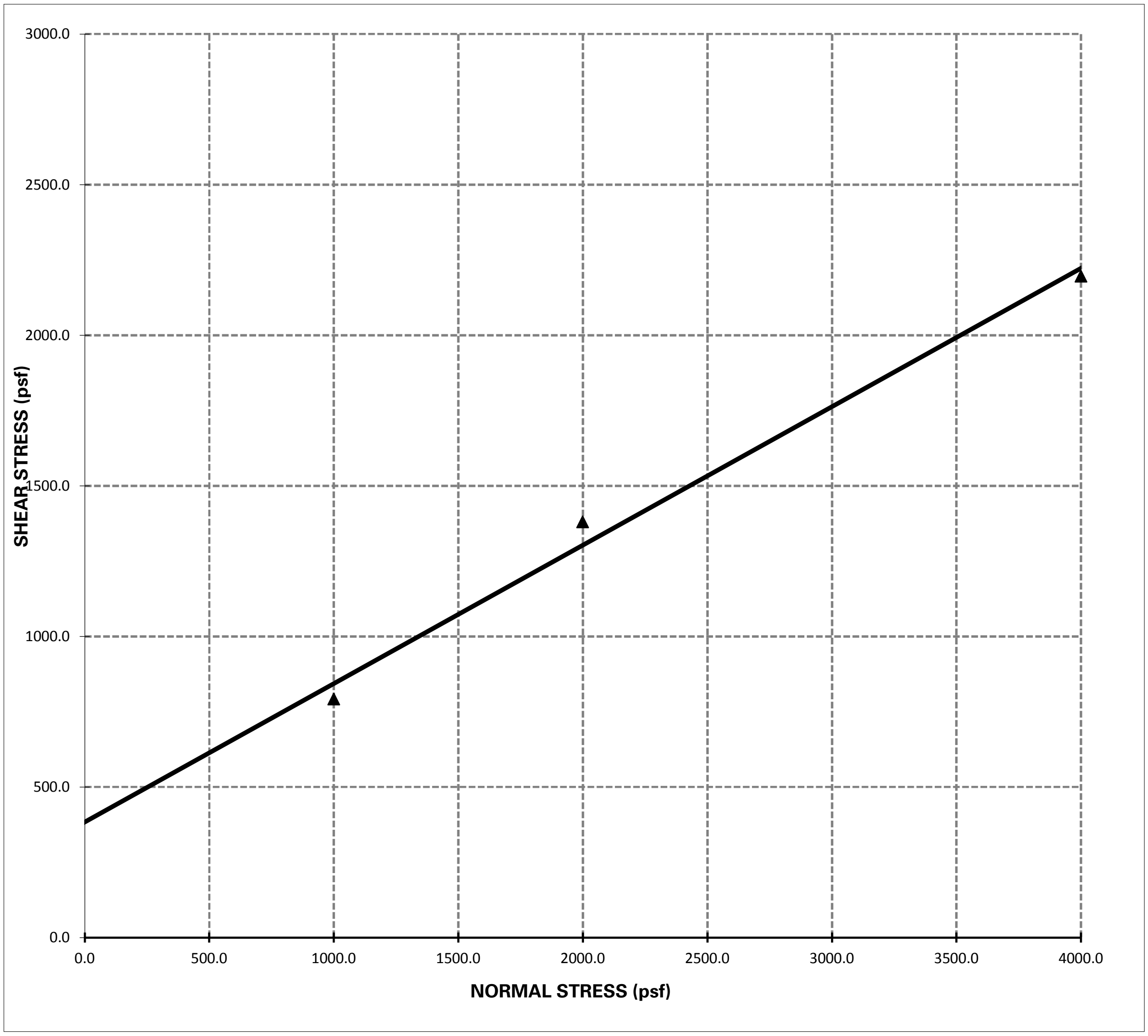
COLLAPSE REPORT

Sample: B-8 @ 2'

534 West Struck Avenue
Orange, California

DIRECT SHEAR TEST

Project Name:	534 West Struck Ave., Orange	***Sample Location:***	B-1 @ 1 - 5
Project Number:	2361-CR	***Date Tested:***	3/25/2020



Shear Strength: Φ = **24.7** $^\circ$, **C = 384.00 psf**

Notes:

1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

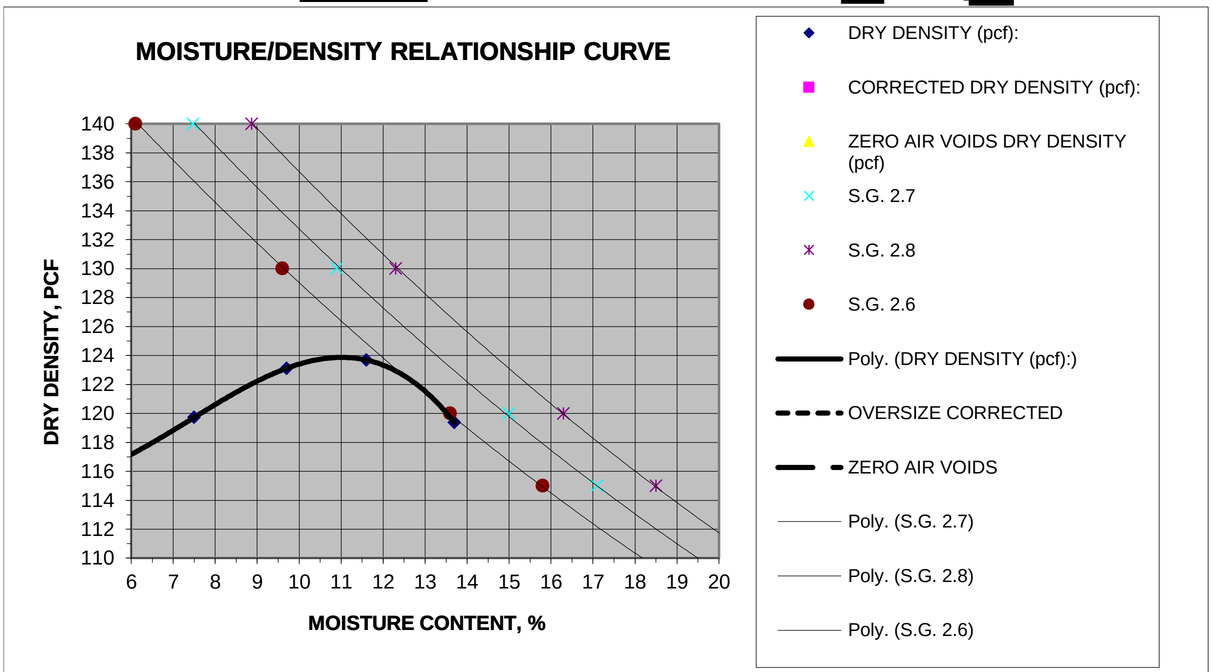
3 - The tests were run at a shear rate of 0.035 in/min.

MOISTURE/DENSITY RELATIONSHIP

Client:	Prologis	**Job No.:**	2361-CR
Project:	534 West Struck Avenue	**Lab No.:**	Corona
Location:	Orange		
Material Type:	Brown Clayey Silt		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-1 @ 1 - 5		
	-		
Sampled By:	DRW	**Date Sampled:**	3/10/2020
Received By:	DLI	**Date Received:**	3/11/2020
Tested By:	DLI	**Date Tested:**	3/21/2020
Reviewed By:	-	**Date Reviewed:**	-

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 5.7 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	124.0	**@ Optimum Moisture, %**	11.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Atterberg Limits:

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

Classification:

Unified Soils Classification: ______

AASHTO Soils Classification: ______

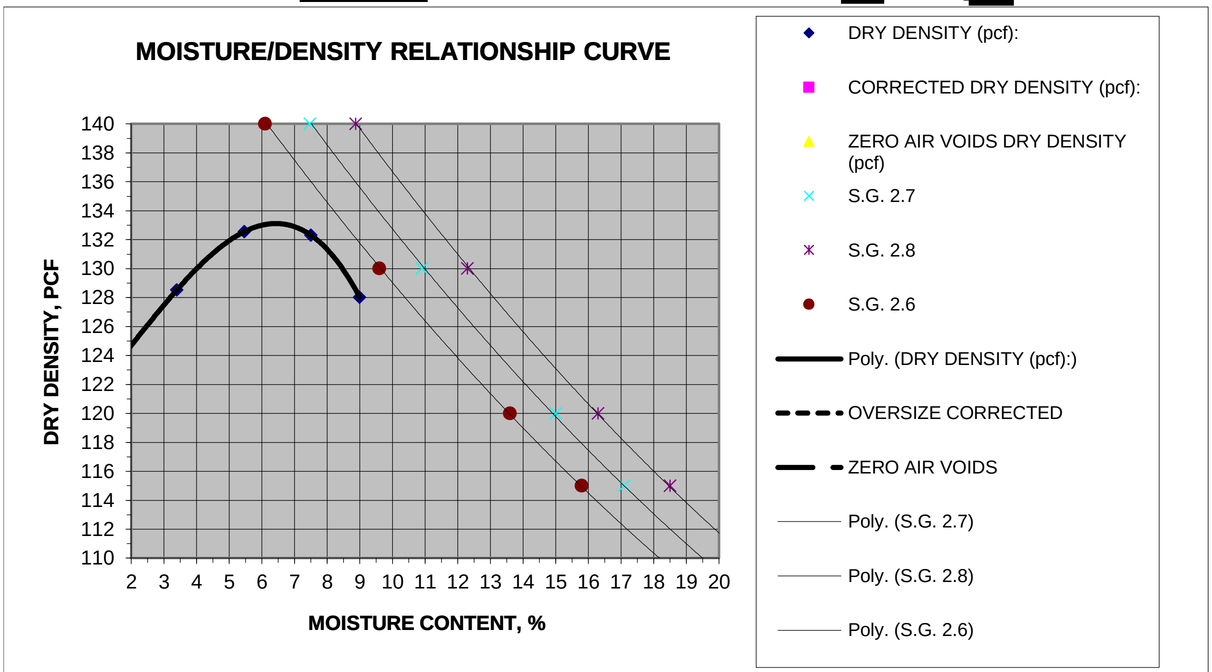
GEOTEK

MOISTURE/DENSITY RELATIONSHIP

Client:	Prologis	**Job No.:**	2361-CR
Project:	534 West Struck Avenue	**Lab No.:**	Corona
Location:	Orange		
Material Type:	Brown Clayey Silt		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-3 @ 1 - 5		
	-		
Sampled By:	DRW	**Date Sampled:**	3/10/2020
Received By:	DLI	**Date Received:**	3/11/2020
Tested By:	DLI	**Date Tested:**	3/21/2020
Reviewed By:	-	**Date Reviewed:**	-

Test Procedure: ASTM D1557 **Method:** C

Oversized Material (%): 29.9 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	133.0	**@ Optimum Moisture, %**	6.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Atterberg Limits:

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

Classification:

Unified Soils Classification: ________

AASHTO Soils Classification: ________

● ANALYSIS
● DESIGN

LaBelle • Marvin

PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

● SOILS, ASPHALT TECHNOLOGY

March 26, 2020

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

Project No. 45887

Attention Ms. Scott:
Laboratory testing of the bulk soil samples delivered to our laboratory on 3/20/2020 has been completed.

Reference:	W.O. # 2361-CR
Project:	ProLogis 534 Struck Drive, Orange
Samples:	B-5 @ 1'-5'
	B-8 @ 1'-5'

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

Very truly yours,

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

Steven R. Marvin
RCE 30659

SRM:tw
Enclosures

2700 S. GRAND AVENUE • SANTA ANA, CA 92705-5404 • (714) 546-3468 • FAX (714) 546-5841
INFO@LABELLEMARVIN.COM

LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 45887
DATE: 3/26/2020

BORING NO. B5 @ 1'-5'
ProLogis 534 Struck Drive, Orange
W.O.# 2361-CR

SAMPLE DESCRIPTION: Brown Gravelly Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	7	8	9
Water added, grams	38	30	25
Initial Test Water, %	8.9	8.2	7.7
Compact Gage Pressure,psi	80	160	260
Exudation Pressure, psi	131	280	504
Height Sample, Inches	2.62	2.56	2.53
Gross Weight Mold, grams	3154	3142	2963
Tare Weight Mold, grams	1953	1948	1772
Sample Wet Weight, grams	1201	1194	1191
Expansion, Inches x 10exp-4	0	0	1
Stability 2,000 lbs (160psi)	26 / 52	15 / 31	14 / 23
Turns Displacement	5.33	4.63	4.34
R-Value Uncorrected	49	69	77
R-Value Corrected	52	70	77
Dry Density, pcf	127.5	130.6	132.4

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.49	0.31	0.24
G. E. by Expansion	0.00	0.00	0.03

Equilibrium R-Value	**71** by EXUDATION	Examined & Checked: 3 /26/ 20
REMARKS:	Gf = 1.25 35.2% Retained on the 3/4" Sieve.	Steven R. Marvin, RCE 30659

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

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 45887

DATE: 3 /26/ 2020

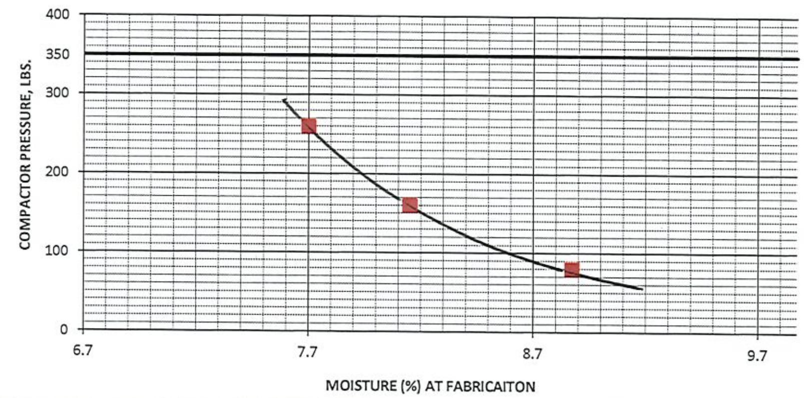
REMARKS:

BORING NO. B5 @ 1'-5'

