Appendix D Geotech Investigation

Appendix

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February 11, 2020 Project No. 184014-02

Mr. Michael Gregg
Shopoff Realty Investments
2 Park Plaza, Suite 700
Irvine, California 92614

Subject: ***Updated Preliminary Geotechnical Report Conclusion Relative to Additional Property, Proposed Lincoln at Euclid Residential Development, Located on the Northeast Corner of Lincoln Avenue and Euclid Street, Anaheim, California***

Reference: *LGC Valley, Inc. 2019, Preliminary Geotechnical Investigation and Infiltration Testing for the Proposed Lincoln at Euclid Multifamily Development, Located on the Northeast Corner of Lincoln Avenue and Euclid Street, Assessor's Parcel Numbers 072-110-21 and 072-110-50, City of Anaheim, California, Project No. 184014-01, dated April 4, 2019*

Introduction

In accordance with your request, LGC Valley, Inc. (LGC) has prepared this letter addressing the property that was added to the proposed Lincoln at Euclid development following issuance of the referenced preliminary geotechnical report, dated April 4, 2019. The purpose of this letter is to provide an opinion as to the applicability of the geotechnical findings, conclusions, and recommendations presented in the project geotechnical report relative to the additional properties.

Project Site Description and Geotechnical Conditions

The study area addressed in the referenced geotechnical report consisted of two parcels: APN 072-110-21 and 072-110-50 (1631 and 1699 West Lincoln Avenue, respectively), totaling approximately 5.9 acres. Following the issuance of our April 4, 2019 report, additional property (i.e. APN 072-110-19 [1621 West Lincoln Avenue] and the western portion of APN 072-110-50 [along the east side of Euclid Avenue]) was added to the proposed development plan which now totals approximately 7.17 acres.

Parcel 072-110-50 is currently undeveloped and is located in the northern portion of the property. At the time of our geotechnical investigation, the western portion of this parcel including a portion of the slope along the east side of Euclid Avenue was not part of the development. However, three test pits were excavated just to the east and south sides of this area. Geologic logging of the test pits (i.e. Test Pits TP-1 through TP-3) indicated similar geologic conditions consisting of 3 to 5.5 feet of undocumented fill underlain by young alluvial fan deposits consisting of silty sands (LGC, 2019).

Parcel 072-110-19 (1621 West Lincoln Avenue) located directly east of Parcel APN 072-110-21 (1631 West Lincoln Avenue) is a City of Anaheim yard used to store vehicles and soil/construction debris. It includes one existing building near the central portion of the site while the remaining portion of the parcel is undeveloped. Small-diameter borings excavated the eastern portion of the proposed development (i.e. Borings B-1 and B-6; as well as Borings FB-1 through FB-4) indicate similar geologic conditions consisting of predominantly poorly sorted sands and silty sands near the ground surface, with near-horizontal layers of silty sand, sands, silts, and sandy to silty clays at depth (LGC, 2019).

Conclusions

Based on our geotechnical evaluation of the site and professional experience in the general vicinity, we anticipate that the subsurface conditions present in the added parcels are similar to the adjacent parcels evaluated during the project preliminary geotechnical investigation. As such, our findings, conclusions, and recommendations presented in the referenced geotechnical report are still considered applicable to the currently planned development area. Furthermore, it is our professional opinion that the proposed site development is feasible from a geotechnical standpoint provided the recommendations included in the referenced report are incorporated into the project plans and specifications, and followed during site grading and construction.

Limitations

Our findings, conclusions and opinions were prepared in accordance with generally accepted professional geotechnical engineering and geologic principles and practice in southern California at this time. We make no other warranty, either express or implied. Soil and geologic conditions revealed during construction/grading may be different from our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and geologist and design(s) adjusted as required or alternate design(s) recommended.

Closure

We appreciate the opportunity to be of service. If you have any questions regarding our letter, please contact this office. We appreciate this opportunity to be of service.

Sincerely,

LGC VALLEY, Inc.

Randall Wagner, CEG 1612
Senior Project Geologist

Adam Rich, PE 85642
Project Engineer

RKW/ACR

Distribution: (1) Addressee (via e-mail)
(1) EPD Solutions, Attention Ms. Norah Jaffan (via e-mail)

LGC Valley, Inc.

Geotechnical Consulting

PRELIMINARY GEOTECHNICAL INVESTIGATION AND INFILTRATION TESTING FOR THE LINCOLN AT EUCLID MULTIFAMILY DEVELOPMENT, CITY OF ANAHEIM, CALIFORNIA

Project No. 184014-01

Dated: April 4, 2019

Prepared For:

SLF-West Lincoln, LLC.
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LGC
LGC Valley, Inc.
Geotechnical Consulting

April 4, 2019 Project No. 184014-01

Mr. Brian Rupp
SLF-West Lincoln, LLC.
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Irvine, California 92614

Subject: ***Preliminary Geotechnical Investigation and Infiltration Testing for the Proposed Lincoln at Euclid Multifamily Development, Located on the Northeast Corner of Lincoln Avenue and Euclid Street, Assessor's Parcel Numbers 072-110-21 and 072-110-50, City of Anaheim, California***

In accordance with your request and authorization, LGC Valley, Inc. (LGC) has performed a preliminary geotechnical investigation and infiltration testing for the proposed Lincoln at Euclid Multifamily Development, located northeast of the intersection of West Lincoln Avenue and Euclid Street, Assessor's Parcel Numbers 072-110-21 and 072-110-50 (1631 and 1699 West Lincoln Avenue) in the City of Anaheim, California. The purpose of our investigation was to evaluate the existing geotechnical conditions relative to the proposed residential development and to provide geotechnical recommendations applicable to the grading operations and future site construction for the project. Our study included: a field investigation consisting of the excavation of six (6) small-diameter borings and ten (10) test pits; an infiltration study consisting of infiltration testing of five (5) test borings and logging three (3) infiltration exploratory borings; laboratory testing of representative on-site soil samples; and geotechnical analysis of the collected data. This report presents the findings, conclusions, opinions, and recommendations relative to the grading and development of the site.

Based on the results of our geotechnical investigation and infiltration testing, it is our professional opinion that the proposed site development is feasible from a geotechnical standpoint provided the recommendations included in this report are incorporated into the project plans and are followed during site grading and construction. If you have any questions regarding our report, please contact this office. We appreciate this opportunity to be of service.

Respectfully Submitted,

LGC Valley, Inc.

Melissa Stayner, CEG 2707
Associate

Basil Hattar, GE 2734
Principal Engineer

Randall K. Wagner, CEG 1612
Senior Project Geologist

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

The purpose of this geotechnical investigation was to identify and evaluate the existing geologic and geotechnical conditions at the site (Figure 1) as they pertain to the proposed development and to provide preliminary geotechnical design criteria. Recommendations for grading, construction, preliminary foundation design, and preliminary pavement sections for the proposed Lincoln multifamily development are included herein to address the on-site geotechnical conditions. This report includes the results of our background review, subsurface investigation, laboratory testing, infiltration testing, and engineering evaluation of the site, and provides our conclusions, opinions, and recommendations with respect to site development. Our scope of work in preparation of this document included:

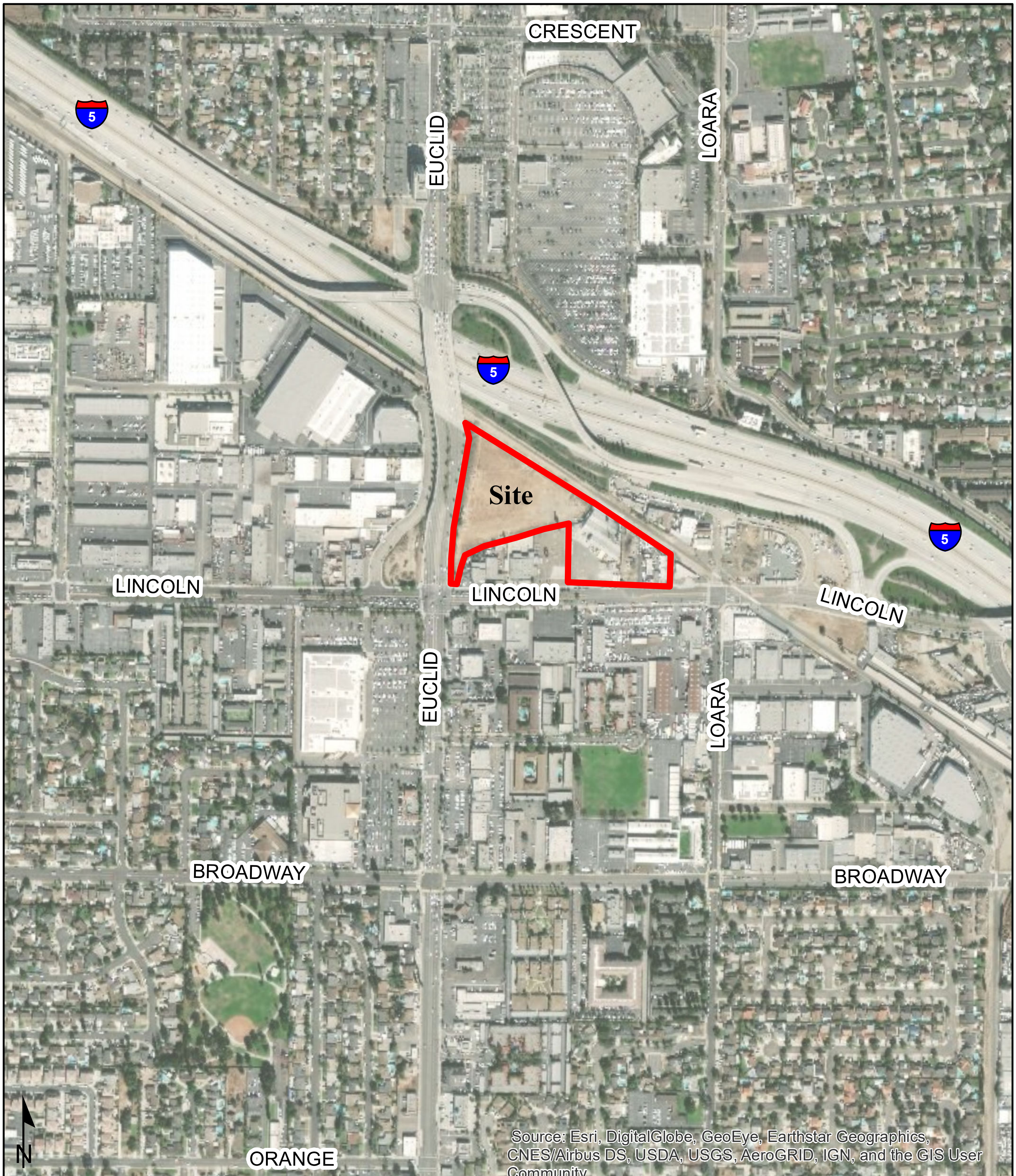
- Review of geotechnical reports, geologic maps and other documents relevant to the site (Appendix A).

- Perform site visits to evaluate the existing conditions and mark the geotechnical/infiltration boring locations.

- Perform a subsurface investigation on June 6, 2018 including drilling, sampling, and logging of six small-diameter exploratory borings that are labeled Borings B-1 through B-6. The boring logs are presented in Appendix B, and their approximate locations are depicted on the Geotechnical Map (Figure 2). The field investigation was performed under the supervision of a licensed engineering geologist from LGC with the goal of evaluating the general characteristics of the subsurface conditions on the site including classification of site soils, determination of depth to competent soil and groundwater, and to obtain representative soil samples.

- Perform laboratory testing on the representative soil samples obtained during our geotechnical investigation. Results of these tests are presented in Appendix C.

- Perform an infiltration investigation on March 14 and 15, 2019 including drilling and logging of five small-diameter infiltration test borings that are labeled Borings I-1 through I-5, and three small-diameter infiltration exploratory borings that are labeled Borings E-1 through E-3. The boring logs are presented in Appendix D, and their approximate locations are depicted on the Geotechnical Map (Figure 2).



LGC

Figure 1
Site Location Map
City of Anaheim
Orange County, California

Project Name	Lincoln At Euclid
Project Number	184001-00
Eng./Geo.	BIH/MS
Date	April 2019

0 0.17 Miles

- Perform infiltration feasibility testing in the locations and at the depths of the bottom of the proposed storm water collection facilities. Percolation rates of the soils were recorded in the field, converted to infiltration rates, and are presented in this report for the use of the civil engineer in final design of the storm water facilities. The investigation also included logging the material as the infiltration test wells were advanced, and filling out Worksheet H: Factor of Safety and Design Infiltration Rate, and Worksheet I: Summary of Groundwater-related Feasibility Criteria, per the Orange County Technical Guidelines Document (TGD) (Orange County, 2013).

- Perform geotechnical analyses and evaluation of the data.

- Preparation of this report presenting our findings, conclusions, opinions and recommendations with respect to the evaluated geologic and geotechnical conditions at the site.

1.2 Site Description and Proposed Development

The subject study area consists of two parcels: APNs 072-110-21 and 072-110-50 (1631 and 1699 West Lincoln Avenue, respectively), totaling approximately 5.8 acres. Parcel 072-110-50 is currently undeveloped and is located in the northwest portion of the property. The existing topography is generally flat. The site currently has a dense cover of native grasses and weeds. The site is bounded to the west by an easement and an ascending slope to Euclid Street, to the northeast by the Southern Pacific Railroad easements and tracks, and to the south by Parcels 072-110-21, -25,-49,-47, and -48 that front West Lincoln Avenue.

There is an active cement manufacturing plant on Parcel 072-110-21 (1631West Lincoln Avenue). There are existing buildings along the northern and northeastern portion of the site, and the remaining portion of the parcel is covered by an asphalt parking lot. This parcel is also relatively flat. This parcel is bounded to the northwest by Parcel 72-110-50, to the northeast by the Southern Pacific Railroad property, to the east by Parcel 072-110-19, to the west by Parcel 072-110-25, and to the south by West Lincoln Avenue.

The proposed development will consist of a total of 101-unit multifamily development consisting of three-story town homes, a recreation center with a swimming pool, driveways, guest parking, concrete flatwork, underground utilities, landscaping, and three storm water infiltration facilities.

1.3 Site History

Historical aerial photos indicate that the existing buildings along the north side of West Lincoln Avenue were constructed in the late 1950's to early 1960's. Based on our review of the State of California GeoTracker Website, a Leaking Underground Storage Tank (LUST) remediation operation was performed within the 1631 West Lincoln Avenue parcel prior to 1996. The LUST operation included the removal of the 10,000-gallon underground double-walled diesel storage tank, remediation of potential diesel impacted soils, and replacement of fill material within the excavation limits. No documentation of the backfill and/or compaction of these backfill soils was available, and therefore, the fill soils are considered undocumented and should be removed to competent soil and replaced with compacted fill during grading operations. The limits of this area is shown on the Geotechnical Map (Figure 2).

Historic aerial photos indicate that there was an off-ramp from Interstate 5 Freeway to Euclid Street that crossed over the railroad right-of-way (ROW) and extended across the southern boundary of Parcel 072-110-50 (which at the time, we understand was owned by Caltrans). The bridge over the railroad consisted of two abutments with fill embankments descending on either side of the railroad ROW. As part of this investigation, a ground penetrating radar subcontractor came to the site and investigated the areas where LGC estimated the abutments were previously located, in attempt to locate any underground structures that may have been left in place, but none were found during the survey.

Sometime between October 1995 and April 2003, the bridge crossing the railroad was demolished and the fill embankments were removed. Between 2003 and March 2005, it appears that soil and construction debris (likely from improvements to Interstate 5) was end-dumped on Parcel 072-110-50 and that sometime prior to March 2005, the end-dumped soil piles were knocked down and spread across the vast majority of the property. Subsequent to 2005, this parcel has remained unchanged with the exception of varying amount of vegetation across the parcel.

1.4 <u>Subsurface Geotechnical Investigation and Laboratory Testing</u>

Our subsurface investigation was performed on June 6, 2018 and consisted of the excavation of six hollow-stem auger borings (Borings B-1 through B-6), ten backhoe test pits (Test Pits TP-1 through TP-10), and a limited ground penetrating radar (GPR) study. The borings were extended to depths ranging from approximately 9 to 51 feet below ground surface (BGS). The test pits were extended to depths of approximately 6 to 8 feet BGS. The approximate locations of the borings, test pits, and area that was surveyed by GPR are shown on the Geotechnical Map (Figure 2). The logs of the geotechnical borings and test pits are presented in Appendix B.

The GPR study was performed along the northeastern property boundary at the approximate location of a previously existing bridge that extended over the railroad tracks and easements. The GPR survey was performed to locate the potential presence of buried concrete foundations and/or caissons associated with the bridge. To that end, we employed a subsurface survey company to attempt to locate any buried concrete structures were left-in-place after the demolition of the bridge and removal of the fill embankment of the freeway off-ramp.

The results of the subsurface survey were inconclusive; in large part due to the presence of the construction debris within the surficial undocumented fill soils (specifically the rebar and welded-wire mesh). There is a possibility that the demolished bridge consisted of a spread footing that was removed during the demolition operations; however, there is a possibility that the bridge abutments were founded on caissons that were not identified during the limited ground penetrating radar and geophysical survey. If encountered, bridge foundation/caissons should be removed to a minimum depth of 10 feet below the proposed structures.

Previous direct-push advancements by FREY Environmental in 2005 as part of the UST investigation on Parcel 072-110-21 were reviewed as a part of this study. The locations of the advancements are shown on the Geotechnical Map (Figure 2), and the logs are presented in Appendix B.

During LGC's subsurface investigation, bulk and relatively undisturbed samples were collected for laboratory testing. Laboratory testing was performed by EGLAB, Inc. (EGL). Laboratory testing was performed on representative soil samples and included moisture and density tests, Atterberg Limits, sieve analysis, corrosive testing, expansive index testing, consolidation, and remolded direct shear testing. LGC has reviewed the laboratory test data, procedures and results performed by EGL with respect to the subject site and concur with and accept responsibility as geotechnical engineer-of-record for their work (laboratory testing). A summary of the laboratory test procedures and test results are presented in Appendix C. The moisture and density test results are presented on the boring logs included in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional and Local Geology

The site is located on the Santa Ana block between the Santa Ana Mountains to the east and the San Joaquin Hills to the southwest in the northwestern portion of the Peninsular Ranges Geomorphic Province. The area is underlain by recent alluvial fan deposits and stream channel deposits, and at depth by Tertiary sedimentary rocks ranging in age from Paleocene through Pliocene and a basement assemblage of Mesozoic metasedimentary and Cretaceous volcanic and batholithic rocks. The Santa Ana Mountains have been uplifted due to compressional forces between the Newport-Inglewood and Whittier-Elsinore Fault zones. As uplift and erosion of the mountain block occurred, the area to the west was filled with poorly consolidated sediments, the youngest of which are of the recent Quaternary period that include alluvial fan and axial channel deposits.

2.2 Site-Specific Geology

The majority of the site is underlain by Quaternary-aged young alluvial fan deposits (Map Symbol Qyf) and localized areas of artificial undocumented fill (Map Symbol Afu). The approximate extent of the geologic units present on the site is depicted on the Geotechnical Map (Figure 2). A brief description of the geologic units encountered on the site is presented below.

2.2.1 Undocumented Artificial Fill (Map Symbol - Afu)

Artificial undocumented fill was encountered mantling the upper approximately 2 to 6 feet of Parcel 072-110-50 (1699 West Lincoln Avenue) and as backfill in the LUST removal area within Parcel 072-110-21 (1631 West Lincoln Avenue). The approximate limits of the artificial undocumented fill is shown on Figure 2. All undocumented fill should be removed during grading operations and replaced with engineered fill.

The undocumented fill located in Parcel 072-110-50 was found to generally consist of silty fine sands, gravelly sands, sandy gravels (i.e. Recycled Caltrans Class II aggregate base material) and lesser amounts of clayey sands and silty sandy conglomerate with cobbles up to 8 inches in maximum dimension. These soils were found to be medium gray brown, medium gray, and orange brown in color, dry to damp, and loose to medium dense. During our investigation we encountered a moderate to abundant amount of construction debris within these undocumented fills. The construction debris generally consisted of concrete and asphalt with minor amounts of brick, clay pipe, rebar, welded wire mesh, and recycled aggregate base. Very minor amounts of wood, plastic Visqueen, and other deleterious materials were also encountered.

Undocumented fill associated with the removal, remediation, and backfill of the LUST within Parcel 072-110-21 was encountered in the southeast portion of the site. Boring B-6 encountered approximately 9 feet of pea gravel above something the drill rig was unable to drill through (the driller thought it might be a concrete slab or structure). Review of the documents pertaining to the removal and remediation of the LUST within Parcel 072-110-21 available on the State of California GeoTracker website (Appendix A) did not contain discussion or descriptions of the backfill operations. The approximate limits of the LUST removal/excavation was presented on the soil boring/direct-push advancement location map (Frey, 2005) and that location is included on Figure 2 of this report.

2.2.2 Quaternary-aged Young Alluvial Fan Deposits (Map Symbol - Qyf)

Based on our geologic logging of the subsurface explorations, this unit consists predominantly of poorly sorted sand to silty sand near the ground surface, with near-horizontal layers of silt, silty clay, and sandy clay below. The soils were found to be slightly moist to moist, loose to very dense (or soft to hard). This unit was found to extend below the maximum depth explored during our boring and test-pit subsurface investigation. Based on our test-pit investigation, the upper 1 to 3 feet of this unit was found to be porous to slightly porous and potentially compressible, and as such, should be removed during grading. Estimated remedial removal depths based on data collected from the geotechnical borings and test pits are presented on Figure 2, Geotechnical Map.

