Appendix C Geotechnical Exploration

Appendix

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GEOTECHNICAL EXPLORATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT PROJECT
1122 N. ANAHEIM BOULEVARD
ANAHEIM, CALIFORNIA

Prepared for:

Renaissance City North Anaheim LLC

4675 MacArthur Court, Suite 550
Newport Beach, California 92660

Project No. 11862.003

September 24, 2019

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

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Project No. 11862.003

Renaissance City North Anaheim LLC
4675 MacArthur Court, Suite 550
Newport Beach, California 92660

Attention: Mr. Robert Kim

Subject: Geotechnical Exploration Report
Proposed Multi-Family Residential Development Project
1122 N. Anaheim Boulevard
Anaheim, California

In accordance with our proposal, dated August 7, 2019, Leighton and Associates, Inc. (Leighton) has performed a geotechnical exploration for the subject project. We understand the proposed development plan is to construct a 4-story, multi-family residential apartment building surrounding a 6-level parking structure. In addition, two underground detention/infiltration facilities are proposed at the site, one in the northwestern portion of the site and one in the southeastern portion of the site. Ancillary improvements such as utility infrastructure, pavement, flatwork, and landscaping are also proposed. The purpose of our geotechnical exploration was to evaluate subsurface conditions at the site, identify potential geologic and seismic hazards that may affect the project, and provide preliminary geotechnical recommendations for design and construction of the proposed improvements as currently planned.

The project is considered feasible from a geotechnical standpoint. The results of our exploration, conclusions and preliminary recommendations are presented in this report.

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We appreciate the opportunity to be of service to you on this project. If you have any questions or concerns, please contact us at your convenience. The undersigned can be reached at **(866) *LEIGHTON***, specifically at the phone extension and e-mail address listed below.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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ATTACHMENTS

1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located to the northeast of the intersection between Anaheim Boulevard and E. La Palma Avenue in the City of Anaheim, California. The site location (latitude 33.8493°, longitude -117.9187°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

The project site covers approximately 4.6 acres and is currently used for industrial purposes. The Orange County Assessor's Office identifies the site as Assessor Parcel Number (APN) 035-010-51. The site is bordered by Anaheim Boulevard to the west, existing industrial properties to the north and east, and a future residential development currently under construction to the south. The project site is currently occupied by a towing business with a majority of the property covered with asphalt concrete paving and three existing structures. Based on review of aerial photographs (NETR, 2019), the property appears to have been used by a towing business or similar to its current configuration since at least 1953.

Review of the United States Geological Survey (USGS) *7.5-Minute Anaheim Quadrangle* (USGS, 1972) indicates the site is relatively flat at approximate elevation (El.) +155 feet mean sea level (msl) with sheet flow generally directed to the southwest.

Based on information provided by you, we understand the proposed development is to construct a 4-story, multi-family residential apartment building surrounding a 6-level parking structure. In addition, two underground detention/infiltration facilities are proposed at the site, one in the northwestern portion of the site and one in the southeastern portion of the site. Ancillary improvements such as utility infrastructure, pavement, flatwork, and landscaping are also proposed.

1.2 Purpose and Scope

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site relative to the proposed development concept and provide preliminary geotechnical recommendations to aid in the design and construction for the project as currently planned. The scope of this geotechnical exploration included the following tasks:

- *Background Review* – We reviewed readily available geotechnical reports, literature, aerial photographs, and maps relevant to the site available from our in-house library or in the public domain. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Appendix A, *References*.

- *Pre-Field Exploration Activities* – A site visit was performed by a member of our technical staff to mark the proposed exploration locations. Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to our subsurface exploration.

- *Field Exploration* – Our field exploration was performed in two phases. The initial phase was performed on December 6, 2017, and consisted of two (2) Cone Penetrometer Test (CPT) soundings (designated CPT-1 and CPT-2). Each CPT was advanced to an approximate depth of 50 feet below ground surface (bgs). Shear wave velocity measurements were recorded at CPT-2 to develop seismic design parameters. The CPT soundings were performed in accordance with ASTM D5778 advanced by a 30-ton CPT rig in which a standard Cone equipped with a 15 cm^2 tip advanced at a constant rate of approximately 1 inch per second. The CPT provides a continuous record of the subsurface stratigraphy via data regarding tip and sleeve resistance which is continuously recorded electronically as the probe is advanced through the subsurface stratigraphy. The recorded data is processed yielding interpretations of soil type based upon the anticipated engineering behavior of the various soil strata though which the probe penetrates.

  The second phase was performed on August 19 and 20, 2019 and included drilling, logging, and sampling of seven (7) hollow-stem auger borings advanced at the site (designated LB-1 through LB-7) to approximate depths between 20 and 51½ feet bgs. In the borings, soil sampling was performed by the Standard Penetration Test (SPT) in accordance with ASTM D 1586 procedures. In addition, relatively undisturbed drive samples were collected by ASTM D 3550 procedures. Samples were collected at 5-foot intervals throughout the depth of exploration. In both test methods, the sampler is driven below the bottom of the borehole by a 140-pound weight (hammer) free-falling 30 inches. The drilling rig was equipped with an automatic hammer to provide greater consistency in the drop height and striking frequency. The number of blows to drive the sampler the final 12 inches of the 18-inch drive interval is termed the “blowcount” or SPT N-value. The N-values provide a measure of relative density in granular (non-cohesive) soils and comparative consistency in cohesive soils. Bulk samples were also obtained from the borings for laboratory analysis

The approximate locations of the borings and CPTs are shown on Figure 2, *Exploration Location Map*. The boring and CPT logs are presented in Appendix B, *Exploration Logs.*

- *Percolation Testing* – Borings LB-1, LB-2, LB-3 and LB-4 (Figure 2) were converted to temporary percolation test wells upon completion of drilling and sampling. In-situ percolation testing was performed on August 20 and 21, 2019 in general accordance with the Orange County Department of Public Works *Technical Guidance Document* (OCPW, 2013). The results of the percolation testing are presented in Appendix C, *Percolation Test Data*. Refer to the discussion of infiltration rate presented in Section 2.3.1, *Infiltration*. Upon completion of the percolation testing, the well casing was removed from each boring and the borings were backfilled with soil cuttings and patched at the surface with cold-mix asphalt concrete to match existing site conditions.

- *Laboratory Testing* – Laboratory tests were performed on representative soil samples to verify the field classification of the samples and to determine the geotechnical properties of the subsurface materials. The following tests were performed:

  - In-situ moisture content and density (ASTM D2216 and ASTM D2937);
  - Atterberg Limits (ASTM D 4318)
  - Consolidation (ASTM D 2435);
  - Direct shear (ASTM D3080);
  - Expansion Index (ASTM D4829);
  - R-value (DOT CA 301);
  - Sand Equivalent (DOT CA 217)
  - Maximum dry density and optimum moisture content (ASTM D1557); and
  - Corrosivity Suite - Sulfate, Chloride, pH and Resistivity (DOT CA 417, 422 and 532/643).

  All laboratory tests were performed in general conformance with American Society of the International Association for Testing and Materials (ASTM) or

Caltrans procedures (DOT CA). The results of the in-situ moisture and density tests are presented on the geotechnical boring logs in Appendix A. The results of other laboratory tests are presented in Appendix D, *Laboratory Test Results.*

- *Engineering Analysis* – The data obtained from our background review and subsurface field exploration were evaluated and analyzed to develop conclusions and preliminary recommendations for the proposed development.

- *Report Preparation* – This report presents our findings, conclusions and preliminary recommendations for the proposed development.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geology

The project site is located on the lowest reach of the Santa Ana River basin within the Peninsular Ranges geomorphic province. The Peninsular Ranges geomorphic province extends southward from the Los Angeles Basin to the tip of Baja California (Yerkes et al., 1965) and is characterized by elongated northwest-trending mountain ranges separated by sediment-floored valleys. The most dominant structural features of the province are the northwest trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province. East of the site are the northwest-trending Santa Ana Mountains, a large range which has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the Pacific Ocean.

Approximately 65 million years ago (at the end of the Cretaceous Period) a deep, structural trough existed off the coast of southern California (Yerkes, 1972). Over time the trough was filled with sediments eroded from the surrounding highlands and mountains. About 7 million years ago the boundary between the Pacific and North American plates shifted to its present position and the geologically modern Los Angeles basin began to form. The deepest part of the Los Angeles basin contains Tertiary to Quaternary-aged (65 million years and younger) marine and non-marine sedimentary rocks that are about 24,000 feet thick (Yerkes, et al, 1965; Wright, 1991). During the Pleistocene epoch (the last two million years) the region was flooded as the sea level rose in response to the worldwide melting of the Pleistocene glaciers depositing sediments across the Los Angeles Basin during transgression and regression of sea level.

The area south and west of the Santa Ana Mountains is generally characterized as a broad, complex, alluvial fan which receives sediments from the Santa Ana River and its tributaries draining the Santa Ana and San Bernardino Mountains. These sediments are comprised of relatively flat-lying, unconsolidated to loosely consolidated clastic deposits that are approximately 3,000 feet thick beneath the site (Sprotte et al., 1980, and Real, 1985). The project site is located approximately 3 miles west of the Santa Ana River. Regional geologic mapping of the project site and vicinity indicates that near-surface native soils beneath the site consist of Quaternary age young alluvial fan deposits comprised of varying proportions of sand, silt and clay (Morton and Miller, 2006). The surficial geologic units mapped in the vicinity of the project site are shown on Figure 3, *Regional Geology Map*.

2.2 Subsurface Soil Conditions

Based on our subsurface explorations, the site is underlain by a thin layer of undocumented artificial fill materials (Afu) overlying Quaternary-aged young alluvial fan deposits (Qyf).

The artificial fill encountered in our borings at the explored locations is generally about 1 to 2½ feet in thickness likely associated with the existing and previous site improvements. The fill soils are variable in type and generally consist primarily of sandy silt and silty sand with minor amounts of clay. Localized thicker accumulations of fill materials and possible foundation remnants should be anticipated during future earthwork construction. The existing artificial fill materials at the site are likely associated with the existing improvements and initial development of the site to its current configuration. However, records documenting observation and testing during fill placement were not available for review. Therefore, for purposes of this report all fill material is considered undocumented and unsuitable in its current configuration for structural support.

Below the artificial fill materials are Quaternary-aged young alluvial fan deposits as encountered in the borings to the maximum depth explored (51½ feet bgs). The alluvium generally consists of a mixture of thick sequences of sand and silty sand to a depth of approximately 15 to 25 feet bgs and below approximately 30 to 35 feet across the site. A zone of interbedded clay, silt, and silty clay of variable thickness exists between approximately 20 to 35 feet bgs and again between approximately 40 to 45 feet bgs.

The stratigraphy of the subsurface soils as interpreted in each boring and CPT is presented on the logs included in Appendix B and the locations of the explorations are shown on Figure 2, *Exploration Location Map*. Some of the engineering properties of these soils are described in the following sections.

2.2.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our exploration, the near surface (upper 5 feet) onsite soils consist predominantly of sand, silty sand, sandy silt, silty clay, and clay. The laboratory test result of representative near-surface (upper 5 feet) bulk soil samples from borings LB-5 and LB-7 indicate very low expansion potential when wetted (EI values of 0 and 2). Accordingly, we recommend that the upper onsite soils be assumed to have very low expansion potential. The Expansion Index test results for the onsite soil from our geotechnical exploration are included in Appendix D of this report.

Variance in expansion potential of onsite soil is anticipated; therefore, additional testing is recommended upon completion of rough grading to confirm the expansion potential result presented in this report. Standard engineering and earthwork construction practices, such as proper foundation design and controlled moisture conditioning will reduce impacts associated with expansive soils.

2.2.2 Soil Corrosivity

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2014). The 2016 California Building Code (CBC), provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes, is 500 ppm per California Test 532. Concentrations of chloride ions above the stated concentration or other characteristics such as soil resistivity or redox potential may warrant special corrosion protection measures.

Representative near-surface (upper 5 feet) bulk soil samples collected from borings LB-5 and LB-7 were tested to evaluate site soil corrosivity. The test results indicates soluble sulfate concentrations of 79 to 114 ppm, chloride contents of 20 to 40 ppm, pH values of 6.23 to 7.10 and minimum resistivity values of 5,220 to 11,980 ohm-cm.

The results of the resistivity test indicate that the underlying soil is mildly corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate content from the soil sample, concrete in contact with the soil is expected to have negligible exposure to sulfate attack per ACI 318-14. The sample tested for water-soluble chloride content

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indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix D of this report.

2.2.3 Soil Compressibility

Two samples of the onsite soils recovered from the borings were subjected to consolidation testing to evaluate the compressibility of these materials under assumed loads representative of anticipated structural bearing stresses. The results of testing indicate these soils did not exhibit a significant compressibility potential. The results of testing are presented in Appendix D.

2.2.4 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory direct shear testing. The results of testing are included in Appendix D as well as summary graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

2.2.5 Shear Wave Velocity Profile

Shear wave velocities were measured in CPT-2, see Figure 2 for location. Results are presented in Appendix B. Based on the average shear wave velocity of about 870 feet per second recorded at CPT-2, from the ground surface down to about 50 feet bgs, the seismic site class is characterized as Site Class D.

2.2.6 Excavation Characteristics

Based on our subsurface explorations performed at the site and our experience from grading jobs in the vicinity of the site, we anticipate the onsite artificial fill and alluvial materials can generally be excavated using conventional excavation equipment in good operating condition.

The soils within the planned excavation depths consist of layers that may contain granular, unconsolidated soils with little or no cementation and few fines. These materials are prone to cave in or collapse in unshored excavations. See Section 3.7, *Temporary Excavations* for additional information on soil type and excavation characteristics.

2.3 Groundwater Conditions

Groundwater was not encountered in our borings advanced at the site to the maximum depth explored of approximately 51½ feet bgs. According to groundwater information obtained through the California Geological Survey (CGS) and presented in the Seismic Hazard Zone Report for the Anaheim Quadrangle (CGS, 1997), the historically shallowest groundwater depth in the vicinity of the project site is greater than 50 feet bgs. In addition, based on review of available groundwater information from the California Department of Water Resources Water Data Library (DWR, 2019) for a nearby groundwater monitoring well located near the eastern project boundary (State Well # 04S10W03P001S), the shallowest groundwater level measured for a monitoring period between February 1971 and August 2003 was approximately 89 feet bgs.

Based on the currently proposed development scheme, groundwater does not pose a constraint during and after construction. Although groundwater is not considered a constraint for the project, seasonal fluctuations in groundwater level, localized zones of perched water including water due to nearby landscaping, and an increase in soil moisture should be anticipated during and following locally intense rainfall or stormwater runoff.

2.3.1 Infiltration

Percolation testing was performed within borings LB-1 through LB-4 to evaluate the infiltration characteristics of subsurface soils. The percolation tests were conducted in general accordance with the Orange County Department of Public Works *Technical Guidance Document* (OCPW, 2013). Results of the percolation testing are presented in Appendix C, *Percolation Test Data*. The test locations and zones tested are shown on Figure 2, *Exploration Location Map*.

A boring percolation test is useful for field measurements of the infiltration rate of soils, and is suited for testing when the design depth of the infiltration device is deeper than current existing grades, especially in areas where it is difficult to dig test pits, or where the depths of these test pits would be considerably deep. At the subject site, testing consisted of advancing the borings to general depths anticipated for the invert of typical infiltration devices.

A falling-head test was implemented at two of the percolation well locations (LB-1 and LB-4) for a test zone approximately 30 to 40 feet bgs. The infiltration rate for the test was calculated by dividing the discharge volume by the infiltration surface area over a period of time. The volume of discharge was calculated by adding the total volume of water that dropped within the PVC pipe and within the annulus, and incorporating a reduction factor to account for the porosity of the annulus material. The infiltration surface area was based on the average water height within the test zone for each time interval.

A constant-head test, or high flowrate test, was implemented at the other two percolation well locations (LB-2 and LB-3) for a test zone approximately 15 to 20 feet bgs due to the generally favorable percolation characteristics of the site soils at the testing depth. The infiltration rate was calculated by recording the approximate volume of water delivered to the test zone while maintaining a relatively constant height of water in the well over the testing period. A water source (garden hose from onsite water source) was used to deliver water to the wells at a relatively constant rate. The measured infiltration rate was calculated by dividing the total volume of water by the total duration of the test, and dividing by the percolation surface area.

Detailed results of the field testing data and measured infiltration rate for the test well are presented in Appendix C, *Percolation Test Data*. The test results are summarized below:

Table 1 – Measured (Unfactored) Infiltration Rate

Test Well Designation	Approximate Depth of Test Zone (feet bgs)	Measured Infiltration Rate (inches per hour)
LB-1	30 to 40	4.18
LB-2	15 to 20	179.5
LB-3	15 to 20	77.1
LB-4	30 to 40	0.81

Based on the results of the percolation tests, the site soils are generally favorable and feasible infiltration at the locations and depths evaluated. However, the results of the testing performed at a depth of 15 to 20 feet bgs at the tested locations of LB-2 and LB-3 indicate significantly higher rates than the deeper zone tested of 30 to 40 feet bgs at the tested locations of

LB-1 and LB-4. Design considerations for infiltration BMPs, including a reduction factor that should be incorporated into design of the system, are presented in Section 3.10.

2.4 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is **not** located within a designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008c). The closest active faults to the site are the Puente Hills fault, Whittier-Elsinore Fault Zone, San Joaquin Hills fault and the Newport-Inglewood Fault Zone, located approximately 1.5 miles, 6.5 miles, 10.6 miles and 12.0 miles from the site, respectively. The Puente Hills and San Joaquin Hills faults are blind thrust faults that are concealed at depth, without the potential for surface fault rupture. The San Andreas fault, which is the largest active fault in California, is approximately 38 miles northeast of the site. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.5 Seismicity and Ground Shaking

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the seismic source, and the site response characteristics. The site should be expected to experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active faults (Figure 4). Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). The 2016 edition of the California Building Code (CBC) is the current edition of the code. Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals for the project, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2016 CBC:

Table 2 – 2016 CBC Seismic Design Parameters

Categorization/Coefficients	Code-Based
Site Longitude (decimal degrees) West	-117.9187°
Site Latitude (decimal degrees) North	33.8493°
Site Class	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.554 g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.593 g
Short Period Site Coefficient at 0.2s Period, F_a	1.0
Long Period Site Coefficient at 1s Period, F_v	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.554 g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.890 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.036 g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.593 g
Site-adjusted geometric mean Peak Ground Acceleration, PGA_m	0.595 g

2.6 Liquefaction Potential

Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine, clean sandy soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose and medium dense, near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential.

In general, liquefaction hazards are the most severe in the upper 50 feet bgs. As shown on the State of California Seismic Hazard Zones map for the Anaheim Quadrangle (CGS, 1998), the project site is **not** located within an area that has been identified as being potentially susceptible to liquefaction (see Figure 5, *Seismic Hazard Map*). In addition, the historically shallowest groundwater depth in the vicinity of the project site is greater than 50 feet bgs (CGS, 1997). Based on these findings, the potential for liquefaction to occur at the project site is low.

2.7 Seismically-Induced Settlement

Seismically induced settlement consists of dynamic settlement of unsaturated soil (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within low density sandy soil due to reduction in volume during and shortly after an earthquake event.

Based on the results of our analysis, seismically-induced settlement at the site due to dry dynamic settlement (above groundwater) at the site was estimated to be on the order of ½ inch across the site. The differential settlement can be taken as one-half the total estimated settlement over a horizontal distance of 30 feet.

2.8 Seismically Induced Lateral Spreading

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Since liquefaction is not considered a hazard at the site and the site is relatively constrained laterally, earthquake induced lateral spreading is also not considered a hazard at the site.

2.9 Seismically Induced Landsliding

The potential for seismically induced landsliding to occur at the site is considered low due to the absence of slopes at the site. In addition, based on review of the State of California Seismic Hazard Zones Map for the Anaheim Quadrangle (CGS, 1998), the site is **not** located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides (see Figure 5, *Seismic Hazard Map*). Proposed slopes, if any, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter.

2.10 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2009), the project site is located within a flood hazard area identified as “Zone X”, which is defined as areas of 0.2 percent annual chance floodplain; areas of 1 percent annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1 percent annual chance flood. Regionally, storm runoff flow is generally directed to the southwest to Carbon Creek channel. As shown on Figure 6, *Flood Hazard Zone Map*, the site **is** located within a 500-year flood hazard zone.

Earthquake induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The project site **is** located within a flood impact zone from Prado Dam as indicated on Figure 7, *Dam Inundation Map*. However, catastrophic failure of this dam is expected to be a very unlikely event in that dam safety regulations exist and are enforced by the Division of Safety of Dams, Army Corp of Engineers and Department of Water Resources. Inspectors may require dam owners to perform work, maintenance or implement controls if issues are found with the safety of the dam.

2.11 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site and the inland location of the site, seiche and tsunami risks at the site are considered negligible.

2.12 Sedimentation and Erosion

The erosion characteristics of the unconsolidated alluvial deposits exposed on any future slopes onsite are expected to be moderately susceptible to erosion. These materials will be particularly prone to erosion during excavation and site development, especially during heavy rains.

The potential for erosion can be mitigated through the application of best management practices (BMPs) and other Storm Water Pollution Prevention Plan (SWPPPs), such as temporary catchment basins and/or sandbagging to control runoff and contain sediment transport within the project site during construction. Following completion of the project, the site is anticipated to be improved with structures, hardscape, landscaping and appropriate drainage infrastructure. Therefore, sedimentation and erosion impacts upon completion of construction are considered less than significant.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

Based upon this study, we conclude that the proposed development for the subject site is feasible from a geotechnical standpoint, provided that the preliminary recommendations presented in this report are properly incorporated in design and construction.

The proposed structure may be supported on shallow spread-type foundations established in engineered fill soils. The floor slab may be supported directly on grade. There may be existing underground utilities that will also be impacted. Information on these utilities should be provided to Leighton for evaluation. All existing undocumented fill is recommended to be removed from the proposed building/structure footprint areas prior to placement of engineered fill.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. The recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to determine the effect upon the recommendations subsequently presented. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the City of Anaheim, the County of Orange and other governing agencies.

Leighton should review the grading and foundation plans and project specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans for this project.

3.1 Site Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, and/or debris within the area of proposed grading. These materials should be removed from the site. Any underground obstructions onsite should be removed. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits. All undocumented fill or man-made debris, unsuitable native soils and former foundation remnants should be excavated and removed from the proposed building/structure footprint areas prior to placement of engineered fill.

3.1.2 Removals and Overexcavations

To provide uniform foundation support and reduce the potential for excessive static settlement, all existing undocumented fill and any unsuitable soil, as deemed by the geotechnical engineer, should be removed to expose suitable native soils and replaced as engineered fill below the proposed building and other structural improvements. Removals and overexcavations should be performed such that all undocumented fill is removed and a minimum of 3 feet of engineered fill is established below the proposed building foundation elements. Based on an assumed footing embedment depth on the order of roughly 2 feet, the depth of overexcavation is anticipated to be on the order of approximately 5 feet below existing grade across the site. The lateral extent of overexcavation beyond foundations should be equal to the depth of overexcavation below the foundation. Deeper overexcavations in localized areas may be recommended during grading by a representative of the geotechnical engineer depending on observed subsurface conditions.

