

**UPDATED GEOTECHNICAL AND INFILTRATION EVALUATION
FOR
PROPOSED 54-UNIT RESIDENTIAL DEVELOPMENT
901 E. KATELA AVENUE
CITY OF ORANGE, ORANGE COUNTY, CALIFORNIA**

**PREPARED FOR

INTRACORP SW, LLC
895 DOVE STREET, SUITE 400
NEWPORT BEACH, CALIFORNIA 926**

**PREPARED BY

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PROJECT No. 3193-CR

JUNE 3, 2022





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June 3, 2022
Project No. 3193-CR

Intracorp SW, LLC
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Attention: Ms. Emilie Simard

Subject: Updated Geotechnical and Infiltration Evaluation
Proposed 54-Unit Residential Development
901 E. Katella Avenue
City of Orange, Orange County, California

Dear Ms. Simard:

We are pleased to provide herein the results of our updated geotechnical and infiltration evaluation for the subject property located in the city of Orange, Orange County, California. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for earthwork, foundation design, and construction. In our opinion, site development is feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development.

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

Respectfully submitted,
GeoTek, Inc.

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Figure 1 – Site Location Map

Figures 2a-b – Exploration Location Maps

Appendix A – Exploratory Boring Logs, Laboratory Test Results, and Infiltration Data by Salem Engineering Group (2019)

Appendix B – Exploratory Boring Logs by GeoTek

Appendix C – Laboratory Test Results by GeoTek

Appendix D – Infiltration Test Data by GeoTek

Appendix E – General Earthwork and Grading Guidelines

I. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to complete an evaluation of the existing geotechnical conditions of the project site with respect to currently anticipated site development. Services provided for this study included the following:

- Research and review of available geologic and general information pertinent to the site,
- Review of the geotechnical report for the property by Salem Engineering Group, Inc. (2019),
- Site reconnaissance,
- Site exploration consisting of the excavation, logging, and sampling of three exploratory hollow-stem auger borings and logging and percolation testing of two hollow-stem auger borings,
- Collection of relatively undisturbed and bulk soil samples of the onsite materials,
- Laboratory testing of the soil samples obtained from the site,
- Review and evaluation of site seismicity,
- Engineering analyses, and
- Compilation of this updated geotechnical and infiltration report which presents our findings, conclusions, and recommendations for site development.

The intent of this report is to aid in the assessment 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. (GeoTek) for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The subject site is located at 901 East Katella Avenue in the city of Orange, Orange County, California. The site is also identified with Orange County Assessor's Parcel Number (APN) 375-



461-41 and consists of a roughly trapezoidal-shaped 2.71-acre property. The parcel is currently occupied by a 20,000 square foot, single-story office building and associated parking lot and other improvements. At the time of our field exploration, the building was vacant.

The site has a generally flat topography with elevations ranging from 229 feet above mean sea level (amsl) to 223 feet amsl descending to the west-southwest. Surface drainage is to the west-southwest.

The site is bounded by residential properties on the north, a commercial development on the east, North Cambridge Street on the west, and East Katella Avenue on the south. The general location of the site is shown in Figure 1. The current and proposed improvements on the site are displayed on Figures 2a-b, Exploration Location Maps.

2.2 PROPOSED DEVELOPMENT

According to the Conceptual Site Plan, prepared by Bassenian Lagoni Architecture and dated February 16, 2022, the property will be developed with 54 residences consisting of two (2) detached single-family dwellings and twenty-six (26) sets of paired single-family dwellings, parking and drive areas, parks, underground utilities, and other improvements. The structures are anticipated to be two to three stories, of wood-framed construction, and will utilize concrete slab-on-grade floors and shallow spread foundations. Cuts and fills to reach design grades are estimated to be minimal. Minor slopes and perimeter/retaining walls are also expected. In addition, we anticipate that stormwater at the site be managed via relatively shallow infiltration systems. Specific locations, depths, and characteristics of the BMP systems are currently unknown.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Final site development plans should be reviewed by GeoTek when they become available. Additional geotechnical field exploration, analyses and recommendations may be necessary upon review of site development plans.

3. REPORT REVIEW

On May 16, 2019, Salem Geotechnical Engineering Group, Inc. (Salem) issued a geotechnical investigation report for the proposed commercial development of the project site. The investigation eight (8) exploratory borings to depths ranging from 5 to 50 feet and three (3)

borings for percolation testing to depths between 5 and 10 feet. Salem's exploration suggested that the site is mostly underlain by medium dense/stiff to very dense/hard alluvial deposits. However, localized areas with undocumented fill up to 15 feet in thickness were also noted. No groundwater was encountered during the site investigation and the potential for liquefaction at the site was considered nil. Salem concluded that the property has surficial soils with medium expansion potential, negligible sulfate concentrations, and poor infiltration rates.

The logs of the exploratory borings, percolation and infiltration test data, and results of laboratory test results by Salem are included in Appendix A. The approximate locations of Salem's explorations are shown on the Exploration Location Maps, Figures 2a-b.

4. FIELD EXPLORATION, LABORATORY TESTING, AND PERCOLATION TESTING

4.1 FIELD EXPLORATION

GeoTek evaluated the subsurface conditions of the site via three (3) exploratory borings excavated on May 5, 2022 to approximately 19 to 22.5 feet below existing grade. In addition, two (2) percolation test borings approximately five (5) feet deep were advanced within the lower portions of the site (west-southwest side) where stormwater BMP systems may be placed. The borings were drilled with a truck-mounted hollow-stem auger drill rig. The approximate locations of our site explorations and past explorations by Salem are shown on the Exploration Location Maps, Figures 2a-b. Logs of the borings are provided in Appendices A and B.

4.2 LABORATORY TESTING

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

4.3 PERCOLATION TESTING

Percolation testing was performed at boring locations I-1 and I-2 to assess the infiltration rates of the site soils underlying potential stormwater system areas. The borings were excavated to approximately five (5) feet below the existing grade, which was assumed to be the approximate invert depth of the infiltration systems. The boring diameter was approximately eight inches.

Percolation testing was performed within the lower approximately 20 inches in the borings by a representative of our firm, in general conformance with the Boring Percolation Test Procedure outlined in the Technical Guidance Document Appendix VII (Orange County, 2013). As required, the percolation rates were corrected to account for discharge of water from both the sides and bottom of the borings. This correction was done using the Porchet Method, obtaining the infiltration rates tabulated below:

SUMMARY OF RESULTS		
Boring	Observed Infiltration Rate, K_{observed} (inches per hour)	Measured Infiltration Rate, K_{measured} (inches per hour)
I-1	4.2	2.1
I-2	0.04	0.02

As recommended by the referenced Technical Guidance Document (Orange County, 2013), a factor of safety of 2.0 was applied to the observed (field) rates to estimate measured infiltration rates. As shown on the table above, poor to favorable infiltration rates (nil to 2 inches per hour) were encountered at the site. Salem (2019) reported relatively similar findings with variable rates ranging from nil to 0.23 inches per hour. Detailed percolation/infiltration test data is included in Appendix D.

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

5. GEOLOGIC AND SOILS CONDITIONS

5.1 REGIONAL SETTING

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

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

More specific to the subject property, Morton, D.M., and Miller, F.K., (2006) map the site to be underlain by old alluvial fan deposits (late to middle Pleistocene age). Additionally, the nearest known active fault to the site is the Elsinore fault- Whittier Section located approximately 7.6 miles to the north.

5.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the earth materials encountered in our explorations and explorations by Salem (2019) is presented in the following sections.

5.2.1 Undocumented Fill

Undocumented fill was encountered in the area of GeoTek's boring B-1 and Salem's boring B-5 beneath the existing pavement section to approximately 9 to 15 feet below grade, respectively. The fill is associated to the construction of the existing site improvements and is generally composed of brown to grayish brown, slightly moist clayey sand and poorly graded to well graded sand with silt and some fine gravel. Because of its variable in-place density (ranging from very loose to very dense) and lack of documentation regarding its placement, the fill is considered to be un-engineered, un-compacted material.

5.2.2 Old Alluvial Fan Deposits

Old alluvial fan deposits were encountered in our borings and borings by Salem below the fill or existing pavement section and extended to the maximum depth explored of about 50 feet. The old alluvial fan deposits consist of interbedded units of clayey sand, silty sand, sandy clay, and poorly graded to well graded sand with silt and some fine gravel. The alluvium was reddish brown to grayish brown in color, slightly moist to moist, and in a medium dense/stiff to very dense/hard in-place state to the total depth explored, based on our field observations, blow counts, and in-place density determinations. Tests conducted on the most unfavorable units of the alluvium (selected based on blow counts) indicated a negligible to slight potential for collapse upon application of water. Test results obtained by Salem suggest similar findings.

The near surface site soils were found to have "very low" to "low" expansion potential based on our laboratory test results in accordance with ASTM D 4829. However, Salem also encountered soil units with "medium" expansion potential.

5.3 SURFACE WATER AND GROUNDWATER

5.3.1 Surface Water

If encountered during the earthwork construction, surface water on this site is the result of precipitation or surface run-off from surrounding sites. Overall drainage in the area is variable, and most commonly directed toward the west-southwest. Provisions for surface drainage will need to be accounted for by the project civil engineer.

5.3.2 Groundwater

Groundwater was not encountered in the deepest site boring, Boring B-5, excavated by Salem (2019) to a maximum depth of 50 feet. The GeoTracker database and the California Water Data Library show several groundwater wells located within 1.5 miles from the site, with a depth to groundwater in excess of 100 feet.

Perched groundwater or localized seepage can occur due to variations in rainfall, irrigation practices, and other factors not evident at the time of this investigation.

5.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an "Alquist-Priolo" Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced liquefaction (CGS, 1997). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced landsliding. The nearest known active fault to the site is the Elsinore fault- Whittier Section located approximately 7.6 miles to the north.

5.4.1 Seismic Design Parameters

The site is located at approximately 33.8097° Latitude and -117.8437° Longitude. Based on the conditions observed in the site excavations and review of regional geologic maps, a Site Class "D" appears to be the appropriate category for the property. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, were determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. As noted using the ASCE 7-16 option on the SEAOC/OSHPD website, the values for S_{M1} and S_{D1} are reported as "null-See Section 11.4.8 (of ASCE 7-16)". As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

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

Assuming that the C_s 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)/ASCE 7-16, are presented on the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.426g
Mapped 1.0 sec Period Spectral Acceleration, S_I	0.507g
Site Coefficient for Site Class “D,” F_a	1.0
Site Coefficient for Site Class “D,” F_v	1.793
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.426g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{MI}	0.908g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.951g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.606g
Site Modified Peak Ground Acceleration, PGA_M	0.658g
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.

5.5 LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT

The project site is not located within an area mapped by the State of California for liquefaction potential. Due to the presence of dense/stiff alluvium and the lack of shallow groundwater, the risk of soil liquefaction at the site is nil.

Loose to medium dense sands tend to densify during strong ground shaking. Based on the blow counts recorded in the deepest site boring (Salem's boring B-5) and utilizing a peak ground acceleration of 0.66g and a modal magnitude weighted (Mw) seismic event of 7.72, we estimate that the seismically induced settlement of the sandy units at the site is less than 0.5 inches total settlement and less than 0.25-inches differential settlement over a 30-foot span.

5.6 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigation. Thus, the potential for landslides is considered negligible.

The site is located about two miles west of the Santiago Creek Recharge Basin. However, as the project site is located at a higher elevation than the basin, the potential for a seiche is considered negligible. The potential for a tsunami is also nil due to the great distance between the site and the ocean.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

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

6.2 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. Site grading plans should be reviewed by this office when they become available. Additional recommendations will likely be offered subsequent to review of these plans.

6.2.1 Site Clearing and Preparation

Site preparation should start with demolition/razing of existing site improvements and removal of deleterious materials and vegetation. Demolition should include removal of all pavements, floor slabs, foundations, and any other below-grade construction. These materials should be properly disposed of off-site. Voids resulting from site clearing (such as removals of underground utilities, foundations, etc.) should be replaced with engineered fill materials.

6.2.2 Removals

Based on the boring data and the field observations, the upper three (3) to fifteen (15) feet of existing site soils or two (2) feet below footing base, whichever is deeper, should be removed in order to provide a homogeneous, dense fill mat for structural support. Deeper removals may be required in some areas to eliminate all undocumented fill and unsuitable alluvium. The bottom of removals should expose competent native alluvial soils which are defined as relatively homogeneous, no visibly porous materials with an in-place density of at least 85 percent of the soil's maximum dry density as determined per ASTM D 1557. As a minimum, removals should extend down and away from foundation elements at a 1:1 (h:v) projection to the recommended removal depth, or a minimum of five feet laterally.

The upper two feet of soil or one foot below pavement subgrade, whichever is deeper, should be removed below asphaltic concrete pavement and Portland cement concrete hardscape areas. Deeper removals may be required in areas containing loose undocumented fills. The horizontal extent of removals should extend at least two feet beyond the edge of the improvements.

The bottom of all removals should be scarified to a minimum depth of 12 inches, brought to slightly above the optimum moisture content, and then recompact to at least 90 percent of the soil's maximum dry density (ASTM D 1557). The bottoms of removals should be observed by a GeoTek representative prior to scarification.

6.2.3 Engineered Fill

The onsite soils are considered suitable for reuse as engineered fill provided they are free from vegetation, roots, and rock/concrete or hard lumps greater than six (6) inches in maximum dimension.

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

The undercut areas should be brought to final pad elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Fill materials should be placed at or above optimum moisture content and should be compacted to a minimum relative compaction of 90 percent as determined by ASTM Test Method D 1557. Additional recommendations pertaining to fill placement are presented in Appendix E.

6.2.4 Excavation Characteristics

Excavation in the onsite soil materials is expected to be easy using heavy-duty grading equipment in good operating conditions.

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 onsite materials should be stable at 1:1 (h:v) inclinations for cuts less than ten (10) feet in height.

6.2.5 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, bulking, and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 10 percent for both the existing fills and upper alluvium may be considered. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction. Bulking is not considered to be a significant factor with the underlying materials within the vicinity of the anticipated construction. Subsidence on the order of up to 0.1-foot could occur.

6.2.6 Trench Excavations and Backfill

Temporary excavations within the onsite materials should be stable at 1:1 (h:v) inclinations for short durations during construction, and where cuts do not exceed ten (10) feet in height. Temporary cuts to a maximum height of four (4) feet can be excavated vertically, but local sloughing and/or failure could occur due to the granular nature of some of the onsite units. Increased caution should be applied when working near or within any excavations at this site.

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

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined per ASTM D 1557). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for road pavements should be compacted to at least 95 percent relative compaction. Much of the onsite materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than six (6) ± inches are removed.

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

6.3 DESIGN RECOMMENDATIONS

6.3.1 Foundation Design Criteria

The site soils are anticipated to have a expansion potential ranging from “very low” to “medium” when tested in accordance with ASTM D 4829. Additional testing of the soils should be performed during construction to evaluate the as-graded conditions. Additional recommendations may be necessary based on the as-graded soils conditions.

A summary of GeoTek’s preliminary conventional foundation design recommendations is presented in the table below:

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONAL REINFORCED SHALLOW FOUNDATIONS			
Design Parameter	“Very Low” Expansion Potential ($0 \leq EI \leq 20$)	“Low” Expansion Potential ($20 \leq EI \leq 50$)	“Medium” Expansion Potential ($51 \leq EI \leq 90$)
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	One-Story – 12 Two-Story – 12 Three-Story – 18	One-Story – 12 Two-Story – 12 Three-Story – 18	One-Story – 18 Two-Story – 18 Three-Story – 24
Minimum Foundation Width (Inches)*	One-Story – 12 Two-Story – 12 Three-Story – 15	One-Story – 12 Two-Story – 12 Three-Story – 15	One-Story – 12 Two-Story – 12 Three-Story – 15
Minimum Slab Thickness (actual)	4 inches (actual)	4 inches (actual)	4 inches (actual)
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab	6" x 6" – W2.9/W2.9 welded wire fabric or No. 3 reinforcing bars at 24 inches on center each way placed in middle of slab	No. 3 reinforcing bars at 18 inches on center each way placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 Reinforcing Bars, one top and one bottom	Two No. 4 Reinforcing Bars, one top and one bottom	Four No. 4 Reinforcing Bars, Two top and two bottom
Effective Plasticity Index**	NA	15	20
Presaturation of Subgrade Soil (Percent of Optimum/Depth in inches)	Minimum 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum 110% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum 120% of the optimum moisture content to a depth of at least 18 inches prior to placing concrete

*Code minimums per Table 1809.7 of the 2019 CBC should be complied with.