2.3 Geologic Structure

Based on our subsurface investigation, review of the geologic maps of the general vicinity (Appendix A), and our professional experience with nearby projects, the alluvial soils are generally massive to thickly-bedded and flat-lying. Adverse structural geology is not anticipated to be a constraint to grading.

2.4 Landslides

Based on our review of geologic maps and our site reconnaissance and field investigation, the site is not on, or located in the vicinity of any known landslides.

2.5 Groundwater

Groundwater was not encountered during our subsurface investigation to the maximum depth explored of 51 feet. The California Department of Water Resources monitored the ground water level in a well located approximately 2000 feet southwest of the site. Readings were taken from 1969 through 2003. The most recent data indicates a groundwater level at an approximate elevation ranging from 12 to 22 feet in elevation, or more than 110 feet below existing ground surface (http://wdl.water.ca.gov/waterdatalibrary/groundwater).

2.6 *Surface Water and Flooding*

Based on our review of the proposed development, surface flow will be directed into storm drains that will outlet into three infiltration facilities on the site. These proposed facility locations are shown on Figure 2, Geotechnical Map. Surface water runoff relative to project design is the purview of the project civil engineer and should be directed away from the planned structures and tops-of-slopes.

LGC reviewed the applicable Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM) relative to the site and general vicinity. Based on our review of FIRM Map Number 06059C0129J, (USFEMA, 2009), the site is located in Zone X- area of minimal flood hazard.

2.7 *Faulting, Seismicity, and Related Effects*

2.7.1 *Faulting*

Based on our review of the Alquist-Priolo Earthquake Fault and Seismic Hazard Zone Maps of the site and general vicinity, the site is not located within a currently established Alquist-Priolo Earthquake Fault Zone for fault rupture hazard (formerly Special Studies Zones for fault rupture hazard). However, movement associated with the nearby active faults could cause significant ground motion at the site. Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region include soil liquefaction and dynamic settlement. Other secondary seismic effects include shallow ground rupture, and seiches and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the on-site geology.

Regional active faults that occur within the Anaheim area include the Newport-Inglewood fault zone to the west and the Whittier, Chino, San Jacinto, and San Andreas faults to the east. The closest known active faults to the site are the Whittier fault located approximately 8 miles (13 kilometers) to the northeast; the Newport-Inglewood fault zone located approximately 10.2 miles (16.5 kilometers) to the southwest, and the San Jacinto fault zone located approximately 37.5 miles (60 kilometers) to the northeast. The location of the site to the regional active faults is presented on Figure 3 - Regional Fault Location Map.

2.7.2 *Seismicity*

The main seismic parameters to be considered when discussing the potential for earthquake-induced damage are the distances to the causative faults, earthquake magnitudes, and expected ground accelerations. We have performed site-specific analysis based on these seismic parameters for the site and the onsite geologic conditions. The results of our analysis are discussed in terms of the potential seismic events that could be produced by the maximum probable earthquakes. A maximum probable earthquake is the maximum earthquake likely to occur given the known tectonic framework.

2.7.3 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 California Building Code (CBC). Representative site coordinates of latitude 33.8339º N and longitude -117.94º W were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 1 on the following page.

Table 1 California Building Code Site Seismic Characteristics	
Selected Parameters from 2016 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class per Chapter 20 of ASCE 7	D
Risk-Targeted Spectral Acceleration for Short Periods (S_S)*	1.50g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)*	0.574g
Site Coefficient F_a per Table 1613.3.3(1)	1.0
Site Coefficient F_v per Table 1613.3.3(2)	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_a S_S$]	1.500g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_v S_1$]	0.862g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.00g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.574g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	1.013
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	1.049

- From https://hazards.atcouncil.org/#/, 2019 Using ASCE-10

Section 1803.5.12 of the 2016 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCEG) Peak Ground Acceleration (PGA) should be used for geotechnical evaluations. The PGAM for the site is equal to 0.538g.

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 7.3 at a distance of approximately 10.4 km (6.46 mi) from the site would contribute the most to this ground motion.

2.7.4 ***Shallow Ground Rupture***

Based on our review of published maps, the site is not located within an Alquist-Priolo Earthquake Fault Zone. Although there are several nearby active and potentially active faults, it is our opinion that the potential for surface fault rupture impacting the site is very low.

2.7.5 ***Liquefaction***

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Liquefaction is typified by a buildup of pore-water pressure in the affected soil layer to a point where a total loss of shear strength occurs, causing the soil to behave as a liquid. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential.

Based on a review of seismic hazard zone map for the Anaheim Quadrangle prepared by the California Geological Survey (CGS, Seismic Hazard Zones-1998), the site is not located within a potential liquefaction seismic hazard area. Based on our limited geotechnical investigation and the relative density of the native on-site soils and the depth to the static groundwater across the site; it is our professional opinion that the potential for seismically induced liquefaction settlement is considered to be negligible.

2.7.6 ***Seismically Induced Settlement***

Based on our geotechnical analysis, seismically induced settlements may occur at the site. The estimation of potential seismic settlements is divided into two separate causative mechanisms: the dynamic settlement of dry coarse-grained soil above the groundwater table and seismic settlement below the groundwater from liquefaction. Dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event. The potential for dry sand settlement is considered low due to the relative density of the native on-site soils and recommended remedial removals for the site. We estimate the total amount of seismically-induced settlement (provided the remedial recommendations discussed herein are performed) is up to approximately 1-inch. The settlement analysis was performed on Borings B-1 and B-4 and was based on the procedures proposed by Tokimatsu and Seed (1987).

2.7.7 ***Tsunamis and Seiches***

Due to the elevation of the site with respect to sea level and its distance from large open bodies of water, the potential of seiches and/or tsunamis is considered to be nil.

2.8 ***Slope Stability***

No significant permanent slopes currently exist onsite or are planned for the subject site, therefore slope stability is not considered an issue with respect to site development.

2.9 ***Laboratory Testing***

Laboratory testing of the onsite soils was performed on representative samples obtained during our subsurface investigation. The testing included in-situ moisture and density tests (ASTM D2216), sieve and hydrometer analyses (ASTM D422), Atterberg limits (ASTM D4318), maximum dry density and optimum moisture (ASTM D1557), expansion index (ASTM D4829), corrosion suite (pH, resistivity, soluble sulfate content, and chloride content) (Caltrans Methods 422, 417, and 643), one-dimensional consolidation (ASTM D2435), and remolded direct shear (ASTM D3080).

Laboratory testing was performed by EGLAB, Inc. (EGL) and LGC, Inc. LGC has reviewed the laboratory test data with respect to the subject site and will take responsibility as geotechnical engineer of record for their work (laboratory testing). A discussion of the tests performed, and printout of the laboratory test results are presented in Appendix C. The moisture and density test results are presented on the boring logs in Appendix B.

A corrosion suite (pH, resistivity, and chloride content) was performed on a representative sample of the on-site soils. The test results indicate a minimum resistivity value of 2,300 ohm-centimeters, a pH value of 7.98, and a chloride content of 145 parts-per-million (ppm). Caltrans defines a corrosive area where any of the following conditions exist: the soil contains more than 500 ppm of chlorides, more than 2,000 ppm (0.2 percent) of sulfates, or a pH of 5.5 or less. On-site soils should be considered *mildly corrosive* to buried metals based on the resistivity values. These results/assumptions should be confirmed with finish grade corrosion testing performed along with the expansion testing upon completion of rough/precise grading to determine the actual corrosion potential for the subject lots. The test results are provided in Appendix C.

Based on the results of laboratory testing by EGL, the anticipated on-site undocumented fill soils are anticipated to have a "very low to low" expansion potential; however, based on the observed fine-grained content of the soils mapped as Quaternary young alluvium, there is a potential for medium expansive layers within that unit. Foundation design should be considered for very low to medium expansion potential and be designed for the anticipated static and seismic settlements. Finish grade expansion testing should be performed upon completion of the rough/precise grading to determine the expansion potentials for the subject building pads. Any expansive soil encountered during the grading operations should be mixed with less expansive soils and/or placed outside the limits of the proposed building pads.

3.0 ***INFILTRATION INVESTIGATION AND TESTING***

3.1 Scope of Infiltration Investigation

On March 14 and March 15, 2019, LGC performed a design-stage infiltration study at the proposed Lincoln at Euclid Development. Our infiltration study was based upon the locations and design of the proposed storm water infiltration facilities shown on the Preliminary Grading Plan (Huitt-Zollars, 2019a) and the project Water Quality Management Plan (WQMP) (Huitt-Zollars, 2019b).

The purpose of this investigation was to measure infiltration rates of the soils in the locations of the proposed infiltration facilities, and to determine the feasibility of infiltration based on the proposed design of the project and site-specific geologic/geotechnical conditions. The locations of the proposed infiltration facilities are shown on the Geotechnical Map (Figure 2). Percolation/infiltration testing was conducted in general accordance with Appendix VII of the Orange County Technical Guidance Document (TGD) (Orange County, 2013).

The infiltration study was performed using the Percolation Test Procedure described in Section VII.3.8. of the TGD. In each of the locations of the storm water infiltration facilities, one or two percolation/infiltration test borings were excavated to the proposed facility subgrade elevation, approximately 10 feet below the proposed finish grade elevation shown on the Preliminary Grading Plan (Huitt-Zollars, 2019). For each of the proposed infiltration facilities, one exploratory boring was also advanced and logged from the existing ground surface to a depth of approximately 10 feet below the bottom of the proposed infiltration elevation for the purpose of describing the encountered soils located along the sides and underlying the facility. The approximate locations of the percolation/infiltration tests and exploratory borings are presented on the Geotechnical Map (Figure 2). The infiltration test locations are labeled Infiltration Borings I-1 through I-5, and the exploratory borings are labeled Borings E-1 through E-3. The exploratory and infiltration boring logs are presented in Appendix D while the percolation test data sheets and infiltration results are presented in Appendix E. Worksheet H: Factor of Safety and Design Infiltration Rate; and Worksheet I: Summary of Groundwater-related Feasibility Criteria are presented in Appendix F.

3.2 Geologic Conditions for Infiltration

The three proposed infiltration facilities are located in areas mapped as Quaternary young alluvial fan deposits. This unit was found to vary in soil composition across the site. The alluvial fan deposits consist of fine-grained silty sand, silty clay, sandy silt, and clayey silt in the area of the eastern and central storm water infiltration facility locations, while the soils consisted of silty sand with less fine-grained material content in the area of the western facility. The varying amount of fine-grained material (silt and/or clay) within the site appears to affect the infiltration rates of the proposed basins across the site. During the first two 25-minute percolation test periods, the soils underlying the eastern and central facilities were classified as non-sandy soils and infiltrated at a slower rate than the tests within the western facility, which were determined during testing to be classified as sandy soils and infiltrated at a faster rate.

3.3 Percolation/Infiltration Testing

On March 14, 2019 our field exploration included drilling and logging infiltration test borings and exploratory test borings in the vicinity of all three of the proposed on-site storm water infiltration facilities. The infiltration test borings were excavated 10 feet below the current ground surface, to the approximate depth of the subgrade of the proposed facilities, as communicated to us by the civil engineer of record. The infiltration test locations are shown on Figure 2 and indicated by symbols I-1 through I-5. Logs of the advancement of these test borings are presented in Appendix D.

One exploratory boring was also advanced within each of the proposed facility and extended approximately 10 feet below the elevation of the bottom of the proposed facilities. Cuttings from these exploratory borings were logged by our field geologist for the purpose of describing/determining the soil profile. The infiltration exploratory boring logs are presented in Appendix D while the exploratory boring locations are shown on Figure 2 and indicated by boring symbols E-1 through E-3.

After the borings were excavated, 2 inches of pea gravel was placed in the bottom of the infiltration test borings. Three-inch perforated PVC pipe was placed in the test hole, and gravel was placed around the outside of the pipe to minimize disturbing the boring sidewalls during the testing. The borings were then pre-soaked by adding water to a level of approximately seven feet above the top of the proposed infiltration basin bottom. Two hours after water was added to test holes for pre-soaking, the geologist noted the water had completely drained from the test locations in the western portion of the site (Infiltration Borings I-4 and I-5), while the test locations in the central and eastern facilities still had water remaining in the holes. For the test locations in which the water had drained completely, the infiltration testing was conducted the same day. For locations that did not drain completely the geologist returned to the site the following day to conduct the testing.

Percolation/infiltration testing was conducted by filling the holes with clear water to a depth of approximately 7 feet above the bottom of the test hole. The water level was monitored during the test period to ensure that the depth of water never dropped below a height equal to 5 times the radius of the boring, in accordance with the Percolation Test Method.

Per the Percolation Test Method procedures outlined in the TGD, two initial 25-minute tests were conducted in each infiltration boring in order to classify the material being tested as either “sandy soils” (more than 6 inches of water seeps away in each of the initial 25-minute test periods) or “non-sandy soils” (less than 6-inches of water seeps away in each of the initial two 25-minute periods). After the initial test periods, I-1, I-2 and I-3 test holes were found to be in “non-sandy soils”, and percolation/infiltration tests were run for 6 hours with water level readings obtained at 30-minute intervals. Test holes I-4 and I-5 were found to be in “sandy soils” and percolation/infiltration tests were run for 1 hour with water level readings obtained at 10-minute intervals.

The measurement of the final 30-minute (non-sandy soils) and 10-minute (sandy soils) test period was used to determine the percolation rate for each of the tests, except in Infiltration Boring I-4, where infiltration rate was calculated using the final 20-minute period, as the measured rate in the final 10-minute period was significantly lower than those measured during the previous test periods. Conversion of the obtained percolation test result to an “infiltration rate” was then performed utilizing the Porchet Method. The infiltration test results and test data sheets are provided in Appendix E, and the final results are presented in Table 2 on Page 16.

3.4 ***Infiltration Factor of Safety***

Given the known potential for infiltration facilities to fail over time, a mandatory factor of safety is applied (Orange County, 2013). The minimum acceptable factor of safety in Orange County is 2. Per the TGD, the geotechnical engineer is to evaluate the site conditions based on Table VII.3: Site Suitability Considerations and Table VII.4: Design Related Considerations for Infiltration, in order to come up with an appropriate factor of safety. The factor of safety deemed appropriate for the project conditions is 2.25. Appendix F presents Worksheet H: Factor of Safety and Design Infiltration Rate.

3.5 ***Field Percolation/Infiltration Test Results***

The results of the percolation testing are summarized in Table 2. The observed infiltration rate column in the Table 2 is based on percolation test results in the field that were then converted to infiltration rates using the Porchet Method. For the central and western locations, two tests were run in each facility location. The average infiltration rate for each facility was calculated, and the adjusted infiltration rate was calculated by dividing the average observed infiltration rate by the factor of safety as determined in Worksheet H: Factor of Safety and Design Infiltration (Appendix F).

According to Section VII.2 of the TGD, infiltration is feasible if the measured infiltration rate (measured infiltration rate = observed infiltration rate in the field/factor of safety) is at least 0.3 inches per minute. Based upon our field testing the western basin is the only location where infiltration is considered feasible.

TABLE 2 SUMMARY OF PERCOLATION/INFILTRATION TESTING							
Infiltration Test Number	**Basin Location**	**Hole Diameter (inches)**	**Total Depth (feet)**	**Final Time Interval (min)**	**Observed Infiltration Rate (in/hr)**	**Average Rate for Basin (in/hr)**	**Adjusted Infiltration Rate (in/hr) (FS=2.25)**
I-1	East	8	10	30	0.14	0.14	0.06
I-2	Central	8	10	30	0.11	0.07	0.03
I-3		8	10	30	0.03		
I-4	West	8	10	10	1.13	0.8	0.36
I-5		8	10	10	0.47		

3.6 Geotechnical Design Considerations

According to the feasibility guidelines set forth in Section VII.2 in the TGD, only the western infiltration facility has infiltration rates over 0.3 in/hour, qualifying it as the only proposed facility location where infiltration is feasible. However, from a geotechnical standpoint, infiltration is feasible at all three proposed locations, as the facilities are not located directly adjacent to slopes or proposed building foundations. However, if infiltration is allowed in the central and eastern facilities, in an effort to minimize the potential for water migration from the infiltration facilities to the proposed utilities along permeable beds within the alluvial fan deposits, we recommend the bottom of the infiltration facility be positioned below the bottom of the adjacent utility trench.

3.7 Groundwater-Related Infiltration Feasibility

Infiltration facilities cannot be used where they would adversely affect groundwater quality or where depth to groundwater would limit infiltration. Per Appendix VIII of the TGD, the following factors must be considered when determining feasibility of onsite infiltration:

- Depth to groundwater and mounding potential,
- Presence of groundwater plumes,
- Wellhead protection and septic systems, and
- Contamination risks from land use activities in the area tributary to the BMP

As mentioned above in Section 2.5, Groundwater, groundwater at the site is thought to be approximately 100 feet below the ground surface. The potential for groundwater mounding is considered a potential impediment to infiltration in the condition of high groundwater (less than 10 feet below the infiltration surface) or shallow perched water. Given the depth to groundwater at the site, and given that we did not encounter any perched conditions in any of our multiple borings across the site to a depth of 20 to 51 feet, groundwater mounding is not considered to be a limiting factor for infiltration at this site.

The subject site is not located within the Plume Protection Boundary identified by the Orange County Water District, as presented on Figure VIII.2 within the TGD. There are no known wellhead or septic systems in the vicinity of the site, therefore, neither of these two factors are considered to be a limitation to infiltration.

According to Appendix VIII, infiltration is prohibited within 250 feet of contaminated sites found in the GeoTracker and EnviroStor databases unless a site specific study demonstrates that infiltration would not adversely impact groundwater conditions (Orange County, 2013). There are two known sites that have at one time been, or presently are considered to be contaminated. The first site is the LUST Cleanup site located on APN 072-110-21 (1631 West Lincoln Avenue). The second area with known contaminants is outside of the project site, on APN 072-110-49 (1683 West Lincoln Avenue). During the Phase 1 Site Assessment for this project, a past dry-cleaning operation was identified on this property (Roux, 2019). Roux Associates, Inc. performed a site specific study assessing the impact of proposed infiltration BMPs within 250 feet of the subject site and concluded that detected concentrations of contaminants in onsite soil samples do not indicate that storm water infiltration will cause adverse impacts to groundwater quality (Appendix G).

4.0 CONCLUSIONS

Based on the results of our geotechnical investigation, evaluation, and review, it is our professional opinion that the proposed site development is feasible from a geotechnical standpoint, provided the following recommendations included in this report are incorporated into the project plans and specifications, and followed during site grading and construction. Our geotechnical conclusions are as follows:

- Based on the subsurface exploration and our review, the site is underlain by undocumented fills and Quaternary young alluvial fan deposits. The upper approximately 5 to 7 feet of onsite soils are considered potentially compressible/collapsible and should be removed during grading operations. Undocumented fill removals in the area of the LUST cleanup site on Parcel 072-110-21 should be completely removed (approximately 10 below ground surface). During grading, the remedial removal depths should be verified in the field by a representative from LGC.

- The undocumented artificial fill encountered across the majority of Parcel 072-110-50 in the northwest portion of the site, extends to a depth between 2 and 6.5 feet below current ground surface elevations as encountered in the test pit excavations. Due to the amount of deleterious material, construction debris, and oversized concrete chunks, this fill should be either removed from the site, or be screened to remove the unsuitable material and oversized rock and/or concrete chunks. Any oversized rock or concrete chunks may be broken up on-site to less than 8 inches and used as fill, providing the amount of this material does not exceed 20 percent of the total amount of volume of engineered fill.

- The backfill soils associated with the LUST removal and remediation operations within Parcel 072-110-21 (i.e. 1631 West Lincoln Avenue) are considered undocumented, and should be removed to competent material and replaced with compacted documented fill. Assuming there are no environmental concerns, the pea gravel encountered within Boring B-6 and the other fill soils within the limits of the LUST removal excavation are suitable to be reprocessed/reused as directed by the geotechnical consultant and may be placed as engineered fill. During the advancement of Boring B-6, an underground structure was encountered 9 feet below ground surface. The nature and extent of this structure is unknown.

- We anticipate removals on the site to be on the order of approximately 5 to 7 feet below existing grade. The fill prism beneath the building footings should extend downward at a 1:1 (horizontal to vertical) slope from the outside edge of the footing bottoms. The removals should extend a minimum of 5 feet beyond the building perimeters.

- After the demolition and removal of existing structures on Parcel 072-110-21, we anticipate encountering additional undocumented fill across the site. Utility lines servicing the existing buildings should be completely removed in the areas of proposed settlement-sensitive structures. Removals should extend into competent material and be replaced with engineered fill.

- Groundwater was not encountered in the geotechnical borings advanced on site. According to our review, groundwater is in excess of 100 feet below ground surface.

- The site is <u>not</u> located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2018) and active or potentially active faults are not known to exist on the site.

- The PGA_M for the site is equal to 0.538g.

- The site is not located within an area deemed to have a potential for liquefaction (CDMG, 1997). Based on our evaluation of onsite soils encountered during advancement of the geotechnical borings, liquefaction is not a concern for the subject site.

- Based on our evaluation and analysis, the potential for seismically induced dry sand settlements to occur at the site is considered low. The differential seismically induced dry sand settlements of up to 0.5-inches should be considered in the foundation design.

- Foundations should be designed for soils with a range of very low to medium expansion potentials.