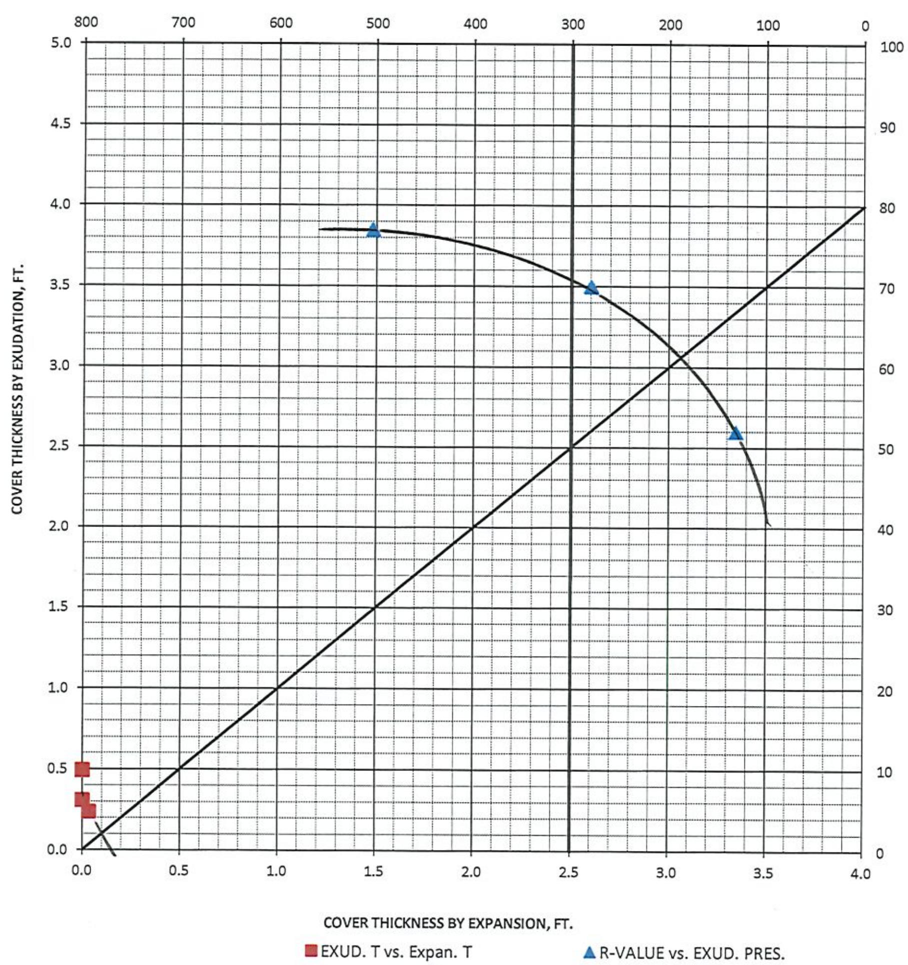
ProLogis 534 Struck Drive, Orange

W.O.# 2361-CR

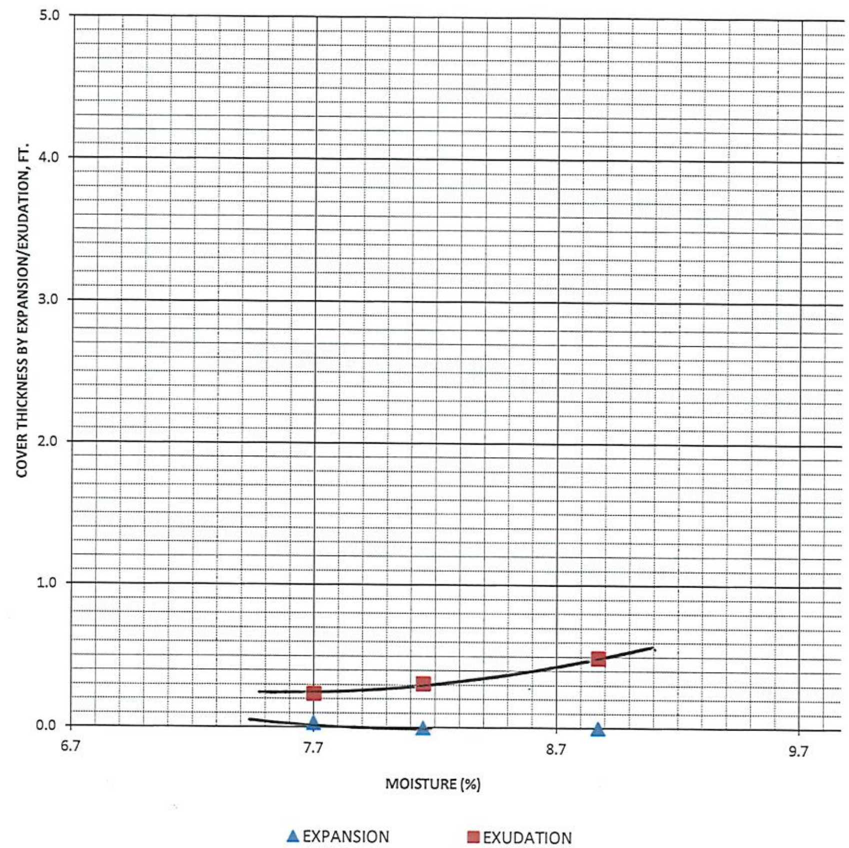
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 45887