**Effective Plasticity Index should be verified at the completion of the rough grading

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

An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings at least 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This allowable soil bearing capacity may be increased by 400 psf for each additional foot of footing depth and 200 psf for each additional foot of footing width to a maximum value of 3,000 psf. An increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).

Structural foundations should be designed in accordance with the 2019 CBC, and to withstand a total static settlement of one-inch and a differential static settlement of one-half of the total

settlement over a horizontal distance of 30 feet. A total seismically induced settlement of approximately 0.5 inches and a differential seismically induced settlement of about 0.25 inches over a horizontal distance of 30 feet are also anticipated.

The passive earth pressure may be computed as an equivalent fluid having a density of 230 psf per foot of depth, to a maximum earth pressure of 2,000 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. The passive pressure and frictional resistance can be combined without reduction. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with concrete or asphalt. The passive pressure and frictional resistance may be combined without reduction.

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

Alternatively, the buildings may rest on post-tensioned slabs. If this foundation option is selected, GeoTek should be contacted to provide geotechnical design parameters.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these systems are provided in the 2019 *California Green Building Standards Code (CALGreen)* Section 4.505.2, the 2019 CBC Section 1907.1, ACI 360R-10, and ACI 302.2R-06.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as the result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the 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. It is GeoTek's opinion that a minimum ten mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. 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 atmospheric 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

permeance) to achieve the desired performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

6.3.2 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.
- Under-slab utility trenches should be compacted to project specifications. Compaction should be achieved with a mechanical compaction device. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

6.3.3 Foundation Set Backs

Foundations should comply with the following setbacks. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The following recommendations are presented:

- The outside bottom edge of all footings should be set back a minimum of $H/2$ (where H is the slope height) from the face of any ascending slope. The setback should be at least five (5) feet and need not to exceed fifteen (15) feet. Where a retaining wall is constructed at the toe of the slope, the height of the slope should be measured from top of the wall to the top of the slope.
- The outside bottom edge of all footings should be set back a minimum of $H/3$ from the face of any descending slope. The setback should be at least seven (7) feet and need not exceed forty (40) feet.
- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom inside edge of the wall foundation.
- The bottom of any planned foundations for structures should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom of the nearest excavation.

6.3.4 Retaining Wall Design and Construction

6.3.4.1 General Design Criteria

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

Retaining wall foundations should be embedded a minimum of 12 inches into “very low” and “low” expansive soils and a minimum of 18 inches into “medium” expansive soils. Foundations should be designed in accordance with Section 6.3.1 of this report. Structural needs may govern and should be evaluated by the project structural engineer.

All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization. The seismic design parameters as discussed in this report remain applicable to all proposed earth retention structures at this site and should be properly incorporated into the design and construction of the structures.

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of Section 6.3.4.4 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 earth retention structure to the base of its 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 and footing 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.

6.3.4.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES		
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) Selected Native Backfill*	Equivalent Fluid Pressure (pcf) Import Granular Backfill**
Level	45	36
2:1	81	53

*The design pressures assume the native backfill material has an expansion index less than or equal to 50 and a friction angle of about 28 degrees or better. 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.

**The design pressures assume that import granular backfill material has an expansion index less than or equal to 20 and a friction angle of at least 34 degrees. 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.

6.3.4.3 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 66 pcf, plus any applicable surcharge loading, for native backfill and level back slope condition. For imported granular backfill, an at-rest equivalent fluid pressure of 57 pcf should be utilized. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

6.3.4.4 Retaining Wall Backfill and Drainage

Retaining wall backfill should be free of deleterious and/or oversized materials and should have properties indicated in Section 6.3.4.2. Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one-cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved

equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining wall backfill should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D 1557. The wall backfill should also include a minimum one-foot wide section of $\frac{3}{4}$ - to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The rock should be separated from the earth with filter fabric. The upper 24 inches should consist of compacted on-site soil.

As an alternative to the drain rock and fabric, Miradrain 2000, or approved equivalent, may be used behind the retaining wall. The Miradrain 2000 should extend from the base of the wall to within two feet of the ground surface. The subdrain should be placed at the base of the wall in direct contact with the Miradrain 2000.

The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. Proper surface drainage needs to be provided and maintained.

6.3.4.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved the project geotechnical engineer or their authorized representative.

6.3.5 Pavement Design Considerations

Pavement design for proposed on-site parking and drive areas was conducted per Caltrans Highway Design Manual guidelines for flexible pavements and the Portland Cement Association for rigid pavements. Based on an assumed design R-value of 20 and for Traffic Indexes (TIs) of 5.0 for car parking areas and 6.0 for drive areas for light traffic with occasional truck traffic, the following preliminary sections were calculated:

PRELIMINARY PAVEMENT SECTIONS		
Traffic Index	Thickness of Flexible Pavement Section	Thickness of Rigid Pavement Section
5.0	3" AC / 7.5" AB	--
6.0	4" AC / 8.5" AB	6.5" PCC/ 4" AB

*AC = Asphalt Concrete, AB = Aggregate Base, PCC = Portland Cement Concrete with a $f'_c > 3,500$ psi.

The TIs used in our 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 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.

No structural reinforcement of the concrete pavements is required. However, temperature and shrinkage control reinforcement should be provided. This reinforcement should consist of No. 4 rebars on 24-inch centers, each way, or equivalent. The reinforcement should be placed at mid-height to the concrete pavement. Also, the concrete pavement should be provided with proper joints to help control cracking. All materials and methods of construction should also conform to the requirements of the City of Orange. These pavement sections should be considered preliminary until reviewed and approved by the City.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinances, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper one foot) after final grading has been completed.

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

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.

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompact to at least 95 percent of the laboratory maximum dry density (ASTM D1557). If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.

6.3.6 Soil Corrosivity

The soil resistivity was tested in the laboratory on two samples collected during our field exploration. The results of the testing (2,750 and 2,814 ohm-cm) indicate that the soil samples are “highly corrosive” to buried ferrous metals, based on the guidelines provided in *Corrosion Basics: An Introduction* (Roberge, 2005). Consideration should be given to consulting with a corrosion engineer.

6.3.7 Soil Sulfate Content

The sulfate content was determined in the laboratory for two soil samples obtained during our field exploration. The results (0.0027 and 0.0103 percent) indicate that the water-soluble sulfate range is less than 0.1 percent by weight which is considered “not applicable” (i.e. negligible) as per Table 4.2.1 of ACI 318. Based upon the test results, no special concrete mix design is required by Code for sulfate attack resistance. Additional testing of soils collected near finish grade should be performed subsequent to site grading.

6.3.8 Import Soils

Import soils should have a “very low” expansion potential. GeoTek also recommends that, as a minimum, proposed import soils be tested for soluble sulfate content. GeoTek should be notified a minimum of 72 hours of potential import sources so that appropriate sampling and laboratory testing can be performed.

6.3.9 Concrete Flatwork

6.3.9.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks, and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required due to the non-structural nature. However, the use of some reinforcement should be considered. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in residential 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 herein.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade with “very low” expansion potential should be pre-saturated to a minimum of 100 percent of optimum moisture content to a depth of at least 12 inches. Soils with “low” and “medium” expansion potentials should be pre-saturated to a minimum of 110 and 120 percent of optimum moisture content, respectively. Presaturation depth should be at least 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 and a City Inspector, if necessary.

6.3.9.2 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 can also undergo 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 also subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are.

GeoTek suggests that control joints be placed in two directions and located a distance apart roughly equal to 24 to 36 times the slab thickness.

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered “non-structural” components. We suggest that the same standards of care be applied to these features as to the structure itself.

6.4 POST CONSTRUCTION CONSIDERATIONS

6.4.1 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. The soils should be maintained in a solid to semi-solid state as defined by the materials Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased 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 foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas.

6.4.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. 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. Pad drainage should be directed toward approved area(s) and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

6.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading plans, pool plans, retaining wall plans, foundation plans, and relevant project specifications be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek 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 onsite and import materials for fill placement and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trenches.
- Perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

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

7. INTENT

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

The scope of our evaluation is limited to the boundaries of the subject site. 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 the



client's needs, our fee estimate (P-04010622-CR) dated April 20, 2022 and geotechnical engineering standards normally used on similar projects in this region.

8. LIMITATIONS

The materials observed on the project site appear to be representative of the area; however, soil materials vary in character between excavations or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusion 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 is expressed or implied. Standards of practice are subject to change with time.

9. SELECTED REFERENCES

- ASCE, 2017, "Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-16".
- ASTM, 2011, "Soil and Rock: American Society for Testing and Materials", vol. 4.08 and 4.09.
- Bassenian Lagoni Architecture, 2022, "Conceptual Site Plan, 901 E. Katella Avenue, Orange, California," dated February 16.
- Bryant, W.A., and Hart, E.W., 2007, "Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps", California Geological Survey: Special Publication 42.
- California Code of Regulations, Title 24, 2019 "California Building Code", 2 volumes.
- California Department of Conservation, 1997, "Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles, Orange County, California", Seismic Hazard Report 3.
- Morton, D.M. and Miller, F.K., 2006, "Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangles, California," US Geological Survey OF 2006-1217, scale 1:100,000.

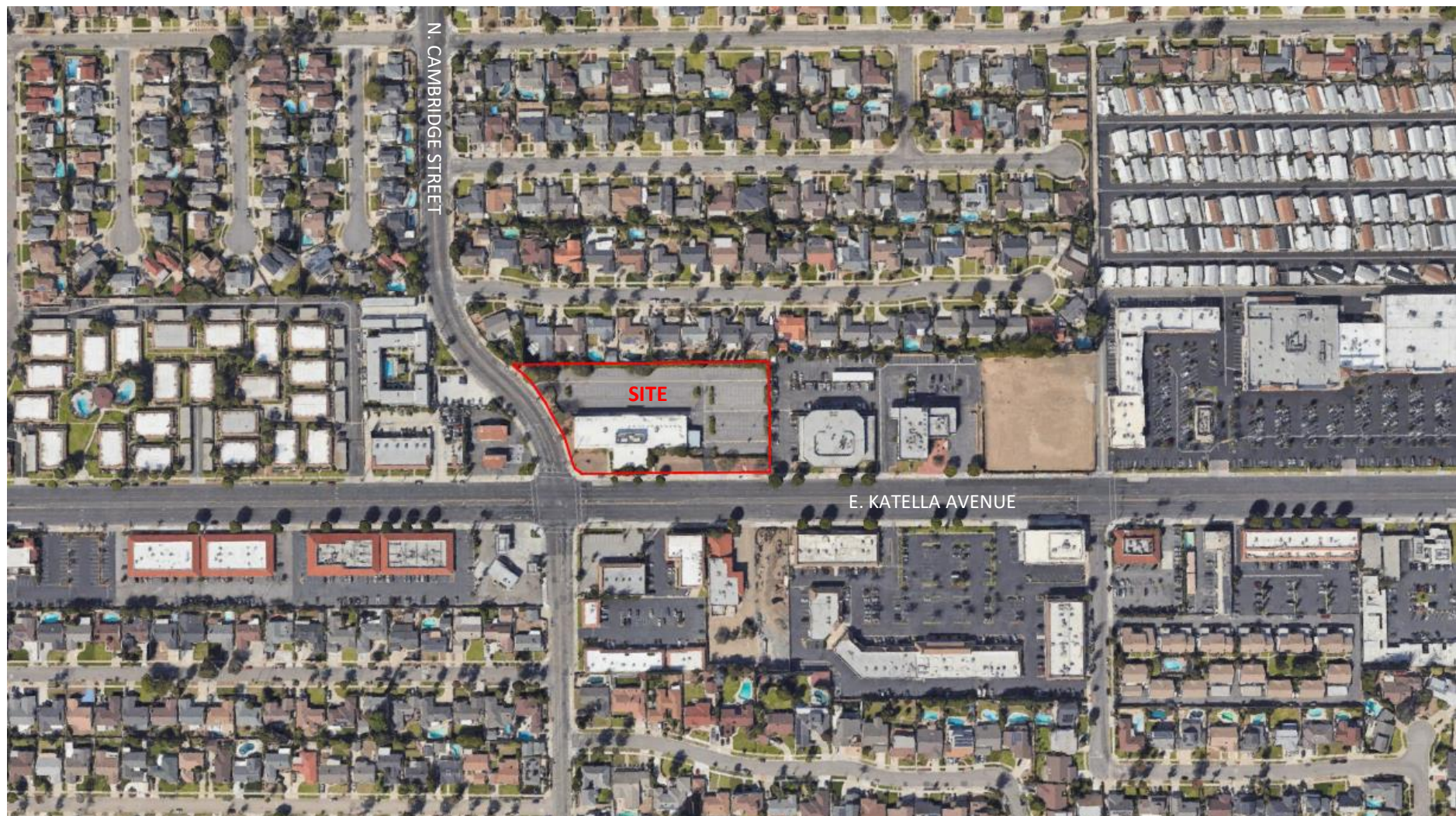


Orange County, 2013, "Technical Guidance Document Appendix VII".

Roberge, P.R., 2005, "Corrosion Basics: An Introduction", 2nd Edition.

Salem Engineering Group, Inc., 2019, "Geotechnical Engineering Investigation, Proposed Commercial Development, 901 E. Katella Avenue, Orange, California," Project No. 3-219-0272, dated May 16.

Seismic Design Values for Buildings (<http://earthquake.usgs.gov/research/hazmaps/design>).