- Laboratory test results of the on-site soils indicate negligible soluble sulfates and are considered mildly corrosive to metals.

- Laboratory test results of the on-site soils indicate a negligible potential of hydro-collapse underlying the recommended remedial removals.

- The on-site soils below recommended remedial grading/excavation depths have a low potential for static settlement (i.e., slightly compressible).

- Based on our evaluations the proposed foundations should consist of post-tension or mat type slab on grade foundations.

- Based on our evaluation and analysis, static settlements for the residential townhome structures of up to 1-inch and a differential settlement of up to 0.5-inches should be considered in the design.

Our Site Percolation Conclusions are as follows:

- Based on data presented in this report for infiltration rate of representative onsite soils, it is our opinion that the infiltration rate measured in the Infiltration Test Borings of 0.06, 0.03, and 0.36 in/hr **are** representative of the upper onsite soils in the east, central, and west facilities, respectively.

- Based upon the results of the infiltration testing (measured infiltration results), the western facility is the only location where infiltration is considered feasible. The east and central facilities had measured infiltration rates below 0.3 in/hr, and therefore, are considered to be not feasible for infiltration per the TGD.

- Groundwater was not encountered to a maximum explored depth of 51.5 feet below the existing grade within the subject site and is not considered a concern for shallow site storm water infiltration design.

- Given the depth to groundwater at the site and given that we did not encounter any perched conditions in any of our multiple borings across the site from 20 to 51 feet, groundwater mounding is not considered to be a limiting factor for infiltration at this site.

- The subject site is not located within the Plume Protection Boundary identified by the Orange County Water District, as presented on Figure VIII.2 within the TGD. Therefore, this is not considered to be a limitation to infiltration.

- There are no known wellhead or septic systems in the vicinity of the site, therefore, this is not considered to be a limitation to infiltration.

- According to Appendix VIII, infiltration is prohibited within 250 feet of contaminated sites found in the GeoTracker and EnviroStor databases unless a site specific study demonstrates that infiltration would not adversely impact groundwater conditions (Orange County, 2013). There are two known sites that have at one time been, or presently are considered to be contaminated. The first site is the LUST Cleanup site located on APN 072-110-21 (1631 West Lincoln Avenue). The second area with known contaminants is outside of the project site, on APN 072-110-49 (1683 West Lincoln Avenue). During the Phase 1 Site Assessment for this project, a past dry-cleaning operation was identified on this property (Roux, 2019). Roux Associates, Inc. performed a site specific study assessing the impact of proposed infiltration BMPs within 250 feet of the subject site and concluded that detected concentrations of contaminants in onsite soil samples do not indicate that storm water infiltration will cause adverse impacts to groundwater quality (Roux, 2019).

- Any proposed stormwater infiltration system should meet the guidelines for proper infiltration with regard to setbacks from buildings, property lines and groundwater levels.

- Any proposed stormwater treatment system should be setback a minimum of 10 feet from the property line.

- Any water infiltration of stormwater runoff is not anticipated to adversely impact soil structure interaction, provided that the percolation area is setback a minimum of 10 feet from any building or wall foundations. Provided that the percolation area is setback a minimum of 10 feet from any building or wall foundations horizontally or vertically; proposed foundations will not be adversely impacted from expansive soils.

- The infiltration facility shall be designed to overflow to the street in the event that the drainage capacity is exceeded or in case of future failure to adequately infiltrate.

- Based upon the relative density of the on-site soils as encountered during this investigation, any water infiltration from a proposed storm water system is not anticipated to result in settlement or hydro-collapse to the soils underlying the site and therefore will not negatively impact any adjacent structures or improvements.
- The proper use and maintenance of the drainage systems are critical to maintain the useful design life per the guidelines set forth by the drain manufacturer.

5.0 RECOMMENDATIONS

5.1 Site Earthwork

We anticipate that earthwork at the site will consist of site preparation, remedial removals, and fill placement to achieve the configuration shown in the Preliminary Grading Plan for Lincoln at Euclid (Huitt-Zollars, 2019a). The rough grade operations will be followed by construction of a slab-on-grade type foundation, installation of utilities, and placement of the driveways, parking spaces, infiltration facilities, and concrete flatwork around the proposed buildings.

We recommend that earthwork onsite be performed in accordance with the recommendations herein, the recommendations provided by the City of Anaheim, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix H. In case of conflict, the recommendations in the following sections shall supersede those included as part of Appendix H.

5.1.1 Site Preparation

Prior to grading of the areas to receive structural fill, the ground surface should be cleared of obstructions and stripped of vegetation. The debris should be removed and properly disposed of offsite. Holes or depressions resulting from the removal of buried obstructions should be replaced with compacted fill.

Following remedial removals, areas to receive fill should be scarified to a minimum depth of 6 to 12-inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

5.1.2 Removal and Recompaction

The undocumented artificial fill encountered across the majority of Parcel 072-110-50 in the northwest portion of the site, generally extends to a depth between 3 and 7 feet below current ground surface elevations as encountered in the test pit excavations. Due to the amount of deleterious material, construction debris, and oversized concrete chunks, this fill should be either removed from the site, or be screened to remove the unsuitable material and oversized rock and/or concrete chunks. Any oversized material may be broken up on-site to less than 8 inches and used as fill, providing the amount of this oversized material does not exceed 20 percent of the total amount of volume of engineered fill.

Based on our geotechnical investigation, the upper 1 to 3 feet of the Quaternary-aged young alluvial fan deposits was found to be porous to slightly porous and potentially compressible. Based on the thickness of the encountered undocumented fill, and porous and potentially compressible upper portion of the alluvial soils, the anticipated removal depths are expected to range from approximately 5 to 7 feet across the site. The anticipated remedial removal depths are shown on the Geotechnical Map (Figure 2). Alluvial fan soils removed as part of remedial removals are considered suitable to be processed and placed as engineered fill under the observation and testing of LGC field personnel.

The backfill soils associated with the LUST removal and remediation operations within Parcel 072-110-21 (i.e. 1631 West Lincoln Avenue) are considered undocumented, and should be removed to competent material and replaced with compacted documented fill. Assuming there are no environmental concerns, the pea gravel encountered within Boring B-6 and the other fill soils within the limits of the LUST removal excavation are suitable to be mixed/reprocessed/reused as directed by the geotechnical consultant and may be placed as engineered fill. During the advancement of Boring B-6, an underground structure was encountered 9 feet below ground surface. The nature and extent of this structure is unknown.

After the demolition of existing on-site structures, we anticipate encountering additional undocumented fill across the site; as well as, during the abandonment and removal of underground utility lines. Utility lines servicing the existing buildings should be completely removed in the areas of proposed settlement-sensitive structures. Removals should extend into competent material and be replaced with engineered fill. From a geotechnical perspective, material that is removed may be placed as fill provided the material is free from rocks greater than 8-inches in maximum dimension, organic material and construction debris, is moisture-conditioned to obtain an above-optimum moisture content, and then recompacted prior to additional fill placement or construction.

5.1.3 Shrinkage/Bulking

Based on the site soils, bulking is not anticipated at the site. The preliminary estimated shrinkage factors of 10 to 20 percent for the undocumented fill and alluvium may be used for consideration of earthwork calculations. These are preliminary rough estimates which will vary with depth of removal, stripping losses, field conditions at the time of grading, etc. In addition, handling losses are not included in the estimates.

5.1.4 Temporary Excavation Stability

In general, all excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Soil conditions should be mapped and frequently checked by a representative of LGC to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical engineer should be maintained to facilitate construction while providing safe excavations. Excavation safety is the responsibility of the contractor.

Temporary excavations maybe cut vertically up to five feet. Excavations over five feet should be slot-cut, shored, or cut no steeper than 1H: 1V (horizontal, H: vertical, V) slope gradient.

Surface water should be diverted away from the exposed cut, and not be allowed to pond on top of the excavations. Temporary cuts should not be left open for an extended period of time.

Planned temporary conditions should be reviewed by the geotechnical consultant of record in order to reduce the potential for sidewall failure. The geotechnical consultant may provide recommendations for controlling the length of sidewall exposed.

5.1.5 *Fill Placement and Compaction*

From a geotechnical perspective, the onsite soils are suitable for use as compacted fill, provided they are screened of rocks greater than 8-inches in maximum dimension, organic material, and construction debris. In addition, we recommend that if highly expansive soils are encountered on the site, these soils should be placed outside the limits of the building pads or at least 5 feet below the proposed finish grade elevations; and replaced with a low expansive material. Areas prepared to receive structural fill and/or other surface improvements should be scarified to a minimum depth of 6 to 12-inches, brought to at least optimum-moisture content, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts generally not exceeding 8 inches in thickness. Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant.

5.1.6 *Trench Backfill and Compaction*

The onsite soils may generally be suitable as trench backfill provided they are screened of rocks and other material over 6 inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 8 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557).

5.2 *Foundation Selection*

5.2.1 *General Foundation Selection*

Recommendations for preliminary foundation design and construction are presented herein. Based on the result of the previous expansion potential testing of representative soils, the proposed structures should be designed for a very low to medium expansion potential. Final foundation design should be based on the results of the finish grade expansion potential testing at the completion of grading. The two foundation options recommended for the proposed structure are: 1) post-tension foundation; or 2) a mat slab.

The information and recommendations presented in this section are not meant to supersede design by the project structural engineer or civil engineer specializing in the structural design nor impede those recommendations by a corrosion consultant. Should conflict arise, modifications to the foundation design provided herein can be provided.

5.2.2 Soil Bearing

Proposed site at-grade improvements may be supported on spread footings provided that the earthwork recommendations outlined in this report are properly implemented. An allowable soil bearing pressure of 2,000 psf may be used for the design of footings placed in compacted fill having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment and 150 psf for each additional foot of foundation width to a maximum value of 3,500 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

Bearing values indicated above are for total dead loads and frequently applied live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

All footing excavations should be cut square and level as much as possible, and should be free of sloughed materials including sand, rocks and gravel, and trash debris. Subgrade soils should be pre-moistened for the assumed very low to medium expansion potential (to be confirmed at the end of grading). These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

5.2.3 Post-Tension Foundation

Based on the site geotechnical conditions and provided the remedial recommendations provided herein are implemented, the site may be considered suitable for the support of the anticipated structures using a post-tensioned slab-on-grade foundation system for very low to medium expansion potential (0-90 Expansion Index). The following section summarizes our recommendations for the foundation system. Table 2 contains the geotechnical recommendations for the construction of a PT slab-on-grade foundation.

TABLE 3

<u>Preliminary Geotechnical Parameters for Post-Tensioned Foundation Design</u>

Parameter	Value	
Expansion Classification (Assumed to be confirmed at the completion of grading):	Very Low to Low and Medium Expansion	
Thornthwaite Moisture Index (From Figure 3.3):	-20	
Constant Soil Suction (From Figure 3.4):	PF 3.6	
Center Lift Edge moisture variation distance (from Figure 3.6), e_m: Center lift, y_m:	**<u>Very Low to Low</u>** 9.0 feet 0.3 inches	**<u>Medium</u>** 9.0 feet 0.5 inches
Edge Lift Edge moisture variation distance (from Figure 3.6), e_m: Edge lift, y_m:	**<u>Low</u>** 5.1 feet 0.61 inches	**<u>Medium</u>** 5.1 feet 1.1 inches
Soluble Sulfate Content for Design of Concrete Mix in Contact with Site Soils in Accordance with American Concrete Institute standard 318, Section 4.3:	Assume Negligible Exposure (to be confirmed at the completion of grading)	
Corrosivity of Earth Materials to Ferrous Metals:	Mildly Corrosive	
Modulus of Subgrade Reaction, k (assuming presaturation as indicated below):	125 pci (very low to low) 85 pci (medium)	
Additional Recommendations: 1. Presaturate slab subgrade to at least optimum-moisture content or to 1.2 times optimum moisture, to minimum depths of 12 and 18 inches below ground surface, respectively for very low to low and medium expansion potentials. 2. Install a 15-mil moisture/vapor barrier (or equivalent) moisture/vapor barrier in direct contact with the concrete (unless superseded by the Structural/Post-tension engineer*) with 1 to 2 inches of sand below the moisture/vapor barrier. 3. Minimum perimeter foundation embedment below finish grade for moisture cut off should be 12 and 18 inches, respectively for very low to low and medium expansion potentials. 4. Minimum slab thickness should be 5 inches (or as determined appropriate by the structural engineer, per the structural design).		

* The above sand and Visqueen recommendations are traditionally included with geotechnical foundation recommendations although they are generally not a major factor influencing the geotechnical performance of the foundation. The sand and Visqueen requirements are the purview of the foundation engineer/corrosion engineer (in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction") and the homebuilder to ensure that the concrete cures more evenly than it would otherwise, is protected from corrosive environments, and moisture penetration of through the floor is acceptable to future homeowners. Therefore, the above recommendations may be superseded by the requirements of the previously mentioned parties.

5.2.4 Mat Foundation

A mat foundation can be used for support of proposed structure. An allowable soil bearing pressure of 1,000 psf may be used for the design of the mat at the ground surface under the slab area. The allowable bearing value is for total dead loads and frequently applied live loads and may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces. A coefficient of vertical subgrade reaction, k, of 85 pounds per cubic inch (pci) may be used to evaluate the pressure distribution beneath the mat foundation. The magnitude of total and differential settlements of the mat foundation will be a function of the structural design and stiffness of the mat.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. Foundations may be designed for a coefficient of friction of 0.35. Minimum perimeter footing embedment provided in the previous sections maybe reduced for the mat slab design.

Coordination with the structural engineer will be required in order to ensure structural loads are adequately distributed throughout the mat foundation to avoid localized stress concentrations resulting in potential settlement. The foundation plan should be reviewed by LGC to confirm preliminary estimated total and differential static settlements.

5.2.5 Foundation Settlement

Based on our current understanding of the project, the results of our site investigation and the recommended remedial grading with shallow foundations embedded into compacted fills, we estimate the post-construction static settlement of the site to be less than 1-inch with a differential settlement of approximately of 0.5-inches in 30 feet. Post-construction settlement should also include the estimated differential seismic settlement up to 0.5-inch in 30 feet.

5.2.6 Foundation Setbacks

All building foundation located close to slopes should have a minimum setback per Figure 1808.7.1 of the 2016 CBC. The setback distances should be measured from competent materials on the outer slope face, excluding any weathered and loose materials.

Per the 2016 CBC Section 1808.7.2 and Figure 1808.7.1, the building foundation constructed on or near a descending slope should be setback or deepened to provide a minimum footing setback equal to the total height of slope (H) divided by 3 (H/3). The footing setback should be a minimum of 5 feet for slopes up to 15 feet in height. The footing setbacks should be measured from the edge of the footing to the competent materials on the outer slope face. Where the slope is steeper than 1H:1V, the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

5.3 Lateral Earth Pressures for Retaining Walls

The following lateral earth pressures may be used for the design of any future site retaining walls. Due to the variable nature of the onsite soils, we recommend site retaining walls be backfilled with select soils or clean sand having a sand equivalence of greater than 30. Select soils should consist of clean, granular soils (less than 15 percent passing the No. 200 sieve) of very low to low expansion potential (expansion index 30 or less based on UBC. 18-2). The recommended lateral pressures for clean sand or approved select soils for level or sloping backfill are presented in Table 3.

Table 4

Lateral Earth Pressures for Retaining Walls

Conditions	**Equivalent Fluid Weight (pcf)**		
	Level Backfill	**2:1 Backfill Sloping Upwards**	**Seismic Earth Pressure (pcf) ***
	Approved Select Material	**Approved Select Material**	
Active	35	55	15
At Rest	55	80	--
Passive	250	150 (Sloping Down)	--

* For walls with greater than 6-feet in backfill height, the above seismic earth pressure should be added to the static pressures given in the table above. The seismic earth pressure should be considered as an inverted triangular distribution with the resultant acting at 0.6H in relation to the base of the retaining wall footing (where H is the retained height). The aforementioned incremental seismic load was determined in general accordance with the standard of practice in the industry for determining earth pressures as a result of seismic events.

Embedded structural walls should be designed for lateral earth pressures exerted on them. The magnitude of these pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for “at-rest” conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the “passive” resistance.

For design purposes, the recommended equivalent fluid pressure for each case for walls founded above the static groundwater and backfilled with low expansive onsite or import soils is provided in the table above. The equivalent fluid pressure values assume free-draining conditions. The backfill soils should be compacted to at least 90 percent relative compaction. The walls should be constructed and backfilled as soon as possible after back-cut excavation. Prolonged exposure of back-cut slopes may result in some localized slope instability. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from any adjacent structures should be evaluated by the geotechnical and structural engineers. Surcharge loading on retaining walls should be considered when any loads are located within a 1:1 (horizontal to vertical) projection from the base of the retaining wall and should be added to the applicable lateral earth pressures. Where applicable, a minimum uniform lateral pressure of 100 psf should be added to the appropriate lateral earth pressures to account for typical vehicle traffic loading.

All retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated on the attached Figure 5. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water, which contains soluble salts, migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

For sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads. For short term loading (i.e. seismic) the allowable bearing capacity may be increased by one-third for seismic loading.

Foundations for retaining walls in properly compacted fill should be embedded at least 18 inches below lowest adjacent grade. At this depth and a minimum of 12 inches in width, an allowable bearing capacity of 2,150 psf may be assumed. A factor of safety greater than 3 was used in evaluating the above bearing capacity value. This value maybe increased by 300 psf for each additional foot in depth and 150 psf for each additional foot of width to a maximum value of 3,500 psf. All excavations should be made in accordance with Cal OSHA. Excavation safety is the sole responsibility of the contractor.

5.4 Preliminary Pavement Recommendations

Based on an assumed R-value of 20 (considering the site soils types), we recommend the following preliminary minimum pavement sections for Traffic Indices of 4.5, 5, and 6 (Table 4). These recommendations should be confirmed with R-value testing of representative near-surface soils at the completion of grading. Final pavement sections should be confirmed by the project civil engineer based upon the projected Traffic Index.

Table 5 Preliminary Pavement Design Sections				
Location	**Traffic Index**	**Design R-Value**	**Asphalt Concrete Thickness (inches)**	**Aggregate Base Thickness (inches)**
Parking Spaces	4.5	20	3.0	6.0
Alleys/Driveways	5.0	20	3.0	8.0
Heavy Truck Lane	6.0	20	3.5	10.0

The aggregate base material should conform to the specifications for Class 2 Aggregate Base (Caltrans), Crushed Aggregate Base, or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction). The base material should be compacted to achieve a minimum relative compaction of 95 percent. The subgrade should achieve a minimum relative compaction of 90 percent through the upper 12 inches. Base and subgrade materials should be moisture-conditioned to relatively uniform moisture content at or slightly over optimum.

Aggregate base should conform to the requirements of the latest edition of the Standard Specifications for Public Works Construction ("Greenbook"). Aggregate base should be compacted to a minimum of 95 percent relative compaction over subgrade compacted to a minimum of 90 percent relative compaction per ASTM-D1557.

Portland Cement Concrete Pavement (PCCP) may be designed using a minimum of 8-inches of Portland cement concrete over 6-inches of compacted aggregate base. The modulus of rupture of the concrete should be a minimum of 500 pounds per square inch (psi) at 28 days. Contraction joints should be placed at maximum 10-foot spacing. Where the outer edge of a concrete pavement connects to an asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10.

The following recommendations are for vehicular concrete pavers designed for vehicular traffic and are underlain by 1-inch of sand. Based on ASCE 58-10 for interlocking pavers, considering a Traffic Index (TI) of 6.0 and a reasonable R-value of 20 for the subgrade soils, we recommend the following minimum base section underlying the proposed pavers. The proposed pavers and sand should be underlain by a minimum 12-inches of crushed aggregate base. The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction) and be place and compacted in maximum 6-inch thick lifts. The base material should be compacted to achieve a minimum relative compaction of 95 percent. The subgrade should achieve a minimum relative compaction of 90 percent through the upper 12 inches. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content near optimum moisture.

5.5 *Corrosivity to Concrete and Metal*

The National Association of Corrosion Engineers (NACE) defines corrosion as "a deterioration of a substance or its properties because of a reaction with its environment." From a geotechnical viewpoint, the "environment" is the prevailing foundation soils and the "substances" are the reinforced concrete foundations or various buried metallic elements such as rebar, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. ACI 318R-11 Table 4.3.1, provides specific guidelines for the concrete mix design when the soluble sulfate content of the soils exceeds 0.1 percent by weight or 1,000 ppm. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532.

Based on site soil testing, the onsite soils are classified as having a negligible sulfate exposure condition in accordance with ACI 318R-11 Table 4.3.1 (ACI, 2011). As a preliminary recommendation due to the results of sulfate content testing, concrete in contact with onsite soils should be designed in accordance with ACI 318R-11 Table 4.3.1 for the negligible category. It is also our opinion that based on soil resistivity measurements, the on-site soils should be preliminarily considered mildly corrosive to buried metals. Finish grade soils should be tested at the conclusion of grading to confirm these results. LGC is not a corrosion consultant and does not provide recommendations related to corrosion.

