APPENDIX A.2: SOILS REPORT AREA D

GEOTECHNICAL EXPLORATION REPORT PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT A-TOWN PARCEL D CITY OF ANAHEIM, ORANGE COUNTY CALIFORNIA

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Project No. 12882.002

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Attention: Ms. Maria Korkosz

Subject: Geotechnical Exploration Report
Proposed Multi-Family Residential Development
A-Town Parcel D
Southwest Corner of East Katella Avenue and Metro Drive
City of Anaheim, Orange County, California

In accordance with our August 14, 2020 proposal, authorized on September 11, 2020, Leighton and Associates, Inc. (Leighton) has completed geotechnical exploration for the subject project. We understand from review of KTGY's *Conceptual Development Review Plan,* dated June 21, 2022, that Parcel D is proposed to consist of a 4-story building with 244 multi-family residential units over one level of subterranean parking in the eastern portion of the site and two levels of shared subterranean parking in the western portion of the site with Parcel C. In addition, we understand that drywells are being considered for the project for stormwater BMPs in the northern and southern portions of the site. Ancillary improvements are anticipated to consist of utility infrastructure, pool in the courtyard above the podium, flatwork, and landscaping.

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site, identify potential geologic and seismic hazards that may impact the project, and provide geotechnical recommendations for design and construction of the proposed project as currently planned.

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

We appreciate the opportunity to be of service to you on this project. If you have any questions or if we can be of further service, please contact us at ***(866) LEIGHTON***; or specifically at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

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TABLE OF CONTENTS

Section Page

1.0 INTRODUCTION ... 1
1.1 Site Description and Proposed Development ... 1
1.2 Purpose and Scope ... 2
1.3 Site Background and Previous Studies ... 5
2.0 GEOTECHNICAL FINDINGS ... 7
2.1 Regional Geologic Setting ... 7
2.2 Surficial Geology ... 7
2.3 Subsurface Conditions ... 7
2.3.1 Expansive Soil Characteristics ... 8
2.3.2 Soil Corrosivity ... 9
2.3.3 Soil Compressibility ... 9
2.3.4 Shear Strength ... 9
2.3.5 Shear Wave Velocity Profile ... 9
2.3.6 Excavation Characteristics ... 10
2.4 Groundwater Conditions ... 10
2.4.1 Infiltration ... 10
2.5 Surface Fault Rupture ... 11
2.6 Strong Ground Shaking ... 12
2.7 Liquefaction Potential ... 13
2.8 Seismically-Induced Settlement ... 14
2.9 Lateral Spreading ... 14
2.10 Earthquake-Induced Landsliding ... 14
2.11 Storm Induced Flooding ... 14
2.12 Earthquake-Induced Flooding ... 14
3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS ... 16
3.1 Site Grading ... 16
3.1.1 Site Preparation ... 16
3.1.2 Excavation Bottom Preparation ... 17
3.1.3 Fill Materials ... 17
3.1.4 Fill Placement and Compaction ... 17
3.1.5 Shrinkage ... 18
3.2 Shoring ... 18
3.2.1 Lateral Earth Pressures ... 18
3.2.2 Design of Soldier Piles ... 19
3.2.3 Lagging ... 19
3.2.4 Deflection ... 20

3.2.5 Monitoring 20

3.3 Foundation Design 20
3.4 Slabs-on-Grade 21
3.5 Sulfate Attack and Ferrous Corrosion Protection 22

3.5.1 Sulfate Exposure 22
3.5.2 Ferrous Corrosivity 23
3.5.3 Corrosivity Test Results 23

3.6 Retaining Walls 24

3.6.1 Sliding and Overturning 25
3.6.2 Drainage 25

3.7 Pavements 26

3.7.1 Asphalt Concrete 26
3.7.2 Portland Cement Concrete Paving 27
3.7.3 Paving Materials 27

3.8 Infiltration BMP Design Considerations 28
3.9 Temporary Excavations 29
3.10 Trench Backfill 29
3.11 Drainage and Landscaping 30
3.12 Additional Geotechnical Services 30

4.0 LIMITATIONS 31

REFERENCES 32

<u>Tables</u>

Table 1 – Measured (Unfactored) Infiltration Rate 11
Table 2 – 2019 CBC Based Ground Motion Parameters (Mapped Values) 13
Table 3 – Sulfate Concentration and Exposure 22
Table 4 – Soil Resistivity and Soil Corrosivity 23
Table 5 – Results of Corrosivity Testing 24
Table 6 – Retaining Wall Design Earth Pressures 25
Table 7 – Asphalt Concrete Pavement Sections 26
Table 8 – PCC Pavement Sections 27

Attachments

Important Information About Your Geotechnical Engineering Report Rear of Text

Figure 1 – Site Location Map Rear of Text
Figure 2 – Regional Geology Map Rear of Text
Figure 3 – Regional Fault Map and Historical Seismicity Map Rear of Text
Figure 4 – Seismic Hazard Map Rear of Text
Figure 5 – Flood Hazard Zone Map Rear of Text
Figure 6 – Dam Inundation Map Rear of Text
Figure 7 – Retaining Wall Backfill and Subdrain Detail (EI≤50) Rear of Text
Plate 1 – Exploration Location Map Rear of Text
Plate 2 – Geotechnical Cross Section A-A' Rear of Text

Appendix A – Exploration Logs
Appendix B – Percolation Test Data
Appendix C – Laboratory Test Results
Appendix D – Earthwork and Grading Guide Specifications

1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The A-Town Parcel D project site is located at the southwest corner of East Katella Avenue and Metro Drive in the city of Anaheim, California. The site location (latitude 33.8028°, longitude -117.8914°) and immediate vicinity are shown on Figure 1, *Site Location Map.*

Site Description: Parcel D is a rectangular plot of land approximately 3.1 acres in size and bounded by East Katella Avenue to the north, Parcel C and Market Street to the west, Park Street to the south, and South Chris Lane, which is to be renamed Metro Drive, to the east. It is our understanding Parcel D was mass graded in 2013 per the *City of Anaheim Mass Grading and Erosion Control Plan for Tr. 17703* (12 sheets), prepared by Hunsaker and Associates Irvine, Inc., dated October 22, 2018.

Parcel D is currently vacant with overall pad grade at Elevation (El.) +153 feet to El. +149 feet mean sea level (msl). Two roughly rectangular basins occupy portions of the northeast and northwest quadrants of Parcel D, with bottoms at approximately El. +140 feet msl and El. +144 feet msl, respectively. Two triangular basins occupy portions of the southeast and southwest quadrants of the site, with bottom at approximately El. +149 feet msl. Descending slopes are inclined roughly at 2.5:1 (horizontal:vertical) or flatter to the toe. The *Preliminary Utility Plan*, Sheet C.3 (Hunsaker, 2020) was utilized as the base map for Plate 1, *Exploration Location Map*.

Aerial Imagery Review: Based on review of historical aerial photographs (NETR, 2022), the site was vacant undeveloped land that appears to have been used for agricultural purposes until at least 1963. Between approximately 1972 and 1980, four (4) commercial buildings were constructed at the site with paved surface parking. By approximately 2009, the buildings and site improvements, roadways and utility infrastructure were removed and the site was graded as a part of the then proposed A-Town Development concept. We understand per review of the compaction report prepared by Group Delta Consultants, Inc. (GDC, 2014), additional grading was performed at the site in 2013 that included placement of engineered fill in the central and southern portions of the overall A-Town development site, which included Parcel D. Between approximately 2014 and 2016, additional grading was performed to bring the site to roughly its current configuration by removing previously constructed streets associated with the former development concept.

Proposed Development of Parcel D: Based on discussions with you and review of the *Concept Development Review,* prepared by KTGY, Sheet A2.7, Leighton understands Parcel D is proposed to consist of a 4-story, 244 multi-family residential units over one level of subterranean parking in the eastern portion of the site and partial two levels of subterranean parking in the western portion of the site. Drywells are being considered for the project for stormwater BMPs in the northern and southern portions of the site. Ancillary improvements are anticipated to consist of utility infrastructure, pool in the courtyard above the podium, flatwork, and landscaping.

The current site layout (KTGY, May 2022) indicates the 4-story residential units will wrap the podium on all four sides. The *Paseo* separating Parcels C and D overlies 2 levels of subterranean parking common to Parcels C and the western half of Parcel D. See Plate 2, *Geotechnical Cross Section AA',* for the currently proposed design concept. The lowest finished floor of the basement parking level in the western half of the site, per KTGY Conceptual Development Review for Parcels C and D, is shown to be approximately 25 feet below current site adjacent grade, with the lowest finish floor elevation of the parking structure in Parcel D at El. +128 feet msl. The finished floor of the single-level subterranean parking in the eastern half of the site is shown to be approximately 10 feet below current site adjacent grade, with the finished floor at El. +144 feet msl. Structural load data was not available at the time this report was prepared. The magnitude of the structural load demand will affect the type of foundation system that is appropriate based upon subsurface conditions. Once structural plans and building loads are prepared they should be provided to the geotechnical engineer for review.

1.2 Purpose and Scope

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site relative to the proposed Parcel D development concept and provide geotechnical recommendations to aid in the design and construction for the project. In accordance with our August 14, 2020 proposal, authorized on September 11, 2020, our scope of work included the following:

- *Research* – We reviewed readily available and provided literature including in-house geotechnical reports, literature, aerial photographs, and maps relevant to the site. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Section 5.0 *References*.

- *Pre-Field Exploration Activities* – Reconnaissance of the site was performed by a certified engineering geologist 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 subsurface exploration (soil borings for Parcel C and D) was performed on June 17 and July 1, 2022, and included drilling, logging, and sampling of two (2) hollow-stem auger borings (designated LB-1 and LB-2) to depths of approximately 41½ feet below the existing ground surface (bgs). One (1) additional boring (designated LP-2) was drilled to an approximate depth of 40 feet bgs for subsequent percolation testing. Approximate location of these explorations are shown on Plate 1, *Exploration Location Map* and corresponding boring logs are presented in Appendix A, *Exploration Logs.*

  During drilling of the hollow-stem auger borings, bulk and drive samples were obtained from the borings for geotechnical laboratory testing. Driven ring samples were collected from the borings using a Modified California ring-lined sampler conducted in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPTs) were also performed within the borings in accordance with ASTM Test Method D 1586. 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. N-values provide a measure of relative density in granular (non-cohesive) soils and comparative consistency in cohesive soils. The number of blows per 6 inches of penetration was recorded on the boring logs included in Appendix A.

  The borings were logged in the field by an engineering geologist from our firm. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory for testing. After completion of drilling, the borings were backfilled to the ground surface with hydrated bentonite chips. Excess soil cuttings from the borings were spread onsite.

Cone Penetrometer Test (CPT) Exploration - In addition to the soil borings, two (2) Cone Penetrometer Test (CPT) soundings were advanced on September 25, 2020 along the southern and eastern margins of the site (designated CPT-2 and CPT-3) to an approximate depth of 70 feet bgs (Plate 1). Shear wave velocity measurements were taken at five-foot intervals to develop seismic design parameters. 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 CPTs provide 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. A graphical log of the interpreted soil conditions at the CPT sounding locations are included in Appendix A.

- *Percolation Testing* – Boring LP-2 (Plate 1) was converted to temporary percolation test wells upon completion of drilling and sampling. The test well consisted of 2-inch slotted (0.020-inch slots) casing in the lower 10 feet of the boring, and solid 2-inch PVC well casing from 10 feet above well bottom to ground surface. In-situ percolation testing was performed on July 1, 2022 in general accordance with the *Orange County Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013). The results of the percolation testing are presented in Appendix B, *Percolation Test Data*. Refer to the discussion of infiltration rate presented in Section 2.4.1, *Infiltration*.

- *Laboratory Testing* – Selected relatively undisturbed and bulk soil samples obtained from our current hollow-stem-auger borings were tested at our in-house Irvine (DSA LEA 063) geotechnical laboratory. This laboratory testing program was designed to evaluate physical geotechnical characteristics of site soils including corrosion potential. A description of geotechnical test procedures and results are presented in Appendix C, *Geotechnical Laboratory Testing*. Tests performed during this investigation include:

  - In-situ Moisture Content and Dry Density (ASTM D 2216 and ASTM D 2937);
  - Expansion Index (ASTM D 4829);
  - Maximum Dry Density (ASTM D 1557);

- Direct Shear (ASTM D 3080);
- Consolidation (ASTM D 2435);
- R-value; and
- Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

Results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A.

- *Engineering Analysis* – Data obtained from these borings and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and recommendations for proposed development.

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

1.3 Site Background and Previous Studies

Parcel D was originally planned to be developed as a part of the overall A-Town Metro Platinum Triangle development project consisting of over 44 acres in size. The initial development plan for the overall project site in 2004 included the construction of high-rise buildings up to 29 stories in height, podium type structures over 2 levels of subterranean parking, various 4- to 5-story mixed use and residential buildings with 1 to 2 levels of subterranean parking and associated streets and utility infrastructure for the project site. Preliminary geotechnical explorations were performed by Leighton and Group Delta Consultants, Inc. to support preliminary design of the proposed development at that time (Leighton, 2004, 2005a and 2005b; GDC, 2006). Copies of the relevant prior exploration logs performed at the site and immediate vicinity by Leighton and others, as available, are included in Appendix A.

Since the original A-Town development scheme included high-rise buildings and podium type structures with 1 to 2 subterranean levels, excavations for the subterranean levels were performed in some of the parcels, including Parcel B (El. +127 feet), Parcel C (El. +132 feet) and Parcel D (El. +137 feet). The excavations, roadway construction, and utility construction were completed in 2006-2007 and a report documenting the geotechnical observation and testing was prepared by GDC (2007). Testing of imported material derived from many sources, according to GDC (2014) generally contained less than 35 percent fines in the upper 7 feet of fill and between 20 to 50 percent below 7 feet. Prior footprints when compared to current dimensions and layout of Parcel D indicate variable thickness of fill material should be expected. The current concept is expected to remove all prior engineered fill placed in support of earlier concept development schemes.

After completion of the utilities and roadways in 2006-2007, the project was put on hold until approximately 2013. Imported fill was required to backfill the excavations performed in 2006-2007. Compacted fill exists below Parcel D in the southwestern corner of the parcel, which was placed under observation and testing by Group Delta Consultants (GDC, 2014) to a minimum of 90 percent relative compaction (ASTM D 1557). The material was characterized as light brown clayey sand with a maximum density of approximately 125 pounds per cubic foot (pcf). The approximate limits of grading and fill placement is shown on Plate 1.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The subject site is located in the Downey Plain, within the southeastern margin of the Los Angeles Basin, a large structural depression within the Peninsular Ranges geomorphic province of California. In general, the Downey Plain is bordered by the Coyote and Peralta Hills on the north, the Santa Ana Mountains and Tustin Plain to the east, the Pacific Ocean to the south and Los Angeles County to the west. Several broadly warped coastal mesas represent uplifted areas along the Newport-Inglewood structural zone. These mesas are separated by erosional gaps which were created by historic routes of the Santa Ana River.

The site lies near the lower reaches of the Santa Ana River. The surface distribution of Holocene sediments, as recorded in early editions of regional soil survey maps (Eckmann and others, 1919) suggests that the Santa Ana River has recently wandered back and forth across the Downey Plain from Alamitos Bay to Newport Bay. Historical accounts and documents further support the process of widespread sheet flooding being the dominant depositional process associated with the Santa Ana River prior to the construction of Prado Dam in 1941 (California Department of Water Resources, 1959).

2.2 Surficial Geology

The surficial deposits at the site and in the vicinity consist of Quaternary age, young alluvial fan and floodplain deposits (alluvium) deposited by the Santa Ana River and tributaries (Bedrossian and Roffers, 2010; Morton and Miller, 2006). Mapped geologic units in the vicinity of the project site is presented as Figure 2, *Regional Geology Map*. These unconsolidated alluvial sediments consist of generally flat-lying, non-marine deposits of sand and minor amount of silt (Sprotte et al., 1980; and Morton and Miller, 2006). These sandy deposits are geologically youthful (Holocene age or less than 11,000 years old) and are reported to be approximately 80 to 100 feet thick beneath the site (Sprotte et al., 1980; and Real, 1985). Beneath the Holocene-aged sediments are the older semi-consolidated deposits of Pleistocene-age (11,000 to 1.6 million years) generally marked by an eroded surface displaying well oxidized soils and an increase in relative density.

2.3 Subsurface Conditions

Based on interpretation of subsurface explorations, the site is underlain by undocumented artificial fill of variable thickness overlying Quaternary-age young alluvial fan deposits. An area of certified engineered fill also exists overlying the alluvial fan deposits in the southwest corner of Parcel D (GDC, 2014). A general description of the earth materials as encountered are described below:

Artificial Fill, undocumented (Afu): The existing near-surface undocumented artificial fill soils encountered in our exploratory borings range in thickness from approximately 3 to 7½ feet below existing grade across the project site. These soils are characterized as light brown to brown, slightly moist to moist, silty sand, clayey sand and silty clayey sand with varying rock and manmade fragments.

Certified Engineered Fill (Afe): Near-surface engineered artificial fill soils are understood to have been placed under the observation and testing of GDC in the southwest corner of Parcel D, as documented in their compaction report (GDC, 2014). Based on elevations provided in the GDC report for nuclear density tests in this area, fill materials are expected to be on the order of approximately 6½ feet below existing grade. These soils are characterized as light brown clayey sand. The approximate limits of engineered fill are shown on Plate 1, *Exploration Location Map*.

Quaternary Age Young Alluvial Fan Deposits (Qyf): The Quaternary age young alluvial fan deposits encountered beneath the fill materials in our exploratory borings generally consist of tan to brown to gray brown, poorly graded, slightly moist to moist, sand and silty sand with thin beds or laminations of silt and clay.

The stratigraphy of the subsurface soils encountered in each soil boring is presented on the boring logs (Appendix A). The interpreted subsurface conditions across the site are depicted on Plate 2, *Geologic Cross-Section A-A'.*

2.3.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and which 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.

Two (2) near-surface bulk soil sample obtained during our subsurface exploration (LP-2 and LB-1) were tested for expansion potential. The test results indicate an Expansion Index (EI) value of 1 (“very low” potential for expansion). The Expansion Index laboratory test result is included in Appendix C of this report.

Expansive soils will likely not impact the proposed construction. Variance in expansion potential of onsite soil is possible but not anticipated.

Therefore, additional testing may be performed upon completion of site grading and excavation to confirm the expansion potential presented in this report. For purposes of this report, and based upon visual characterization of alluvial materials at approximate foundation depth, very low expansion potential of site materials may be considered to support design.

2.3.2 Soil Corrosivity

Two (2) near-surface bulk soil samples (LB-1 and LP-2) obtained during our subsurface exploration were tested for corrosivity to assess corrosion potential to buried concrete. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix C of this report.

The test results indicate *Soluble Sulfate* concentrations of 230 and 177 parts per million (ppm), *Chloride* contents of 40 and 61 ppm, *pH* values of 7.88 and 8.30, and *Minimum Resistivity* values of 1,249 and 2,399 ohm-cm for LB-1 and LP-2, respectively.

The results of the resistivity tests indicate the underlying soil is moderately to severely corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate contents from the soil samples, concrete in contact with the soil is expected to have negligible exposure to sulfate attack (S0) per ACI 318 (ACI, 2014). The samples tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete (C1) due to the chloride content of the soil.

2.3.3 Soil Compressibility

Four (4) 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 exhibit a low compressibility potential. The results of testing are presented in Appendix C.

2.3.4 Shear Strength

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

2.3.5 Shear Wave Velocity Profile

Shear wave velocities were measured in CPT-2 and CPT-3, during prior exploration for the A-Town development (Leighton, 2020), and results are presented in Appendix A. Based on the average shear wave velocity of about 845 feet per second and 809 feet per second recorded at CPT-2 and CPT-3, respectively, the Seismic Site Class is classified as Site Class D.

2.3.6 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 native earth materials can generally be excavated using conventional excavation equipment in good operating condition.

2.4 Groundwater Conditions

Groundwater was encountered in our prior subsurface investigation in boring BH-5 (Plate 1) at a depth of 81.5 feet bgs corresponding to El. +63.5 feet msl. 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.

Based on these findings, groundwater is not expected to pose a constraint during or after construction. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture, should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff, or from stormwater infiltration.

2.4.1 Infiltration

Percolation testing was performed within temporary percolation wells installed within boring LP-2 to evaluate the infiltration characteristics of subsurface soils. The percolation tests were conducted in general accordance with the *Orange County Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013). Results of the percolation testing are presented in Appendix B, *Percolation Test Data*. The test locations and zones tested are shown on Plate 1, *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 boring to general depths anticipated for the invert of typical infiltration devices below the planned basement levels, approximately 30 to 40 feet below Parcel D subgrade corresponding to El. ±123 feet to El. ±113 feet msl.

The tests were performed using a constant-head method which records 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. Since the subsurface materials were generally favorable for percolation (sandy soils), a water source was used to deliver water to each well at a relatively constant rate while recording the water height in the well. The measured infiltration rate for each percolation test was calculated by dividing the total volume of water infiltrated 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 wells are presented in Appendix B. The test results are summarized below:

Table 1 – Measured (Unfactored) Infiltration Rate

Test Well Designation	Approximate Depth of Test Zone (feet bgs)	Approximate Elevation of Test Zone (feet msl)	Measured Infiltration Rate (inches per hour)
LP-2	30 to 40	+123 to +113	11.9
GDC 2015-P3	30 to 40	+122 to +112	45.4

The results of the percolation tests indicate favorable rates of infiltration at the specific locations and depths tested. The measured infiltration rates are the result of small-scale test performed at specific locations and depths. The actual infiltration rate over the area of the proposed infiltration device could vary significantly from the test locations. Therefore, care must be used in the selection of infiltration rate for use in design and the potential for variances in soil conditions that could significantly affect field performance. The infiltration rate will decline over time between maintenance cycles as the BMP surface becomes occluded and particulates accumulate in the infiltrative layer.

2.5 Surface Fault Rupture

Our review of available literature indicates that no known active faults have been mapped across the site, and the site is **not** located within a currently established *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, 2008). The closest active faults to the site with the potential for surface fault rupture are the Whittier-Elsinore fault and the Newport-Inglewood Fault Zone (NIFZ), located approximately 8.9 and 10.4 miles from the site, respectively. The San Andreas fault, which is the largest active fault in California, is approximately 40.5 miles northeast of the site on the north side of the San Gabriel Mountains. Major regional faults with surface expression in proximity to the site are shown on Figure 3, *Regional Fault and Historic Seismicity Map*.

2.6 Strong 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 (Figure 3). The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics.

Accordingly, design of the project should be performed 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 2019 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, potential effects relating to seismic shaking can be reduced.

The following code-based seismic parameters should be considered for design under the 2019 CBC:

Table 2 – 2019 CBC Based Ground Motion Parameters (Mapped Values)

Categorization Coefficient	Code-Based
Site Latitude	33.8028°
Site Longitude	-117.8914°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.402 g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.497 g
Short Period (0.2 sec) Site Coefficient, F_a	1.0
Long Period (1 sec) Site Coefficient, F_v	null[1]
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.402 g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	null[1]
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	0.935 g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	null[1]
Site-adjusted geometric mean Peak Ground Acceleration, PGA_M	0.651 g

[1]Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient C_S to be determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_s$ or Eq. 12.8-4 for $T > T_L$

2.7 Liquefaction Potential

The term liquefaction is generally referenced to loss of strength and stiffness in soils due to build-up of pore water pressure when subject to cyclic or monotonic loading. Both sandy and clayey soils are susceptible to loss of strength and stiffness. Because of the difference in strength characteristic and methods for evaluating strength loss potential for granular and clayey soils, the term liquefaction is used for granular soils while cyclic softening is used for fine-grained soils (i.e. clays and plastic silts).

In general, adverse effects of liquefaction or cyclic softening include excessive ground settlement, loss of bearing support for structural foundations, and seismically-induced lateral ground deformations such as lateral spreading. Depending upon the relative thickness of the liquefied strata with respect to overlying non-liquefiable soils, other potentially adverse effects such as ground oscillation and ground fissuring may occur.

As shown on the *Seismic Hazard Zones* map for the Anaheim Quadrangle (CGS, 1998), the project site is **not** located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 4, *Seismic*

Hazard Map). In addition, the current and historic depth to groundwater are both greater than 50 feet bgs. Based on these findings, liquefaction is not considered a hazard at the site.

2.8 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 our evaluation of the site soils, the total seismically-induced settlement is estimated to be less than ½ inch. The differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet.

2.9 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 the site is relatively flat and constrained laterally, earthquake-induced lateral spreading is not considered a hazard at the site.

2.10 Earthquake-Induced Landsliding

As shown on Figure 4, the site is **not** mapped within a seismically-induced landslide hazard zone identified by the State of California (CGS, 1998). In addition, due to project site being relatively flat, it is our opinion that the potential for seismically-induced landslide hazard at the site is negligible.

2.11 Storm Induced 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 an area with a 0.2 percent annual chance flood hazard. As shown on Figure 5, *Flood Hazard Zone Map*, the site **is** located within a 500-year flood hazard zone. Regionally, storm runoff flow is generally directed to the south.

2.12 Earthquake-Induced Flooding

Strong seismic ground motion can cause dams and levees to fail, resulting in damage to structures and properties located downstream. The site is located within the inundation zone of Prado Dam, located on the lower Santa Ana River, approximately 20 miles upstream from the site as indicated on Figure 6, *Dam Inundation Map*. The potential for earthquake induced flooding exists if Prado Dam were to fail during a large earthquake.

Prado Dam, owned and operated by the Los Angeles District of the Army Corps of Engineers (Army Corps), has been continuously improved and is regularly maintained by the Army Corps to maintain operational capacity. There is no evidence, reports or documentations that indicate the dam has a high potential for failure during an earthquake. 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 Corps and Department of Water Resources. Therefore, the potential for flooding or earthquake-induced flooding due to dam failure is considered less than significant.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

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

The planned excavation will remove unsuitable soil deposits and expose undisturbed natural soils. The planned building may be supported on shallow foundations (spread footings or mat foundations) established in undisturbed natural soils.

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, and other governing agencies.

Leighton should review the grading, shoring and foundation plans 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

Earthwork guide specifications are presented in Appendix D, *Earthwork and Grading Guide Specifications*. Earthwork for Parcel D is expected to include overexcavation and recompaction, shoring and slope cutting operations, basement wall backfill, and utility installation/paving.

3.1.1 <u>Site Preparation</u>

Prior to construction, the site should be cleared of any vegetation, trash, and/or debris within the area of proposed grading. Active utilities should be removed or rerouted to maintain their function. These materials should be removed from the site. The onsite soils may be used as compacted structural fill and should be free of organic material or construction debris.

For any improvements outside of proposed basement areas, we anticipate a preliminary overexcavation depth of 3 feet bgs under proposed improvements and under any new fill used to raise site grades. If some future distress to paving is deemed acceptable, the overexcavation depth under areas planned for paving and concrete flatwork outside of basement areas may be reduced to 18 inches and proof-rolled with heavy compaction equipment to identify soft spots requiring localized overexcavation.

3.1.2 Excavation Bottom Preparation

Excavation bottom-surfaces should be observed by Leighton prior to placement of any backfill or new construction. After excavations are completed, and prior to any fill placement or foundation construction, exposed surfaces should be observed by Leighton to identify are any unsuitable areas requiring remediation. Unsuitable areas will have to be overexcavated down to competent materials, scarified to a minimum depth of 6 inches, moisture-conditioned to or slightly above optimum moisture content, and recompacted or proof rolled to achieve a minimum 95 percent relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve).

3.1.3 Fill Materials

On-site soil that is free of construction debris, organics, or rock larger than 4 inches in largest dimension is suitable to be used as fill for support of structures. Onsite clayey soils (not anticipated) if encountered during site grading, should not be used within 2 feet of concrete slabs-on-grade. Any imported fill soil should be approved by the geotechnical engineer prior to import or use onsite. Import soils should be uncontaminated, granular in nature, free of organic material (loss on ignition less than 2 percent), have a very low expansion potential (with an Expansion Index less than 21) and have a low corrosion impact to the proposed improvements.

3.1.4 Fill Placement and Compaction

Onsite soils free of organics, debris and oversized material (greater-than 6-inches in largest dimension) are suitable for use as compacted structural fill. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton, and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill must be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 3 percent **above** optimum moisture content, and compacted to a minimum 95% relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve). Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction.

Fills placed on slopes steeper than 5:1 (horizontal:vertical) should be benched into dense soils. Benching should be of sufficient depth to remove all loose material. A minimum bench height of 3 feet into approved material should be maintained at all times.

3.1.5 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 of native alluvial soils and engineered fill 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 5 to 10 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.

3.2 Shoring

The planned shoring system for the site may consist of soldier piles and lagging. Soldier piles may consist of steel H-beams set in pre-drilled holes and backfilled with lean-mix concrete to the ground surface. The pre-drilling auger diameter should be smaller than the diagonal dimension of the H-beam. Since the depth of the excavation is anticipated to be on the order of approximately 24 feet below existing adjacent road grade in the western portion of the site, and 10 feet below existing adjacent road grade in the eastern portion of the site, tieback anchors and internal bracing are expected to be required for a portion of the planned shoring system. The potential for raveling and caving of sand layers may pose difficulties in the installation of the soldier piles. Accordingly, the shoring contractor should be prepared to use special techniques and measures, if necessary, to permit the proper installation of the soldier piles.

3.2.1 Lateral Earth Pressures

For design of cantilevered shoring, where the surface of the backfill is level, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot (pcf). In addition to the recommended earth pressure, the shoring should be designed to resist any applicable surcharge loads due to foundation, storage, traffic, or other anticipated loads.

In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to streets should be designed to resist a uniform lateral pressure

100 psf, acting as a result of an assumed 100 psf surcharge behind the shoring due to normal street traffic. The recommended lateral surcharge due to traffic also applies to permanent basement walls. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. We can determine lateral surcharge pressures for specific cases, such as construction crane, concrete trucks, and other heavy construction equipment adjacent to shoring, if requested.

3.2.2 Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers (OC), the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 psf at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and soils.

Concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads from the soldier pile to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the design load. For piles encased in concrete, the coefficient of friction between the soldier piles and the retained earth may be taken as 0.4. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth. In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 psf.

3.2.3 Lagging

Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth.

The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. For clear spans up to 8 feet, we recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 psf at the midline between soldier piles, and 0 psf at the soldier piles.

3.2.4 Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. The maximum allowable horizontal shoring deflection adjacent to existing buildings, as measured at the top of the excavation, is ½ inch. The maximum allowable horizontal shoring deflection, as measured at the top of the excavation, should be limited to 1 inch in other areas.

If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent structures and of any utilities in the adjacent streets. To reduce the deflection of the shoring, if desired, a greater active pressure could be used in the shoring design.

3.2.5 Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles or though installation of inclinometers. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system is finalized.

We recommend that the adjacent existing streets be surveyed for horizontal and vertical locations. Also, a careful pre-construction survey of existing cracks and offsets in the streets should be performed and recorded along with photographic records. A pre-construction benchmark survey establishing horizontal locations and vertical elevations for the adjacent buildings combined with documentation of existing cracks and offsets may be useful in responding to claims of distress and damage (if any).

3.3 Foundation Design

Conventional spread footings established in undisturbed natural soils or 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 5,000 pounds per square foot (psf) may be used for footings established in undisturbed natural soils below the planned basement level at El. +144 feet msl. An allowable soil bearing pressure of 6,000 pounds per square foot (psf) may be used for footings established in undisturbed natural soils below the planned deepest basement level at El. +128 feet msl. Footings should be embedded a minimum 24 inches below the lowest adjacent grade and have a minimum width of 18 inches for continuous footings and 24 inches for isolated footings.

The ultimate bearing capacity can be taken as 18,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.45 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 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 0.3 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.

3.4 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.

3.5 Sulfate Attack and Ferrous Corrosion Protection

3.5.1 Sulfate Exposure

Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2019 California Building Code (CBC) defers to the American Concrete Institute's (ACI's) ACI 318-14 for concrete durability requirements. Table 19.3.1.1 of ACI 318-14 lists "*Exposure categories and classes*," including sulfate exposure as follows:

Table 3 – Sulfate Concentration and Exposure

Water-Soluble Sulfate (SO_4) in soil (percentage by weight ppm)	ACI 318-14 Sulfate Class
0.00 - 0.10	S0 (negligible)
0.10 - 0.20	S1 (moderate*)
0.20 - 2.00	S2 (severe)
>2.00	S3 (very severe)

*or seawater

3.5.2 Ferrous Corrosivity

Many factors can modify corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "*Effects of Soil Characteristics on Corrosion*" (February 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as follows:

Table 4 – Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in modifying corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

3.5.3 Corrosivity Test Results

To evaluate corrosion potential of soils sampled from this site, we tested a bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below:

Table 5 – Results of Corrosivity Testing

Boring Number	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
LB-1	0-5	230	40	7.88	1,249
LP-2	0-5	177	61	8.30	2,399

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

These results are discussed as follows:

- **Sulfate Exposure**: Based on test results and Table 19.3.1.1 of ACI 318-14, in our opinion, the Water-Soluble Sulfate (SO_4) in soil (percentage by weight), sulfate exposure should be considered "**negligible**" with an **Exposure Class S0**.
- **Ferrous Corrosivity**: As shown above, minimum soil resistivity of **1,249 ohm-centimeters or less** was measured in our laboratory test. In our opinion, based on resistivity correlation presented in Table 4 Section 3.5.2, it appears for site soils that corrosion potential to buried steel may be characterized as "**severely corrosive**" at the site.

As standard design concepts, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials.

3.6 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 very low expansion potential, field and laboratory verification are recommended before use. Should site soil be considered or available for reuse behind basement retaining walls, it 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 7, *Retaining Wall Backfill and Subdrain Detail* are as follows:

Table 6 – 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

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 the seismic condition, the pressure should be distributed as an equivalent fluid pressure with the dynamic thrust should be applied at a height of 1/3 H above the base of the wall.

3.6.1 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.

3.6.2 Drainage

Adequate drainage may be provided by a subdrain system positioned behind earth retaining walls. 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), 2021 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), 2021 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.

3.7 Pavements

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 95 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.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. Landscape areas must be separated from pavements with concrete curbs and/or edge drains. Excessive over-irrigation will have an adverse impact on adjacent pavements. *Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from paving, will result in premature pavement failure.*

3.7.1 <u>Asphalt Concrete</u>

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 45, compacted to at least 95 percent as recommended, the minimum recommended paving thicknesses are presented in the following table. Results of R-value testing on a near surface samples of existing onsite soils indicates a value of 77 and 79.

Table 7 – Asphalt Concrete Pavement Sections

Design Traffic Index (TI)	Asphalt Concrete (inches)	Base Course (inches)
5	3	4
6	3	6
7	4	6

A minimum of 3-inches of asphalt is recommended due to hot weather oxidation and degradation common in southern California. Traffic Indexes (TIs) used in our pavement design are considered reasonable values for proposed auto parking lots, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Higher TIs should be used in heavy truck traffic areas or high-volume lanes.

3.7.2 Portland Cement Concrete Paving

For light axle loads and average daily truck traffic (ADT) less than (<) 500, fire lanes subject to outrigger loads, trash corral aprons, or other areas where point loads are possible, should be paved with Portland Cement Concrete (PCC) with a minimum thickness of 7-inches over properly compacted fill. However, for medium/heavy axle loads and an ADT of (≥) 500 or more over properly compacted fill subgrades, a minimum PCC thickness of 8-inches should be used, such as for loading docks, etc. All PCC pavements should have a minimum 28-day concrete compressive strength of 4,000 pounds-per-square-inch (psi) and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. PCC subgrades supporting axle loads are recommended to be compacted to 95 percent relative compaction.

Table 8 – PCC Pavement Sections

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

This 4-inch layer of Class 2 aggregate may be used beneath other areas of PCC pavement to improve performance. Additional details should be added to plans indicating pavement thickness transitions, pavement joint dowels, expansion joints and saw cut joints. Use of concrete cutoff or edge barriers should be considered at the perimeter of common parking or driveway areas when abutting either open (unfinished) or landscaped areas.

3.7.3 Paving Materials

Asphalt concrete, aggregate base and Portland Cement Concrete (PCC) should conform to *Caltrans Standard Specifications* (current Edition):

https://dot.ca.gov/-/media/dot-media/programs/design/documents/f00203402018stdspecsa11y.pdf

Recommended structural pavement materials should conform to the specified provisions in the Caltrans *Standard Specifications* including grading and quality requirements, shown below:

- **Asphalt Concrete (Hot Mixed Asphalt)** for pavement should be Type A and should conform to Section 39 of the *Standard Specifications*. Asphalt concrete specimens should be tested for surface abrasion in accordance with CT-360.
- **Class 2 Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*.
- **Portland Cement Concrete (PCC)** pavement should conform to Section 40 of the *Standard Specifications*. PCC pavement materials (pavement, structures, minor concrete) should conform to Section 90 of the *Standard Specifications*.

As an alternative, asphalt concrete can conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book), 2021 Edition. Crushed aggregate base or crushed miscellaneous base can conform to Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), 2021 Edition, respectively.

3.8 Infiltration BMP Design Considerations

The small-scale infiltration rates presented in Section 2.4.1 should be converted to a large-scale rate using a reduction factor. In addition, infiltration rates will 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 small-scale infiltration rate using a minimum factor of safety of 3 or more should be used to establish a more realistic infiltration rate for the service life of the system(s).

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 and public pavements as soil saturation could lead to a loss of soil support, settlement or collapse, and internal erosion (piping). 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, although not anticipated at this time, 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.

3.9 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. Near-surface onsite soils are to be considered Type B soils (granular cohesionless).