0 400 ft

Intracorp SW, LLC
 901 E Katella Avenue
 Orange, Orange County, California

Project No. 3193-CR



Figure I
 Site Location Map





Intracorp SW, LLC
 901 E Katella Avenue
 Orange, Orange County, California

GeoTek Project No. 3193-CR

Scale As Shown



Figure 2

Boring and Infiltration
 Test Location Map





Intracorp SW, LLC
901 E Katella Avenue
Orange, Orange County, California

Project No. 3193-CR



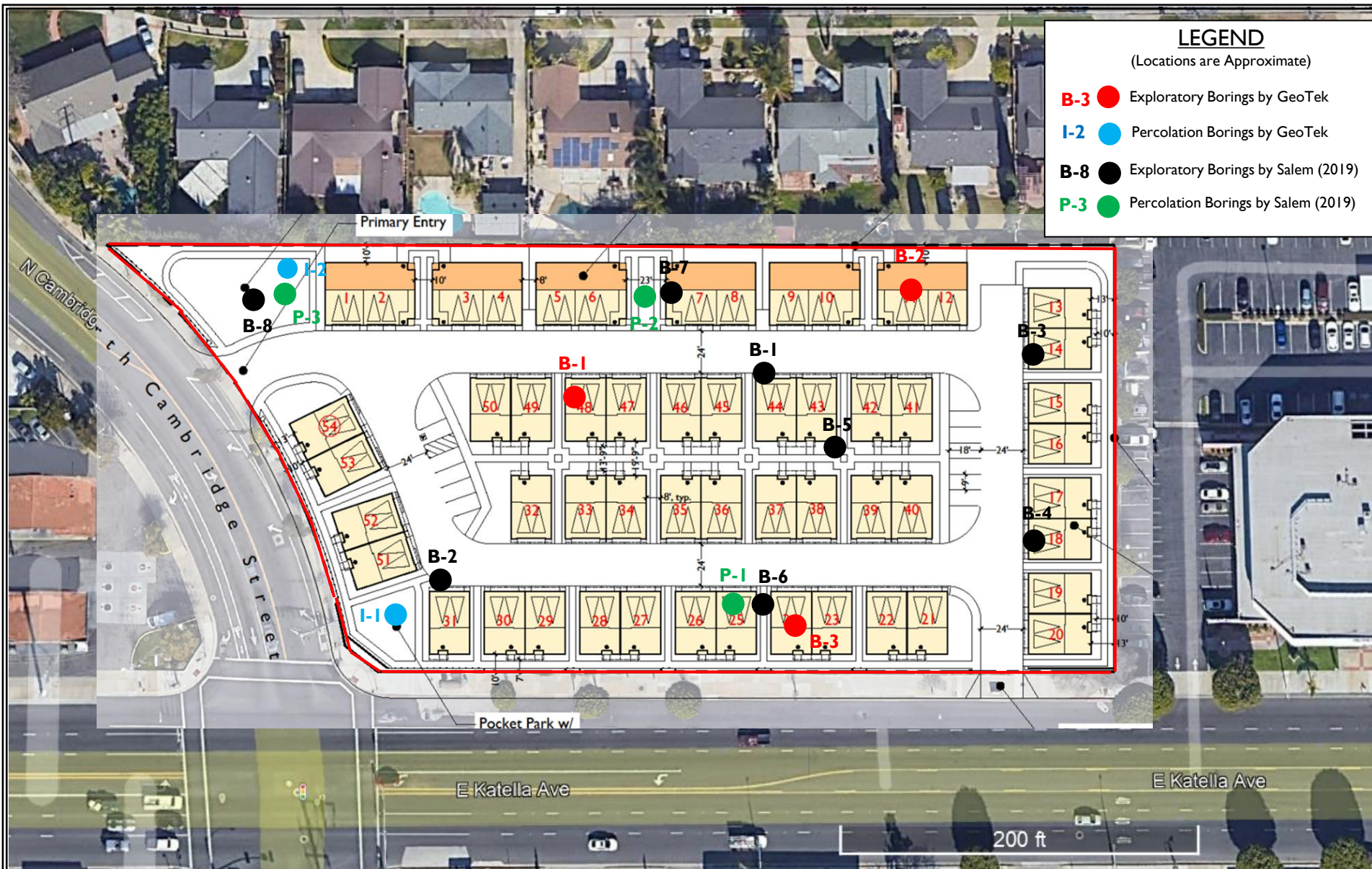
Figure 2a
Exploration Location
Map



LEGEND

(Locations are Approximate)

- B-3** ● Exploratory Borings by GeoTek
- I-2** ● Percolation Borings by GeoTek
- B-8** ● Exploratory Borings by Salem (2019)
- P-3** ● Percolation Borings by Salem (2019)



Intracorp SW, LLC
 901 E Katella Avenue
 Orange, Orange County, California

Scale As Shown



Figure 2b

Exploration Location
 Map



Project No. 3193-CR

APPENDIX A

EXPLORATORY BORING LOGS, LABORATORY TEST RESULTS, AND INFILTRATION DATA BY SALEM ENGINEERING GROUP (2019)

**Updated Geotechnical and Infiltration Evaluation
901 E. Katella Avenue, Orange, California
Project No. 3193-CR**



Boring No. B-1

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-1
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Asphalt Concrete = 3.5 in							
		Aggregate Base = 6.5 in							
		Sandy CLAY (CL) Very stiff; moist; reddish brown; fine grain sand.	121.3	12.7	MCS		28		
5		Clayey SAND (SC) Dense; moist; grayish brown; fine to coarse grain sand; with fine gravel.	125.2	7.9	MCS		49		
10		Grades as above; slightly moist.	115.9	5.0	MCS		53		
15		Grades as above; medium dense.	-	4.4	SPT		27		
20		Poorly graded SAND with Silt (SP-SM) Dense; slightly moist; grayish brown; fine to coarse grain sand; with fine gravel.	-	4.3	SPT		31		
25		Sandy CLAY (CL) Stiff; moist; reddish brown; fine grain sand.	-	11.9	SPT		9		

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 2

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-1

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-1
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
30		Grades as above; hard; grayish brown; with fine gravel.	-	6.0	SPT		64		
35		Well-graded SAND with Silt (SW-SM) Very dense; slightly moist; grayish brown; fine to coarse grain sand; with fine gravel.	-	5.6	SPT		61		
40		Grades as above.	-	4.2	SPT		54		
45		Grades as above; with fine to coarse gravel; with some cobbles.	-	-	SPT		50		
50		Grades as above.	-	4.7	SPT		50		
		End of Borehole							

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 2 of 2

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-2

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-2
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Sandy CLAY (CL) Stiff; moist; reddish brown; fine grain sand.	113.6	9.4	MCS		19		
5		Clayey SAND (SC) Dense; slightly moist; reddish brown; fine to coarse grain sand; with fine gravel.	117.9	4.9	MCS		59		
10		Silty SAND (SM) Medium dense; moist; grayish brown; fine to coarse grain sand; trace clay; with fine gravel.	-	6.0	SPT		26		
15		Clayey SAND (SC) Dense; moist; reddish brown; fine to coarse grain sand; with fine gravel.	-	7.6	SPT		38		
		End of Borehole							
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-3

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-3
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Asphalt Concrete = 4 in							
		Sandy CLAY (CL) Very stiff; moist; reddish brown; fine grain sand.	119.4	12.3	MCS		28		
5		Clayey SAND (SC) Very dense; slightly moist; reddish brown; fine to coarse grain sand; with fine gravel.	106.6	5.1	MCS		74		
10		Grades as above; dense; moist.	-	9.5	SPT		31		
15		Grades as above; very dense; slightly moist.	-	4.1	SPT		56		
		End of Borehole							
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-4

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-4
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Asphalt Concrete = 2.75 in							
		Aggregate Base = 4 in							
		Sandy CLAY (CL) Very stiff; moist; reddish brown; fine grain sand.	120.5	12.9	MCS		26		
5		Clayey SAND (SC) Very dense; slightly moist; reddish brown; fine to coarse grain sand; with fine gravel.	126.3	5.8	MCS		96		
10		Silty SAND (SM) Dense; slightly moist; grayish brown; fine to coarse grain sand; with fine gravel.	-	5.1	SPT		46		
15		Grades as above; medium dense; trace clay.	-	5.3	SPT		27		
		End of Borehole							
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-5

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-5
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Asphalt Concrete = 3 in							
		Aggregate Base = 4 in							
		Well-graded SAND with Silt (SW-SM) Medium dense; moist; grayish brown; fine to medium grain sand. Possible FILL.	108.8	5.6	MCS		22		
5		Well-graded SAND (SW) Very loose; slightly moist; grayish brown; fine to coarse grain sand; with fine gravel. Possible FILL.	107.0	3.2	MCS		3		
10		Poorly graded SAND (SP) Very loose; slightly moist; grayish brown; fine to coarse grain sand; trace clay; with fine to coarse gravel. Possible FILL.	-	3.1	SPT		2		
15		Grades as above; medium dense.	-	5.4	SPT		29		
		End of Borehole							
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-6

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-6
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Sandy CLAY (CL) Very stiff; moist; reddish brown; fine grain sand.	111.4	8.5	MCS		23		
5		Clayey SAND (SC) Dense; moist; reddish brown; fine to coarse grain sand.	-	8.1	SPT		33		
10		Grades as above.	-	6.2	SPT		35		
		End of Borehole							
15									
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-7

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-7
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface							
		Asphalt Concrete = 6 in							
		Aggregate Base = 12 in							
		Sandy CLAY (CL) Very stiff; moist; reddish brown; fine grain sand.	121.0	12.9	MCS		29		
5		Clayey SAND (SC) Dense; moist; grayish brown; fine to coarse grain sand; with fine gravel.	-	5.1	SPT		44		
10		Silty SAND (SM) Dense; moist; grayish brown; fine to coarse grain sand; trace clay; with fine gravel.	-	4.7	SPT		32		
		End of Borehole							
15									
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Boring No. B-8

Project: Proposed Commercial Development
Client: The Festival Companies
Location: 901 E. Katella Avenue, Orange, California
Grnd. Surf. Elev. (Ft. MSL) -

Project No: 3-219-0272
Figure No.: A-8
Logged By: JH
Initial: -
At Completion: -

Depth to Water>

SUBSURFACE PROFILE			SAMPLE					Penetration Test	Water Level
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture Content (%)	Sampler Type	Penetration	Blow Count		
0		Ground Surface						20 40 60 80	
		Asphalt Concrete = 3 in							
		Aggregate Base = 6 in							
		Sandy CLAY (CL)	125.0	10.7	MCS		46		
		Hard; moist; reddish brown; fine grain sand.							
5		Grades as above; with fine gravel.	-	11.1	SPT		41		
		End of Borehole							
10									
15									
20									
25									

Drill Method: Solid Flight
Drill Rig: CME 55
Driller: SALEM
Sheet: 1 of 1

Drill Date: 04/25/19
Borehole Size: 6 in
Hammer Type: Automatic Trip
Weight & Drop: 140 lb/30 in



Percolation Test Worksheet

Project: Proposed Commercial Development
901 E. Katella Avenue
Orange, California

Job No.: 3-219-0272
Date Drilled: 4/25/2019
Soil Classification: Clayey SAND (SC)

Hole Radius: 4 in.

Pipe Dia.: 3 in.

Total Depth of Hole: 120 in.

Test Hole No.: P-1

Presoaking Date: 4/25/2019

Tested by: JH

Test Date: 4/26/2019

Drilled Hole Depth: 10 ft.

Pipe Stick up: 0.2 ft.

Time Start	Time Finish	Depth of Test Hole (ft) [#]	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level [#] (ft)	Final Water Level [#] (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
8:00	8:30	10.2	Y	0:30	9.12	9.15	0.36	30	83.3	13.0	12.6	12.8	0.10
8:30	9:00	10.2	N	0:30	9.15	9.18	0.36	30	83.3	12.6	12.2	12.4	0.10
9:00	9:30	10.2	N	0:30	9.18	9.21	0.36	30	83.3	12.2	11.9	12.1	0.10
9:30	10:00	10.2	N	0:30	9.21	9.24	0.36	30	83.3	11.9	11.5	11.7	0.11
10:00	10:30	10.2	N	0:30	9.24	9.27	0.36	30	83.3	11.5	11.2	11.3	0.11
10:30	11:00	10.2	N	0:30	9.27	9.29	0.24	30	125.0	11.2	10.9	11.0	0.07
11:00	11:30	10.2	N	0:30	9.29	9.31	0.24	30	125.0	10.9	10.7	10.8	0.08
11:30	12:00	10.2	N	0:30	9.31	9.32	0.12	30	250.0	10.7	10.6	10.6	0.04
12:00	12:30	10.2	N	0:30	9.32	9.33	0.12	30	250.0	10.6	10.4	10.5	0.04
12:30	13:00	10.2	N	0:30	9.33	9.34	0.12	30	250.0	10.4	10.3	10.4	0.04
13:00	13:30	10.2	N	0:30	9.34	9.35	0.12	30	250.0	10.3	10.2	10.3	0.04
13:30	14:00	10.2	N	0:30	9.35	9.36	0.12	30	250.0	10.2	10.1	10.1	0.04
Recommended for Design:										Infiltration Rate		0.04	

Percolation Test Worksheet

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

Project: Proposed Commercial Development		Job No.: 3-219-0272	
901 E. Katella Avenue		Date Drilled: 4/25/2019	
Orange, California		Soil Classification: Silty SAND (SM) w/Gravel, trace Clay	Hole Radius: <u>4</u> in.
			Pipe Dia.: <u>3</u> in.
Test Hole No.: P-2		Presoaking Date: 4/25/2019	Total Depth of Hole: <u>120</u> in.
Tested by: JH		Test Date: 4/26/2019	
Drilled Hole Depth: 10	ft.		Pipe Stick up: <u>0.5</u> ft.

[illegible]

Recommended for Design:	Infiltration Rate	0.23
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Recommended for Design:	Infiltration Rate	0.23
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Recommended for Design:	Infiltration Rate	0.23
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Percolation Test Worksheet

Project: Proposed Commercial Development
901 E. Katella Avenue
Orange, California

Job No.: 3-219-0272
Date Drilled: 4/25/2019
Soil Classification: Sandy CLAY (CL)

Hole Radius: 4 in.

Pipe Dia.: 3 in.

Total Depth of Hole: 60 in.

Test Hole No.: P-3

Presoaking Date: 4/25/2019

Tested by: JH

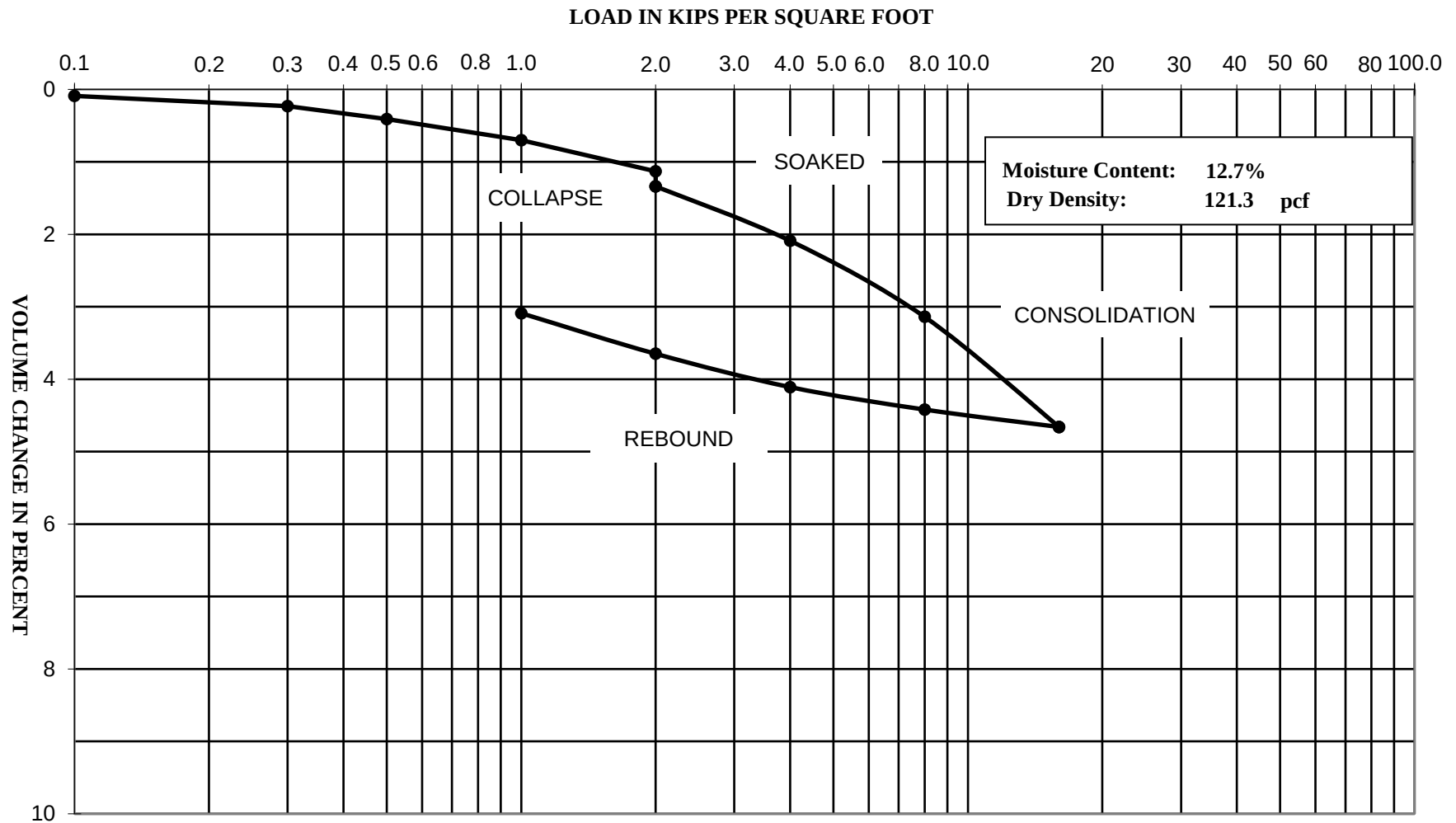
Test Date: 4/26/2019

Drilled Hole Depth: 5 ft.

Pipe Stick up: 1.25 ft.