5.6 *Nonstructural Concrete Flatwork*

Concrete flatwork (such as walkways, etc.) have a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations because these slabs are typically much thinner than foundation slabs and are not reinforced with the same dynamic as foundation elements. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 5. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

Table 6 Nonstructural Concrete Flatwork				
	Private Sidewalks	**Private Driveways**	**Patio/Entryways**	**Sidewalk, Curb, and Gutter**
Minimum Thickness (in inches)	4	5	5	City/Agency Standard
Presaturation	Wet down subgrade soils prior to placement	Presoak to 12 inches	Presoak to 12 inches	City/Agency Standard
Reinforcement	--	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge	--	8” x 8”	--	City/Agency Standard
Crack Control	Saw cut or deep tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base	--	2	2	City/Agency Standard

5.7 Swimming Pool and Spa

The proposed pool, spa and associated improvements should be constructed in accordance with the attached Figure 6, Geotechnical Guidelines for Swimming Pool Construction. Pool excavation will occur in newly placed compacted fills and is anticipated to be relatively uniform. Consideration should be given to the medium expansive potential of onsite soils in design of the pool, and associated decking. Also concrete in contact with onsite soils should be designed in accordance with the negligible category of ACI 318R-11 Table 4.3.1.

Due to inherent differences in supporting capacity and expansive potential of different layers of the alluvium/fill, it is undesirable to have structures partially supported on soils having different geotechnical characteristics or materials having different engineering characteristics. If a cut/fill transition or expansive soil condition exists, the cut portion of the transition or expansive soil should be excavated (usually impractical for pool/spa construction), or the pool/spa can be designed with additional reinforcement and/or a thicker shell in order to cope with potential differences in supporting capacity and expansive potential.

Excavation and subsequent fill placement for pool including the placement of drains, outlets, water-proofing, etc. should be performed under the observation and testing of a geotechnical consultant. Observation and testing should be performed by the geotechnical consultant during pool excavation to verify that the exposed soil conditions are consistent with the design assumptions.

Concrete flatwork adjacent to the pool should be a minimum of 5 inches thick reinforced with No. 3 rebar at 18-inches on center each way with a 12-inch deep perimeter cut-off footing. Construction joints or weakened plane joints should be provided in all flatwork to a minimum depth of 1.5 inches at frequent internals (5 feet or less). The concrete slab should be underlain by a minimum of 4 inches of clean sand or base. Presoaking of the subgrade should be performed to a minimum depth of 12 inches. The subgrade should be inclined so that any moisture that seeps through cracks in the concrete due to irrigation, rain, or pool splash will be directed away from the pool.

5.8 <u>Control of Surface Water and Drainage Control</u>

Positive drainage of surface water away from structures is very important. No water should be allowed to pond adjacent to the building. The 2016 California Building Code, Section 1804.3 states that the ground immediately adjacent to the foundation should be sloped a minimum of 5-percent away from the building for a minimum distance of 10 feet and should further be diverted into a swale with a slope of at least 2-percent. If there is an impervious surface immediately adjacent to the foundation, the slope may be reduced to a 2-percent gradient. However, based on site soils, positive drainage may be accomplished by providing drainage away from the building at a gradient of at least 2-percent for a distance of at least 5 feet, and further maintained by a swale or drainage path at a gradient of at least 1-percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes.

5.9 Construction Observation and Testing

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC.

Geotechnical observation and testing should be performed by the geotechnical consultant during site excavations, subgrade for slab/foundation, backfill of utility trenches, preparation of any subgrade and placement of aggregate base, or when any unusual soil conditions are encountered at the site. Grading plans, foundation plans, and final project drawings should be reviewed by this office prior to construction.

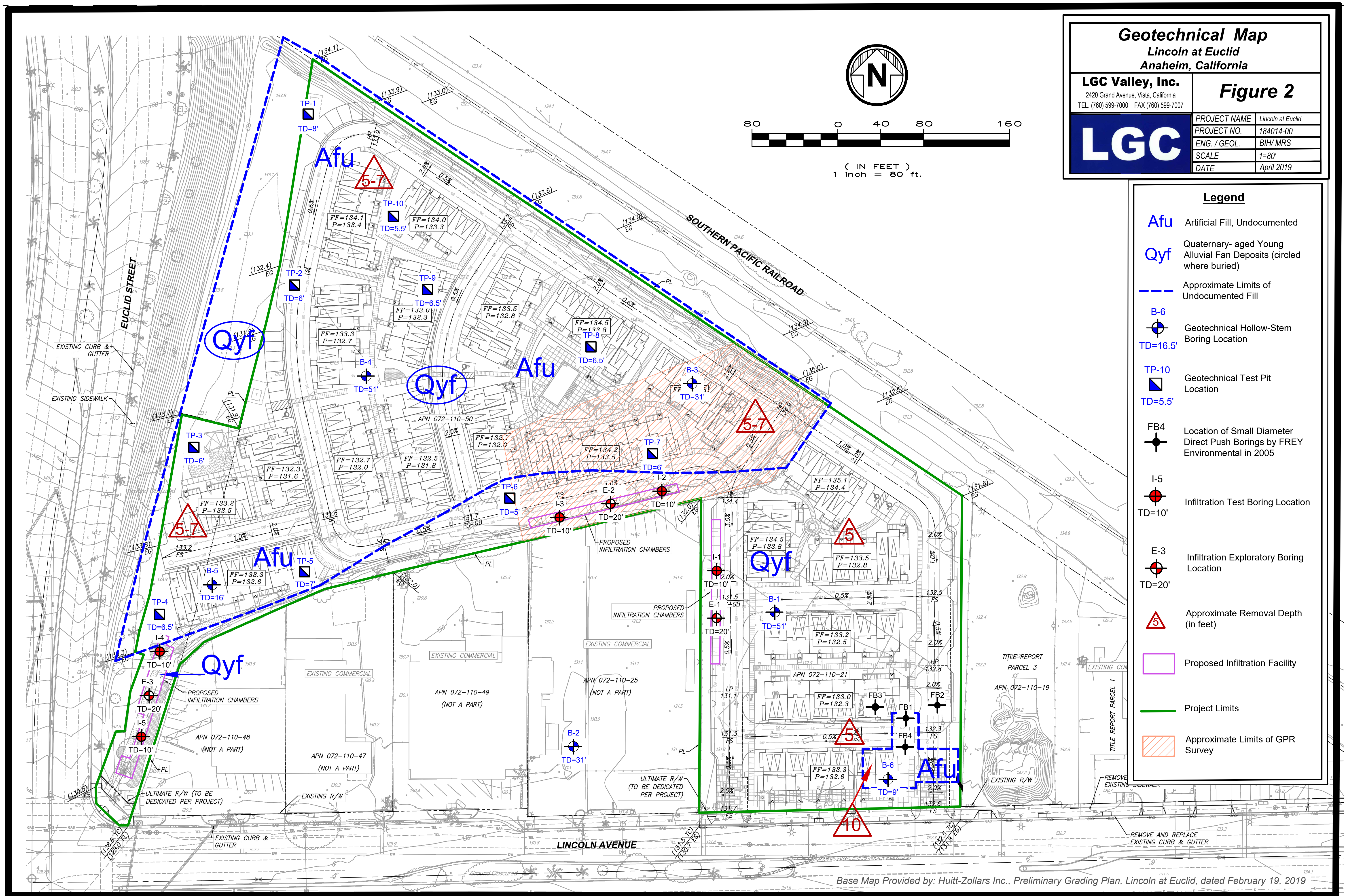
6.0 LIMITATIONS

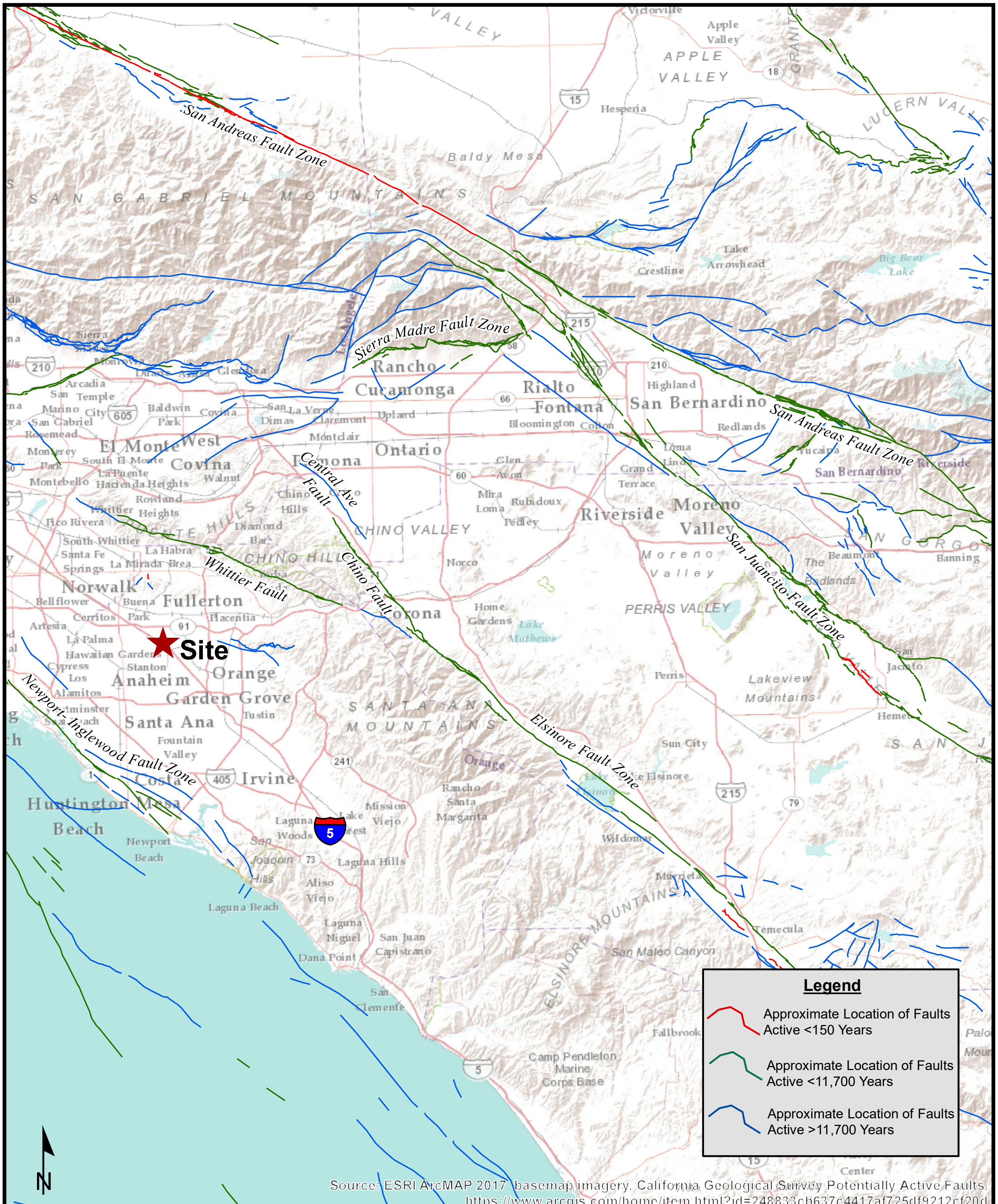
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made, and the in-situ field-testing performed are believed representative of the entire project; however, soil and geologic conditions revealed by excavation may be different from our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control.


Geotechnical Map
Lincoln at Euclid
Anaheim, California
LGC Valley, Inc.
2420 Grand Avenue, Vista, California
TEL. (760) 599-7000 FAX (760) 599-7007
LGC
Figure 2
PROJECT NAME Lincoln at Euclid
PROJECT NO. 184014-00
ENG. / GEOL. BIH/ MRS
SCALE 1=80'
DATE April 2019
N
80 0 40 80 160
(IN FEET)
1 inch = 80 ft.
Legend
Afu Artificial Fill, Undocumented
Qyf Quaternary- aged Young Alluvial Fan Deposits (circled where buried)
Approximate Limits of Undocumented Fill
B-6 TD=16.5' Geotechnical Hollow-Stem Boring Location
TP-10 TD=5.5' Geotechnical Test Pit Location
FB4 Location of Small Diameter Direct Push Borings by FREY Environmental in 2005
I-5 TD=10' Infiltration Test Boring Location
E-3 TD=20' Infiltration Exploratory Boring Location
5 Approximate Removal Depth (in feet)
Proposed Infiltration Facility
Project Limits
Approximate Limits of GPR Survey
SOUTHERN PACIFIC RAILROAD
EUCLID STREET
LINCOLN AVENUE
EXISTING CURB & GUTTER
EXISTING SIDEWALK
PROPOSED INFILTRATION CHAMBERS
EXISTING COMMERCIAL
APN 072-110-50
APN 072-110-49 (NOT A PART)
APN 072-110-48 (NOT A PART)
APN 072-110-47 (NOT A PART)
APN 072-110-25 (NOT A PART)
APN 072-110-21
APN 072-110-19
TITLE REPORT PARCEL 3
TITLE REPORT PARCEL 1
ULTIMATE R/W (TO BE DEDICATED PER PROJECT)
EXISTING R/W
REMOVE AND REPLACE EXISTING CURB & GUTTER
Base Map Provided by: Huitt-Zollars Inc., Preliminary Grading Plan, Lincoln at Euclid, dated February 19, 2019

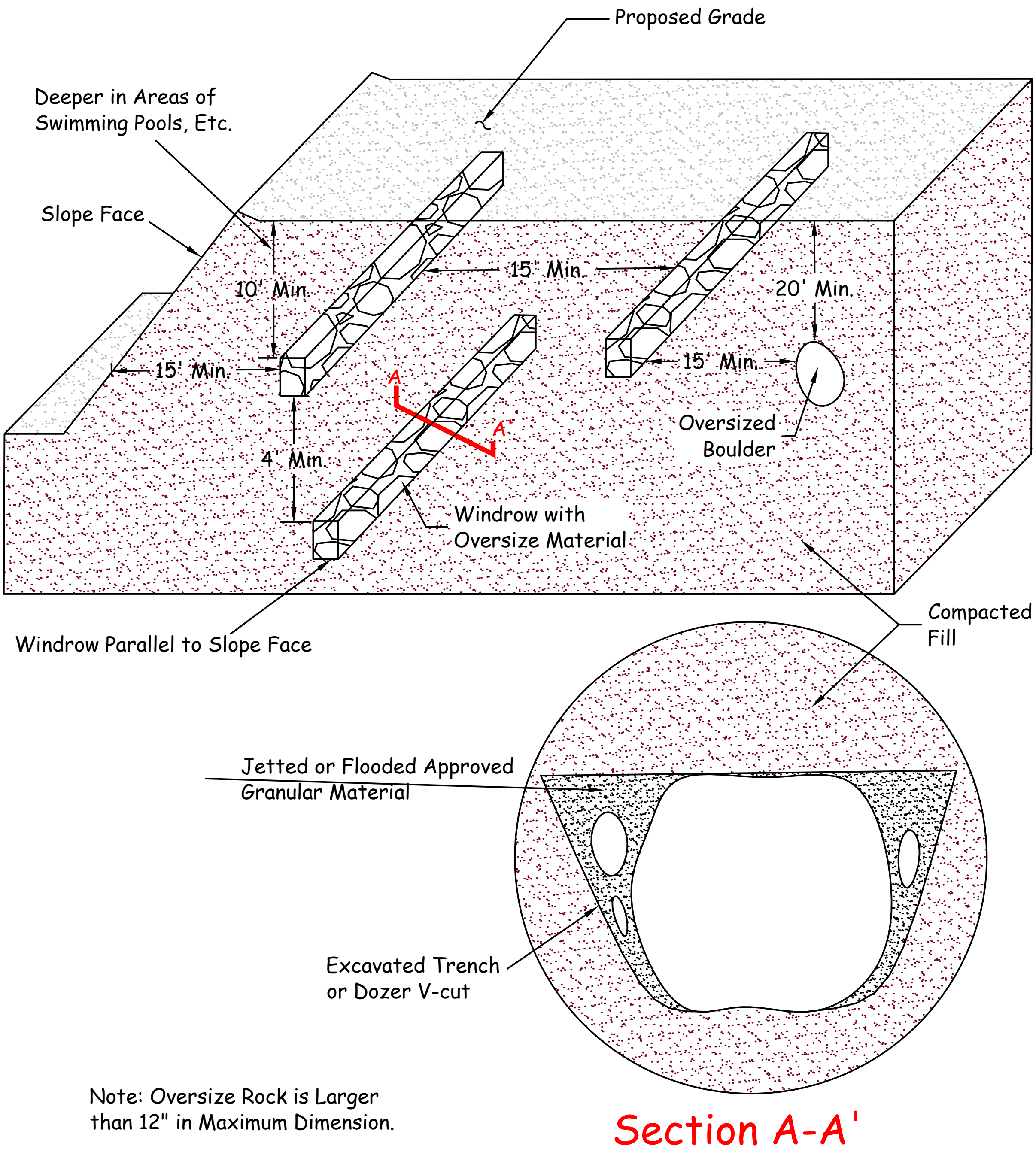


LGC

Figure 3
Regional Fault Location Map
City of Anaheim
Orange County, California

Project Name	Lincoln At Euclid
Project Number	184014-01
Eng./Geo.	BIH/MRS
Date	April 2019

0 5 10 Miles

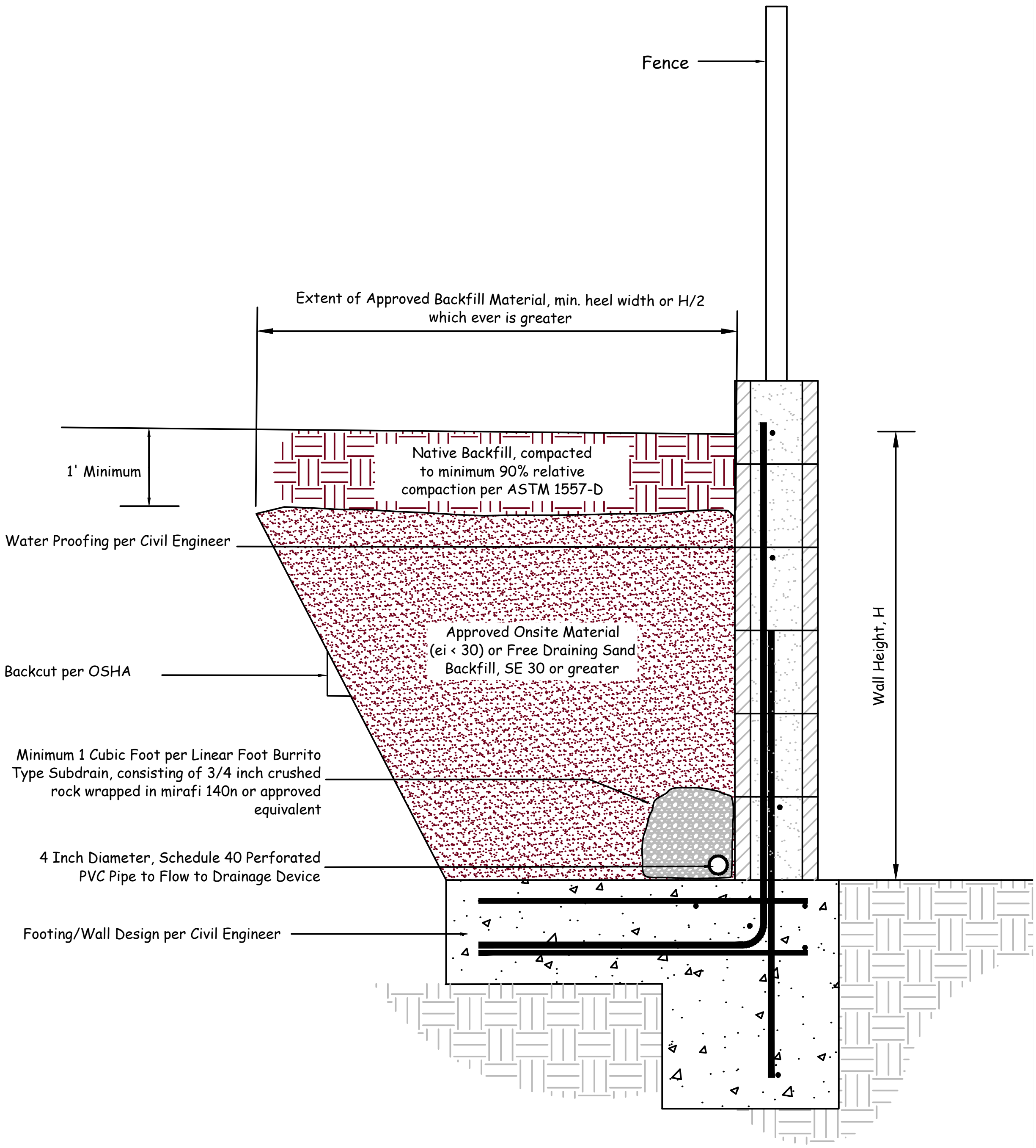


LGC

Figure 4

Oversize Rock Disposal Detail

Project Name	Lincoln at Euclid
Project No.	184014-00
Eng. / Geol.	BIH/MRS
Scale	NTS
Date	April 2019



LGC

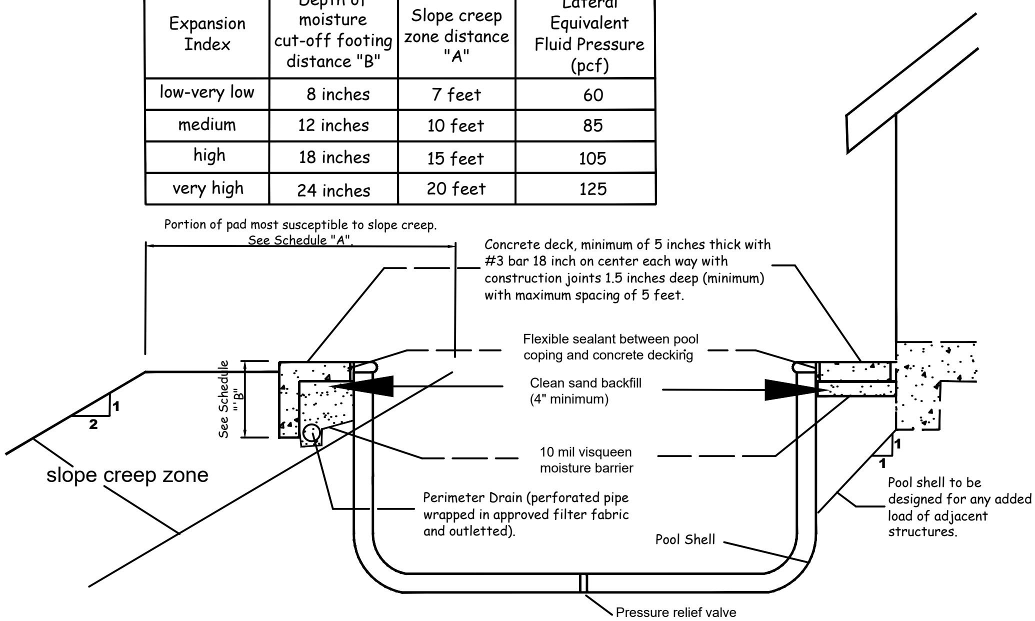
Figure 5

Retaining Wall Detail, Sand Backfill

Project Name	Lincoln At Euclid
Project No.	184014-00
Eng. / Geol.	BIH/MRS
Scale	NTS
Date	April 2019

SCHEDULE

Expansion Index	Depth of moisture cut-off footing distance "B"	Slope creep zone distance "A"	Lateral Equivalent Fluid Pressure (pcf)
low-very low	8 inches	7 feet	60
medium	12 inches	10 feet	85
high	18 inches	15 feet	105
very high	24 inches	20 feet	125



For pools adjacent to descending slopes, the pool shell should be designed assuming total loss of soil support for the portion of the pool located within the assumed "creep zone". For design purposes, the creep zone should be considered to extend a distance "A" from the top of slope (see schedule "A" above). The creep zone should be considered as parallel to the slope face.