3.1.3 Excavation Bottom Preparation

All excavation or removal bottoms should be observed by a representative of the geotechnical engineer prior to placement of fill or other improvements to determine that geotechnically suitable soil is exposed. Excavation bottoms observed to be suitable for fill placement or other improvements should be scarified to a depth of at least 8 inches, moisture-conditioned as necessary to achieve a moisture content of at least 2 percentage points above the optimum moisture content, and then compacted to a minimum of 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557 (Modified Proctor).

3.1.4 Fill Materials

On-site soil that is free of construction debris, organics, cobbles, boulders, rubble, or rock larger than 6-inches in largest dimension is suitable to be used as fill for support of structures. Oversized materials larger than 6-inches in diameter encountered during site grading may require special handling, and may be placed in non-structural areas or areas of deep fill at depth below anticipated excavations such as for any footings, utilities, future developments, etc. Any imported fill soil should be approved by the geotechnical engineer prior to import or use onsite.

3.1.5 Fill Placement and Compaction

Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to at least 2 percent above optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

3.1.6 Shrinkage

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Field and laboratory data used in our calculations included laboratory-measured maximum dry density for the general soil type encountered at the subject site, the measured in-place densities of near surface soils encountered and our experience.

Based upon the results of the in-place density and the moisture-density relationship exhibited by representative bulk samples of the near surface soils, recompaction of the soils is anticipated to result in volume shrinkage in the range of 10 to 15 percent. The estimated shrinkage does not include material losses due to removal of organic material or other unsuitable bearing materials (debris, rubble, oversize material greater than 6-inches) and the actual shrinkage that occurs during grading may vary throughout the site.

3.1.7 Reuse of Concrete and Asphalt Rubble

If encountered during site clearing and/or during preparation activities, construction rubble (i.e., Portland cement concrete and asphalt concrete) may be incorporated in the proposed development. For use as structural fill, the processed material should be crushed to develop a relatively well-graded mixture with a maximum particle size of 3-inch nominal diameter. Concrete rubble should be free of rebar; processed asphalt pavement rubble may be used if mixed with the existing base course (where present) and soils in proportion of 1 part processed asphalt to 3 parts soil. For use as pavement base course, rubble should be crushed to satisfy gradation requirements of Section 200-2.4 of the SSPWC. Such materials must be free of and segregated from any hazardous materials and/or organic material of any kind.

3.2 Foundation Design

Conventional spread footings established in engineered fill may be used to support the proposed building. Footings should be embedded a minimum 18 inches below the lowest adjacent grade. An allowable soil bearing pressure of 4,000 pounds per square foot (psf) may be used for footings with a minimum width of 12 inches for continuous footings and 18 inches for isolated footings.

A one-third increase in the bearing value for short duration loading, such as wind or seismic forces may be used. The ultimate bearing capacity can be taken as 12,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.5 should be used for initial bearing capacity evaluation with factored loads.

The allowable bearing capacity for shallow footings is based on a total static settlement of 1 inch. Differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

For static loading, 50 pounds per cubic inch (pci) may be assumed as the modulus of subgrade reaction (*k*). For seismic loading, a *k* value of 150 pci may be assumed.

Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. Once developed by the structural

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engineer, we should review total dead and sustained live loads for each column including plan location and span distance, to evaluate if differential settlements between dissimilarly loaded columns will be tolerable. Excessive differential settlement can be mitigated with the use of reduced bearing pressures, deeper footing embedment, possibly changing overexcavation schemes and using imported base material under spread footings, or possibly other methods.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings structures. For calculating lateral resistance, a passive pressure of 300 psf per foot of depth to a maximum of 3,000 psf and a frictional coefficient of 0.30 may be used. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

3.3 Slabs-on-Grade

Concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 3.1. From a geotechnical standpoint, we recommend slab-on-grade be a minimum 5 inches thick with No. 3 rebar placed at the center of the slab at 24 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. Where moisture-sensitive floor coverings or equipment is planned, the slabs should be protected by a minimum 10-mil-thick vapor barrier between the slab and subgrade. A coefficient of friction of 0.35 can be used between the floor slab and the vapor barrier.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential but not eliminate for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

3.4 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil are expected to have negligible exposure to water-soluble sulfates in the soil. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with CBC 2016 requirements. However, concrete exposed to recycled water should be designed using Type V cement.

Based on our laboratory testing, the onsite soil is considered mildly corrosive to ferrous metals. Ferrous pipe should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Ferrous pipe, if used, should be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from onsite soils.

3.5 Retaining Walls

Recommended lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft. or pcf. These values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

On-site soils are likely suitable to be used as retaining wall backfill due to its low expansion potential, field and laboratory verification are recommended before use. However, site soils can be variable in composition, clast size and expansive characteristics. Should site soil for reuse behind retaining walls should be tested to ensure expansion potential is less than 20 (EI<20). Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions as shown on Figure 8, *Retaining Wall Backfill and Subdrain Detail* are as follows:

Table 3 – Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	35
At-Rest (braced)	60
Passive Resistance (compacted fill)	300
Seismic Increment (add to active pressure)	20
*Only for level and drained properly compacted backfill	

Walls that are free to rotate or deflect may be designed using active earth pressure. For basement walls or walls that are fixed against rotation, the at-rest pressure should be used. For seismic condition, the pressure should be distributed as an inverted triangular distribution and the dynamic thrust should be applied at a height of 0.6H above the base of the wall.

3.5.1 Sliding and Overturning

Total depth of retained earth for design of walls and for uplift resistance, should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, if drained, or 60 pcf if submerged, for properly compacted backfill.

3.5.2 Drainage

Adequate drainage may be provided by a subdrain system positioned behind the walls (Figure 8). Typically, this system consists of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the Standard Specifications for Public Works Construction (Green Book), 2018 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the Standard Specifications for

Public Works Construction (Green Book), 2018 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill, particularly where horizontal space is limited adjacent to shoring (where walls are cast against shoring). These drainage panels should be connected to the perforated drainpipe at the base of the wall.

3.6 Paving

To provide support for paving, the subgrade soils should be prepared as recommended in the Section 3.1. Compaction of the subgrade, including trench backfills, to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

3.6.1 Asphalt Concrete

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of engineered fill with an R-value greater than 50, compacted to at least 90 percent as recommended, the minimum recommended paving thicknesses are presented in the following table. Results of R-value testing on near surface samples of existing onsite soils indicate values of 70 and 71.

Table 4 – Asphalt Concrete Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
5	3	4
6	3	6
7	4	6
8	5	6
9	5	8

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The asphalt paving sections were determined using the Caltrans design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

3.6.2 Portland Cement Concrete Paving

We have assumed that such a subgrade will have an R-value of at least 50, which will need to be verified after the completion of site grading.

Portland cement concrete (PCC) paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented in the following table. We have assumed that the Portland cement concrete will have a compressive strength of at least 3,000 pounds per square inch.

Table 5 – PCC Pavement Sections

Traffic Index	PCC (inches)	Base Course (inches)
5	5	4
6	6	4
7	6½	4
8	7	4
9	8	4

The paving should be provided with expansion joints at regular intervals no more than 15 feet in each direction. Load transfer devices, such as dowels or keys, are recommended at joints in the paving to reduce possible offsets. The paving sections in the above table have been developed based on the strength of unreinforced concrete. Steel reinforcing may be added to the paving to reduce cracking and to prolong the life of the paving.

3.6.3 Base Course

The base course for both asphalt concrete and Portland cement concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D 1557.

3.7 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and foundation excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 4 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure.

Temporary excavations should be treated in accordance with the State of California version of OSHA excavation regulations, Construction Safety Orders for Excavation General Requirements, Article 6, Section 1541, effective October 1, 1995. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Onsite sandy soils are to be considered Type C soils which are subject to collapse in shallow unbraced excavations (i.e. approximately 3-feet in vertical height).

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.8 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1 and 306-6 of the Standard Specifications for Public Works Construction, ("Greenbook"), 2018 Edition. Utility trenches can be backfilled with onsite sandy material free of rubble, debris, organic and oversized material up to (≤) 3-inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Sand:** A uniform, sand material that has a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), water densified in place, or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), 2018 Edition. CLSM should not be jetted.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. Native and clean fill soils can be used as backfill over the pipe bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.9 Drainage and Landscaping

Surface drainage should be designed to direct water away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

3.10 Infiltration BMP Design Considerations

It should be noted that the measured infiltration rates presented in Section 2.3.1 may degrade over time due to complete saturation of underlying soils, and fines build-up and plugging if pretreatment of the storm water is not performed. As such, a reduction of the measured infiltration rates using a factor of safety of 6 or more should be considered to establish a conservative infiltration rate for the service life of the system. This factor should not be less than 6, but may be higher at the discretion of the design engineer.

In general, a vast majority of geotechnical distress issues are related to improper drainage. Distress in the form of foundation movement could occur. Direct infiltration to the subsurface is not recommended adjacent to curb and gutter, public pavements or within 10 feet away from the design saturation zone as soil saturation could lead to a loss of soil support, settlement or collapse, and internal erosion (piping). The design saturation zone may be assumed as a 1:1 plane projected downward from the top of an infiltration device's discharge zone. Additionally, infiltration water will migrate along pipe backfill (typically sand or gravel bedding) affecting improvements far from the point of infiltration. Proposed direct open bottom infiltration systems, should be located as far away from existing or proposed foundations, rigid improvements and utilities as is practical in order to reduce the geotechnical distress issues related to water. Where sufficient distance from improvements cannot be achieved, additional recommendations may be warranted and can be provided during plan review.

Prior to construction of any infiltration device intended for the site, the plans should be reviewed by the geotechnical consultant to verify that our geotechnical recommendations have been appropriately incorporated into the plans and not compromised by the addition of an infiltration system to the site. The designer of any infiltration system should contact the geotechnical consultant for geotechnical input during the design process as they feel necessary.

3.11 Additional Geotechnical Services

Leighton should review the grading plans, foundation plans, and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated.

Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- Subgrade Preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When any unusual conditions are encountered.

4.0 LIMITATIONS

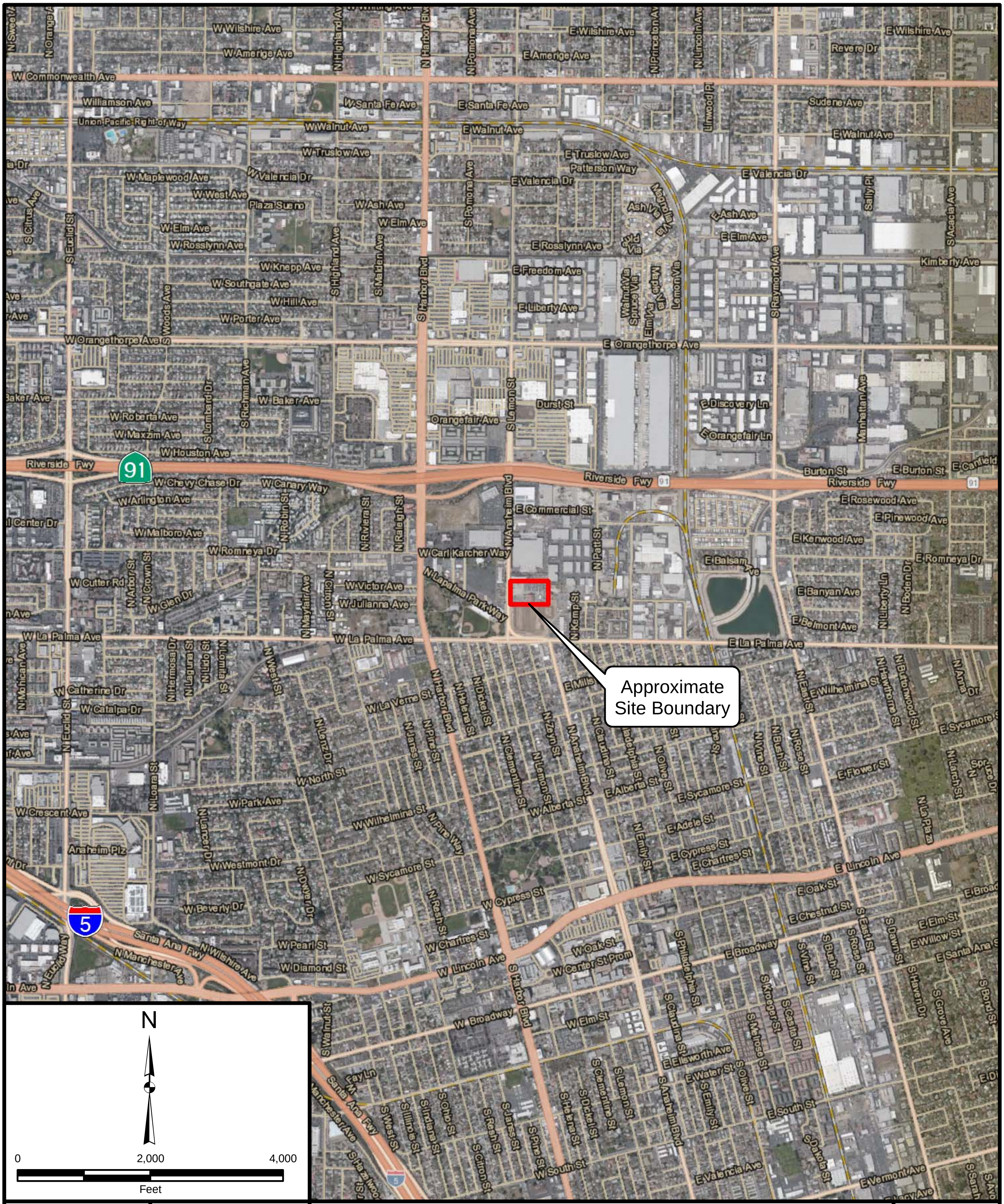
Leighton's work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in California at this time. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

This report is issued with the understanding that it is the responsibility of the owner or a duly authorized agent acting on behalf of the owner, to ensure that information and preliminary recommendations contained herein are brought to the attention of the necessary design consultants for this project and incorporated into plans and specifications.

Until reviewed and accepted by the local governing Agency, this report may be subject to change. Changes may be required as part of the Agency review process. Leighton assumes no risk or liability for consequential damages that may arise due to design work progressing before this report is reviewed and accepted by the reviewing Agency.

The findings of this report are considered valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on the subject or adjacent properties. In addition, changes in standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may at some future time be invalidated wholly or partially by changes outside Leighton's control. Conditions revealed in construction excavations may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by Leighton and additional recommendations may be warranted based on additional observations and findings.

The conclusions and recommendations in this report are based in part upon data that were obtained from a necessarily limited number of observations, site visits, excavations, samples and testes. Such information can be obtained only with respect to the specific locations explored, and therefore may not completely define all subsurface conditions throughout the site. The nature of many sites is that differing geotechnical and/or geological conditions can occur within small distances and under varying climatic conditions. Furthermore, changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report should be considered preliminary if unanticipated conditions are encountered and additional explorations, testing and analyses may be necessary to develop alternative recommendations.



Project: 11862.003	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: March 2016

Base Map: ESRI ArcGIS Online 2019

Author: Leighton Geomatics (btran)

SITE LOCATION MAP

1122 N. Anaheim Boulevard
Anaheim, California

Figure 1

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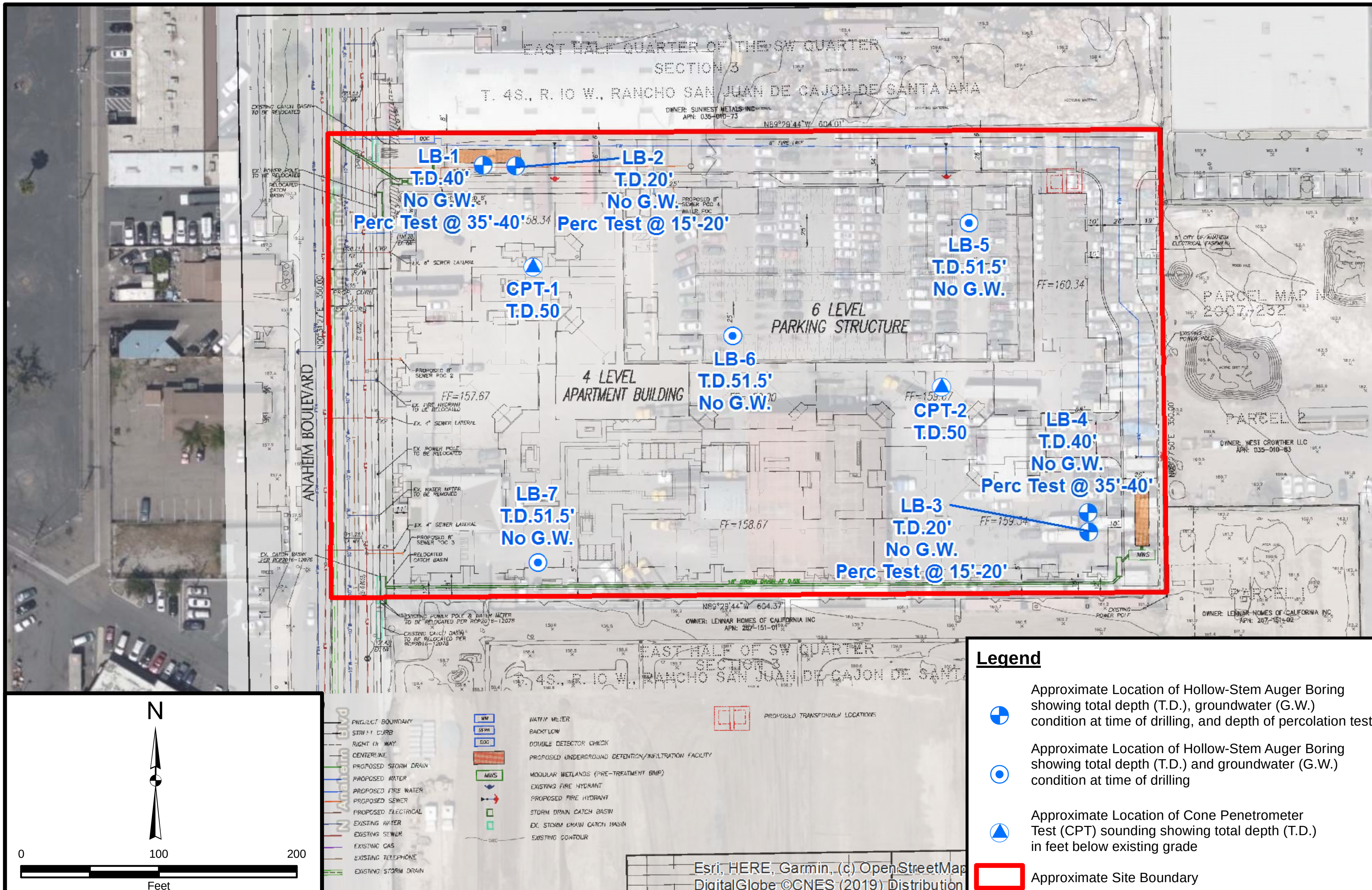

Legend

- Approximate Location of Hollow-Stem Auger Boring showing total depth (T.D.), groundwater (G.W.) condition at time of drilling, and depth of percolation test
- Approximate Location of Hollow-Stem Auger Boring showing total depth (T.D.) and groundwater (G.W.) condition at time of drilling
- Approximate Location of Cone Penetrometer Test (CPT) sounding showing total depth (T.D.) in feet below existing grade
- Approximate Site Boundary

Project: 11862.003	Eng/Geol: CCK/JMP
Scale:1 " = 100 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

EXPLORATION LOCATION MAP

1122 N. Anaheim Boulevard
Anaheim, California

Figure 2

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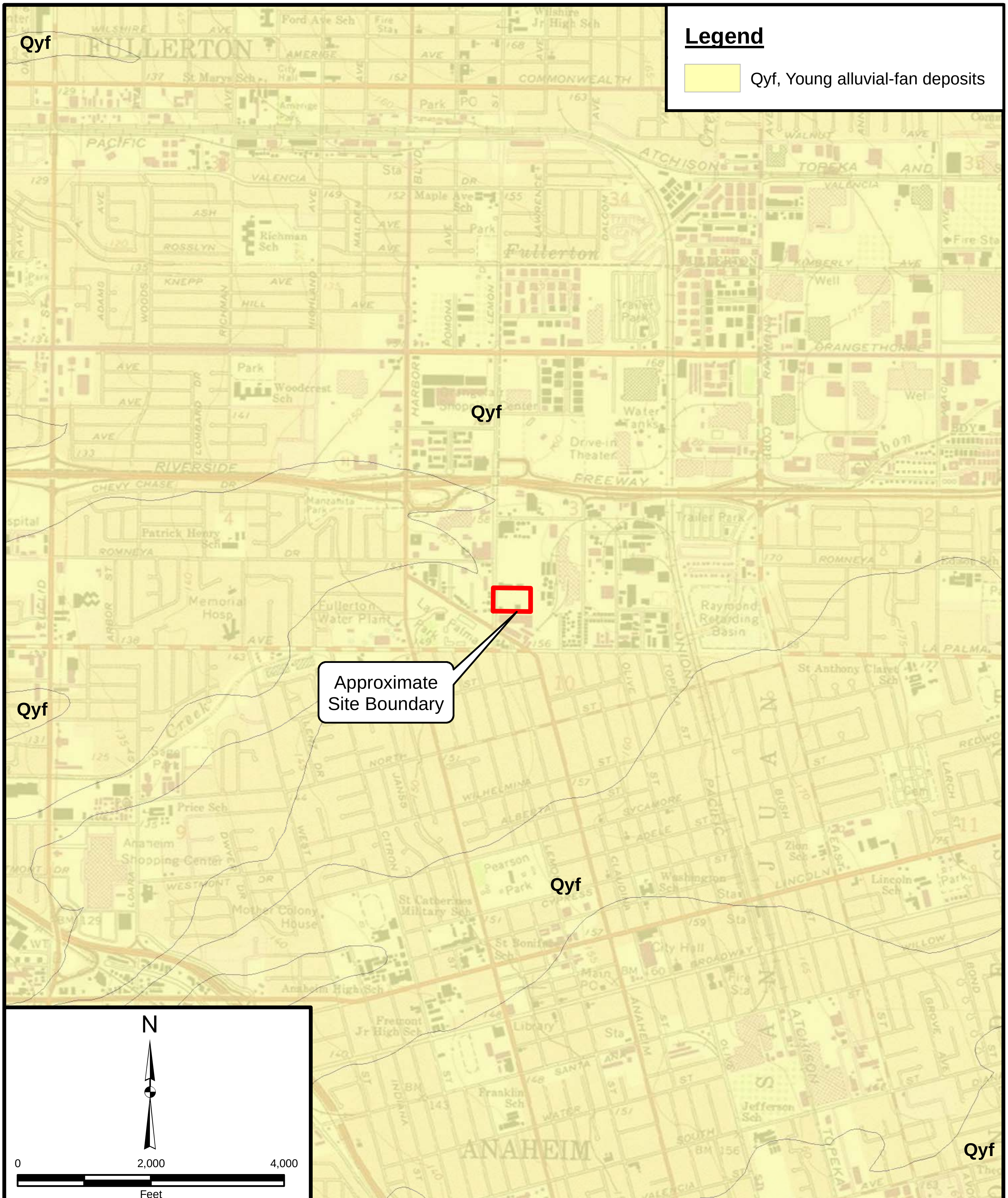

Project: 11862.003	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: September 2019

Base Map: USGS, 2006, Geologic map of the San Bernardino and Santa Ana 30'x60' quadrangles, California, version 1.

Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

1122 N. Anaheim Boulevard
Anaheim, California

Figure 3

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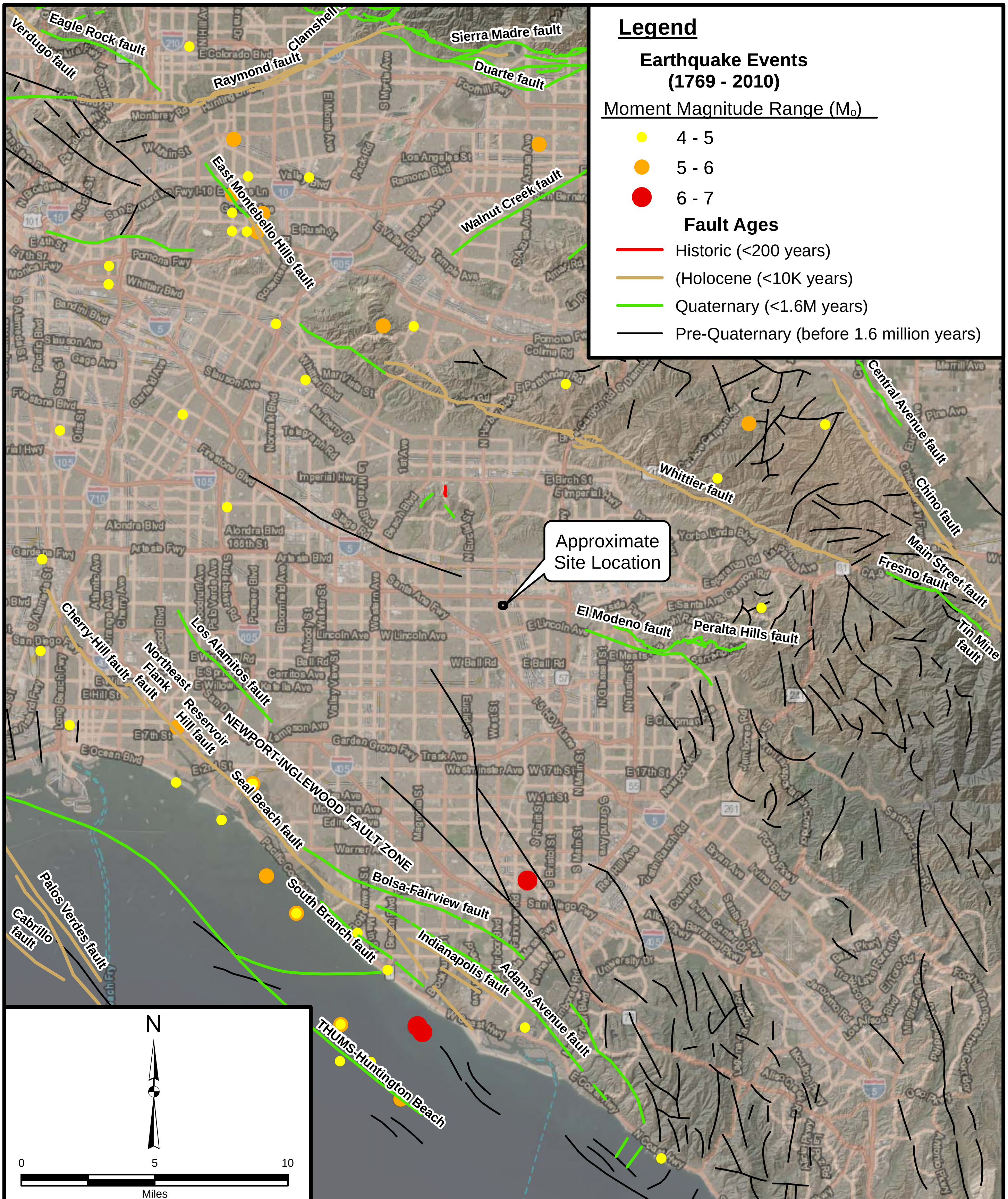

Project: 11862.003	Eng/Geol: CCK/EBP	Figure 4
Scale:1 " = 5 miles	Date: September 2019	

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (btran)

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

1122 N. Anaheim Boulevard
Anaheim, California

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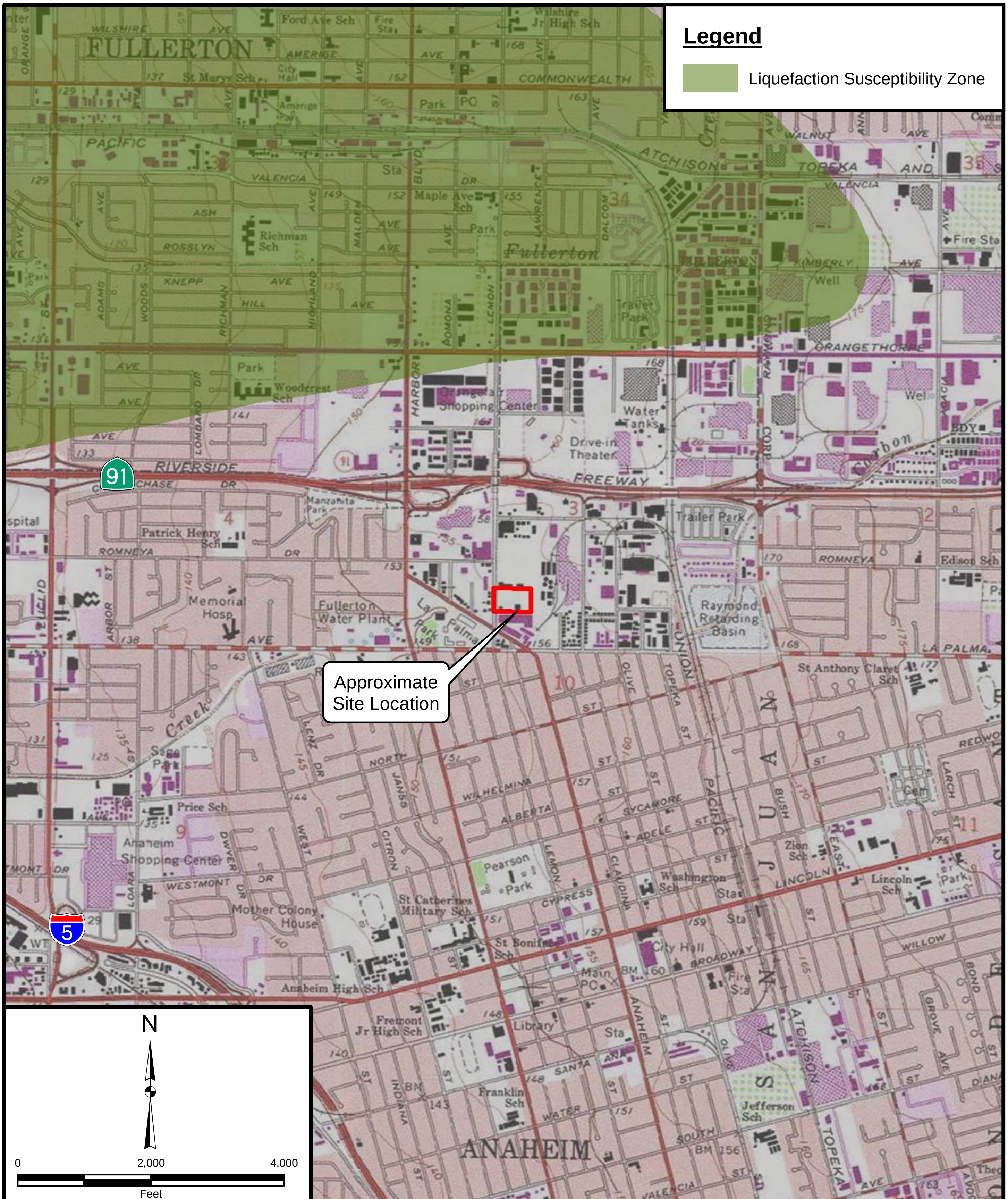

Project: 11862.003	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: March 2016

Base Map, CGS

Author: Leighton Geomatics (btran)

SEISMIC HAZARD MAP

1122 N. Anaheim Boulevard
Anaheim, California

Figure 5

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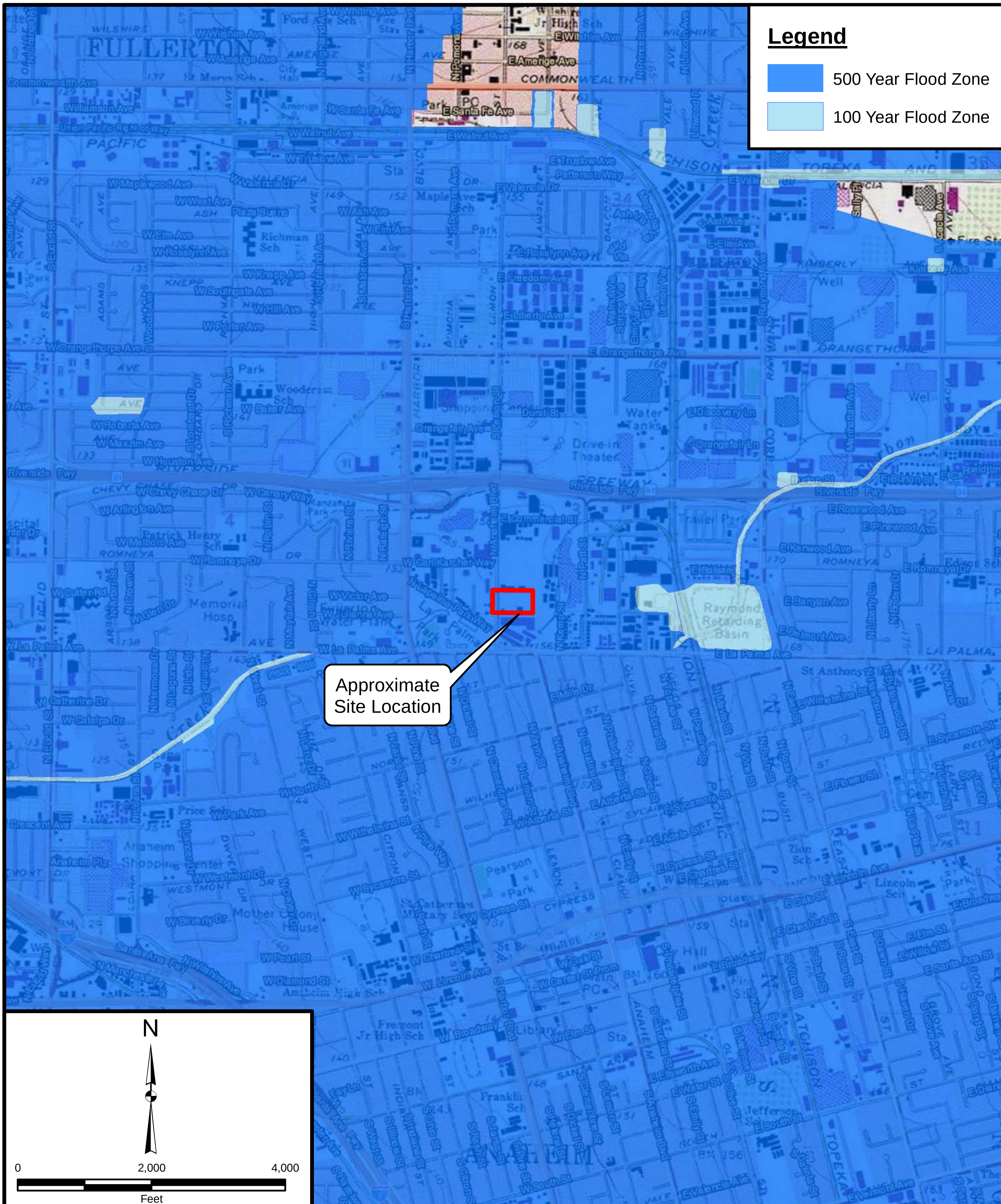

Project: 11862.003	Eng/Geol: CCK/JMP
Scale:1 " = 2,000 '	Date: March 2016

Base Map: CA DWR, FEMA

Author: Leighton Geomatics (btran)

FLOOD HAZARD MAP

1122 N. Anaheim Boulevard
Anaheim, California

Figure 6

Leighton

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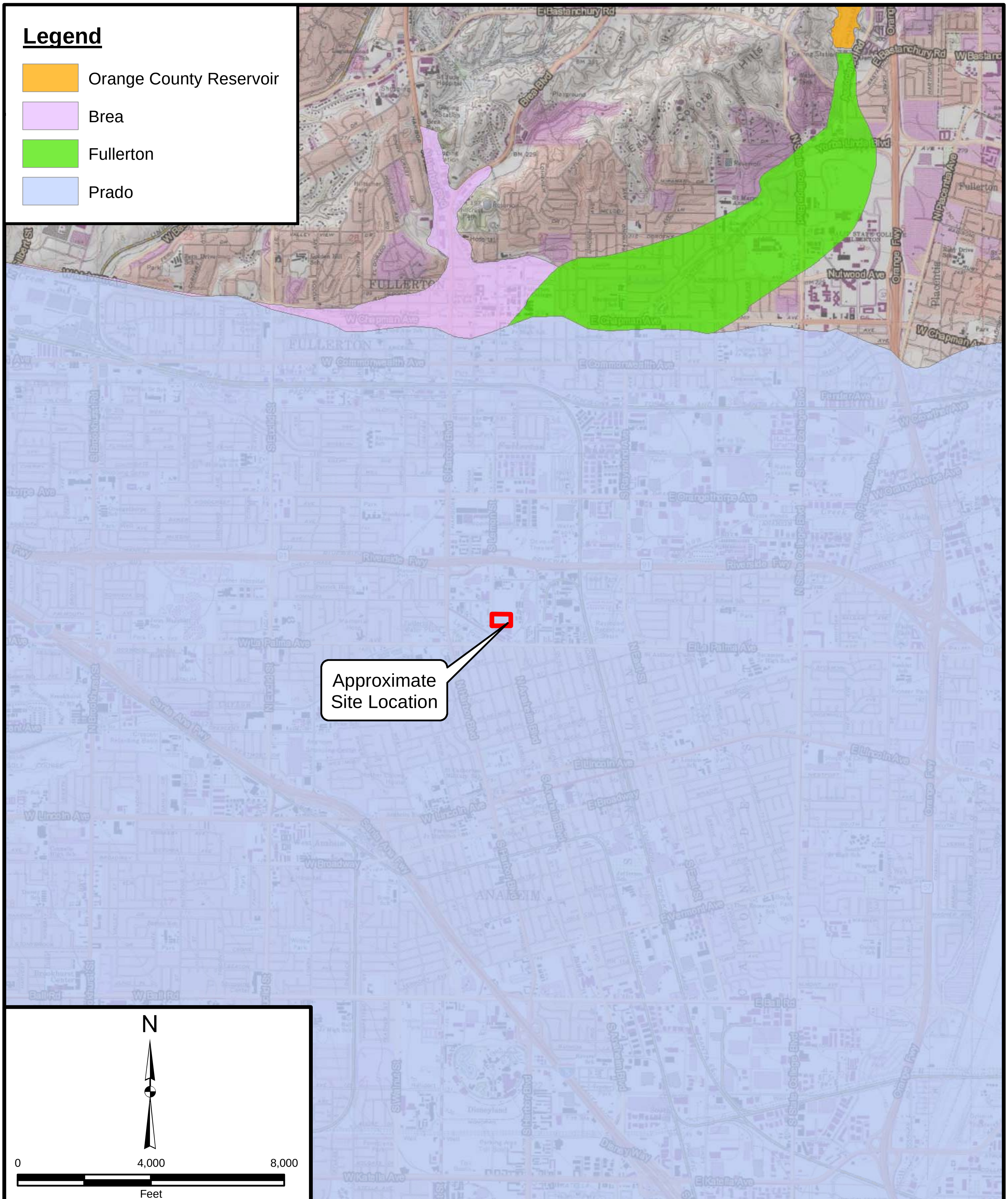

Project: 11862.003	Eng/Geol: CCK/JMP
Scale:1 " = 4,000 '	Date: March 2016

Base Map: CGS

Author: Leighton Geomatics (btran)

DAM INUNDATION MAP

1122 N. Anaheim Boulevard
Anaheim, California

Figure 7

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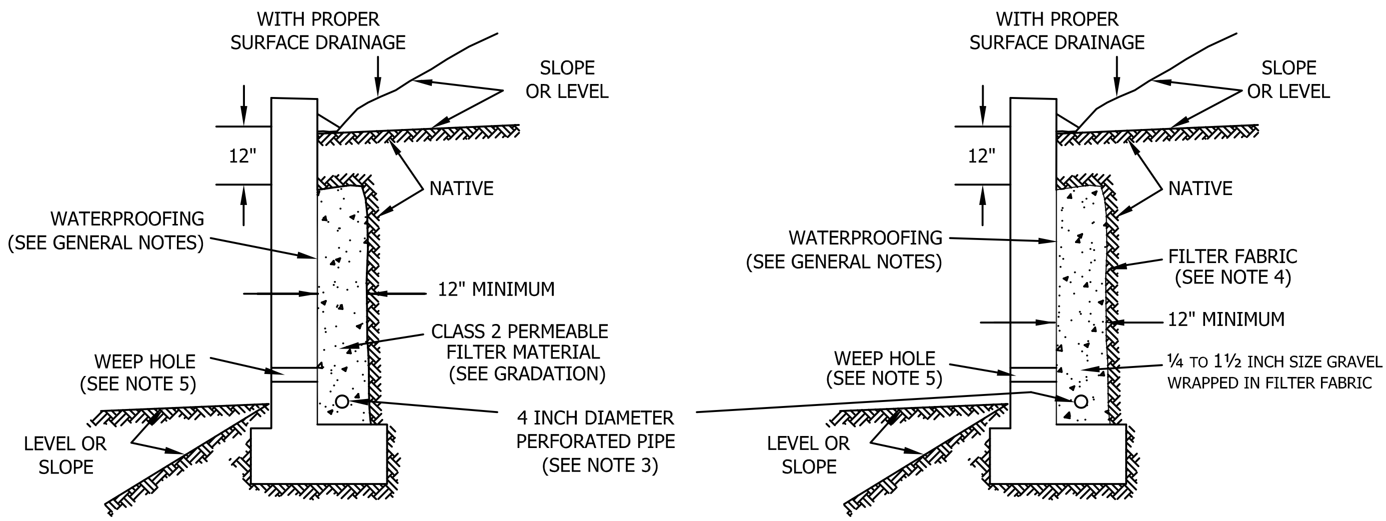
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

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Figure 8

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APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~158'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								@Surface: 3" asphalt concrete over 2" aggregate base	
								SM	**Artificial Fill, undocumented (Afu):** @0.42': Silty SAND with Clay, dark brown, moist to very moist, loose/soft	
								SP-SM	**Quaternary Young Alluvial Fan Deposits (Qyf):** @1.5': SAND to Silty SAND, light yellow brown, slightly moist to moist, fine to medium sand	
	5			R1	5 8 9	97	2	SP	@5': SAND, light yellow brown, moist, medium dense, fine to medium sand	
				R2	4 6 10	97	2		@7': Loose, moist, trace coarse sand, few coarse gravels	
	10			R3	4 5 9	107	16	SP-ML	@10': Sandy SILT to SAND, brown to orange brown, moist, stiff/loose, fine to medium sand	
	15			S1	4 9 12		2	SP	@15': SAND, light orange brown, slightly moist, medium dense, fine to medium sand	
	20			R4	7 10 8	97	2		@20': Orange-brown, fine sand	
	25			S2	4 10 13		2		@25': Light yellow brown	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~158'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R5	5 17 29	98	14	SP-CL	**Quaternary Young Alluvial Fan Deposits (Qyf), continued:** @30': Interlayered SAND and CLAY, light yellow brown to olive brown, slightly moist to very moist, medium dense/very stiff, fine sand	
	35			S3	8 14 15		3	SP	@35': SAND, light yellow brown, slightly moist, medium dense, fine sand	
				R6	9 17 27	117	5	SP-ML	@38.5': SAND to Sandy SILT, light yellow brown to gray brown, moist, medium dense/very stiff, fine to coarse sand	
	40								**Total Depth: 40 feet bgs** **No groundwater encountered during drilling** **Temporary percolation well installed:** **2-inch solid PVC @ 0-30 feet bgs** **2-inch slotted PVC (0.020") @ 30-40 feet bgs** **#3 Monterey Sand @ 29-40 feet bgs** **Upon completion of percolation testing, well casing removed, boring backfilled with soil cuttings**	
	45									
	50									
	55									
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-2

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~157'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B1				SM	@Surface: 3" asphalt concrete over artificial fill **Artificial Fill, undocumented (Afu):** @0.25': Silty SAND with Clay, brown, moist, fine to medium sand	
								SP-SM	**Quaternary Young Alluvial Fan Deposits (Qyf):** @2': SAND to Silty SAND, light yellow brown, slightly moist to moist, fine to medium sand	
	5			R1	3 4 10	105	2	SP	@5': SAND, light yellow brown, slightly moist, loose, fine to coarse sand	
	10			S1	3 5 5		2		@10': Loose to medium dense	
	15			R2	6 11 23	108	2		@15': Medium dense	
				S2	5 9 8		2			
	20								**Total Depth 20 feet bgs** **No groundwater encountered during drilling** **Temporary percolation well installed:** **2-inch solid PVC @ 0-15 feet bgs** **2-inch slotted PVC (0.020") @ 15-20 feet bgs** **#3 Monterey Sand @ 14-20 feet bgs** **Upon completion of percolation testing, well casing removed, boring backfilled with soil cuttings**	
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-3

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~159'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B1				ML	@Surface: 3" asphalt concrete over artificial fill	
								SP	**Artificial Fill, undocumented (Afu):** @0.25': Sandy SILT, brown, moist, fine sand	
									Quaternary Young Alluvial Fan Deposits (Qyf): @1': SAND, light yellow brown, slightly moist to moist, fine to medium sand	
	5			R1	3 5 7	95	3		@5': SAND, light yellow brown, slightly moist, loose, fine to medium sand	
	10			S1	1 3 4		3		@10': SAND, light orange brown, moist, loose, fine to coarse sand	
	15			R2	8 19 27	102	4		@15': SAND, light yellow brown, moist, medium dense, fine to medium sand	
				S2	4 9 6		4			
	20									
									Total Depth 20 feet bgs **No groundwater encountered during drilling** **Temporary percolation well installed:** **2-inch solid PVC @ 0-15 feet bgs** **2-inch slotted PVC (0.020") @ 15-20 feet bgs** **#3 Monterey Sand @ 14-20 feet bgs** **Upon completion of percolation testing, well casing removed, boring backfilled with soil cuttings**	
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-4

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~158'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							ML-SM	@Surface: 2" asphalt concrete over artificial fill **Artificial Fill, undocumented (Afu):** @0.17': Sandy SILT to Silty SAND, brown, moist, fine sand	
								SP	**Quaternary Young Alluvial Fan Deposits (Qyf):** @1.5': SAND, light yellow brown, moist, fine to medium sand, uniform	
	5			R1	3 6 11	100	3		@5': Medium dense	
				R2	4 8 11	95	3			
	10			R3	6 12 13	104	5		@10': Few fine gravels	
	15			S1	4 9 12		4			
	20			R4	11 14 9	102	4		@20': Fine sand	
	25			S2	2 4 2		23	CL-ML	@25': CLAY to Clayey SILT, olive brown, moist to very moist, medium stiff, trace fine sand, micaceous	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-4