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 engineering geologist or geotechnical engineer of record should be maintained to facilitate construction while providing safe excavations.

3.10 Trench Backfill

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"), 2021 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 95 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.11 Drainage and Landscaping

Building walls below grade should be waterproofed or at least damp proofed, depending upon the degree of moisture protection desired. Surface drainage should be designed to direct water away from foundations and toward approved drainage devices or drywells. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

3.12 Additional Geotechnical Services

Leighton should review the grading, shoring and foundation plans 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:

- Installation of shoring (solider beams) and during testing of tieback anchors;
- 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

This geotechnical exploration does not address the potential for encountering hazardous soil at this site. In addition, this report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is, by necessity, incomplete. Please also refer GBA's *Important Information About Your Geotechnical Report* (included at the rear of the text), presenting additional information and limitations regarding geotechnical engineering studies and reports. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton and Associates, Inc. has the opportunity to observe subsurface conditions during grading and construction, to confirm that our data are representative for the site. Leighton and Associates, Inc. should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing at this time in Orange County. We do not make any warranty, either expressed or implied.

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will <u>not</u> likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do <u>not</u> rely on an executive summary. Do <u>not</u> read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

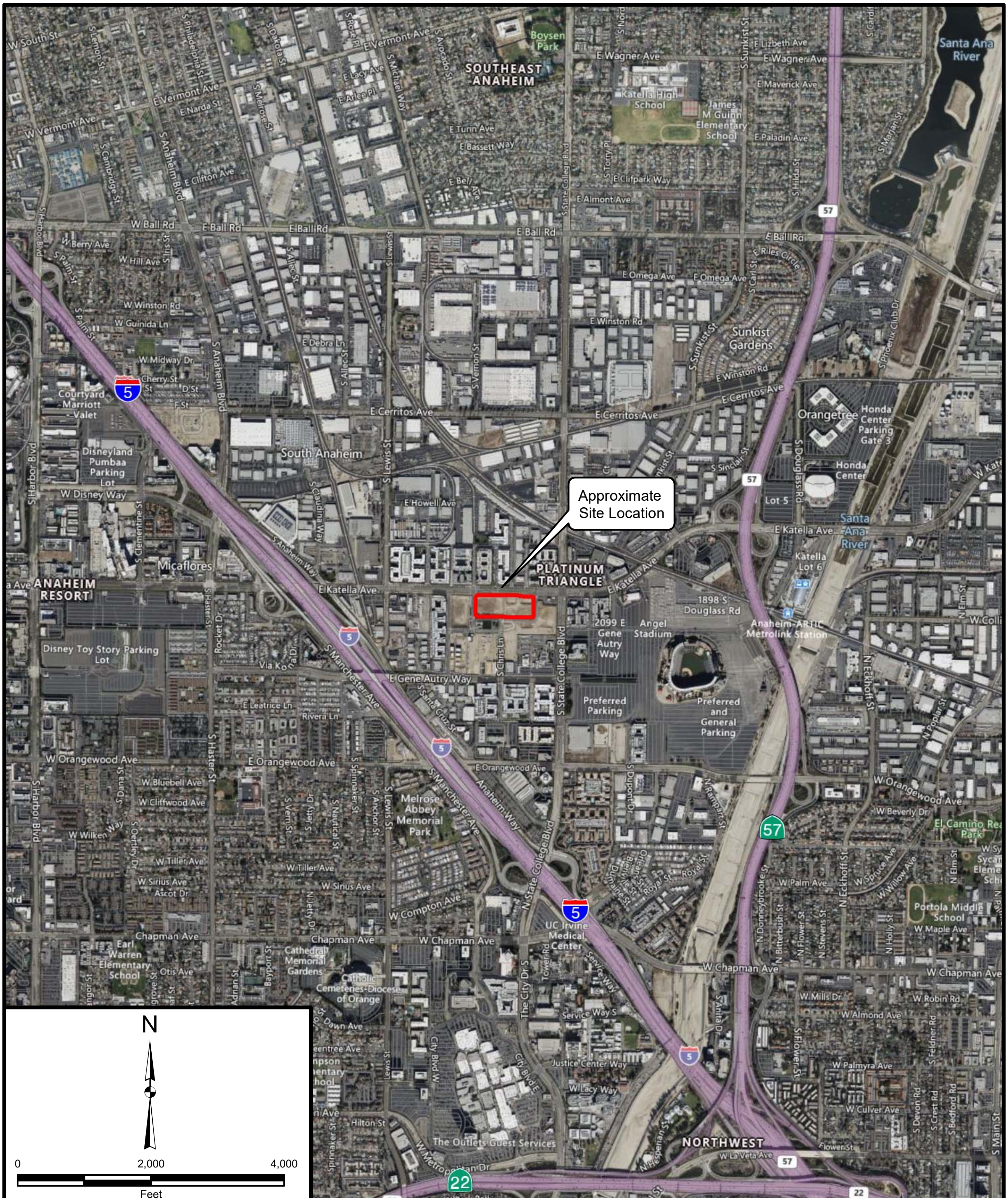
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 12882.002	Eng/Geol: CCK/JAR
Scale:1 " = 2,000 '	Date: June 2022

Reference: © 2022 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus

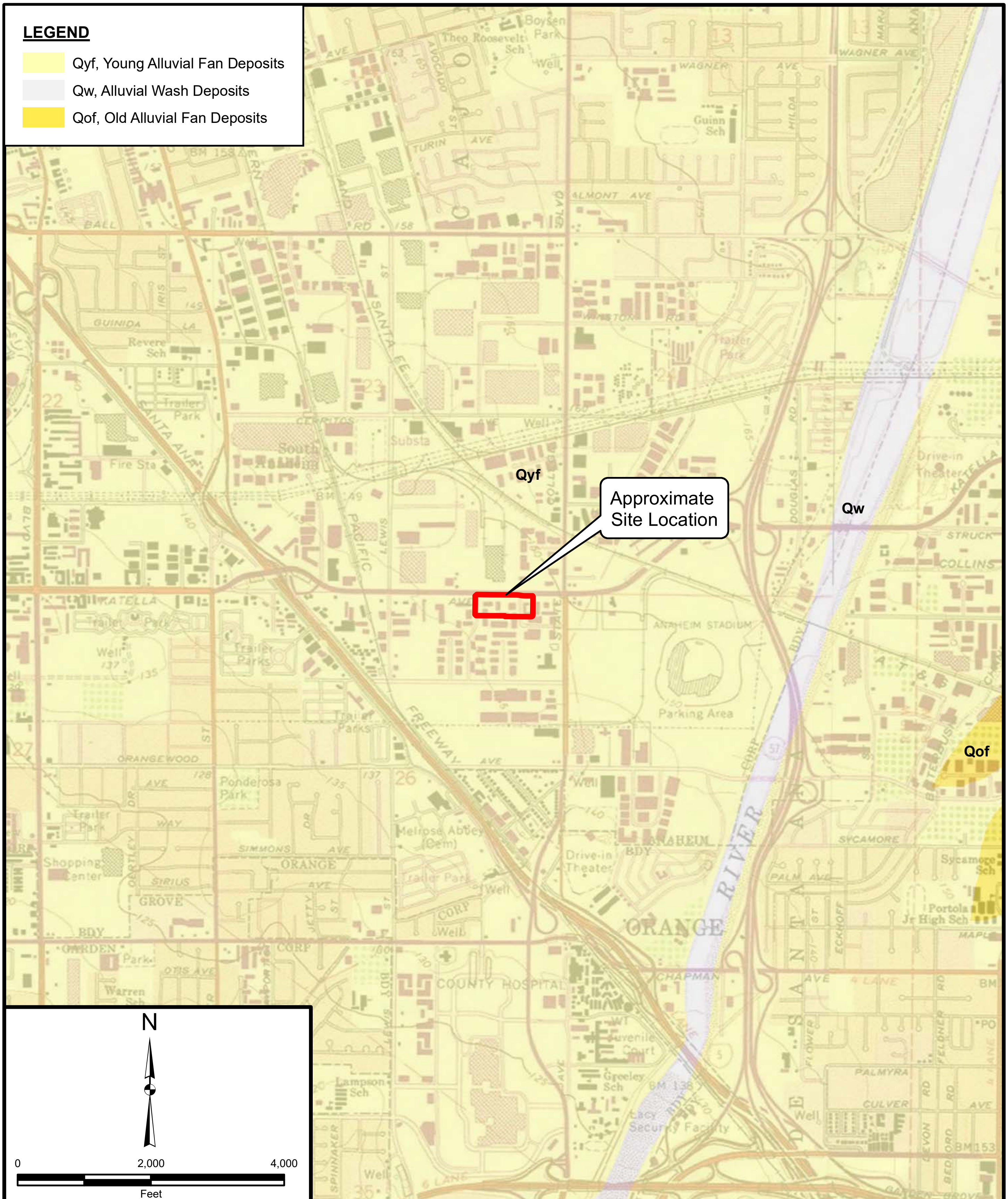
SITE LOCATION MAP

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California

FIGURE 1

Leighton

Map Saved as V:\Drafting\12882\002\Maps\12882-002_F01_SLM_2022-06-24.mxd on 6/23/2022 5:24:40 PM Author: KVM (btran)



Project: 12882.002	Eng/Geol: CCK/JAR	
Scale:1 " = 2,000 '	Date: June 2022	

Basemap: USGS Topo Map Service from Esri, 2021
Reference: USGS Geologic Map of the Sotuthern California GIS Compilation by CGS

REGIONAL GEOLOGY MAP

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California

FIGURE 2

Leighton

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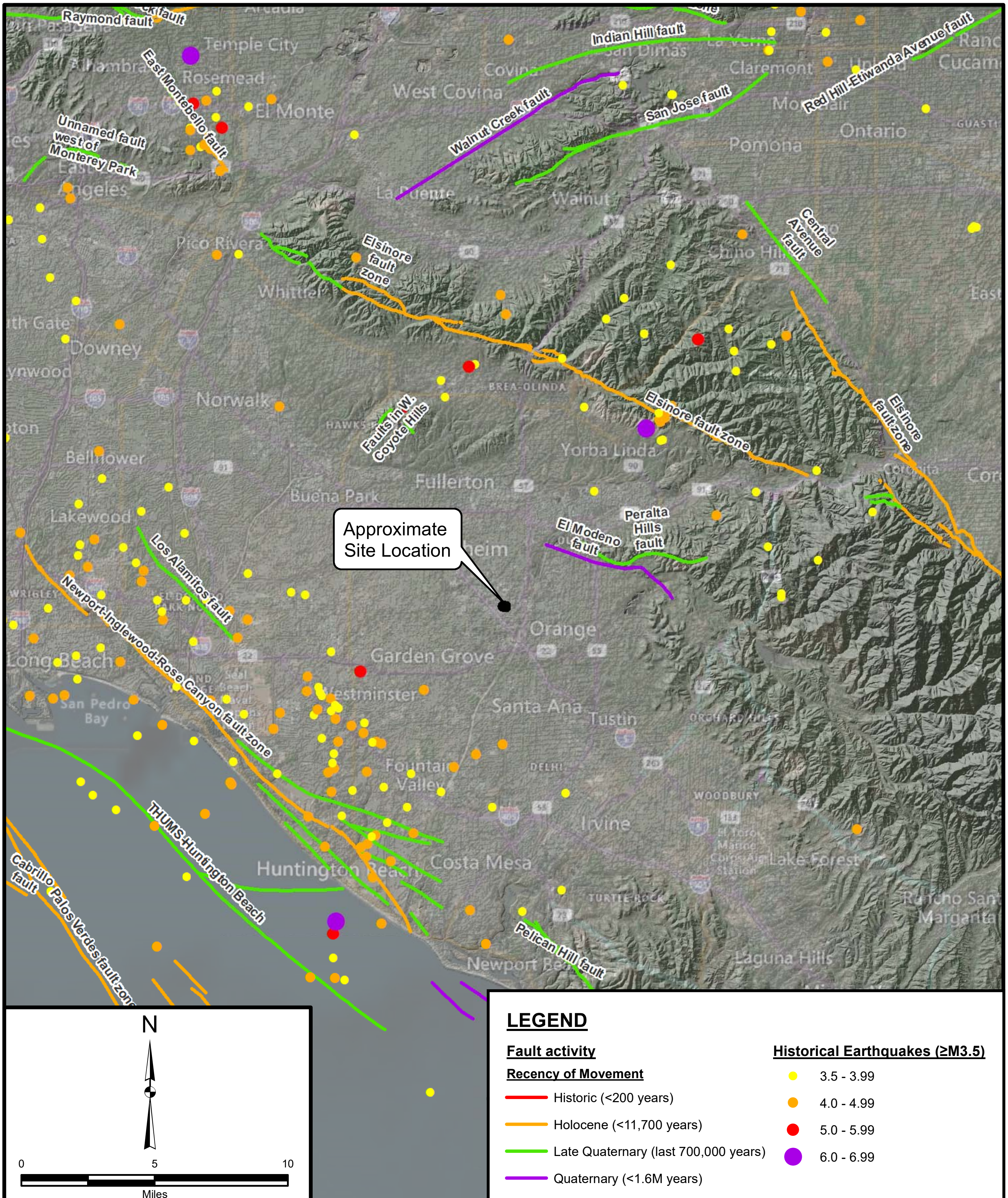

Project: 12882.002	Eng/Geol: CCK/JAR	**REGIONAL FAULT AND HISTORIC SEISMICITY MAP**	**FIGURE 3**
Scale: 1 " = 5 miles	Date: June 2022	Proposed Multi Family Residential Development A Town Parcel C and D Anaheim, California	Leighton

Basemap Reference: © 2022 Microsoft Corporation
Earthstar Geographics SIO © 2022 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

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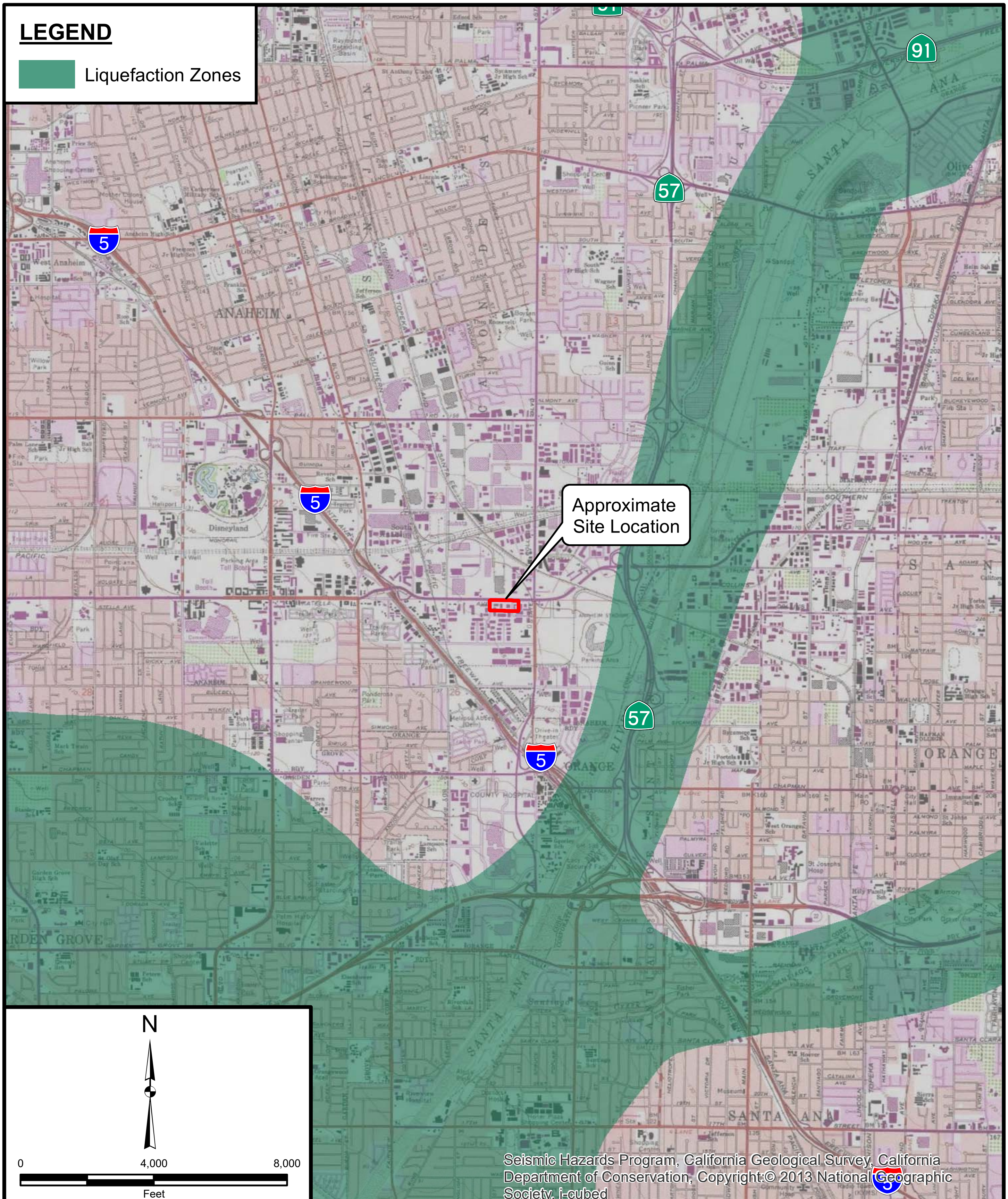


Project: 12882.002	Eng/Geol: CCK/JAR
Scale:1 " = 4,000 '	Date: June 2022

Base Map: San Bernardino County Land Use Plan, Geologic Hazard Overlays (FH29 C, Fontana)

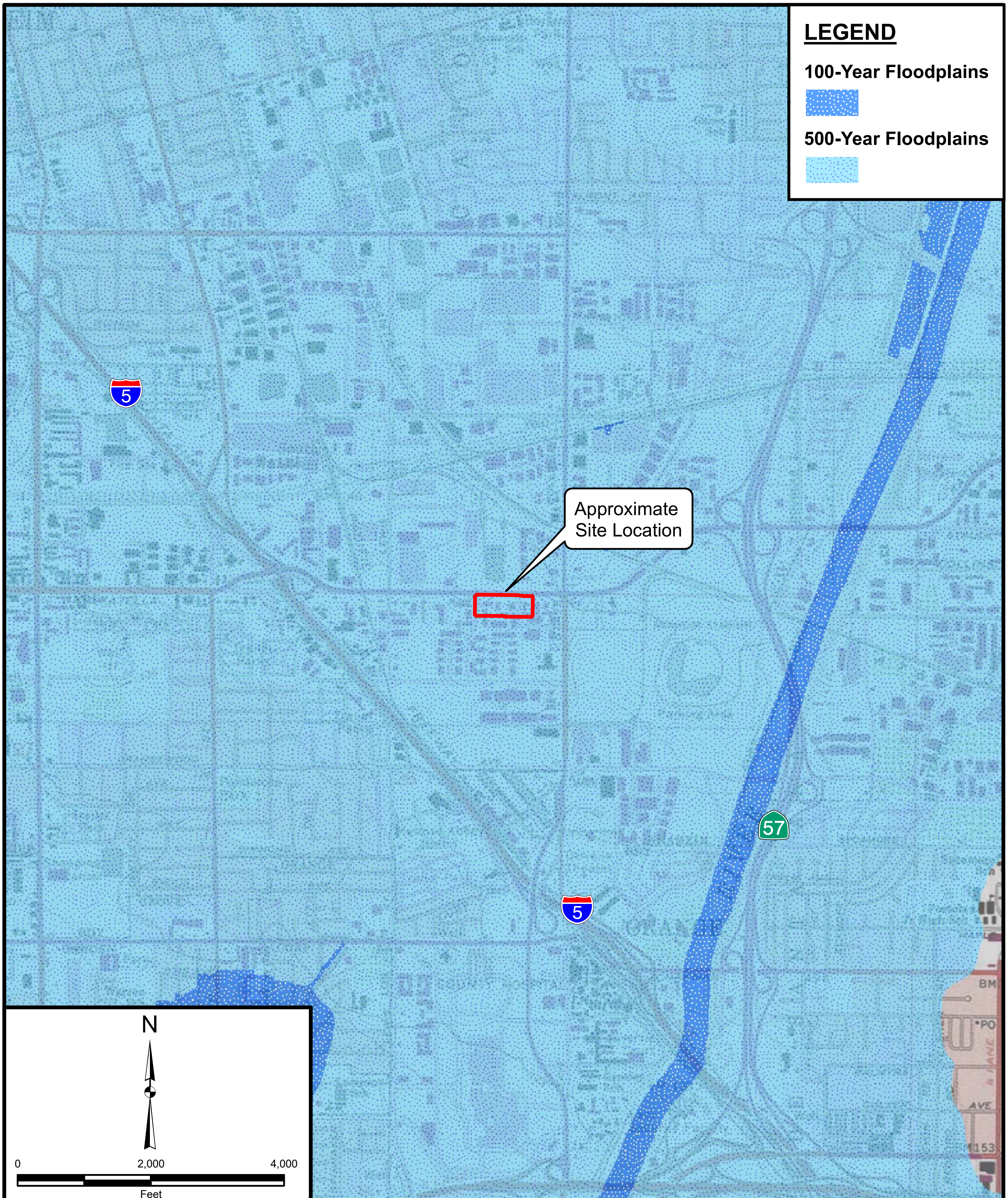
SEISMIC HAZARD MAP

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California

FIGURE 4

Leighton

Map Saved as V:\Drafting\12882\002\Maps\12882-002_F04_SHM_2022-06-24.mxd on 6/23/2022 5:28:25 PM Author: KVM (btran)



Project: 12882.002	Eng/Geol: CCK/JAR
Scale:1 " = 2,000 '	Date: June 2022

Reference: Copyright:© 2013 National Geographic Society, i-cubed
FEMA (http://www.fema.gov/index.shtm), DWR (http://www.dwr.ca.gov)

FLOOD HAZARD ZONE MAP

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California

FIGURE 5

Leighton

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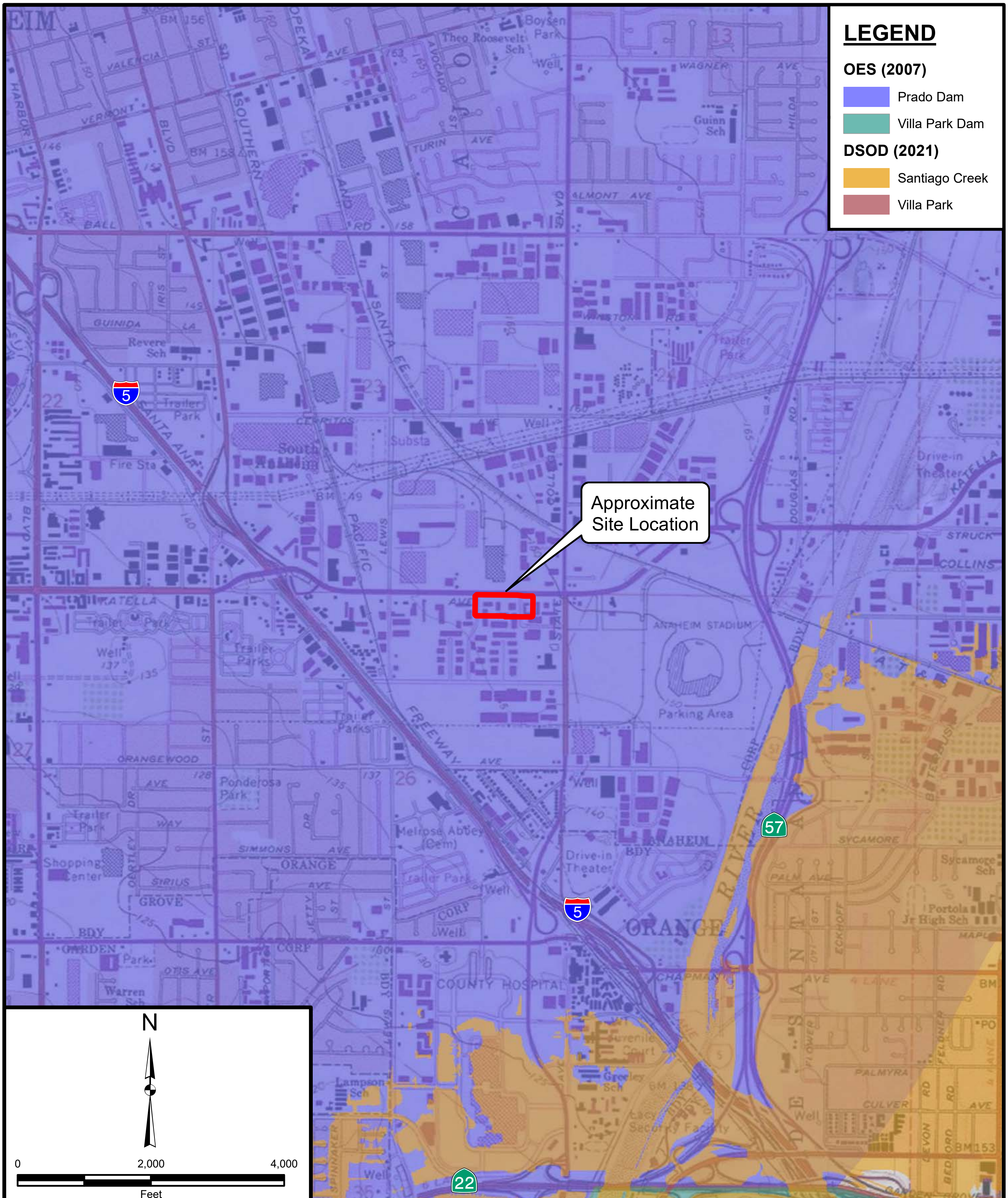

Project: 12882.002 | Eng/Geol: CCK/JAR
Scale: 1 " = 2,000 ' | Date: June 2022

Basemap Reference: Copyright:© 2013 National Geographic Society, i-cubed
Reference: Office of Emergency Services (2007),
Dept of Safety of Dams (2021)
National Inventory of Dams, Army Corps of Engrs (2021)

DAM INUNDATION MAP

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California

FIGURE 6

Leighton

Map Saved as V:\Drafting\12882\002\Maps\12882-002_F06_DIM_2022-06-24.mxd on 6/23/2022 5:27:43 PM Author: KVM (btran)

LEGEND

LB-2 — Approximate Location of Hollow Stem Auger boring shown with Total Depth (T.D.), depth to groundwater and Earth Units in Feet below ground surface (This Study).

LP-2 — Approximate Location of Percolation Test Boring shown with Total Depth (T.D.), depth to groundwater Earth Units in Feet below ground surface, and Infiltration Rate (I.R.) in inches per hour (This Study)

CPT-3 — Existing Cone Penetrometer Test (CPT) Leighton 2021

BH-6 — Approximate Location of Hollow Stem Auger Boring by Leighton, March 2005

GDC P3 — Approximate Locaiton of Infiltration Test Boring by Group Delta GDC, November 6, 2015

A — A' Geotechnical Cross Section

Approximate Limits of Certified Engineered Fill (GDC, 2014)

Approximate Site Boundary

EXPLORATION LOCATION MAP

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California.

Leighton

PLATE 1
Scale: 1 " = 30 feet
Date: July 2022
Proj: 12882.002
Eng/Geol: CCK/JAR

Base Map: Preliminary Utility Plan, Sheet C3, dated; 11/24/2020 by Hunsaker & Associates/A Lennar Company

GEOTECHNICAL CROSS SECTION A-A'

Proposed Multi Family Residential Development
A Town Parcel C and D
Anaheim, California

Leighton

PLATE 2	
Scale: 1"=30'	
Date: July 2022	
Proj: 12882.002	
Eng/Geol: CCK/JAR	

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APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	150'
Location	See Plate 1 - *Exploration Location Map*	**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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
150	0		B-1				SM-SC	**Artifical fill, undocumented: (Afu)** @0': Silty Clayey SAND (SM-SC), brown, moist, fine sand, some medium to coarse sand, some scattered gravels	CN, CR EI, MX, RV
145	5		R-1	2 3 4	106	5	SC	@5': Clayey SAND with Gravel (SC), brown, moist, loose, predominantly fine sand, some medium to coarse sand, and fine gravels @7': Light grinding/chiriping	
								Quaternary young alluvial fan deposits: (Qyf)	
140	10		S-1	2 3 4		2	SP	@10': SAND (SP), light tan, moist, loose, predominantly medium to coarse sand, few fine gravels, grading finer to predominantly fine sand, some medium to coarse sand, little fines	
135	15		R-2	8 11 15	103	2		@15': Tan, moist, medium dense, predominantly fine sand, some medium sand, massive	
130	20		S-2	7 12 14		3		@20': Light tan, slightly moist to moist, fine sand, massive	
125	25		R-3	8 23 33	107	5		@25': Light grey brown, moist, dense, predominantly very fine to fine sand, few medium to coarse sand, micaceous, thin bed of fine to medium sand @~26.4'	
120	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

Leighton

GEOTECHNICAL BORING LOG LB-1

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	150'
Location	See Plate 1 - *Exploration Location Map*	**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	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.	
120	30			S-3	4 6 8		14	SM	@30': Silty SAND (SM), brown, very moist, medium dense, very fine to fine sand, micaceous, few to some clay, grading coarser to SAND with Silt (SP-SM), little to no clay	
115	35			R-4	7 15 17	101	3	SP	@35': SAND (SP), light tan, slightly moist to moist, medium dense, predominantly fine to medium sand, some medium to coarse sand, few small clay blebs	
110	40			S-4	5 4 9		19	SM	@40': Silty SAND (SM), grey-brown, moist, medium dense, laminated, micaceous, grading to Sandy SILT (ML), then mottled brown and reddish brown SILT (ML), very moist, medium stiff, laminated very thin FeO veins @41.3': Sharp horizontal contact with SAND (SP), light tan, moist, medium dense, very fine sand, weakly laminated, FeO stained @Contact	
105	45								**Total Depth of Boring: 41.5 Feet bgs** **No groundwater encountered during drilling** **Boring backfilled with cuttings to ground surface upon completion of drilling on 6-14-2022.**	
100	50									
95	55									
90	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

Leighton

GEOTECHNICAL BORING LOG LB-2

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	153'
Location	See Plate 1 - *Exploration Location Map*	**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			B-1				SM	**Artifical fill, undocumented: (Afu)** @0': Silty SAND (SM), brown, slightly moist, predominantely fine sand, some medium to coarse sand	
150										
									@5': Silty SAND (SM), brown, slightly moist to moist, medium dense, fine sand	
	5			S-1	6 8 8		1	SP	**Quaternary young alluvial fan deposits: (Qyf)** @5.3': SAND (SP), tan, slightly moist, medium dense, fine to medium sand, few coarse sand, few fine gravels	
145										
	10			R-1	5 6 7	97	1	SPg	@10': Gravelly SAND (SPg), light tan, moist, medium dense, fine to coarse sand, fine subrounded gravels, few fines, grading finer by 11.25' to predominantly	
140										
	15			S-2	3 4 7		2	SP	@15': SAND (SP), tan, moist, medium dense, fine to coarse sand, little fines	
135										
	20			R-2	5 7 10	105	2	SPg	@20': SAND with Gravel (SPg), tan, moist, medium dense, predominantly fine to medium sand, with some coase sand and fine gravels, few thin lams of fine grey-brown Silty CLAY (ML-CL), one coarse gravel-sized ball of hard Sandy CLAY (CL), red brown, coated with coarse sand grains, fine matrix, MnO spots, trace charcoal in clay lams	
130										
	25			S-3	1 2 4		16	SC-SM SP	@25': Clayey Silty SAND (SC-SM), loose, very moist, very fine sand, FeO staned, grading with depth to very fine to fine light orange SAND (SP), erosional contact with massive brown Sandy CLAY/Clayey SAND (SC-CL), very moist, firm/loose @26.4: Sharp horizontal contact with tan fine SAND (SP)	
125										
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

Leighton

GEOTECHNICAL BORING LOG LB-2

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	153'
Location	See Plate 1 - *Exploration Location Map*	**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	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			R-3	15 25 41	103	2	SP	@30': SAND (SP), tan, slightly moist to moist, dense, fine sand, few medium sand, massive	
120										
	35			S-4	5 7 10		8	SC SP	@35': Clayey SAND (SC), brown, very moist, fine sand, grading to SAND (SP) by 35.5', tan, moist, medium dense, fine sand	
115										
	40			R-4	16 30 45	104	3	SP	@40': SAND (SP), tan, moist, very dense, very fine sand, few silt, few thin zones with medium to coarse sand, few thin Silty SAND (SM) zones	
110									**Total Depth of Boring: 41.5 Feet bgs** **No groundwater encountered during drilling** **Boring backfilled with cuttings to ground surface upon completion of drilling on 6-14-2022.**	
	45									
105										
	50									
100										
	55									
95										
	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

Leighton

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

GEOTECHNICAL BORING LOG LB-3

Project No.	12282.002	**Date Drilled**	7-1-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	MR Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	136'
Location	See Plate 1 - *Exploration Location Map*	**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
135	0			B-1				SM	**Artificial fill, undocumented: (Afu)** @0': Silty SAND, light brown, slightly moist, predominately fine sand, some medium to coarse sand and fine to medium gravels, some construction debris (AC, concrete)	
									Quaternary young alluvial fan deposits: (Qyf)	
130	5			R-1	4 9 8			SP	@5': SAND, tan, moist, medium dense, multiple gradational changes from predominately fine sand to fine to medium sand with some coarse sand and trace fine gravels, slightly micaceous, few fines	
125	10			S-1	5 6 6				@10': SAND, tan, moist, medium dense, very fine to fine sand, trace medium to coarse sand and fine gravel, little fines	
120	15			R-2	5 11 12				@15': SAND, tan, moist, medium dense, fine to medium sand with few to some coarse sand, trace fine gravel; grades finer to predominately fine sand with few medium to coarse sand, then very fine to fine sand with few to some silt, 1" bed of SILT at 15.5', brown, moist, stiff, very micaceous, few very fine to fine sand, massive	
115	20			S-2	7 7 7				@20': SAND, tan, moist, medium dense, fine sand, few medium sand, trace coarse sand, slightly micaceous	
110	25			R-3	6 9 12				@25': SAND, light tan, moist, medium dense, fine sand, some medium to coarse sand, little to no fines; grading finer to predominately very fine sand, few fine sand, micaceous, little fines	
	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

Leighton

GEOTECHNICAL BORING LOG LB-3

Project No.	12282.002	**Date Drilled**	7-1-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	MR Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	136'
Location	See Plate 1 - *Exploration Location Map*	**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
105	30			S-3	4 10 9			SP-ML	@30': Interbedded SILT with clay and SAND; SILT with clay, reddish brown, very moist, micaceous, laminated, iron oxidation stained SAND, light grey, moist, medium dense, very fine sand, micaceous, biotite-rich	
100	35			R-4	4 5 6			SP ML/CL	@35': SAND with silt, light brown, moist, loose, fine sand, grading finer to grey laminated Silty SAND, then to brown Sandy SILT, massive, iron oxidation spotting @35.5': Clayey Silty SAND, brown, moist, loose, very fine to fine sand, massive, micaceous, some iron oxidation spotting, trace calcium carbonate filled pinhole pores	
95	40			S-4	3 7 14			SM-SC	@40': Clayey SILT to Silty CLAY, mottled brown and reddish brown, moist, stiff, laminated, iron oxidation stains, irregularly shaped pockets of sand, light tan, fine sand with few medium sand, some sand from 41 to 41.5', SILT/CLAY possibly a rip-up clast or bank slough **Total Depth: 41.5' bgs** **No groundwater encountered during drilling** **Backfilled with soil cuttings to ground surface**	
90	45									
85	50									
80	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

Leighton

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GEOTECHNICAL BORING LOG LP-1

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	148'
Location	See Plate 1 - *Exploration Location Map*	**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			B-1				SM	**Artificial fill, undocumented: (Afu)** @0': Silty SAND (SM), brown, slightly moist to dry, very fine to fine sand, some medium to coarse sand, scattered fine to medium gravels @surface	
145								SP	**Quaternary young alluvial fan deposits: (Qyf)** @3': SAND (SP), tan	
	5			S-1	2 3 5		2		@5': Slightly moist, loose, fine sand, few medium sand, massive	
140										
	10			R-1	6 6 8				@10': Light tan, moist, medium dense, fine to coarse sand, few fine subrounded gravels, little fines	DS
135										
	15			S-2	3 3 3		2		@15': Loose, fine sand, few medium sand, trace to few coarse sand, massive, unconsolidated	
130										
	20			R-2	8 12 13	104	2		@20': Medium dense, gap-graded, predominantly fine sand, some scattered subrounded to rounded coarse sand, micaceous, few fines	
125										
	25			S-3	6 9 10		3		@25': Medium dense, predominantly fine sand, same medium to coarse sand, massive, unconsolidated, little fines	
120										
	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

GEOTECHNICAL BORING LOG LP-1

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	148'
Location	See Plate 1 - *Exploration Location Map*	**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			R-3	3 15 35			SM	@30': Silty SAND (SM), brown, very moist to wet, fine sand, few silt laminations, otherwise massive	CN, DS
								SP	@31.2': SAND (SP), light tan, moist, medium dense, fine sand, few medium sand	
115										
	35			S-4	4 8 9		2		@35': Tan, moist, medium dense, fine sand, some medium to coarse sand, micaceous, unconsolidated, massive	
110										
	40			R-4	5 13 14				@40': Interbedded SAND and SILT with Clayey SAND (SM-SC), light grey, moist, medium dense, very fine sand, very micaceous, laminated, heavily oxidized spots and laminated planes, SILT with Clay, mottled olive gray and reddish brown, very moist, stiff, laminated, oxidation spots abundant	CN, DS
105										
				S-5	3 5 6		13	SM-SC SM-ML	@45': Thickly interbedded SAND, Silty SAND, and Sandy SILT (SM-ML), grey brown, moist, medium dense, very fine sand, micaceous, silty sand to sandy silt is brown, very moist, medium dense/stiff, very fine sand, massive	
	45								**Total Depth of Boring: 45 Feet bgs** **No groundwater encountered during drilling** **Boring converted to Percolation Test Boring with 2-inch slotted PVC from 35' to 45 feet; Solid PVC Riser from 0 to 35 feet on 6-14-2022.** **Percolation Test performed on 7-01-2022.**	
100										
	50									
95										
	55									
90										
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