Concrete flatwork adjacent to the pool should be a minimum of 5 inches thick reinforced with No. 3 rebar at 18-inches on center each way with a perimeter cut-off footing per the above schedule. Construction joints or weakened plane joints should be provided in all flatwork to a minimum depth of 1.5 inches at frequent internals (5 feet or less). The concrete slab should be underlain by a minimum of 4 inches of clean sand underlain inturn by a 10-mil Visqueen barrier. Presoaking of the subgrade prior to placing the Visqueen barrier should be performed in accordance with the recommendations included in the project geotechnical report. The presoaking should saturate the subgrade to a minimum depth of 12 inches. The subgrade below the Visqueen barrier should be inclined so that any moisture that seeps through cracks in the concrete due to irrigation, rain, or pool splash will be directed away from the pool. A perforated pipe wrapped in approved filter fabric should be installed to transport the collected moisture away from the pool area. The drain pipe is not considered necessary for soils of low to medium expansion potential. The contractor must ensure that the Visqueen is properly lapped, sealed and not punctured during construction.

All pool design should be performed by a qualified designer, using the equivalent fluid pressures shown in the schedule.

A geotechnical consultant should be contacted to review the final design which is based on the recommendations of this detail. This is not a design document and has been provided for INFORMATIONAL PURPOSES ONLY unless stamped and signed by LGC and pertaining to a specific pool.

To reduce the potential of lifting and cracking of the pool decking, landscape planters should not be located in islands within the decking unless they are lined with a waterproof membrane and provided with a subdrainage system to prevent moisture variations below the decking.

The pool shell should be designed to account for any additional loading due to improvements (building, raised planters, etc.)

Raised planters should not be located at the top of slopes unless specially designed by the geotechnical consultant.

The recommendations above will not eliminate all movement of the pool and associated improvements, however they should reduce the degree of movement, and promote cracking along construction joints, not flatwork.

LGC

Figure 6

Swimming Pool Detail

Project Name	Lincoln at Euclid
Project No.	184014-00
Eng. / Geol.	BIH/MRS
Scale	NTS
Date	April 2019

APPENDIX A

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APPENDIX B

Test Pit Logs by LGC
Boring Logs by LGC
Direct Push Logs by FREY Environmental

Boring and Test Pit Logs (continued)

<table>
<tr><th>Test Pit Number and Location</th><th>Depth and Description</th></tr>
<tr><td>TP-1

Elevation 131 feet

Location
N 33.834477°
E-117.940772°</td><td>UNDOCUMENTED ARTIFICIAL FILL (Afu):
@0’ Silty sandy CONGLOMERATE; medium gray brown, dry, medium dense; scattered roots and construction debris (concrete, AC, clay pipe); rock fragments generally 2-6” in size

@2.5’ 2” diameter PVC pipe (abandoned)

@3’ becomes damp

YOUNG ALLUVIAL FAN DEPOSITS (Qyf):
@ 5.5’ Silty fine SAND; dark gray brown, damp, medium dense; relatively homogeneous

Total Depth = 8 Feet
No Ground Water Encountered</td></tr>
<tr><td>TP-2

Elevation 129 feet

Location
N 33.834065°
E -117.940871°</td><td>UNDOCUMENTED ARTIFICIAL FILL (Afu):
@0’ Gravelly to cobbly silty fine SAND; medium gray to medium gray brown, dry, medium dense, minor construction debris

YOUNG ALLUVIAL FAN DEPOSITS (Qyf):
@3’ Silty fine SAND; medium gray brown, damp, medium dense; few subrounded to rounded gravels

@4’ Silty to very silty fine SAND, dark gray brown, damp, medium dense; slightly porous to 5.5’, pores generally 1/10th or less in diameter

@5’ bulk sample obtained
Total Depth = 6 Feet
No Ground Water Encountered</td></tr>
</table>

Boring and Test Pit Logs (continued)

Test Pit Number and Location	Depth and Description
TP-3 Elevation 130 feet Location N 33.833620° E -117.941016°	UNDOCUMENTED ARTIFICIAL FILL (Afu): @0' Gravelly silty fine SAND; medium gray brown, damp, medium dense, scattered rock fragments @2' scattered construction debris (brick, wood, AC) YOUNG ALLUVIAL FAN DEPOSITS (Qyf): @3' Silty fine SAND; dark gray brown, damp, medium dense; moderately porous, up to 1/8 – 1/16"diameter pores Total Depth = 6 Feet No Ground Water Encountered
TP-4 Elevation 131 feet Location N 33.833138° E -117.941105°	UNDOCUMENTED ARTIFICIAL FILL (Afu): @0' Gravelly silty fine SAND; medium gray and orange brown, dry, loose to medium dense @1' Silty to clayey SAND; orange to red brown, with abundant rock fragments up to 6-8" in diameter @2.5' 1-foot diameter concrete chunk @3' slightly gravelly silty fine SAND, medium gray brown, medium dense YOUNG ALLUVIAL FAN DEPOSITS (Qyf): @4.5' Silty fine SAND; dark gray brown, damp, medium dense to dense; very slightly porous to 6' Total Depth = 6.5 Feet No Ground Water Encountered

Boring and Test Pit Logs (continued)

<table>
<tr><th>Test Pit Number and Location</th><th>Depth and Description</th></tr>
<tr><td>TP-5

Elevation 132 feet

Location
N 33.833320°
E -117.940669°</td><td><u>UNDOCUMENTED ARTIFICIAL FILL </u>(Afu):
@0’ Silty sandy to clayey sandy GRAVEL; dark gray and orange brown, dry, medium dense; occasional 6-8” diameter rock fragments; minor construction debris

@2’ becomes damp; scattered sandy clay chunks

@4.5’ rebar piece

<u>YOUNG ALLUVIAL FAN DEPOSITS </u>(Qyf):
@5.5’ Silty fine SAND; dark gray brown, damp, medium dense; scattered iron oxide blebs and small black carbon chunks generally 1/16”or less in size; very slightly porous to 6.5’
@6.5’ slightly silty fine SAND, medium brown, damp, medium dense, to dense; homogeneous

Total Depth = 7 Feet
No Ground Water Encountered</td></tr>
<tr><td>TP-6

Elevation 130 feet

Location
N 33.833501°
E -117.940050°</td><td><u>UNDOCUMENTED ARTIFICIAL FILL </u>(Afu):
@0’ Silty sandy GRAVEL; medium gray, dry, medium dense; scattered concrete chunks up to 6-8”, slightly cemented due to concrete chunks and dust

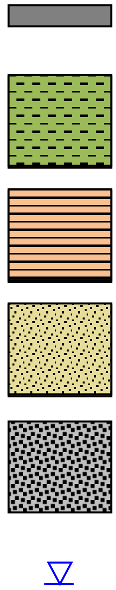
<u>YOUNG ALLUVIAL FAN DEPOSITS </u>(Qyf):
@1.5’ Silty fine SAND; minor medium sand; medium gray brown, dry, dense; slightly porous to 3’; few fine subrounded gravel
@3’ Silty fine SAND, dark gray brown, damp, medium dense
@4.5’ becomes medium brown
Total Depth = 5 Feet
No Ground Water Encountered</td></tr>
</table>

Boring and Test Pit Logs (continued)

Test Pit Number and Location	Depth and Description
TP-7 Elevation 130 feet Location N 33.833591° E -117.939608°	UNDOCUMENTED ARTIFICIAL FILL (Afu): @0' Silty sandy GRAVEL, recycled class II base; medium gray, dry, medium dense @1' becomes damp @2.5' silty fine to medium SAND, minor gravel and cobble sized rock fragments up to 6-8", one 4" thick by 12"x12" concrete chunk YOUNG ALLUVIAL FAN DEPOSITS (Qyf): @3.5' Silty fine SAND; medium to dark gray brown, damp, medium dense to dense, few subrounded fine gravel, homogeneous @5.5' fine SAND, medium brown, damp, dense Total Depth = 6 Feet No Ground Water Encountered
TP-8 Elevation 131 feet Location N 33.833885° E -117.939799°	UNDOCUMENTED ARTIFICIAL FILL (Afu): @0' Silty sandy GRAVEL; medium gray to medium gray brown, dry, medium dense to dense; moderately cemented; minor construction debris, brick, AC, a bic pen, black plastic Visqueen, PVC pipe pieces, etc. @1' becomes dark gray brown, damp @2' abundant AC and concrete chunks, 4-8" thick chunks, very difficult to excavate YOUNG ALLUVIAL FAN DEPOSITS (Qyf): @4' Silty fine SAND; dark gray brown, damp, medium dense; slightly porous to 5' @6' becomes medium brown silty fine SAND Total Depth = 6.5 Feet No Ground Water Encountered

Boring and Test Pit Logs (continued)

Test Pit Number and Location	Depth and Description
TP-9 Elevation 130 feet Location N 33.834030° E -117.940298°	UNDOCUMENTED ARTIFICIAL FILL (Afu): @0' Silty sandy GRAVEL; medium gray, damp, medium dense @0.5' Silty to clayey sandy CONGLOMERATE; orange brown, damp, dense; rock fragments up to 6" @3' becomes a gravelly silty fine SAND, light to medium gray, damp, dense; scattered sandy clay chunks; rock fragments up to 3-4"; bulk sample obtained YOUNG ALLUVIAL FAN DEPOSITS (Qyf): @5.5' silty fine SAND; medium gray brown and dark gray brown, damp, dense; very slightly porous Total Depth = 6.5 Feet No Ground Water Encountered
TP-10 Elevation 129 feet Location N 33.834267° E -117.940364°	UNDOCUMENTED ARTIFICIAL FILL (Afu): @0' Gravelly silty SAND; medium gray brown, dry, medium dense; few rock fragments up to 3-4" in diameter @2.5' abundant concrete chunks, up to 12"x18"x12", one red colored concrete chunk and AC; scattered small rebar and welded wire mesh pieces YOUNG ALLUVIAL FAN DEPOSITS (Qyf): @5' silty fine SAND; medium gray brown, damp, medium dense to dense Total Depth = 5.5 Feet No Ground Water Encountered



ASPHALT CONCRETE

CLAY

SILT

SAND

GRAVEL/COBBLES

APPROXIMATE GROUNDWATER LEVEL

Laboratory Test Symbols

Symbol	Laboratory Test
SA	Sieve Analysis
H	Hydrometer Analysis
SHA	Sieve & Hydrometer Analysis
-200	Percent Passing #200 Sieve
AL	Atterberg Limits
MAX	Maximum Density
DS	Undisturbed Direct Shear
RDS	Remolded Direct Shear
TRI	Triaxial Shear
EI	Expansion Index
P	Permeability
CN	Consolidation
COL	Collapse
UC	Unconfined Compression
S	Sulfate Content
pHR	pH & Resistivity
COR	Corrosion Suite (pH, Resistivity, Chloride, Sulfate)
RV	R-Value

Geotechnical Boring Log B-1

Date: June 6, 2018	Page: 1 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 132 Feet	Hole Location: See Map

Logged By: LF

Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
132	0							@0' Asphalt, 4"	
							ML-CL	**Young Alluvial Fan Deposits (Qyf):** @0.3' clayey SILT to silty CLAY; dark brown, damp to moist, medium stiff to very stiff	
			1	20	110.2	6.1			
127	5		2	20					CN
			3	34	110.9	13.9			
122	10		4	19					CN
			5	5				@12.5' becomes medium stiff to soft	SA
117	15		6	27	109.9	9.7	SP-SM	@15' fine SAND with minor silt; light orange brown, damp, medium dense	
			7	20					SA
112	20		8	21				@20' subrounded fine gravels present	CN
			9	10			SC	@22.5' fine clayey SAND; light brown, moist, loose	
107	25		10	29	116.2	13.2			
			11	9			CL	@27.5' sandy CLAY; medium brown, moist, stiff	SA
102	30								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

LGC VALLEY, INC.
THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS

Geotechnical Boring Log B-1

Date: June 6, 2018	Page: 2 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 132 Feet	Hole Location: See Map

DESCRIPTION

Logged By: LF

Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Description	Type of Test
102	30		12	15	109.7	19.8	CL	**Young Alluvial Fan Deposits Cont'd (Qyf):**	
			13	9					
97	35		14	21			SM	@35' fine silty SAND; medium brown, moist, medium dense	CN
			15	27			SP-SM	@37.5' fine SAND with silt; medium brown, moist, medium dense	
92	40		16	46					
			17	21					SA
87	45		18	49				@45' becomes coarse SAND; dense	
			19	30				@47.5' becomes fine SAND; medium dense to dense	
82	50		20	68					
								Total Depth = 51 Feet **No Groundwater Encountered** **Backfilled with Native Soil on June 6, 2018**	
77	55								
72	60								

LGC

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Geotechnical Boring Log B-2

Date: June 6, 2018	Page: 1 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 131 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
131	0						SM-SP	@0' Asphalt, 4" thick **Young Alluvial Fan Deposits (Qyf):** @0.4' fine silty SAND; light brown, moist, loose	
			1	12	93.1	13.2			
126	5		2	17	102.9	20.5	ML	@5' clayey SILT; brown, moist, stiff	
			3	11	93.5	27.4	CL	@7.5' silty CLAY; dark brown, moist, medium stiff	
121	10		4	18	78	21.5		@10' fine sandy CLAY; dark brown, moist, stiff	
			5	9			SP	@12.5' fine SAND; dark brown, moist, loose	
116	15		6	17			SP-SW	@15' fine to medium SAND; light brown, moist, medium dense	CN
			7	30				@17.5' becomes fine to coarse SAND; medium dense	
111	20		8	17			SM	@20' fine silty SAND; dark brown, moist, medium dense	
			9	9			ML	@22.5' fine sandy SILT; dark brown, moist, stiff	
106	25		10	16	116.2	13.2	SM	@25' fine silty SAND; dark brown, moist, loose	
			11	9			ML	@27.5' clayey SILT; dark brown, moist, stiff	
101	30								

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Geotechnical Boring Log B-2

Date: June 6, 2018	Page: 2 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 131 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number		Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
101	30		12		18			ML	**Young Alluvial Fan Deposits Cont'd (Qyf):** @30' fine sandy SILT, dark brown, moist, stiff	
									Total Depth = 31 Feet **No Groundwater Encountered** **Backfilled with Native Soil on June 6, 2018**	
96	35									
91	40									
86	45									
81	50									
76	55									
71	60									

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Geotechnical Boring Log B-3

Date: June 6, 2018	Page: 1 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 134 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Sample Type	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
134	0							SP-SM	**Undocumented Artificial Fill (Afu):** @0' Gravelly silty SAND; medium gray to medium gray brown, dry to damp, medium dense; scattered construction debris with pieces of concrete **Young Alluvial Fan Deposits (Qyf):**	
								SM	@4' fine silty SAND, light brown, moist, loose	
129	5		1	Ring	14					
			2	Ring	13	96.0	27.3	ML	@7.5' clayey SILT; light brown, moist, medium stiff to stiff	
124	10		3	Ring	16	109.3	16.7			
			4	Ring	12	103.7	21.1			
119	15		5	SPT	8					
			6	Ring	20	99.9	4.9	SP	@17.5' fine SAND; light brown, moist, medium dense	
114	20		7	SPT	18					
			8	Ring	39					
109	25		9	SPT	32			ML	@25' clayey SILT; dark brown, moist, hard	
			10	Ring	15			SP	@27.5' fine SAND; light brown, moist, loose	
104	30									

LGC

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Geotechnical Boring Log B-3

Date: June 6, 2018	Page: 2 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 134 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number		Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
104	30		11	☒	72			CL	**Young Alluvial Fan Deposits (Qyf) Cont'd:** @30' silty CLAY; dark brown, moist, hard	
99	35								**Total Depth = 31 Feet** **No Groundwater Encountered** **Backfilled with Native Soil on June 6, 2018**	
94	40									
89	45									
84	50									
79	55									
74	60									

LGC

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Geotechnical Boring Log B-4

Date: June 6, 2018	**Page: 1 of 2**
Project Name: Lincoln at Euclid	**Project Number: 184014-00**
Drilling Company: Baja Exploration	**Type of Rig: Hollow Stem Auger**
Drive Weight: 140 lbs.	**Drop: 30"** **Hole Dia: 8"**
Elevation of Top of Hole: 132 Feet	**Hole Location: See Map**

Logged By: LF
Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Sample Type	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
132	0							GP	**Undocumented Artificial Fill (Afu):** @0' Silty sandy GRAVEL; medium gray brown, damp, medium dense	
			1	Ring	27	102.2	5.6	ML	**Young Alluvial Fan Deposits (Qyf):** @2.5' fine sandy SILT; light brown, damp, stiff	
127	5		2	Ring	13			SM-ML	@5' silty SAND to sandy SILT; light brown, damp, medium dense to medium stiff	CN
			3	Ring	20	99.1	1.8	ML	@7.5' clayey SILT; light brown, moist, medium stiff to stiff	
122	10		4	Ring	28	103.3	22			
			5	Ring	13					CN
117	15		6	SPT	9			SC-CL	@15' fine clayey SAND to sandy CLAY; light brown, moist, medium dense to medium stiff	SA
			7	Ring	40	109.6	2.8	SP	@17.5' fine to medium SAND; light brown, moist, medium dense	
112	20		8	SPT	23					SA
			9	Ring	25			ML	@22.5' clayey SILT with minor sand; dark brown, moist, very stiff	
107	25		10	SPT	18			SC	@25' fine clayey SAND; dark brown, moist, medium dense	
			11	Ring	30	106.7	8.5	CL	@27.5' fine sandy CLAY; light brown, moist, very stiff	
102	30									

LGC

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Geotechnical Boring Log B-4

Date: June 6, 2018	Page: 2 of 2
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 132 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
102	30		12	8			ML	**Young Alluvial Fan Deposits (Qyf) Cont'd:** @30' clayey SILT; dark brown, moist, medium stiff to very stiff	
			13	30	109.5	19.5			
97	35		14	11					
			15	29					
92	40		16	9					
			17	16			CL	@42.5' silty CLAY; light brown, very moist, stiff	SA
87	45							@45' clayey SILT; light brown, very moist, stiff	
			18	8			ML		
								@47.5' fine clayey SAND; light brown, moist, medium dense	
			19	43			SC		
82	50		20	34				@50' decrease in clay, dense	
77	55							**Total Depth = 51 Feet** **No Groundwater Encountered** **Backfilled with Native Soil on June 6, 2018**	
72	60								

LGC

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Geotechnical Boring Log B-5

Date: June 6, 2018	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 133 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
133	0						SP-SM	**Undocumented Artificial Fill (Afu):** @0' fine silty SAND with subrounded fine gravels; dark brown, moist to damp, dense	
			1	56	110.8	4.1		**Young Alluvial Fan Deposits (Qyf):**	
128	5		2	35	105.1	6.1	ML	@5' fine sandy SILT; dark brown, damp, very stiff	
			3	29	103.6	2.4	SP	@7.5' fine SAND; light brown, moist, medium dense	
123	10		4	31	104.9	17.3	ML	@10' fine sandy SILT; dark brown, moist, stiff to very stiff	
			5	30					
118	15		6	23					
113	20							**Total Depth = 16 Feet** **No Groundwater Encountered** **Backfilled with Native Soil on June 6, 2018**	
108	25								
103	30								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

Geotechnical Boring Log B-6

Date: June 6, 2018	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 133 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
133	0						GP	@0' Asphalt, 4" thick **Undocumented Artificial Fill (Afu):** @0.3' Pea gravel encountered	
128	5								
								@9' refusal on hard unknown object (concrete slab?)	
123	10							**Total Depth = 9 Feet** **No Groundwater Encountered** **Backfilled with Native Gravel** **on June 6, 2018**	
118	15								
113	20								
108	25								
103	30								

LGC

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Date drilled/completed	May 2, 2005	Top of casing elevation	NA
Geologist	J. Moeller	Boring depth	Approx. 40 feet BGS
Drilling equipment	Direct Push, Geoprobe 6600	Water depth	NA
Surface elevation	Not surveyed	Well screen depth	NA

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
0									Backfill for tank cavity	
1										
2										
3										
4										
5								ML	Brown, damp SILT with some fine grained SAND	No petroleum hydrocarbon odor
6	ND<0.5	<1			-	5				
7										
8										
9										
10									No Sand, some Clay	
11	ND<0.5	<1			-	10				
12										
13										
14										
15								CL	Brown, damp CLAY, trace Silt, medium to low plasticity	
16	ND<0.5	<1			-	15				
17										
18										
19										
20								SP	Tan gray, dry, fine to medium grained SAND	
21	ND<0.5	<1			-	20				
22										
23										
24										
25										
26	ND<0.5	<1			-	25		ML	Brown, damp SILT with trace fine grained Sand	
27										
28										
29										▼
30										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB1	1

FREY ENVIRONMENTAL, INC.