Time Start	Time Finish	Depth of Test Hole (ft) [#]	Refill- Yes or No	Elapsed Time (hrs:min)	Initial Water Level [#] (ft)	Final Water Level [#] (ft)	Δ Water Level (in.)	Δ Min.	Meas. Perc Rate (min/in)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Infiltration Rate, It (in/hr)
7:50	8:20	6.3	Y	0:30	4.94	4.95	0.12	30	250.0	15.7	15.6	15.7	0.03
8:20	8:50	6.3	N	0:30	4.95	4.96	0.12	30	250.0	15.6	15.5	15.5	0.03
8:50	9:20	6.3	N	0:30	4.96	4.97	0.12	30	250.0	15.5	15.4	15.4	0.03
9:20	9:50	6.3	N	0:30	4.97	4.98	0.12	30	250.0	15.4	15.2	15.3	0.03
9:50	10:20	6.3	N	0:30	4.98	4.99	0.12	30	250.0	15.2	15.1	15.2	0.03
10:20	10:50	6.3	N	0:30	4.99	5.00	0.12	30	250.0	15.1	15.0	15.1	0.03
10:50	11:20	6.3	N	0:30	5.00	5.01	0.12	30	250.0	15.0	14.9	14.9	0.03
11:20	11:50	6.3	N	0:30	5.01	5.02	0.12	30	250.0	14.9	14.8	14.8	0.03
11:50	12:20	6.3	N	0:30	5.02	5.03	0.12	30	250.0	14.8	14.6	14.7	0.03
12:20	12:50	6.3	N	0:30	5.03	5.04	0.12	30	250.0	14.6	14.5	14.6	0.03
12:50	13:20	6.3	N	0:30	5.04	5.05	0.12	30	250.0	14.5	14.4	14.5	0.03
13:20	13:50	6.3	N	0:30	5.05	5.06	0.12	30	250.0	14.4	14.3	14.3	0.03
Recommended for Design:										Infiltration Rate		0.03	

CONSOLIDATION - PRESSURE TEST DATA ASTM D2435



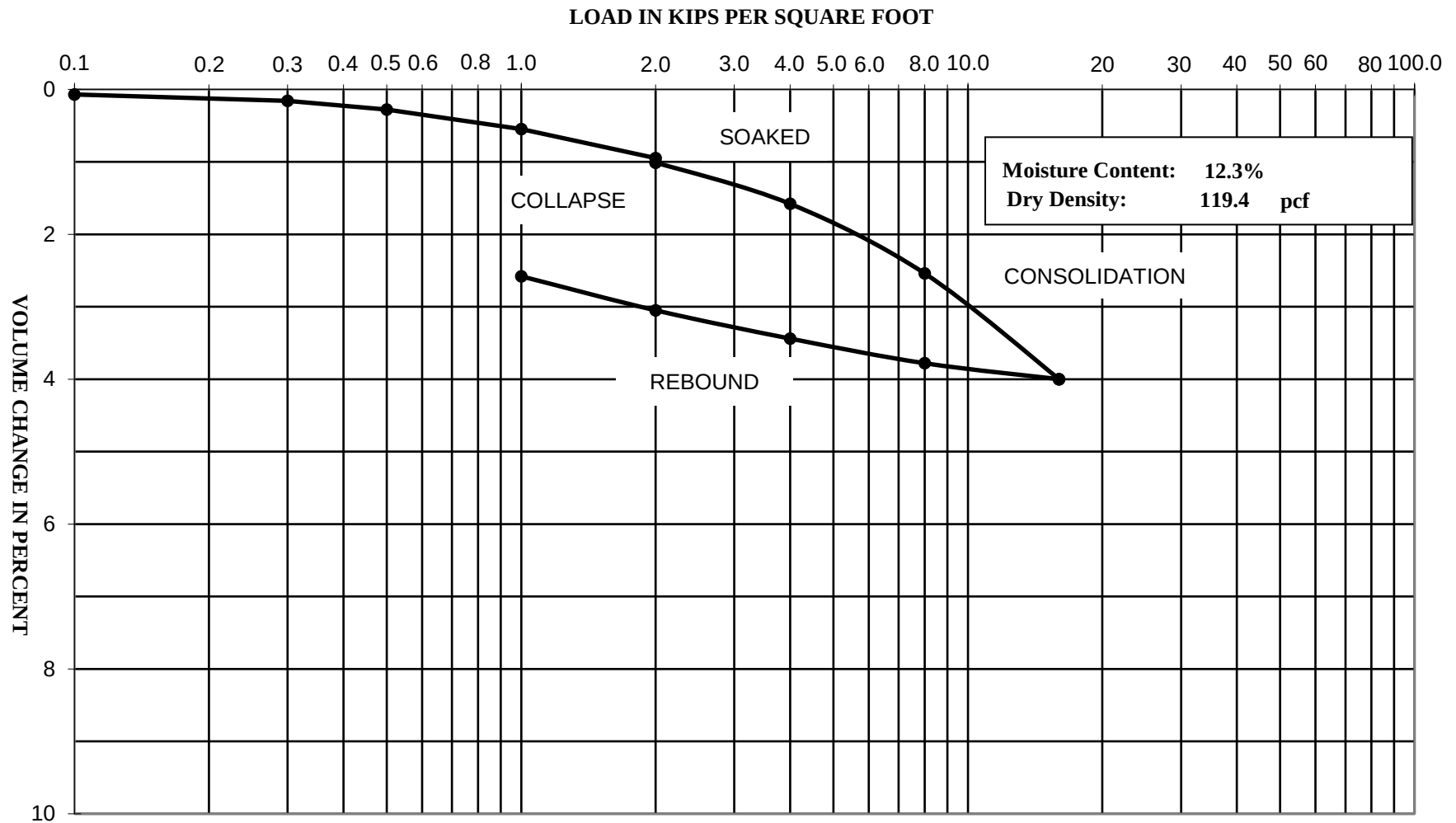
Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Boring: B-1 @ 2'

CONSOLIDATION - PRESSURE TEST DATA

ASTM D2435

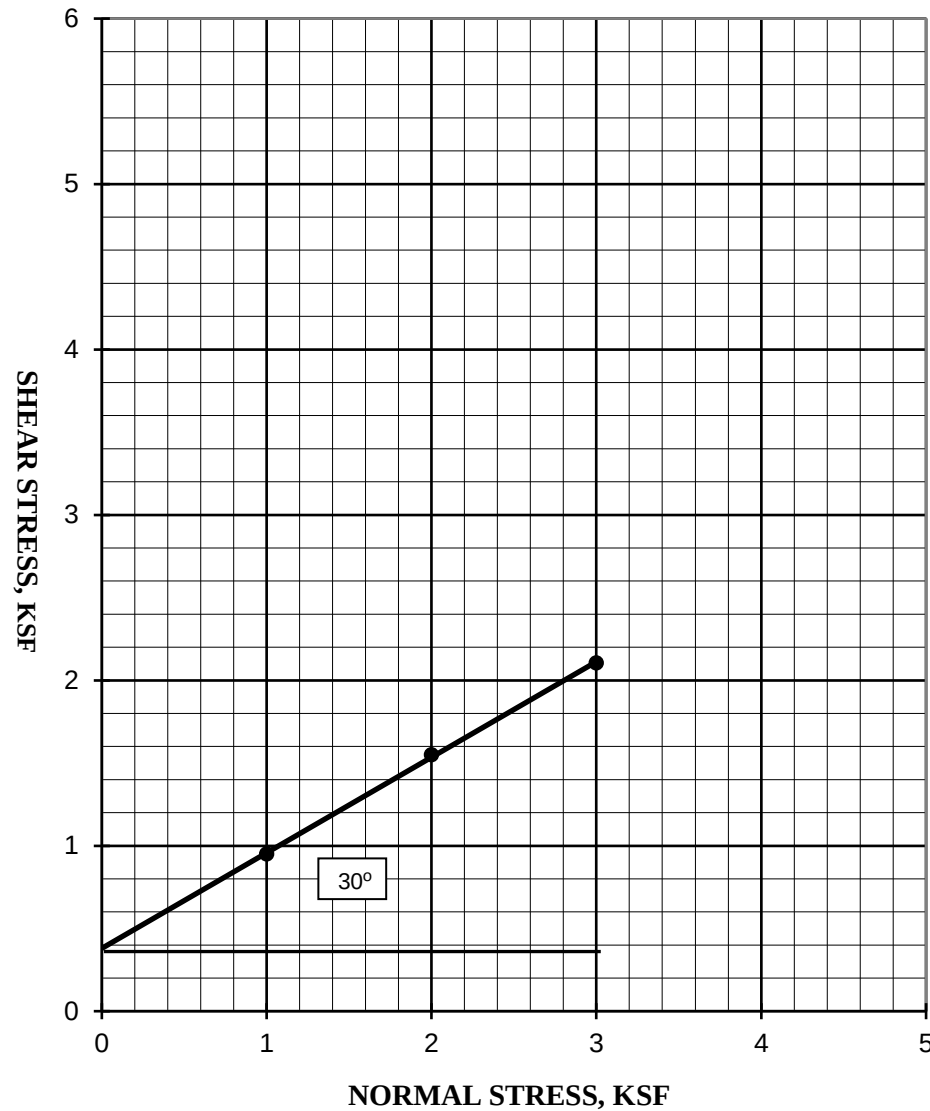


Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Boring: B-3 @ 2'

**SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM D - 3080**



Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

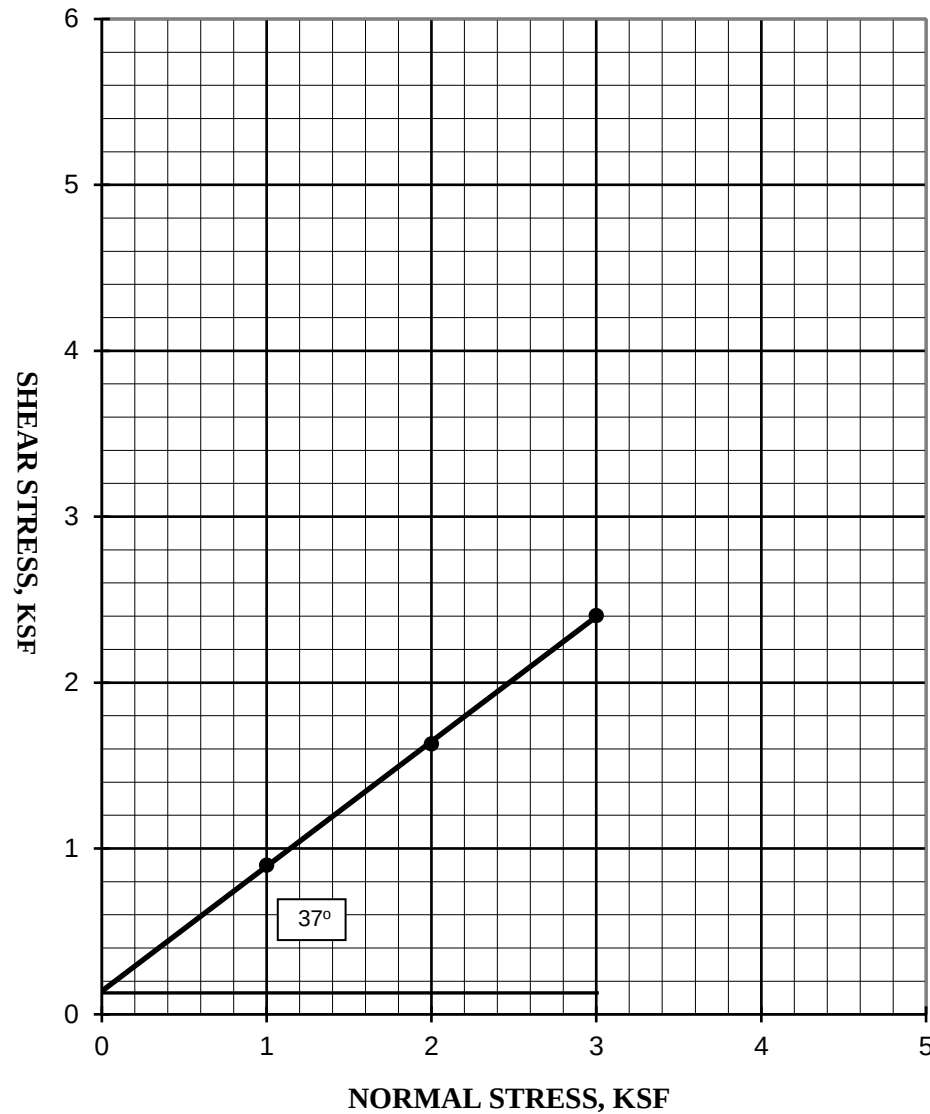
Boring: B-1 @ 2'
Soil Type: Sandy CLAY (CL)

Friction Angle: 30.0 degrees
Cohesion: 380 psf

Moisture Content 12.7%

Dry Density 121.3 pcf

**SHEAR STRENGTH DIAGRAM
(DIRECT SHEAR)
ASTM D - 3080**



Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Boring: B-5 @ 2'

Soil Type: Well-graded SAND with Silt (SW-SM)

Friction Angle: 37.0 degrees

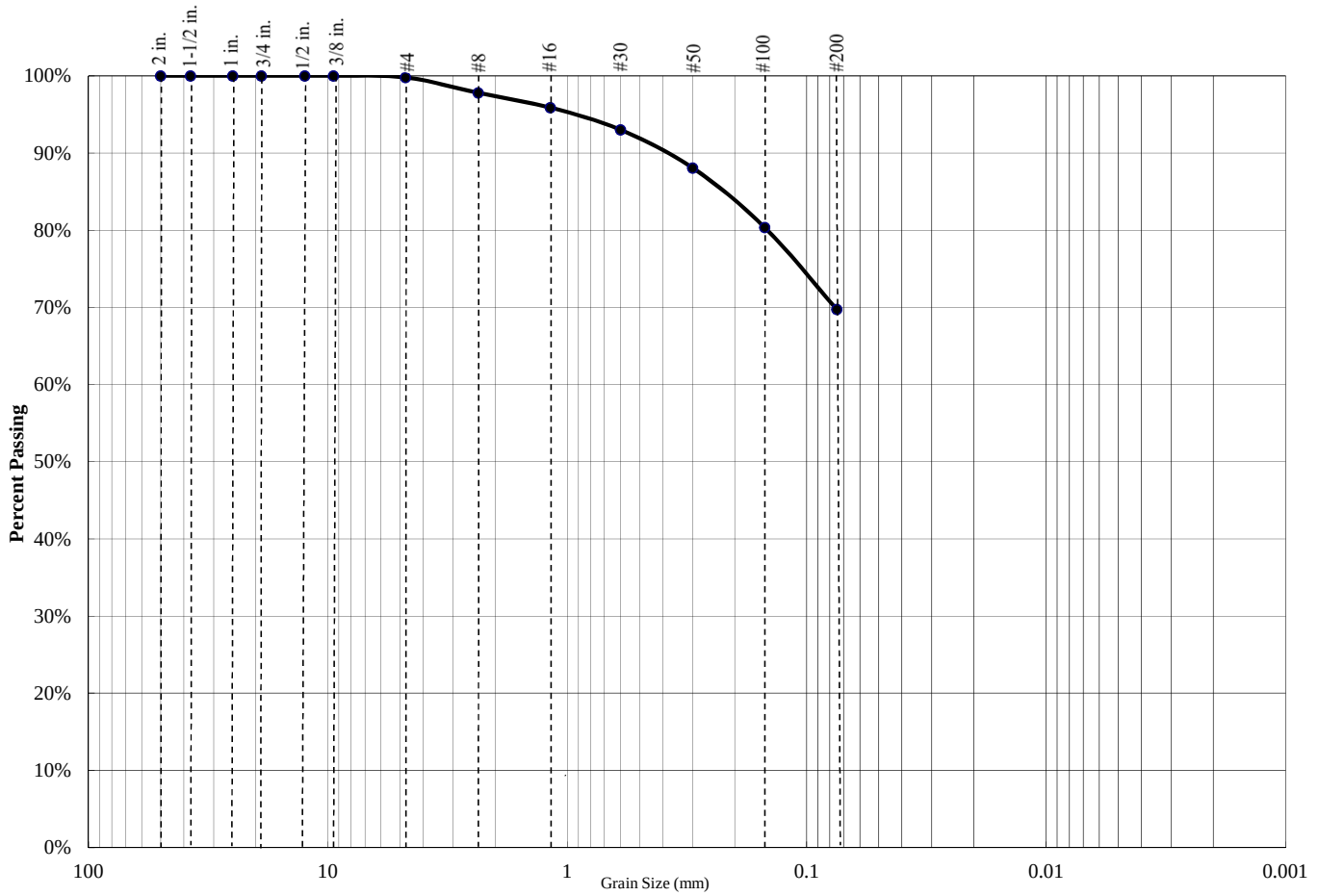
Cohesion: 140 psf

Moisture Content 5.6%

Dry Density 108.8 pcf

PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	30%	70%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.8%
#8	97.8%
#16	95.9%
#30	93.0%
#50	88.1%
#100	80.4%
#200	69.7%