Leighton

GEOTECHNICAL BORING LOG LP-2

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	152'
Location	See Plate 1 - *Exploration Location Map*	**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			B-1				SM	**Artificial fill, undocumented: (Afu)** @0': Silty SAND (SM), brown, moist, predominantly fine sand, some medium to coarse sand, scattered fine to medium gravels	CN, CR EI, MX, RV
150										
								SP	**Quaternary young alluvial fan deposits: (Qyf)** @3': SAND (SP), light tan, slightly moist, fine to coarse sand, angular to subangular coarse sand grains	
	5			R-1	10 11 14	120	1		@5': Medium dense, fine to coarse sand, subangular to angular grains, little fines, trace fine subrounded gravel	
145										
	10			S-1	2 3 5		1		@10': Medium dense, slightly moist, fine to medium sand, some coarse sand, grading coarse with depth	
140										
	15			R-2	6 9 15				@15': Fining upward sequences, a few inches-thick, medium to coarse sand, with trace to few gravels @base, fining to predominantly fine sand, with few medium sand, moist, medium dense, weak FeO staining	DS
135										
	20			S-2	9 16 19		2		@20': SAND (SP), light tan, moist, medium dense, predominantly fine to medium sand, few coarse sand, faint micaceous lams	
130										
	25			R-3	7 9 16	110	7		@25': SAND with Silt and Clay (SM-SC), brown, moist to very moist, medium dense, predominantly fine to medium sand, some coarse sand @26.5': 1-inch of Sandy SILT with Clay (ML-CL), grey brown and reddish brown, very fine sand, laminated, FeO stained lams, few clay	
125										
	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

Leighton

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

GEOTECHNICAL BORING LOG LP-2

Project No.	12282.002	**Date Drilled**	6-14-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	152'
Location	See Plate 1 - *Exploration Location Map*	**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			S-3	6 9 18		3	SP	@30': SAND (SP), light tan, moist, medium dense, fine to medium sand, little fines @31.25': Clear contact with fine to very fine micaceous sand	
120										
	35			R-4	10 20 27	114	10	SM	@35': Silty SAND (SM), brown, very moist, dense, very fine to fine sand, some medium sand, trace coase sand, slightly micaceous, coarse biotite flakes, massive	
115										
				S-4	5 10 11		4	SP	@38.5': SAND (SP), light brown, moist, medium dense, predominantly fine sand, few medium sand, massive	
	40								**Total Depth of Boring: 40 Feet bgs** **No groundwater encountered during drilling** **Boring converted to Percolation Test Boring with 2-inch slotted PVC from 30' to 40 feet; Solid PVC Riser from 0 to 30 feet on 6-14-2022.** **Percolation Test performed on 7-01-2022.** **PVC Removed, boring backfilled with cuttings on 7-01-2022.**	
110										
	45									
105										
	50									
100										
	55									
95										
	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

Leighton

GEOTECHNICAL BORING LOG LP-3

Project No.	12282.002	**Date Drilled**	7-1-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	MR Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	151'
Location	See Plate 1 - *Exploration Location Map*	**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			B-1				SC/SM	@0': 3 -inches Aggregate Base **Arificial Fill, undocumented (Afu)** @0.25': Clayey Silty SAND, brown, moist, fine to medium sand, some coarse sand	
150										
	5			S-1	4 7 11			SC	@5': Clayey SAND, brown, moist, medium dense, predominately fine sand, few coarse sand, some AC fragments	
145								SP	**Quaternary young alluvial fan deposits: (Qyf)** @6.5': SAND, tan, moist, predominately fine sand, some medium sand, little fines	
	10			R-1	5 8 11				@10': SAND, tan, moist, medium dense, predominately fine to medium sand, some coarse sand, few fine subround to subangular gravels	
140										
	15			S-2	3 5 5				@15': SAND, tan, moist, medium dense, fewer coarse sand and fine gravel	
135										
	20			R-2	8 13 16				@20': SAND, tan, moist, medium dense, predominately fine sand, few medium to coarse sand, trace fine gravel, slightly micaceous	
130										
	25			S-3	4 7 9				@25': SAND, tan, moist, medium dense, very fine to fine sand, few silt, micaceous, massive	
125										
	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

Leighton

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

GEOTECHNICAL BORING LOG LP-3

Project No.	12282.002	**Date Drilled**	7-1-22
Project	A-Town Parcels C and D	**Logged By**	KMD
Drilling Co.	MR Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	151'
Location	See Plate 1 - *Exploration Location Map*	**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
120	30			R-3	8 12 12				@30': SAND, tan, moist, medium dense, fine to medium sand with some coarse sand, trace to few fine gravels, grades to fine sand, trace medium sand, massive, micaceous by 31'	
115	35			S-4	5 3 8			SM	@35': Silty SAND with clay, light brown, moist, medium dense, fine sand, few 1-2" beds of Sandy CLAY (@~35.25, 35.75), firm to stiff, plastic, some MnO spotting	
				R-4	8 14 16			SP	@38.5': SAND, light brown, moist, medium dense, very fine to fine sand, few silt, slightly micaceous	
110	40								**Total Depth of Boring: 40 Feet bgs** **No groundwater encountered during drilling** **Boring converted to Percolation Test Boring with 2-inch slotted PVC from 30' to 40 feet; Solid PVC Riser from 0 to 30 feet on 7-01-2022.** **Percolation Test performed on 7-01-2022.**	
105	45									
100	50									
95	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

Leighton

BORING RECORD

PROJECT NAME	PT metro
PROJECT NUMBER	IR 607A
HOLE ID	P-2
SITE LOCATION	1404 East Katella Avenue, Anaheim, CA
START	10/19/2015
FINISH	10/19/2015
SHEET NO.	1 of 2
DRILLING COMPANY	2R Drilling
DRILL RIG	CME 75
DRILLING METHOD	Hollow Stem Auger
LOGGED BY	E. Smith
CHECKED BY	M. DiNicola
HAMMER TYPE (WEIGHT/DROP)	Hammer: 140 lbs., Drop: 30 in.
HAMMER EFFICIENCY (ERi)	84.5%
BORING DIA. (in)	8
TOTAL DEPTH (ft)	28.5
GROUND ELEV (ft)	135
DEPTH/*ELEV.* GW (ft)	∇ NE / *NE* DURING DRILLING; ▼ NE / *NE* AFTER DRILLING
DRIVE SAMPLER TYPE(S) & SIZE (ID)	Bulk, SPT (1.4"), MC (2.4")
NOTES	N_{60} = 1.41 Nspt = 0.94 Nmc

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			B-1												Poorly-graded SAND (SP); brown; damp; mostly fine to medium SAND; trace fines; nonplastic.
			S-2	2 3 7	10	14									Medium dense.
5	130														
			R-3	9 16 20	36	34									Dense; light gray.
			S-4	5 6 7	13	18									Medium dense; brown.
10	125														
			R-5	11 18 25	43	40			4.9	104		PA			Dense; trace coarse SAND. 96% SAND; 4% fines
			S-6	7 11 12	23	32									
15	120														
			R-7	9 10 9	19	18									Medium dense.
			S-8	3 5 8	13	18			10.3			#200			SILTY SAND (SM); medium dense; brown; damp; mostly fine SAND; little fines; trace GRAVEL; nonplastic. 78% SAND; 21% fines; 1% GRAVEL
20	115														Poorly graded SAND (SP); medium dense; brown; moist; mostly fine to medium SAND; trace fines; nonplastic.
			R-9	11 17 24	41	39									Dense.
			S-10	6 9 11	20	28									Medium dense.

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FIGURE

a

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
PT metro	IR 607A	P-2

SITE LOCATION	START	FINISH	SHEET NO.
1404 East Katella Avenue, Anaheim, CA	10/19/2015	10/19/2015	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 75	Hollow Stem Auger	E. Smith	M. DiNicola

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	84.5%	8	28.5	135	⊻ NE / *NE* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk, SPT (1.4"), MC (2.4")	N_{60} = 1.41 Nspt = 0.94 Nmc	▼ NE / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															(Poorly graded SAND (SP), continued.) Dense.
			R-11	16 21 26	47	44									
			S-12	8 9 9	18	25			11.5			PA			SILTY SAND (SM); medium dense; olive brown; moist; mostly fine SAND; few medium SAND; some fines; nonplastic. 63% SAND; 37% fines
															Groundwater not encountered. Bottom of borehole at 28.5 ft. Boring was completed at the planned depth. Percolation test completed. Boring backfilled with cement grout tremmie.
30	105														This Boring Record was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
35	100														
40	95														
45	90														

BORING RECORD

PROJECT NAME: PT metro		PROJECT NUMBER: IR 607A		HOLE ID: **P-3**
SITE LOCATION: 1404 East Katella Avenue, Anaheim, CA		START: 10/19/2015	FINISH: 10/19/2015	SHEET NO.: 1 of 2
DRILLING COMPANY: 2R Drilling	DRILL RIG: CME 75	DRILLING METHOD: Hollow Stem Auger	LOGGED BY: E. Smith	CHECKED BY: M. DiNicola
HAMMER TYPE (WEIGHT/DROP): Hammer: 140 lbs., Drop: 30 in.	HAMMER EFFICIENCY (ERi): 84.5%	BORING DIA. (in): 8	TOTAL DEPTH (ft): 39	GROUND ELEV (ft): 145
DEPTH/*ELEV.* GW (ft): ⊻ NE / *NE* DURING DRILLING; ▼ NE / *NE* AFTER DRILLING				
DRIVE SAMPLER TYPE(S) & SIZE (ID): Bulk, SPT (1.4"), MC (2.4")	NOTES: N_{60} = 1.41 Nspt = 0.94 Nmc			

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	145														Poorly-graded SAND (SP); brown; moist; mostly medium SAND (subangular - subrounded); trace fines; nonplastic.
		Bulk	B-1												
5	140		R-2	7 13 12	25	24									Medium dense.
10	135		S-3	4 8 11	19	27									
			R-4	16 28 33	61	57									Very dense; trace GRAVEL.
15	130		S-5	5 14 17	31	44									Dense.
			R-6	7 11 21	32	30									Trace coarse SAND.
20	125		S-7	7 12 14	26	37									
			R-8	12 15 19	34	32									

GDC_LOG_BORING_2013 IR-607A.GPJ GDC2013.GDT 11/3/15

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
PT metro	IR 607A	P-3

SITE LOCATION	START	FINISH	SHEET NO.
1404 East Katella Avenue, Anaheim, CA	10/19/2015	10/19/2015	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
2R Drilling	CME 75	Hollow Stem Auger	E. Smith	M. DiNicola

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	84.5%	8	39	145	∇ NE / *NE* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk, SPT (1.4"), MC (2.4")	N_{60} = 1.41 Nspt = 0.94 Nmc	▼ NE / *NE*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		SPT	S-9	4, 7, 10	17	24									SILTY SAND (SM); medium dense; brown; moist; mostly medium SAND; little fines; nonplastic.
		MC	R-10	4, 9, 14	23	22			5.9	107		PA			Mostly fine to medium SAND. 84% SAND; 16% fines
30	115	SPT	S-11	9, 11, 14	25	35									Poorly graded SAND with SILT (SP-SM); dense; brown; moist; mostly fine to medium SAND; few fines; nonplastic.
		MC	R-12	6, 15, 23	38	36			3.2			#200			94% SAND; 6% fines
35	110	SPT	S-13	9, 13, 15	28	39									Mostly medium SAND.
		MC	R-14	6, 5, 8	13	12			35.8	85		PA			SILT (ML); stiff; brown; moist; mostly fines; few fine SAND; low plasticity. 91% fines; 9% SAND
40	105														Groundwater not encountered. Bottom of borehole at 39.0 ft. Boring was completed at the planned depth. Percolation test completed. Boring backfilled with cement grout tremmie. This Boring Record was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
45	100														

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FIGURE b

GEOTECHNICAL BORING LOG BH-3

Date	3-1-05	**Sheet**	1 of 3
Project	Platinum Triangle	**Project No.**	011331-011-
Drilling Co.	West HazMat	**Type of Rig**	CME-75
Hole Diameter	8"	**Drive Weight**	140
Drop	30"		
Elevation Top of Hole	148'	**Location**	Anaheim, California

Logged By JAR
Sampled By JAR

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								@ 0': 3" Asphalt over 4" Sandy Gravel base Fill	
									@ 7": Silty Sand (SM), fine to medium grained sand, trace fine gravel, asphalt debris, dark brown to dark blackish brown Alluvium (Qal)	
145									@ 3': Sand (SP), fine to coarse grained sand, trace fine gravel, moist, light yellowish brown	
	5			1	1 5 7	98.8	2.6	SP		MD,CR
									@ 7': Becoming light orange brown	
140				2			7.4	SP		
	10			3	3 7 8			SP	@ 10': Sand (SP), medium to coarse grained sand, fine gravel, moist, light yellowish brown	
135										
	15			4	5 6 11	103.9	3.7	SP	@ 15': Sand (SP), fine to coarse grained sand, moist, micaceous, some silt, light yellow brown	DS
130										
	20			5	7 8 11			SP	@ 21': Sand (SP), fine grained sand, moist, medium dense, light greyish white	
125										
	25			6	5 8 14	103.7	5.0	SP	@ 25': Silty Sand (SM), fine to medium grained sand, moist, medium dense, light brown to medium brown	DS
120										
	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
CU TRIAXIAL SHEAR
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-3

Date 3-1-05 **Sheet** 2 **of** 3
Project Platinum Triangle **Project No.** 011331-011-
Drilling Co. West HazMat **Type of Rig** CME-75
Hole Diameter 8" **Drive Weight** 140 **Drop** 30"
Elevation Top of Hole 148' **Location** Anaheim, California

Logged By JAR
Sampled By JAR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30			7	4 6 7			ML	@ 30': Sandy Silt (ML), very fine grained sand, moist, medium stiff, micaceous, brown	
115										
	35			8	2 7 13	100.6	7.5	SP	@ 35': Sand (SP), fine to medium grained sand, some silt, moist, medium dense, light yellowish brown	DS
110										
	40			9	6 11 11			SM	@ 40': Silty Sand (SM), fine grained sand, moist, medium dense, micaceous, light brown	
105										
	45			10	4 7 11	82.6	37.5	CL	@ 45': Silty Clay (CL), thinly interbedded fine to medium grained micaceous sand, wet, medium stiff, mottled olive brown to orange brown	DS,CN
100										
	50			11	8 12 15			SM	@ 50': Silty Sand (SM), fine to medium grained sand, interbedded silt and clayey silt, moist, medium dense, light olive brown	
95										
	55			12	3 12 23	101.5	2.6	SP	@ 55': Sand (SP), fine to coarse grained sand, fine gravel, moist, medium dense, light whitish grey	
90										
	60									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
CU TRIAXIAL SHEAR
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-3

Date 3-1-05 | Sheet 3 of 3
Project Platinum Triangle | Project No. 011331-011-
Drilling Co. West HazMat | Type of Rig CME-75
Hole Diameter 8" | Drive Weight 140 | Drop 30"
Elevation Top of Hole 148' | Location Anaheim, California

Logged By JAR
Sampled By JAR

Elevation Feet	Depth Feet	Graphic Log (N–S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	60			13	8 16 21			SP	@ 60': Sand (SP), fine to medium grained sand, some silt, moist, dense, micaceous, light greyish white	
85										
	65			14	5 20 28	105.8	5.2	SP	@ 65': Sand (SP), fine to medium grained sand, very moist, encountered perched groundwater, dense, micaceous, light whitish gray	
80										
	70			15	4 7 10			CL	@ 70': Clay (CL), some silt and very fine sand, very moist, medium stiff, medium plasticity, light orange brown	
75									Total depth: 71.5' Perched groundwater encountered at 65' below ground surface Boring backfilled with soil cuttings and patched with asphalt upon completion	
	75									
70										
	80									
65										
	85									
60										
	90									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
CU TRIAXIAL SHEAR
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-5

Date 4-19-05 | Sheet 1 of 5

Project Platinum Triangle | Project No. 011331-011-

Drilling Co. Martini Drilling | Type of Rig CME-75

Hole Diameter 8" | Drive Weight 140 | Drop 30"

Elevation Top of Hole 145' | Location Anaheim, California

Logged By JAR

Sampled By JAR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
145	0								@0': 3" Asphalt concrete over Gravelley Sand (SP) base @.7': Fill Sand (SP), fine to coarse grained sand and gravel, some silt and asphalt debris, moist, orange brown	
				1				SP		
140	5			2	3 4 5			SP	@5': Alluvium (Qal) Sand (SP), fine to coarse grained sand, trace silt, moist, loose, micaceous, orange grey	
135	10			3	3 9 15			SP	@10': Sand (SP), fine to coarse grained sand, moist, micaceous, medium dense, orange grey	
130	15			4	2 6 9			SP	@15': Sand (SP), fine to coarse grained sand, some silt, micaceous, moist, medium dense, light orange brown	
125	20			5	12 20 22			SP	@20': Sand (SP), fine to coarse grained sand, some silt, fine to coarse gravel, wet, dense, orange brown	
120	25			6	5 8 8			SP-SM	@25': Sand with Silt (SP/SM), fine to coarse sand, moist, medium dense, orange brown	
115	30									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- SH SHELBY TUBE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- CU TRIAXIAL SHEAR
- EI EXPANSION INDEX
- RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-5

Date	4-19-05	**Sheet**	2 of 5
Project	Platinum Triangle	**Project No.**	011331-011-
Drilling Co.	Martini Drilling	**Type of Rig**	CME-75
Hole Diameter	8"	**Drive Weight**	140
Drop	30"		
Elevation Top of Hole	145'	**Location**	Anaheim, California

Logged By JAR
Sampled By JAR

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
115	30		7	4 8 12			SM	@30': Silty Sand (SM), fine grained sand, very moist, medium dense, micaceous, orange brown	
110	35		8	4 6 8			SM	@35': Silty Sand (SM), fine grained sand, very moist, orange brown	
105	40		9	8 19 37			ML	@40': Sandy Silt (ML), fine grained sand, micaceous, very stiff, moist, orange brown	
100	45		10	3 5 8			ML	@45': Clayey Silt (ML), interbedded fine grained sand, micaceous, medium stiff, moist, orange brown	
95	50		11	9 16 15			SM	@50': Silty Sand (SM), fine grained sand, moist, micaceous, dense	
90	55		12	6 13 15			SP	@55': Sand (SP), fine grained sand, some silt, moist, dense, orange brown	
85	60								

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
CU TRIAXIAL SHEAR
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-5

Date 4-19-05 | Sheet 3 of 5
Project Platinum Triangle | Project No. 011331-011-
Drilling Co. Martini Drilling | Type of Rig CME-75
Hole Diameter 8" | Drive Weight 140 | Drop 30"
Elevation Top of Hole 145' | Location Anaheim, California

Logged By JAR
Sampled By JAR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
85	60			13	7 25 47			SM	@60': Silty Sand (SM), fine to coarse grained sand, trace of fine gravel, moist, very dense, orange brown	
80	65			14	12 25 20			SP	@65': Gravelley Sand (SP), fine to coarse grained sand, fine to coarse gravel, very dense, orange brown	
75	70			15	8 15 17			CL	@70': Silty Clay (CL), trace of fine slaty gravel, porous, stiff, moist, dark reddish brown to orange brown	
70	75			16	3 5 7			CL	@75': Silty Clay (CL), same as above	
									@77': encountered gravel	
65	80			17	20 50/6"			SP	@80': Gravelley Sand (SP), fine to coarse grained sand, some silt, fine to coarse slaty gravel, very dense, wet, reddish brown	
									@81.5': encountered groundwater	
									added bentonite mud to augers	
60	85			18	35 23 29			SP	@85': Gravelley Sand (SP), fine to coarse grained sand, fine to coarse gravel, wet, very dense, orange brown	
55	90									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
CU TRIAXIAL SHEAR
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-5

Date	4-19-05	**Sheet**	4 **of** 5
Project	Platinum Triangle	**Project No.**	011331-011-
Drilling Co.	Martini Drilling	**Type of Rig**	CME-75
Hole Diameter	8"	**Drive Weight**	140
Drop	30"		
Elevation Top of Hole	145'	**Location**	Anaheim, California

Logged By JAR
Sampled By JAR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
55	90			19	18 16 13			SP	@90': Gravelley Sand (SP), coarse grained sand, fine gravel, wet, dense, orange brown	
50	95			20	3 3 5			CL	@95': Silty Clay (CL), wet, loose, mottled orange brown	
45	100			21	9 23 25			CL	@100': Silty Clay (CL), some very fine grained sand, very stiff, wet, orange brown	
40	105			22	3 5 5			CL	@105': Silty Clay (CL), loose, moist, mottled orange brown	
35	110			23	3 6 14			CL	@110':" Silty Clay (CL), trace of fine grained sand, medium stiff, moist, light greyish brown	
30	115			24	6 23 29			SM	@116" Silty Sand (SM), fine to coarse grained sand, moist, dense, brown	
									@120': Silty Sand (SM), fine grained sand, wet, dense, light yellow brown	
25	120				18					

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- SH SHELBY TUBE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- CU TRIAXIAL SHEAR
- EI EXPANSION INDEX
- RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG BH-5

Date 4-19-05 **Sheet** 5 **of** 5
Project Platinum Triangle **Project No.** 011331-011-
Drilling Co. Martini Drilling **Type of Rig** CME-75
Hole Diameter 8" **Drive Weight** 140 **Drop** 30"
Elevation Top of Hole 145' **Location** Anaheim, California

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION Logged By JAR Sampled By JAR	Type of Tests
25	120			25	40 33			SM		
									Total depth: 121.5' Encountered groundwater @ 81.5' below ground surface Boring backfilled with soil cuttings and patched with asphalt upon completion	
20	125									
15	130									
10	135									
5	140									
0	145									
-5	150									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
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TYPE OF TESTS:
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LEIGHTON AND ASSOCIATES, INC.

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

E. Katella Avenue & Market Street
Anaheim, CA
September 28, 2020

Prepared for:

Mr. Jeff Pflueger
Leighton & Associates
17781 Cowan
Irvine, CA 92614-6009
Office (800) 253-4567 / Fax (949) 250-1114

Prepared by:

KTE

KEHOE TESTING & ENGINEERING

5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

1. INTRODUCTION
2. SUMMARY OF FIELD WORK
3. FIELD EQUIPMENT & PROCEDURES
4. CONE PENETRATION TEST DATA & INTERPRETATION

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Summary of Shear Wave Velocities
- CPT Data Files (sent via email)

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at E. Katella Avenue & Market Street in Anaheim, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 28, 2020. The scope of work was performed as directed by Leighton & Associates personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at five locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	71	
CPT-2	70	
CPT-3	70	
CPT-4	70	
CPT-5	70	

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At locations CPT-1, CPT-2, CPT-3, CPT-4 & CPT-5, shear wave measurements were obtained at various depths. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Steven P. Kehoe
President

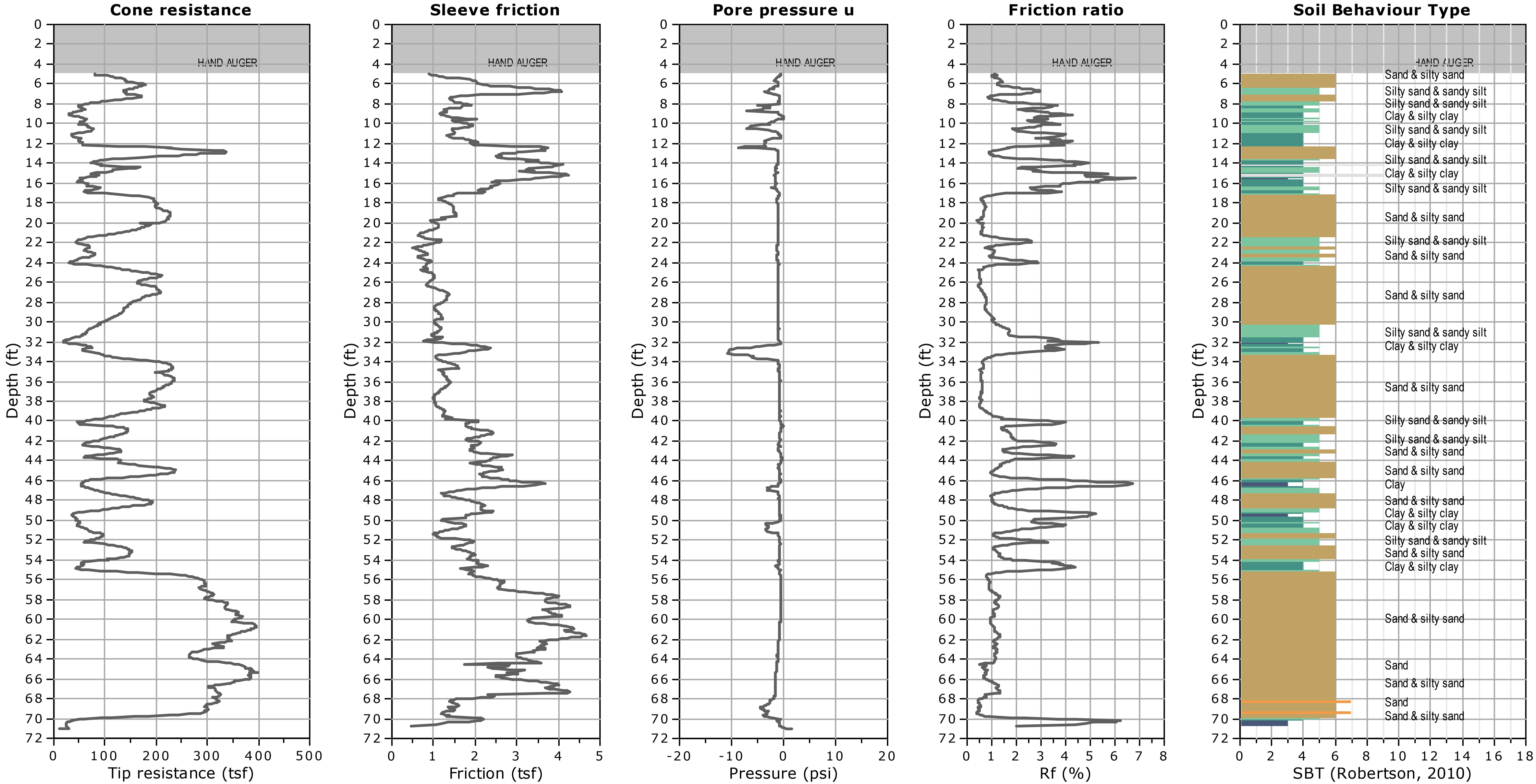
09/30/20-wt-2192

APPENDIX

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

Project: Leighton & Associates
Location: E. Katella Ave & Market St, Anaheim, CA

CPT-1
Total depth: 71.07 ft, Date: 9/28/2020



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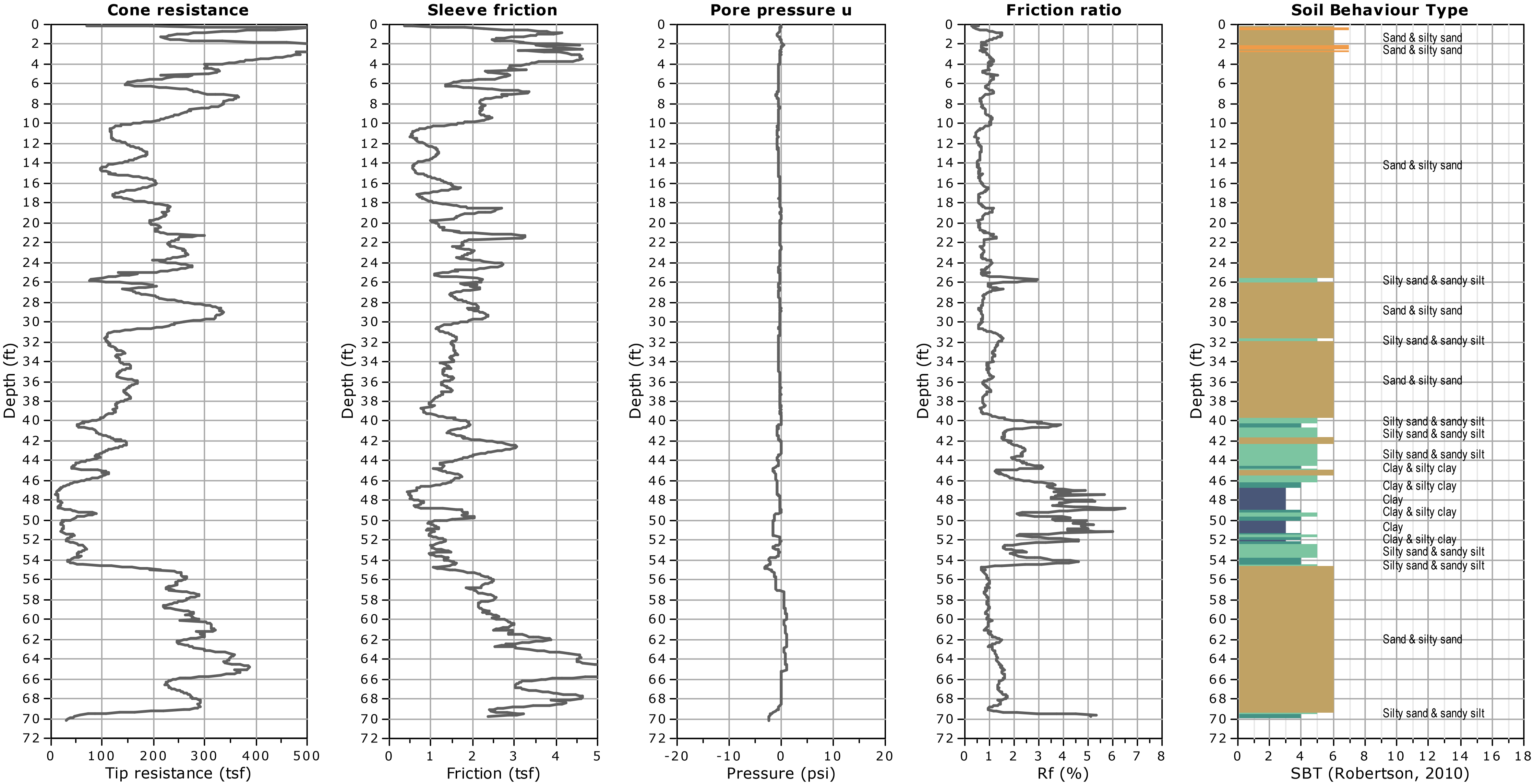
Project: Leighton & Associates
Location: E. Katella Ave & Market St, Anaheim, CA

CPT-2
Total depth: 70.17 ft, Date: 9/28/2020



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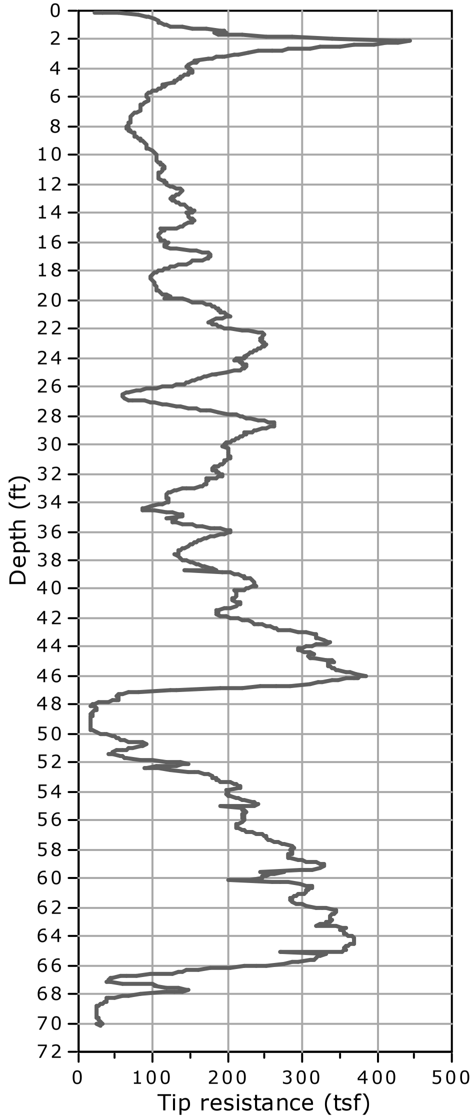
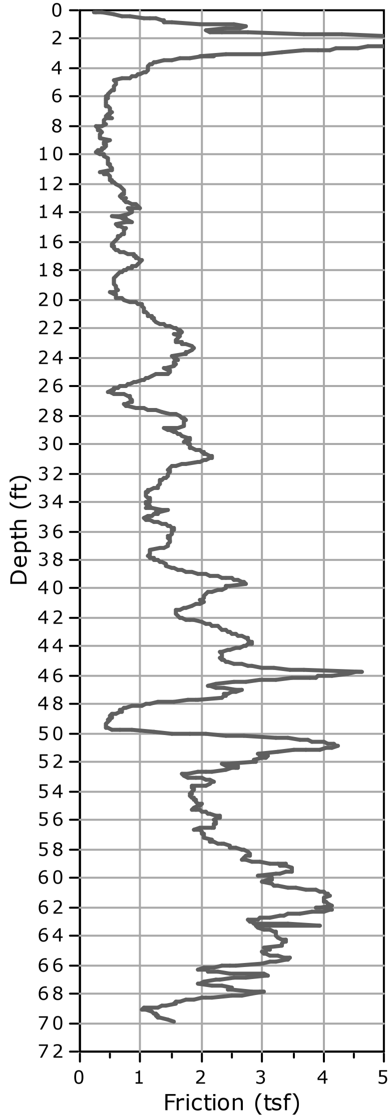
Project: Leighton & Associates
Location: E. Katella Ave & Market St, Anaheim, CA