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No	Graphic Log	U.S.C.S. Classification	Description	Remarks
30										No petroleum hydrocarbon odor
31	ND<0.5	<1			-	30		**ML**	Brown, damp SILT with trace fine grained SAND	
32										
33										
34										
35										
36	ND<0.5	<1			-	35				
37										
38										
39										
40								**SP**	Grayish tan, dry, fine to medium grained SAND	
41	ND<0.5	<1			-	40				▼
42									Bottom of boring at 40 feet BGS	
43										
44										
45										
46										
47										
48										
49										
50										
51										
52										
53										
54										
55										
56										
57										
58										
59										
60										

Project Name	**1631 W. LINCOLN AVENUE, ANAHEIM, CA**	Log of Boring	Figure No.
Project Number	**527-01**	**FB1**	2

***FREY** ENVIRONMENTAL, INC.*

Date drilled/completed	May 2, 2005	Top of casing elevation	NA
Geologist	J. Moeller	Boring depth	Approx. 40 feet BGS
Drilling equipment	Direct Push, Geoprobe 6600	Water depth	NA
Surface elevation	Not surveyed	Well screen depth	NA

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
0									Asphalt 6-inches thick	Hand Auger to 5 feet BGS
5	ND<0.5	<1			-	5		SP	Brown, dry, fine grained SAND with some Silt	No petroleum hydrocarbon odor
10	ND<0.5	<1			-	10			Becomes Siltier	
15	ND<0.5	<1			-	15			Trace amounts of Clay	
20	ND<0.5	<1			-	20			Tan gray, dry, fine to medium grained SAND	
25	ND<0.5	<1			-	25			Becomes fine grained and Silty	
30										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB2	1

FREY ENVIRONMENTAL, INC.

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
30										
31	ND<0.5	<1			-	30		SP	Light brown, dry, fine grained SAND	No petroleum hydrocarbon odor
32										
33										
34										
35										
36	ND<0.5	<1			-	35		ML	Brown, dry SILT with some fine grained SAND	
37										
38										
39										
40										
41	ND<0.5	<1			-	40		SP	Brown, damp, fine to medium grained SAND with trace Silt	▼
42									Bottom of boring at 40 feet BGS	
43										
44										
45										
46										
47										
48										
49										
50										
51										
52										
53										
54										
55										
56										
57										
58										
59										
60										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB2	2

FREY ENVIRONMENTAL, INC.

Date drilled/completed	May 2, 2005	Top of casing elevation	NA
Geologist	J. Moeller	Boring depth	Approx. 40 feet BGS
Drilling equipment	Direct Push, Geoprobe 6600	Water depth	NA
Surface elevation	Not surveyed	Well screen depth	NA

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
0									Asphalt 6-inches thick	Hand Auger to 5 feet BGS
1										
2										
3										
4										▼
5								ML	Brown, damp SILT with some fine grained Sand	No petroleum hydrocarbon odor
6	ND<0.5	<1			-	5				
7										
8										
9										
10										
11	ND<0.5	<1			-	10			No more Sand	
12										
13										
14										
15										
16	ND<0.5	<1			-	15			Trace amounts of Clay	
17										
18										
19										
20								SP	Tan grey, dry, fine to medium grained SAND	
21	ND<0.5	<1			-	20				
22										
23										
24										
25										
26	ND<0.5	<1			-	25			Becomes olive brown, fine grained only	
27										
28										
29										▼
30										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB3	1

FREY ENVIRONMENTAL, INC.

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
30										
31	ND<0.5	<1			-	30		ML	Brown, dry SILT with some fine grained SAND	No petroleum hydrocarbon odor
32										
33										
34										
35										
36	ND<0.5	<1			-	35		SP	Brown, dry, fine grained SAND with some Silt	
37										
38										
39										
40										
41	ND<0.5	<1			-	40			Becomes tan gray and fine to medium grained, no Silt	▼
42									Bottom of boring at 40 feet BGS	
43										
44										
45										
46										
47										
48										
49										
50										
51										
52										
53										
54										
55										
56										
57										
58										
59										
60										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB3	2

FREY ENVIRONMENTAL, INC.

Date drilled/completed	May 2, 2005	Top of casing elevation	NA
Geologist	J. Moeller	Boring depth	Approx. 40 feet BGS
Drilling equipment	Direct Push, Geoprobe 6600	Water depth	NA
Surface elevation	Not surveyed	Well screen depth	NA

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
0									Backfill for UST cavity	Hand Auger to 5 feet BGS
5	ND<0.5	<1			-	5		ML	Brown, damp SILT with some Clay and fine grained Sand	No petroleum hydrocarbon odor
10	ND<0.5	<1			-	10			More Clay present, no Sand encountered	
15	ND<0.5	<1			-	15		CL	Brown, damp CLAY with some Silt, medium plasticity	
20	ND<0.5	<1			-	20		SP	Tan gray, dry, fine to medium grained SAND	
25	ND<0.5	<1			-	25			Becomes fine grained only	
30										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB4	1

FREY ENVIRONMENTAL, INC.

Depth	EPA Method 8015 TPHg (mg/kg)	Headspace (ppmv)	Well Construction Detail	Sample Type	Blow Counts	Sample No.	Graphic Log	U.S.C.S. Classification	Description	Remarks
30								SP	Brown, dry, fine grained SAND with some Silt	No petroleum hydrocarbon odor
31	ND<0.5	<1			-	30				
32										
33										
34										
35										
36	ND<0.5	<1			-	35				
37										
38										
39										
40										
41	ND<0.5	<1			-	40			Becomes gray	▼
42									Bottom of boring at 40 feet BGS	
43										
44										
45										
46										
47										
48										
49										
50										
51										
52										
53										
54										
55										
56										
57										
58										
59										
60										

Project Name	1631 W. LINCOLN AVENUE, ANAHEIM, CA	Log of Boring	Figure No.
Project Number	527-01	FB4	2

FREY ENVIRONMENTAL, INC.

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and the results are presented on the following pages. LGC has reviewed the laboratory test data, procedures and results with respect to the subject site, concurs with, and accepts responsibility as geotechnical engineer of record for their work (laboratory testing).

Soil Classification: Soils were classified according the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. This system uses relies on the Atterberg limits and grain size distribution of a soil. The soil classifications (or group symbol) are shown on the laboratory test data and excavation logs.

Atterberg Limits: The liquid and plastic limits ("Atterberg limits") were determined in accordance with ASTM Test Method D4318 for engineering classification of fine-grained material and presented on the following table:

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Test Pit TP-2 #A @ 5'	24	18	6

Chloride Content: Chloride content was tested in accordance with CTM 422. The results are presented below:

Sample Location	Sample Description	Chloride Content (ppm)	Potential Degree of Chloride Attack*
Test Pit TP-2 #A @ 5'	Dark gray brown very silty fine SAND	145	Negligible

* Extrapolation from California Test Method 532, Method for Estimating the Time to Corrosion of Reinforced Concrete Substructures and previous experience.

Laboratory Testing Procedures and Test Results (continued)

Consolidation: Consolidation tests were performed on selected, relatively undisturbed ring samples (per Modified ASTM Test Method D2435). Samples (2.42 inches in diameter and 1 inch in height) were placed in a consolidometer and increasing loads were applied. The samples were allowed to consolidate under "double drainage" and total deformation for each loading step was recorded. The percent consolidation for each load step was recorded as the ratio of the amount of vertical compression to the original sample height. The consolidation pressure curves are presented on the attached figures at the end of this appendix.

Grain Size Distribution: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve. The portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D422 (CTM 202). Where an appreciable amount of fines were encountered (greater than 20 percent passing the No. 200 sieve) a hydrometer analysis was done to determine the distribution of soil particles passing the No. 200 sieve. The sieve and hydrometer curves are presented on the attached figures at the end of this appendix. The percent passing the #200 sieve is presented on the following table:

Sample Location	Sample Description	Percent Passing #200 Sieve
Boring B-1 #5 @ 12.5'	Dark brown silty CLAY to clayey SILT (CL-ML)	51
Boring B-1 #7 @ 17.5'	Light brown fine SAND with minor clay (SP-SM)	7
Boring B-1 #11 @ 27.5'	Medium brown fine sandy CLAY (CL)	84
Boring B-1 #17 @ 42.5'	Medium brown slightly silty fine SAND (SP-SM)	8
Boring B-4 #6 @ 15'	Light brown clayey very fine to fine SAND (SC)	42
Boring B-4 #8 @ 20'	Light brown fine to medium SAND (SP)	3
Boring B-4 #17 @ 42.5'	Light brown silty CLAY (CL)	93

Expansion Index Tests: The expansion potential of selected materials was evaluated by the Expansion Index Test, UBC Standard No. 18-I-B and/or ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential
Test Pit TP-2 #A @ 5'	Dark gray brown very silty fine SAND	20	Low

Laboratory Testing Procedures and Test Results (continued)

Direct Shear (Remolded or Undisturbed): Direct shear tests were performed on selected remolded and/or undisturbed samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inch per minute (depending upon the soil type). The test results are presented on the following table and/or on the attached figures at the end of this appendix.

Sample Location	Sample Description	Peak/Ultimate Friction Angle (degrees)	Peak/Ultimate Apparent Cohesion (psf)
Boring B-1 #A @ 2.5 to 5’	Dark brown clayey fine SAND	237/90	30/30

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented on the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Maximum Dry Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM Test Method D1557. The results of these tests are presented in the table below:

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Boring B-1 #A @ 2.5 to 5’	Dark brown clayey fine SAND	131.0	8.5

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. As results of soil's resistivity decreases corrosivity increases. The results are presented in the table below:

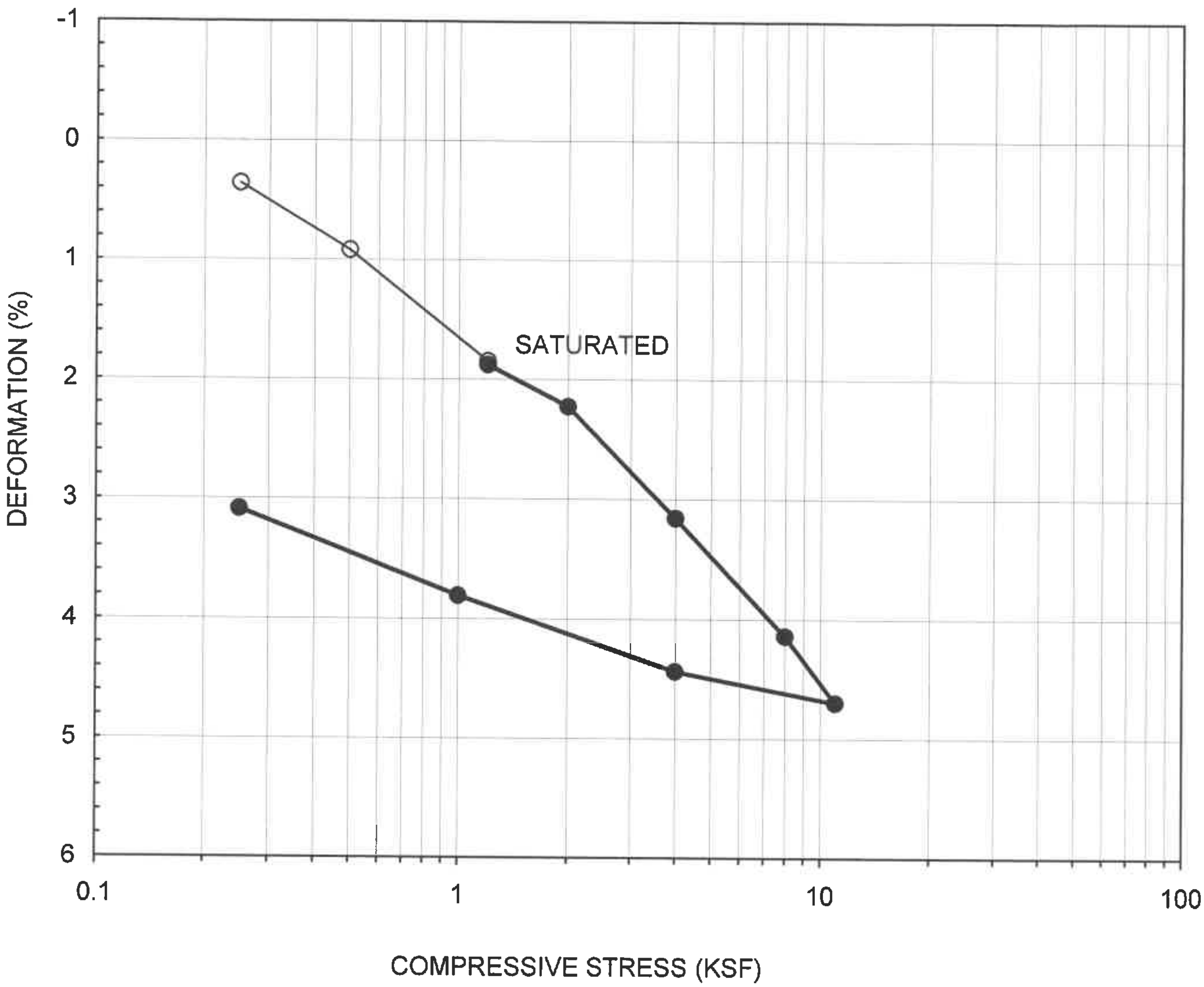
Sample Location	Sample Description	pH	Minimum Resistivity (ohms-cm)	Potential Degree of Corrosivity*
Test Pit TP-2 #A @ 5'	Dark gray brown very silty fine SAND	7.98	2,300	Mildly Corrosive

* NACE Corrosion Basics

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below:

Sample Location	Sample Description	Sulfate Content (% by weight)	Potential Degree of Sulfate Attack*
Test Pit TP-2 #A @ 5'	Dark gray brown very silty fine SAND	0.004	Negligible

* Per ACI 318R-08 Table 4.3.1 (ACI, 2008).



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	2	5.0	ML	12.5	112.1	0.503

EGLAB, INC.

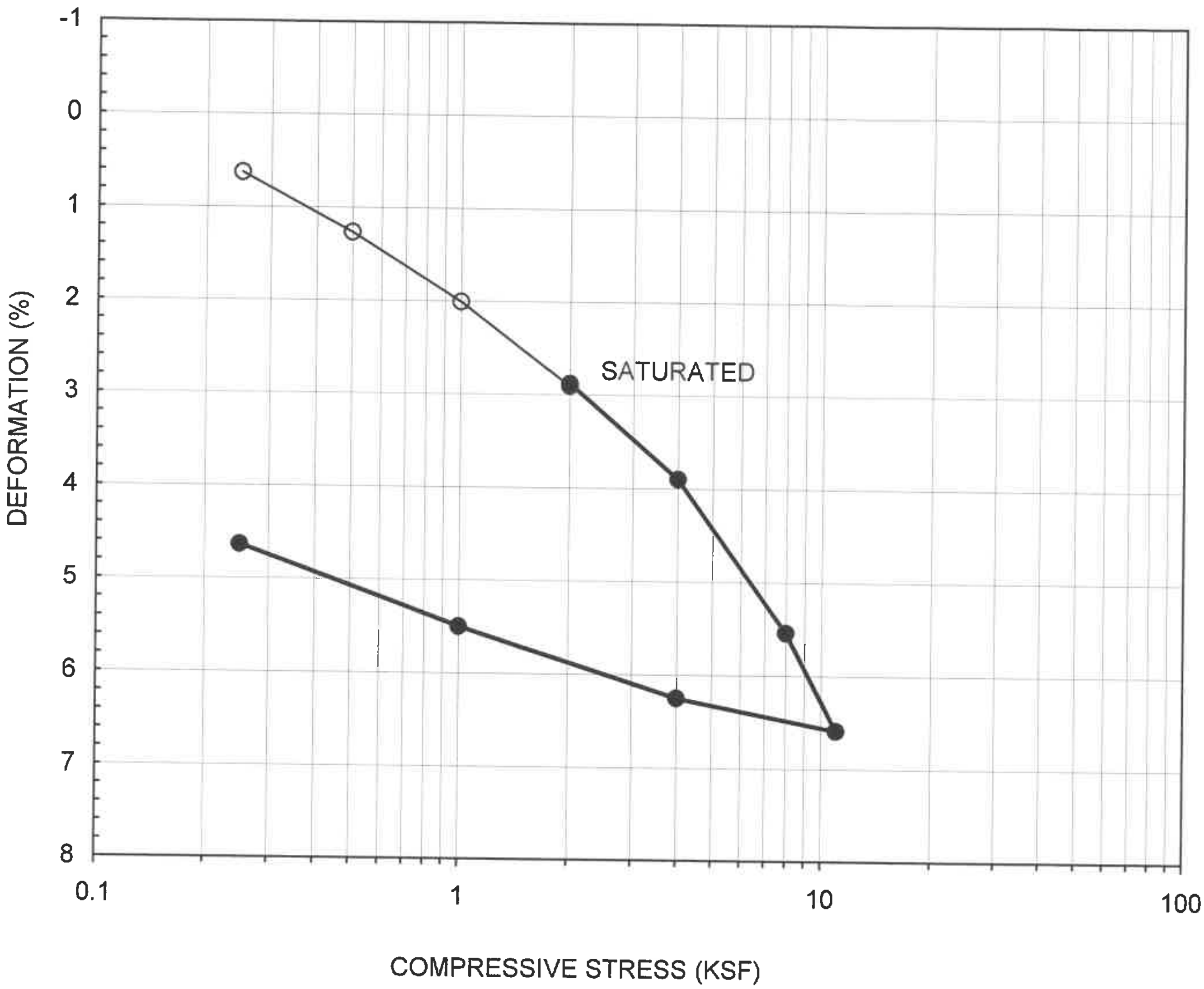
Project Name: Shopoff Anaheim 2.
Client: LGC Valley, Inc.
Job No.: 184014-00
EGLAB Project No.: 18-059-014

CONSOLIDATION

(ASTM D2435)

06/18

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	4	10.0	CL	24.2	105.4	0.599

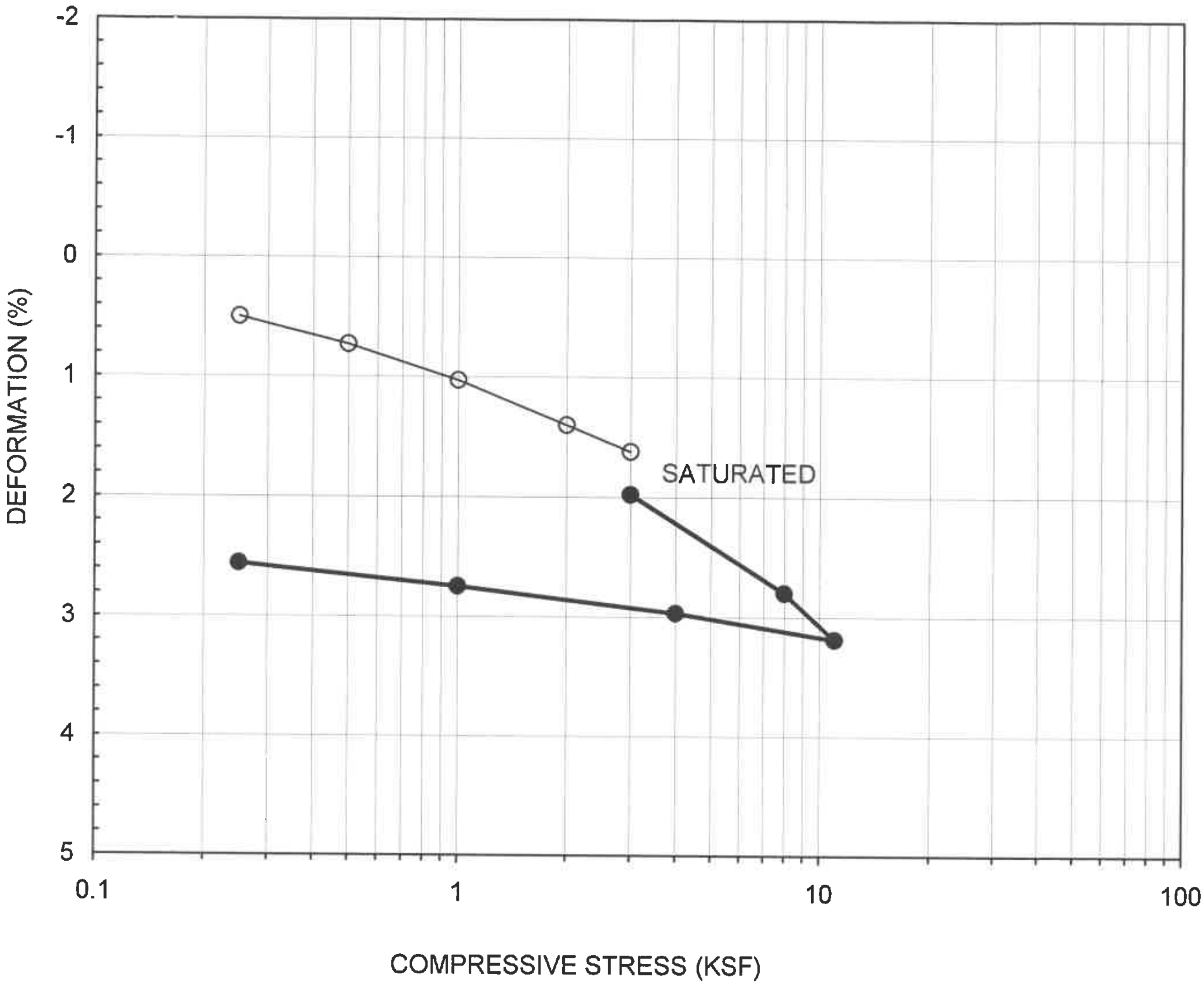
EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client: LGC Valley, Inc. Job No.: 184014-00 EGLAB Project No.: 18-059-014

CONSOLIDATION

(ASTM D2435)

06/18

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	8	20.0	SP	1.9	107.3	0.570

EGLAB, INC.