DATE: 3/25/2020

BORING NO. B8 @ 1'-5'

ProLogis 534 Struck Drive, Orange

W.O.# 2361-CR

SAMPLE DESCRIPTION: Brown Clayey Silt

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	1	3	4
Water added, grams	50	95	35
Initial Test Water, %	13.0	17.5	11.5
Compact Gage Pressure,psi	130	40	180
Exudation Pressure, psi	440	230	730
Height Sample, Inches	2.46	2.68	2.37
Gross Weight Mold, grams	3067	3104	3055
Tare Weight Mold, grams	1954	1958	1957
Sample Wet Weight, grams	1113	1146	1098
Expansion, Inches x 10exp-4	85	12	180
Stability 2,000 lbs (160psi)	39 / 90	67 / 145	24 / 66
Turns Displacement	3.43	4.65	3.03
R-Value Uncorrected	36	5	54
R-Value Corrected	36	5	51
Dry Density, pcf	121.3	110.3	125.9

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.66	0.97	0.50
G. E. by Expansion	2.83	0.40	6.00

Equilibrium R-Value	**13** by EXPANSION
Examined & Checked:	3 /25/ 20

REMARKS:

Gf = 1.25

0.0% Retained on the 3/4" Sieve.

Steven R. Marvin, RCE 30659

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

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 45887

DATE: 3 /25/ 2020

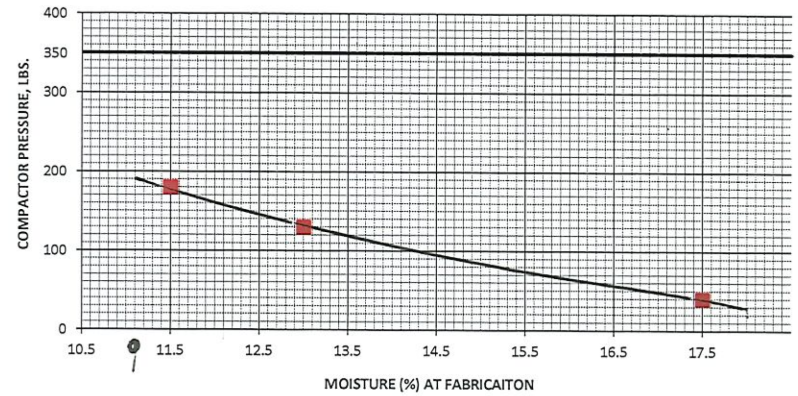
REMARKS:

BORING NO. B8 @ 1'-5'