CPT-3
Total depth: 70.24 ft, Date: 9/28/2020

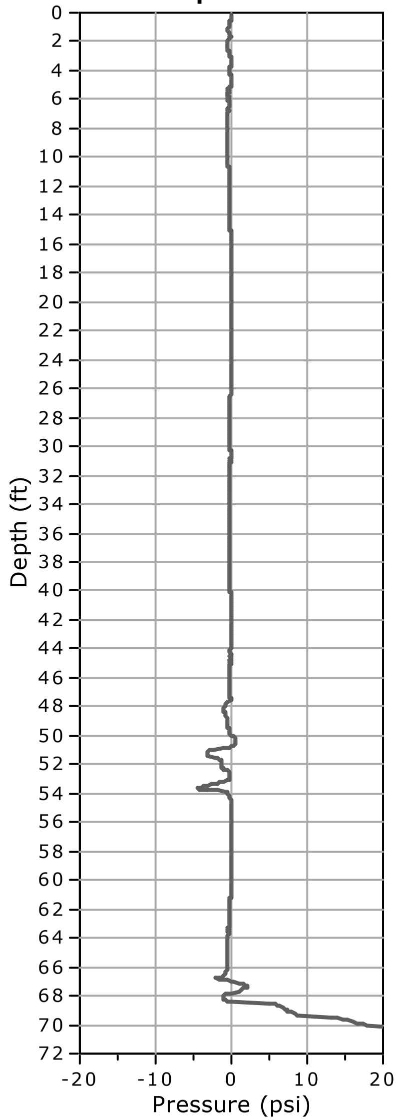
Cone resistance



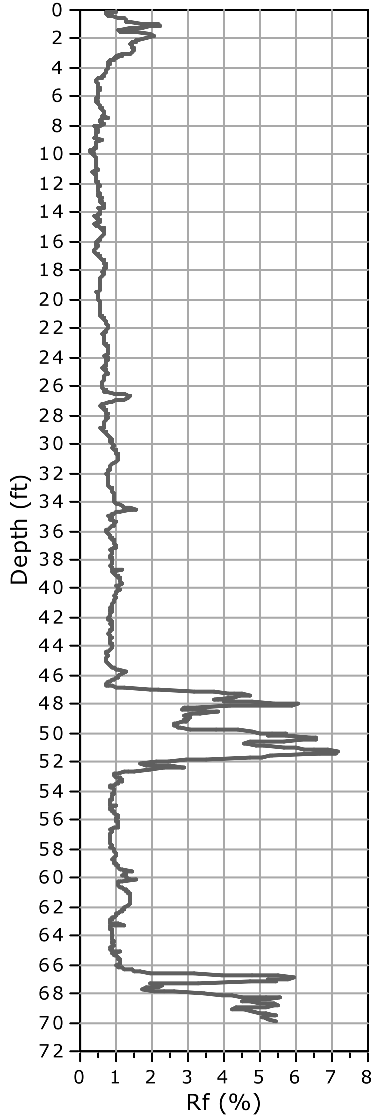
Sleeve friction



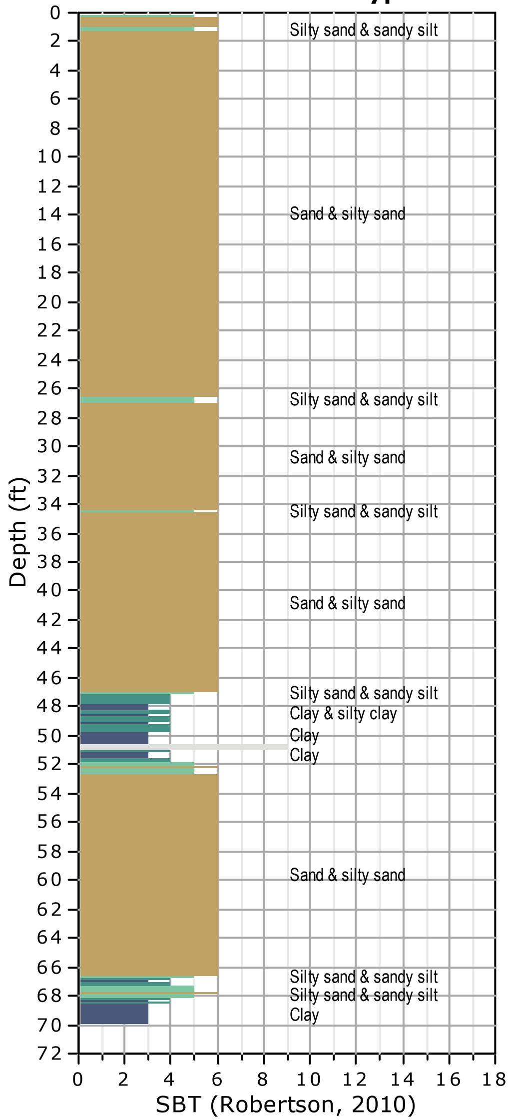
Pore pressure u



Friction ratio



Soil Behaviour Type



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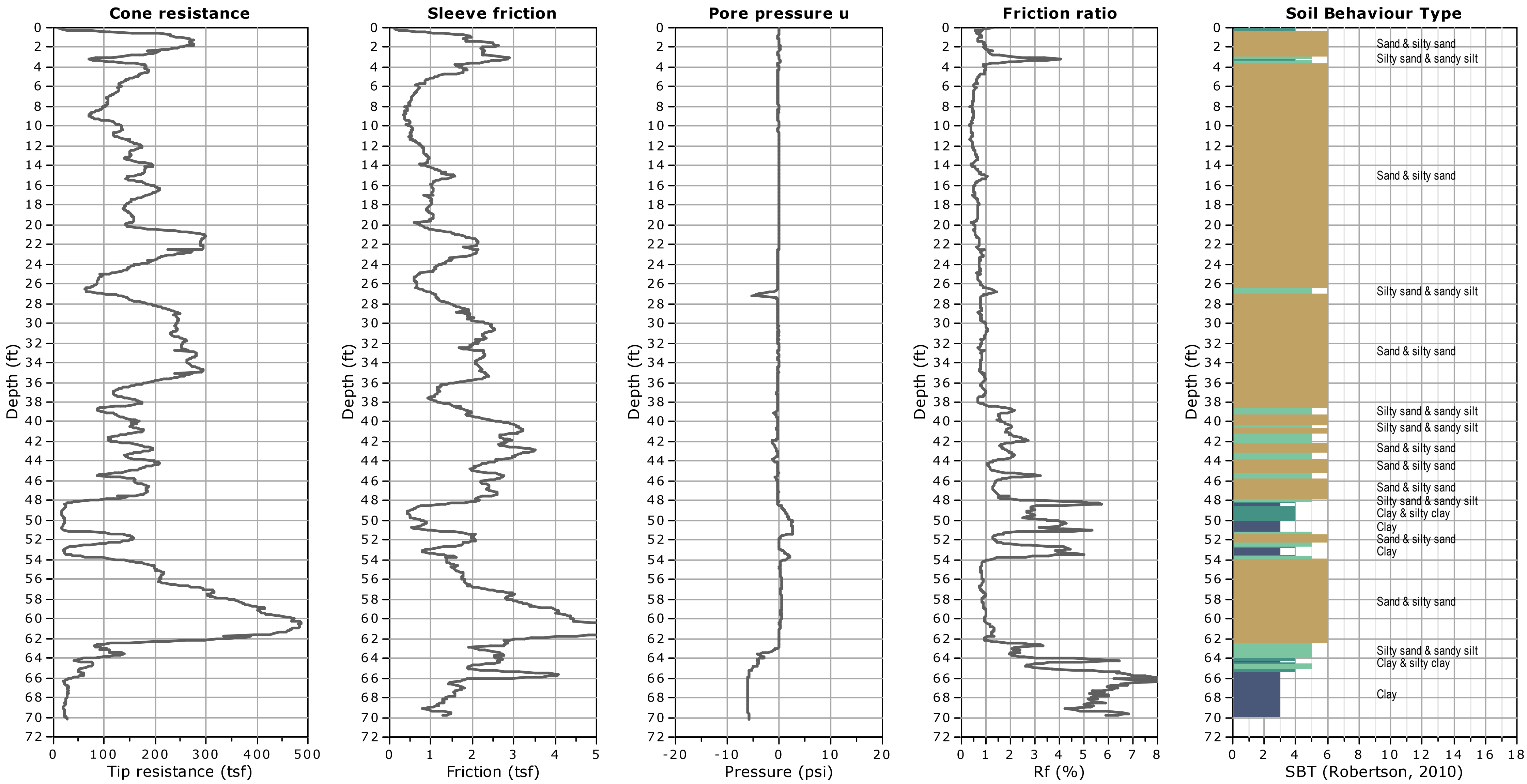
www.kehoetesting.com

Project: Leighton & Associates
Location: E. Katella Ave & Market St, Anaheim, CA

CPT-4
Total depth: 70.21 ft, Date: 9/28/2020



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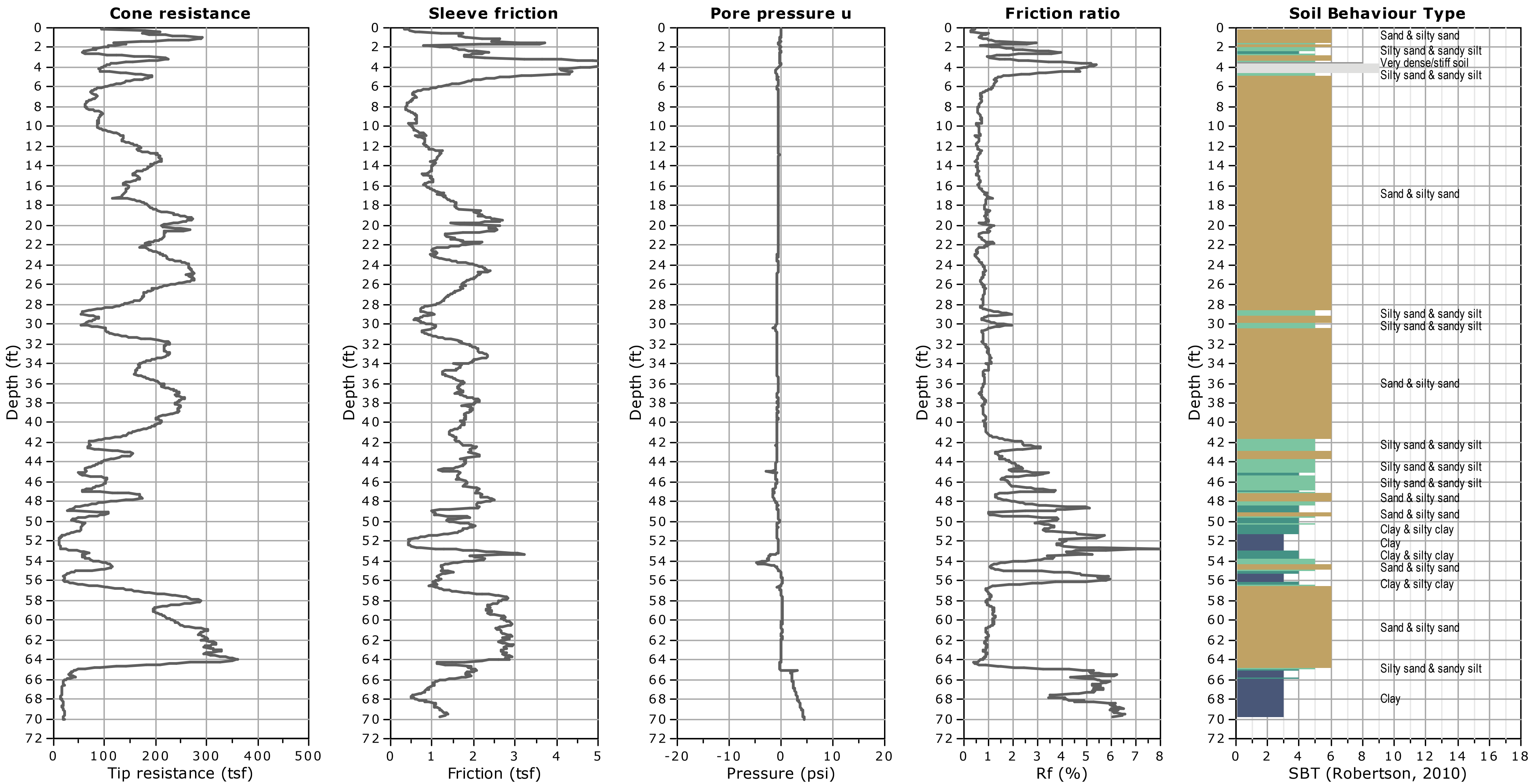
www.kehoetesting.com

Project: Leighton & Associates
Location: E. Katella Ave & Market St, Anaheim, CA

CPT-5
Total depth: 70.10 ft, Date: 9/28/2020



KTE
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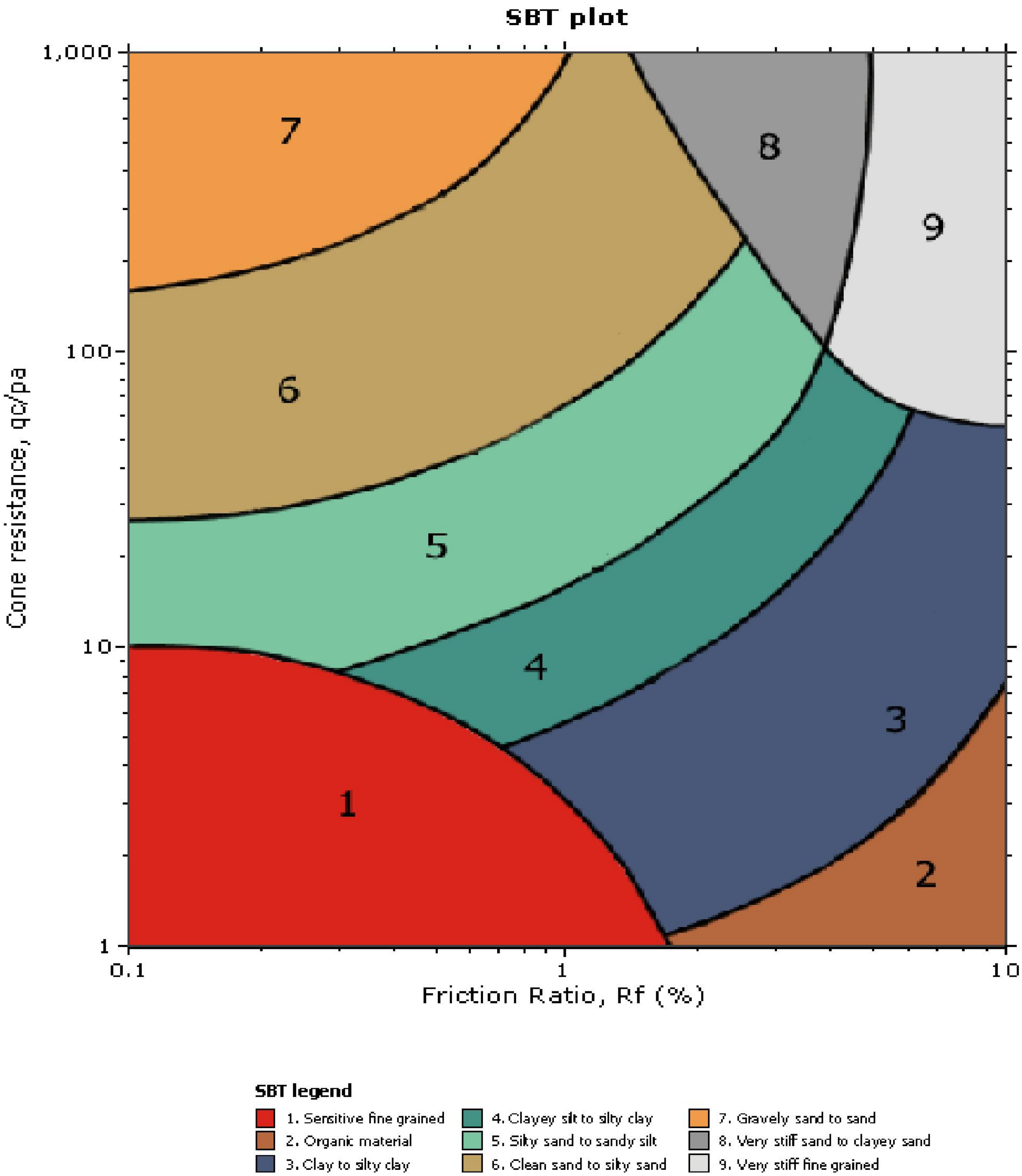

Leighton & Associates
E. Katella Ave & Market St.
Anaheim, CA

CPT Shear Wave Measurements

Location	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)
CPT-1	5.05	4.05	4.52	5.02	900	
	10.04	9.04	9.26	10.04	922	945
	15.06	14.06	14.20	15.52	915	902
	20.08	19.08	19.18	21.60	888	820
	25.07	24.07	24.15	27.52	878	839
	30.02	29.02	29.09	34.36	847	722
	35.07	34.07	34.13	41.76	817	681
	40.09	39.09	39.14	48.80	802	712
	45.01	44.01	44.06	54.28	812	897
	50.10	49.10	49.14	61.44	800	710
	55.09	54.09	54.13	67.58	801	812
	60.04	59.04	59.07	72.68	813	970
	65.06	64.06	64.09	76.96	833	1172
	70.14	69.14	69.17	81.70	847	1071
CPT-2	5.02	4.02	4.49	4.56	985	
	10.10	9.10	9.32	11.14	836	734
	15.09	14.09	14.23	17.12	831	822
	20.08	19.08	19.18	24.18	793	702
	25.07	24.07	24.15	30.40	795	799
	30.09	29.09	29.16	37.04	787	754
	35.07	34.07	34.13	42.80	797	863
	40.09	39.09	39.14	47.96	816	971
	45.11	44.11	44.16	54.28	813	793
	50.10	49.10	49.14	61.44	800	696
	55.02	54.02	54.06	67.24	804	848
	60.04	59.04	59.07	72.06	820	1041
	65.09	64.09	64.12	77.08	832	1005
	70.18	69.18	69.21	81.88	845	1060

Shear Wave Source Offset - 2 ft

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

Leighton & Associates
E. Katella Ave & Market St.
Anaheim, CA

CPT Shear Wave Measurements

Location	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)
CPT-3	5.05	4.05	4.52	6.00	753	
	10.04	9.04	9.26	13.80	671	608
	15.06	14.06	14.20	21.42	663	649
	20.05	19.05	19.15	28.76	666	675
	25.03	24.03	24.11	35.22	685	768
	30.05	29.05	29.12	41.10	708	851
	35.01	34.01	34.07	46.68	730	887
	40.06	39.06	39.11	52.14	750	924
	45.08	44.08	44.13	57.92	762	868
	50.10	49.10	49.14	64.88	757	721
	55.02	54.02	54.06	70.24	770	917
	60.07	59.07	59.10	75.26	785	1005
	65.09	64.09	64.12	80.08	801	1041
	70.08	69.08	69.11	85.44	809	931
CPT-4	6.04	5.04	5.42	8.12	668	
	10.04	9.04	9.26	15.20	609	542
	15.06	14.06	14.20	22.36	635	690
	20.08	19.08	19.18	28.24	679	847
	25.07	24.07	24.15	35.32	684	702
	30.05	29.05	29.12	41.96	694	748
	35.01	34.01	34.07	47.74	714	856
	40.03	39.03	39.08	53.20	735	918
	45.05	44.05	44.10	58.56	753	935
	50.03	49.03	49.07	64.78	757	800
	55.02	54.02	54.06	70.34	769	897
	60.04	59.04	59.07	75.40	783	991
	65.03	64.03	64.06	80.12	800	1057
	70.24	69.24	69.27	85.98	806	889

Shear Wave Source Offset - 2 ft

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

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Anaheim, CA

CPT Shear Wave Measurements

Location	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)
CPT-5	5.05	4.05	4.52	7.38	612	
	10.01	9.01	9.23	13.28	695	799
	15.06	14.06	14.20	19.80	717	763
	20.14	19.14	19.24	26.20	735	788
	25.03	24.03	24.11	32.96	732	720
	30.09	29.09	29.16	38.96	748	841
	35.04	34.04	34.10	44.96	758	823
	40.09	39.09	39.14	50.82	770	860
	45.05	44.05	44.10	56.52	780	869
	50.03	49.03	49.07	61.88	793	928
	55.05	54.05	54.09	67.68	799	865
	60.07	59.07	59.10	72.80	812	980
	65.06	64.06	64.09	77.28	829	1113
	70.11	69.11	69.14	82.94	834	892

Shear Wave Source Offset - 2 ft

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

APPENDIX B

PERCOLATION TEST DATA

Boring Percolation Test Data Sheet

Project Number:	12882.002	**Test Hole Number:**	LP-1
Project Name:	A-Town Parcels C and D	**Date Excavated:**	6/14/2022
Earth Description:	Alluvium	**Date Tested:**	7/1/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	45
Tested By:	AS/KMD	**Radius of boring, r (in):**	4
		Radius of casing (in):	1
		Length of slotted of casing (ft):	10
		Depth to Initial Water Depth (ft):	34
		Porosity of Annulus Material, *n* :	1
		Bentonite Plug at Bottom:	No

Field Percolation Data - Constant Head Test

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	9:22	-	34.40	127.2	-
2	9:30	8	34.09	130.9	32.0
3	9:35	5	34.08	131.0	48.0
4	10:28	-	33.46	138.5	-
5	10:40	12	33.11	142.7	71.0
6	10:56	16	33.38	139.4	93.0
7	11:06	10	33.46	138.5	108.0
8	11:12	6	33.50	138.0	115.0
9	11:49	-	34.00	132.0	-
10	11:59	10	34.30	128.4	122.0
11	12:09	10	34.30	128.4	132.0
12	12:20	11	34.80	122.4	138.0
13	12:37	17	33.96	132.5	152.0
14	12:47	10	34.20	129.6	162.0
15	12:58	11	34.50	126.0	172.0
16	13:08	10	34.68	123.8	177.0
17	13:41	-	34.68	123.8	-
18	13:51	10	34.68	123.8	189.0
19	14:01	10	34.68	123.8	202.0
20	14:11	10	34.68	123.8	212.0
21	14:22	11	34.68	123.8	219.0
22	14:31	9	34.68	123.8	228.0

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	228.0
Total Volume of Water Delivered (cubic inches)	52668
Average Water Height (inches)	129.7
Average Percolation Surface Area (cubic Inches)	3308.8
Duration of Test (minutes)	186
Duration of Test (hours)	3.10

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

Measured Infiltration Rate (inches per hour) = 5.1

Boring Percolation Test Data Sheet

Project Number:	12882.002	**Test Hole Number:**	LP-2
Project Name:	A-Town Parcels C and D	**Date Excavated:**	6/14/2022
Earth Description:	Alluvium	**Date Tested:**	7/1/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	40
Tested By:	AS/KMD	**Radius of boring, r (in):**	4
		Radius of casing (in):	1
		Length of slotted of casing (ft):	10
		Depth to Initial Water Depth (ft):	32
		Porosity of Annulus Material, *n* :	1
		Bentonite Plug at Bottom:	No

Field Percolation Data - Constant Head Test

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	8:15	-	-	-	-
2	8:25	10	31.92	97.0	20.7
3	8:35	10	31.99	96.1	41.4
4	8:45	10	31.98	96.2	62.1
5	8:55	10	32.04	95.5	82.8
6	9:23	28	32.08	95.0	140.7
7	9:39	16	32.10	94.8	173.8
8	9:51	12	32.12	94.6	198.6
9	10:02	11	32.13	94.4	221.4
10	10:11	9	32.12	94.6	240.0
11	10:32	21	32.13	94.4	283.4
12	10:48	16	32.15	94.2	316.6
13	11:18	30	32.30	92.4	378.6
14	11:36	18	32.38	91.4	415.9
15	12:13	37	32.65	88.2	492.4

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	492.4
Total Volume of Water Delivered (cubic inches)	113747.586
Average Water Height (inches)	94.2
Average Percolation Surface Area (cubic Inches)	2418.0
Duration of Test (minutes)	238
Duration of Test (hours)	3.97

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

Measured Infiltration Rate (inches per hour) = 11.9

Boring Percolation Test Data Sheet

Project Number:	12882.002	**Test Hole Number:**	LP-3
Project Name:	A-Town Parcels C and D	**Date Excavated:**	7/1/2022
Earth Description:	Alluvium	**Date Tested:**	7/1/2022
Liquid Description:	Tap water	**Depth of boring (ft):**	40
Tested By:	AS/KMD	**Radius of boring, r (in):**	4
		Radius of casing (in):	1
		Length of slotted of casing (ft):	10
		Depth to Initial Water Depth (ft):	30
		Porosity of Annulus Material, *n* :	1
		Bentonite Plug at Bottom:	No

Field Percolation Data - Constant Head Test

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	10:15	-	-	-	-
2	11:15	60	38.80	14.4	124.1
3	12:52	10	29.49	126.1	144.8
4	13:02	10	29.52	125.8	165.5
5	13:12	10	29.67	124.0	186.2
6	13:22	10	29.80	122.4	206.9
7	13:32	10	29.84	121.9	227.6
8	13:42	10	29.98	120.2	248.3
9	13:52	10	29.91	121.1	269.0
10	14:02	10	29.90	121.2	289.7
11	14:12	10	29.93	120.8	310.3
12	14:22	10	29.95	120.6	331.0
13	14:32	10	29.91	121.1	351.7

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	351.7
Total Volume of Water Delivered (cubic inches)	81248.2759
Average Water Height (inches)	113.3
Average Percolation Surface Area (cubic Inches)	2897.8
Duration of Test (minutes)	170
Duration of Test (hours)	2.83

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

Measured Infiltration Rate (inches per hour) = 9.9

APPENDIX C

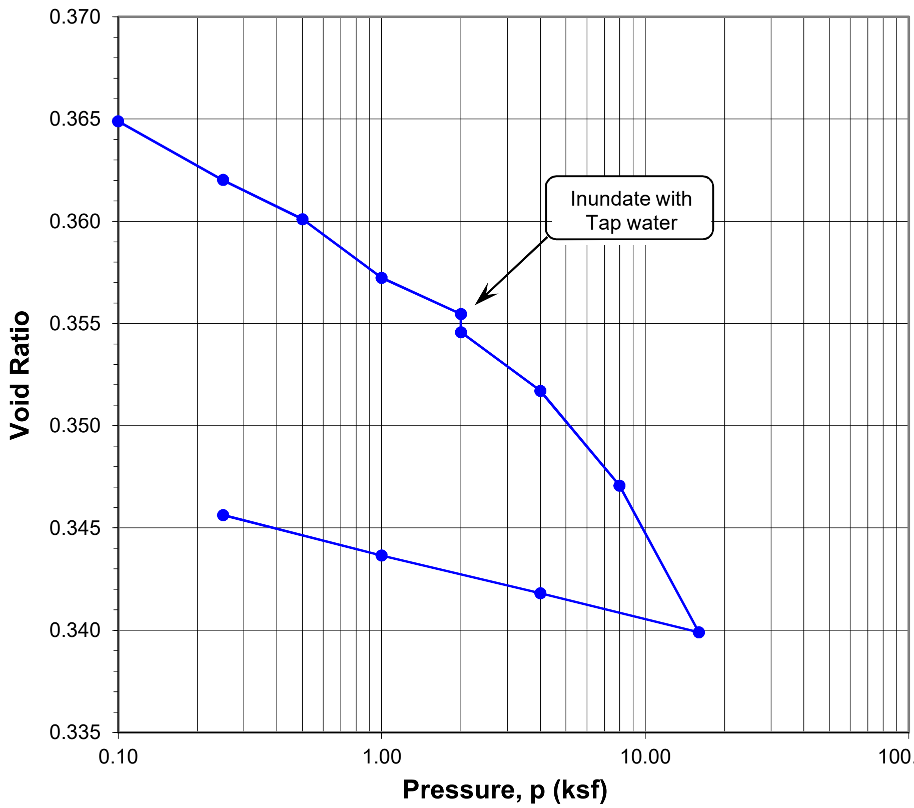
LABORATORY TEST RESULTS

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	A-Town Parcels C & D	Tested By: G. Bathala	Date: 06/21/22
Project No.:	12882.002	Checked By: J. Ward	Date: 07/27/22
Boring No.:	LB-1	Depth (ft.): 0-5	
Sample No.:	B-1	Sample Type:	95% Remold
Soil Identification:	Olive brown silty sand with gravel (SM)g		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	205.17
Weight of Ring (g):	44.68
Height after consol. (in.):	0.9856
Before Test	
Wt. of Wet Sample+Cont. (g):	176.60
Wt. of Dry Sample+Cont. (g):	168.43
Weight of Container (g):	67.68
Initial Moisture Content (%)	8.1
Initial Dry Density (pcf)	123.5
Initial Saturation (%):	60
Initial Vertical Reading (in.)	0.0914
After Test	
Wt. of Wet Sample+Cont. (g):	267.60
Wt. of Dry Sample+Cont. (g):	252.14
Weight of Container (g):	58.95
Final Moisture Content (%)	10.41
Final Dry Density (pcf):	125.3
Final Saturation (%):	81
Final Vertical Reading (in.)	0.1089
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

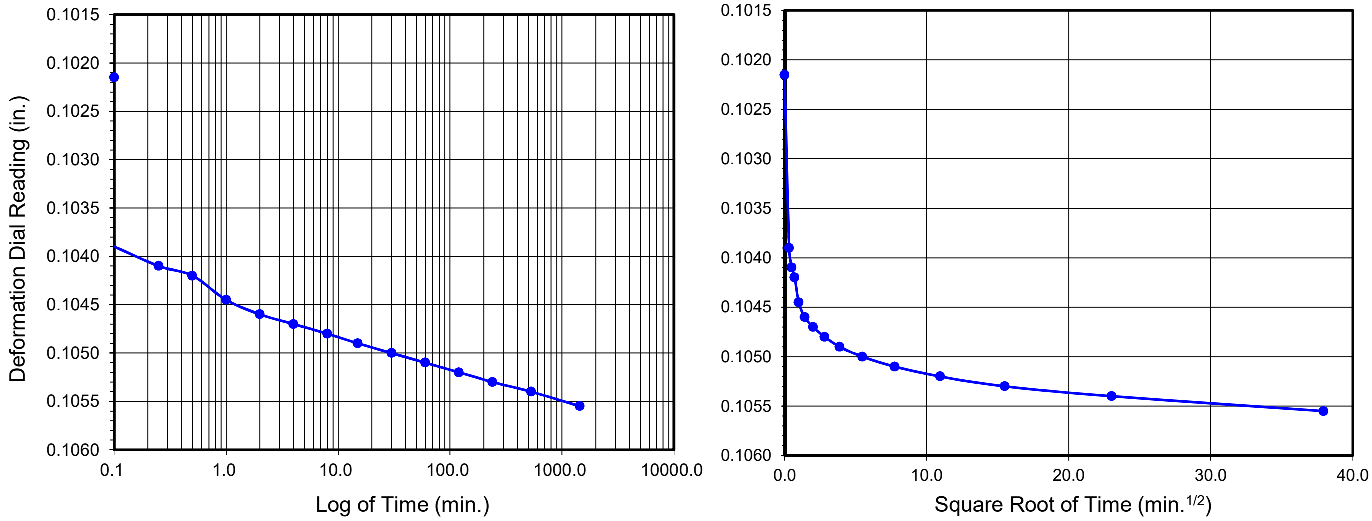


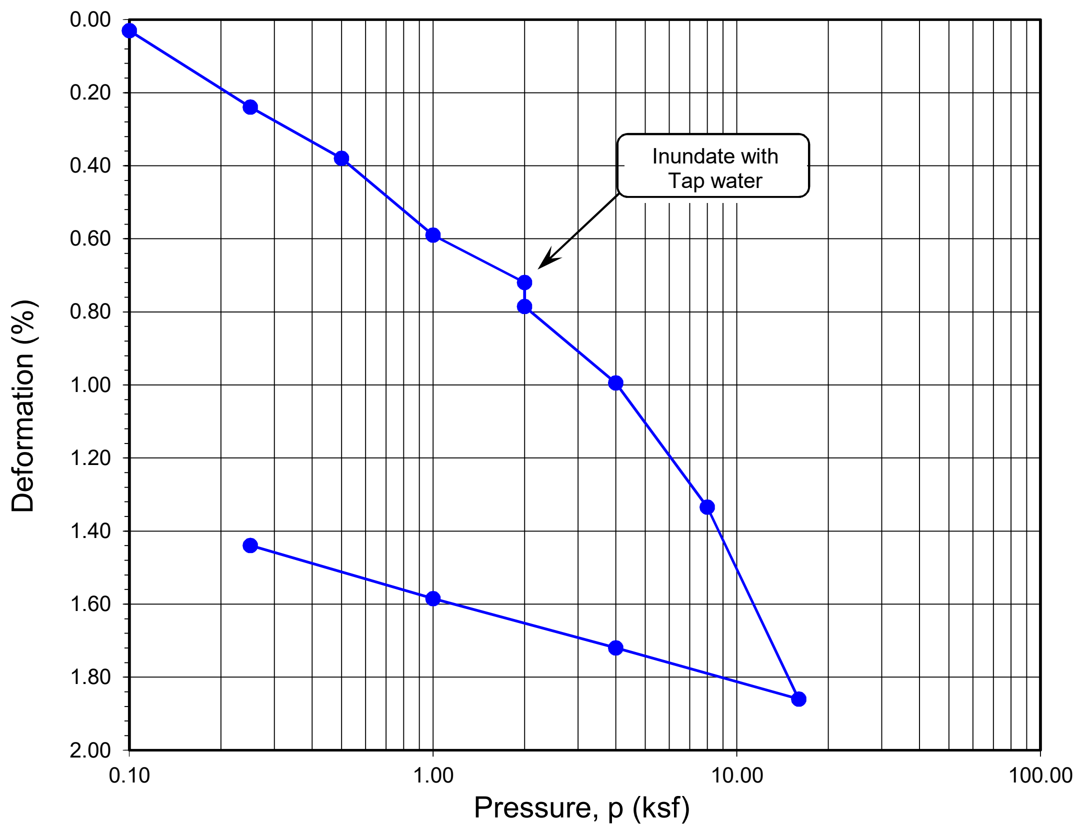
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0917	0.9997	0.00	0.03	0.365	0.03
0.25	0.0943	0.9971	0.05	0.29	0.362	0.24
0.50	0.0963	0.9951	0.11	0.49	0.360	0.38
1.00	0.0992	0.9922	0.19	0.78	0.357	0.59
2.00	0.1015	0.9899	0.29	1.01	0.355	0.72
2.00	0.1022	0.9893	0.29	1.08	0.355	0.79
4.00	0.1056	0.9859	0.42	1.42	0.352	1.00
8.00	0.1103	0.9812	0.55	1.89	0.347	1.34
16.00	0.1169	0.9745	0.69	2.55	0.340	1.86
4.00	0.1141	0.9773	0.55	2.27	0.342	1.72
1.00	0.1114	0.9801	0.41	2.00	0.344	1.59
0.25	0.1089	0.9825	0.31	1.75	0.346	1.44

Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
6/24/22	7:30:00	0.0	0.0	0.1022
6/24/22	7:30:06	0.1	0.3	0.1039
6/24/22	7:30:15	0.2	0.5	0.1041
6/24/22	7:30:30	0.5	0.7	0.1042
6/24/22	7:31:00	1.0	1.0	0.1045
6/24/22	7:32:00	2.0	1.4	0.1046
6/24/22	7:34:00	4.0	2.0	0.1047
6/24/22	7:38:00	8.0	2.8	0.1048
6/24/22	7:45:00	15.0	3.9	0.1049
6/24/22	8:00:00	30.0	5.5	0.1050
6/24/22	8:30:00	60.0	7.7	0.1051
6/24/22	9:30:00	120.0	11.0	0.1052
6/24/22	11:30:00	240.0	15.5	0.1053
6/24/22	16:20:00	530.0	23.0	0.1054
6/25/22	7:30:00	1440.0	37.9	0.1056

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-1	**B-1**	**0-5**	**8.1**	**10.4**	**123.5**	**125.3**	**0.365**	**0.346**	**60**	**81**

Soil Identification: Olive brown silty sand with gravel (SM)g

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 12882.002

A-Town Parcels C & D

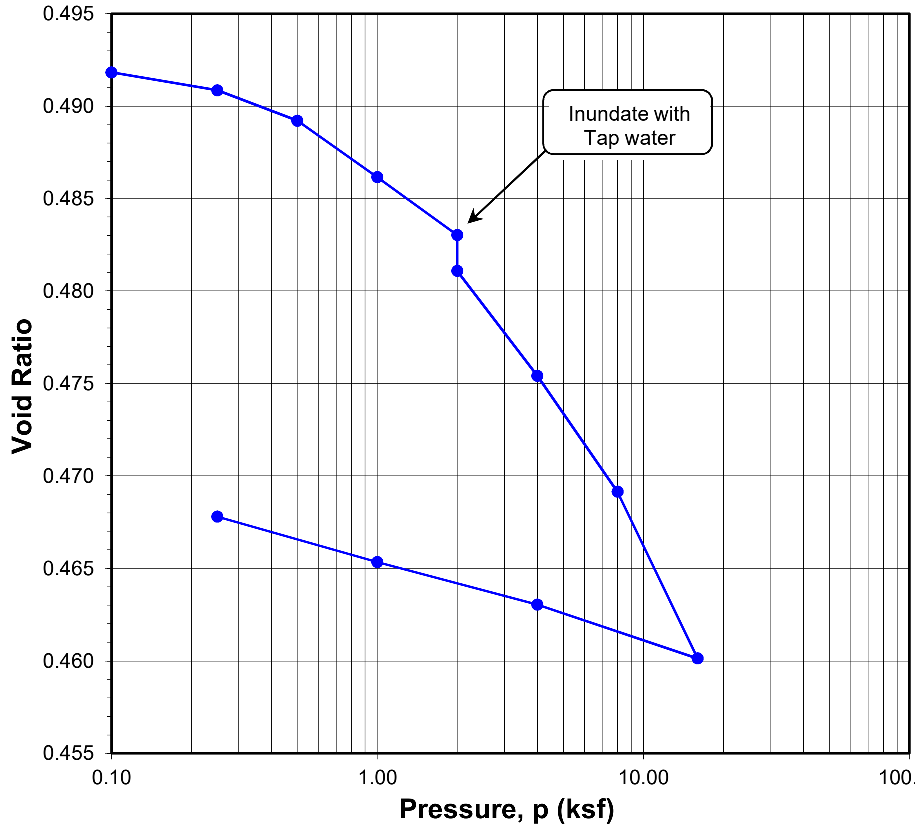
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	A-Town Parcels C & D	Tested By: G. Bathala	Date:	06/20/22
Project No.:	12882.002	Checked By: J. Ward	Date:	07/27/22
Boring No.:	LP-1	Depth (ft.): 30.0		
Sample No.:	R-3	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)	190.60
Weight of Ring (g)	45.68
Height after consol. (in.)	0.9838
Before Test	
Wt.Wet Sample+Cont. (g)	193.05
Wt.of Dry Sample+Cont. (g)	185.10
Weight of Container (g)	66.07
Initial Moisture Content (%)	6.7
Initial Dry Density (pcf)	113.0
Initial Saturation (%)	37
Initial Vertical Reading (in.)	0.2910
After Test	
Wt.of Wet Sample+Cont. (g)	260.98
Wt. of Dry Sample+Cont. (g)	240.95
Weight of Container (g)	60.89
Final Moisture Content (%)	14.91
Final Dry Density (pcf)	113.6
Final Saturation (%)	83
Final Vertical Reading (in.)	0.2711
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