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~158'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			R5	4 8 11	102	23	CL	**Quaternary Young Alluvial Fan Deposits (Qyf), continued:** @30': Silty CLAY, olive brown, very moist, stiff, trace fine sand, micaceous	
	35			S3	3 5 7		19	ML	@35': SILT to Clayey SILT, olive brown, moist to very moist, stiff, micaceous	
				R6	3 7 14	102	22	CL	@38.5': Silty CLAY, olive brown, very moist, stiff, micaceous	
	40								**Total Depth: 40 feet bgs** **No groundwater encountered during drilling** **Temporary percolation well installed:** **2-inch solid PVC @ 0-30 feet bgs** **2-inch slotted PVC (0.020") @ 30-40 feet bgs** **#3 Monterey Sand @ 29-40 feet bgs** **Upon completion of percolation testing, well casing removed, boring backfilled with soil cuttings**	
	45									
	50									
	55									
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-5

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~157'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							ML	@Surface: 3" asphalt concrete over artificial fill **Artificial Fill, undocumented (Afu):** @0.25': Sandy SILT, brown, very moist, fine sand	CR, EI, MD, RV
								SP	**Quaternary Young Alluvial Fan Deposits (Qyf):** @1.5': SAND, light yellow brown, slightly moist, fine to medium sand	
	5			R1	2 4 7	99	4		@5': Loose, some fine gravel	DS
				R2	7 13 16			SP-SM	@7': SAND with Silt, light yellow brown, slightly moist, medium dense, fine to medium sand	
	10			R3	6 11 13	102	5		@10': Fine sand	
	15			R4	7 14 20				@15': SAND with Silt, light yellow brown, moist, medium dense, fine to coarse sand, some fine gravel	DS
	20			S1	3 3 3		19	CL-ML	@20': Silty CLAY to Sandy SILT, olive brown, moist to very moist, medium stiff, micaceous, fine sand	
	25			R5	4 3 6				@25': Silty CLAY, olive brown, moist, medium stiff, fine sand, micaceous	CN
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-5

Project No.	11862.003	**Date Drilled**	8-19-19
Project	RCNA LLC Anaheim	**Logged By**	JMP
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~157'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			S2	4 7 10		13	SP-ML	**<u>Quaternary Young Alluvial Fan Deposits (Qyf), continued:</u>** @30': Interlayered Sandy SILT and SAND, olive brown to light yellow brown, moist, very stiff, medium dense, fine sand	
	35			R6	11 20 31	99	2	SP	@35': SAND, light yellow brown, slightly moist, dense, fine sand	
	40			S3	7 13 15		5		@40': Medium dense	
	45			R7	12 21 35	106	4		@45': Dense, fine to coarse sand	
	50			S4	9 12 16		5		@50': Medium dense, fine sand	
	55								**Total Depth 51.5 feet bgs** **No groundwater encountered during drilling** **Boring backfilled with soil cuttings** **Bentonite plug placed in bottom of hole and near surface** **Surface patched with asphalt concrete**	
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-6

Project No.	11862.003	**Date Drilled**	8-20-19
Project	RCNA LLC Anaheim	**Logged By**	KMD
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~155'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	KMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@Surface: 3" asphalt concrete over 5" aggregate base	
				B1				SM	**Artificial Fill, undocumented (Afu):** @0.67': Silty SAND, dark brown, moist, fine sand, some fine to medium gravels	
								SP-SM	**Quaternary Young Alluvial Fan Deposits (Qyf):** @2': SAND with Silt to Silty SAND, yellow brown, moist, fine to medium sand, some coarse sand, some fine to medium subround gravels, slightly micaceous	
	5			R1	4 5 8	98	3	SP	@5': SAND, tan, moist, medium dense, fine to medium sand, some coarse sand, few trace fine gravels, micaceous	
				R2	6 11 16	101	5		@7': Tan-brown, fine sand, few medium sand, some oxidation staining, weakly bedded to laminated, coarsening with depth	
	10			R3	5 3 5	98	24	ML	@10.75': SILT with Clay, olive, very moist, stiff, low plasticity, oxidized, some medium sand	
	15			S1	3 6 8		2	SP	@15': SAND, tan, moist, medium dense, medium sand, some fine and coarse sand, trace fine gravels, few to trace silt	
	20			R4	7 12 8	109	10	SM	@20': Silty SAND, olive, moist to very moist, medium dense, very fine to fine sand, laminated	
	25			S2	4 6 2		8	SP	@25': SAND, olive brown, moist, medium dense, very fine to fine sand, few medium to coarse sand, trace fine gravels, some to few silt	
								CL-ML	@26': Sandy Silty CLAY, olive, moist to very moist, fine grained sand, some oxidized laminations	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-6

Project No.	11862.003	**Date Drilled**	8-20-19
Project	RCNA LLC Anaheim	**Logged By**	KMD
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~155'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	KMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R5	10 17 27	107	5	SP	**Quaternary Young Alluvial Fan Deposits (Qyf), continued:** @30': SAND, tan/brown, very moist, dense, fine sand, some medium sand, few silt	
	35			S3	5 10 14		2		@35': SAND, pale tan, slightly moist to moist, medium dense, fine sand, some medium sand, few coarse sand, trace fine gravels, micaceous, few to trace silt	
	40			R6	19 25 43	109	3		@40': Moist, very dense, fine to medium sand, few coarse sand	
	45			S4	6 6 8		19		@45': Medium dense	
								ML	@45.83': SILT, variegated olive and red brown, moist, nonplastic, laminated, some oxidized laminations, few interlaminations of fine sand	
	50			R7	15 23 32	105	6	SP	@50': SAND, pale tan, very moist, dense, medium to coarse sand, some fine sand with lainations of sandy SILT, dark olive, very moist, fine sand, stiff	
									Total Depth 51.5 feet bgs **No groundwater encountered during drilling** **Boring backfilled with soil cuttings** **Bentonite plug placed in bottom of hole and near surface** **Surface patched with asphalt concrete**	
	55									
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-7

Project No.	11862.003	**Date Drilled**	8-20-19
Project	RCNA LLC Anaheim	**Logged By**	KMD
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~155'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	KMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								@Surface: 4" asphalt concrete over 6" aggregate base	
				B1				SM	**Artificial Fill, undocumented (Afu):** @0.83': Silty SAND with Clay, dark brown, moist, fine to medium sand, fine to coarse gravels	CR, EI, MD, RV
								SP	**Quaternary Young Alluvial Fan Deposits (Qyf):** @2.5': SAND with Silt, pale brown to tan, moist, fine to medium sand, some coarse sand	
	5			R1	6 10 7			SP-SM	@5': SAND, pale tan, slightly moist to moist, medium dense, fine sand, few medium sand, trace silt, micaceous	DS
				R2	6 10 15	101	3		@7.5': Moist, medium to coarse sand, trace fine gravel, fining with depth to fine to medium sand, few coarse sand	
	10			R3	5 9 11			SP	@10': SAND, light yellow brown, slightly moist, medium dense, coarsening to medium to coarse sand and trace fine gravels	DS
	15			R4	5 11 11	101	2		@15': Increase in grain size to coarse sand, some medium sand	
	20			S1	4 6 4		15	SM-CL	@20': Silty SAND with Clay, medium olive, very moist, medium dense, fine sand, slightly micaceous, grades to silty CLAY with sand, olive, very moist, stiff, low plasticity, some fine sand	
	25			R5	10 16 9			SM	@25': Silty SAND, tan, very moist, medium dense, fine sand, some medium to coarse sand	CN
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
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- DS DIRECT SHEAR
- EI EXPANSION INDEX
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GEOTECHNICAL BORING LOG LB-7

Project No.	11862.003	**Date Drilled**	8-20-19
Project	RCNA LLC Anaheim	**Logged By**	KMD
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	~155'
Location	1122 N. Anaheim Boulevard, Anaheim, CA	**Sampled By**	KMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S2	3 8 9		12	CL	**Quaternary Young Alluvial Fan Deposits (Qyf), continued:** @30': Sandy CLAY, dark olive, very moist, stiff, fine sand, medium to low plasticity, grades to Clayey Silty SAND, dark olive, very moist, medium dense, fine sand, some medium sand, then SAND, light brown, very moist, fine to medium sand, some silt	
								SC-SM		
								SP		
	35			R6	10 21 32	104	5		@35': SAND, pale tan, very moist, dense, fine sand, few medium sand, grades to medium to coarse sand at bottom of sample	
	40			S3	7 13 15		5		@40': Fine sand, few medium sand	
	45			R7	11 8 15	97	22			
								ML	@46': Becoming SILT, variegated olive and red brown, stiff, moist to very moist, nonplastic, laminated with oxidated laminations, very micaceous laminations	
	50			S4	7 13 13		7	SP	@50': SAND, pale tan to gray, slightly moist to moist, medium dense, very fine to fine sand, few silt, coarsening slightly with depth, single coarse gravel in sampler shoe	
									Total Depth 51.5 feet bgs **No groundwater encountered during drilling** **Boring backfilled with soil cuttings** **Bentonite plug placed in bottom of hole and near surface** **Surface patched with asphalt concrete**	
	55									
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
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- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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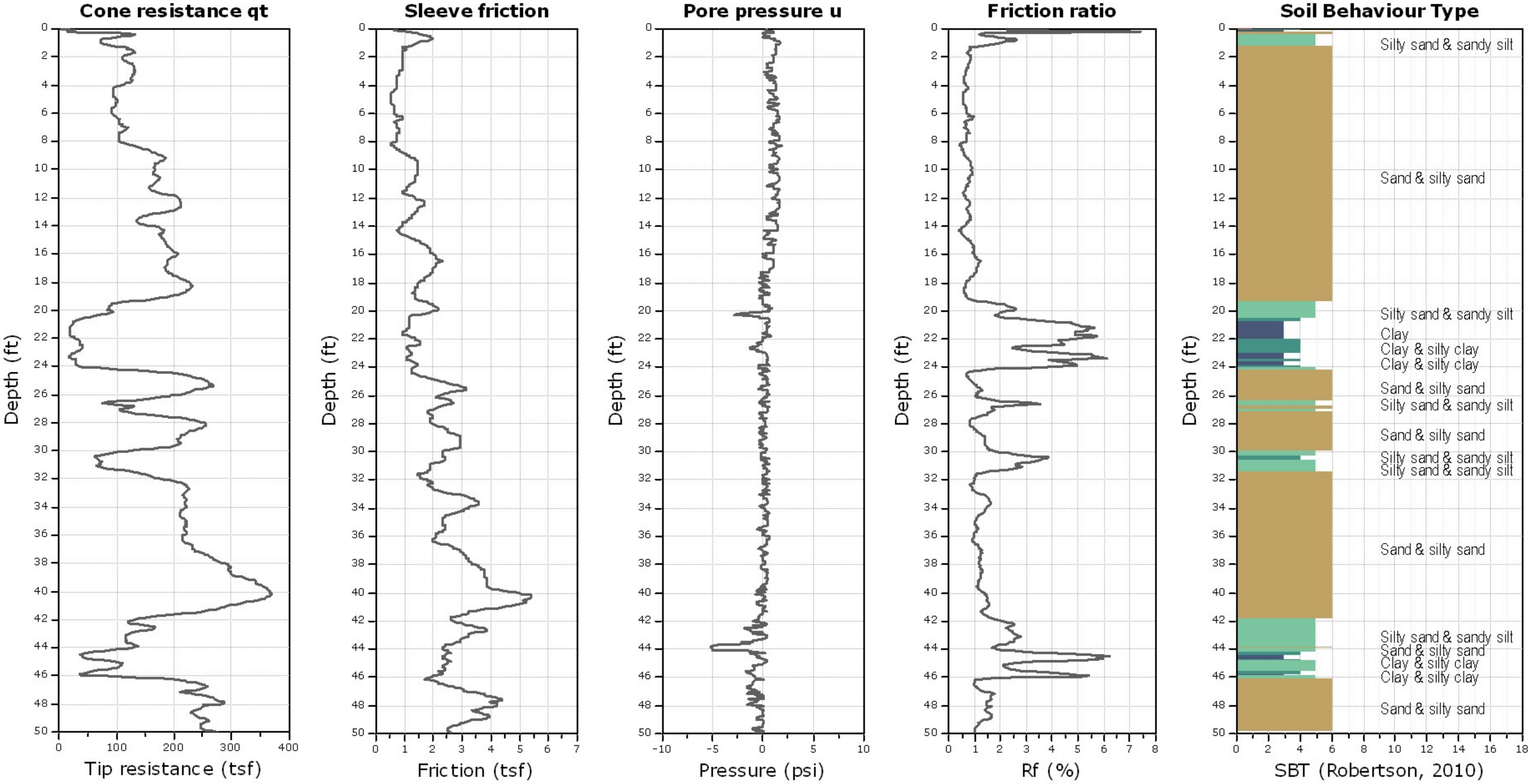
Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-1

Project: **Leighton & Asscociates/RPP Anaheim**
Location: **1041-1071 N. Kemp St & 1122 N. Anaheim Blvd Anaheim, CA**

Total depth: 50.28 ft, Date: 12/6/2017
Cone Type: Vertek



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

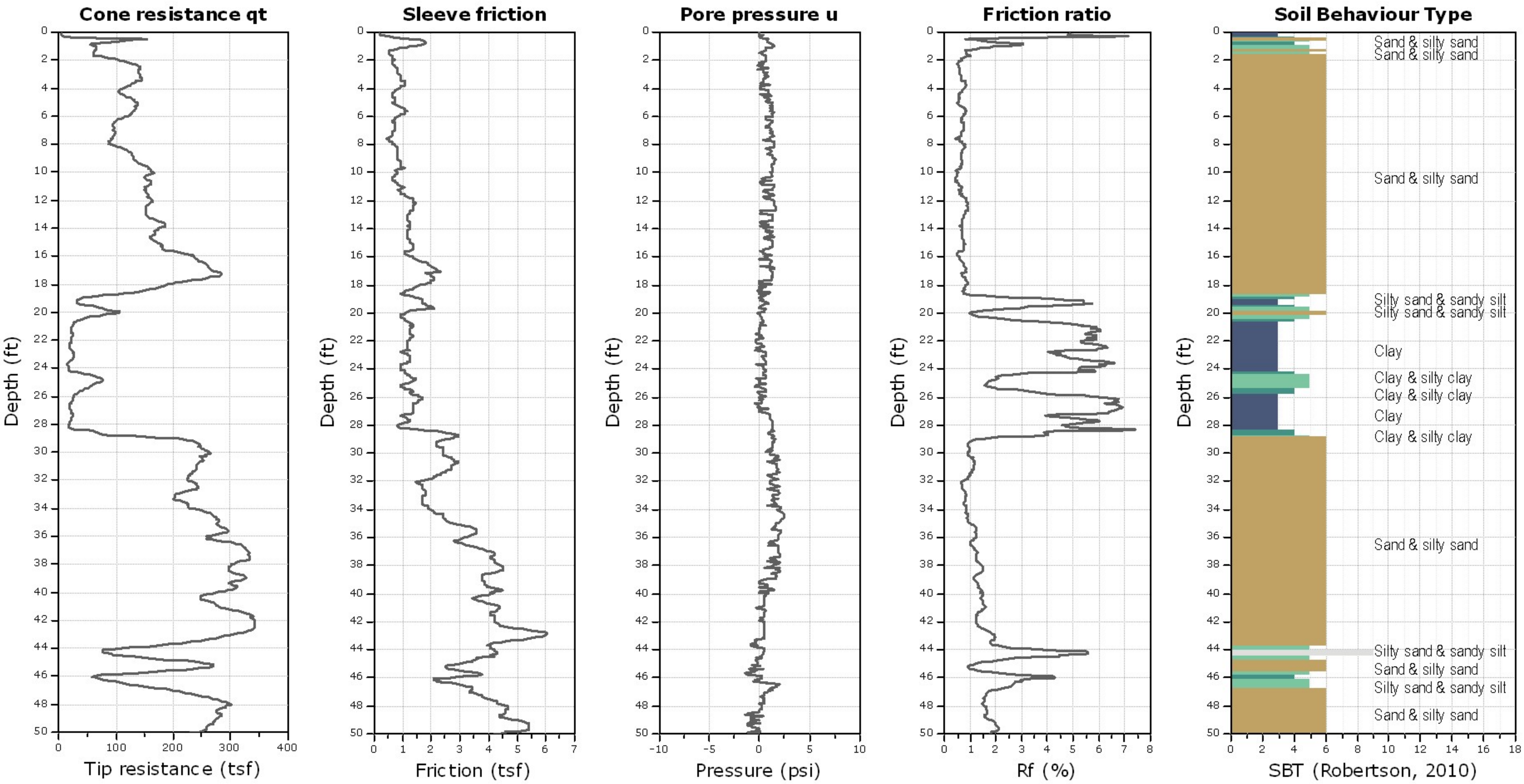
CPT-2

Project: **Leighton & Asscociates/RPP Anaheim**

Total depth: 50.40 ft, Date: 12/6/2017

Location: **1041-1071 N. Kemp St & 1122 N. Anaheim Blvd Anaheim, CA**

Cone Type: Vertek



N., Kemp St & N. Anaheim Blvd
Anaheim, CA

CPT Shear Wave Measurements

CPT-2	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
	10.07	9.07	10.36	17.14	604.25	
	20.11	19.11	19.75	29.28	674.63	774.00
	30.28	29.28	29.70	43.76	678.79	687.19
	40.12	39.12	39.44	53.26	740.49	1024.67
	50.07	49.07	49.32	63.28	779.46	986.61

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

APPENDIX C

PERCOLATION TEST DATA

Boring Percolation Test Data Sheet

Project Number:	11862.003	**Test Hole Number:**	LB-1
Project Name:	RCNA Anaheim	**Date Excavated:**	8/19/2019
Earth Description:	Alluvium	**Date Tested:**	8/20/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	40
Tested By:	JMP	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	7:34	**Length of slotted of casing (ft):**	10
Start Time for Standard:	8:35	**Depth to Initial Water Depth (ft):**	30
Standard Time Interval Between Readings, mins:	10	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
P1	7:34	25	30.00	120.0	91.2	0.27	2.24
	7:59		37.60	28.8			
P2	8:02	25	30.00	120.0	79.8	0.31	1.82
	8:27		36.65	40.2			
1	8:35	10	30.00	120.0	78.4	0.13	4.44
	8:45		36.53	41.6			
2	8:46	10	30.00	120.0	78.1	0.13	4.42
	8:56		36.51	41.9			
3	8:57	10	30.00	120.0	77.6	0.13	4.38
	9:07		36.47	42.4			
4	9:09	10	30.00	120.0	77.0	0.13	4.33
	9:19		36.42	43.0			
5	9:20	10	30.00	120.0	76.6	0.13	4.29
	9:30		36.38	43.4			
6	9:32	10	30.00	120.0	75.5	0.13	4.20
	9:42		36.29	44.5			
7	9:44	10	30.00	120.0	75.8	0.13	4.23
	9:54		36.32	44.2			
8	9:55	10	30.00	120.0	75.6	0.13	4.21
	10:05		36.30	44.4			
9	10:07	10	30.00	120.0	75.5	0.13	4.20
	10:17		36.29	44.5			
10	10:20	10	30.00	120.0	75.2	0.13	4.18
	10:30		36.27	44.8			

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Infiltration Rate, I (Last Reading) = $\boxed{\textbf{4.18}}$ **in./hr.**

Boring Percolation Test Data Sheet

Project Number:	11862.003	**Test Hole Number:**	LB-2
Project Name:	RCNA Anaheim	**Date Excavated:**	8/19/2019
Earth Description:	Alluvium	**Date Tested:**	8/20/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	20
Tested By:	JMP	**Radius of boring, r (in):**	4
		Radius of casing (in):	1
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	15
		Porosity of Annulus Material, *n* :	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	10:38	-	-	-	-
2	10:43	5	17.83	26.0	57.7
3	10:48	5	17.62	28.6	115.4
4	10:53	5	17.54	29.5	173.1
5	10:58	5	17.47	30.4	230.8
6	11:03	5	17.45	30.6	288.5
7	11:08	5	17.39	31.3	346.2
8	11:13	5	17.30	32.4	403.9
9	11:18	5	17.25	33.0	461.6
10	11:23	5	17.20	33.6	519.3
11	11:28	5	17.18	33.8	577.0
12	11:33	5	17.16	34.1	634.7
13	11:38	5	17.13	34.4	692.4
14	11:43	5	17.14	34.3	750.1
15	11:48	5	17.11	34.7	807.8
16	11:53	5	17.09	34.9	865.5
17	11:58	5	17.08	35.0	923.2
18	12:03	5	17.06	35.3	980.9
19	12:08	5	17.05	35.4	1038.6
20	12:13	5	17.04	35.5	1096.3
21	12:18	5	17.03	35.6	1154.0
22	12:23	5	17.00	36.0	1211.7
23	12:28	5	17.01	35.9	1269.4
24	12:33	5	16.99	36.1	1327.1
25	12:38	5	16.98	36.2	1384.8

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	1384.8
Total Volume of Water Delivered (cubic inches)	319888.8
Average Water Height (inches)	33.5
Average Percolation Surface Area (cubic Inches)	891.0
Duration of Test (minutes)	120
Duration of Test (hours)	2.00

$$\text{Measured Infiltration Rate} = (\text{Total Volume})/(\text{Test Duration})/(\text{Surface Area})$$

Measured Infiltration Rate (inches per hour) = 179.5

Boring Percolation Test Data Sheet

Project Number:	11862.003	**Test Hole Number:**	LB-3
Project Name:	RCNA Anaheim	**Date Excavated:**	8/19/2019
Earth Description:	Alluvium	**Date Tested:**	8/21/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	20
Tested By:	JMP	**Radius of boring, r (in):**	4
		Radius of casing (in):	1
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	15
		Porosity of Annulus Material, *n* :	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	7:20	-	-	-	-
2	7:25	5	16.45	42.6	39.2
3	7:30	5	16.20	45.6	78.4
4	7:35	5	15.96	48.5	117.6
5	7:40	5	15.83	50.0	156.8
6	7:45	5	15.79	50.5	196.0
7	7:50	5	15.90	49.2	235.2
8	7:55	5	15.64	52.3	274.4
9	8:00	5	15.59	52.9	313.6
10	8:05	5	15.63	52.4	352.8
11	8:10	5	15.50	54.0	392.0
12	8:15	5	15.47	54.4	431.2
13	8:20	5	15.57	53.2	470.4
14	8:25	5	15.42	55.0	509.6
15	8:30	5	15.40	55.2	548.8
16	8:35	5	15.37	55.6	588.0
17	8:40	5	15.34	55.9	627.2
18	8:45	5	15.31	56.3	666.4
19	8:50	5	15.28	56.6	705.6
20	8:55	5	15.25	57.0	744.8
21	9:00	5	15.25	57.0	784.0
22	9:05	5	15.25	57.0	823.2
23	9:10	5	15.20	57.6	862.4
24	9:15	5	15.18	57.8	901.6
25	9:20	5	15.18	57.8	940.8
26	9:25	5	15.15	58.2	980.0
27	9:30	5	15.13	58.4	1019.2
28	9:35	5	15.12	58.6	1058.4