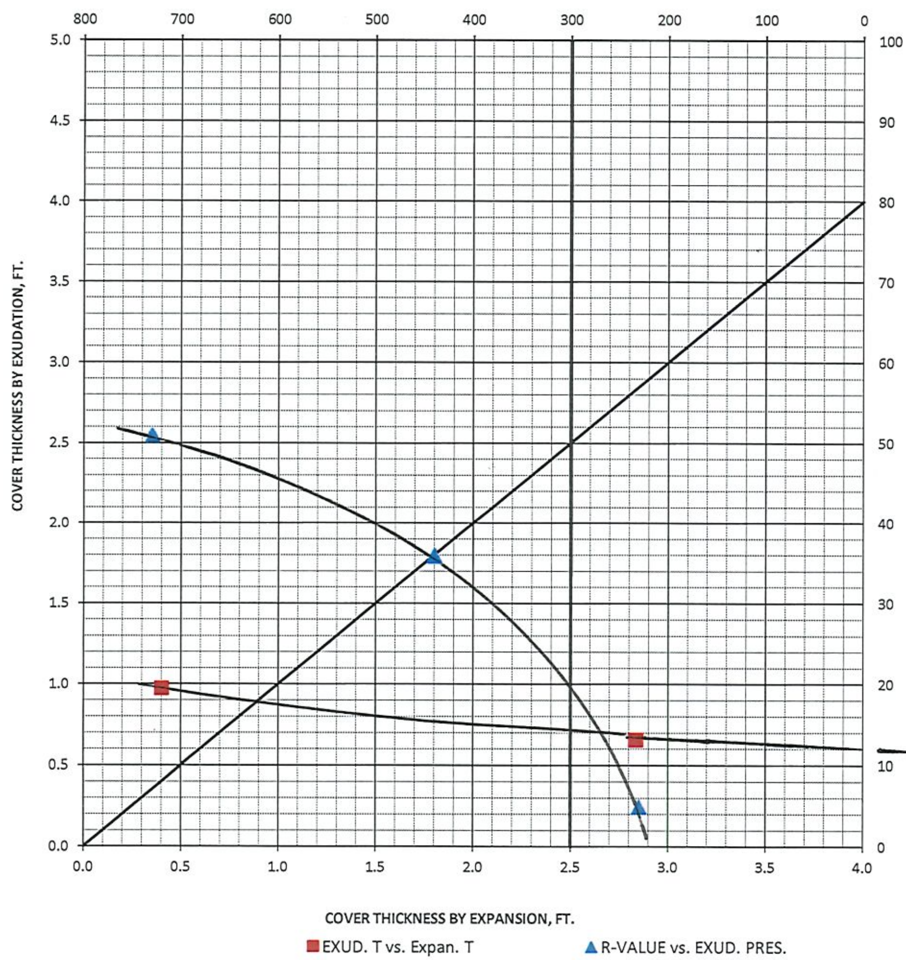
ProLogis 534 Struck Drive, Orange

W.O.# 2361-CR

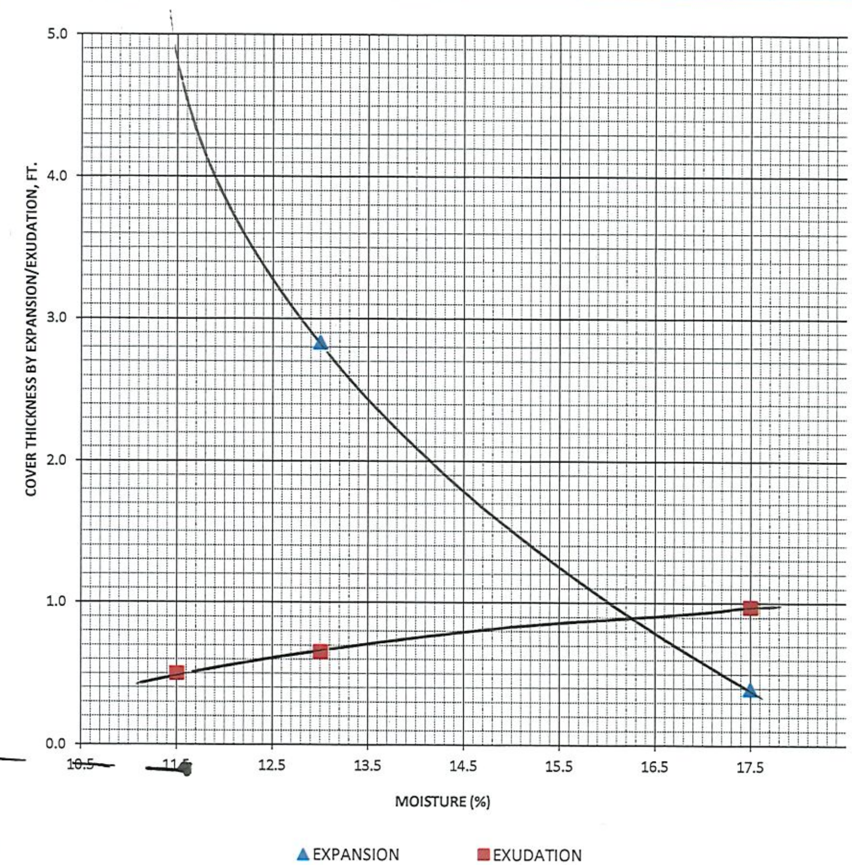
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



Results Only Soil Testing for 534 Struck Ave, Orange

March 19, 2020

Prepared for:
Anna Scott
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92880
ascott@geotekusa.com

Project X Job#: S200316H
Client Job or PO#: 2361-CR

Respectfully Submitted,

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

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

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: 534 Struck Ave, Orange
Client Job Number: 2361-CR
Project X Job Number: S200316H
March 19, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Flouride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B1	1.0-5.0	308.9	0.0309	7.6	0.0008	48,240	1,340	8.6	147.0	0.1	241.1	ND	ND	112.5	ND	7.3	38.9	6.6	ND
B3	1.0-5.0	109.8	0.0110	4.7	0.0005	36,850	4,020	8.5	118.0	0.6	40.1	ND	ND	44.8	0.8	6.2	50.7	2.2	6.5

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

APPENDIX C

INFILTRATION TEST DATA

Proposed Warehouse Facility
City of Orange, Orange County, California
Project No. 2361-CR

GEOTEK

Client:	**ProLogis**
Project:	**Orange**
Project No:	**2361-CR**
Date:	**3/11/2020**

Boring No. **I-1**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	52.125
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	52
Total Test Hole Depth, D_T =	72

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	19.875
$\Delta H = \Delta D = H_O - H_F$ =	0.125
$Havg = (H_O+H_F)/2$ =	19.9375

I_t = 0.02 **Inches per Hour**

GEOTEK

Client:	**ProLogis**
Project:	**Orange**
Project No:	**2361-CR**
Date:	**3/11/2020**

Boring No. **I-2**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	64.125
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	64
Total Test Hole Depth, D_T =	84

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	19.875
$\Delta H = \Delta D = H_O - H_F$ =	0.125
Havg = $(H_O+H_F)/2$ =	19.9375