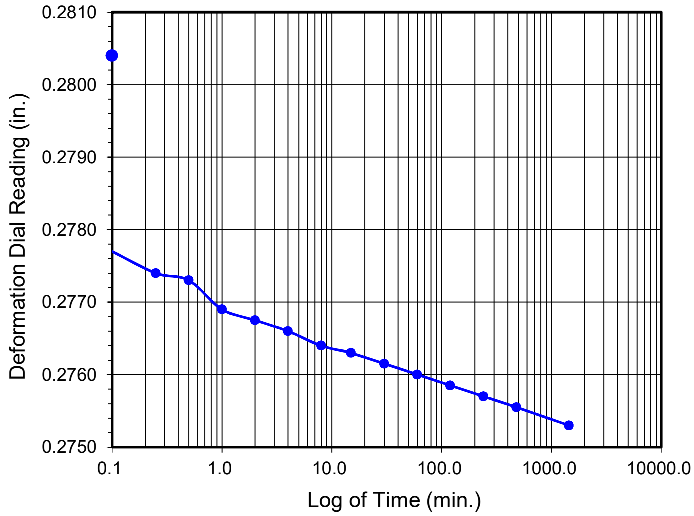


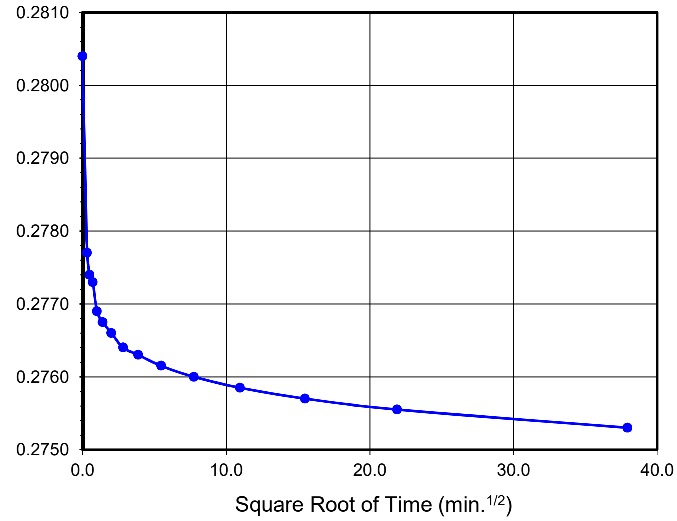
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2909	0.9999	0.00	0.01	0.492	0.01
0.25	0.2896	0.9986	0.07	0.14	0.491	0.07
0.50	0.2879	0.9969	0.13	0.31	0.489	0.18
1.00	0.2850	0.9940	0.21	0.60	0.486	0.39
2.00	0.2817	0.9907	0.33	0.93	0.483	0.60
2.00	0.2804	0.9894	0.33	1.06	0.481	0.73
4.00	0.2753	0.9843	0.46	1.57	0.475	1.11
8.00	0.2693	0.9783	0.64	2.17	0.469	1.53
16.00	0.2611	0.9701	0.86	2.99	0.460	2.13
4.00	0.2648	0.9738	0.68	2.62	0.463	1.94
1.00	0.2682	0.9772	0.50	2.29	0.465	1.79
0.25	0.2711	0.9801	0.37	1.99	0.468	1.62

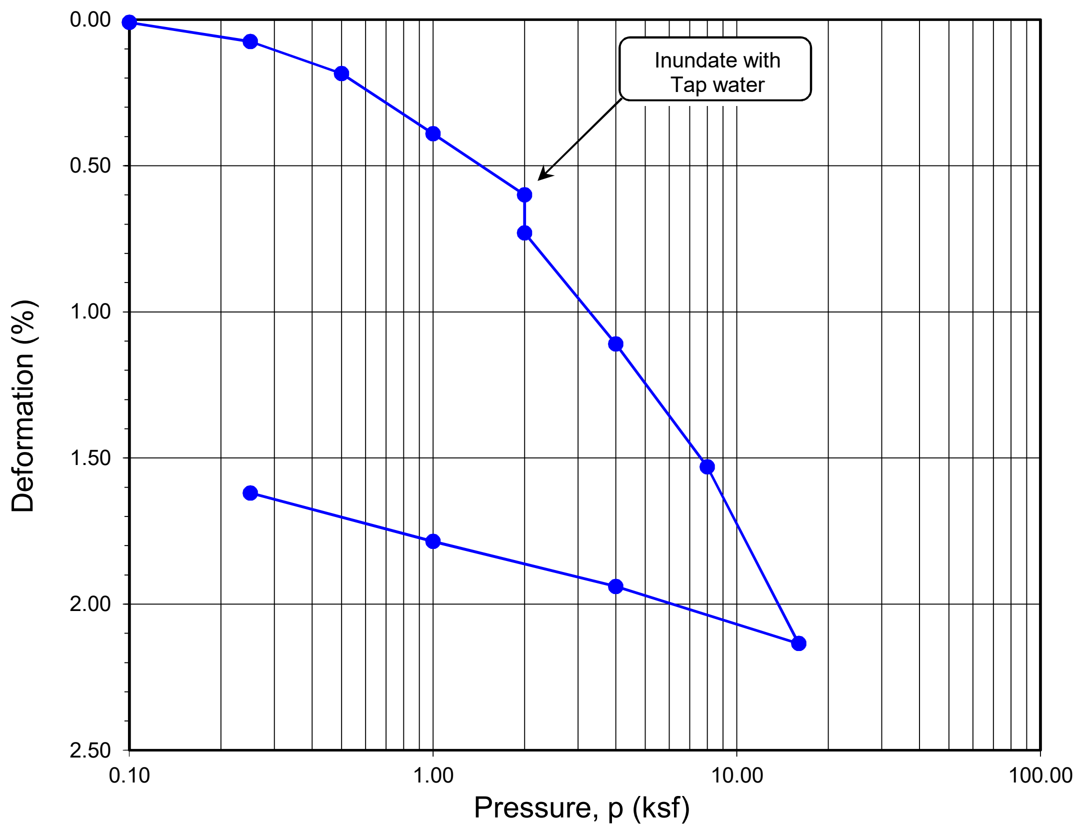
Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
6/23/22	7:50:00	0.0	0.0	0.2804
6/23/22	7:50:06	0.1	0.3	0.2777
6/23/22	7:50:15	0.2	0.5	0.2774
6/23/22	7:50:30	0.5	0.7	0.2773
6/23/22	7:51:00	1.0	1.0	0.2769
6/23/22	7:52:00	2.0	1.4	0.2768
6/23/22	7:54:00	4.0	2.0	0.2766
6/23/22	7:58:00	8.0	2.8	0.2764
6/23/22	8:05:00	15.0	3.9	0.2763
6/23/22	8:20:00	30.0	5.5	0.2762
6/23/22	8:50:00	60.0	7.7	0.2760
6/23/22	9:50:00	120.0	11.0	0.2759
6/23/22	11:50:00	240.0	15.5	0.2757
6/23/22	15:50:00	480.0	21.9	0.2756
6/24/22	7:50:00	1440.0	37.9	0.2753

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
LP-1	**R-3**	**30.0**	**6.7**	**14.9**	**113.0**	**113.6**	**0.492**	**0.468**	**37**	**83**

Soil Identification: Light olive brown silty sand (SM)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 12882.002

A-Town Parcels C & D

07-22

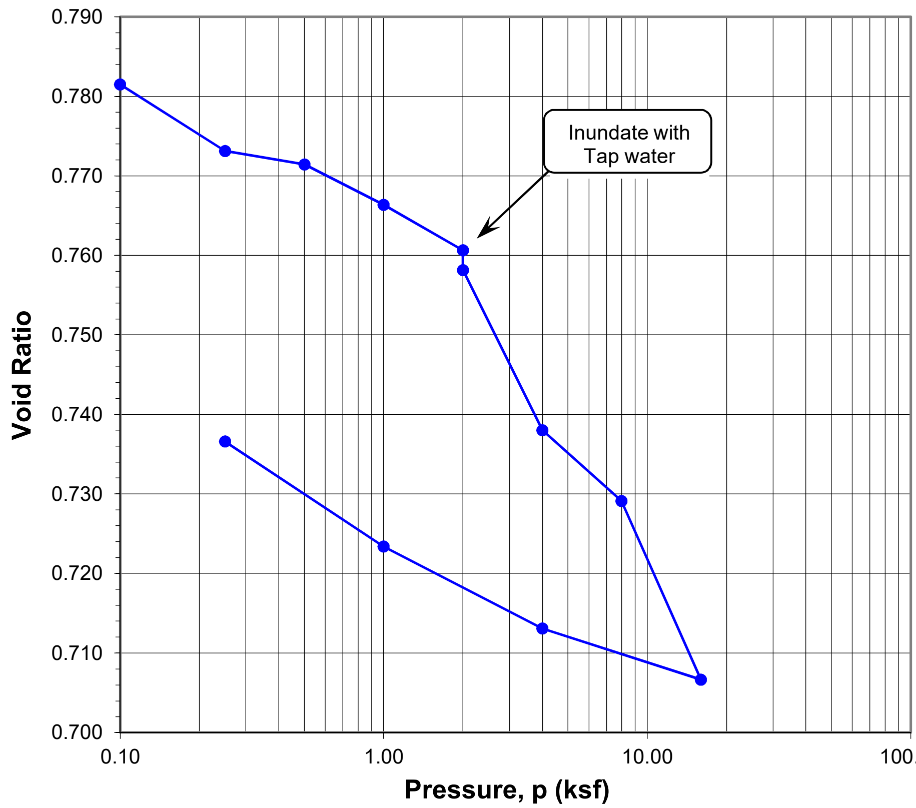
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	A-Town Parcels C & D	Tested By: G. Bathala	Date: 06/24/22
Project No.:	12882.002	Checked By: J. Ward	Date: 07/27/22
Boring No.:	LP-1	Depth (ft.): 40.0	
Sample No.:	R-4	Sample Type:	Ring
Soil Identification:	Olive silt (ML)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	170.03
Weight of Ring (g):	45.14
Height after consol. (in.):	0.9746
Before Test	
Wt. of Wet Sample+Cont. (g):	176.52
Wt. of Dry Sample+Cont. (g):	166.81
Weight of Container (g):	67.69
Initial Moisture Content (%)	9.8
Initial Dry Density (pcf)	94.6
Initial Saturation (%):	34
Initial Vertical Reading (in.)	0.0738
After Test	
Wt. of Wet Sample+Cont. (g):	225.09
Wt. of Dry Sample+Cont. (g):	195.75
Weight of Container (g):	37.73
Final Moisture Content (%)	25.99
Final Dry Density (pcf):	96.3
Final Saturation (%):	94
Final Vertical Reading (in.)	0.1017
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

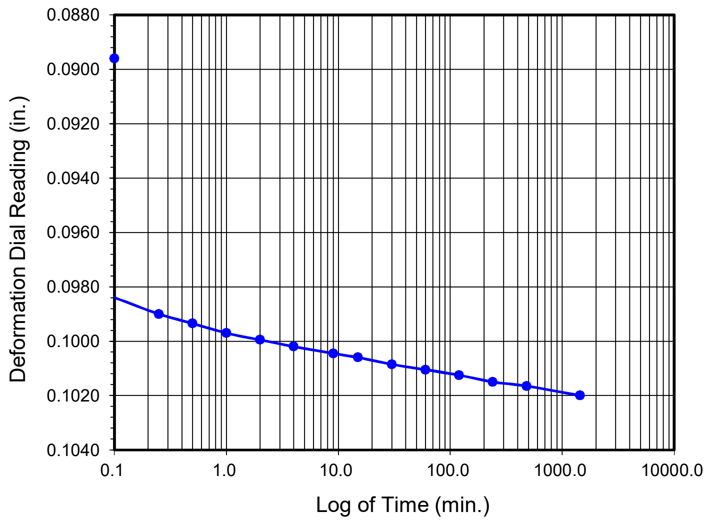


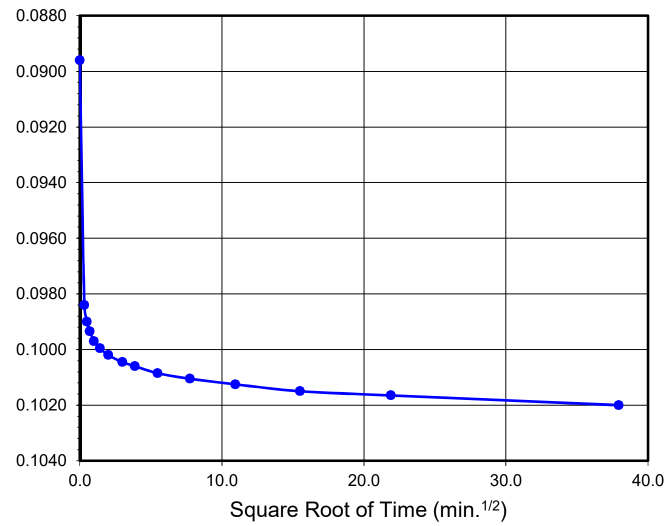
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0740	0.9998	0.00	0.02	0.781	0.02
0.25	0.0790	0.9948	0.03	0.52	0.773	0.49
0.50	0.0805	0.9934	0.08	0.67	0.771	0.59
1.00	0.0840	0.9898	0.15	1.02	0.766	0.87
2.00	0.0882	0.9856	0.25	1.44	0.761	1.19
2.00	0.0896	0.9842	0.25	1.58	0.758	1.33
4.00	0.1020	0.9718	0.36	2.82	0.738	2.46
8.00	0.1083	0.9655	0.49	3.45	0.729	2.96
16.00	0.1224	0.9514	0.64	4.86	0.707	4.22
4.00	0.1171	0.9567	0.47	4.33	0.713	3.86
1.00	0.1100	0.9638	0.34	3.62	0.723	3.28
0.25	0.1017	0.9721	0.25	2.79	0.737	2.54

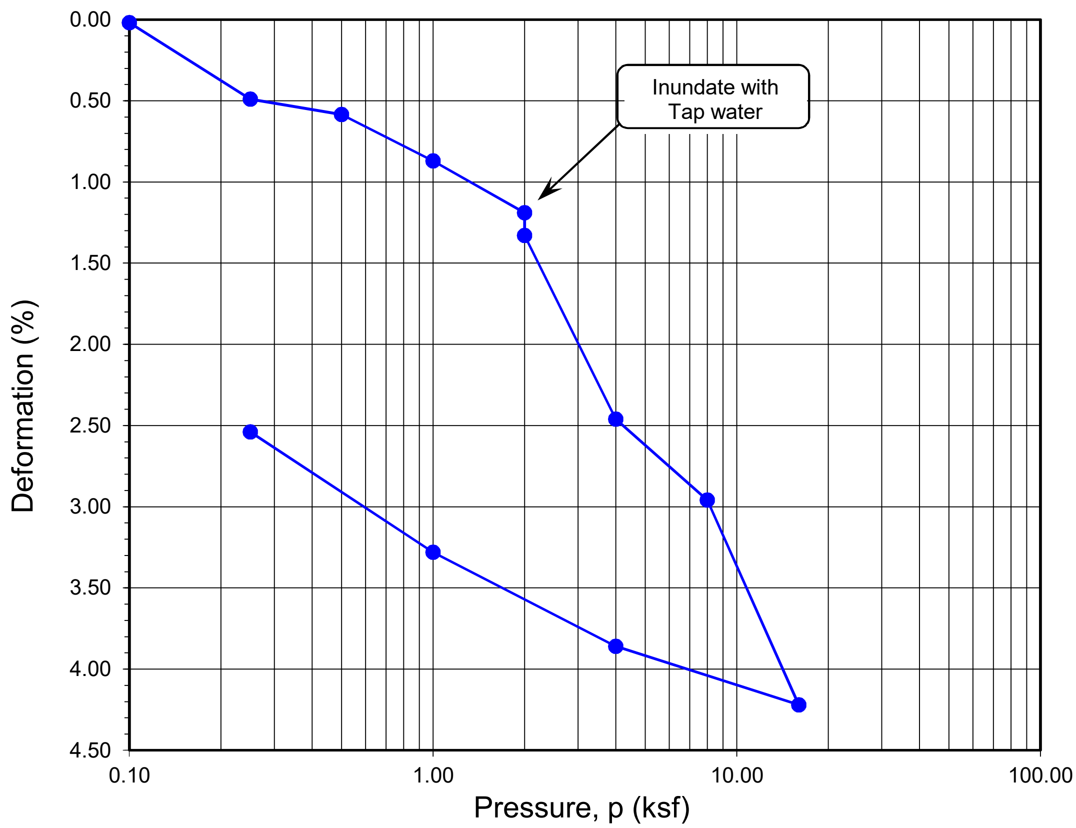
Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
6/28/22	7:30:00	0.0	0.0	0.0896
6/28/22	7:30:06	0.1	0.3	0.0984
6/28/22	7:30:15	0.2	0.5	0.0990
6/28/22	7:30:30	0.5	0.7	0.0994
6/28/22	7:31:00	1.0	1.0	0.0997
6/28/22	7:32:00	2.0	1.4	0.1000
6/28/22	7:34:00	4.0	2.0	0.1002
6/28/22	7:39:00	9.0	3.0	0.1005
6/28/22	7:45:00	15.0	3.9	0.1006
6/28/22	8:00:00	30.0	5.5	0.1009
6/28/22	8:30:00	60.0	7.7	0.1011
6/28/22	9:30:00	120.0	11.0	0.1013
6/28/22	11:30:00	240.0	15.5	0.1015
6/28/22	15:30:00	480.0	21.9	0.1017
6/29/22	7:30:00	1440.0	37.9	0.1020

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
LP-1	**R-4**	**40**	**9.8**	**26.0**	**94.6**	**96.3**	**0.782**	**0.737**	**34**	**94**

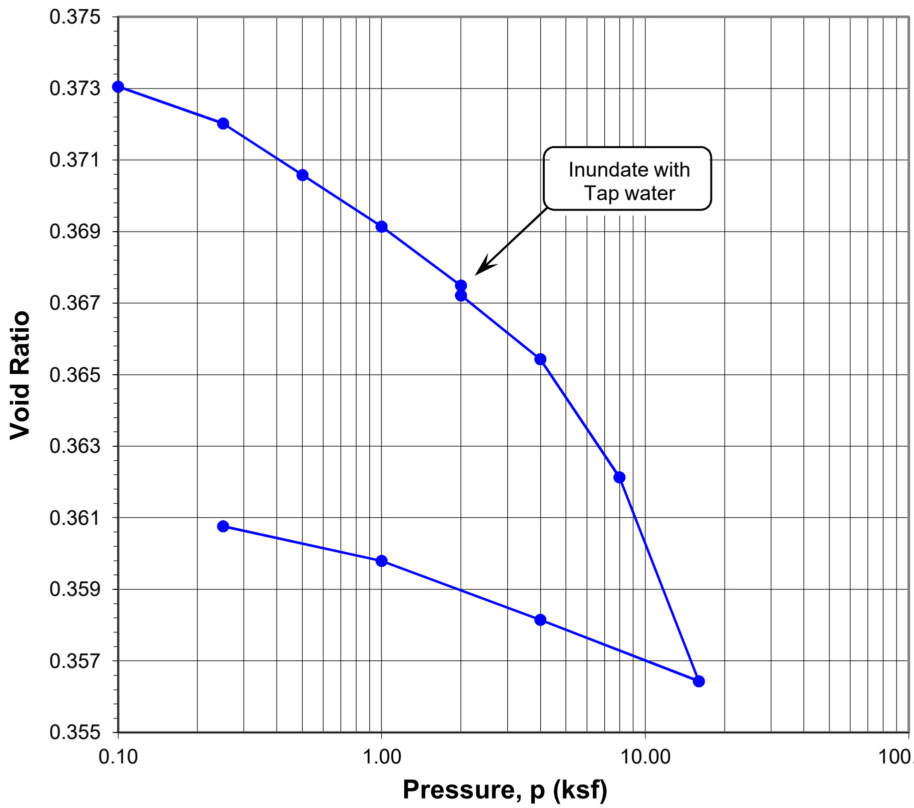
Soil Identification: Olive silt (ML)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	A-Town Parcels C & D	Tested By: G. Bathala	Date: 06/23/22
Project No.:	12882.002	Checked By: J. Ward	Date: 07/27/22
Boring No.:	LP-2	Depth (ft.): 0-5	
Sample No.:	B-1	Sample Type:	95% Remold
Soil Identification:	Olive brown silty sand (SM)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	204.28
Weight of Ring (g):	45.38
Height after consol. (in.):	0.9910
Before Test	
Wt. of Wet Sample+Cont. (g):	172.37
Wt. of Dry Sample+Cont. (g):	163.86
Weight of Container (g):	52.64
Initial Moisture Content (%)	7.7
Initial Dry Density (pcf)	122.8
Initial Saturation (%):	55
Initial Vertical Reading (in.)	0.0865
After Test	
Wt. of Wet Sample+Cont. (g):	265.09
Wt. of Dry Sample+Cont. (g):	250.85
Weight of Container (g):	58.03
Final Moisture Content (%)	9.66
Final Dry Density (pcf):	123.7
Final Saturation (%):	72
Final Vertical Reading (in.)	0.0990
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

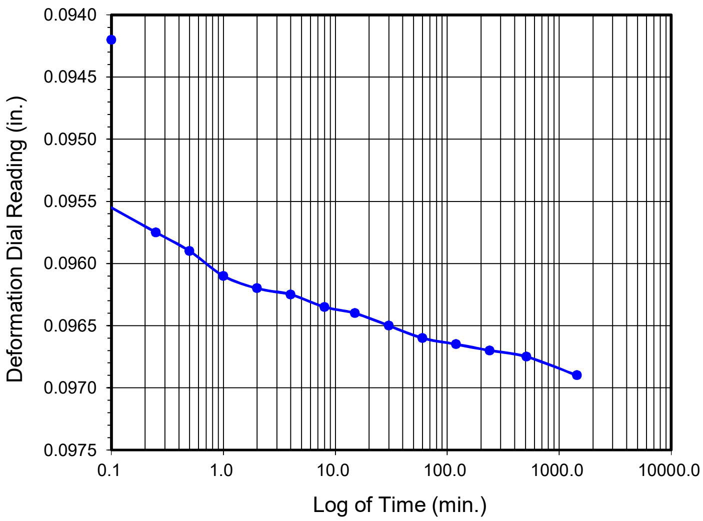


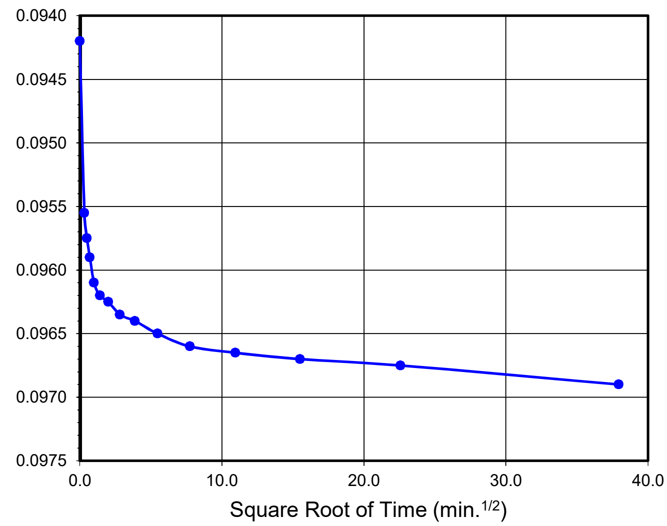
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0866	1.0000	0.00	0.00	0.373	0.00
0.25	0.0878	0.9987	0.05	0.13	0.372	0.08
0.50	0.0897	0.9969	0.13	0.31	0.371	0.18
1.00	0.0916	0.9949	0.22	0.51	0.369	0.29
2.00	0.0940	0.9925	0.34	0.75	0.367	0.41
2.00	0.0942	0.9923	0.34	0.77	0.367	0.43
4.00	0.0969	0.9896	0.48	1.04	0.365	0.56
8.00	0.1009	0.9856	0.64	1.44	0.362	0.80
16.00	0.1073	0.9793	0.86	2.08	0.356	1.22
4.00	0.1035	0.9830	0.61	1.70	0.358	1.09
1.00	0.1008	0.9857	0.46	1.43	0.360	0.97
0.25	0.0990	0.9875	0.35	1.25	0.361	0.90

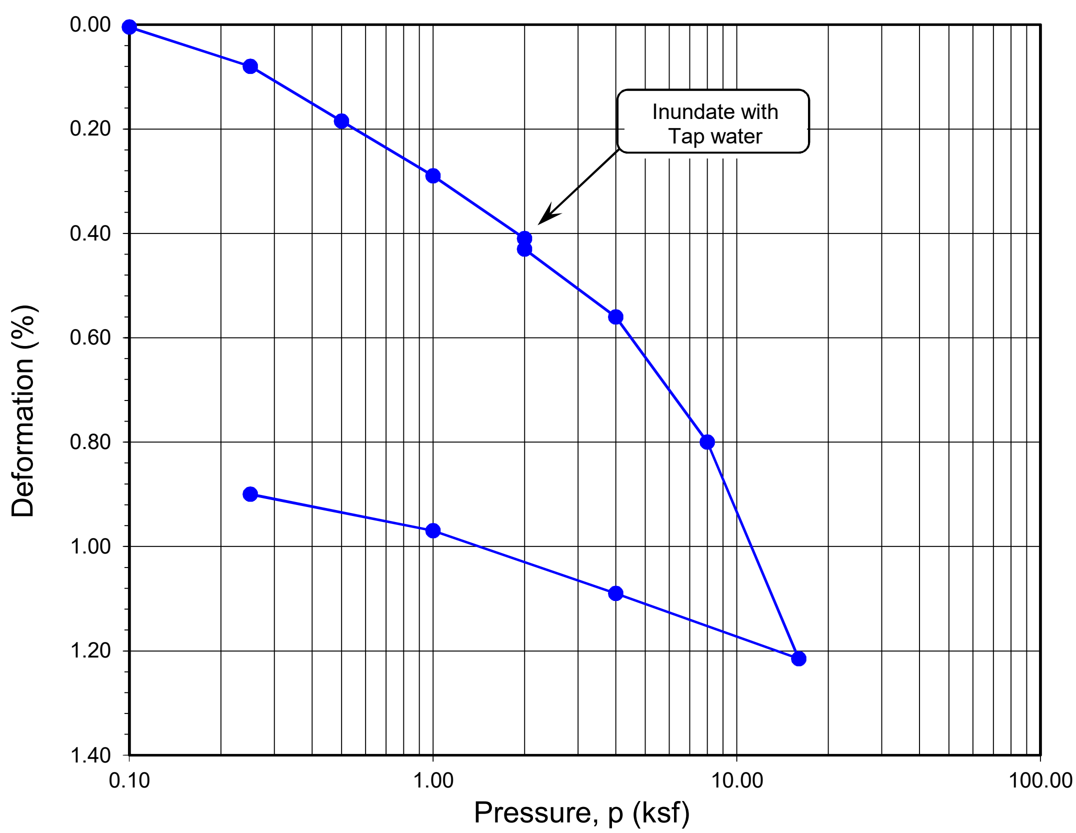
Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
6/27/22	7:30:00	0.0	0.0	0.0942
6/27/22	7:30:06	0.1	0.3	0.0956
6/27/22	7:30:15	0.2	0.5	0.0958
6/27/22	7:30:30	0.5	0.7	0.0959
6/27/22	7:31:00	1.0	1.0	0.0961
6/27/22	7:32:00	2.0	1.4	0.0962
6/27/22	7:34:00	4.0	2.0	0.0963
6/27/22	7:38:00	8.0	2.8	0.0964
6/27/22	7:45:00	15.0	3.9	0.0964
6/27/22	8:00:00	30.0	5.5	0.0965
6/27/22	8:30:00	60.0	7.7	0.0966
6/27/22	9:30:00	120.0	11.0	0.0967
6/27/22	11:30:00	240.0	15.5	0.0967
6/27/22	16:00:00	510.0	22.6	0.0968
6/28/22	7:30:00	1440.0	37.9	0.0969

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
LP-2	**B-1**	**0-5**	**7.7**	**9.7**	**122.8**	**123.7**	**0.373**	**0.361**	**55**	**72**

Soil Identification: Olive brown silty sand (SM)

Leighton	**ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS** **ASTM D 2435**	Project No.: 12882.002 A-Town Parcels C & D 07-22

Leighton

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name:	A-Town Parcels C & D	Tested By :	GEB/JD	Date:	06/28/22
Project No. :	12882.002	Checked By:	J. Ward	Date:	07/27/22

Boring No.	LB-1	LP-2		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Olive brown (SM)g	Olive brown SM		
Wet Weight of Soil + Container (g)	0.00	94.10		
Dry Weight of Soil + Container (g)	0.00	92.75		
Weight of Container (g)	1.00	37.02		
Moisture Content (%)	0.00	2.42		
Weight of Soaked Soil (g)	100.20	100.60		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	95	5		
Crucible No.	3	6		
Furnace Temperature (°C)	860	860		
Time In / Time Out	7:30/8:15	7:30/8:15		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	24.5192	25.7397		
Wt. of Crucible (g)	24.5136	25.7355		
Wt. of Residue (g) (A)	0.0056	0.0042		
PPM of Sulfate (A) x 41150	230.44	172.83		
PPM of Sulfate, Dry Weight Basis	**230**	**177**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.4	0.5		
PPM of Chloride (C -0.2) * 100 * 30 / B	40	60		
PPM of Chloride, Dry Wt. Basis	**40**	**61**		

pH TEST, DOT California Test 643

pH Value	**7.88**	**8.30**		
Temperature °C	20.5	20.7		

SOIL RESISTIVITY TEST
DOT CA TEST 643

Project Name:	A-Town Parcels C & D	Tested By :	J. Domingo Date: 07/15/22
Project No. :	12882.002	Checked By:	J. Ward Date: 07/27/22
Boring No.:	LB-1	Depth (ft.) :	0-5
Sample No. :	B-1		

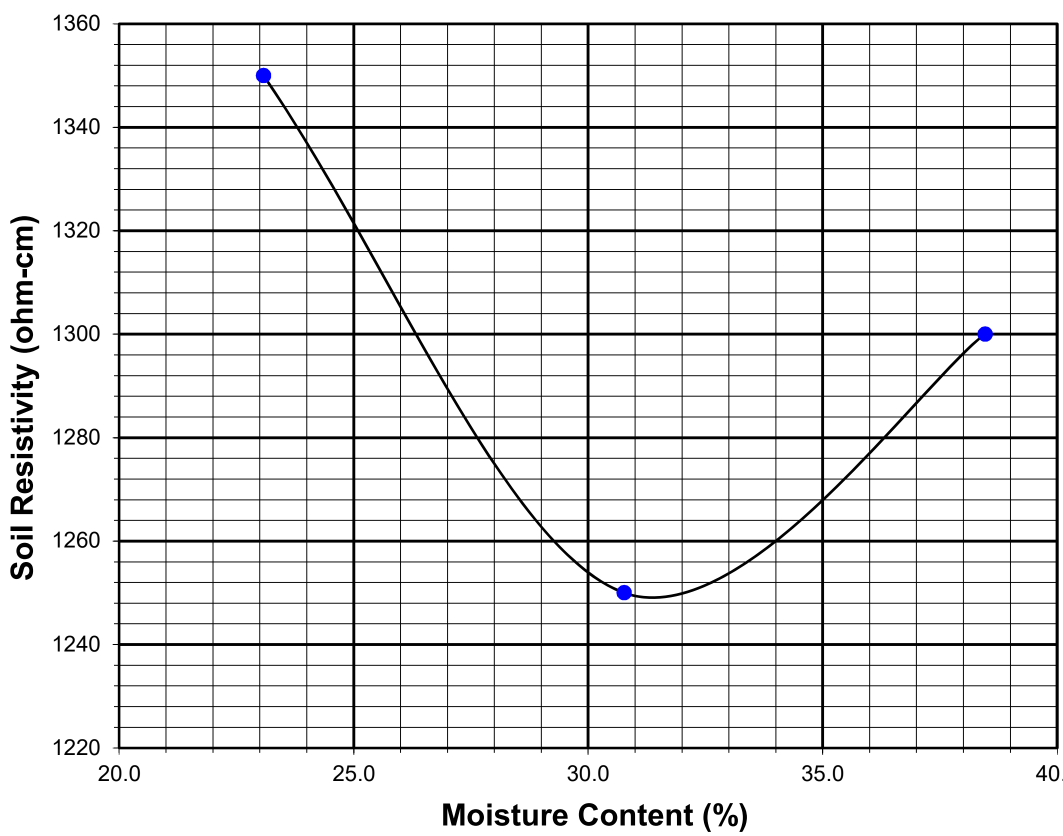
Soil Identification:* Olive brown (SM)g

*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	23.08	1350	1350
2	40	30.77	1250	1250
3	50	38.46	1300	1300
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
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	
1249	**31.4**	**230**	**40**	**7.88**	**20.5**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	A-Town Parcels C & D	Tested By :	J. Domingo Date: 07/15/22
Project No. :	12882.002	Checked By:	J. Ward Date: 07/27/22
Boring No.:	LP-2	Depth (ft.) :	0-5
Sample No. :	B-1		

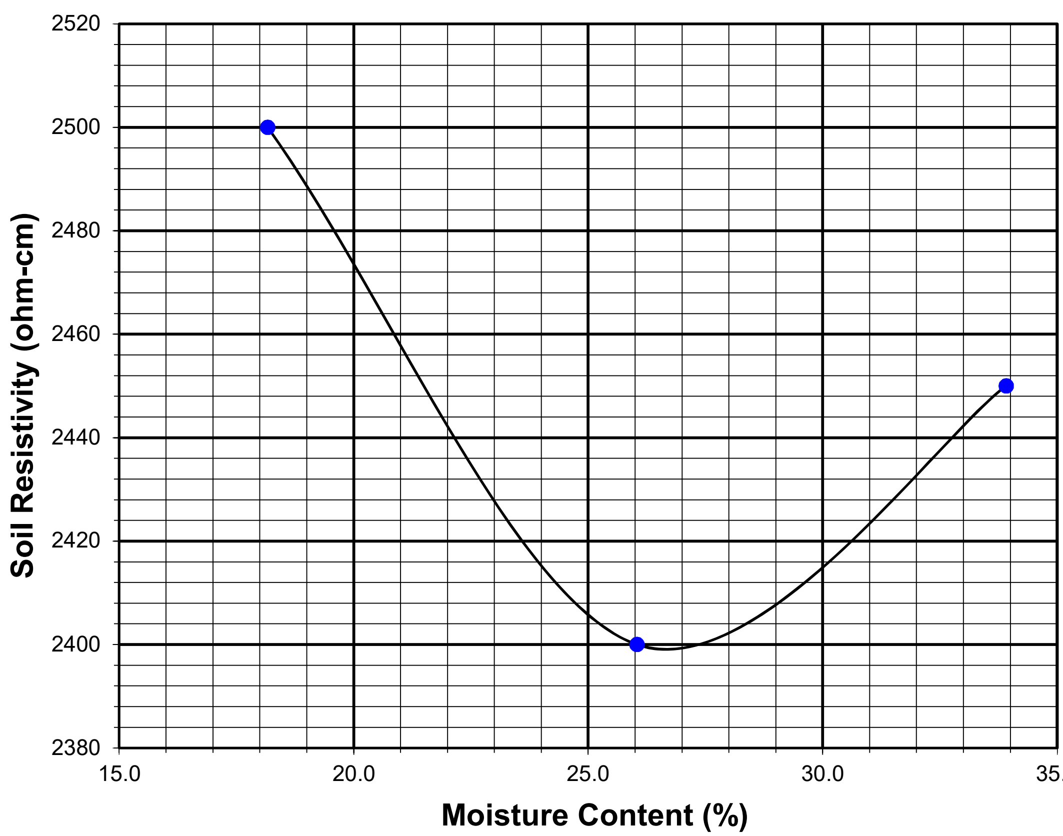
Soil Identification:* Olive brown SM

*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	18.17	2500	2500
2	30	26.04	2400	2400
3	40	33.91	2450	2450
4				
5				

Moisture Content (%) (MCi)	2.42
Wet Wt. of Soil + Cont. (g)	94.10
Dry Wt. of Soil + Cont. (g)	92.75
Wt. of Container (g)	37.02
Container No.	
Initial Soil Wt. (g) (Wt)	130.10
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	
2399	**26.7**	**177**	**61**	**8.30**	**20.7**



DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: A-Town Parcels C & D | Tested By: G. Bathala | Date: 06/22/22

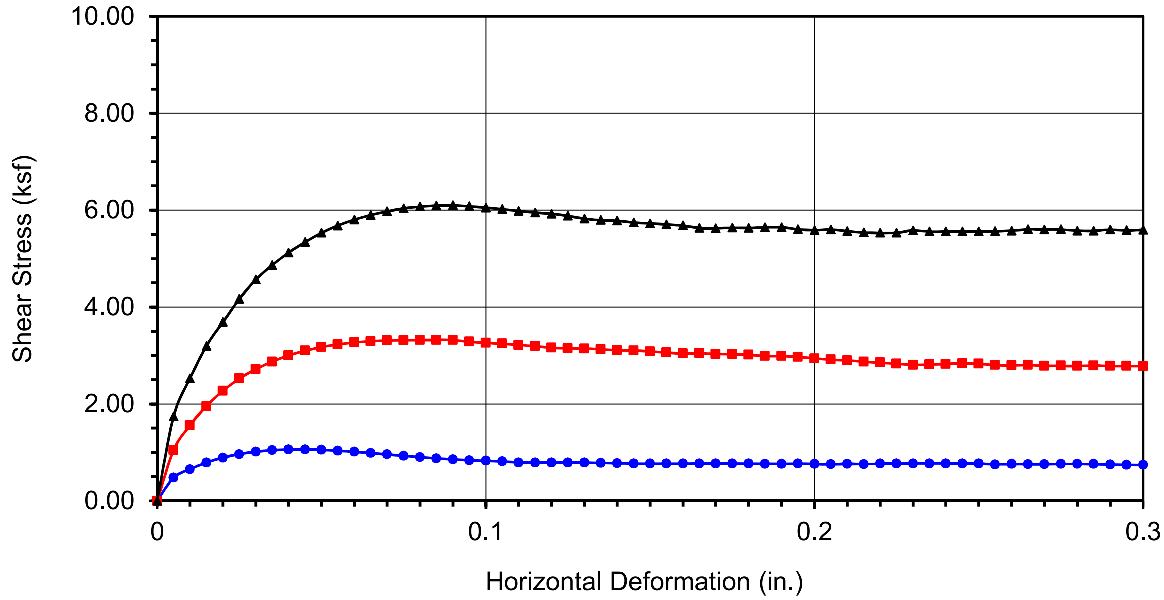
Project No.: 12882.002 | Checked By: J. Ward | Date: 07/27/22