Atterberg Limits		
PL=	LL=	PI=

Coefficients		
D ₈₅ =	D ₆₀ =	D ₅₀ =
D ₃₀ =	D ₁₅ =	D ₁₀ =
C _u =	N/A	C _c = N/A

USCS CLASSIFICATION
Sandy CLAY (CL)

Project Name: Proposed Commercial - Orange, Ca

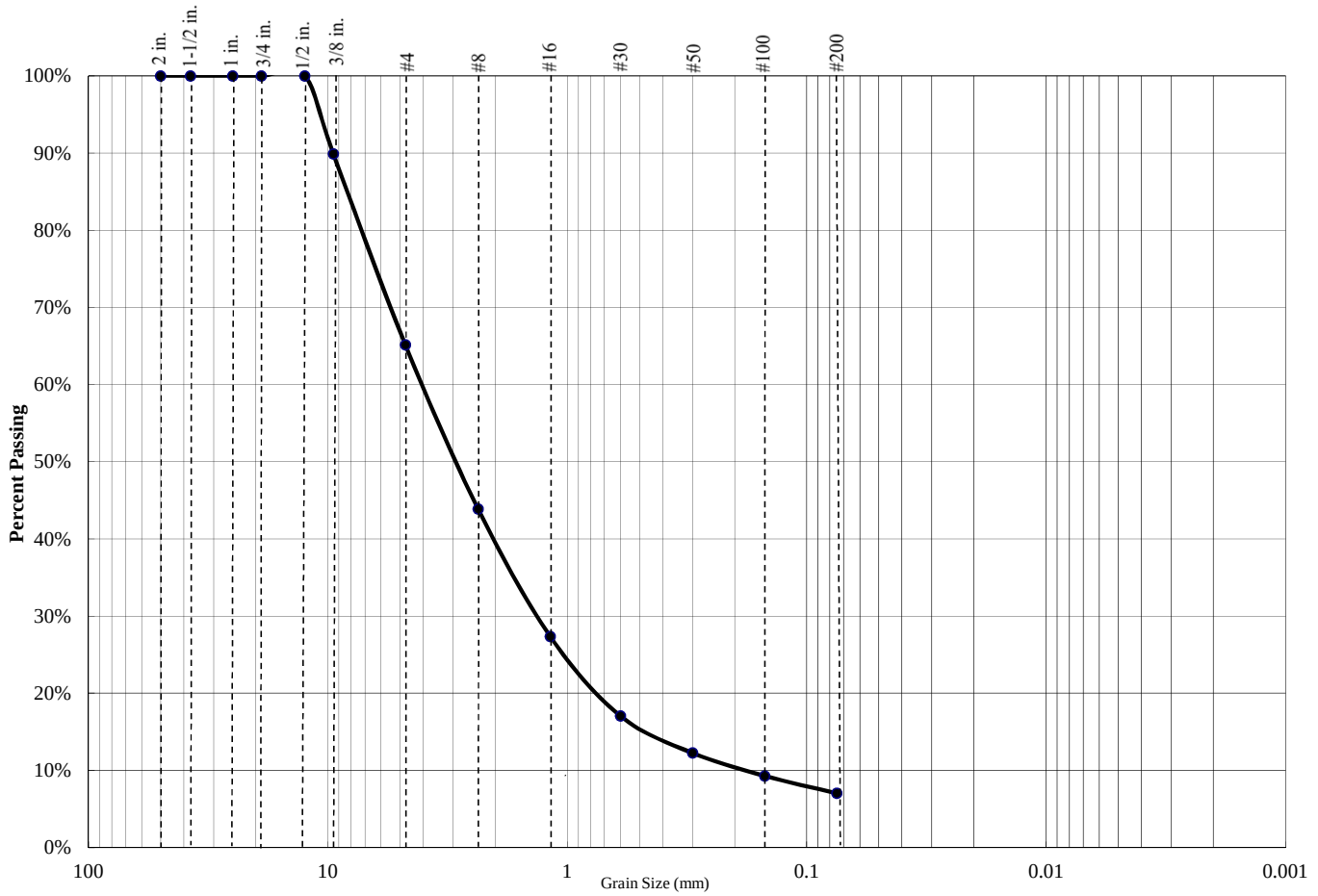
Project Number: 3-219-0272

Boring: B-1 @ 2'



PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
35%	58%	7%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	89.9%
#4	65.1%
#8	43.9%
#16	27.4%
#30	17.1%
#50	12.3%
#100	9.3%
#200	7.0%

Atterberg Limits		
PL=	LL=	PI=

Coefficients			
D85=	D60=	4	D50=
D30=	1.5	D15=	D10= 0.2
C _u =	20.00	C _c =	2.81

USCS CLASSIFICATION
Poorly graded SAND with Silt (SP-SM)

Project Name: Proposed Commercial - Orange, Ca

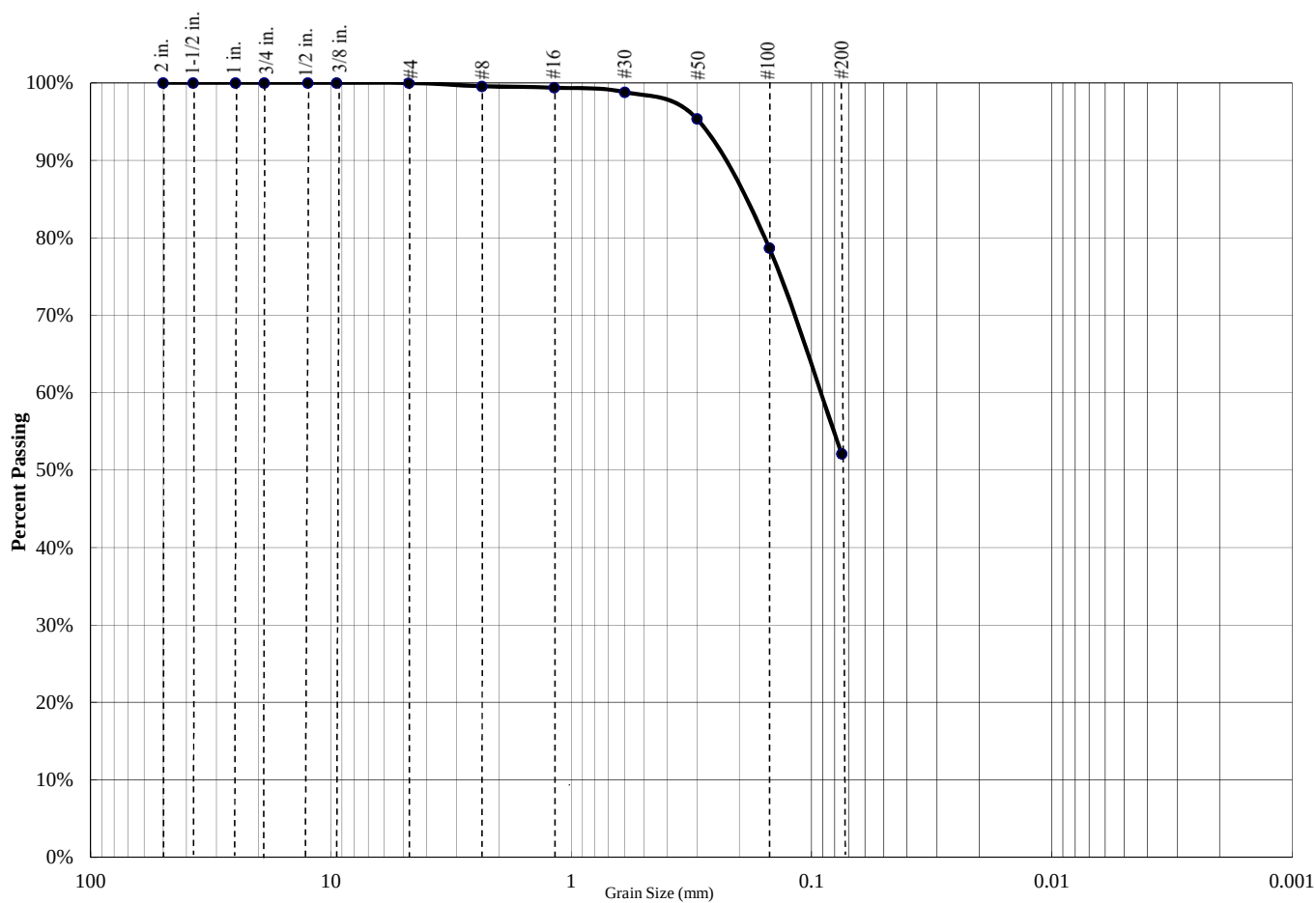
Project Number: 3-219-0272

Boring: B-1 @ 20'



PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
0%	48%	52%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	100.0%
#8	99.6%
#16	99.4%
#30	98.8%
#50	95.3%
#100	78.7%
#200	52.1%

Atterberg Limits		
PL=	LL=	PI=

Coefficients		
D ₈₅ =	D ₆₀ =	D ₅₀ =
D ₃₀ =	D ₁₅ =	D ₁₀ =
C _u =	N/A	C _c = N/A

USCS CLASSIFICATION
Sandy CLAY (CL)

Project Name: Proposed Commercial - Orange, Ca

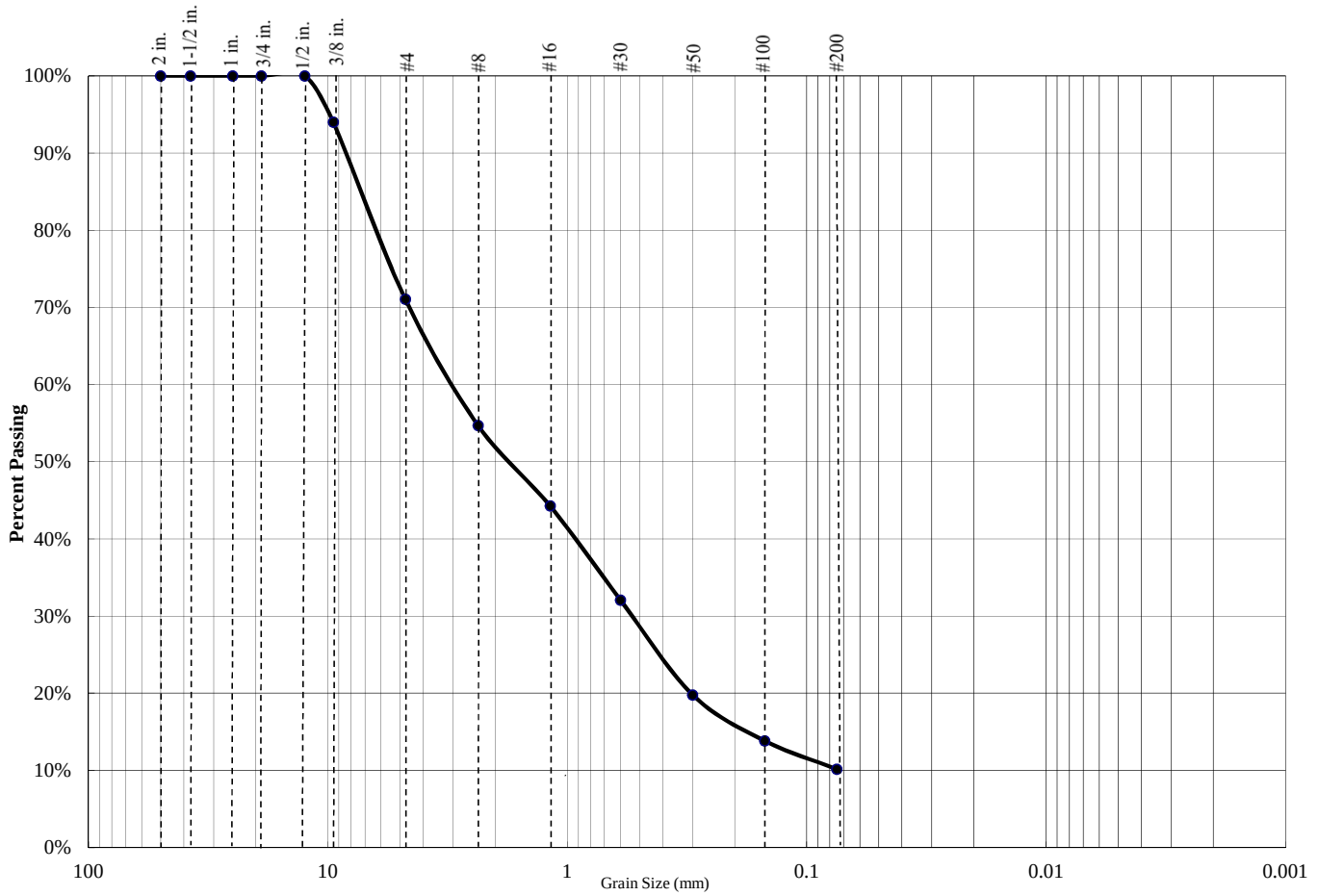
Project Number: 3-219-0272

Boring: B-1 @ 25'



PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
29%	61%	10%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	94.0%
#4	71.1%
#8	54.7%
#16	44.3%
#30	32.1%
#50	19.8%
#100	13.8%
#200	10.2%

Atterberg Limits		
PL=	LL=	PI=

Coefficients		
D ₈₅ =	D ₆₀ = 3	D ₅₀ =
D ₃₀ = 0.55	D ₁₅ =	D ₁₀ = 0.075
C _u = 40.00	C _c = 1.34	

USCS CLASSIFICATION
Well-graded SAND with Silt (SW-SM)

Project Name: Proposed Commercial - Orange, Ca

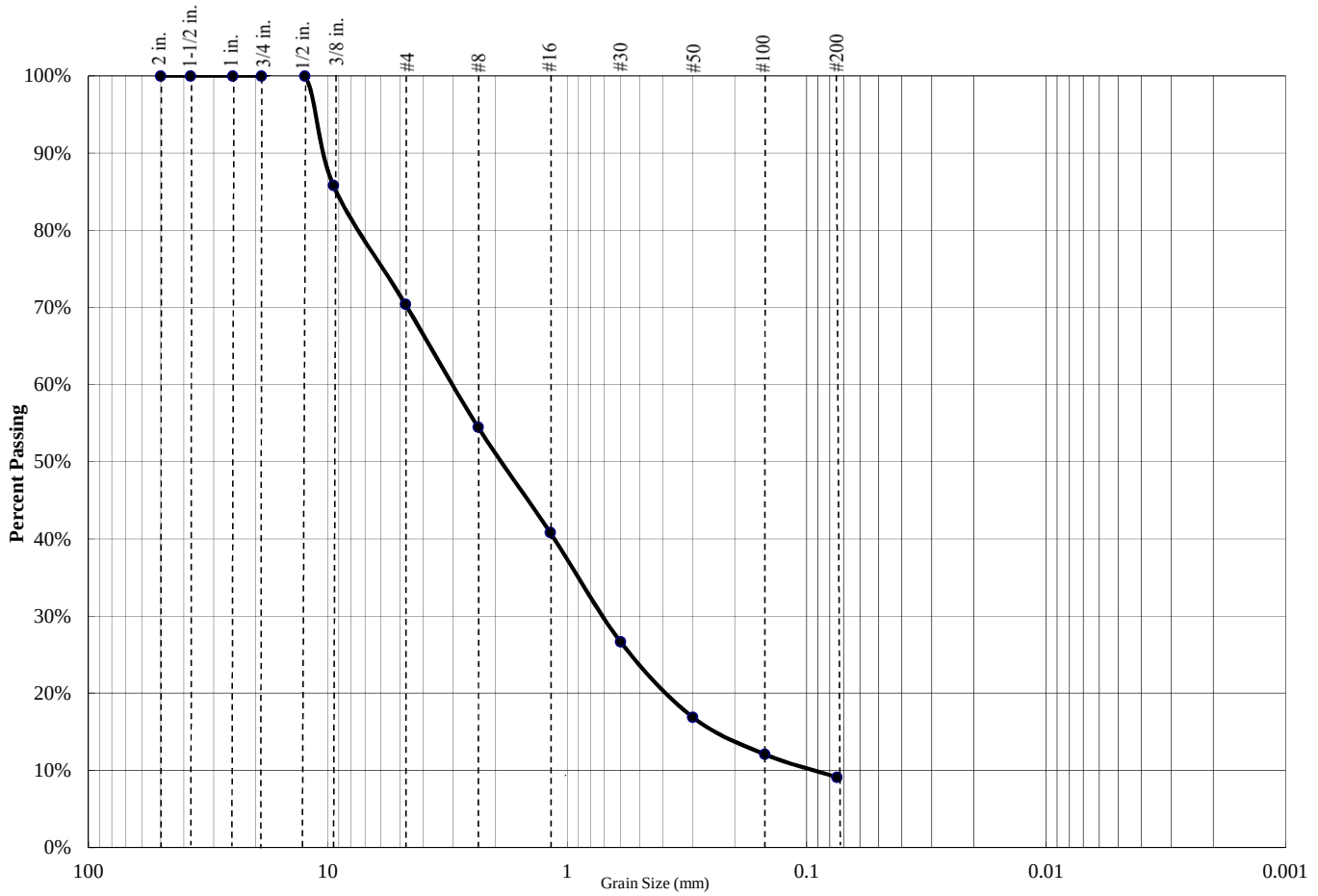
Project Number: 3-219-0272

Boring: B-1 @ 35'



PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
30%	61%	9%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	85.8%
#4	70.4%
#8	54.5%
#16	40.8%
#30	26.7%
#50	16.9%
#100	12.1%
#200	9.1%

Atterberg Limits		
PL=	LL=	PI=

Coefficients			
D ₈₅ =	D ₆₀ =	3	D ₅₀ =
D ₃₀ =	0.7	D ₁₅ =	D ₁₀ = 0.09
C _u =	33.33	C _c =	1.81

USCS CLASSIFICATION
Well-graded SAND with Silt (SW-SM)

Project Name: Proposed Commercial - Orange, Ca

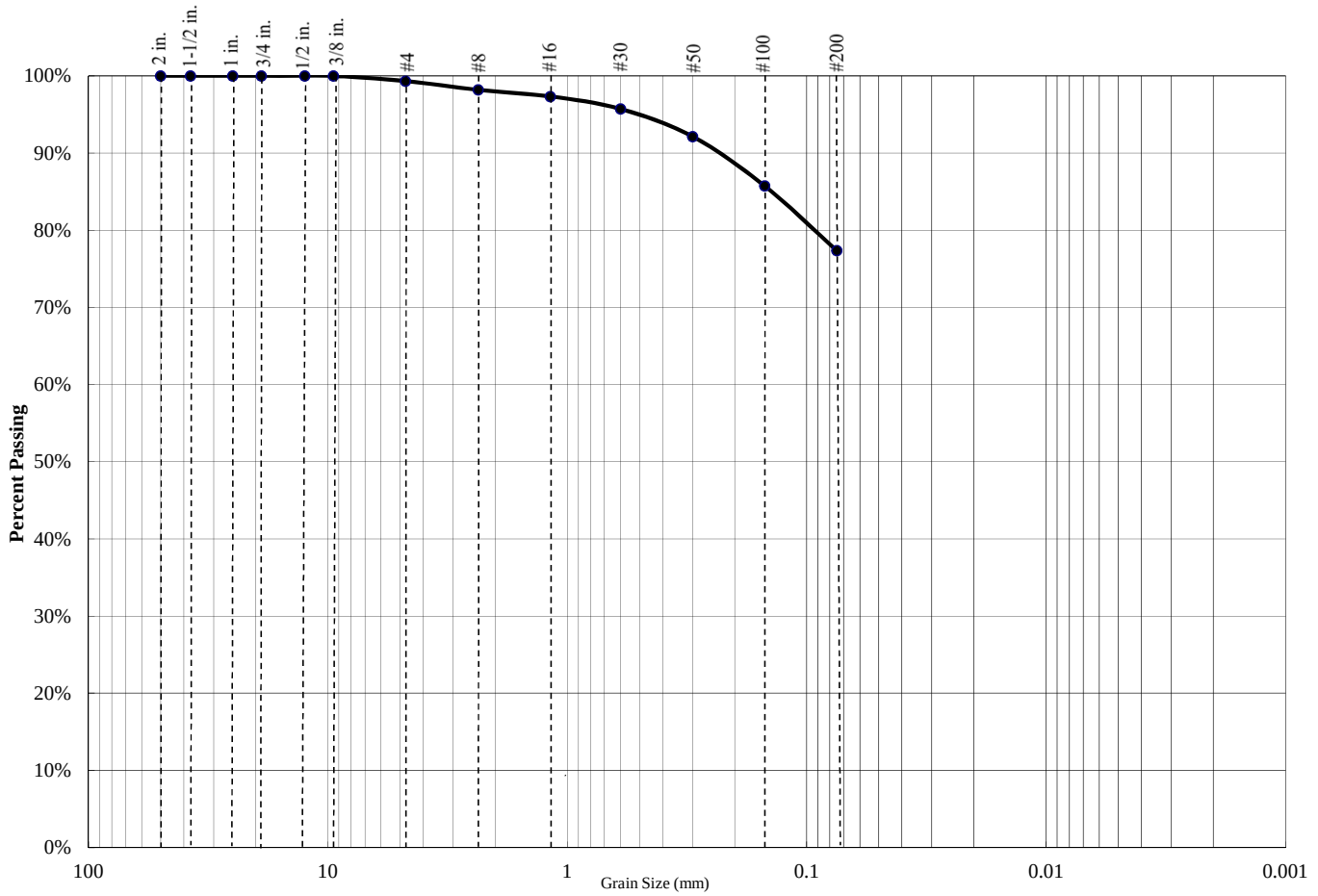
Project Number: 3-219-0272

Boring: B-1 @ 48.5



PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
1%	22%	77%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	99.3%
#8	98.2%
#16	97.3%
#30	95.7%
#50	92.1%
#100	85.7%
#200	77.4%

Atterberg Limits		
PL=	LL=	PI=

Coefficients		
D85=	D60=	D50=
D30=	D15=	D10=
C _u =	N/A	C _c = N/A

USCS CLASSIFICATION
CLAY with Sand (CL)

Project Name: Proposed Commercial - Orange, Ca

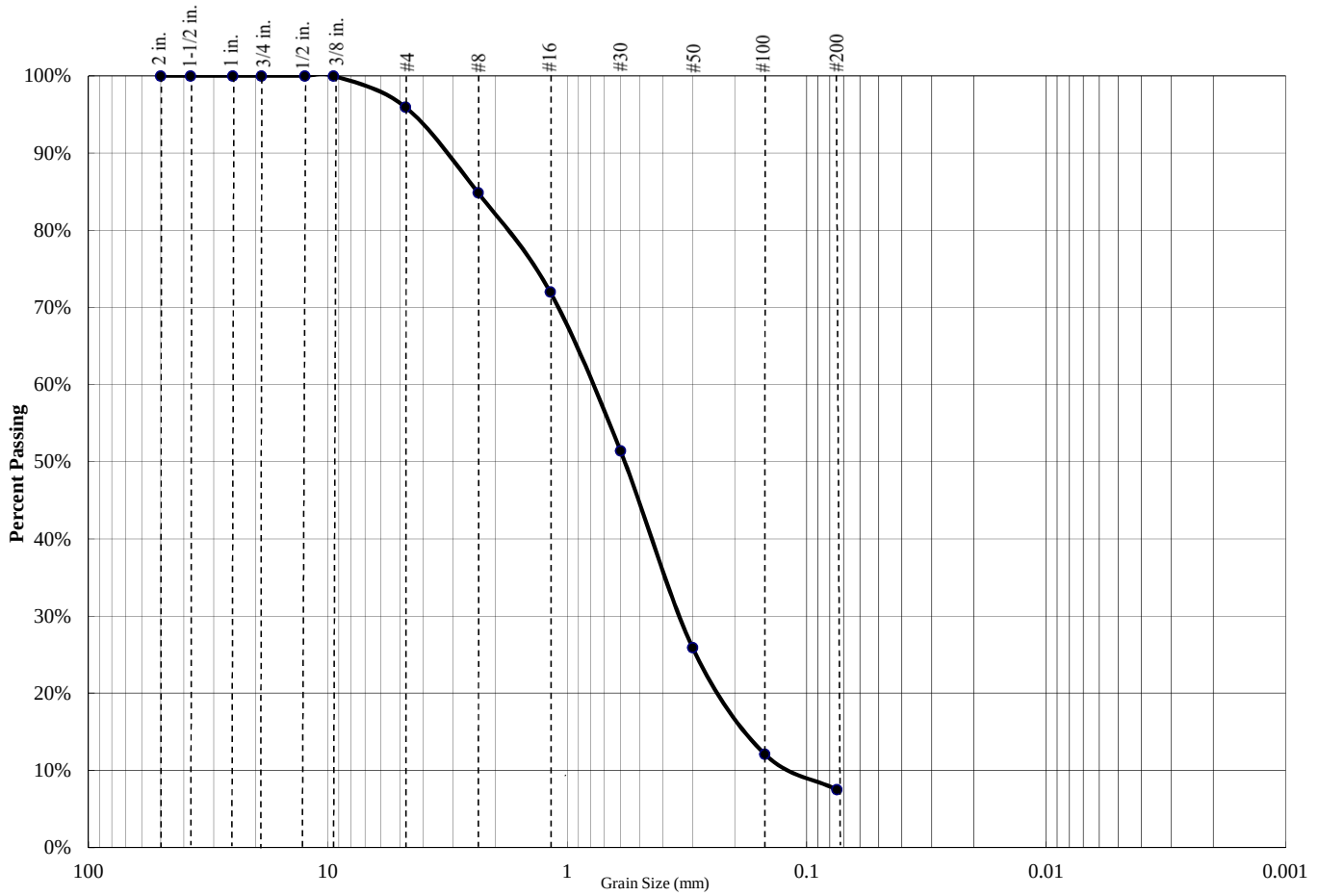
Project Number: 3-219-0272

Boring: B-3 @ 2'



PARTICLE SIZE DISTRIBUTION DIAGRAM

GRADATION TEST - ASTM C136



Percent Gravel	Percent Sand	Percent Silt/Clay
4%	88%	8%

Sieve Size	Percent Passing
3/4 inch	100.0%
1/2 inch	100.0%
3/8 inch	100.0%
#4	96.0%
#8	84.9%
#16	72.0%
#30	51.4%
#50	25.9%
#100	12.1%
#200	7.5%

Atterberg Limits		
PL=	LL=	PI=

Coefficients			
D ₈₅ =	D ₆₀ =	0.8	D ₅₀ =
D ₃₀ =	0.35	D ₁₅ =	D ₁₀ = 0.125
C _u =	6.40	C _c =	1.23

USCS CLASSIFICATION
Well-graded SAND with Silt (SW-SM)

Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Boring: B-5 @ 2'



EXPANSION INDEX TEST

ASTM D4829

Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Date Sampled: 4/25/19

Date Tested: 5/3/19

Sampled By: SEG

Tested By: NL

Sample Location: B-1 @ 0 - 3'

Soil Description: Reddish Brown Sandy CLAY (CL)

Trial #	1	2	3
Weight of Soil & Mold, g.	586.3		
Weight of Mold, g.	188.2		
Weight of Soil, g.	398.1		
Wet Density, pcf	120.1		
Weight of Moisture Sample (Wet), g.	851.0		
Weight of Moisture Sample (Dry), g.	772.9		
Moisture Content, %	10.1		
Dry Density, pcf	109.0		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	50.1		

Time	Initial	30 min	1 hr	6 hrs	12 hrs	24 hrs
Dial Reading	0	0.0345	0.0442	--	--	0.054

Expansion Index_{measured} = 54

Expansion Index₅₀ = 54.0

Expansion Index =

54

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

CHEMICAL ANALYSIS

SO₄ - Modified CTM 417 & Cl - Modified CTM 417/422

Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Date Sampled: 4/25/19

Date Tested: 5/2/19

Sampled By: SEG

Tested By: DZ

Soil Description: Reddish Brown Sandy CLAY (CL)

Sample Number	Sample Location	Soluble Sulfate SO ₄ -S	Soluble Chloride Cl	pH
1a.	B-1 @ 0 - 3'	530 mg/kg	68 mg/kg	7.7
1b.	B-1 @ 0 - 3'	530 mg/kg	67 mg/kg	7.7
1c.	B-1 @ 0 - 3'	530 mg/kg	65 mg/kg	7.7
Average:		530 mg/kg	67 mg/kg	7.7

Laboratory Compaction Curve ASTM D1557

Project Name: Proposed Commercial - Orange, Ca

Project Number: 3-219-0272

Date Sampled: 4/25/19

Date Tested: 5/2/19

Sampled By: SEG

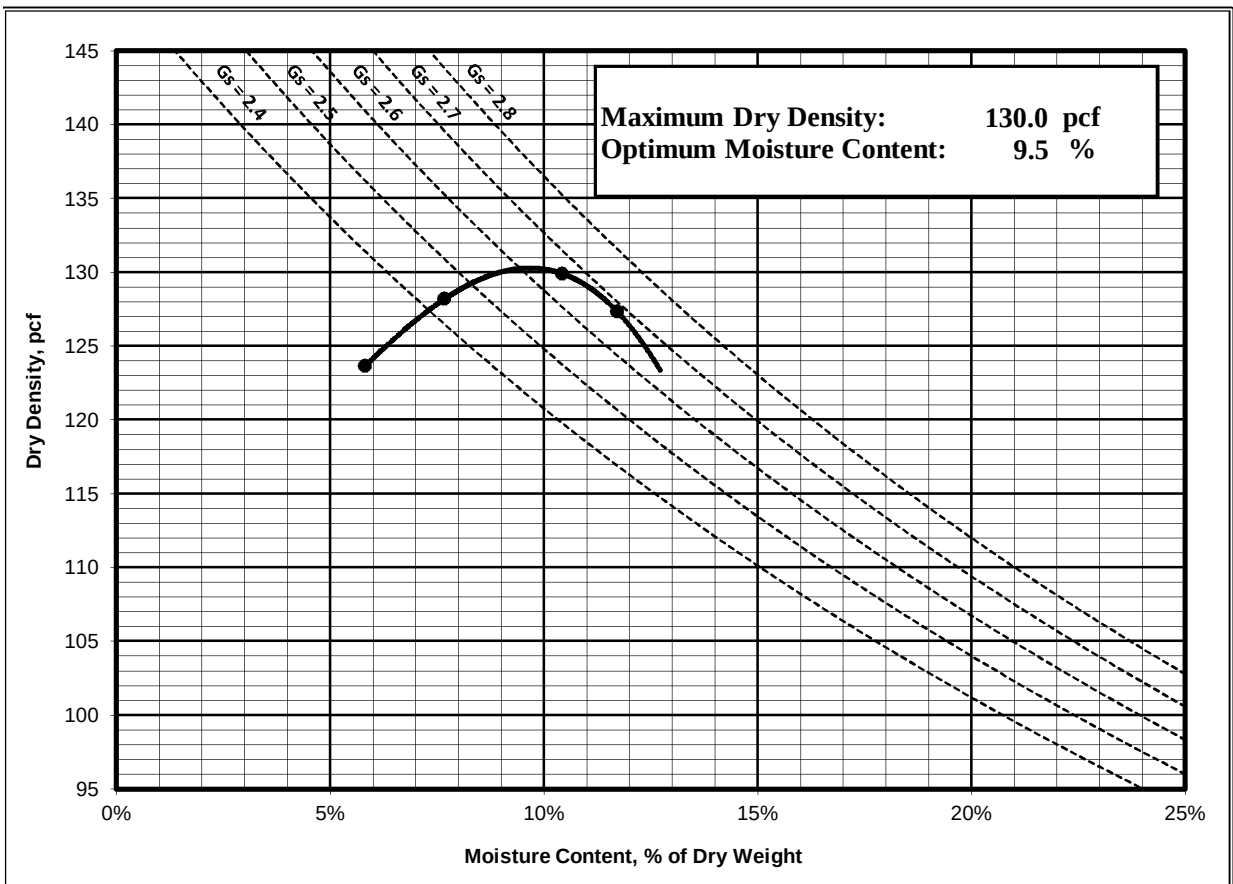
Tested By: MZ

Sample Location: B-1 @ 0 - 3'