I_t = 0.02 **Inches per Hour**

GEOTEK

Client:	**ProLogis**
Project:	**Orange**
Project No:	**2361-CR**
Date:	**3/11/2020**

Boring No. **I-3**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	76.125
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	76
Total Test Hole Depth, D_T =	96

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	19.875
$\Delta H = \Delta D = H_O - H_F$ =	0.125
Havg = $(H_O+H_F)/2$ =	19.9375

I_t = 0.02 **Inches per Hour**

GEOTEK

Client:	**ProLogis**
Project:	**Orange**
Project No:	**2361-CR**
Date:	**3/11/2020**

Boring No. **I-4**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	52.125
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	52
Total Test Hole Depth, D_T =	72

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	19.875
$\Delta H = \Delta D = H_O - H_F$ =	0.125
$H_{avg} = (H_O+H_F)/2$ =	19.9375

I_t = 0.02 **Inches per Hour**

GEOTEK

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361-CR

Test Hole No.: I-1 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 72 Before Test: 72" After Test: 72".

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
							PRESOAK 5 GAL 3/10/2020
	815		72	20			BEGIN 3/11/2020
	840	25			19 7/8	1/8	1ST 25 MIN.
	842		72	20			
	907	25			19 7/8	1/8	2ND 25 MIN.
	909		72	20			
	939	30			19 7/8	1/8	1ST 30 MIN.
	941		72	20			
	1011	30			19 7/8	1/8	2ND 30 MIN
	1013		72	20			
	1043	30			19 7/8	1/8	3RD 30 MIN
	1045		72	20			
	1115	30			19 7/8	1/8	4TH 30 MIN.
	1117		72	20			
	1147	30			19 7/8	1/8	5TH 30 MIN.
	1149		72	20			
	1219	30			19 7/8	1/8	6TH 30 MIN.
	1221		72	20			
	1251	30			19 7/8	1/8	7TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361 - CR

Test Hole No.: I-1 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 72" Before Test: 72" After Test: 72"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	1253		72	20			
	123	30			19 7/8	1/8	8TH 30 MIN.
	125		72	20			
	155	30			19 7/8	1/8	9TH 30 MIN.
	157		72	20			
	227	30			19 7/8	1/8	10TH 30 MIN
	229		72	20			
	259	30			19 7/8	1/8	11TH 30 MIN.
	301		72	20			
	331	30			19 7/8	1/8	12TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361-CR

Test Hole No.: I-2 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 84" Before Test: 84" After Test: 84"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
							PRESOAK 5 GAL 3/10/2020
	822		84	20			BEGIN 3/11/2020
	847	25			19 7/8	1/8	1ST 25 MIN.
	849		84	20			
	914	25			19 7/8	1/8	2ND 25 MIN.
	916		84	20			
	946	30			19 7/8	1/8	1ST 30 MIN.
	948		84	20			
	1018	30			19 7/8	1/8	2ND 30 MIN.
	1020		84	20			
	1050	30			19 7/8	1/8	3RD 30 MIN.
	1052		84	20			
	1122	30			19 7/8	1/8	4TH 30 MIN.
	1124		84	20			
	1154	30			19 7/8	1/8	5TH 30 MIN.
	1156		84	20			
	1226	30			19 7/8	1/8	6TH 30 MIN
	1228		84	20			
	1258	30			19 7/8	1/8	7TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361 - CR.

Test Hole No.: I - 2 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 84" Before Test: 84" After Test: 84".

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	106		84	20			
	130	30			19 7/8	1/8	8TH 30 MIN.
	132		84	20			
	202	30			19 7/8	1/8	9TH 30 MIN.
	204		84	20			
	234	30			19 7/8	1/8	10TH 30 MIN.
	236		84	20			
	306	30			19 7/8	1/8	11TH 30 MIN.
	308		84	20			
	338	30			19 7/8	1/8	12TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361 - CR

Test Hole No.: I-3 Tested By: DVG Date: 3/10, 11/2020

Depth of Hole As Drilled: 96" Before Test: 96" After Test: 96"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
							PRESOAK 5 GAL 3/10/2020
	829		96	20			BEGIN 3/11/2020
	854	25			19 7/8	1/8	1ST 25 MIN.
	856		96	20			
	921	25			19 7/8	1/8	2ND 25 MIN.
	923		96	20			
	953	30			19 7/8	1/8	1ST 30 MIN.
	955		96	20			
	1025	30			19 7/8	1/8	2ND 30 MIN.
	1027		96	20			
	1057	30			19 7/8	1/8	3RD 30 MIN.
	1059		96	20			
	1129	30			19 7/8	1/8	4TH 30 MIN.
	1131		96	20			
	1201	30			19 7/8	1/8	5TH 30 MIN.
	1203		96	20			
	1233	30			19 7/8	1/8	6TH 30 MIN.
	1235		96	20			
	105	30			19 7/8	1/8	7TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361-CR