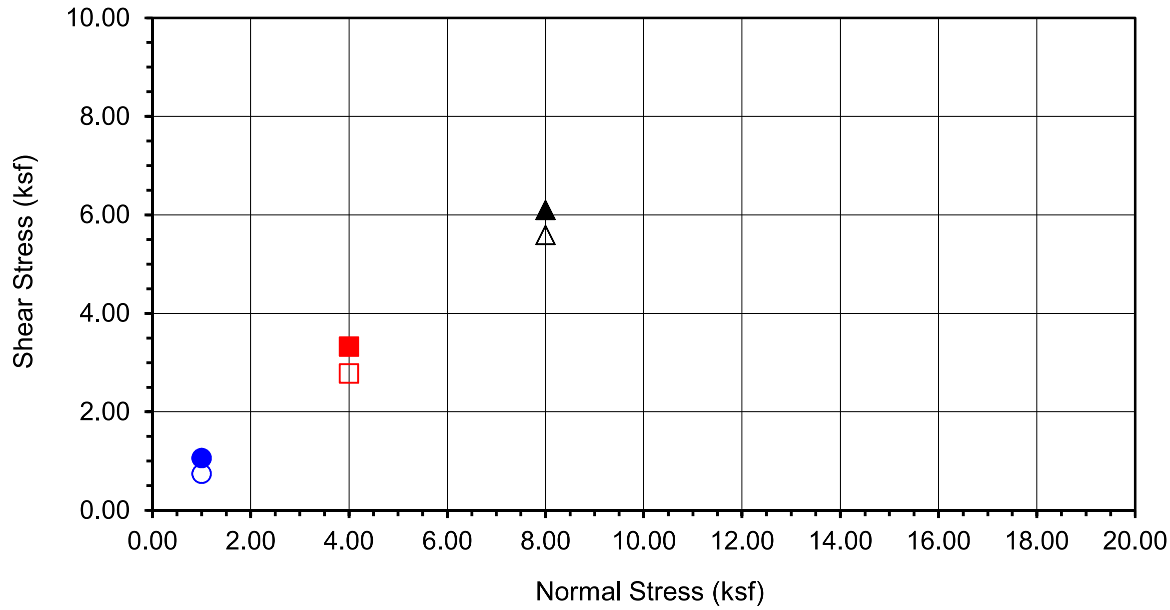
Boring No.: LB-1 | Sample Type: 95% Remold

Sample No.: B-1 | Depth (ft.): 0-5

Soil Identification: Olive brown silty sand with gravel (SM)g

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	205.65	205.67	205.85
Weight of Ring(gm):	45.44	45.40	45.46
Before Shearing			
Weight of Wet Sample+Cont.(gm):	176.60	176.60	176.60
Weight of Dry Sample+Cont.(gm):	168.43	168.43	168.43
Weight of Container(gm):	67.68	67.68	67.68
Vertical Rdg.(in): Initial	0.0000	0.2563	0.2568
Vertical Rdg.(in): Final	-0.0073	0.2717	0.2740
After Shearing			
Weight of Wet Sample+Cont.(gm):	202.43	230.22	214.69
Weight of Dry Sample+Cont.(gm):	185.78	214.07	199.27
Weight of Container(gm):	39.56	67.68	52.64
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0-5**

<u>Sample Type:</u>

95% Remold

<u>Soil Identification:</u>
Olive brown silty sand with gravel (SM)g

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.059	■ 3.320	▲ 6.099
Shear Stress @ End of Test (ksf)	○ 0.742	□ 2.779	△ 5.593
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 (%)	8.11	8.11	8.11
Dry Density (pcf)	123.2	123.3	123.4
Saturation (%)	59.5	59.6	59.8
Soil Height Before Shearing (in.)	0.9927	0.9846	0.9828
Final Moisture Content (%)	11.4	11.0	10.5

Leighton

DIRECT SHEAR TEST RESULTS

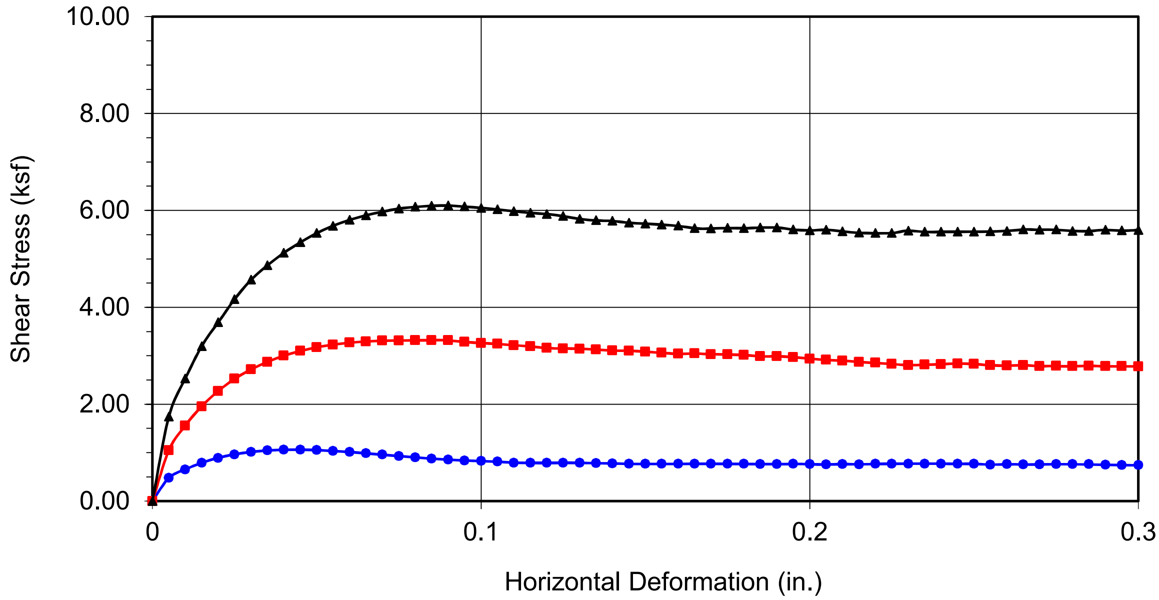
Consolidated Drained - ASTM D 3080

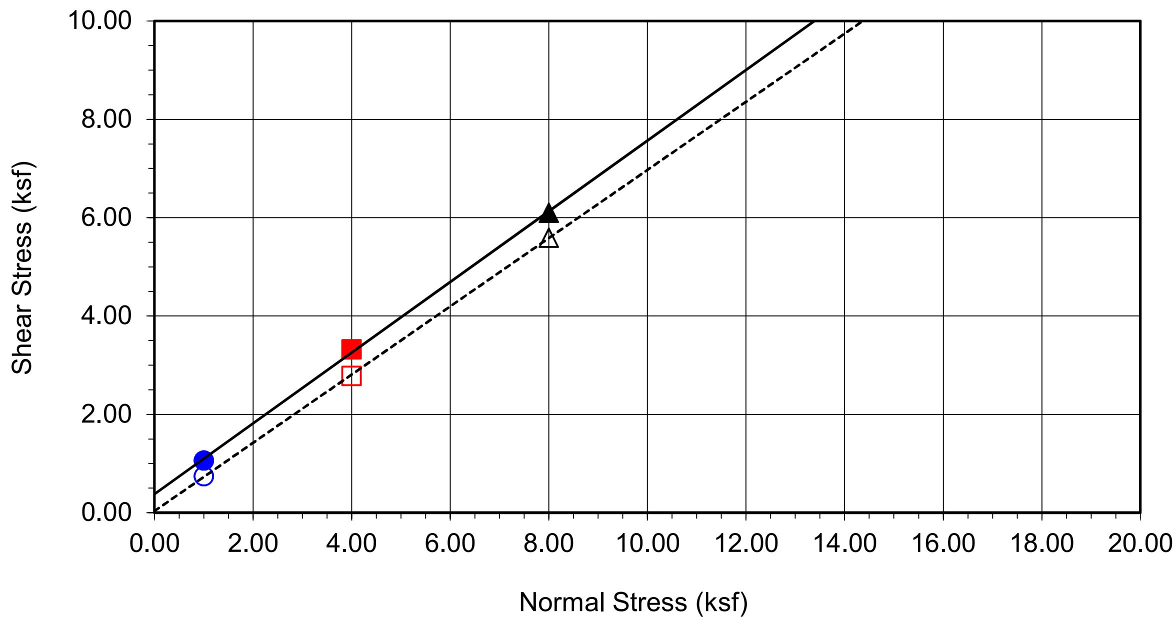
Project No.: 12882.002

A-Town Parcels C & D

06-22

DS LB-1, B-1 @ 0-5





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	95% Remold
Soil Identification: Olive brown silty sand with gravel (SM)g	

Strength Parameters

	C (psf)	ϕ (°)
Peak	379	36
Ultimate	33	35

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.059	■ 3.320	▲ 6.099
Shear Stress @ End of Test (ksf)	○ 0.742	□ 2.779	△ 5.593
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 (%)	8.11	8.11	8.11
Dry Density (pcf)	123.2	123.3	123.4
Saturation (%)	59.5	59.6	59.8
Soil Height Before Shearing (in.)	0.9927	0.9846	0.9828
Final Moisture Content (%)	11.4	11.0	10.5

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22

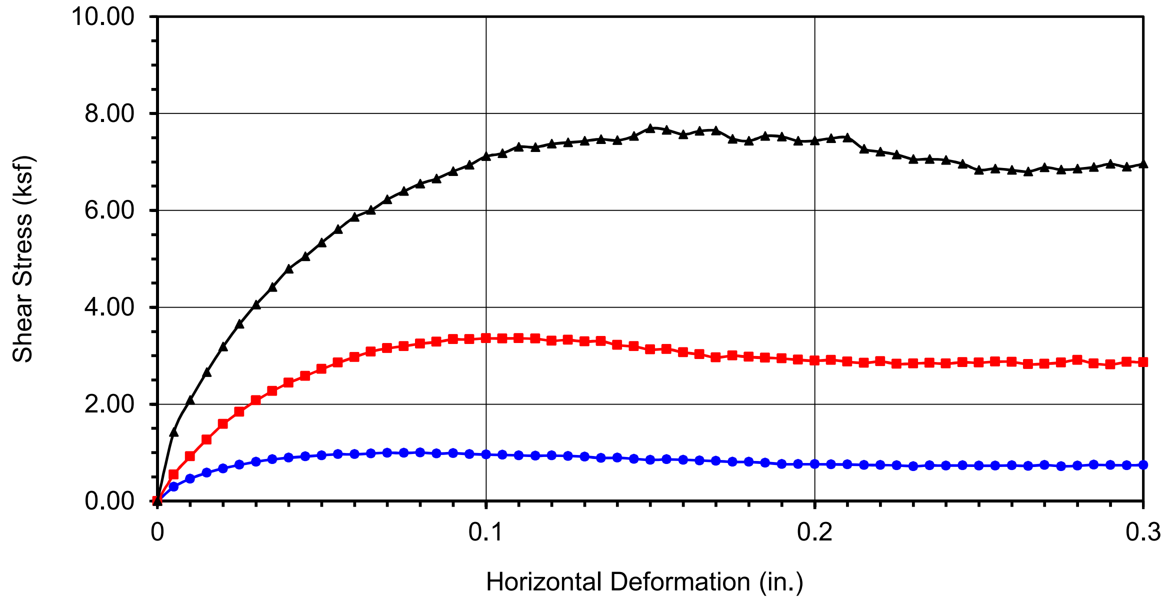
DS LB-1, B-1 @ 0-5

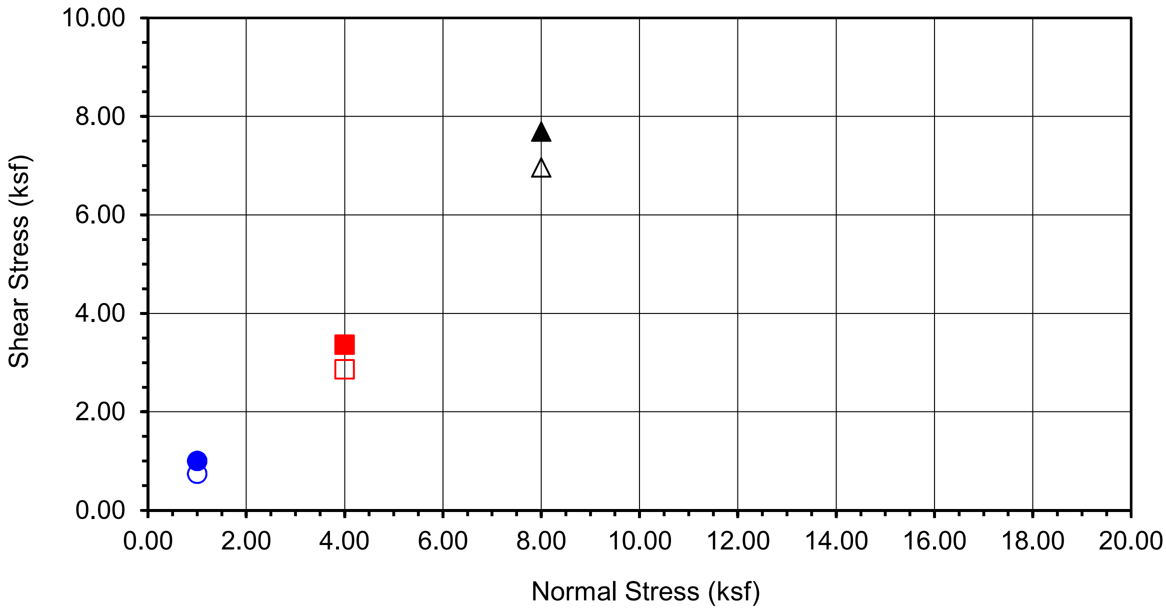
DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: A-Town Parcels C & D
Project No.: 12882.002
Boring No.: LP-1
Sample No.: R-1
Soil Identification: Light yellowish brown poorly-graded sand (SP)

Tested By: G. Bathala — Date: 06/23/22
Checked By: J. Ward — Date: 07/27/22
Sample Type: Ring
Depth (ft.): 10.0

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	172.36	174.43	177.37
Weight of Ring(gm):	44.09	44.78	43.98
Before Shearing			
Weight of Wet Sample+Cont.(gm):	191.10	191.10	191.10
Weight of Dry Sample+Cont.(gm):	189.91	189.91	189.91
Weight of Container(gm):	58.47	58.47	58.47
Vertical Rdg.(in): Initial	0.0000	0.2356	0.2472
Vertical Rdg.(in): Final	-0.0078	0.2569	0.2783
After Shearing			
Weight of Wet Sample+Cont.(gm):	181.05	213.40	183.35
Weight of Dry Sample+Cont.(gm):	159.85	192.61	165.13
Weight of Container(gm):	38.31	69.45	39.56
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LP-1**
Sample No.	**R-1**
Depth (ft)	**10**

Sample Type:

Ring

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

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.003	■ 3.361	▲ 7.690
Shear Stress @ End of Test (ksf)	○ 0.745	□ 2.861	△ 6.963
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 (%)	0.91	0.91	0.91
Dry Density (pcf)	105.7	106.9	109.9
Saturation (%)	4.1	4.2	4.6
Soil Height Before Shearing (in.)	0.9922	0.9787	0.9689
Final Moisture Content (%)	17.4	16.9	14.5

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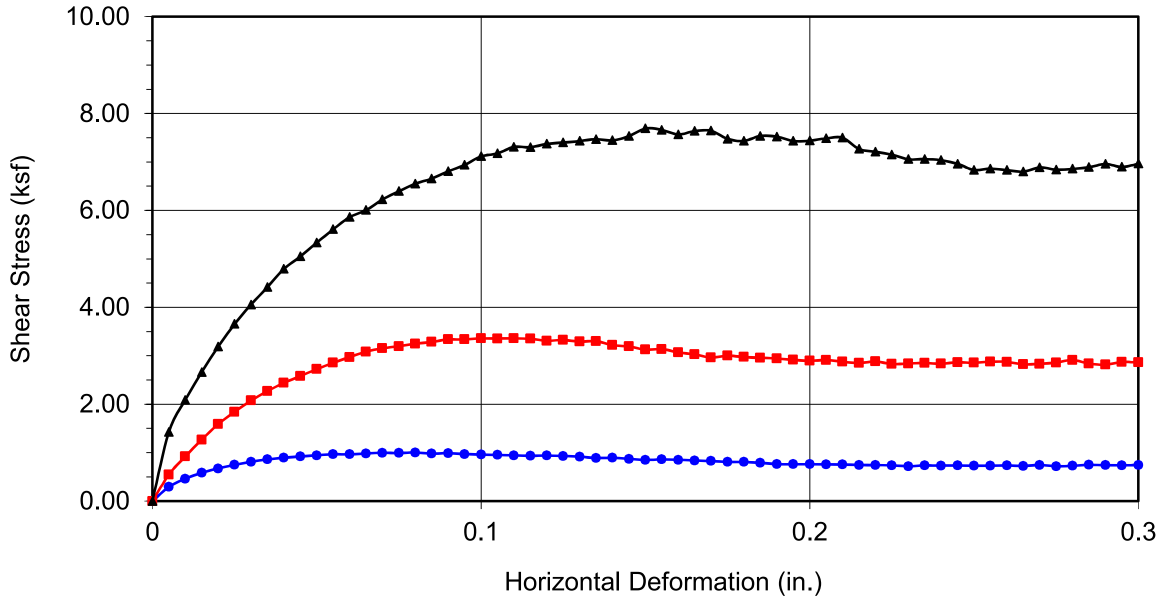
DIRECT SHEAR TEST RESULTS

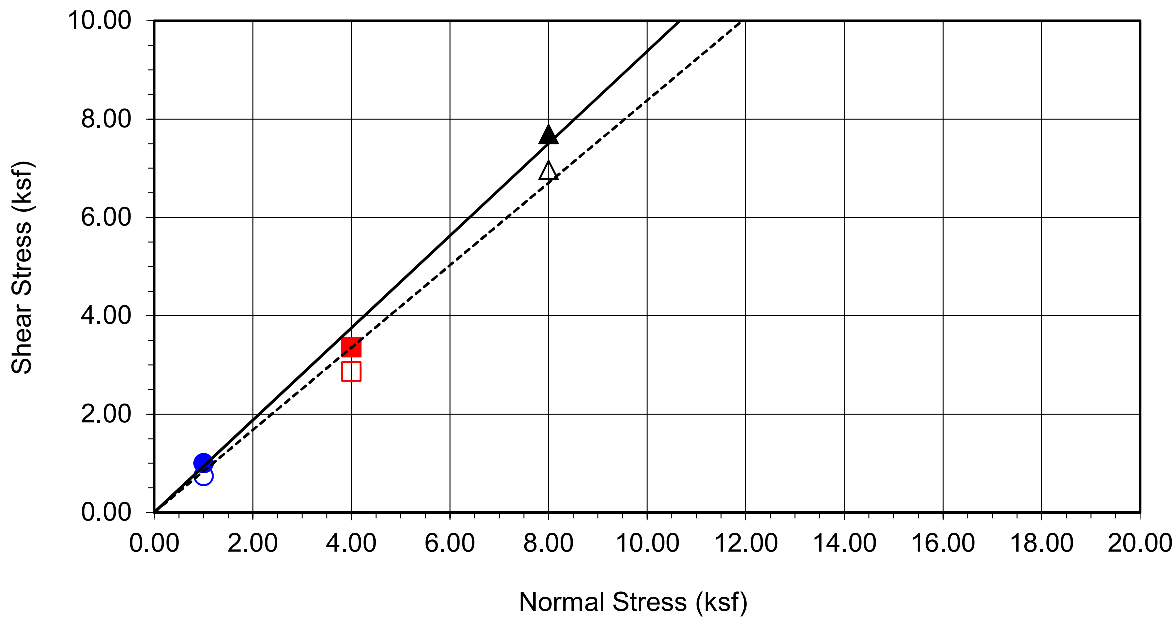
Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22





Boring No.	**LP-1**
Sample No.	**R-1**
Depth (ft)	**10**
Sample Type:	Ring

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

Strength Parameters

	C (psf)	ϕ (°)
Peak	0	43
Ultimate	0	40

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.003	■ 3.361	▲ 7.690
Shear Stress @ End of Test (ksf)	○ 0.745	□ 2.861	△ 6.963
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 (%)	0.91	0.91	0.91
Dry Density (pcf)	105.7	106.9	109.9
Saturation (%)	4.1	4.2	4.6
Soil Height Before Shearing (in.)	0.9922	0.9787	0.9689
Final Moisture Content (%)	17.4	16.9	14.5

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22

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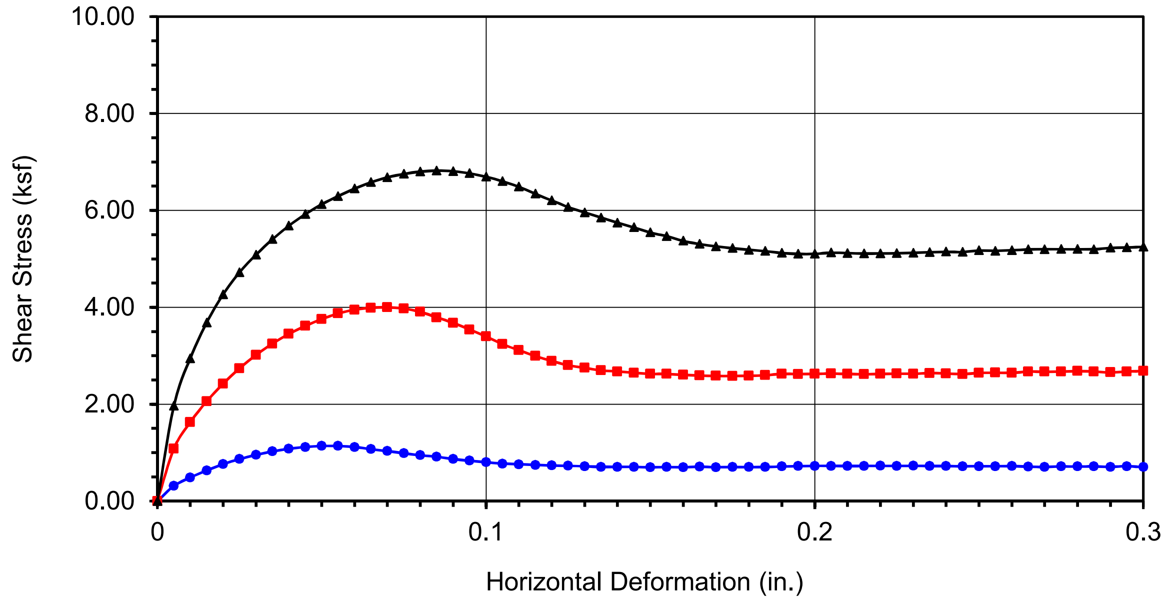
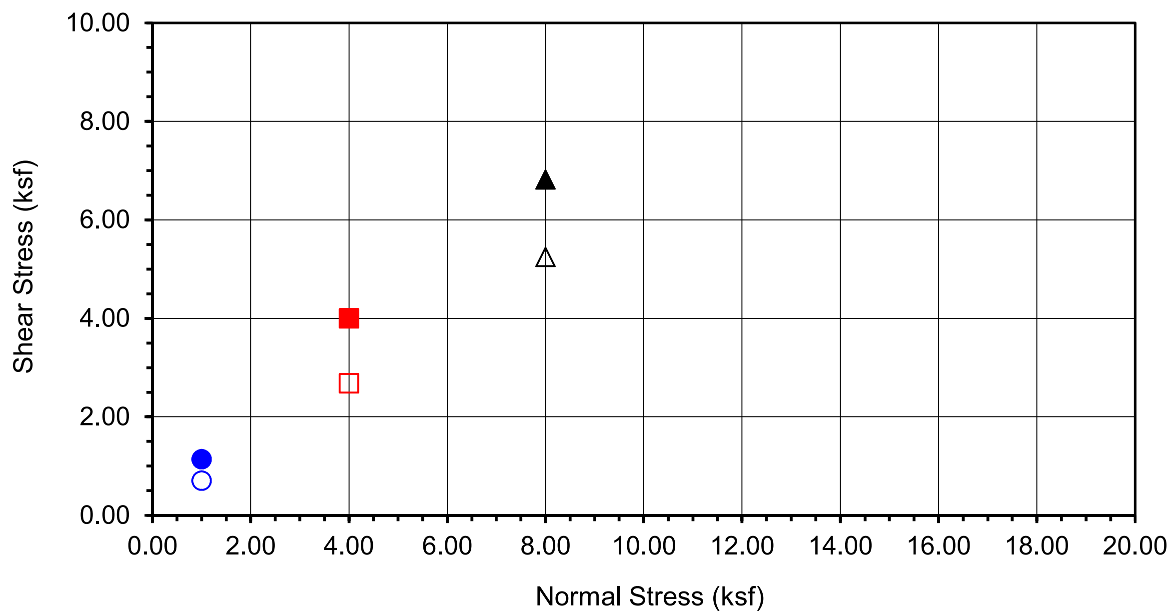
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	A-Town Parcels C & D	Tested By:	G. Bathala	Date:	06/29/22
Project No.:	12882.002	Checked By:	J. Ward	Date:	07/27/22
Boring No.:	LP-1	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.):	30.0		
Soil Identification:	Light olive brown silty sand (SM)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	180.47	187.98	193.02
Weight of Ring(gm):	40.17	45.14	42.42
Before Shearing			
Weight of Wet Sample+Cont.(gm):	193.05	193.05	193.05
Weight of Dry Sample+Cont.(gm):	185.10	185.10	185.10
Weight of Container(gm):	66.07	66.07	66.07
Vertical Rdg.(in): Initial	0.2553	0.2470	0.0000
Vertical Rdg.(in): Final	0.2647	0.2638	-0.0283
After Shearing			
Weight of Wet Sample+Cont.(gm):	199.21	207.28	220.95
Weight of Dry Sample+Cont.(gm):	179.24	186.70	201.28
Weight of Container(gm):	52.66	55.82	66.12
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LP-1**
Sample No.	**R-3**
Depth (ft)	**30**

Sample Type:

Ring

Soil Identification:
Light olive brown silty sand (SM)

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.138	■ 3.999	▲ 6.819
Shear Stress @ End of Test (ksf)	○ 0.701	□ 2.682	△ 5.247
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 (%)	6.68	6.68	6.68
Dry Density (pcf)	109.4	111.4	117.4
Saturation (%)	33.3	35.1	41.4
Soil Height Before Shearing (in.)	0.9906	0.9832	0.9717
Final Moisture Content (%)	15.8	15.7	14.6

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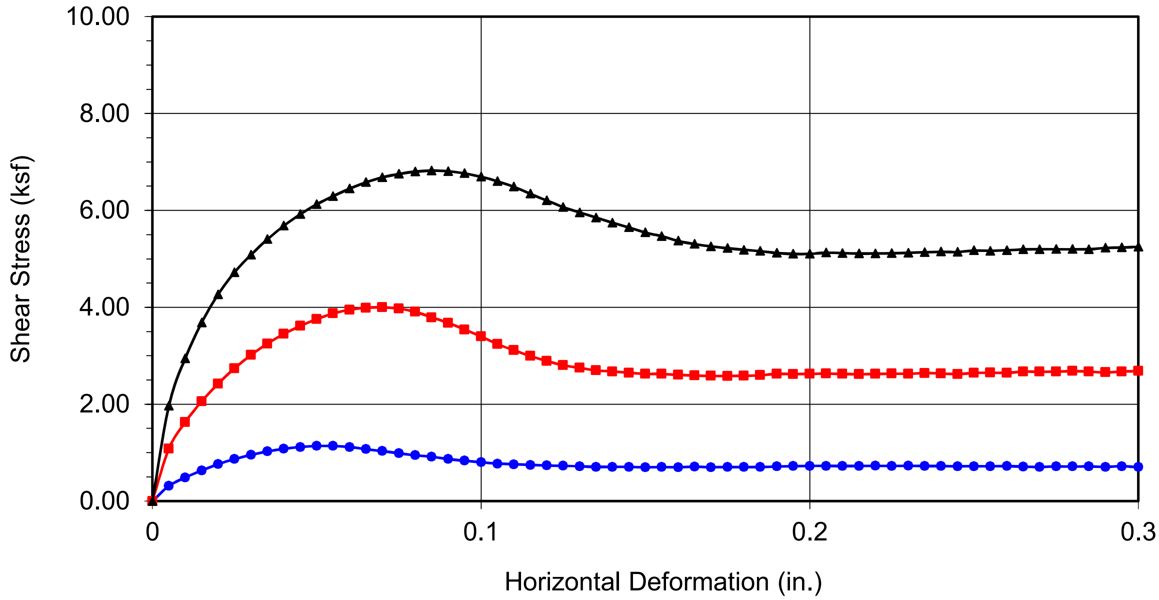
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

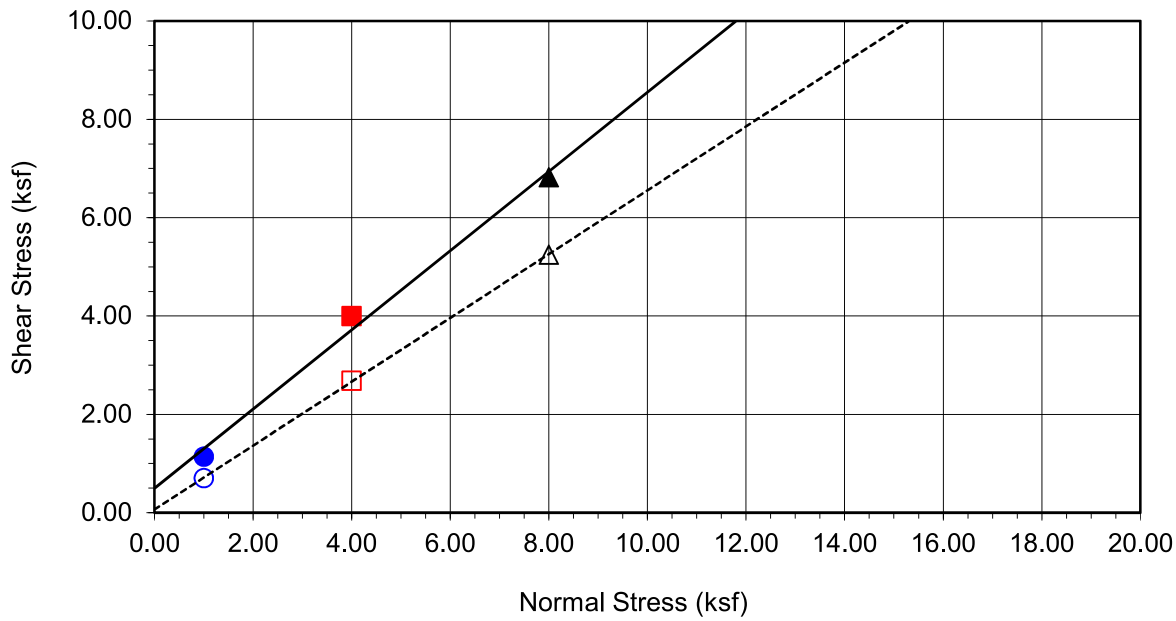
Project No.: 12882.002

A-Town Parcels C & D

06-22

DS LP-1, R-3 @ 30





Boring No.	**LP-1**
Sample No.	**R-3**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:
Light olive brown silty sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	493	39
Ultimate	64	33

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.138	■ 3.999	▲ 6.819
Shear Stress @ End of Test (ksf)	○ 0.701	□ 2.682	△ 5.247
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 (%)	6.68	6.68	6.68
Dry Density (pcf)	109.4	111.4	117.4
Saturation (%)	33.3	35.1	41.4
Soil Height Before Shearing (in.)	0.9906	0.9832	0.9717
Final Moisture Content (%)	15.8	15.7	14.6

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

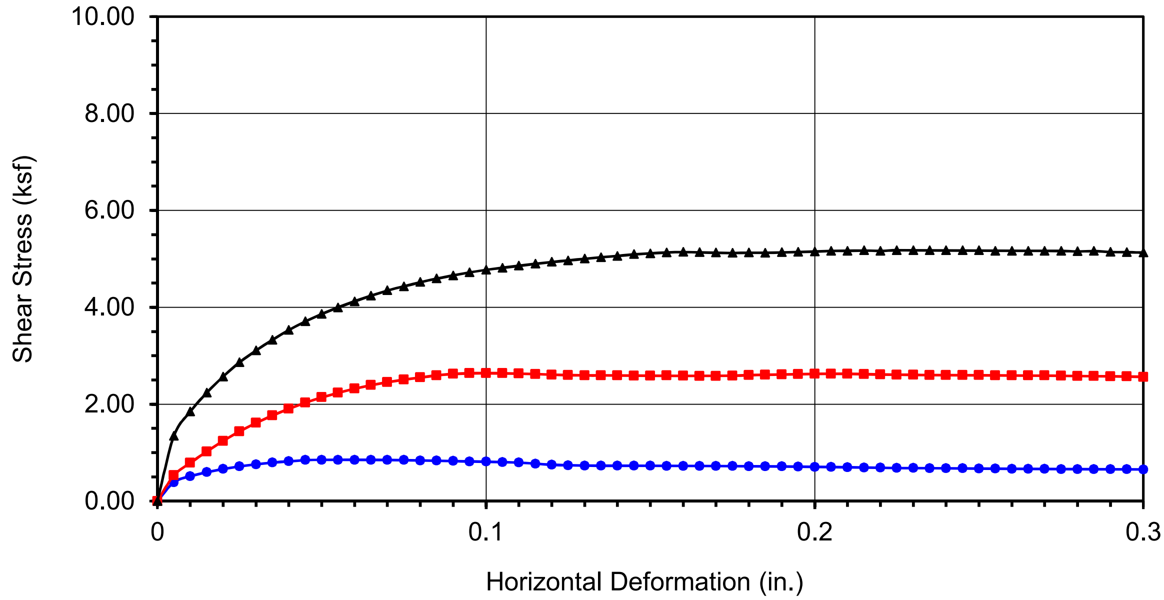
06-22

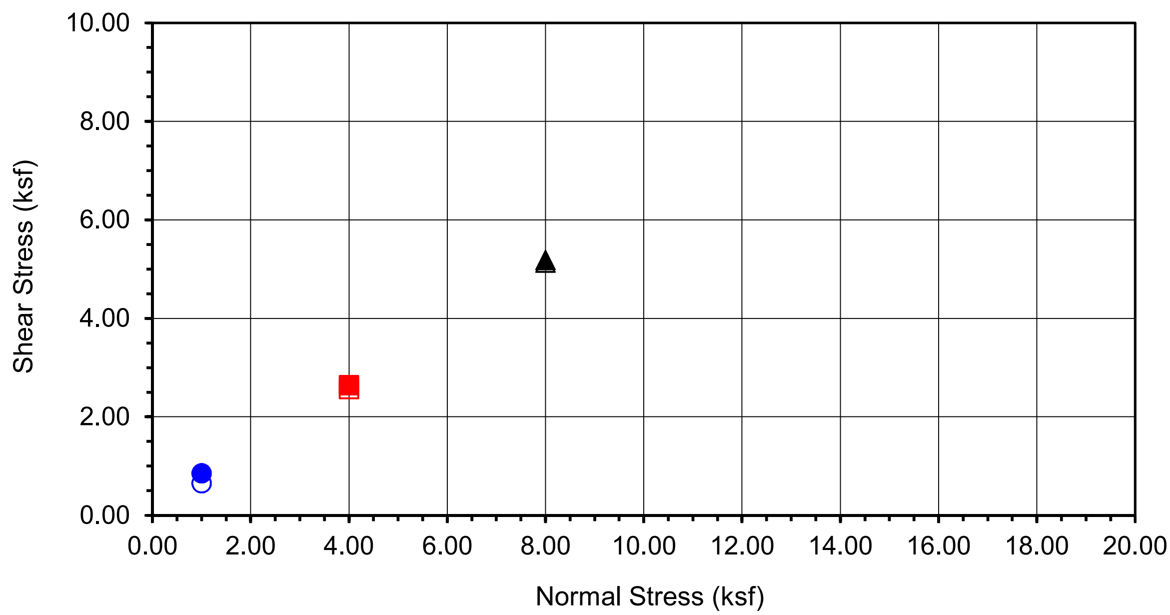
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	A-Town Parcels C & D	Tested By:	G. Bathala	Date:	06/29/22
Project No.:	12882.002	Checked By:	J. Ward	Date:	07/27/22
Boring No.:	LP-1	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.):	40.0		
Soil Identification:	Olive silt (ML)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	166.78	169.07	174.74
Weight of Ring(gm):	43.36	45.51	46.71
Before Shearing			
Weight of Wet Sample+Cont.(gm):	176.52	176.52	176.52
Weight of Dry Sample+Cont.(gm):	166.81	166.81	166.81
Weight of Container(gm):	67.69	67.69	67.69
Vertical Rdg.(in): Initial	0.0000	0.2381	0.2646
Vertical Rdg.(in): Final	-0.0094	0.2690	0.3011
After Shearing			
Weight of Wet Sample+Cont.(gm):	176.71	197.77	191.71
Weight of Dry Sample+Cont.(gm):	147.21	167.14	162.59
Weight of Container(gm):	37.73	59.52	52.54
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LP-1**
Sample No.	**R-4**
Depth (ft)	**40**
Sample Type: Ring	
Soil Identification: Olive silt (ML)	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.852	■ 2.638	▲ 5.178
Shear Stress @ End of Test (ksf)	○ 0.651	□ 2.562	△ 5.127
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.80	9.80	9.80
Dry Density (pcf)	93.5	93.6	97.0
Saturation (%)	32.9	33.0	35.8
Soil Height Before Shearing (in.)	0.9906	0.9691	0.9635
Final Moisture Content (%)	26.9	28.5	26.5