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	1058.4
Total Volume of Water Delivered (cubic inches)	244494.024
Average Water Height (inches)	54.1
Average Percolation Surface Area (cubic Inches)	1409.0
Duration of Test (minutes)	135
Duration of Test (hours)	2.25

$$\text{Measured Infiltration Rate} = (\text{Total Volume})/(\text{Test Duration})/(\text{Surface Area})$$

Measured Infiltration Rate (inches per hour) = 77.1

Boring Percolation Test Data Sheet

Project Number:	11862.003	**Test Hole Number:**	LB-4
Project Name:	RCNA Anaheim	**Date Excavated:**	8/19/2019
Earth Description:	Alluvium	**Date Tested:**	8/21/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	40
Tested By:	KMD	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	10:14	**Length of slotted of casing (ft):**	10
Start Time for Standard:	11:06	**Depth to Initial Water Depth (ft):**	30
Standard Time Interval Between Readings, mins:	10	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
P1	10:14	25	30.00	120.0	43.4	0.58	0.81
	10:39		33.62	76.6			
P2	10:40	25	30.00	120.0	41.6	0.60	0.77
	11:05		33.47	78.4			
1	11:06	10	30.00	120.0	22.8	0.44	0.97
	11:16		31.90	97.2			
2	11:17	10	30.00	120.0	21.0	0.48	0.88
	11:27		31.75	99.0			
3	11:31	10	30.00	120.0	19.4	0.51	0.81
	11:41		31.62	100.6			
4	11:42	10	30.00	120.0	21.0	0.48	0.88
	11:52		31.75	99.0			
5	11:53	10	30.00	120.0	19.0	0.53	0.79
	12:03		31.58	101.0			
6	12:06	10	30.00	120.0	21.0	0.48	0.88
	12:16		31.75	99.0			
7	12:17	10	30.00	120.0	21.0	0.48	0.88
	12:27		31.75	99.0			
8	12:29	10	30.00	120.0	19.7	0.51	0.82
	12:39		31.64	100.3			
9	12:41	10	30.00	120.0	19.8	0.51	0.83
	12:51		31.65	100.2			
10	12:53	10	30.00	120.0	19.3	0.52	0.81
	13:03		31.61	100.7			
11	13:05	10	30.00	120.0	19.4	0.51	0.81
	13:15		31.62	100.6			

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Infiltration Rate, I (Last Reading) = $\boxed{0.81}$ **in./hr.**

APPENDIX D

LABORATORY TEST RESULTS

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	RCNA Anaheim	Tested By:	O. Figueroa	Date:	09/12/19
Project No.:	11862.003	Input By:	J. Ward	Date:	09/17/19
Boring No.:	LB-5	Depth (ft.):	0-5		
Sample No.:	B1				
Soil Identification:	Brown poorly-graded sand with silt (SP-SM)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03320

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3650	3697	3728	3746		
Weight of Mold (g)	1817	1817	1817	1817		
Net Weight of Soil (g)	1833	1880	1911	1929		
Wet Weight of Soil + Cont. (g)	378.3	434.7	463.5	496.2		
Dry Weight of Soil + Cont. (g)	353.9	400.5	416.2	436.6		
Weight of Container (g)	38.2	62.5	39.2	38.7		
Moisture Content (%)	7.73	10.12	12.55	14.98		
Wet Density (pcf)	121.7	124.8	126.9	128.1		
Dry Density (pcf)	113.0	113.4	112.8	111.4		

Maximum Dry Density (pcf) **113.4** **Optimum Moisture Content (%)** **9.8**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

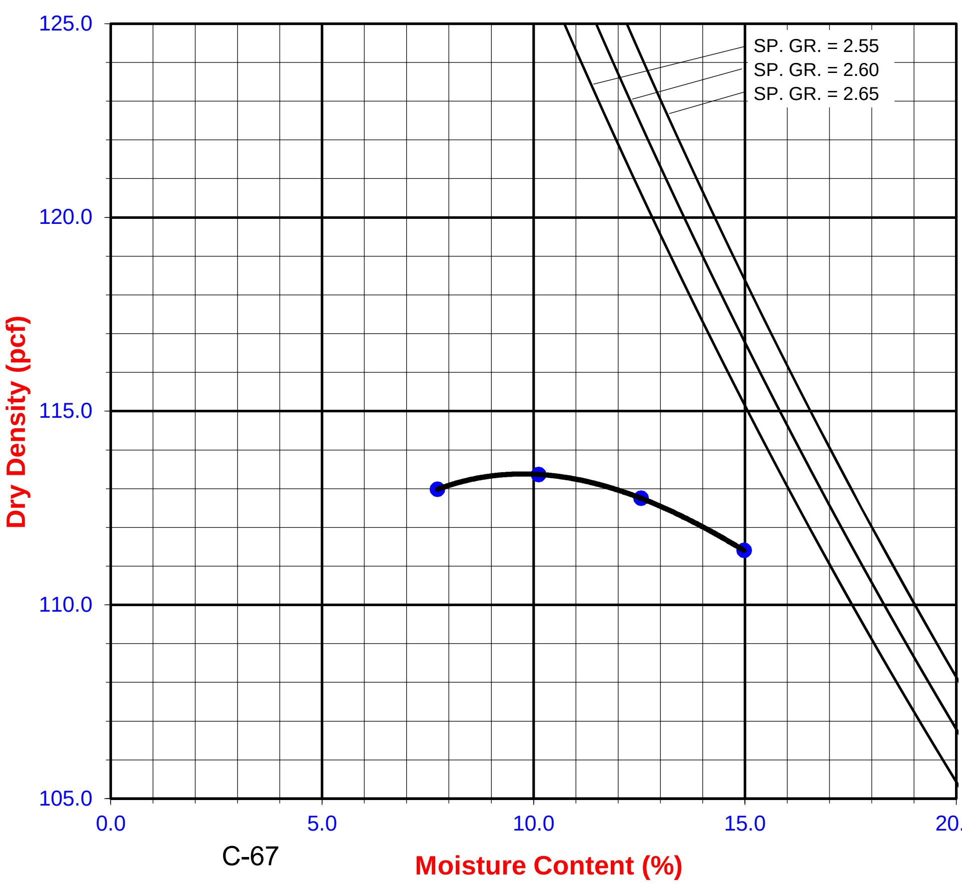
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MX LB-5, B1 @ 0-5

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	RCNA Anaheim	Tested By:	R. Manning	Date:	09/16/19
Project No.:	11862.003	Input By:	J. Ward	Date:	09/17/19
Boring No.:	LB-7	Depth (ft.):	0-5		
Sample No.:	B1				
Soil Identification:	Light olive brown poorly-graded sand with silt (SP-SM)				

Preparation Method: [X] Moist [ ] Dry [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03320 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3776	3906	3851			
Weight of Mold (g)	1817	1817	1817			
Net Weight of Soil (g)	1959	2089	2034			
Wet Weight of Soil + Cont. (g)	499.8	425.2	481.9			
Dry Weight of Soil + Cont. (g)	464.3	387.4	429.3			
Weight of Container (g)	39.3	39.0	37.7			
Moisture Content (%)	8.35	10.85	13.43			
Wet Density (pcf)	130.1	138.7	135.1			
Dry Density (pcf)	120.1	125.1	119.1			

Maximum Dry Density (pcf) 125.1 **Optimum Moisture Content (%)** 10.8

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

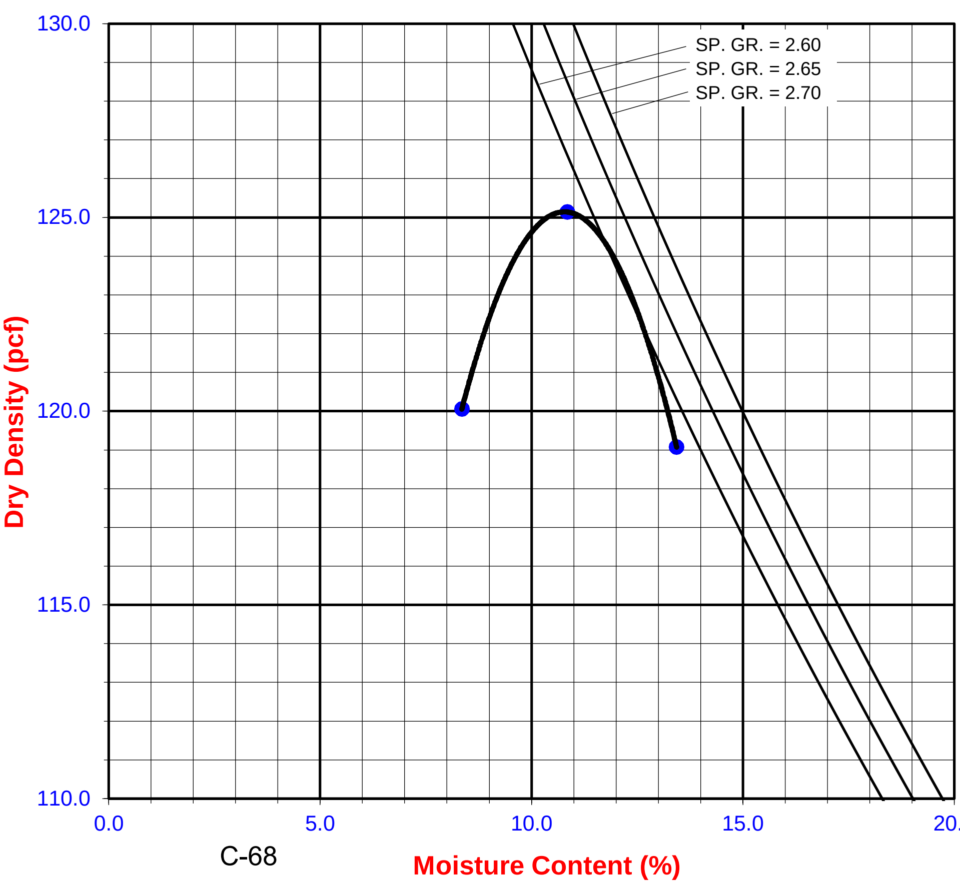
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MX LB-7, B1 @ 0-5

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	RCNA Anaheim	Tested By:	S. Felter	Date:	09/11/19
Project No.:	11862.003	Checked By:	J. Ward	Date:	09/17/19
Boring No.:	LB-5	Depth (ft.):	0-5		
Sample No.:	B1				
Soil Identification:	Brown poorly-graded sand with silt (SP-SM)				

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0000
Wt. Comp. Soil + Mold (g)	567.20	394.46
Wt. of Mold (g)	178.90	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	787.90	573.36
Dry Wt. of Soil + Cont. (g)	713.10	530.22
Wt. of Container (g)	0.00	178.90
Moisture Content (%)	10.49	12.28
Wet Density (pcf)	117.1	119.0
Dry Density (pcf)	106.0	106.0
Void Ratio	0.590	0.591
Total Porosity	0.371	0.371
Pore Volume (cc)	76.8	76.9
Degree of Saturation (%) [S_{meas}]	**48.0**	56.1

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
09/11/19	8:00	1.0	0	0.4760
09/11/19	8:10	1.0	10	0.4760
Add Distilled Water to the Specimen				
09/11/19	11:13	1.0	183	0.4760
09/12/19	6:45	1.0	1355	0.4760
09/12/19	7:43	1.0	1413	0.4760

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**0**

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	RCNA Anaheim	Tested By:	S. Felter	Date:	09/11/19
Project No.:	11862.003	Checked By:	J. Ward	Date:	09/17/19
Boring No.:	LB-7	Depth (ft.):	0-5		
Sample No.:	B1				
Soil Identification:	Light olive brown poorly-graded sand with silt (SP-SM)				

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0015
Wt. Comp. Soil + Mold (g)	584.60	418.59
Wt. of Mold (g)	184.60	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	798.10	603.19
Dry Wt. of Soil + Cont. (g)	725.50	548.06
Wt. of Container (g)	0.00	184.60
Moisture Content (%)	10.01	15.17
Wet Density (pcf)	120.7	126.1
Dry Density (pcf)	109.7	109.5
Void Ratio	0.537	0.540
Total Porosity	0.349	0.351
Pore Volume (cc)	72.3	72.7
Degree of Saturation (%) [S_{meas}]	**50.3**	75.8

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
09/11/19	8:25	1.0	0	0.3955
09/11/19	8:35	1.0	10	0.3955
Add Distilled Water to the Specimen				
09/11/19	11:12	1.0	157	0.3960
09/12/19	6:46	1.0	1331	0.3970
09/12/19	7:44	1.0	1389	0.3970

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**2**

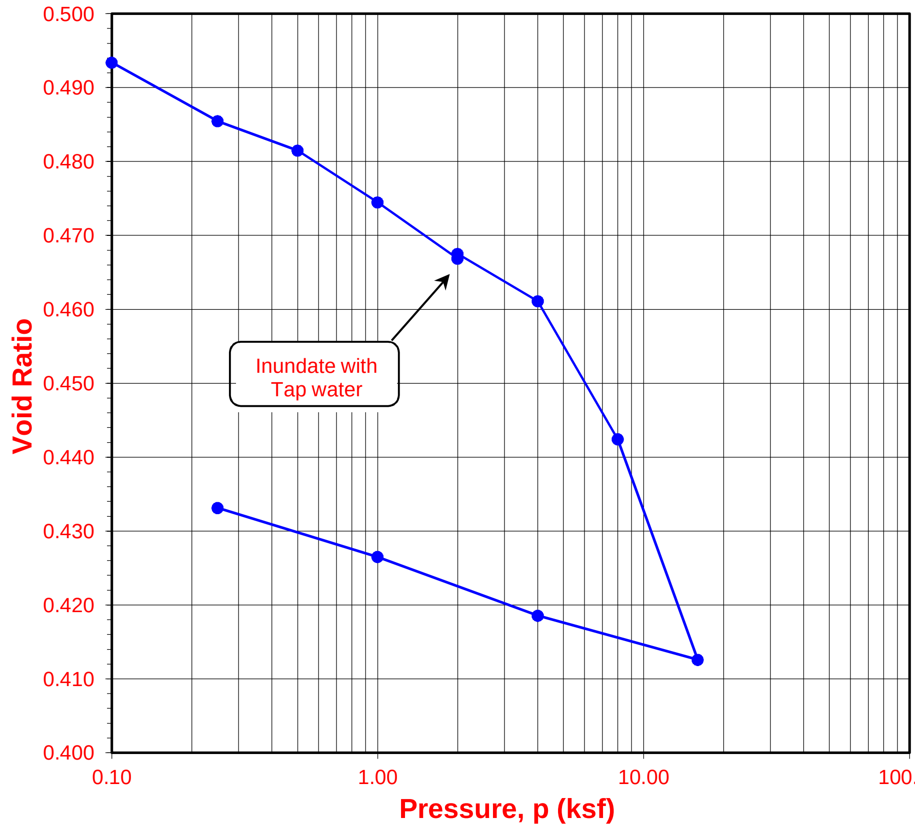
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	RCNA Anaheim	Tested By: G. Bathala	Date:	08/26/19
Project No.:	11862.003	Checked By: J. Ward	Date:	09/17/19
Boring No.:	LB-5	Depth (ft.): 25.0		
Sample No.:	R5	Sample Type:	Ring	
Soil Identification:	Olive brown silty clay (CL-ML)			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	203.51
Weight of Ring (g)	44.25
Height after consol. (in.)	0.9587
Before Test	
Wt.Wet Sample+Cont. (g)	211.12
Wt.of Dry Sample+Cont. (g)	188.54
Weight of Container (g)	59.24
Initial Moisture Content (%)	17.5
Initial Dry Density (pcf)	112.8
Initial Saturation (%)	95
Initial Vertical Reading (in.)	0.3139
After Test	
Wt.of Wet Sample+Cont. (g)	240.93
Wt. of Dry Sample+Cont. (g)	219.71
Weight of Container (g)	39.71
Final Moisture Content (%)	15.63
Final Dry Density (pcf)	117.8
Final Saturation (%)	98
Final Vertical Reading (in.)	0.2693
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

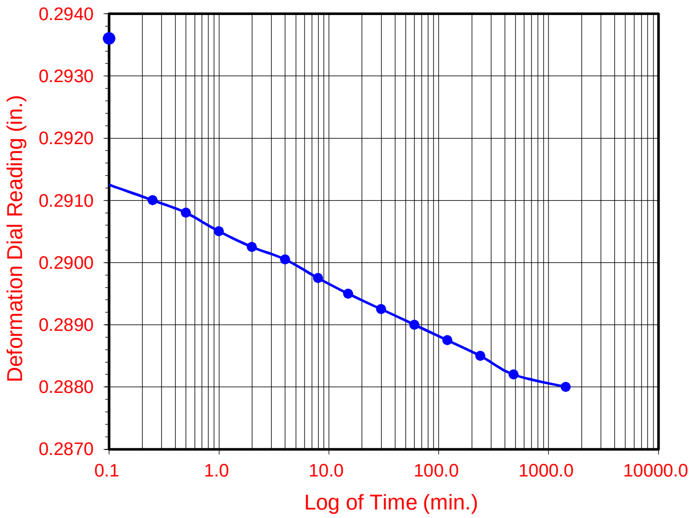


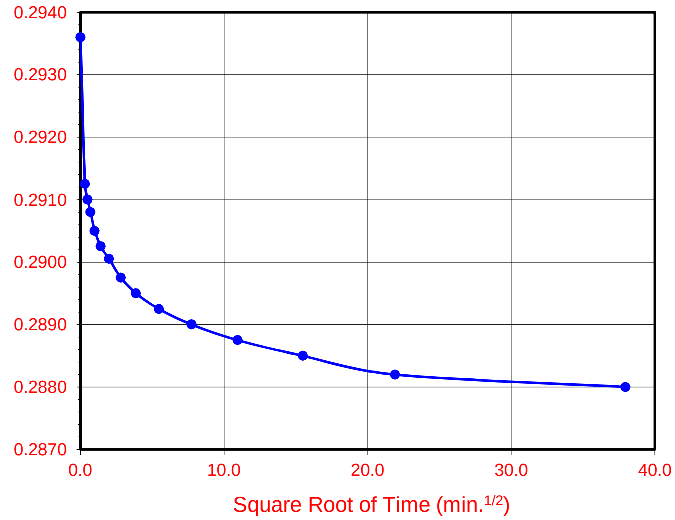
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3129	0.9990	0.00	0.10	0.493	0.10
0.25	0.3073	0.9934	0.03	0.66	0.485	0.63
0.50	0.3044	0.9905	0.06	0.95	0.482	0.89
1.00	0.2992	0.9853	0.11	1.48	0.474	1.37
2.00	0.2932	0.9793	0.20	2.08	0.467	1.88
2.00	0.2936	0.9797	0.20	2.03	0.468	1.83
4.00	0.2880	0.9741	0.33	2.59	0.461	2.26
8.00	0.2740	0.9601	0.48	3.99	0.442	3.51
16.00	0.2522	0.9383	0.67	6.18	0.413	5.51
4.00	0.2579	0.9440	0.50	5.60	0.419	5.10
1.00	0.2643	0.9504	0.39	4.97	0.426	4.58
0.25	0.2693	0.9554	0.33	4.46	0.433	4.13

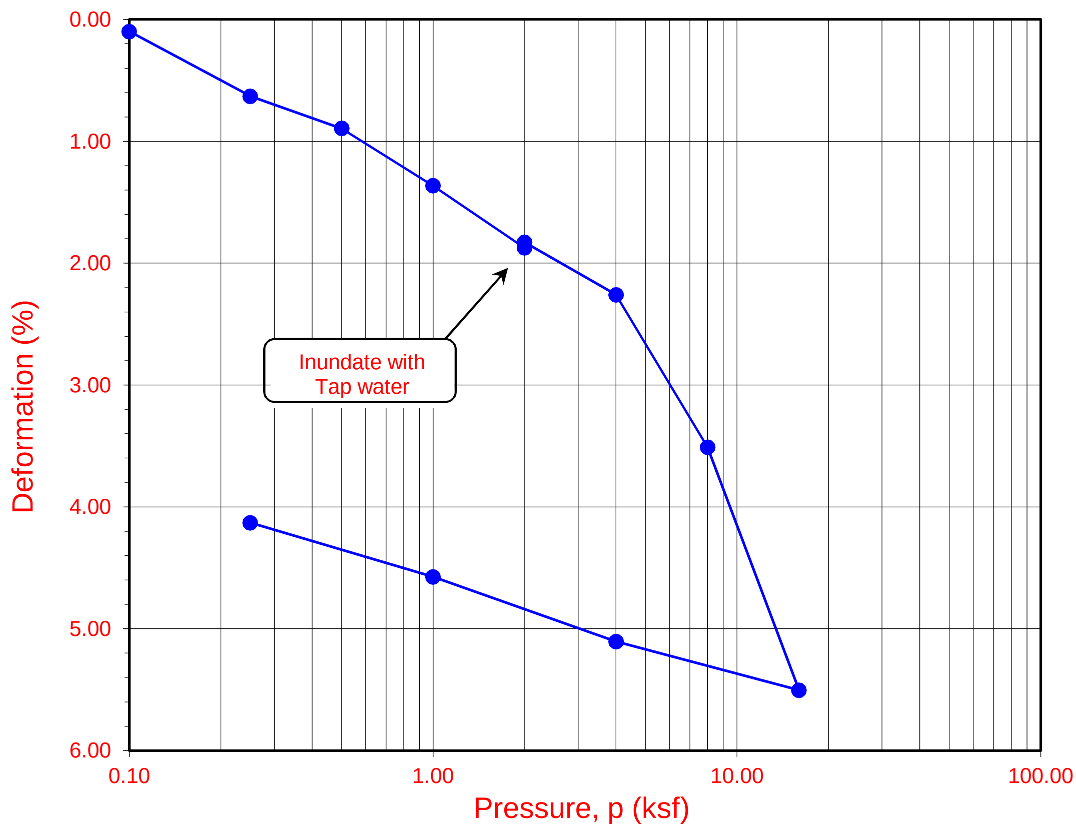
Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/29/19	8:00:00	0.0	0.0	0.2936
8/29/19	8:00:06	0.1	0.3	0.2913
8/29/19	8:00:15	0.2	0.5	0.2910
8/29/19	8:00:30	0.5	0.7	0.2908
8/29/19	8:01:00	1.0	1.0	0.2905
8/29/19	8:02:00	2.0	1.4	0.2903
8/29/19	8:04:00	4.0	2.0	0.2901
8/29/19	8:08:00	8.0	2.8	0.2898
8/29/19	8:15:00	15.0	3.9	0.2895
8/29/19	8:30:00	30.0	5.5	0.2893
8/29/19	9:00:00	60.0	7.7	0.2890
8/29/19	10:00:00	120.0	11.0	0.2888
8/29/19	12:00:00	240.0	15.5	0.2885
8/29/19	16:00:00	480.0	21.9	0.2882
8/30/19	8:00:00	1440.0	37.9	0.2880

Time Readings @ 4.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-5	**R5**	**25.0**	**17.5**	**15.6**	**112.8**	**117.8**	**0.495**	**0.433**	**95**	**98**

Soil Identification: Olive brown silty clay (CL-ML)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11862.003