Test Hole No.: I-3 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 96" Before Test: 96" After Test: 96"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	107		96	20			
	137	30			19 7/8	1/8	8TH 30 MIN.
	139		96	20			
	209	30			19 7/8	1/8	9TH 30 MIN.
	211		96	20			
	241	30			19 7/8	1/8	10TH 30 MIN.
	243		96	20			
	313	30			19 7/8	1/8	11TH 30 MIN.
	315		96	20			
	345	30			19 7/8	1/8	12TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361-CR

Test Hole No.: I-4 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 72" Before Test: 72" After Test: 72"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
							PRESOAK 5 GAL. 3/10/2020
	836		72	20			BEGIN 3/11/2020
	901	25			19 7/8	1/8	1ST 25 MIN.
	903		72	20			
	928	25			19 7/8	1/8	2ND 25 MIN.
	930		72	20			
	1000	30			19 7/8	1/8	1ST 30 MIN.
	1002		72	20			
	1032	30			19 7/8	1/8	2ND 30 MIN.
	1034		72	20			
	1104	30			19 7/8	1/8	3RD 30 MIN.
	1106		72	20			
	1136	30			19 7/8	1/8	4TH 30 MIN.
	1138		72	20			
	1208	30			19 7/8	1/8	5TH 30 MIN.
	1210		72	20			
	1240	30			19 7/8	1/8	6TH 30 MIN.
	1242		72	20			
	112	30			19 7/8	1/8	7TH 30 MIN.

PERCOLATION DATA SHEET

Project: 534 WEST STRUCK AVENUE ORANGE Job No.: 2361-CR

Test Hole No.: I-4 Tested By: DVG Date: 3/10,11/2020

Depth of Hole As Drilled: 72" Before Test: 72" After Test: 72"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	114		72	20			
	144	30			19 7/8	1/8	8TH 30 MIN.
	146		72	20			
	216	30			19 7/8	1/8	9TH 30 MIN.
	218		72	20			
	248	30			19 7/8	1/8	10TH 30 MIN.
	250		72	20			
	320	30			19 7/8	1/8	11TH 30 MIN.
	322		72	20			
	352	30			19 7/8	1/8	12TH 30 MIN.

APPENDIX D

GENERAL GRADING GUIDELINES

Proposed Warehouse Facility
City of Orange, Orange County, California
Project No. 2361-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 (2016) and the guidelines presented below.

Preconstruction Meeting

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

Grading Observation and Testing

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

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

Site Clearing

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

Treatment of Existing Ground

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

Fill Placement

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

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

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

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

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

JOB SAFETY

General

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

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

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

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

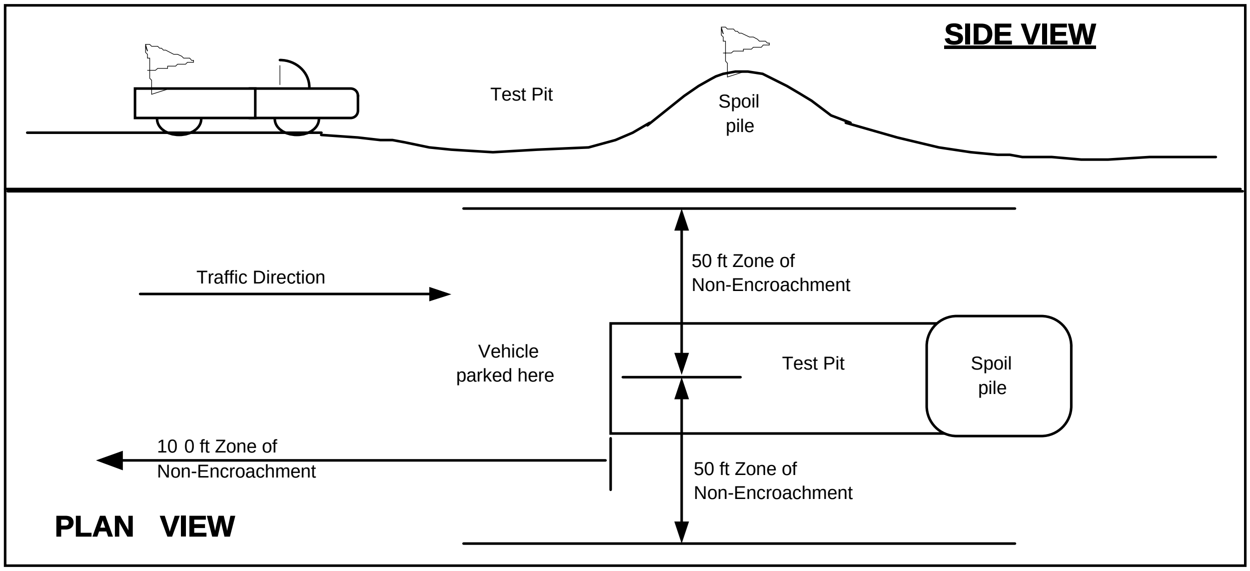
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

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

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

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

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