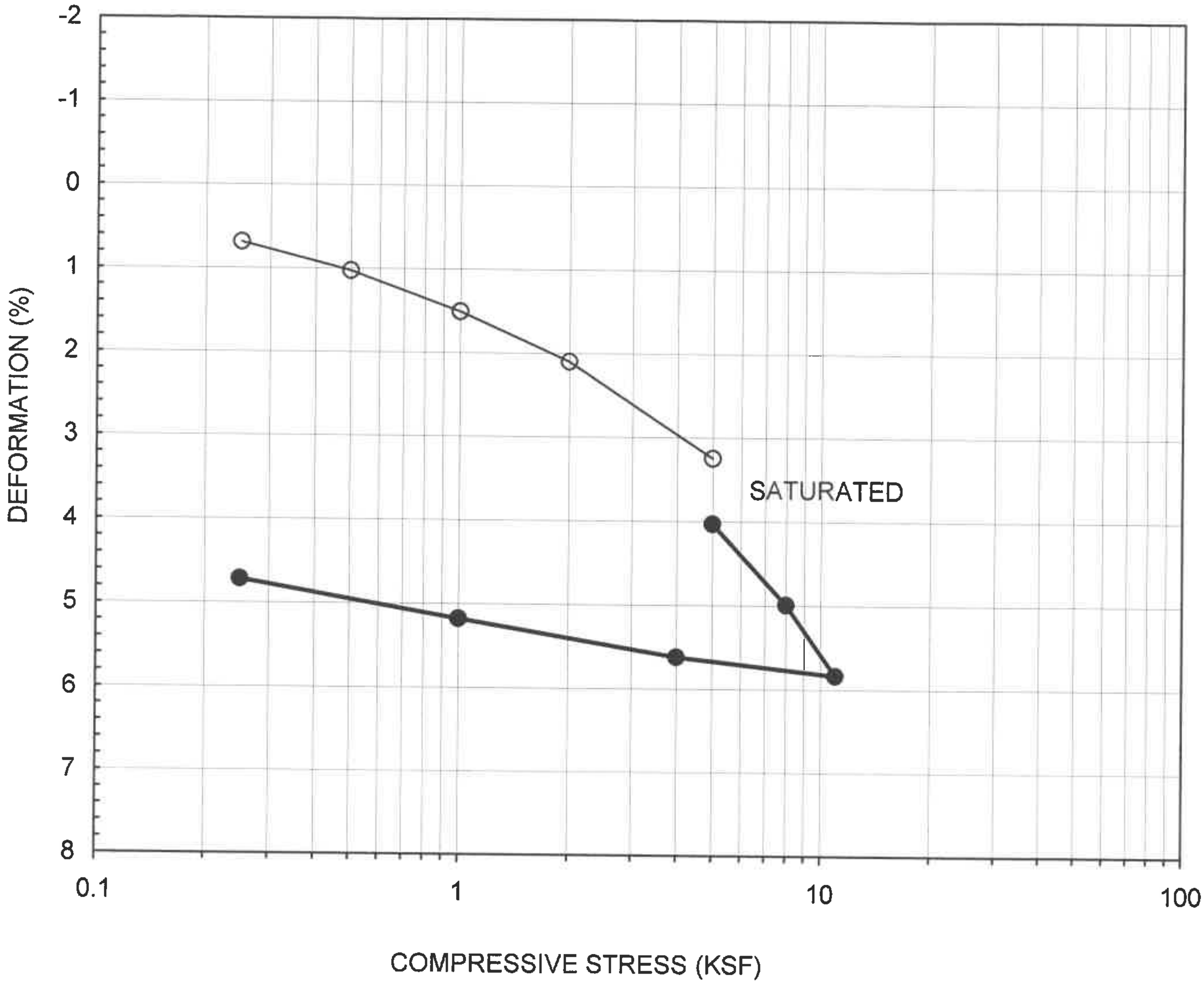
Project Name:
Shopoff Anaheim 2.
Client: LGC Valley, Inc.
Job No.: 184014-00
EGLAB Project No.: 18-059-014

CONSOLIDATION

06/18

(ASTM D2435)

Figure

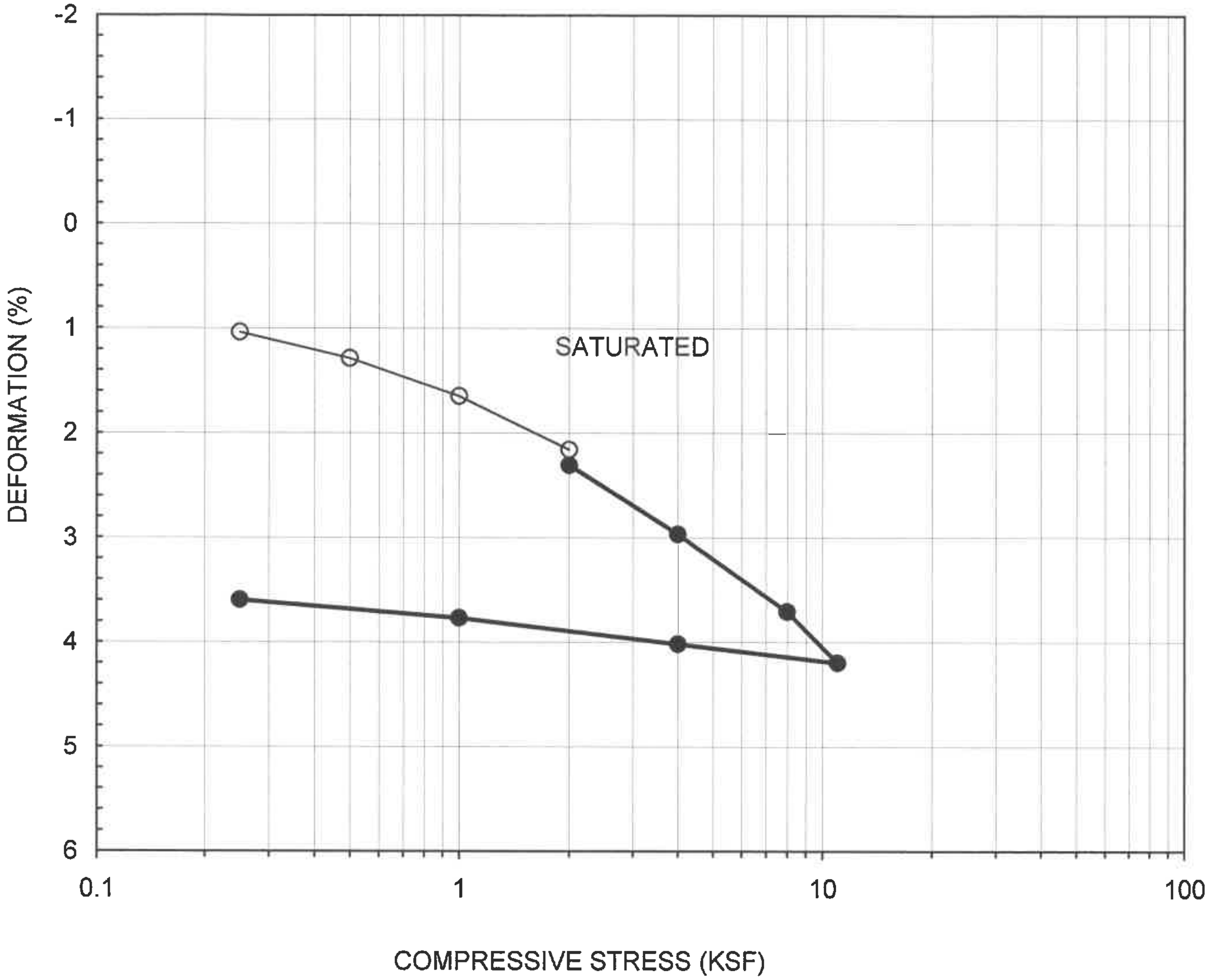


Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	14	35.0	SM	11.4	102.5	0.644

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client: LGC Valley, Inc. Job No.: 184014-00 EGLAB Project No.: 18-059-014

CONSOLIDATION

06/18 (ASTM D2435) Figure



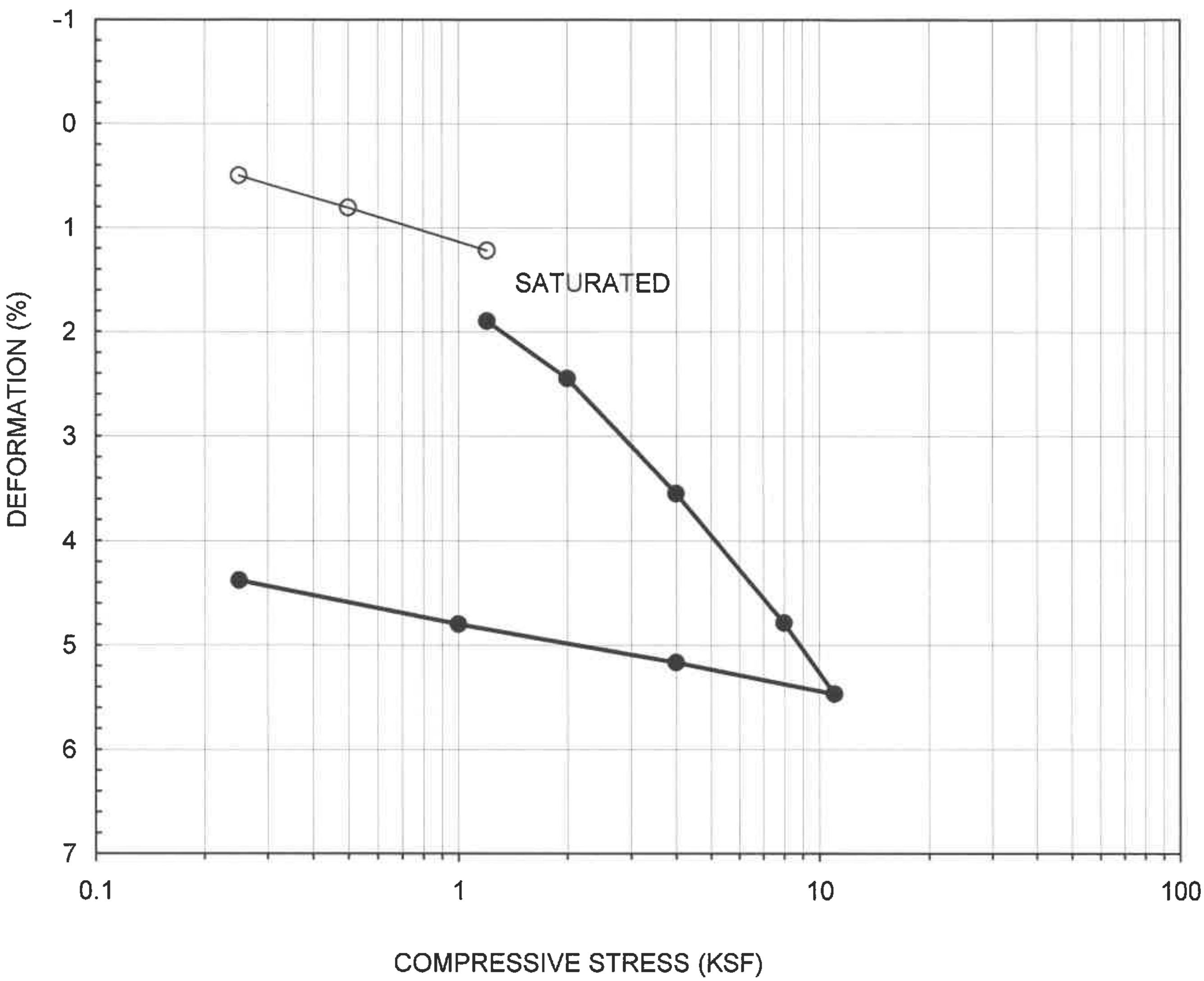
Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-2	6	15.0	SP-SM	3.4	107.2	0.572

EGLAB, INC.

Project Name: Shopoff Anaheim 2.
Client: LGC Valley, Inc.
Job No.: 184014-00
EGLAB Project No.: 18-059-014

CONSOLIDATION

06/18 (ASTM D2435) Figure

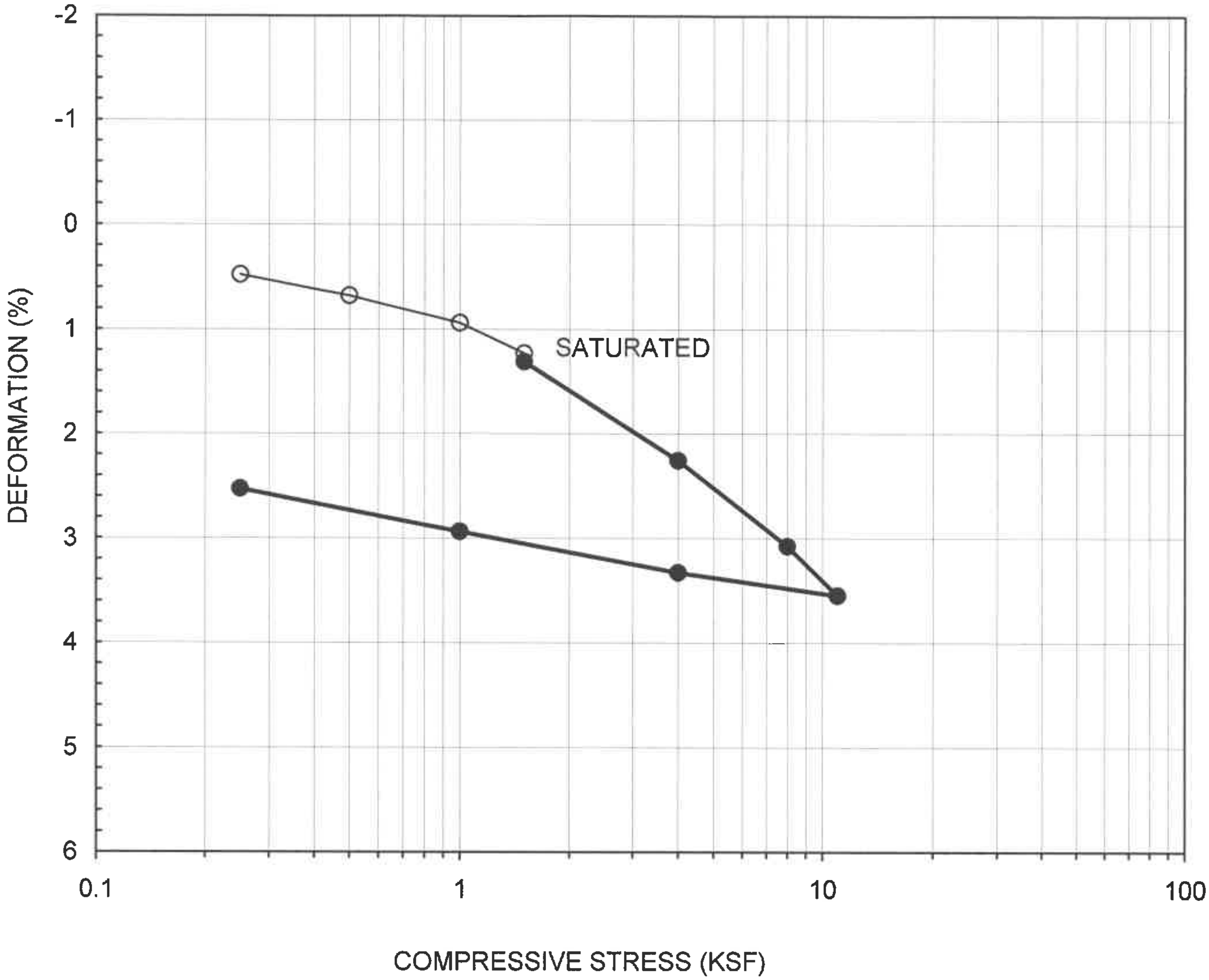


Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-4	2	5.0	SM	4.7	106.4	0.584

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client: LGC Valley, Inc. Job No.: 184014-00 EGLAB Project No.: 18-059-014

CONSOLIDATION

06/18 (ASTM D2435) Figure



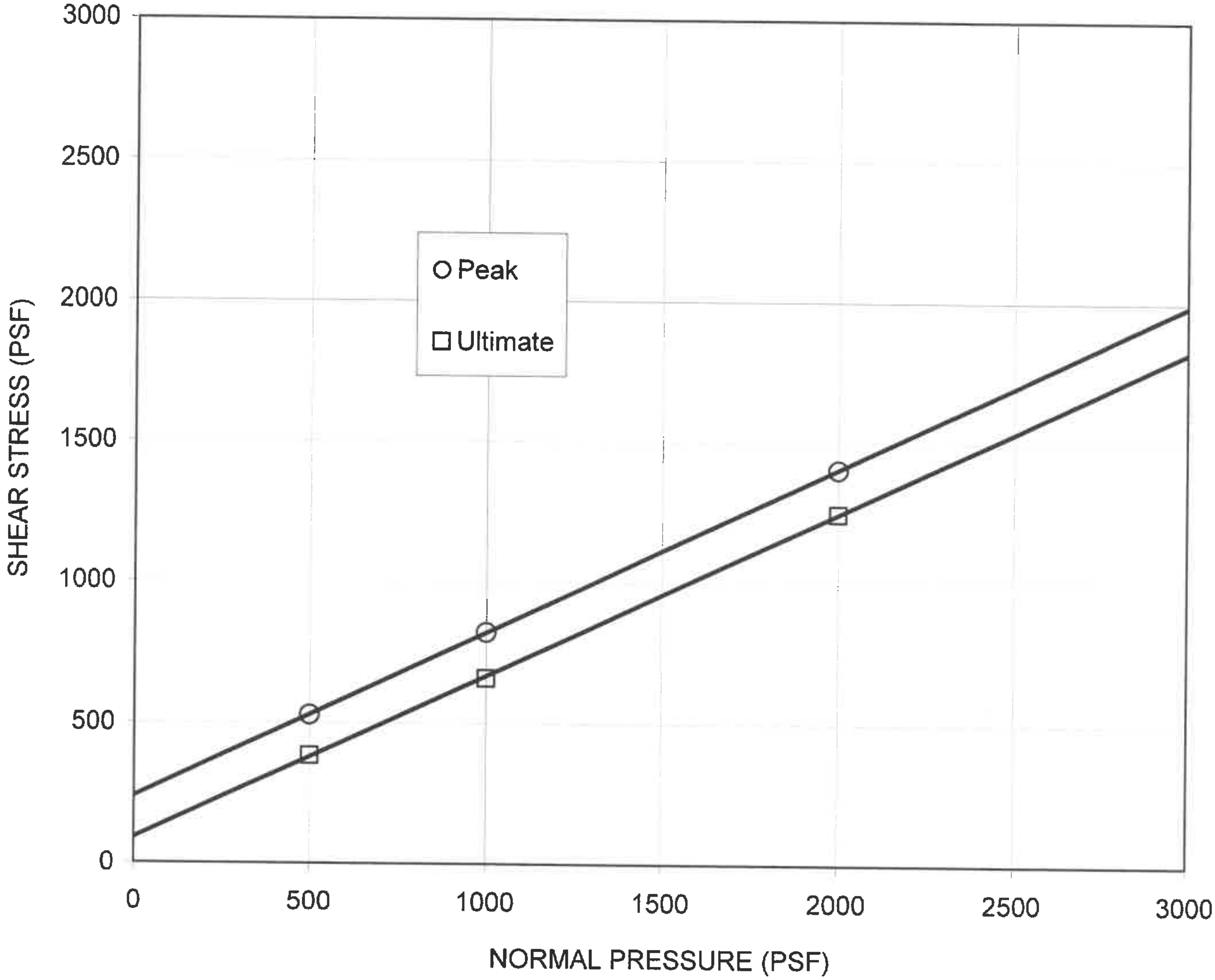
Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-4	5	12.5	ML	16.8	114.2	0.475

Project Name:
Shopoff Anaheim 2.
Client: LGC Valley, Inc.
Job No.: 184014-00
EGLAB Project No.: 18-059-014

EGLAB, INC.