RCNA Anaheim

09-19

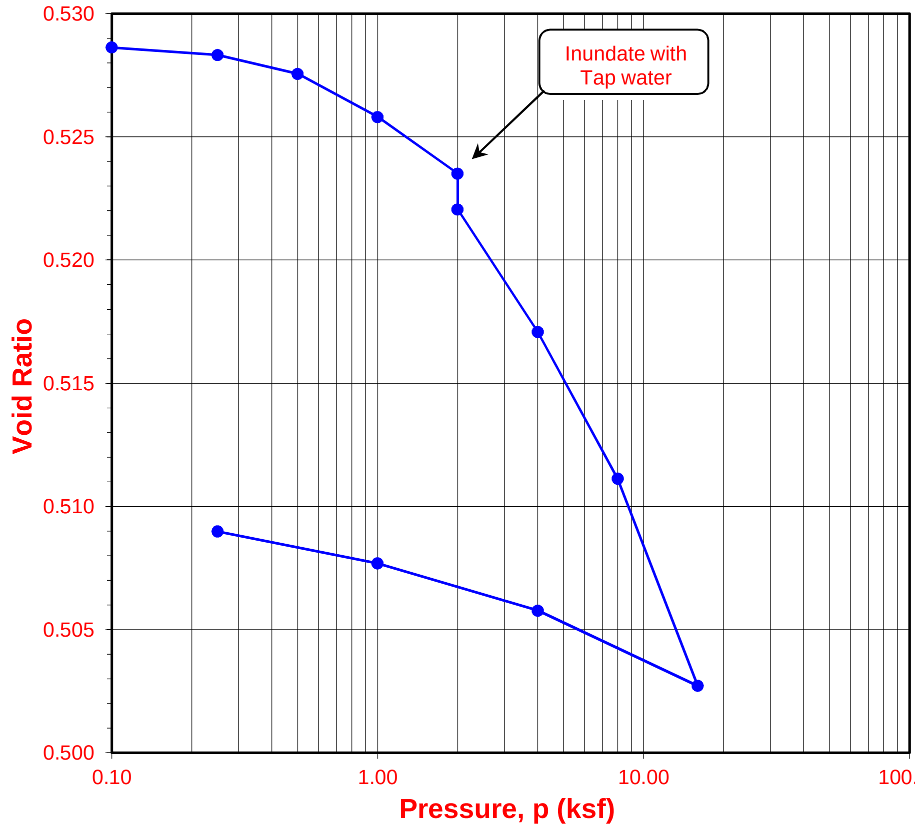
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	RCNA Anaheim	Tested By: G. Bathala	Date: 08/26/19
Project No.:	11862.003	Checked By: J. Ward	Date: 09/17/19
Boring No.:	LB-7	Depth (ft.): 25.0	
Sample No.:	R5	Sample Type: Ring	
Soil Identification:	Light olive brown silty sand (SM)		

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	179.28
Weight of Ring (g)	41.33
Height after consol. (in.)	0.9870
Before Test	
Wt.Wet Sample+Cont. (g)	173.25
Wt.of Dry Sample+Cont. (g)	168.71
Weight of Container (g)	56.85
Initial Moisture Content (%)	4.1
Initial Dry Density (pcf)	110.3
Initial Saturation (%)	21
Initial Vertical Reading (in.)	0.2844
After Test	
Wt.of Wet Sample+Cont. (g)	270.66
Wt. of Dry Sample+Cont. (g)	249.36
Weight of Container (g)	77.40
Final Moisture Content (%)	16.31
Final Dry Density (pcf)	110.1
Final Saturation (%)	83
Final Vertical Reading (in.)	0.2677
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

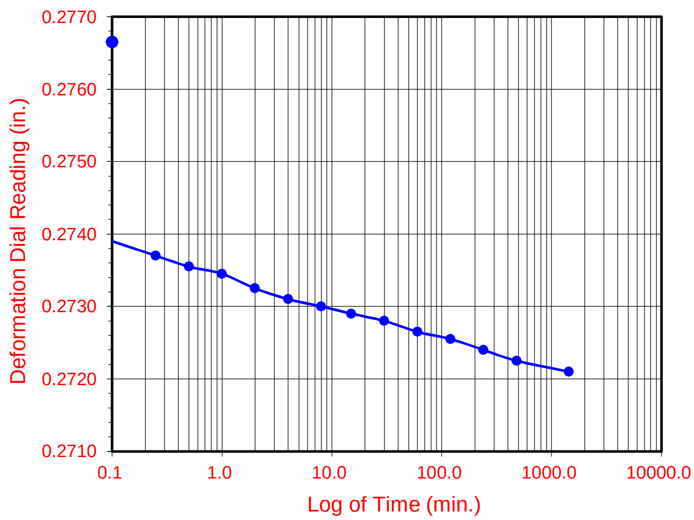


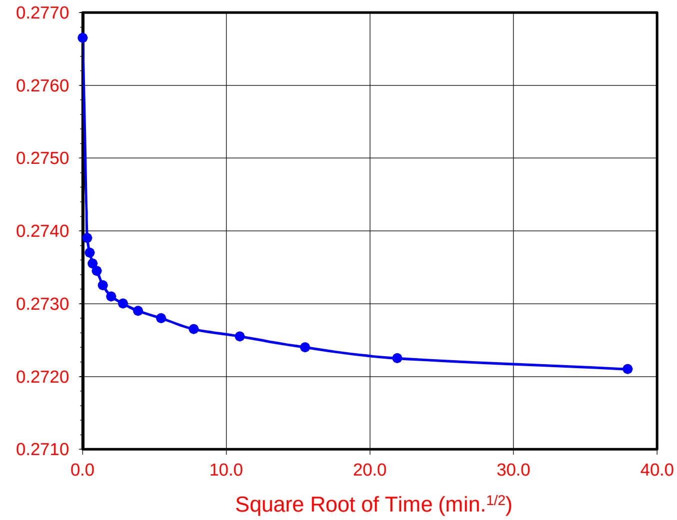
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2843	0.9999	0.00	0.01	0.529	0.01
0.25	0.2834	0.9990	0.07	0.11	0.528	0.03
0.50	0.2823	0.9979	0.13	0.21	0.528	0.08
1.00	0.2803	0.9959	0.21	0.41	0.526	0.20
2.00	0.2776	0.9932	0.33	0.68	0.524	0.35
2.00	0.2767	0.9923	0.33	0.77	0.522	0.44
4.00	0.2721	0.9877	0.46	1.23	0.517	0.77
8.00	0.2664	0.9820	0.64	1.80	0.511	1.16
16.00	0.2587	0.9743	0.86	2.57	0.503	1.71
4.00	0.2625	0.9781	0.68	2.19	0.506	1.51
1.00	0.2656	0.9812	0.50	1.89	0.508	1.39
0.25	0.2677	0.9833	0.37	1.67	0.509	1.30

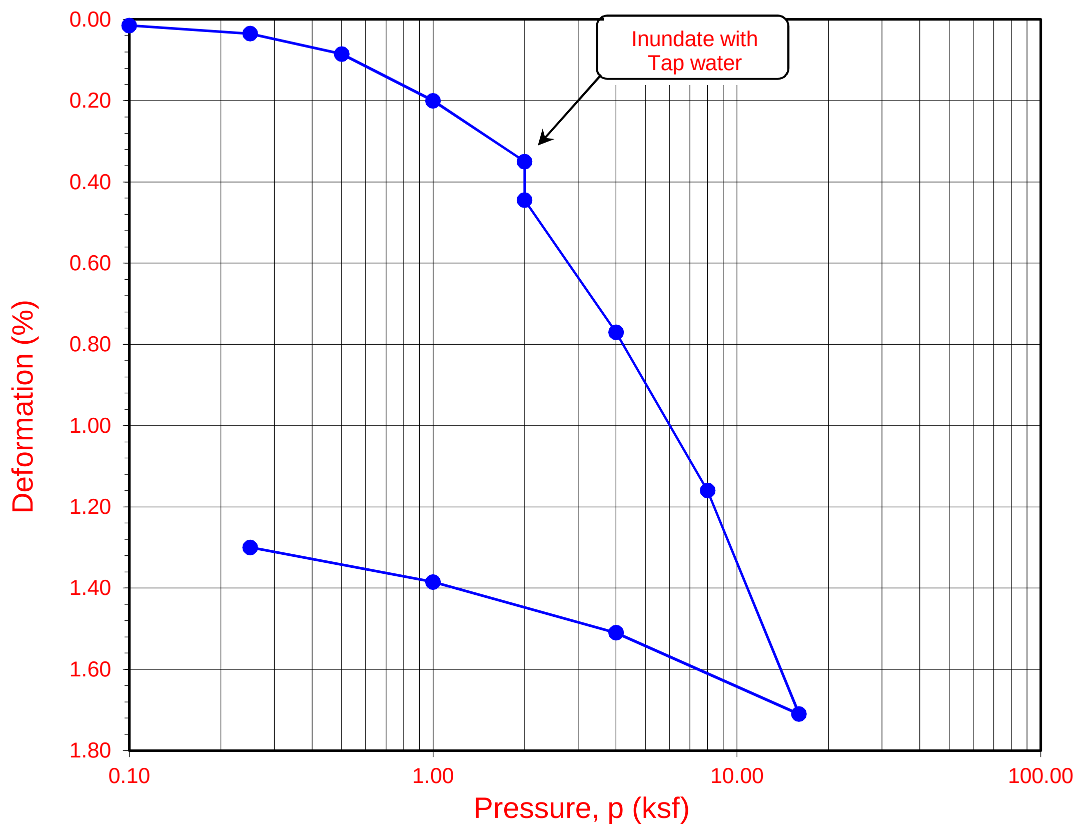
Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/29/19	8:05:00	0.0	0.0	0.2767
8/29/19	8:05:06	0.1	0.3	0.2739
8/29/19	8:05:15	0.2	0.5	0.2737
8/29/19	8:05:30	0.5	0.7	0.2736
8/29/19	8:06:00	1.0	1.0	0.2735
8/29/19	8:07:00	2.0	1.4	0.2733
8/29/19	8:09:00	4.0	2.0	0.2731
8/29/19	8:13:00	8.0	2.8	0.2730
8/29/19	8:20:00	15.0	3.9	0.2729
8/29/19	8:35:00	30.0	5.5	0.2728
8/29/19	9:05:00	60.0	7.7	0.2727
8/29/19	10:05:00	120.0	11.0	0.2726
8/29/19	12:05:00	240.0	15.5	0.2724
8/29/19	16:05:00	480.0	21.9	0.2723
8/30/19	8:05:00	1440.0	37.9	0.2721

Time Readings @ 4.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-7	**R5**	**25.0**	**4.1**	**16.3**	**110.3**	**110.1**	**0.529**	**0.509**	**21**	**83**

Soil Identification: Light olive brown silty sand (SM)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11862.003

RCNA Anaheim

09-19

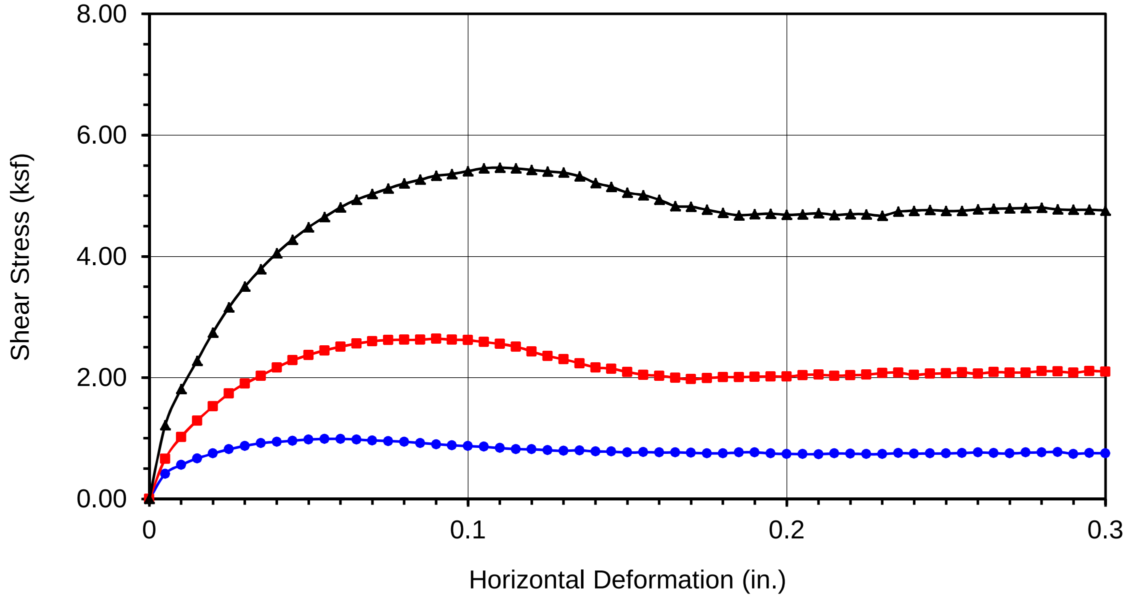
Leighton

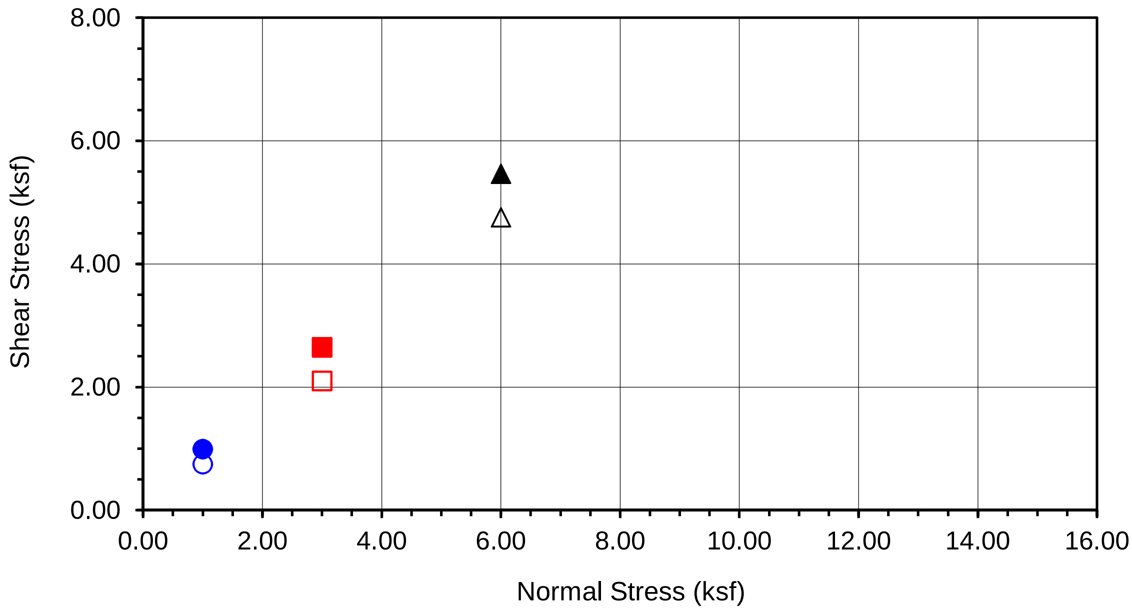
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: RCNA Anaheim Tested By: G. Bathala Date: 09/05/19
Project No.: 11862.003 Checked By: J. Ward Date: 09/17/19
Boring No.: LB-5 Sample Type: Ring
Sample No.: R2 Depth (ft.): 7.0
Soil Identification: Light olive brown poorly-graded sand with silt (SP-SM)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	173.14	176.62	178.92
Weight of Ring(gm):	44.17	44.55	45.65
Before Shearing			
Weight of Wet Sample+Cont.(gm):	188.80	188.80	188.80
Weight of Dry Sample+Cont.(gm):	183.79	183.79	183.79
Weight of Container(gm):	57.20	57.20	57.20
Vertical Rdg.(in): Initial	0.0000	0.2521	0.2552
Vertical Rdg.(in): Final	-0.0086	0.2664	0.2765
After Shearing			
Weight of Wet Sample+Cont.(gm):	180.41	182.29	214.03
Weight of Dry Sample+Cont.(gm):	157.83	161.24	193.04
Weight of Container(gm):	37.24	39.70	69.88
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-5**
Sample No.	**R2**
Depth (ft)	**7**

Sample Type:

Ring

Soil Identification:
Light olive brown poorly-graded sand with silt (SP-SM)

Normal Stress (kip/ft²)	1.000	3.000	6.000
Peak Shear Stress (kip/ft²)	● 0.990	■ 2.644	▲ 5.464
Shear Stress @ End of Test (ksf)	○ 0.748	□ 2.100	△ 4.757
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.96	3.96	3.96
Dry Density (pcf)	103.2	105.7	106.6
Saturation (%)	16.9	17.9	18.4
Soil Height Before Shearing (in.)	0.9914	0.9857	0.9787
Final Moisture Content (%)	18.7	17.3	17.0

Leighton

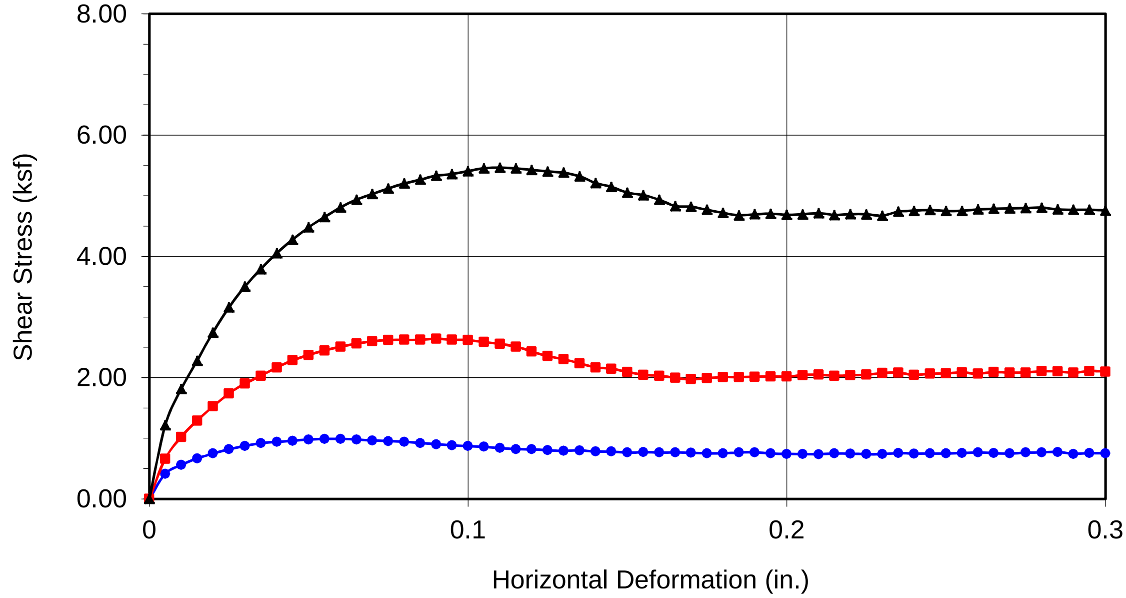
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

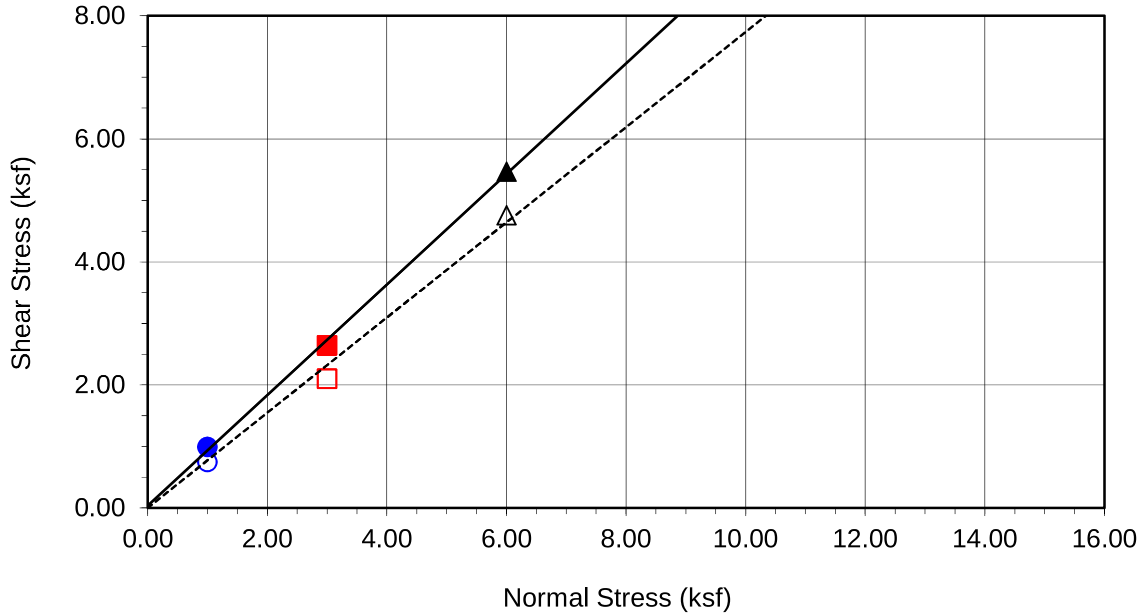
Project No.: 11862.003

RCNA Anaheim

09-19

DS LB-5, R2 @ 7





Boring No.	**LB-5**
Sample No.	**R2**
Depth (ft)	**7**
Sample Type:	Ring

Soil Identification:
Light olive brown poorly-graded sand with silt (SP-SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	38	42
Ultimate	0	38

Normal Stress (kip/ft²)	1.000	3.000	6.000
Peak Shear Stress (kip/ft²)	● 0.990	■ 2.644	▲ 5.464
Shear Stress @ End of Test (ksf)	○ 0.748	□ 2.100	△ 4.757
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.96	3.96	3.96
Dry Density (pcf)	103.2	105.7	106.6
Saturation (%)	16.9	17.9	18.4
Soil Height Before Shearing (in.)	0.9914	0.9857	0.9787
Final Moisture Content (%)	18.7	17.3	17.0