Soil Description: Reddish Brown Sandy CLAY (CL)

Test Method: Method A

	1	2	3	4
Weight of Moist Specimen & Mold, (g)	3955.3	4064.2	4145.8	4127.7
Weight of Compaction Mold, (g)	1977.1	1977.1	1977.1	1977.1
Weight of Moist Specimen, (g)	1978.2	2087.1	2168.7	2150.6
Volume of Mold, (ft ³)	0.0333	0.0333	0.0333	0.0333
Wet Density, (pcf)	130.8	138.0	143.4	142.2
Weight of Wet (Moisture) Sample, (g)	336.5	336.5	336.5	336.5
Weight of Dry (Moisture) Sample, (g)	318.0	312.5	304.7	301.2
Moisture Content, (%)	5.8%	7.7%	10.4%	11.7%
Dry Density, (pcf)	123.6	128.2	129.9	127.3



APPENDIX B

EXPLORATORY BORING LOGS BY GEOTEK

**Updated Geotechnical and Infiltration Evaluation
901 E. Katella Avenue, Orange, California
Project No. 3193-CR**



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 logs of borings. 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 five pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – BORING LOG LEGEND

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

SOILS

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

GEOLOGIC

B: Attitudes Bedding: strike/dip

J: Attitudes Joint: strike/dip

C: Contact line

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

(Additional denotations and symbols are provided on the logs of borings)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Intracorp SW, LLC
PROJECT NAME: 901 E Katella Ave
PROJECT NO.: 3193-CR
LOCATION: Orange, CA

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"

LOGGED BY: C. Diaz
OPERATOR: Jeff
RIG TYPE: CME 75
DATE: 5/12/2022

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-1	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS							
5					4" Asphalt, 7" Base Fill:			EI, MD, SR
		30 35 44	R1	SC	Clayey f-c SAND, brown, slightly moist, very dense, few f gravel	6.4	130.8	
		50/5	R2		No Recovery			
10		50/6	R3	SM	Alluvium: Silty f-c SAND, brown, slightly moist, very dense, little f gravel	4.6	130.4	HC
		50/5	R4		Silty f-c SAND, brown, slightly moist to moist, very dense, little f gravel, disturbed sample			
		14 16 23	R5		Silty f-m SAND, brown, slightly moist to moist, medium dense	8.1	98.4	
	9 15 24	R6		Silty f-m SAND, brown, moist, medium dense	15.1	116.9		
25					BORING TERMINATED AT 22.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
30								

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

LEGEND

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

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

CLIENT:	Intracorp SW, LLC	DRILLER:	2R Drilling	LOGGED BY:	C. Diaz
PROJECT NAME:	901 E Katella Ave	DRILL METHOD:	Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	3193-CR	HAMMER:	140H/30"	RIG TYPE:	CME 75
LOCATION:	Orange, CA			DATE:	5/12/2022

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-2	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
5 <								

CLIENT:	Intracorp SW, LLC	DRILLER:	2R Drilling	LOGGED BY:	C. Diaz
PROJECT NAME:	901 E Katella Ave	DRILL METHOD:	Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	3193-CR	HAMMER:	140#/30"	RIG TYPE:	CME 75
LOCATION:	Orange, CA			DATE:	5/12/2022

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-3	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
5 								

CLIENT:	Intracorp SW, LLC	DRILLER:	2R Drilling	LOGGED BY:	C. Diaz
PROJECT NAME:	901 E Katella Ave	DRILL METHOD:	Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	3193-CR	HAMMER:	140#/30"	RIG TYPE:	CME 75
LOCATION:	Orange, CA			DATE:	5/12/2022

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

LEGEND

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

Lab testing:

AL = Atterberg Limits

SR = Sulfate/Resistivity Test

EI = Expansion Index

SH = Shear Test

SA = Sieve Analysis

HC= Consolidation

RV = R-Value Test

MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: Intracorp SW, LLC
PROJECT NAME: 901 E Katella Ave
PROJECT NO.: 3193-CR
LOCATION: Orange, CA

DRILLER: 2R Drilling
DRILL METHOD: Hollow Stem
HAMMER: 140#/30"

LOGGED BY: C. Diaz
OPERATOR: Jeff
RIG TYPE: CME 75
DATE: 5/12/2022

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: I-2	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
5 10 15 20 25 30				CL 4" Asphalt, 6" Base <u>Alluvium:</u> Sandy CLAY, reddish brown, slightly moist				
				BORING TERMINATED AT 5 FEET No groundwater encountered Boring set with pipe, sock, and gravel				

LEGEND

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

APPENDIX C

LABORATORY TEST RESULTS

**Updated Geotechnical and Infiltration Evaluation
901 E. Katella Avenue, Orange, California
Project No. 3193-CR**



SUMMARY OF LABORATORY TESTING

Classification

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

Collapse

Collapse tests were conducted in accordance with ASTM D4546. The results of these tests are presented herewith.

Expansion Index

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

In Situ Moisture Content and Unit Weight

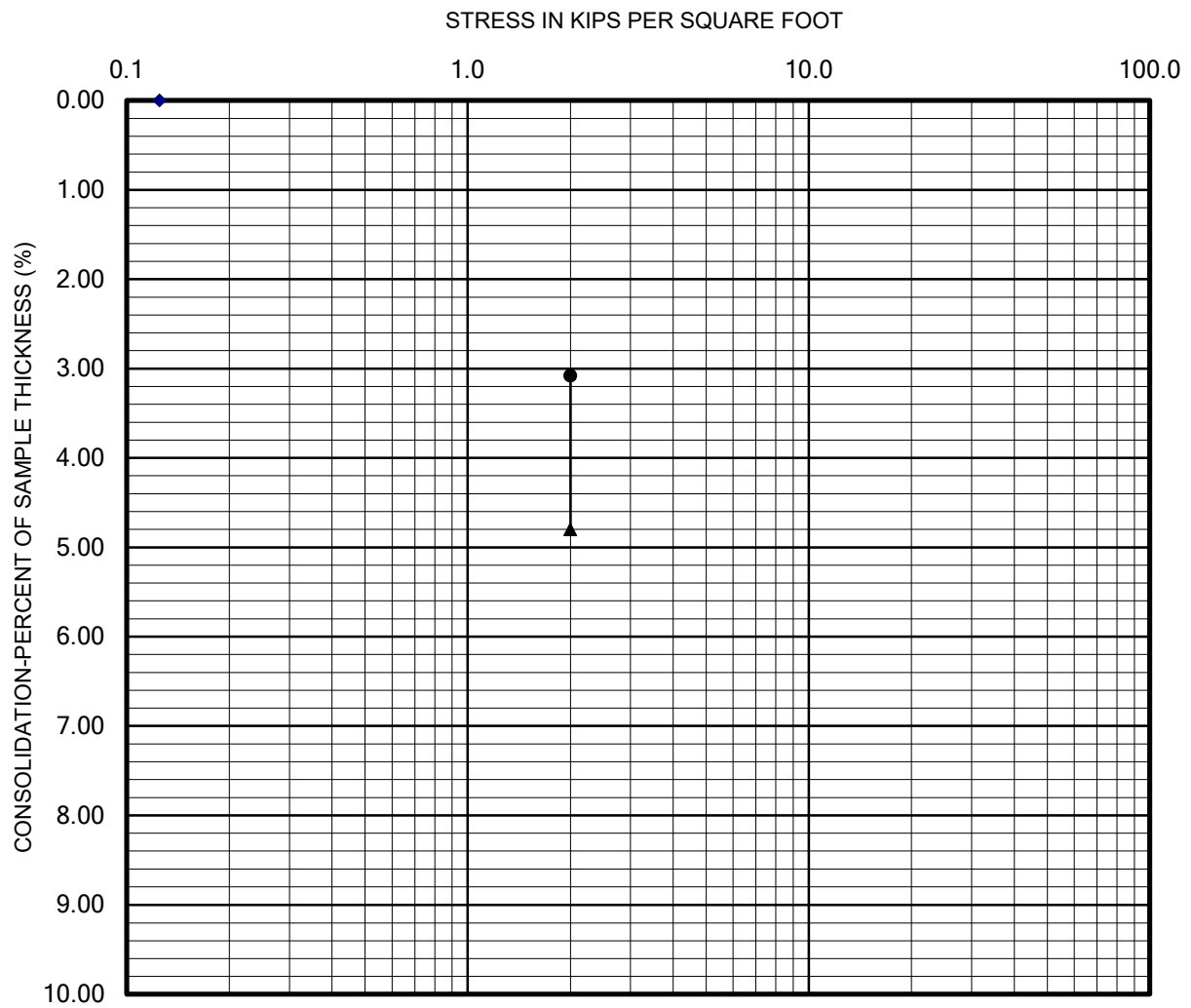
The field moisture content was measured in the laboratory on selected samples collected during the field investigation. The field moisture content is determined as a percentage of the dry unit weight. The dry density was measured in the laboratory on selected ring samples. The results are shown on the logs of exploratory borings in Appendix B.

Moisture-Density Relationship

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

Sulfate Content, Resistivity and Chloride Content

Testing to determine the water-soluble sulfate content, minimum resistivity, and chloride concentration in selected soil samples was performed by others. The results are included herewith.



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

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



COLLAPSE REPORT

CHECKED BY: DA	Lab: Corona
3193-CR	5/17/2022

Sample: B-1 @ 18 feet



EXPANSION INDEX TEST

(ASTM D4829)

Client: Intracorp SW LLC
Project Number: 3193-CR
Project Location: 901 E Katella Avenue, Orange

Tested/ Checked By: RL Lab No Corona
Date Tested: 5/17/2022
Sample Source: B-1 @ 0-5 feet
Sample Description:

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	786.5
Weight of ring (gm)	362.6
Net weight of sample (gm)	423.9
Wet Density, lb / ft ³ (C*0.3016)	127.8
Dry Density, lb / ft ³ (D/1.F)	118.4

SATURATION DETERMINATION

Moisture Content, %	8.0
Specific Gravity, assumed	2.70
Unit Wt. of Water @ 20 °C, (pcf)	62.4
% Saturation	51.0

READINGS

DATE	TIME	READING	
5/17/2022		0.7450	Initial
5/17/2022		0.7430	10 min/Dry
5/18/2022		0.7430	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
789.2	8.6

EXPANSION INDEX = 0



EXPANSION INDEX TEST

(ASTM D4829)

Client: Intracorp SW LLC
Project Number: 3193-CR
Project Location: 901 E Katella Avenue, Orange

Tested/ Checked By: RL Lab No Corona
Date Tested: 5/17/2022
Sample Source: B-3 @ 0-5 feet
Sample Description:

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

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	778.3
Weight of ring (gm)	370.3
Net weight of sample (gm)	408.0
Wet Density, lb / ft ³ (C*0.3016)	123.1
Dry Density, lb / ft ³ (D/1.F)	112.4

SATURATION DETERMINATION

Moisture Content, %	9.5
Specific Gravity, assumed	2.70
Unit Wt. of Water @ 20 °C, (pcf)	62.4
% Saturation	51.4

READINGS

DATE	TIME	READING	
5/17/2022		0.1300	Initial
5/17/2022		0.1290	10 min/Dry
5/18/2022		0.1580	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
806.7	16.5

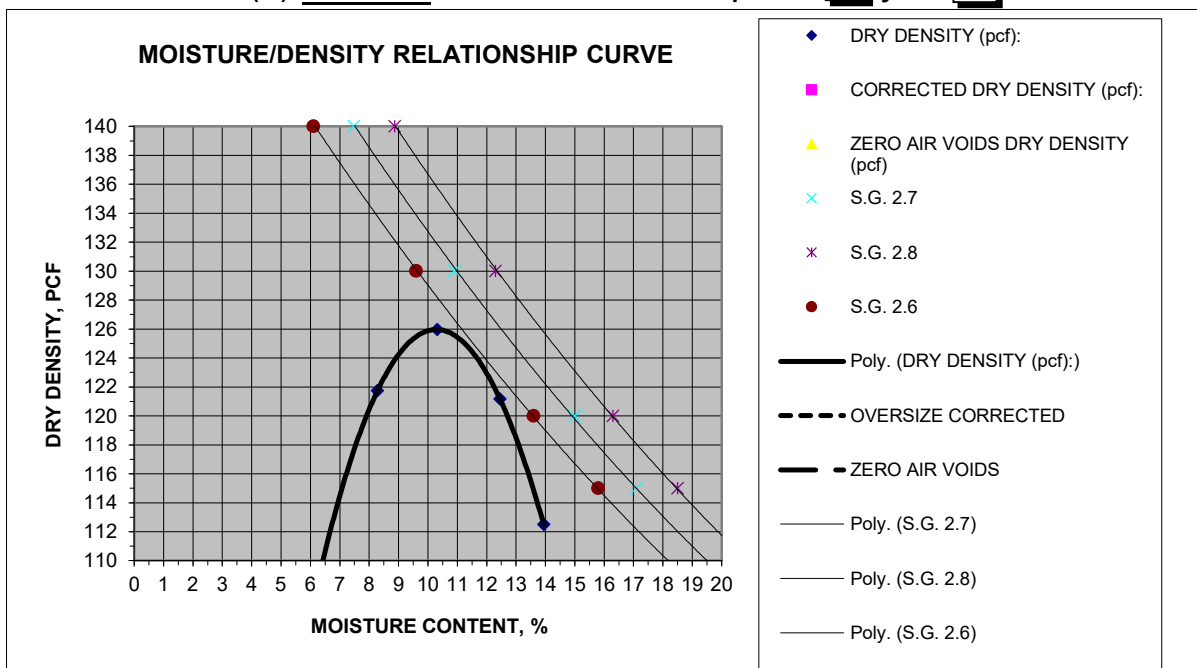
EXPANSION INDEX = 29



MOISTURE/DENSITY RELATIONSHIP

Client: Intracorp SW LLC	Job No.: 3193-CR
Project: 901 E Katella Avenue	Lab No.: Corona
Location: Orange	
Material Type: Yellow brown medium to coarse sand	
Material Supplier: -	
Material Source: -	
Sample Location: B-1 @ 0-5 feet	
-	
Sampled By: CD	Date Sampled: -
Received By: MP	Date Received: 5/16/2022
Tested By: MP	Date Tested: 5/18/2022
Reviewed By: DA	Date Reviewed: 5/26/2022

Test Procedure: ASTM D1557 Method: A
 Oversized Material (%): 11.2 Correction Required: ☒ yes ☐ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	126.0	@ Optimum Moisture, %	10.5
Corrected Maximum Dry Density, pcf		@ Optimum Moisture, %	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Classification:

Unified Soils Classification: _____
 AASHTO Soils Classification: _____

Atterberg Limits:

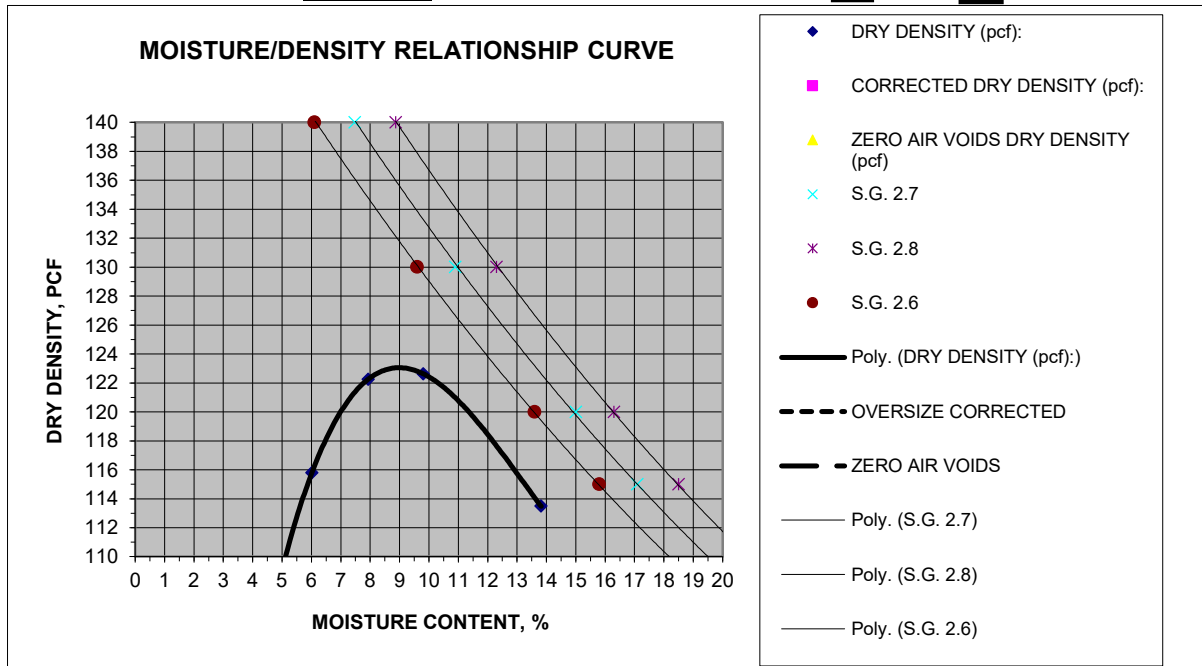
	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %



MOISTURE/DENSITY RELATIONSHIP

Client: Intracorp SW LLC	Job No.: 3193-CR
Project: 901 E Katella Avenue	Lab No.: Corona
Location: Orange	
Material Type: Yellow brown medium to coarse sand	
Material Supplier: -	
Material Source: -	
Sample Location: B-3 @ 0-5 feet	
Sampled By: CD	Date Sampled: -
Received By: MP	Date Received: 5/16/2022
Tested By: MP	Date Tested: 5/23/2022
Reviewed By: DA	Date Reviewed: 5/26/2022

Test Procedure: ASTM D1557 **Method:** A
Oversized Material (%): 3.4 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	123.0	@ Optimum Moisture, %	9.0
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)

Classification:

Unified Soils Classification: _____
AASHTO Soils Classification: _____

Atterberg Limits:

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



Soil Analysis Lab Results

Client: GeoTek, Inc.

Job Name: 901 E. Katella Avenue Orange, CA

Client Job Number: 3193-CR Intracropsin, LLC

Project X Job Number: S220518B

May 19, 2022

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates		Chlorides		Resistivity		pH	Redox	Sulfide	Nitrate	Ammonium	Lithium	Sodium	Potassium	Magnesium	Calcium	Fluoride	Phosphate
		SO ₄ ²⁻		Cl ⁻		As Rec'd Minimum				S ²⁻	NO ₃ ⁻	NH ₄ ⁺	Li ⁺	Na ⁺	K ⁺	Mg ²⁺	Ca ²⁺	F ₂ ⁻	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	0-5	26.5	0.0027	12.0	0.0012	3,417	2,747	8.7	132	12.70	1.0	46.0	ND	9.9	4.4	2.8	7.0	0.8	11.6
B3	0-5	103.2	0.0103	24.3	0.0024	26,130	2,814	7.9	144	3.50	42.5	13.7	ND	57.0	2.3	14.1	18.4	0.7	7.5

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

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

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

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

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

APPENDIX D

INFILTRATION TEST DATA BY GEOTEK

**Updated Geotechnical and Infiltration Evaluation
901 E. Katella Avenue, Orange, California
Project No. 3193-CR**



Client: Intracorp SW, LLC
Project: 901 E. Katella, Orange
Project No: 3193-CR
Date: 5/13/2002

Boring No. I-I

Infiltration Rate (Porchet Method)

Time Interval, $\Delta t =$	10	min
Final Depth to Water, $D_F =$	43.75	in
Test Hole Radius, $r =$	4	in
Initial Depth to Water, $D_O =$	36	in
Total Test Hole Depth, $D_T =$	60	in

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

$H_O = D_T - D_O =$	24	in
$H_F = D_T - D_F =$	16.25	in
$\Delta H = \Delta D = H_O - H_F =$	7.75	in
$H_{avg} = (H_O + H_F)/2 =$	20.125	in

$I_t =$
4.20
 Inches per Hour

Client: Intracorp SW, LLC
Project: 901 E. Katella, Orange
Project No: 3193-CR
Date: 5/13/2002

Boring No. I-2

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	min
Final Depth to Water, D_F =	39.5	in
Test Hole Radius, r =	4	in
Initial Depth to Water, D_O =	39.25	in
Total Test Hole Depth, D_T =	60	in

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

$H_O = D_T - D_O =$	20.75	in
$H_F = D_T - D_F =$	20.5	in
$\Delta H = \Delta D = H_O - H_F =$	0.25	in
$H_{avg} = (H_O + H_F)/2 =$	20.625	in

$I_t =$
0.04
 Inches per Hour

PERCOLATION DATA SHEET

Project: Intra corp SW, LLC 901 E Katella Ave.
Orange

Job No.: 3193-CR

Test Hole No.: I-1 Tested By: _____

Date: 5/12/2022

Depth of Hole As Drilled: 60 Before Test: 60

After Test: 60

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final water Level (Inches)	Δ in Water Level (Inches)	Rate (minutes per Inch)	Comments
								<u>Pre Soak</u> <u>5 Gal</u>
	<u>11:27</u>		<u>60</u>	<u>24</u>				
	<u>11:57</u>	<u>30</u>			<u>4 3/4</u>	<u>19 1/4</u>		<u>1st 30 min</u>
	<u>11:59</u>		<u>60</u>	<u>24</u>				
	<u>12:29</u>	<u>30</u>			<u>5 1/2</u>	<u>18 1/2</u>		<u>2nd 30 min</u>
	<u>12:31</u>		<u>60</u>	<u>24</u>				
	<u>12:41</u>	<u>10</u>			<u>15</u>	<u>9</u>		<u>1st 10 min</u>
	<u>12:43</u>		<u>60</u>	<u>24</u>				
	<u>12:53</u>	<u>10</u>			<u>16 1/4</u>	<u>7 3/4</u>		<u>2nd 10 min</u>
	<u>12:55</u>		<u>60</u>	<u>24</u>				
	<u>1:05</u>	<u>10</u>			<u>16</u>	<u>8</u>		<u>3rd 10 min</u>
	<u>1:07</u>		<u>60</u>	<u>24</u>				
	<u>1:17</u>	<u>10</u>			<u>16</u>	<u>8</u>		<u>4th 10 min</u>
	<u>1:19</u>		<u>60</u>	<u>24</u>				
	<u>1:29</u>	<u>10</u>			<u>16 1/4</u>	<u>7 3/4</u>		<u>5th 10 min</u>
	<u>1:31</u>		<u>60</u>	<u>24</u>				
	<u>1:41</u>	<u>10</u>			<u>16 1/4</u>	<u>7 3/4</u>		<u>6th 10 min</u>

PERCOLATION DATA SHEET

Project: Intra corp SW, LLC 901 E. Katella Ave
Orange Job No.: 3193-CR
 Test Hole No.: I-2 Tested By: C. Diaz Date: 5/13/2022
 Depth of Hole As Drilled: 60 Before Test: 60 After Test: 60

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final water Level (Inches)	Δ in Water Level (Inches)	Rate (minutes per Inch)	Comments
								<u>Pre Soak</u> <u>5 Gal.</u>
	<u>5:48</u> <u>6:18</u>	<u>30</u>	<u>60</u>	<u>24</u>	<u>23 3/4</u>	<u>1/4</u>		<u>1st 30 min</u>
	<u>6:18</u> <u>6:48</u>	<u>30</u>	<u>60</u>	<u>23 3/4</u>	<u>23 1/2</u>	<u>1/4</u>		<u>2nd 30 min</u>
	<u>6:48</u> <u>7:18</u>	<u>30</u>	<u>60</u>	<u>23 1/2</u>	<u>23 1/4</u>	<u>1/4</u>		<u>1st 30 min</u>
	<u>7:18</u> <u>7:48</u>	<u>30</u>	<u>60</u>	<u>23 1/4</u>	<u>23</u>	<u>1/4</u>		<u>2nd 30 min</u>
	<u>7:48</u> <u>8:18</u>	<u>30</u>	<u>60</u>	<u>23</u>	<u>22 3/4</u>	<u>1/4</u>		<u>3rd 30 min</u>
	<u>8:18</u> <u>8:48</u>	<u>30</u>	<u>60</u>	<u>22 3/4</u>	<u>22 1/2</u>	<u>1/4</u>		<u>4th 30 min</u>
	<u>8:48</u> <u>9:18</u>	<u>30</u>	<u>60</u>	<u>22 1/2</u>	<u>22 1/4</u>	<u>1/4</u>		<u>5th 30 min</u>
	<u>9:18</u> <u>9:48</u>	<u>30</u>	<u>60</u>	<u>22 1/4</u>	<u>22</u>	<u>1/4</u>		<u>6th 30 min</u>
	<u>9:48</u> <u>10:18</u>	<u>30</u>	<u>60</u>	<u>22</u>	<u>21 3/4</u>	<u>1/4</u>		<u>7th 30 min</u>

PERCOLATION DATA SHEET

Project: Intra Corp. SW, LLC 901 E. Katella Ave
Orange

Job No.: 3193-CR

Test Hole No.: I-2 Tested By: C. Diaz

Date: 5/13/2022

Depth of Hole As Drilled: 60 Before Test: 60

After Test: 60

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final water Level (Inches)	in Water Level (Inches)	Rate (minutes per Inch)	Comments
	<u>10:18</u>		<u>60</u>	<u>21 3/4</u>	<u>21 1/2</u>			
	<u>10:48</u>	<u>30</u>				<u>1/4</u>		<u>8th 30 min</u>
	<u>10:48</u>		<u>60</u>	<u>21 1/2</u>	<u>21 1/4</u>			
	<u>11:18</u>	<u>30</u>				<u>1/4</u>		<u>9th 30 min</u>
	<u>11:18</u>		<u>60</u>	<u>21 1/4</u>	<u>21</u>			
	<u>11:48</u>	<u>30</u>				<u>1/4</u>		<u>10th 30 min</u>
	<u>11:48</u>		<u>60</u>	<u>21</u>	<u>20 3/4</u>			
	<u>12:18</u>	<u>30</u>				<u>1/4</u>		<u>11th 30 min</u>
	<u>12:18</u>		<u>60</u>	<u>20 3/4</u>	<u>20 1/2</u>			
	<u>12:48</u>	<u>30</u>				<u>1/4</u>		<u>12th 30 min</u>
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APPENDIX E

GENERAL EARTHWORK AND GRADING GUIDELINES

**Updated Geotechnical and Infiltration Evaluation
901 E. Katella Avenue, Orange, California
Project No. 3193-CR**



GENERAL GRADING GUIDELINES

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

General

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

Preconstruction Meeting

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

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.

6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
 - a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
 - b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

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

Treatment of Existing Ground

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

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).

2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
 - a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
 - b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
 - a) They are not placed in concentrated pockets;
 - b) There is a sufficient percentage of fine-grained material to surround the rocks;
 - c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

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

The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.

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

JOB SAFETY

General

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

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

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

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

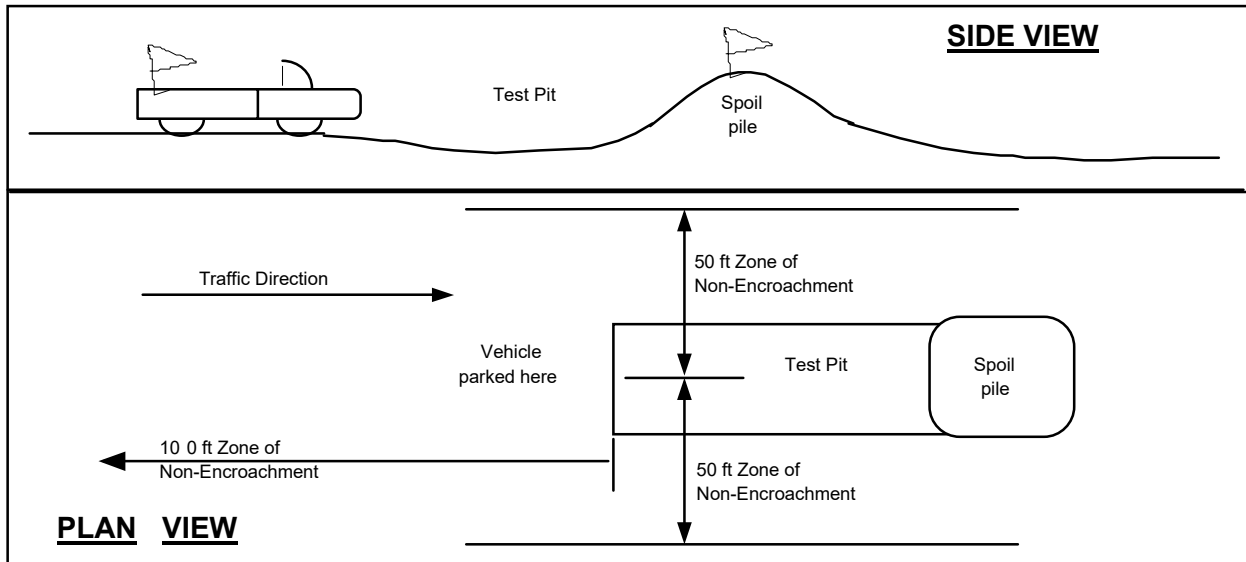
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to 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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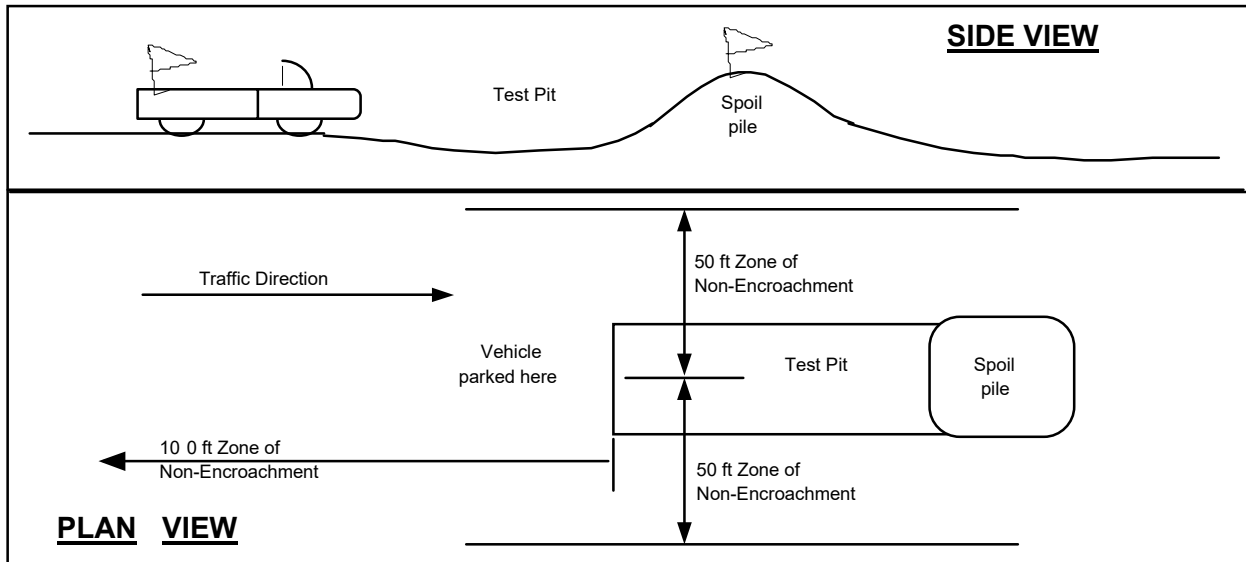
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to 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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