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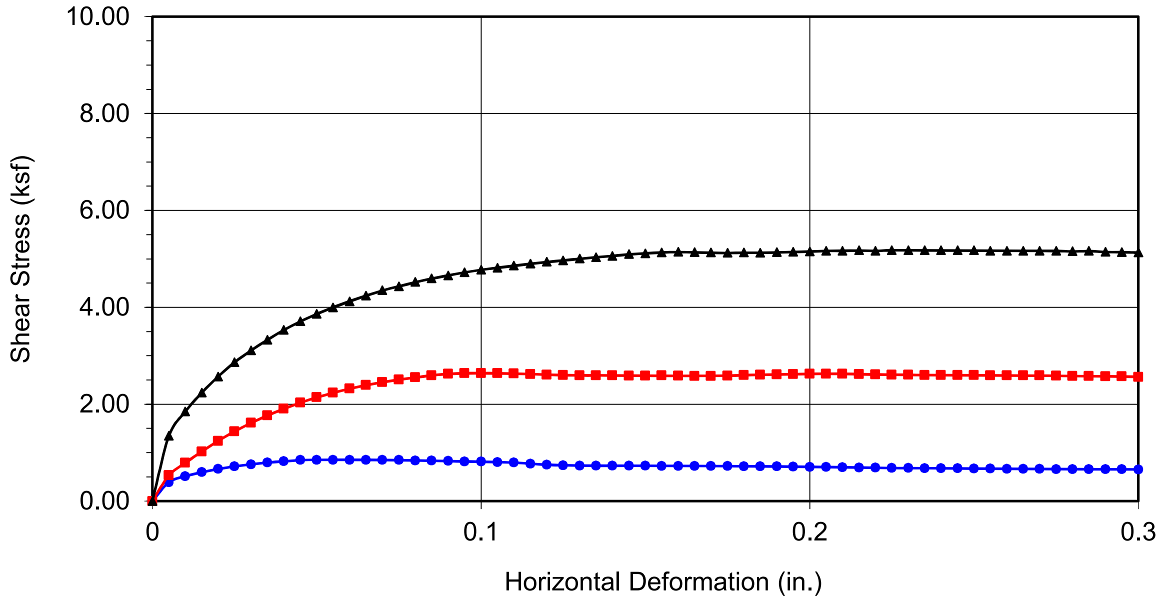
DIRECT SHEAR TEST RESULTS

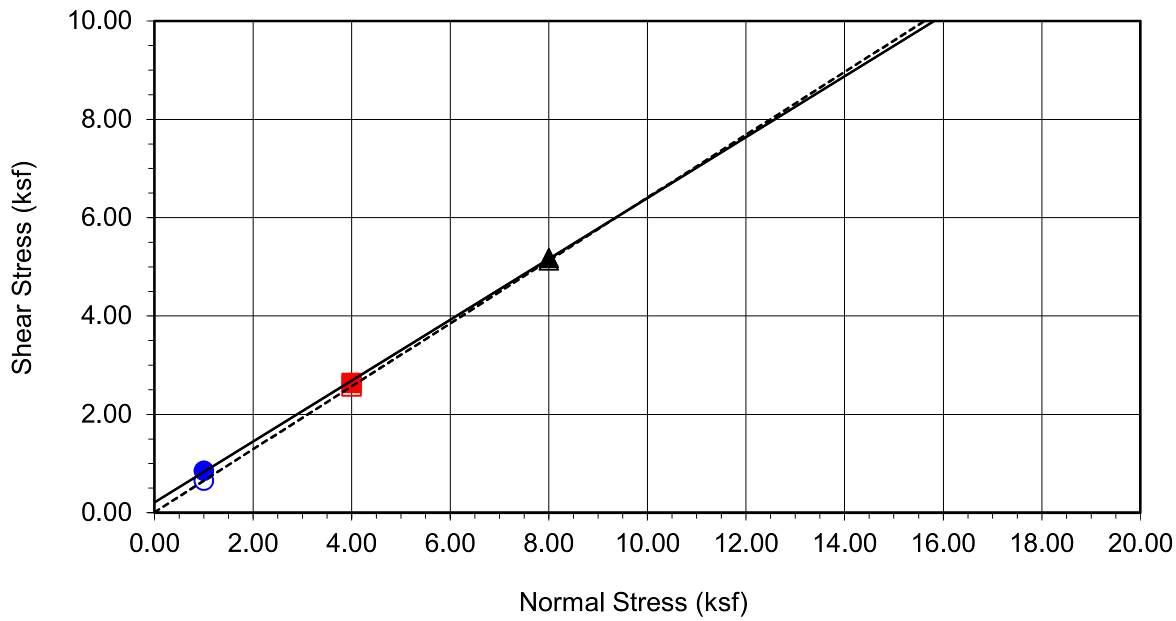
Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22





Boring No.	**LP-1**
Sample No.	**R-4**
Depth (ft)	**40**
Sample Type:	Ring
Soil Identification: Olive silt (ML)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	207	32
Ultimate	9	33

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.852	■ 2.638	▲ 5.178
Shear Stress @ End of Test (ksf)	○ 0.651	□ 2.562	△ 5.127
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.80	9.80	9.80
Dry Density (pcf)	93.5	93.6	97.0
Saturation (%)	32.9	33.0	35.8
Soil Height Before Shearing (in.)	0.9906	0.9691	0.9635
Final Moisture Content (%)	26.9	28.5	26.5

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22

DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: A-Town Parcels C & D
Tested By: G. Bathala
Date: 06/22/22

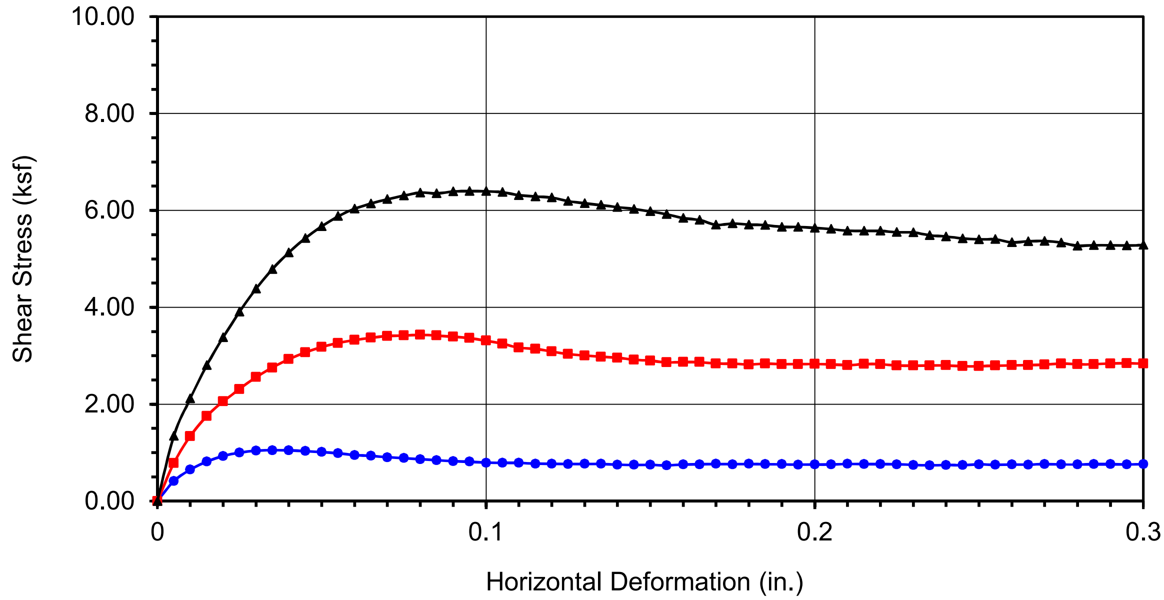
Project No.: 12882.002
Checked By: J. Ward
Date: 07/27/22

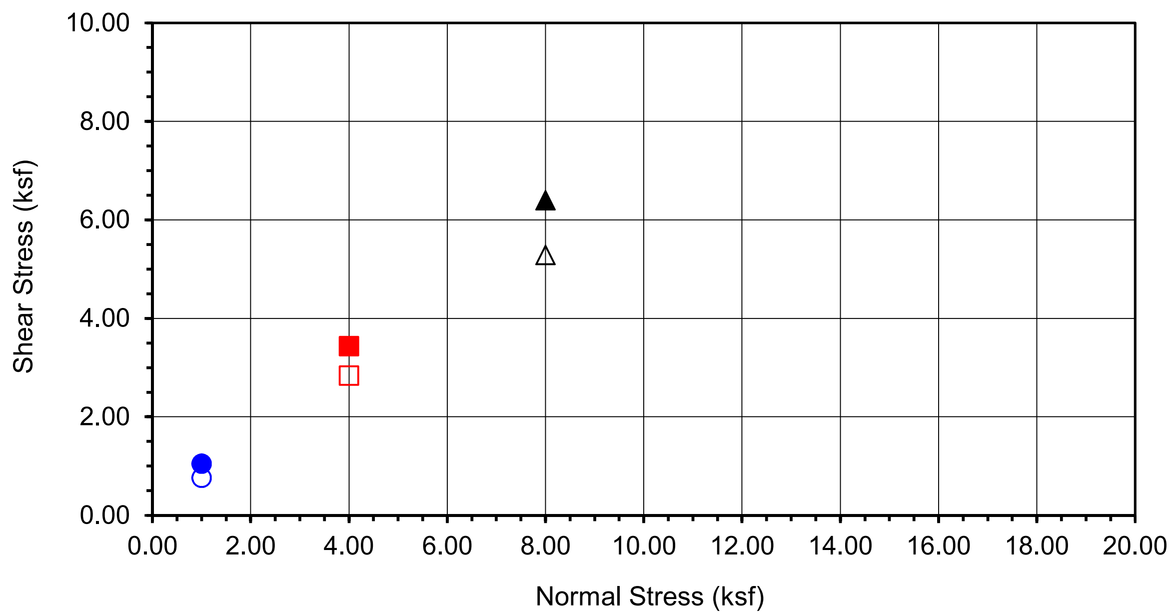
Boring No.: LP-2
Sample Type: 95% Remold

Sample No.: B-1
Depth (ft.): 0-5

Soil Identification: Olive brown silty sand (SM)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	204.55	204.75	204.82
Weight of Ring(gm):	45.45	45.47	45.40
Before Shearing			
Weight of Wet Sample+Cont.(gm):	172.37	172.37	172.37
Weight of Dry Sample+Cont.(gm):	163.86	163.86	163.86
Weight of Container(gm):	52.64	52.64	52.64
Vertical Rdg.(in): Initial	0.2316	0.2538	0.0000
Vertical Rdg.(in): Final	0.2378	0.2795	-0.0284
After Shearing			
Weight of Wet Sample+Cont.(gm):	230.03	198.13	217.14
Weight of Dry Sample+Cont.(gm):	213.89	182.78	202.03
Weight of Container(gm):	69.45	38.30	57.28
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LP-2**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type: 95% Remold	
Soil Identification: Olive brown silty sand (SM)	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.050	■ 3.430	▲ 6.398
Shear Stress @ End of Test (ksf)	○ 0.764	□ 2.836	△ 5.285
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 (%)	7.65	7.65	7.65
Dry Density (pcf)	122.9	123.1	123.2
Saturation (%)	55.6	55.9	56.0
Soil Height Before Shearing (in.)	0.9938	0.9743	0.9716
Final Moisture Content (%)	11.2	10.6	10.4

Leighton

DIRECT SHEAR TEST RESULTS

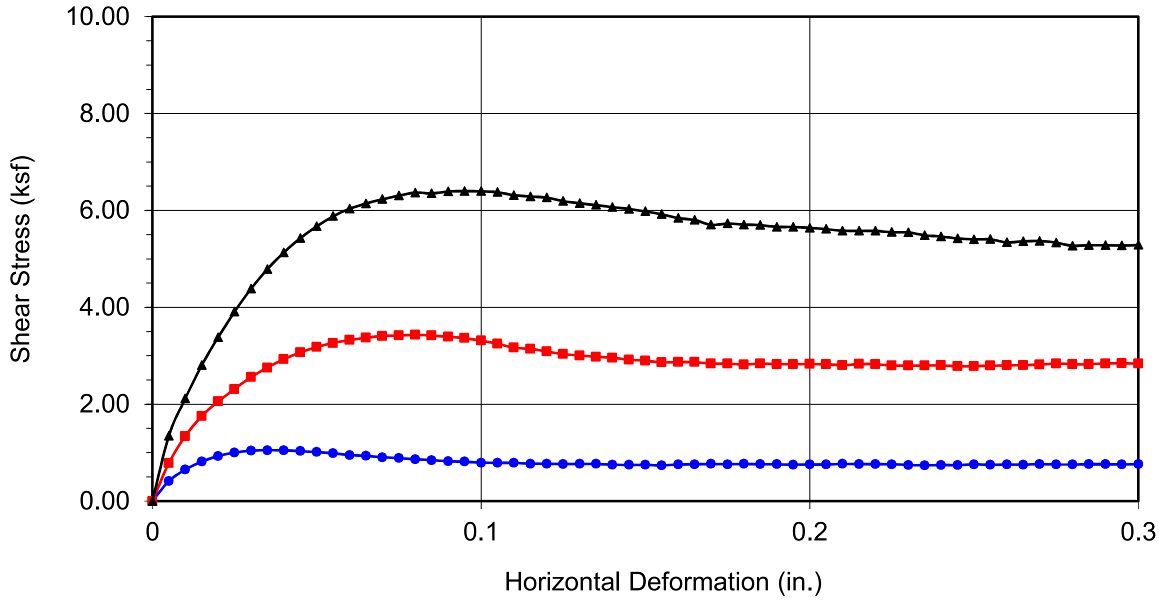
Consolidated Drained - ASTM D 3080

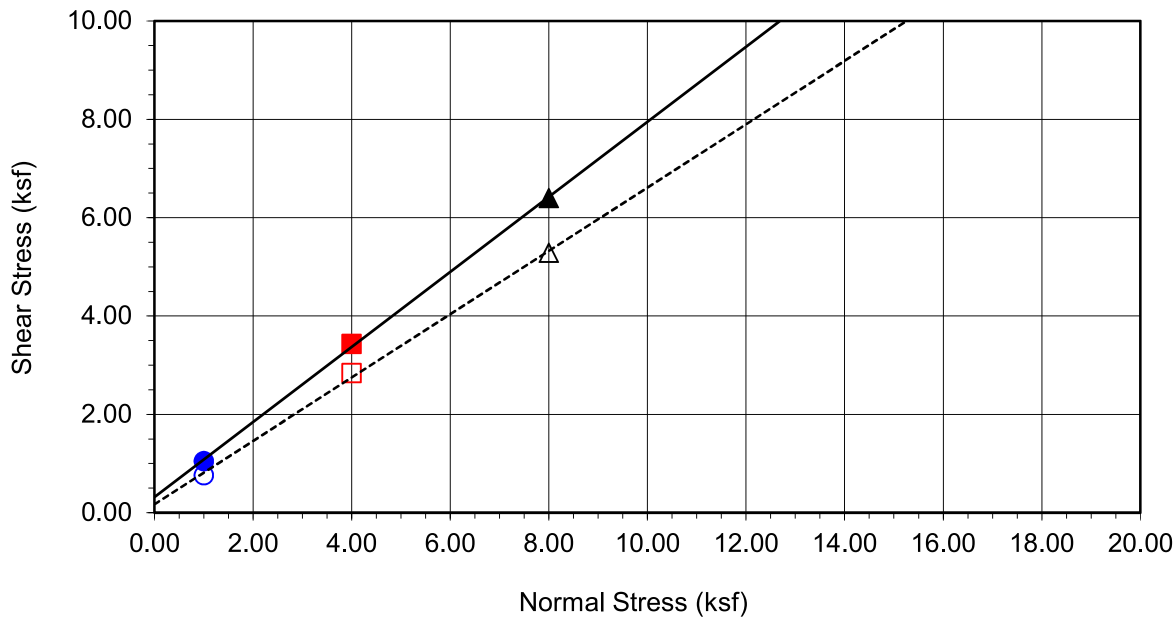
Project No.: 12882.002

A-Town Parcels C & D

06-22

DS LP-2, B-1 @ 0-5





Boring No.	**LP-2**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	95% Remold

Soil Identification:
Olive brown silty sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	320	37
Ultimate	171	33

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.050	■ 3.430	▲ 6.398
Shear Stress @ End of Test (ksf)	○ 0.764	□ 2.836	△ 5.285
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 (%)	7.65	7.65	7.65
Dry Density (pcf)	122.9	123.1	123.2
Saturation (%)	55.6	55.9	56.0
Soil Height Before Shearing (in.)	0.9938	0.9743	0.9716
Final Moisture Content (%)	11.2	10.6	10.4

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22

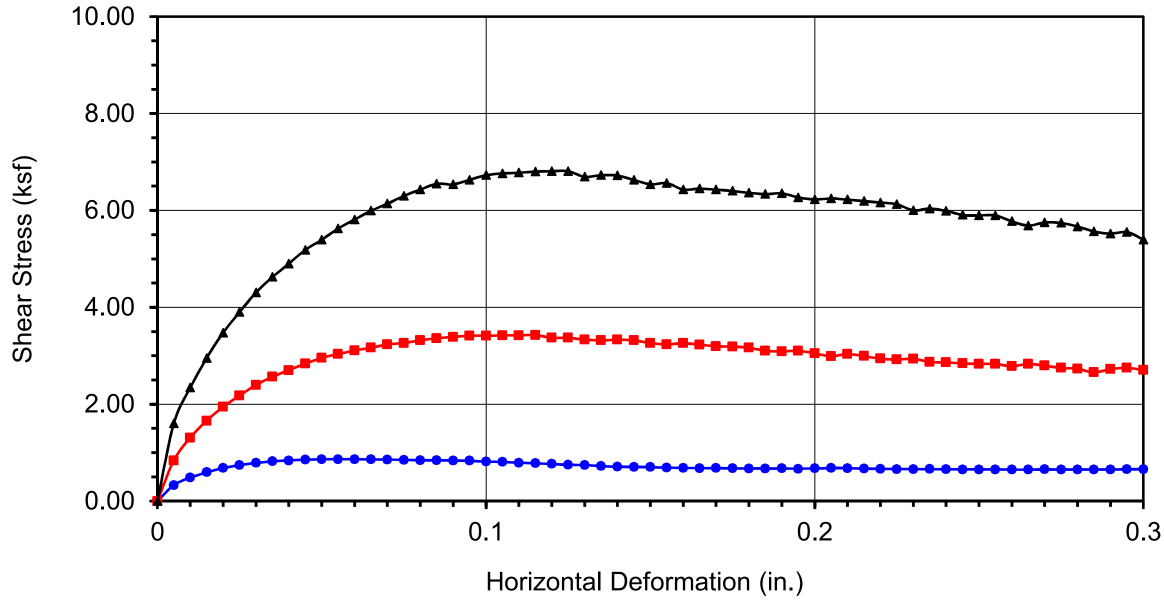
DIRECT SHEAR TEST

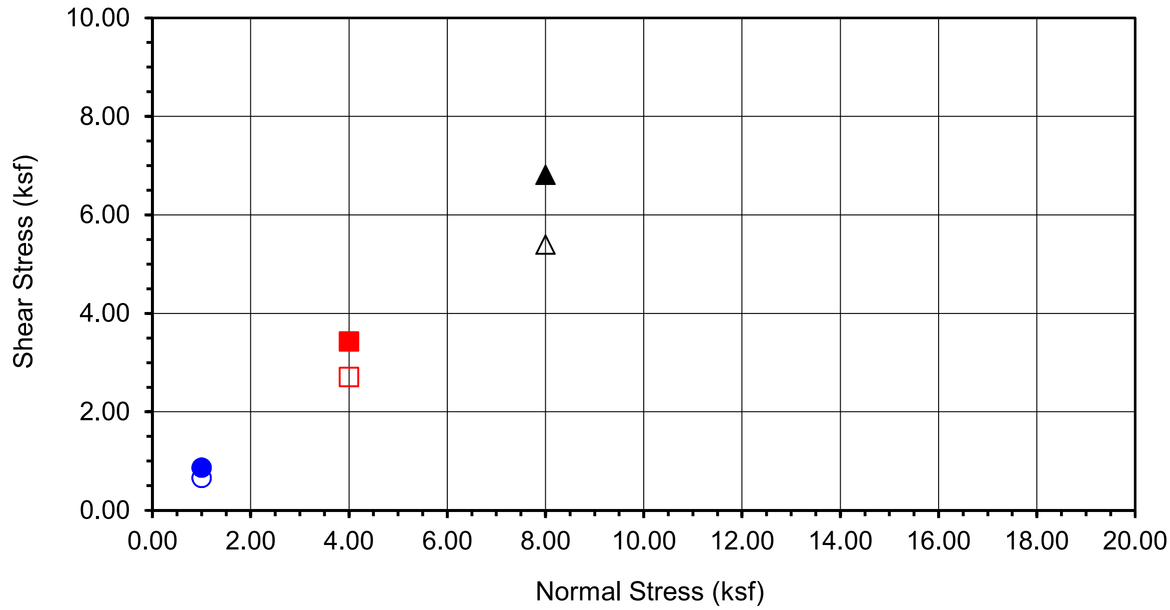
Consolidated Drained - ASTM D 3080

Project Name:	A-Town Parcels C & D	Tested By:	G. Bathala	Date:	06/20/22
Project No.:	12882.002	Checked By:	J. Ward	Date:	07/27/22
Boring No.:	LP-2	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.):	15.0		

Soil Identification: Light yellowish 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):	161.38	169.02	173.12
Weight of Ring(gm):	41.70	43.41	45.52
Before Shearing			
Weight of Wet Sample+Cont.(gm):	193.26	193.26	193.26
Weight of Dry Sample+Cont.(gm):	191.52	191.52	191.52
Weight of Container(gm):	58.27	58.27	58.27
Vertical Rdg.(in): Initial	0.2341	0.2491	0.0000
Vertical Rdg.(in): Final	0.2425	0.2748	-0.0320
After Shearing			
Weight of Wet Sample+Cont.(gm):	188.03	193.25	179.41
Weight of Dry Sample+Cont.(gm):	164.56	173.33	159.23
Weight of Container(gm):	52.53	57.29	39.56
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LP-2**
Sample No.	**R-2**
Depth (ft)	**15**

Sample Type:

Ring

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

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.865	■ 3.427	▲ 6.809
Shear Stress @ End of Test (ksf)	○ 0.657	□ 2.707	△ 5.395
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 (%)	1.31	1.31	1.31
Dry Density (pcf)	98.3	103.1	104.8
Saturation (%)	4.9	5.6	5.8
Soil Height Before Shearing (in.)	0.9916	0.9743	0.9680
Final Moisture Content (%)	20.9	17.2	16.9

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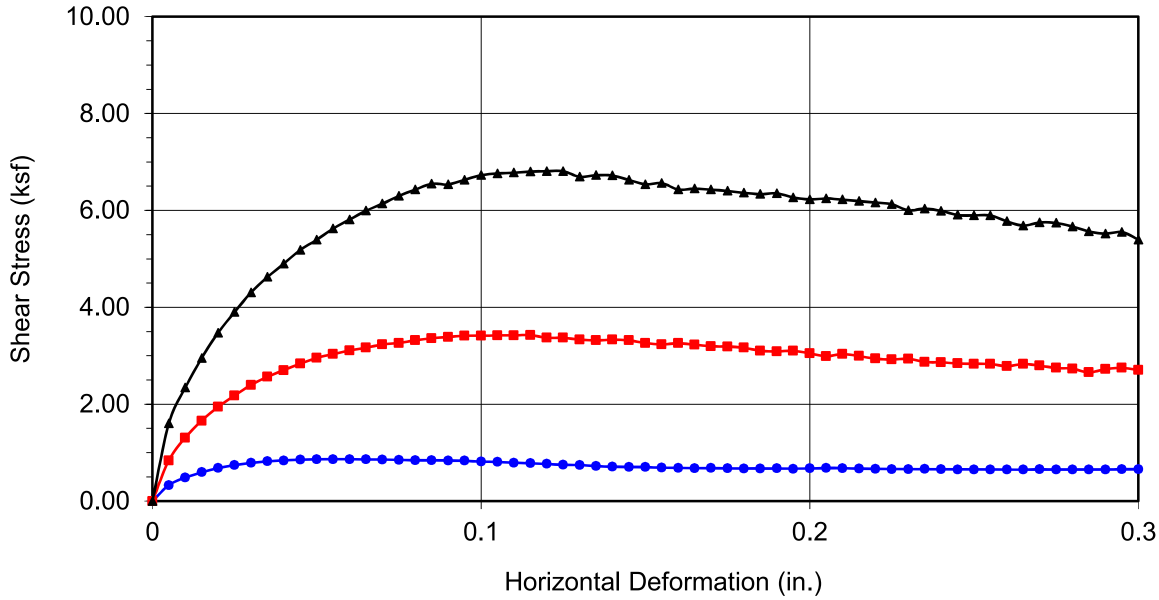
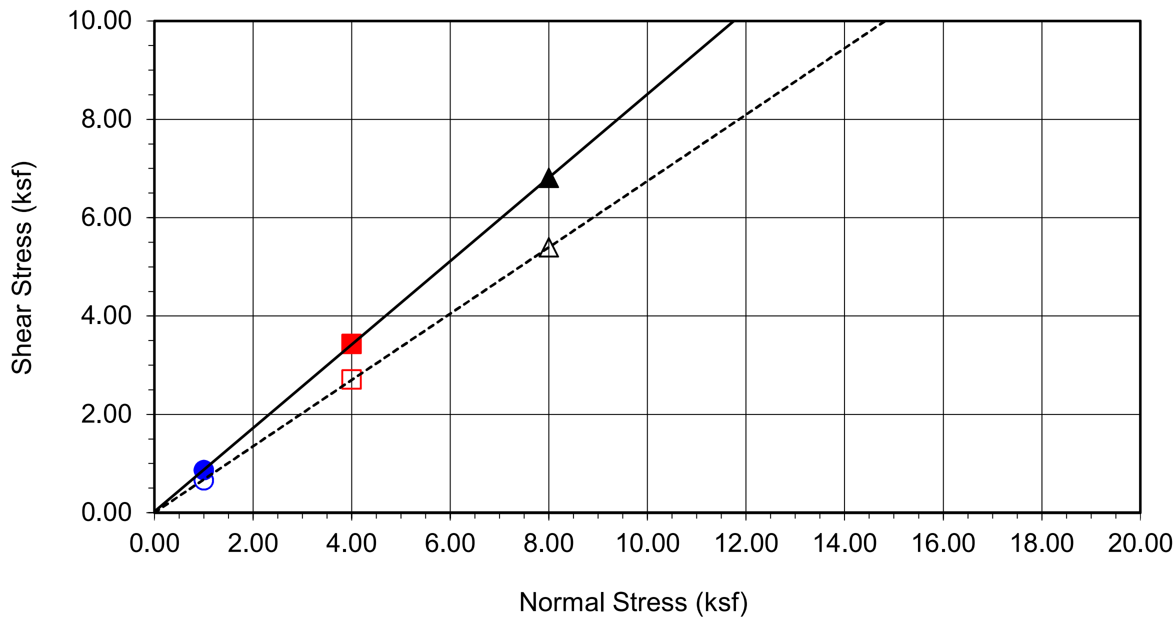
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22





Boring No.	**LP-2**
Sample No.	**R-2**
Depth (ft)	**15**
Sample Type:	Ring

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

Strength Parameters

	C (psf)	ϕ (°)
Peak	22	40
Ultimate	0	34

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.865	■ 3.427	▲ 6.809
Shear Stress @ End of Test (ksf)	○ 0.657	□ 2.707	△ 5.395
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 (%)	1.31	1.31	1.31
Dry Density (pcf)	98.3	103.1	104.8
Saturation (%)	4.9	5.6	5.8
Soil Height Before Shearing (in.)	0.9916	0.9743	0.9680
Final Moisture Content (%)	20.9	17.2	16.9

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12882.002

A-Town Parcels C & D

06-22

DS LP-2, R-2 @ 15

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	A-Town Parcels C & D	Tested By:	GEB/OHF	Date:	06/29/22
Project No.:	12882.002	Checked By:	J. Ward	Date:	07/27/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand with gravel (SM)g				

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)	643.80	458.90
Wt. of Mold (g)	203.30	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	875.50	662.20
Dry Wt. of Soil + Cont. (g)	820.50	616.13
Wt. of Container (g)	0.00	203.30
Moisture Content (%)	6.70	11.16
Wet Density (pcf)	132.9	138.4
Dry Density (pcf)	124.5	124.5
Void Ratio	0.354	0.354
Total Porosity	0.261	0.261
Pore Volume (cc)	54.1	54.1
Degree of Saturation (%) [S_{meas}]	**51.2**	85.2

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.)
06/29/22	13:58	1.0	0	0.5465
06/29/22	14:08	1.0	10	0.5460
Add Distilled Water to the Specimen				
06/29/22	14:30	1.0	22	0.5460
06/30/22	6:38	1.0	990	0.5465
06/30/22	13:00	1.0	1372	0.5465

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**1**

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	A-Town Parcels C & D	Tested By:	GEB/OHF	Date:	06/20/22
Project No.:	12882.002	Checked By:	J. Ward	Date:	07/27/22
Boring No.:	LP-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand (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)	643.00	445.30
Wt. of Mold (g)	207.00	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	867.60	652.30
Dry Wt. of Soil + Cont. (g)	810.80	614.48
Wt. of Container (g)	0.00	207.00
Moisture Content (%)	7.01	9.28
Wet Density (pcf)	131.5	134.3
Dry Density (pcf)	122.9	122.9
Void Ratio	0.372	0.372
Total Porosity	0.271	0.271
Pore Volume (cc)	56.1	56.1
Degree of Saturation (%) [S_{meas}]	**50.9**	67.4

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.)
06/20/22	14:27	1.0	0	0.5610
06/20/22	14:37	1.0	10	0.5610
Add Distilled Water to the Specimen				
06/20/22	15:00	1.0	23	0.5610
06/21/22	6:00	1.0	923	0.5610
06/21/22	8:01	1.0	1044	0.5610

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

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	A-Town Parcels C & D	Tested By:	J. Gonzalez	Date:	06/19/22
Project No.:	12882.002	Checked By:	A. Santos	Date:	06/20/22
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand with gravel (SM)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

			Scalp Fraction (%)	
Preparation Method:	X	Moist	#3/4	
		Dry	#3/8	
Compaction Method	X	Mechanical Ram	#4	16.8
		Manual Ram		

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) = 0.03330

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3853	3942	3887			
Weight of Mold (g)	1826	1826	1826			
Net Weight of Soil (g)	2027	2116	2061			
Wet Weight of Soil + Cont. (g)	527.5	493.2	560.2			
Dry Weight of Soil + Cont. (g)	498.8	456.1	508.1			
Weight of Container (g)	40.1	39.1	39.3			
Moisture Content (%)	6.26	8.90	11.11			
Wet Density (pcf)	134.2	140.1	136.4			
Dry Density (pcf)	126.3	128.6	122.8			

Maximum Dry Density (pcf) **129.0** **Optimum Moisture Content (%)** **8.2**

Corrected Dry Density (pcf) **134.3** **Corrected Moisture Content (%)** **7.0**

[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

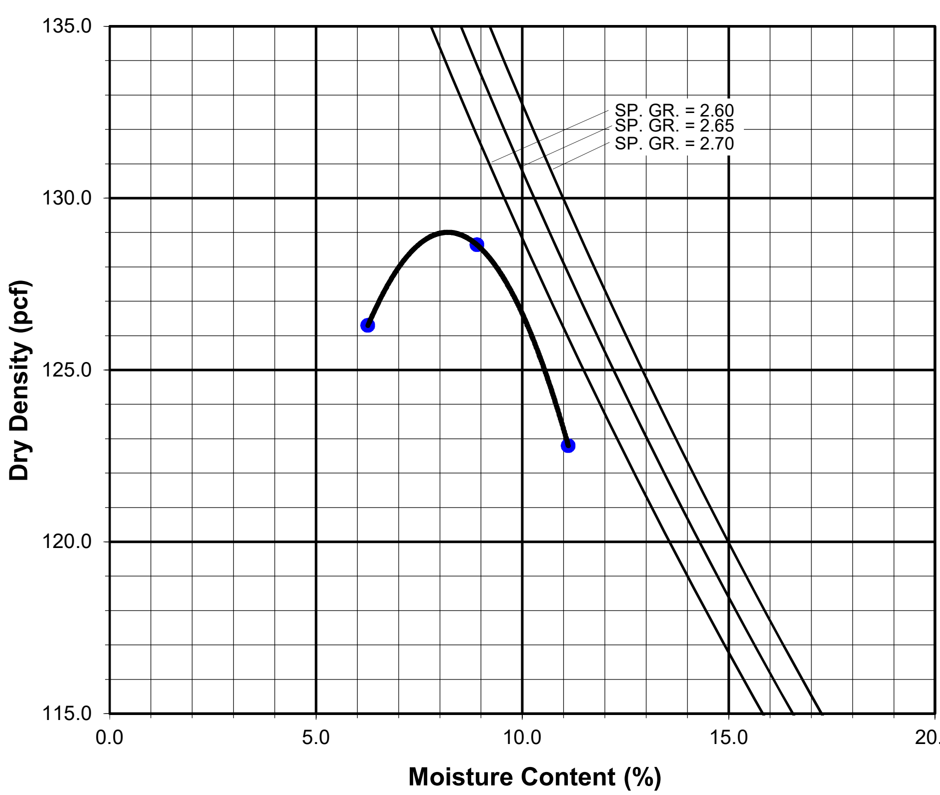
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Project Name:	A-Town Parcels C & D	Tested By:	J. Gonzalez	Date:	06/19/22
Project No.:	12882.002	Checked By:	A. Santos	Date:	06/20/22
Boring No.:	LP-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Olive brown silty sand (SM)				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

			Scalp Fraction (%)			
Preparation Method:	X	Moist	#3/4		Rammer Weight (lb.) =	10.0
		Dry			Height of Drop (in.) =	18.0
Compaction Method	X	Mechanical Ram	#3/8			
		Manual Ram	#4	4.9	Mold Volume (ft³)	0.03330

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3819	3925	3910			
Weight of Mold (g)	1826	1826	1826			
Net Weight of Soil (g)	1993	2099	2084			
Wet Weight of Soil + Cont. (g)	514.2	499.0	535.9			
Dry Weight of Soil + Cont. (g)	489.8	465.2	489.9			
Weight of Container (g)	37.8	39.6	39.5			
Moisture Content (%)	5.40	7.94	10.21			
Wet Density (pcf)	131.9	139.0	138.0			
Dry Density (pcf)	125.2	128.7	125.2			

Maximum Dry Density (pcf)	**128.8**	**Optimum Moisture Content (%)**	**7.8**
Corrected Dry Density (pcf)	**130.3**	**Corrected Moisture Content (%)**	**7.5**

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

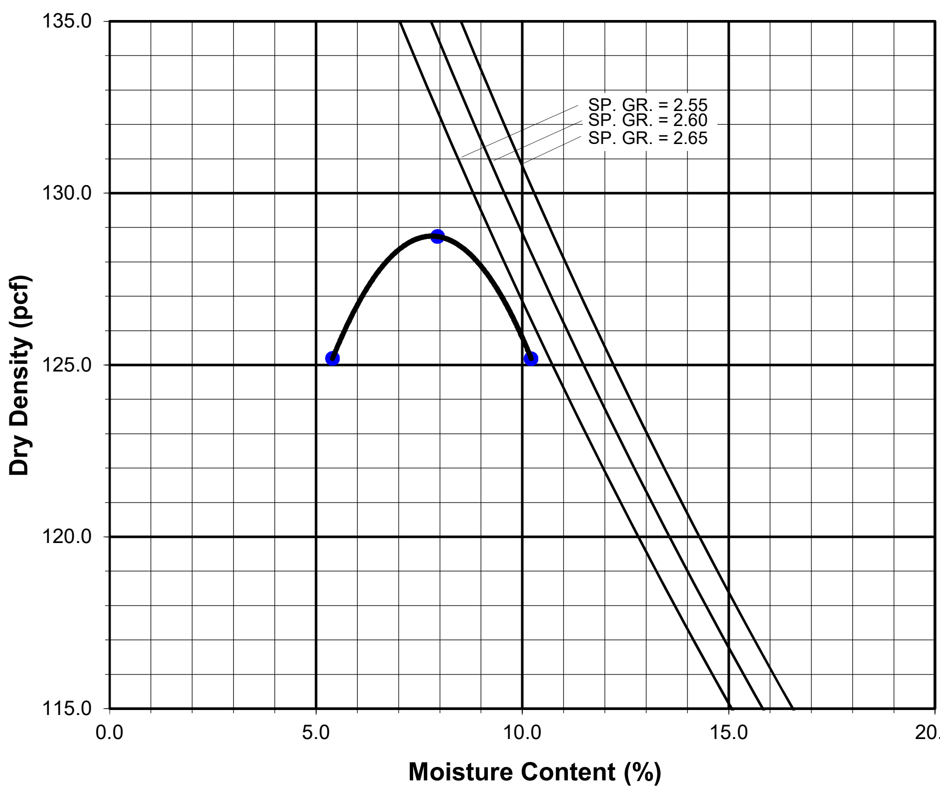
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI

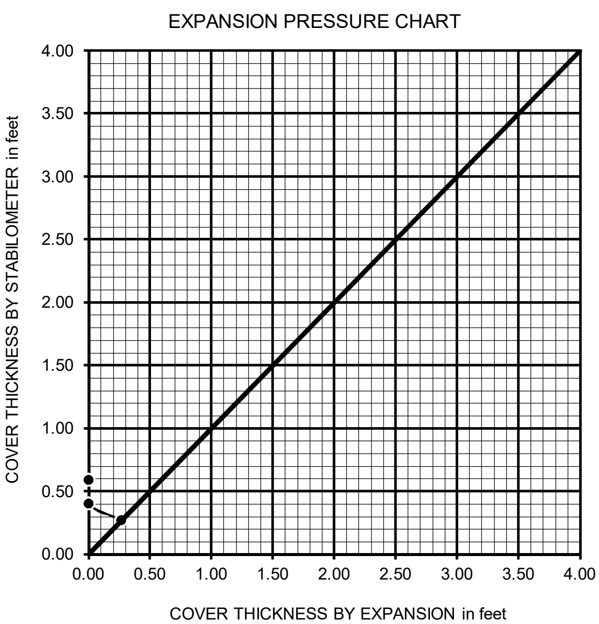


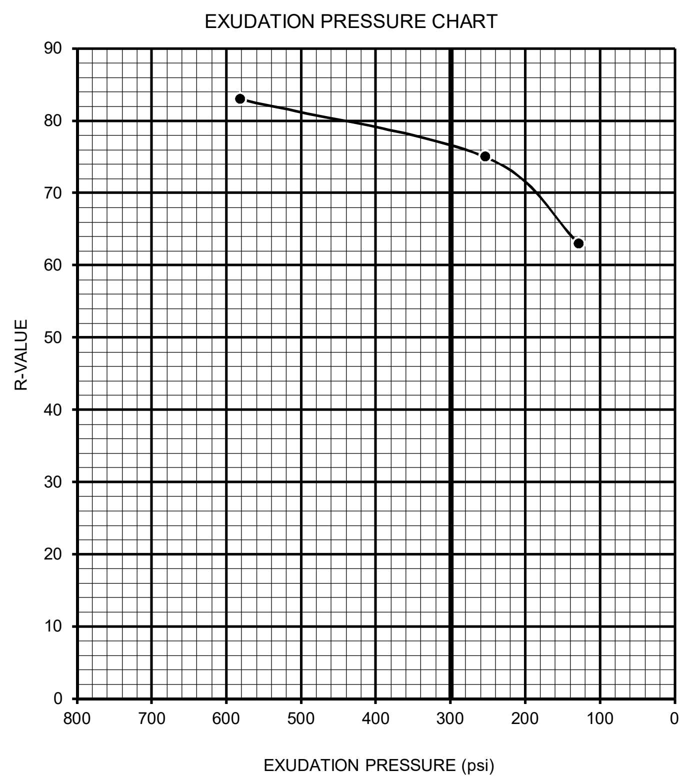
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	A-Town Parcels C & D	PROJECT NUMBER:	12882.002
BORING NUMBER:	LB-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Olive brown silty sand with gravel (SM)g	DATE COMPLETED:	6/23/2022

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	7.8	8.6	9.5
HEIGHT OF SAMPLE, Inches	2.45	2.49	2.52
DRY DENSITY, pcf	128.8	128.2	128.0
COMPACTOR PRESSURE, psi	350	275	175
EXUDATION PRESSURE, psi	582	254	128
EXPANSION, Inches x 10exp-4	8	0	0
STABILITY Ph 2,000 lbs (160 psi)	16	22	35
TURNS DISPLACEMENT	4.71	5.12	5.23
R-VALUE UNCORRECTED	83	75	63
R-VALUE CORRECTED	83	75	63

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.	0.27	0.40	0.59
EXPANSION PRESSURE THICKNESS, ft.	0.27	0.00	0.00





R-VALUE BY EXPANSION:	83
R-VALUE BY EXUDATION:	77
EQUILIBRIUM R-VALUE:	77

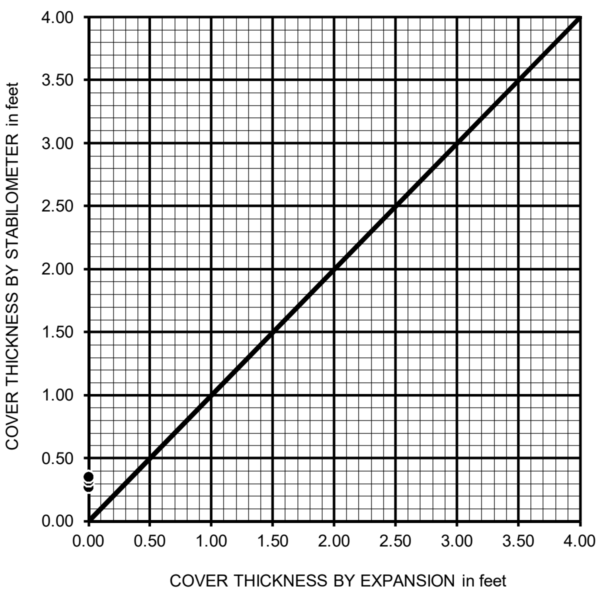
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	A-Town Parcels C & D	PROJECT NUMBER:	12882.002
BORING NUMBER:	LP-2	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Olive brown silty sand (SM)	DATE COMPLETED:	6/25/2022

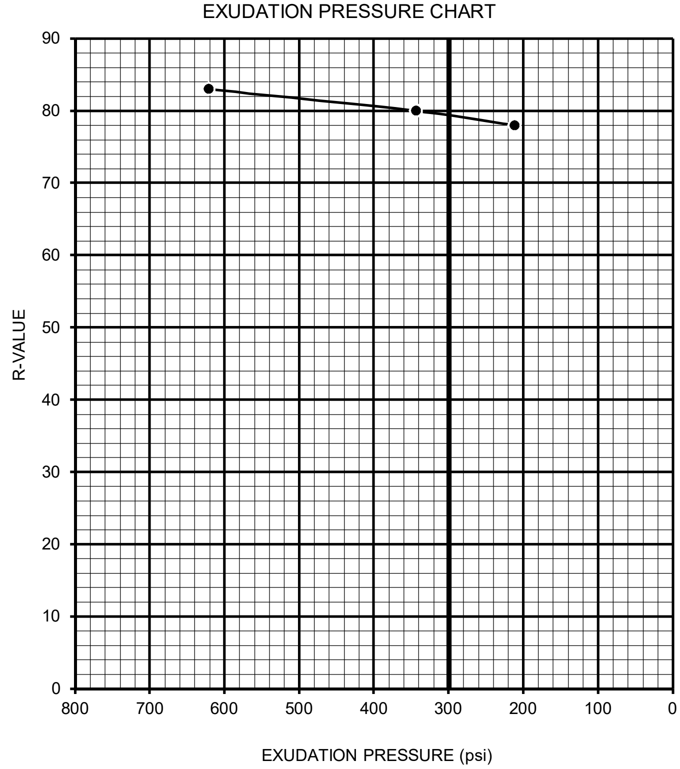
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	8.0	8.5	8.7
HEIGHT OF SAMPLE, Inches	2.50	2.51	2.51
DRY DENSITY, pcf	127.6	127.5	127.5
COMPACTOR PRESSURE, psi	350	325	300
EXUDATION PRESSURE, psi	621	344	212
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	15	17	19
TURNS DISPLACEMENT	5.06	5.18	5.31
R-VALUE UNCORRECTED	83	80	78
R-VALUE CORRECTED	83	80	78

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.	0.27	0.32	0.35
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:	79
EQUILIBRIUM R-VALUE:	79

APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

APPENDIX D
LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

TABLE OF CONTENTS

<u>Section</u> <u>Appendix D Page</u>

1.0 GENERAL 1
- 1.1 Intent 1
- 1.2 The Geotechnical Consultant of Record 1
- 1.3 The Earthwork Contractor 2

2.0 PREPARATION OF AREAS TO BE FILLED 2
- 2.1 Clearing and Grubbing 2
- 2.2 Processing 3
- 2.3 Overexcavation 3
- 2.4 Benching 3
- 2.5 Evaluation/Acceptance of Fill Areas 4

3.0 FILL MATERIAL 4
- 3.1 General 4
- 3.2 Oversize 4
- 3.3 Import 4

4.0 FILL PLACEMENT AND COMPACTION 5
- 4.1 Fill Layers 5
- 4.2 Fill Moisture Conditioning 5
- 4.3 Compaction of Fill 5
- 4.4 Compaction of Fill Slopes 5
- 4.5 Compaction Testing 5
- 4.6 Frequency of Compaction Testing 6
- 4.7 Compaction Test Locations 6

5.0 SUBDRAIN INSTALLATION 6

6.0 EXCAVATION 6

7.0 TRENCH BACKFILLS 7
- 7.1 Safety 7
- 7.2 Bedding and Backfill 7
- 7.3 Lift Thickness 7
- 7.4 Observation and Testing 7

TABLE OF CONTENTS (CONT'D)

ATTACHMENTS

Standard Details

A - Keying and Benching	Rear of Text
B - Oversize Rock Disposal	Rear of Text
C - Canyon Subdrains	Rear of Text
D - Buttress or Replacement Fill Subdrains	Rear of Text
E - Transition Lot Fills and Side Hill Fills	Rear of Text

1.0 GENERAL

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 PREPARATION OF AREAS TO BE FILLED

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 FILL MATERIAL

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 SUBDRAIN INSTALLATION

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 TRENCH BACKFILLS

7.1 <u>Safety</u>

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 <u>Bedding and Backfill</u>

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

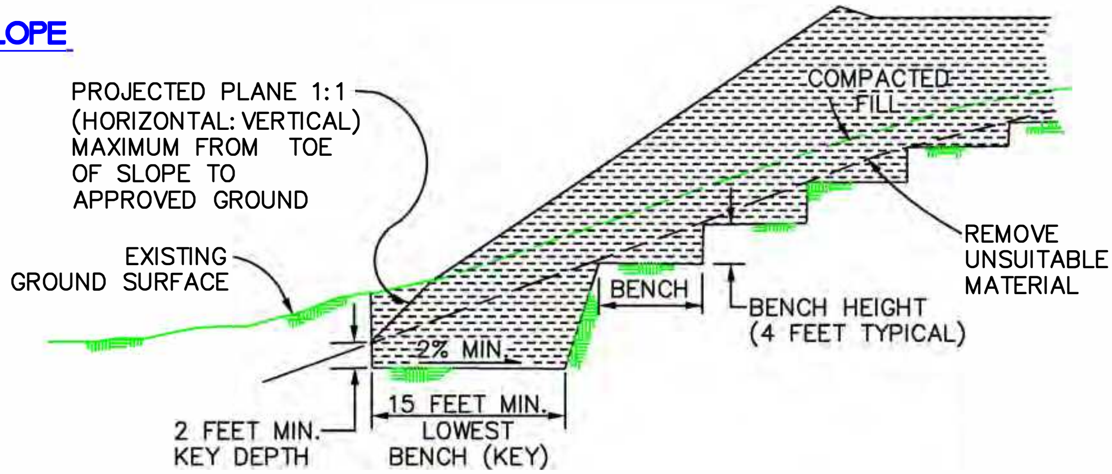
7.3 <u>Lift Thickness</u>

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

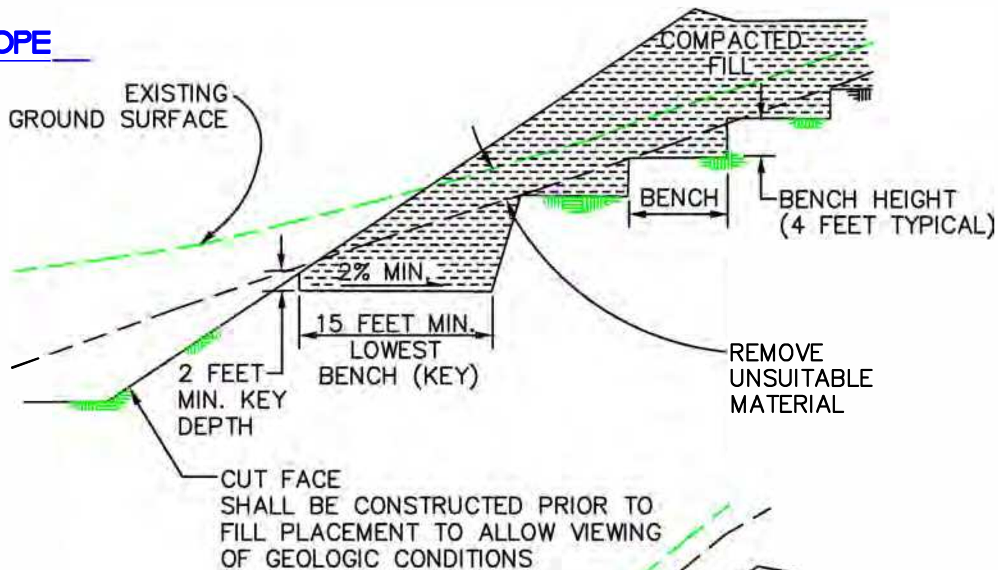
7.4 <u>Observation and Testing</u>

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

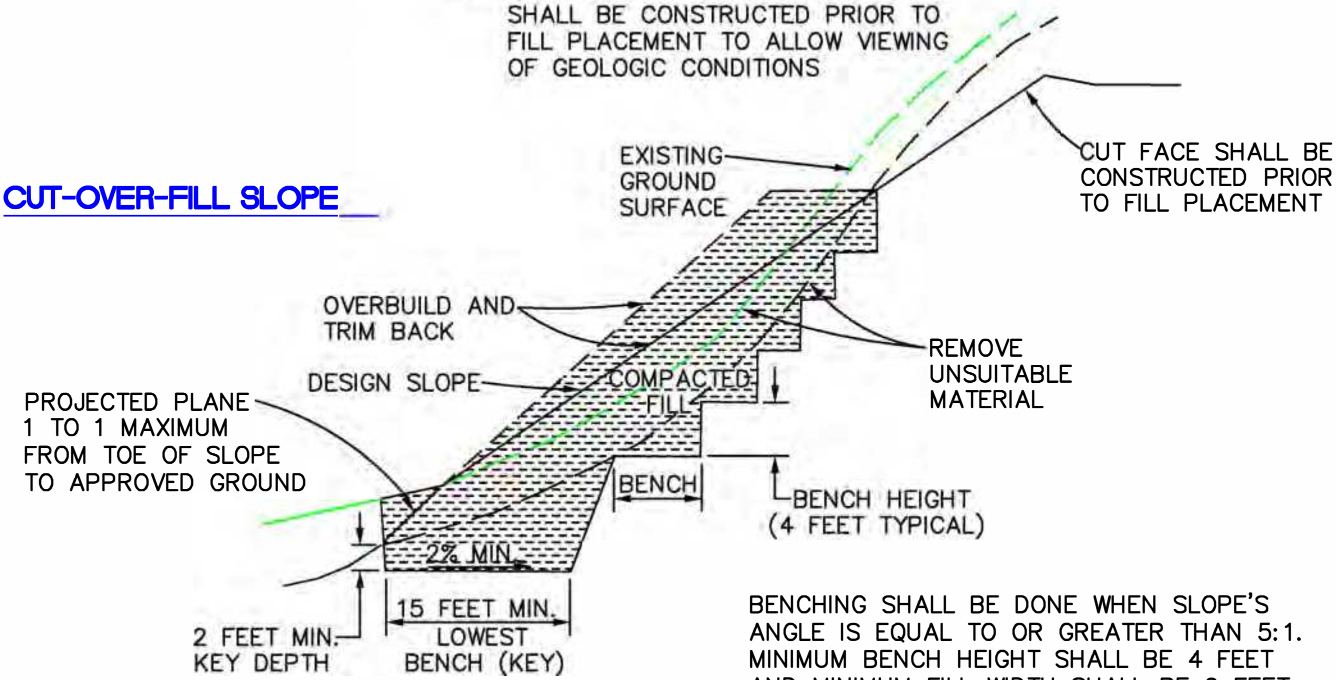
FILL SLOPE



FILL-OVER-CUT SLOPE

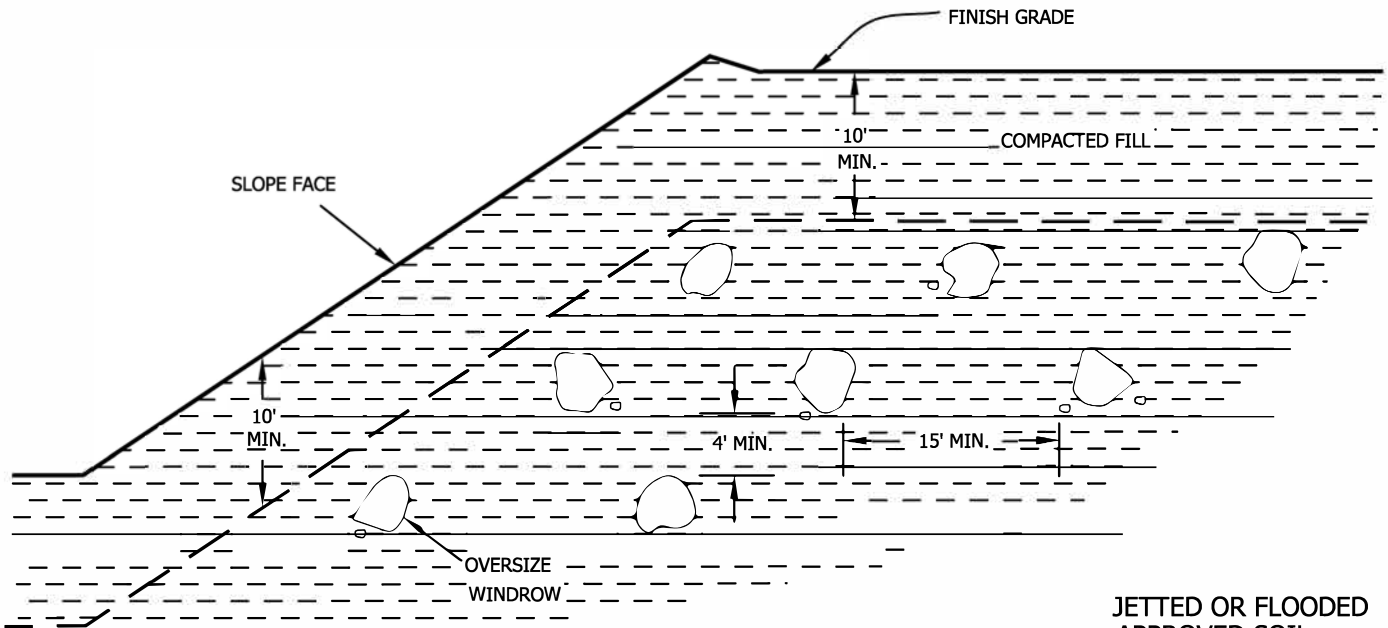


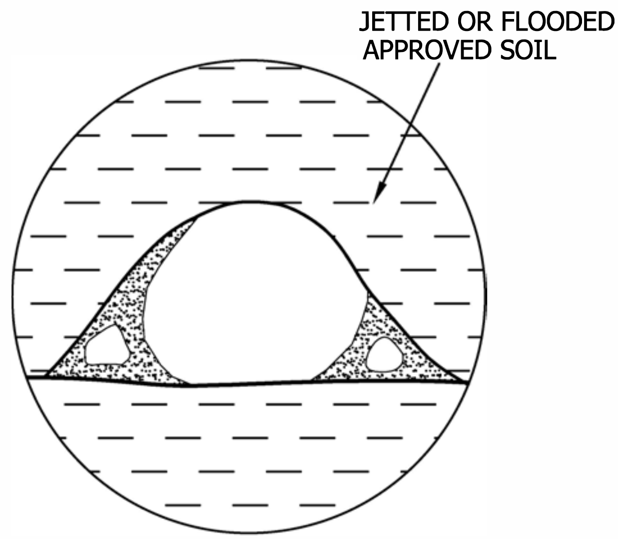
CUT-OVER-FILL SLOPE



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

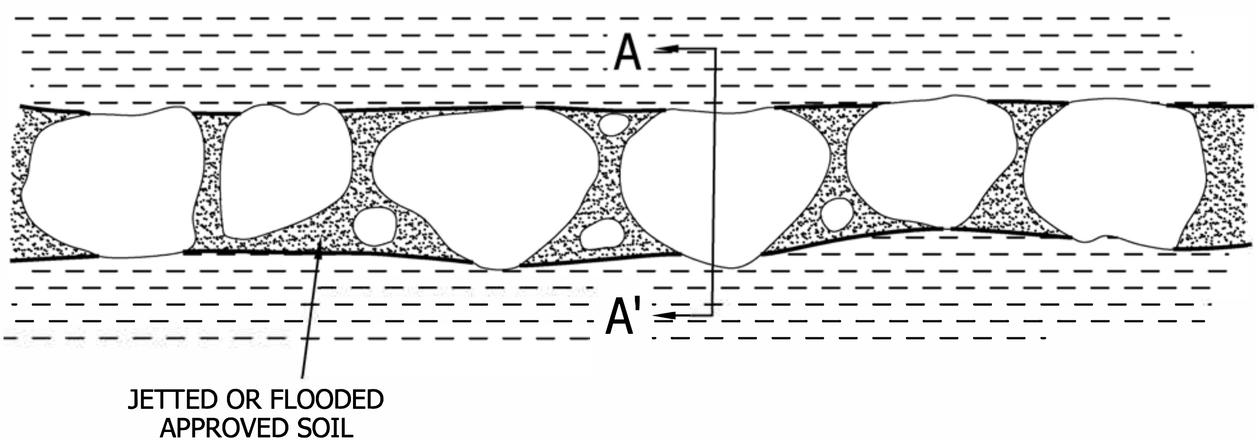




SECTION A-A'

- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	Leighton

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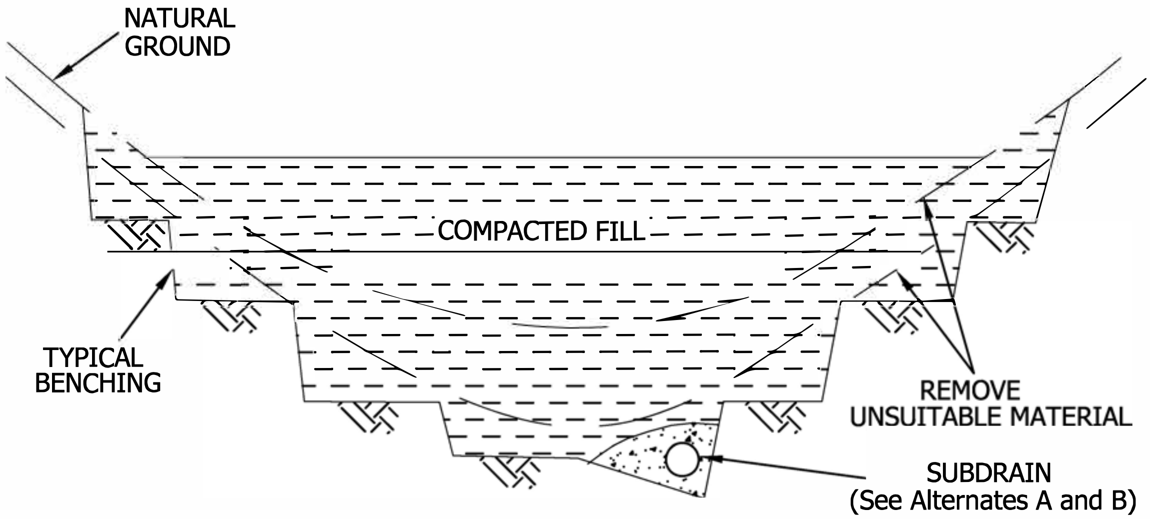
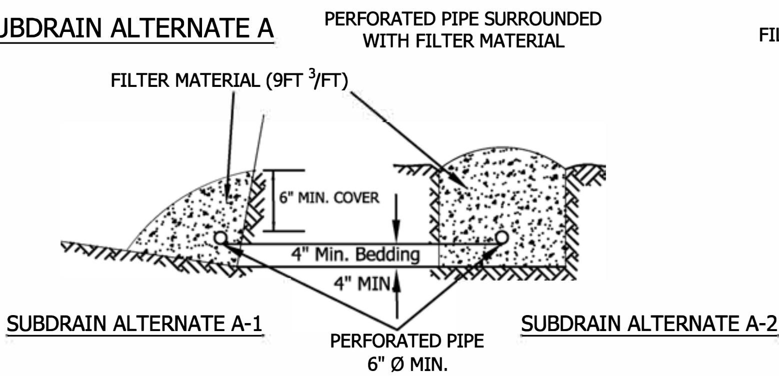


SUBDRAIN ALTERNATE A

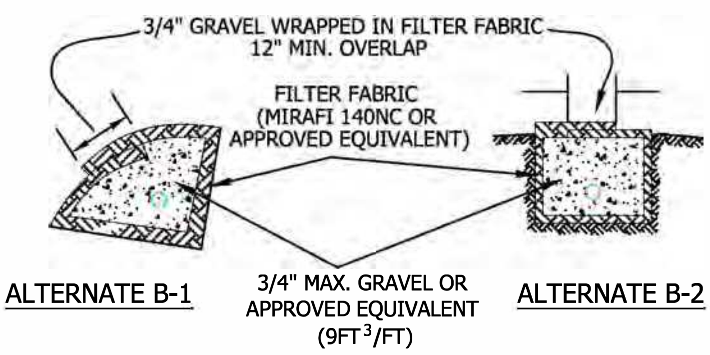


FILTER MATERIAL

FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

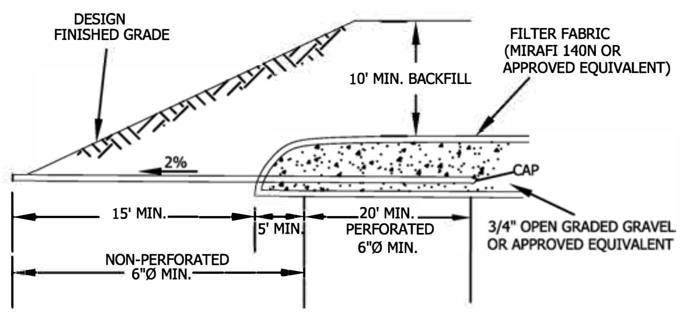
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B

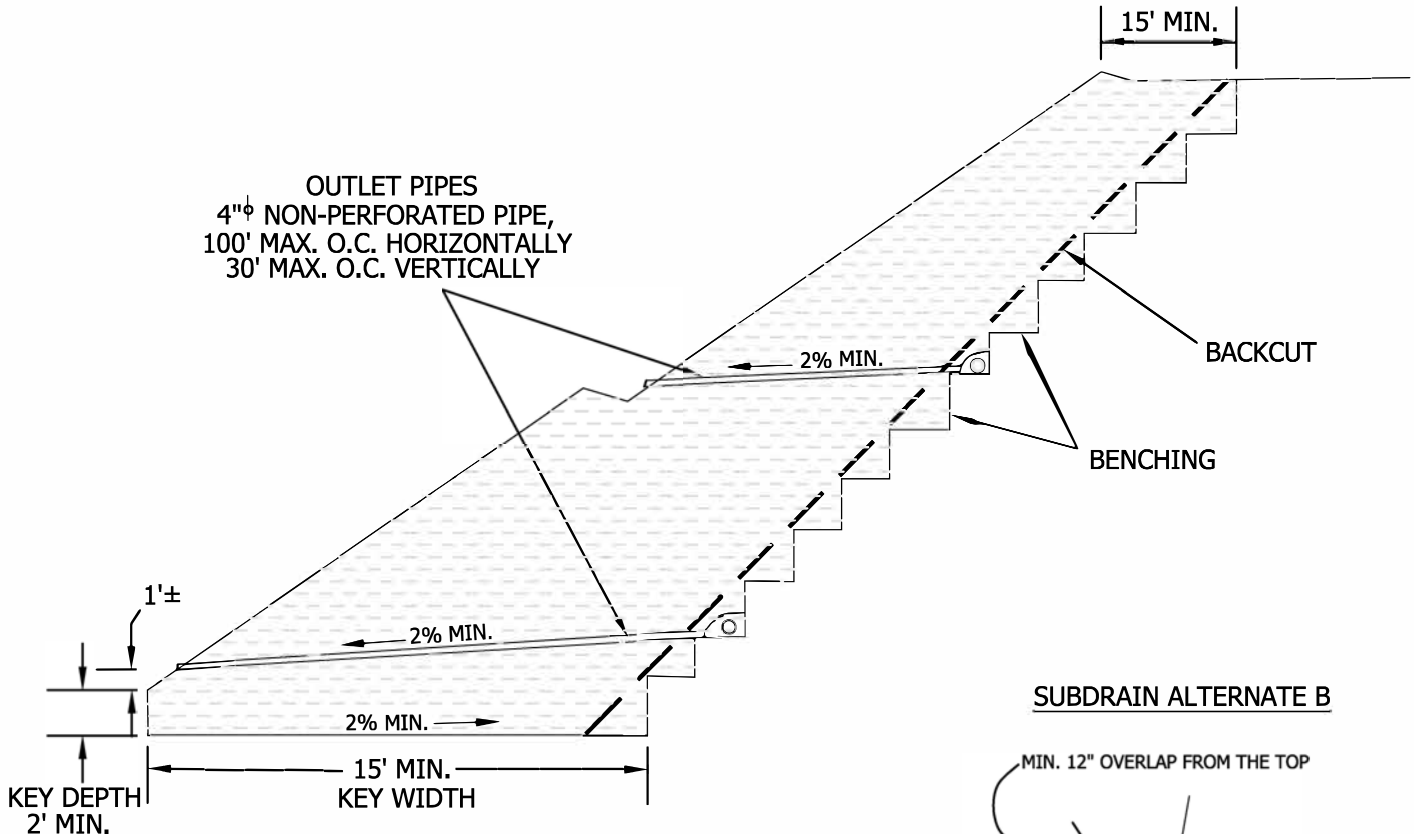


○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

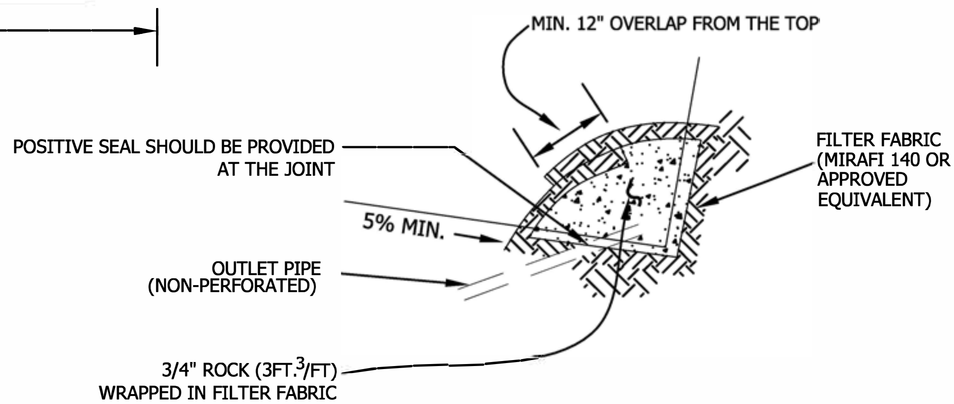
DETAIL OF CANYON SUBDRAIN TERMINAL



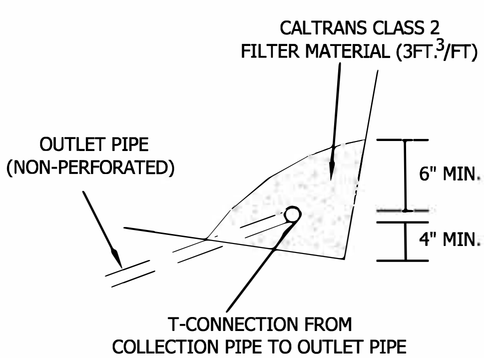
CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton



SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	Leighton

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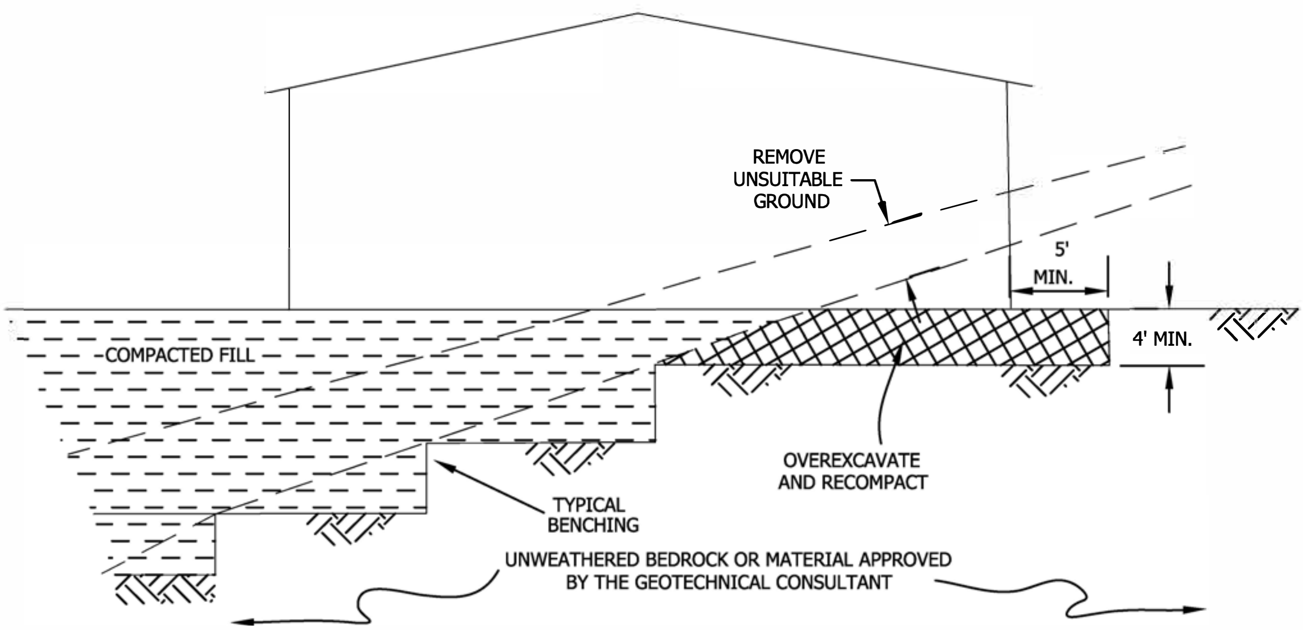
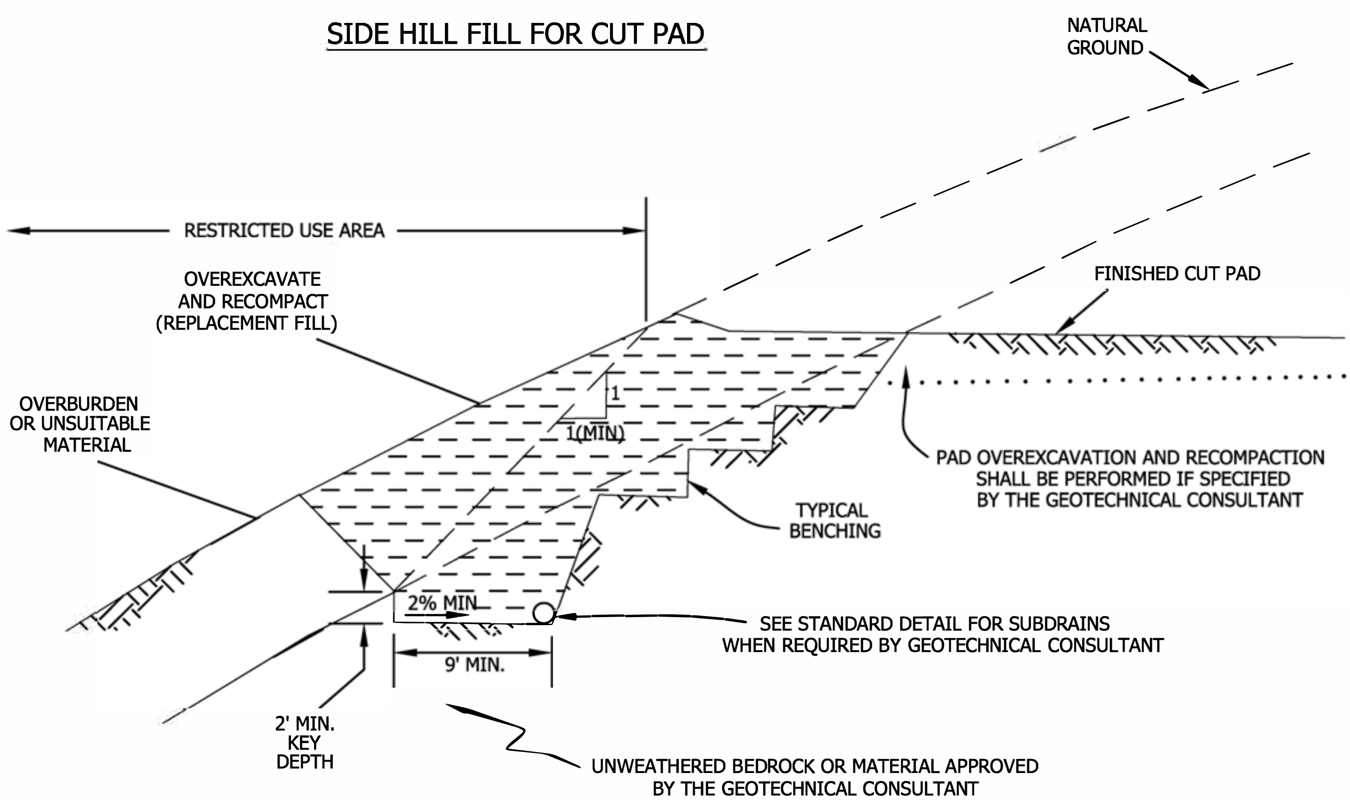
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E	Leighton

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