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11862.003

RCNA Anaheim

09-19

DS LB-5, R2 @ 7

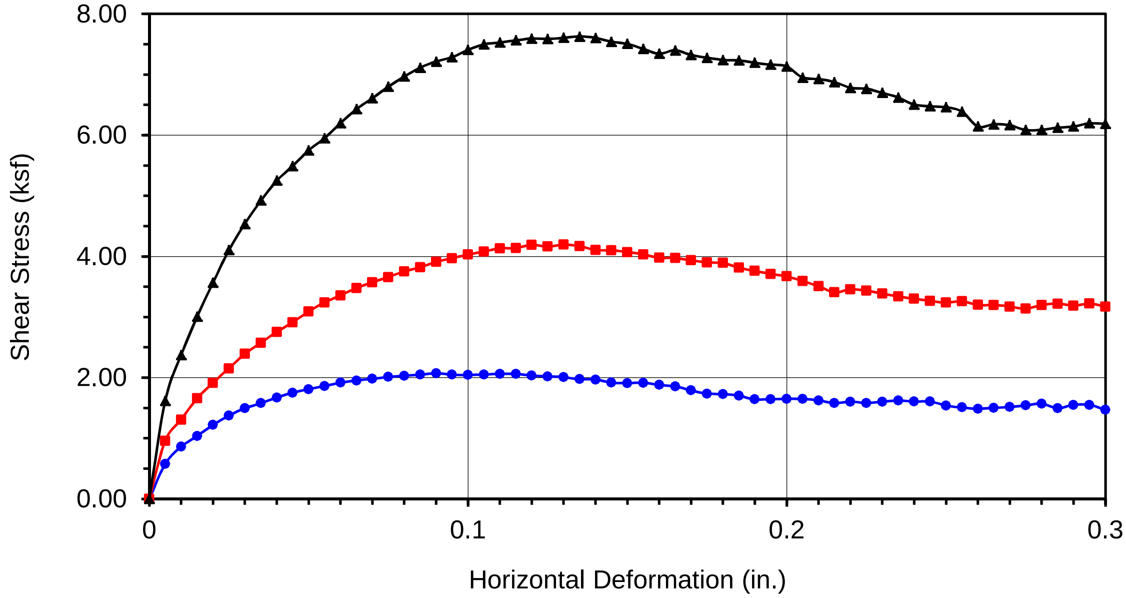
DIRECT SHEAR TEST

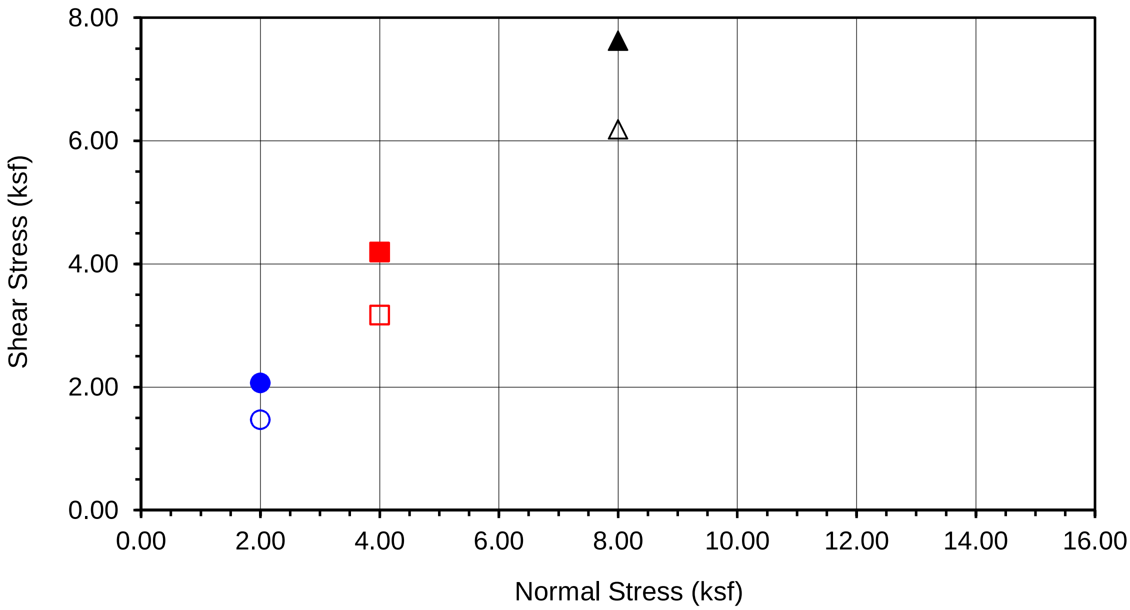
Consolidated Drained - ASTM D 3080

Project Name:	RCNA Anaheim	Tested By:	G. Bathala	Date:	09/07/19
Project No.:	11862.003	Checked By:	J. Ward	Date:	09/17/19
Boring No.:	LB-5	Sample Type:	Ring		
Sample No.:	R4	Depth (ft.):	15.0		

Soil Identification: Light olive brown poorly-graded sand with silt (SP-SM)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	172.11	175.99	177.33
Weight of Ring(gm):	40.46	41.68	42.80
Before Shearing			
Weight of Wet Sample+Cont.(gm):	226.30	226.30	226.30
Weight of Dry Sample+Cont.(gm):	221.23	221.23	221.23
Weight of Container(gm):	57.51	57.51	57.51
Vertical Rdg.(in): Initial	0.2293	0.0000	0.0000
Vertical Rdg.(in): Final	0.2412	-0.0173	-0.0302
After Shearing			
Weight of Wet Sample+Cont.(gm):	205.50	208.93	204.81
Weight of Dry Sample+Cont.(gm):	183.77	189.08	186.24
Weight of Container(gm):	58.53	63.40	61.49
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-5**
Sample No.	**R4**
Depth (ft)	**15**

Sample Type:

Ring

Soil Identification:
Light olive brown poorly-graded sand with silt (SP-SM)

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.069	■ 4.194	▲ 7.627
Shear Stress @ End of Test (ksf)	○ 1.468	□ 3.169	△ 6.187
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.10	3.10	3.10
Dry Density (pcf)	106.2	108.3	108.5
Saturation (%)	14.2	15.0	15.1
Soil Height Before Shearing (in.)	0.9881	0.9827	0.9698
Final Moisture Content (%)	17.4	15.8	14.9

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DIRECT SHEAR TEST RESULTS

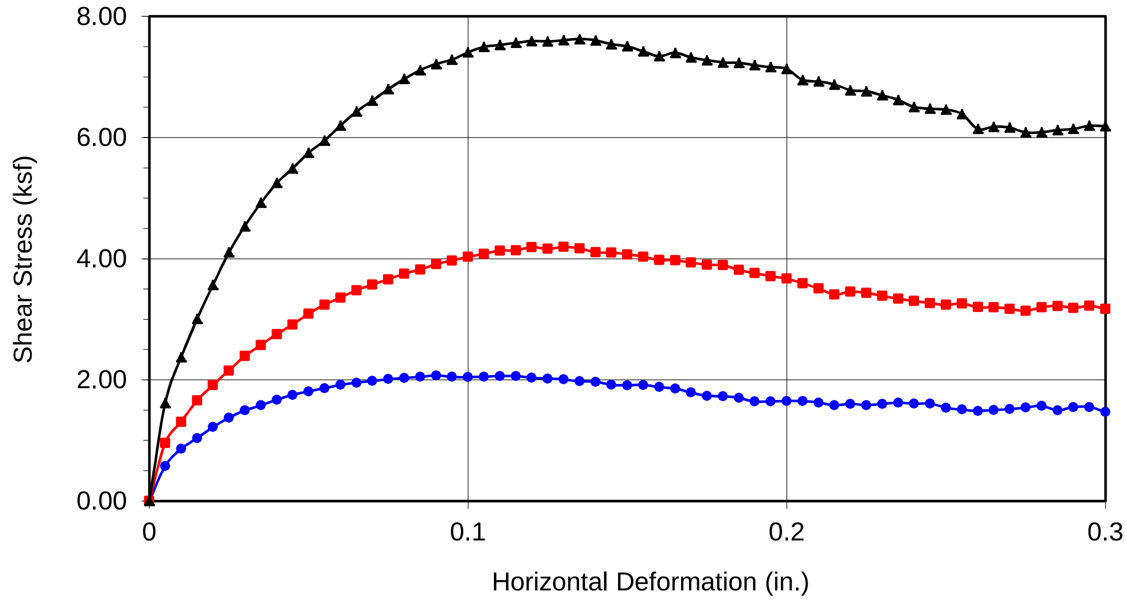
Consolidated Drained - ASTM D 3080

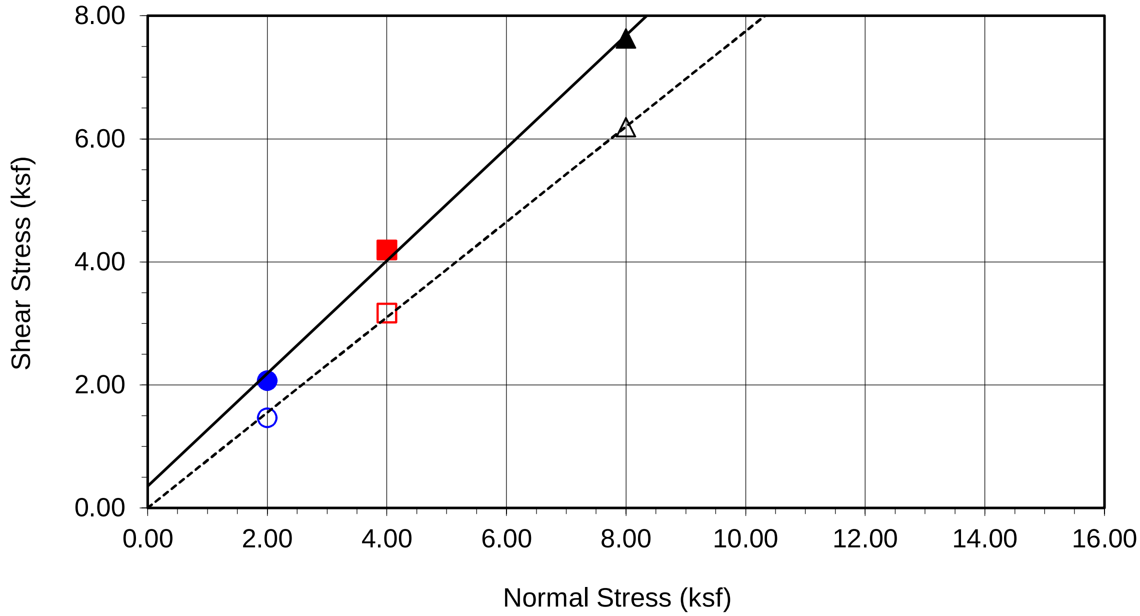
Project No.: 11862.003

RCNA Anaheim

09-19

DS LB-5, R4 @ 15





Boring No.	**LB-5**
Sample No.	**R4**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:
Light olive brown poorly-graded sand with silt (SP-SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	352	43
Ultimate	0	38

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.069	■ 4.194	▲ 7.627
Shear Stress @ End of Test (ksf)	○ 1.468	□ 3.169	△ 6.187
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.10	3.10	3.10
Dry Density (pcf)	106.2	108.3	108.5
Saturation (%)	14.2	15.0	15.1
Soil Height Before Shearing (in.)	0.9881	0.9827	0.9698
Final Moisture Content (%)	17.4	15.8	14.9

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11862.003

RCNA Anaheim

09-19

DS LB-5, R4 @ 15

DIRECT SHEAR TEST

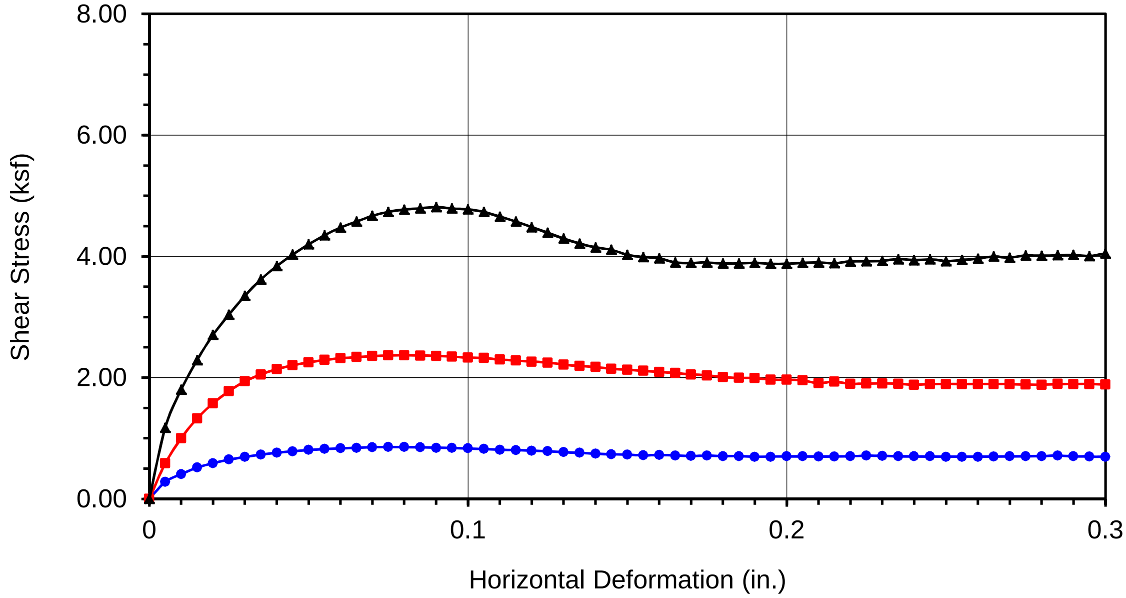
Consolidated Drained - ASTM D 3080

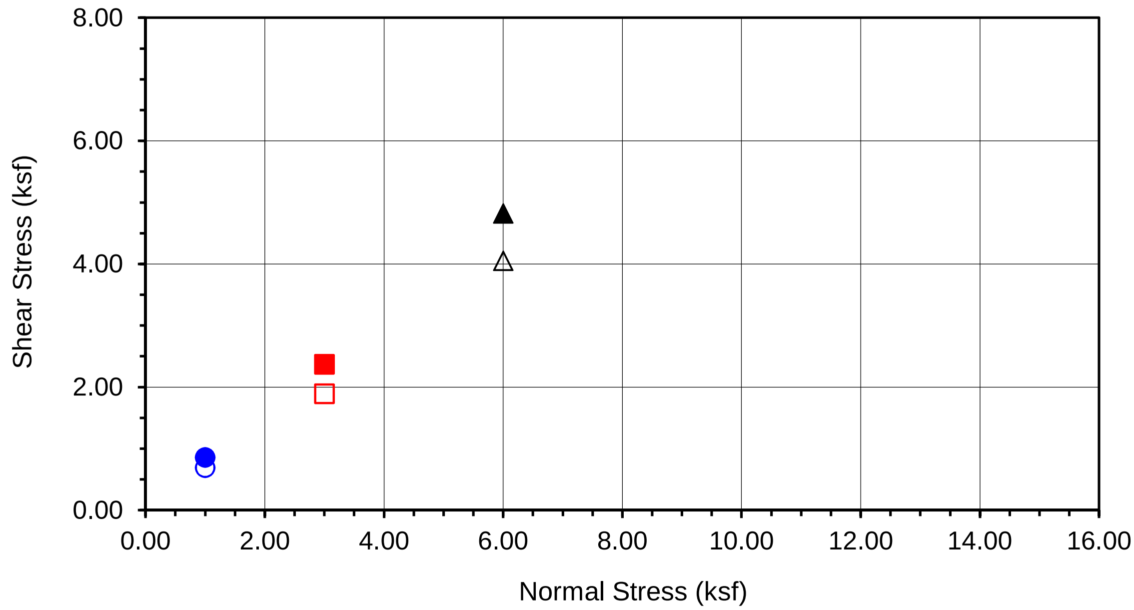
Leighton

Project Name:	RCNA Anaheim	Tested By:	G. Bathala	Date:	09/07/19
Project No.:	11862.003	Checked By:	J. Ward	Date:	09/17/19
Boring No.:	LB-7	Sample Type:	Ring		
Sample No.:	R1	Depth (ft.):	5.0		

Soil Identification: Light olive brown poorly-graded sand with silt (SP-SM)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	167.26	166.31	168.56
Weight of Ring(gm):	45.49	43.76	44.00
Before Shearing			
Weight of Wet Sample+Cont.(gm):	186.17	186.17	186.17
Weight of Dry Sample+Cont.(gm):	183.42	183.42	183.42
Weight of Container(gm):	57.28	57.28	57.28
Vertical Rdg.(in): Initial	0.2477	0.2467	0.0000
Vertical Rdg.(in): Final	0.2539	0.2693	-0.0269
After Shearing			
Weight of Wet Sample+Cont.(gm):	203.70	194.07	198.94
Weight of Dry Sample+Cont.(gm):	180.87	173.15	176.93
Weight of Container(gm):	67.21	58.94	59.05
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-7**
Sample No.	**R1**
Depth (ft)	**5**

Sample Type:

Ring

Soil Identification:
Light olive brown poorly-graded sand with silt (SP-SM)

Normal Stress (kip/ft²)	1.000	3.000	6.000
Peak Shear Stress (kip/ft²)	● 0.855	■ 2.370	▲ 4.816
Shear Stress @ End of Test (ksf)	○ 0.692	□ 1.889	△ 4.049
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	2.18	2.18	2.18
Dry Density (pcf)	99.1	99.7	101.4
Saturation (%)	8.4	8.5	8.9
Soil Height Before Shearing (in.)	0.9938	0.9774	0.9731
Final Moisture Content (%)	20.1	18.3	18.7

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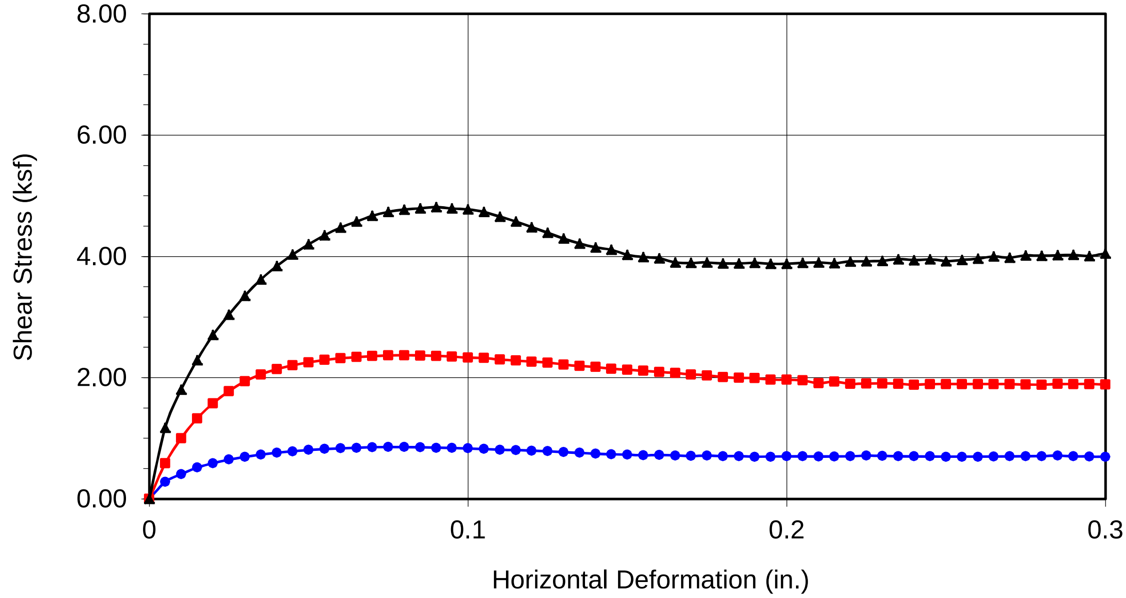
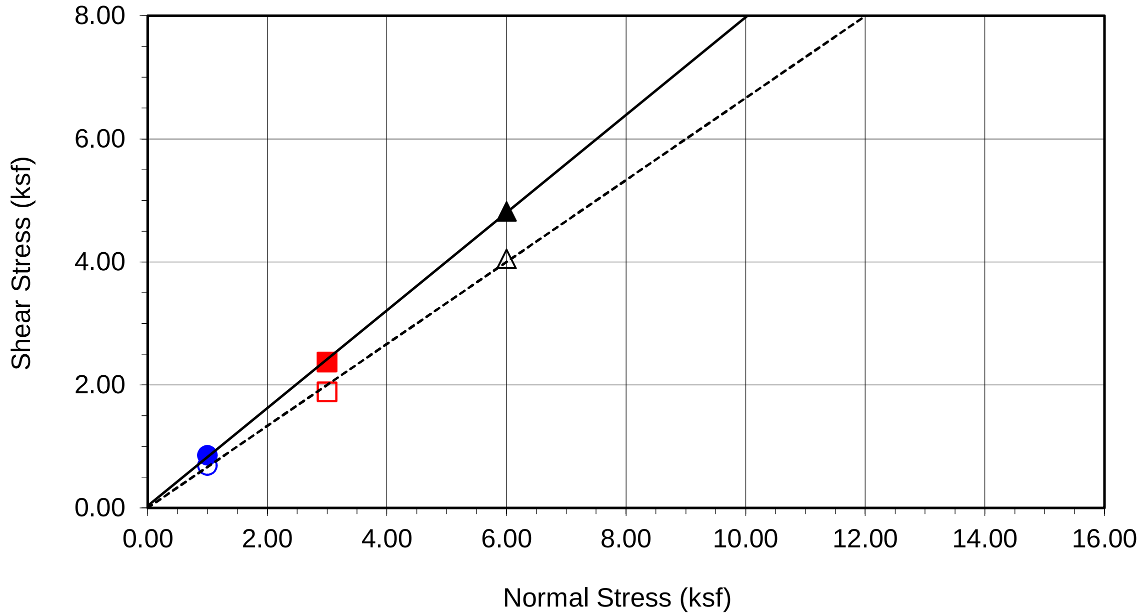
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11862.003

RCNA Anaheim

09-19





Boring No.	**LB-7**
Sample No.	**R1**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:
Light olive brown poorly-graded sand with silt (SP-SM)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	34	38
Ultimate	0	34

Normal Stress (kip/ft²)	1.000	3.000	6.000
Peak Shear Stress (kip/ft²)	● 0.855	■ 2.370	▲ 4.816
Shear Stress @ End of Test (ksf)	○ 0.692	□ 1.889	△ 4.049
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	2.18	2.18	2.18
Dry Density (pcf)	99.1	99.7	101.4
Saturation (%)	8.4	8.5	8.9
Soil Height Before Shearing (in.)	0.9938	0.9774	0.9731
Final Moisture Content (%)	20.1	18.3	18.7

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11862.003

RCNA Anaheim

09-19

DIRECT SHEAR TEST

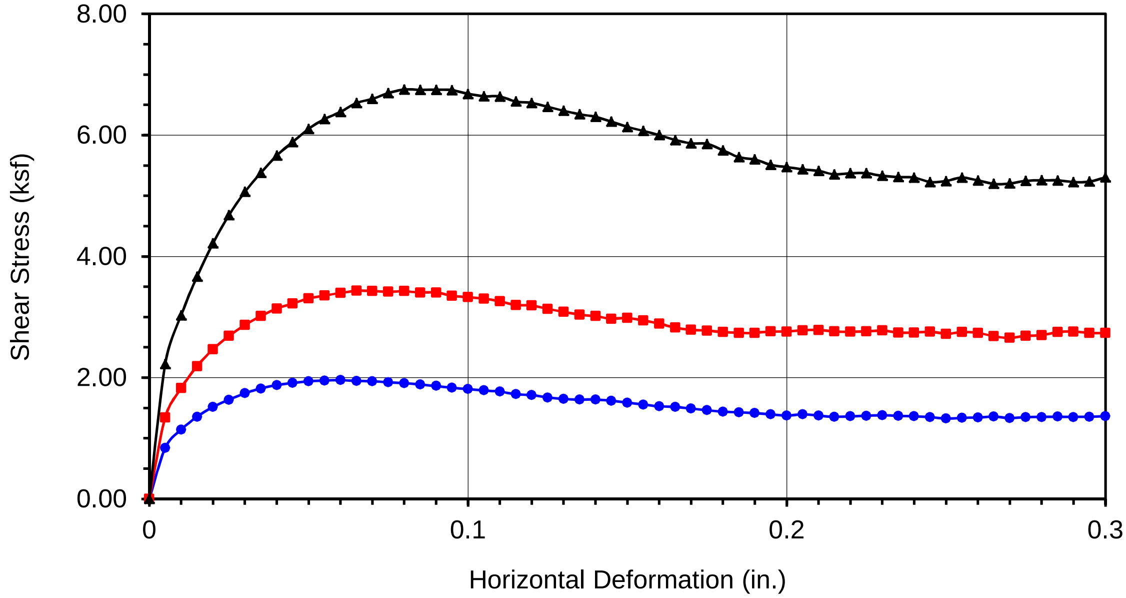
Consolidated Drained - ASTM D 3080

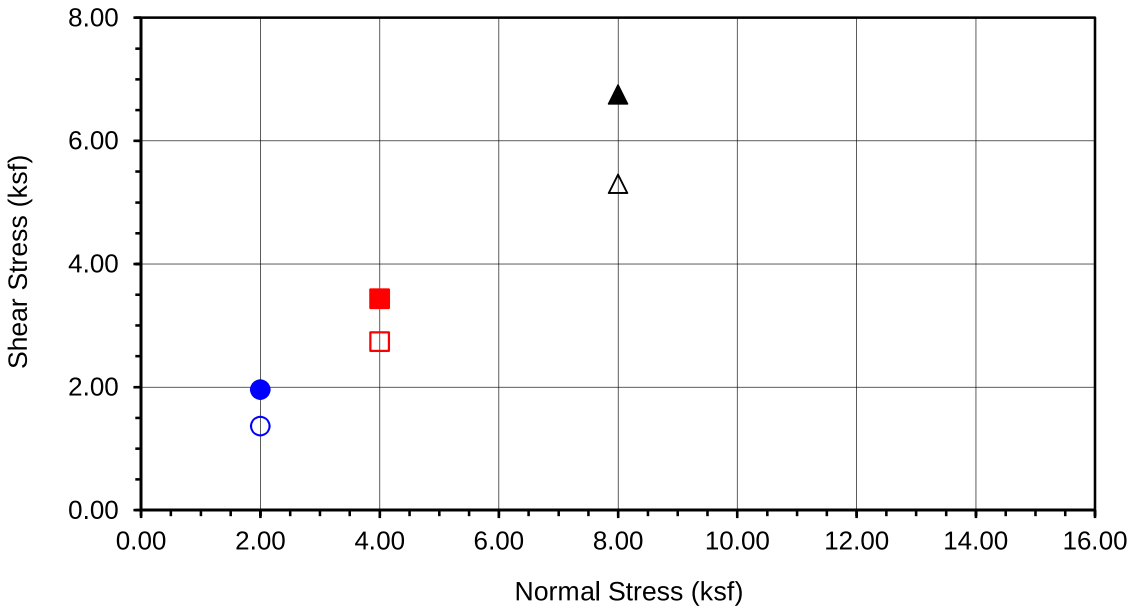
Leighton

Project Name:	RCNA Anaheim	Tested By:	G. Bathala	Date:	08/27/19
Project No.:	11862.003	Checked By:	J. Ward	Date:	09/17/19
Boring No.:	LB-7	Sample Type:	Ring		
Sample No.:	R3	Depth (ft.):	10.0		

Soil Identification: Light yellowish brown poorly-graded sand (SP)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	162.86	171.13	168.47
Weight of Ring(gm):	40.91	45.86	42.59
Before Shearing			
Weight of Wet Sample+Cont.(gm):	186.07	186.07	186.07
Weight of Dry Sample+Cont.(gm):	184.72	184.72	184.72
Weight of Container(gm):	77.39	77.39	77.39
Vertical Rdg.(in): Initial	0.0000	0.2488	0.2580
Vertical Rdg.(in): Final	-0.0121	0.2735	0.2954
After Shearing			
Weight of Wet Sample+Cont.(gm):	195.22	208.30	202.19
Weight of Dry Sample+Cont.(gm):	174.25	186.86	181.87
Weight of Container(gm):	59.07	69.52	61.51
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-7**
Sample No.	**R3**
Depth (ft)	**10**

Sample Type:

Ring

Soil Identification:
Light yellowish brown poorly-graded sand (SP)

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.959	■ 3.433	▲ 6.753
Shear Stress @ End of Test (ksf)	○ 1.364	□ 2.738	△ 5.304
Deformation Rate (in./min.)	0.0050	0.0050	0.0050
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	1.26	1.26	1.26
Dry Density (pcf)	100.2	102.9	103.4
Saturation (%)	5.0	5.3	5.4
Soil Height Before Shearing (in.)	0.9879	0.9753	0.9626
Final Moisture Content (%)	18.2	18.3	16.9

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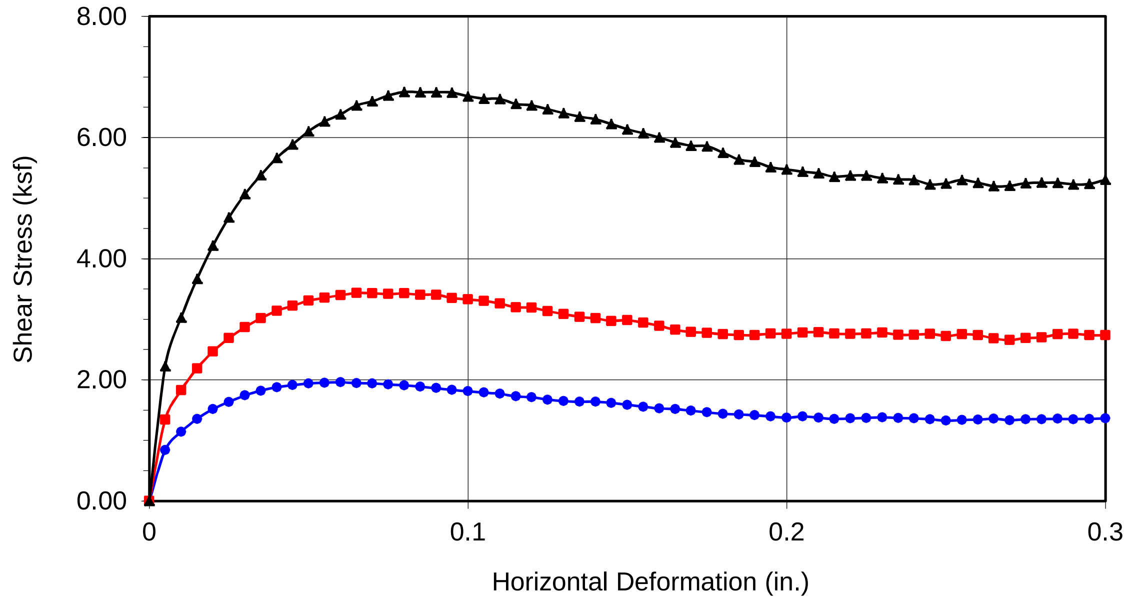
DIRECT SHEAR TEST RESULTS

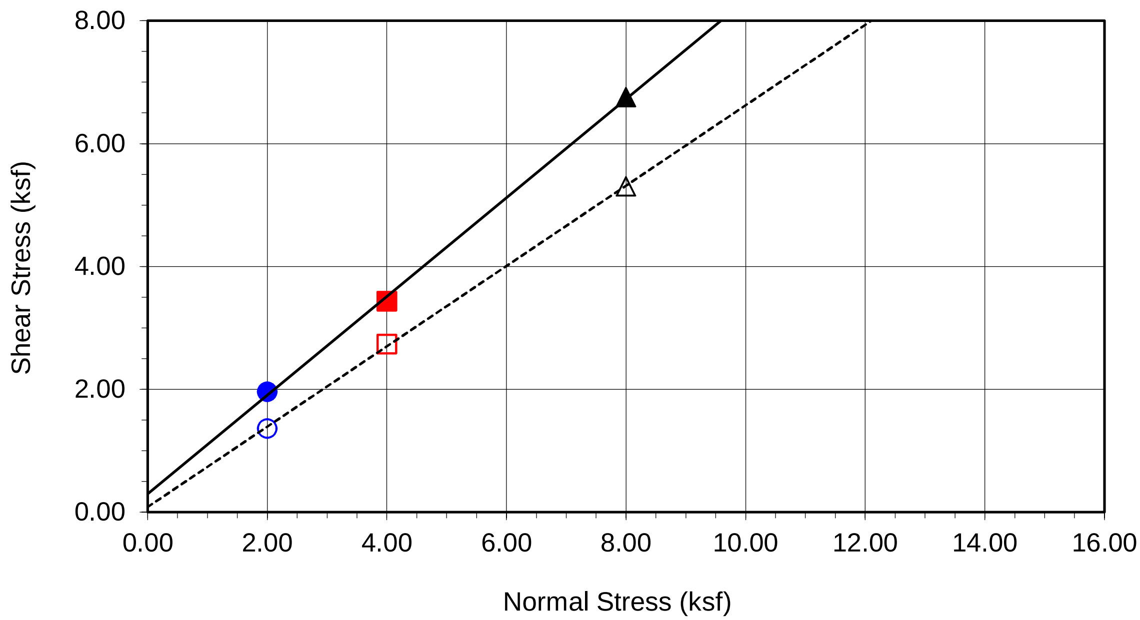
Consolidated Drained - ASTM D 3080

Project No.: 11862.003

RCNA Anaheim

08-19





Boring No.	**LB-7**
Sample No.	**R3**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:
Light yellowish brown poorly-graded sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	299	39
Ultimate	81	33

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.959	■ 3.433	▲ 6.753
Shear Stress @ End of Test (ksf)	○ 1.364	□ 2.738	△ 5.304
Deformation Rate (in./min.)	0.0050	0.0050	0.0050
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	1.26	1.26	1.26
Dry Density (pcf)	100.2	102.9	103.4
Saturation (%)	5.0	5.3	5.4
Soil Height Before Shearing (in.)	0.9879	0.9753	0.9626
Final Moisture Content (%)	18.2	18.3	16.9

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11862.003

RCNA Anaheim

08-19

DS LB-7, R3 @ 10

R-VALUE TEST RESULTS

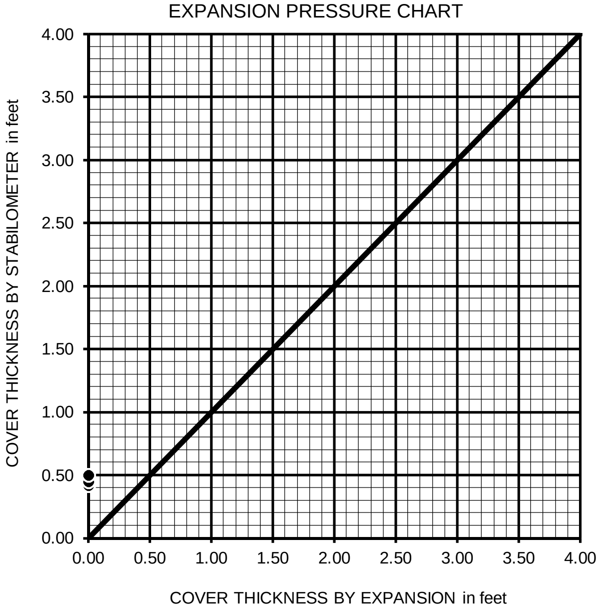
DOT CA Test 301

PROJECT NAME:	RCNA Anaheim	PROJECT NUMBER:	11862.003
BORING NUMBER:	LB-5	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B1	TECHNICIAN:	R. Manning
SAMPLE DESCRIPTION:	Brown poorly-graded sand with silt (SP-SM)	DATE COMPLETED:	9/11/2019

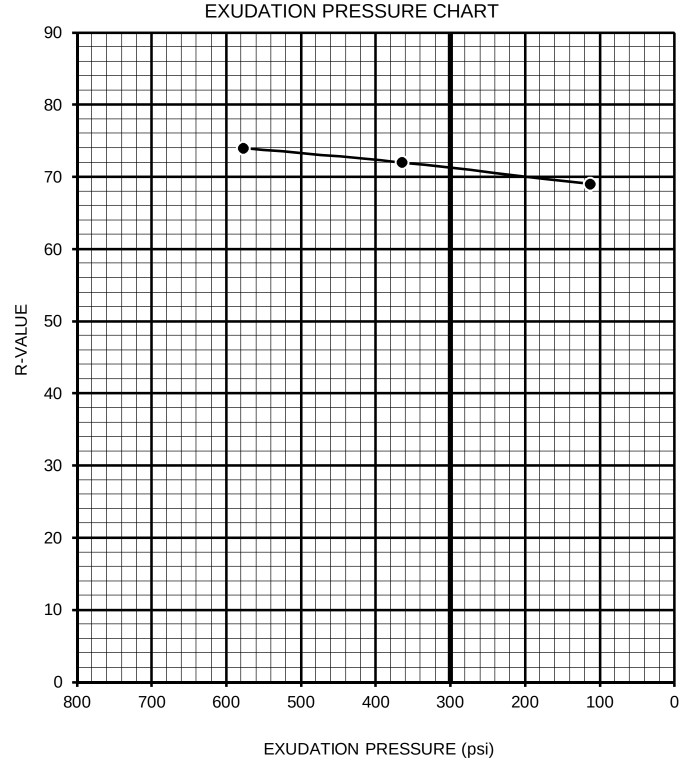
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	8.7	9.8	11.9
HEIGHT OF SAMPLE, Inches	2.52	2.53	2.54
DRY DENSITY, pcf	112.3	111.8	111.4
COMPACTOR PRESSURE, psi	200	175	150
EXUDATION PRESSURE, psi	577	365	113
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	22	24	27
TURNS DISPLACEMENT	5.43	5.43	5.63
R-VALUE UNCORRECTED	74	72	69
R-VALUE CORRECTED	74	72	69

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.42	0.45	0.50
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	71
EQUILIBRIUM R-VALUE:	71

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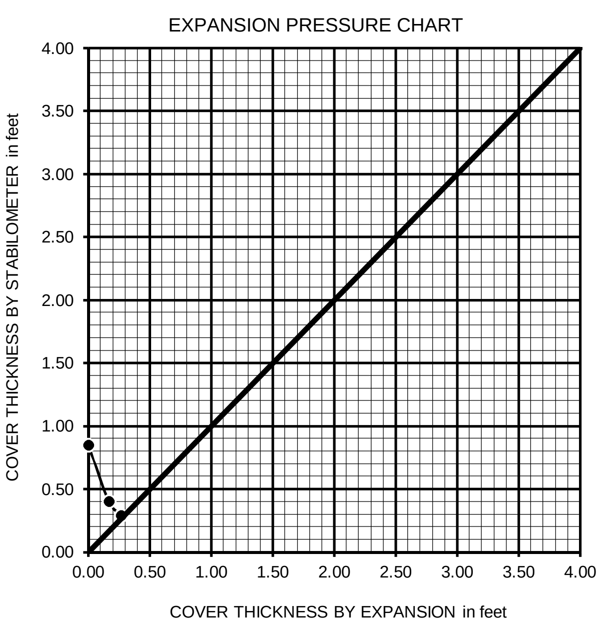
R-VALUE TEST RESULTS

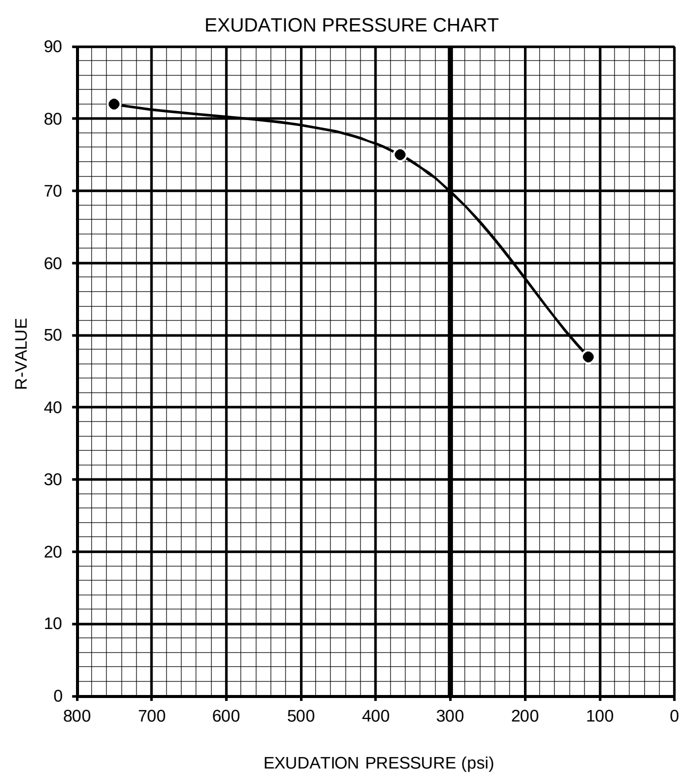
DOT CA Test 301

PROJECT NAME:	RCNA Anaheim	PROJECT NUMBER:	11862.003
BORING NUMBER:	LB-7	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B1	TECHNICIAN:	R. Manning
SAMPLE DESCRIPTION:	Light olive brown SP-SM	DATE COMPLETED:	9/11/2019

TEST SPECIMEN	**a**	**b**	**c**
MOISTURE AT COMPACTION %	10.7	12.0	14.1
HEIGHT OF SAMPLE, Inches	2.46	2.48	2.50
DRY DENSITY, pcf	119.3	118.4	117.4
COMPACTOR PRESSURE, psi	200	150	100
EXUDATION PRESSURE, psi	750	368	115
EXPANSION, Inches x 10exp-4	8	5	0
STABILITY Ph 2,000 lbs (160 psi)	19	22	62
TURNS DISPLACEMENT	3.98	5.17	4.52
R-VALUE UNCORRECTED	82	75	47
R-VALUE CORRECTED	82	75	47

DESIGN CALCULATION DATA	**a**	**b**	**c**
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.29	0.40	0.85
EXPANSION PRESSURE THICKNESS, ft.	0.27	0.17	0.00





R-VALUE BY EXPANSION:	83
R-VALUE BY EXUDATION:	70
EQUILIBRIUM R-VALUE:	70

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	RCNA Anaheim	Tested By :	G. Berdy	Date:	09/10/19
Project No. :	11862.003	Input By:	J. Ward	Date:	09/17/19

Boring No.	LB-5	LB-7		
Sample No.	B1	B1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Brown SP-SM	Light olive brown SP-SM		
Wet Weight of Soil + Container (g)	240.75	177.12		
Dry Weight of Soil + Container (g)	239.51	174.17		
Weight of Container (g)	39.89	51.37		
Moisture Content (%)	0.62	2.40		
Weight of Soaked Soil (g)	100.32	100.40		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	0	61		
Crucible No.	5	15		
Furnace Temperature (°C)	860	860		
Time In / Time Out	7:45/8:30	7:45/8:30		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	18.4891	25.5599		
Wt. of Crucible (g)	18.4872	25.5572		
Wt. of Residue (g) (A)	0.0019	0.0027		
PPM of Sulfate (A) x 41150	78.18	111.10		
PPM of Sulfate, Dry Weight Basis	**79**	**114**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.4	0.3		
PPM of Chloride (C -0.2) * 100 * 30 / B	40	20		
PPM of Chloride, Dry Wt. Basis	**40**	**20**		

pH TEST, DOT California Test 643

pH Value	**6.23**	**7.10**		
Temperature °C	20.2	20.3		

SOIL RESISTIVITY TEST

DOT CA TEST 643

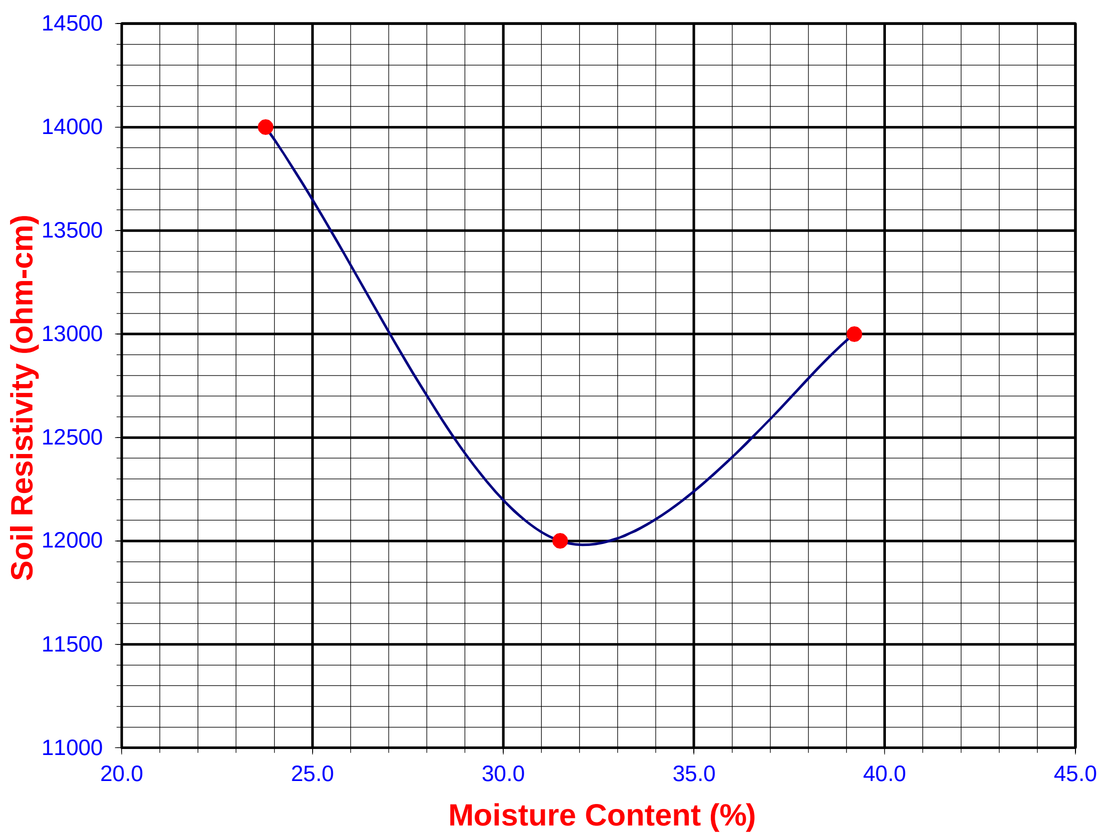
Project Name:	RCNA Anaheim	Tested By :	G. Berdy	Date:	09/12/19
Project No. :	11862.003	Input By:	J. Ward	Date:	09/17/19
Boring No.:	LB-5	Depth (ft.) :	0-5		
Sample No. :	B1				
Soil Identification:*	Brown SP-SM				

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	23.77	14000	14000
2	40	31.49	12000	12000
3	50	39.21	13000	13000
4				
5				

Moisture Content (%) (MCi)	0.62
Wet Wt. of Soil + Cont. (g)	240.75
Dry Wt. of Soil + Cont. (g)	239.51
Wt. of Container (g)	39.89
Container No.	
Initial Soil Wt. (g) (Wt)	130.38
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
11980	**32.1**	**79**	**40**	**6.23**	**20.2**



Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	RCNA Anaheim	Tested By :	G. Berdy	Date:	09/12/19
Project No. :	11862.003	Input By:	J. Ward	Date:	09/17/19
Boring No.:	LB-7	Depth (ft.) :	0-5		
Sample No. :	B1				

Soil Identification:* Light olive brown SP-SM

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	18.10	6400	6400
2	30	25.94	5250	5250
3	40	33.79	5600	5600
4				
5				

Moisture Content (%) (MCi)	2.40
Wet Wt. of Soil + Cont. (g)	177.12
Dry Wt. of Soil + Cont. (g)	174.17
Wt. of Container (g)	51.37
Container No.	
Initial Soil Wt. (g) (Wt)	130.50
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
5220	**27.0**	**114**	**20**	**7.10**	**20.3**