CONSOLIDATION

06/18 (ASTM D2435) Figure



Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-1	A	2.5-5	Bulk	SC	○	237	30
					□	90	30

Note: Sample was remolded to 90% maximum relative density and optimum moisture

Maximum Dry Density: 131.0 pcf   Optimum Moisture: 8.5 %

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)
500	8.5	16.0
1000	8.5	15.8
2000	8.5	15.5

EGLAB, INC.

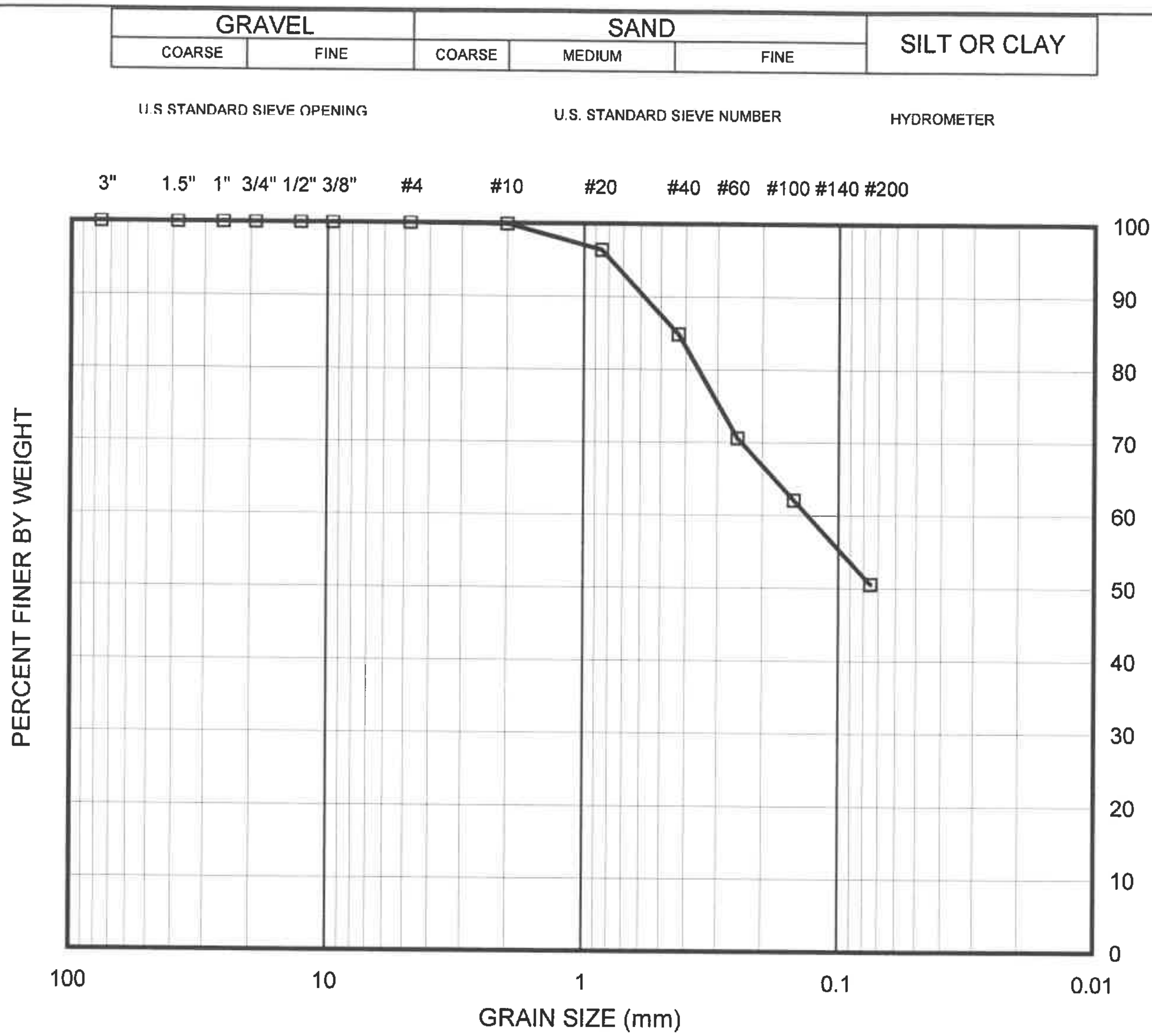
Project Address:
Shopoff Anaheim 2.
Client: LGC Valley, Inc.
Project No.: 184014-00
EGLAB Project No.: 18-059-014

DIRECT SHEAR

(ASTM D3080)

06/18

Figure

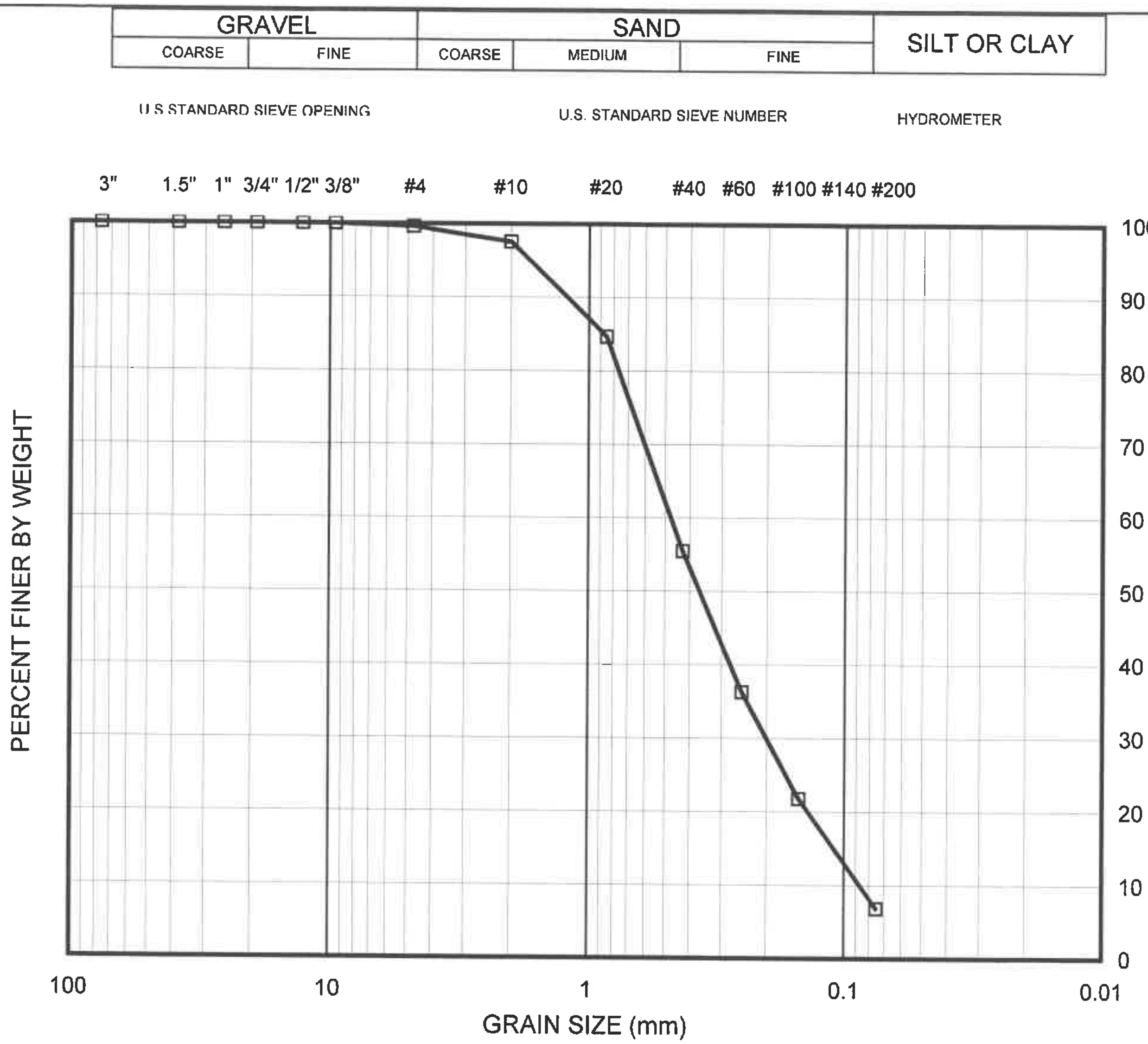


SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
☐	B-1	5	12.5	Bag	CL-ML	N/A	N/A

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client Job No.: 184014-00 Client Name: LGC Valley, Inc. EGLAB Project No.: 18-059-014

GRAIN SIZE DISTRIBUTION CURVE

06/18/18 (ASTM D422) FIGURE



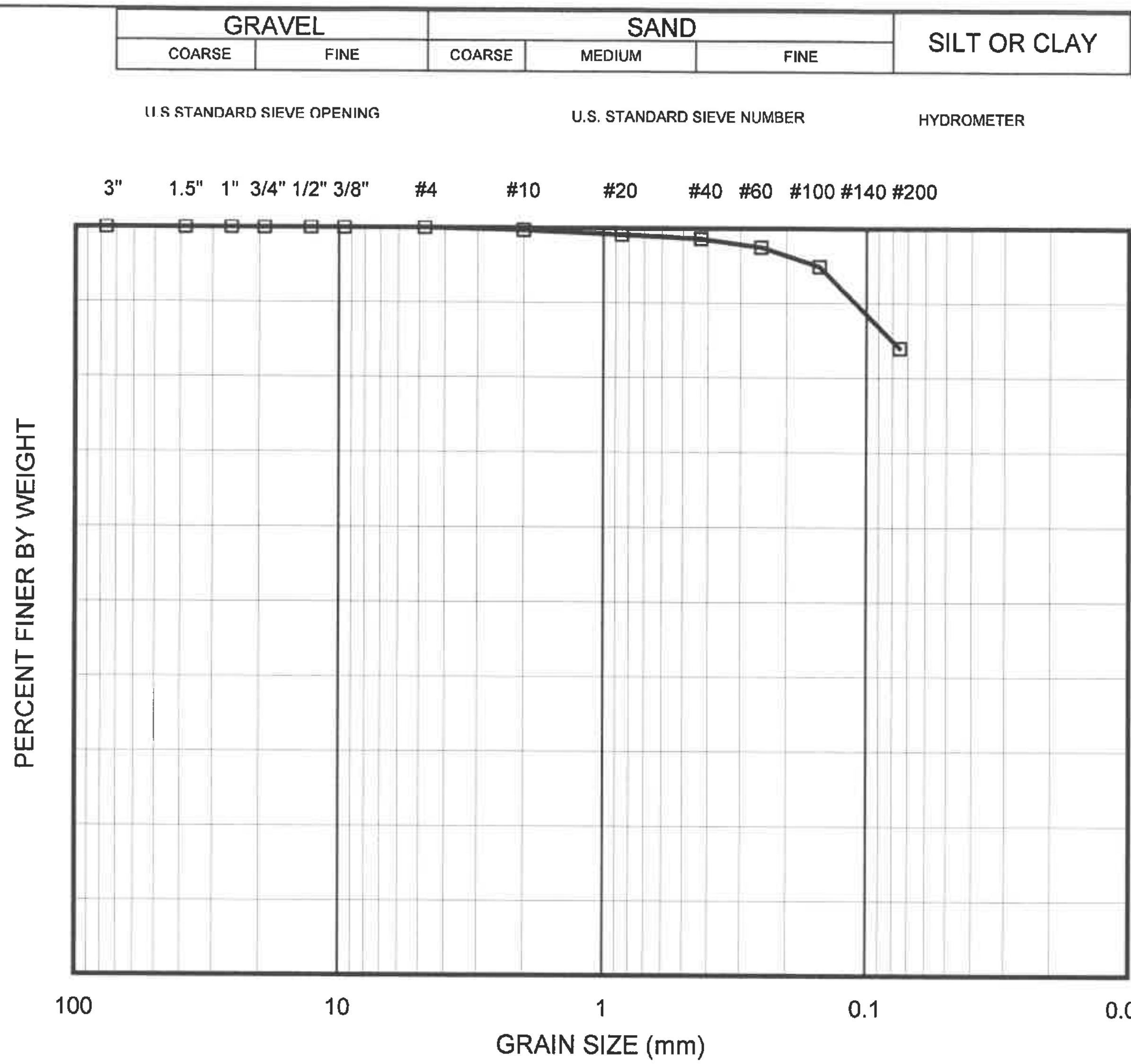
SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	7	17.5	Bag	SP-SM	N/A	N/A

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client Job No.: 184014-00 Client Name: LGC Valley, Inc. EGLAB Project No.: 18-059-014

GRAIN SIZE DISTRIBUTION CURVE
(ASTM D422)

06/18/18

FIGURE



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	11	27.5	Bag	CL	N/A	N/A

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client Job No.: 184014-00 Client Name: LGC Valley, Inc. EGLAB Project No.: 18-059-014

GRAIN SIZE DISTRIBUTION CURVE
(ASTM D422)

06/18/18

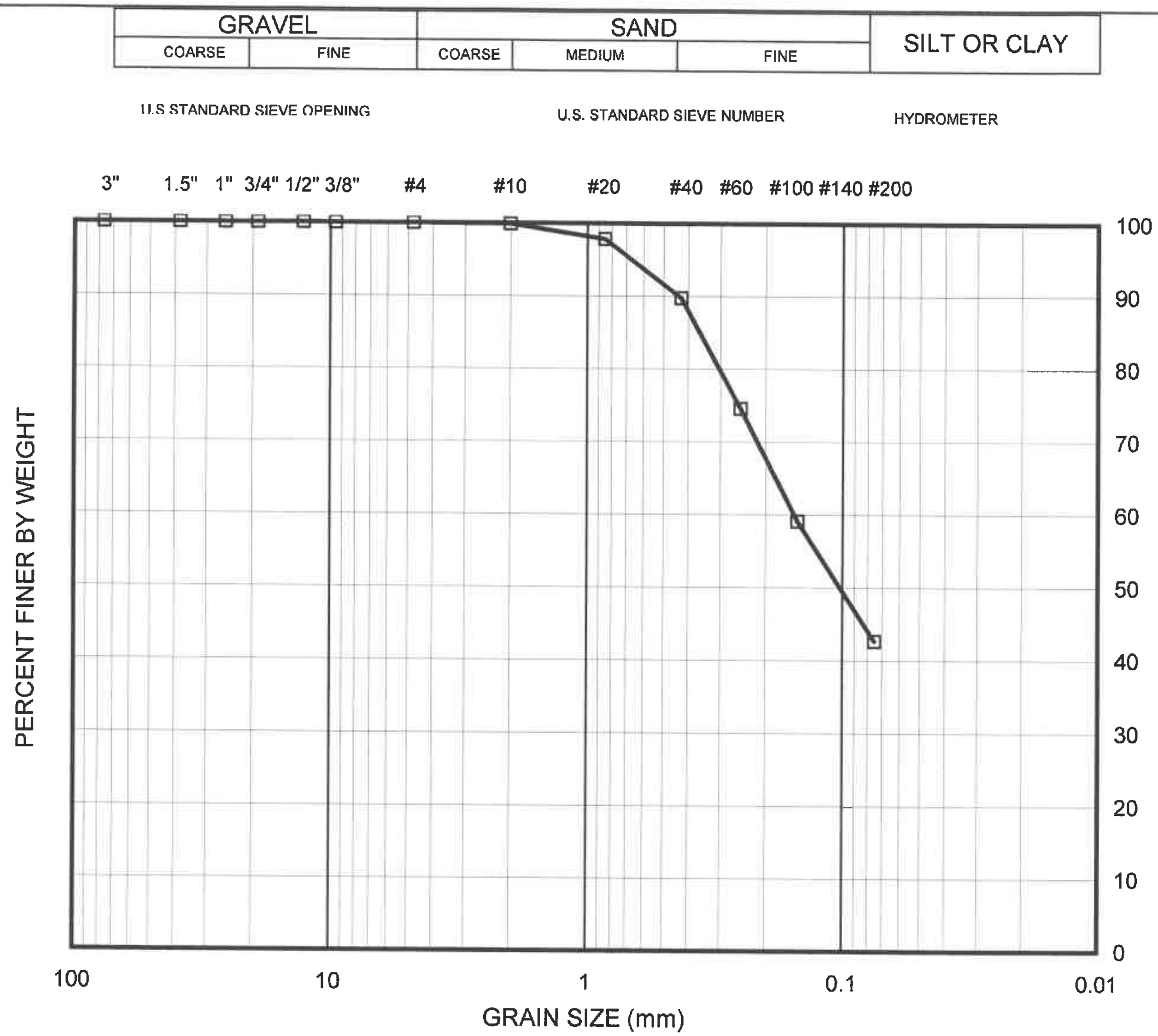
FIGURE

SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
☐	B-1	17	42.5	Bag	SP-SM	N/A	N/A

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client Job No.: 184014-00 Client Name: LGC Valley, Inc. EGLAB Project No.: 18-059-014

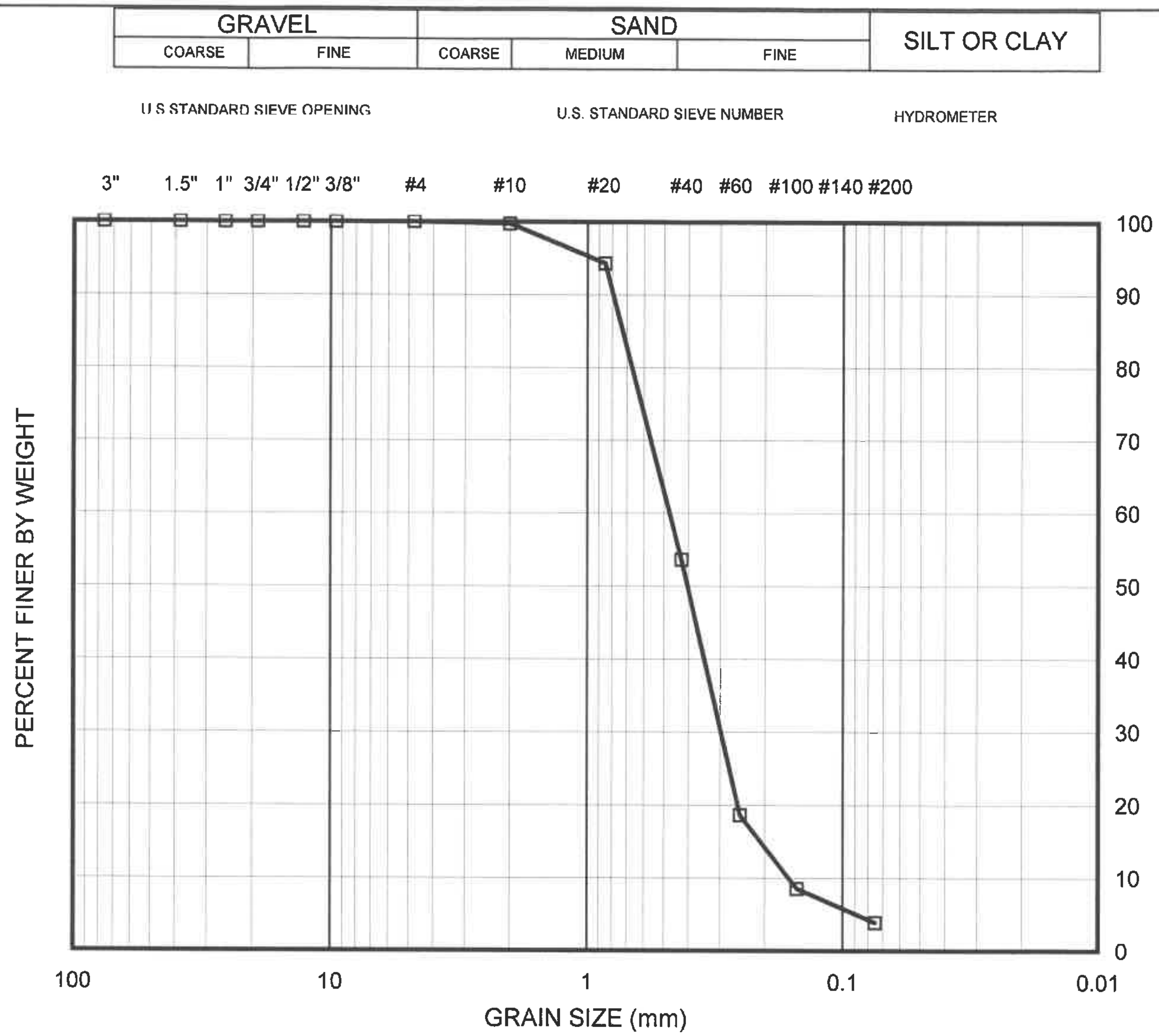
GRAIN SIZE DISTRIBUTION CURVE

06/18/18 (ASTM D422) FIGURE



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
☐	B-4	6	15.0	Bag	SC	N/A	N/A

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client Job No.: 184014-00 Client Name: LGC Valley, Inc. EGLAB Project No.: 18-059-014
06/18/18	**GRAIN SIZE DISTRIBUTION CURVE** (ASTM D422) FIGURE



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
☐	B-4	8	20.0	Bag	SP	N/A	N/A

EGLAB, INC.

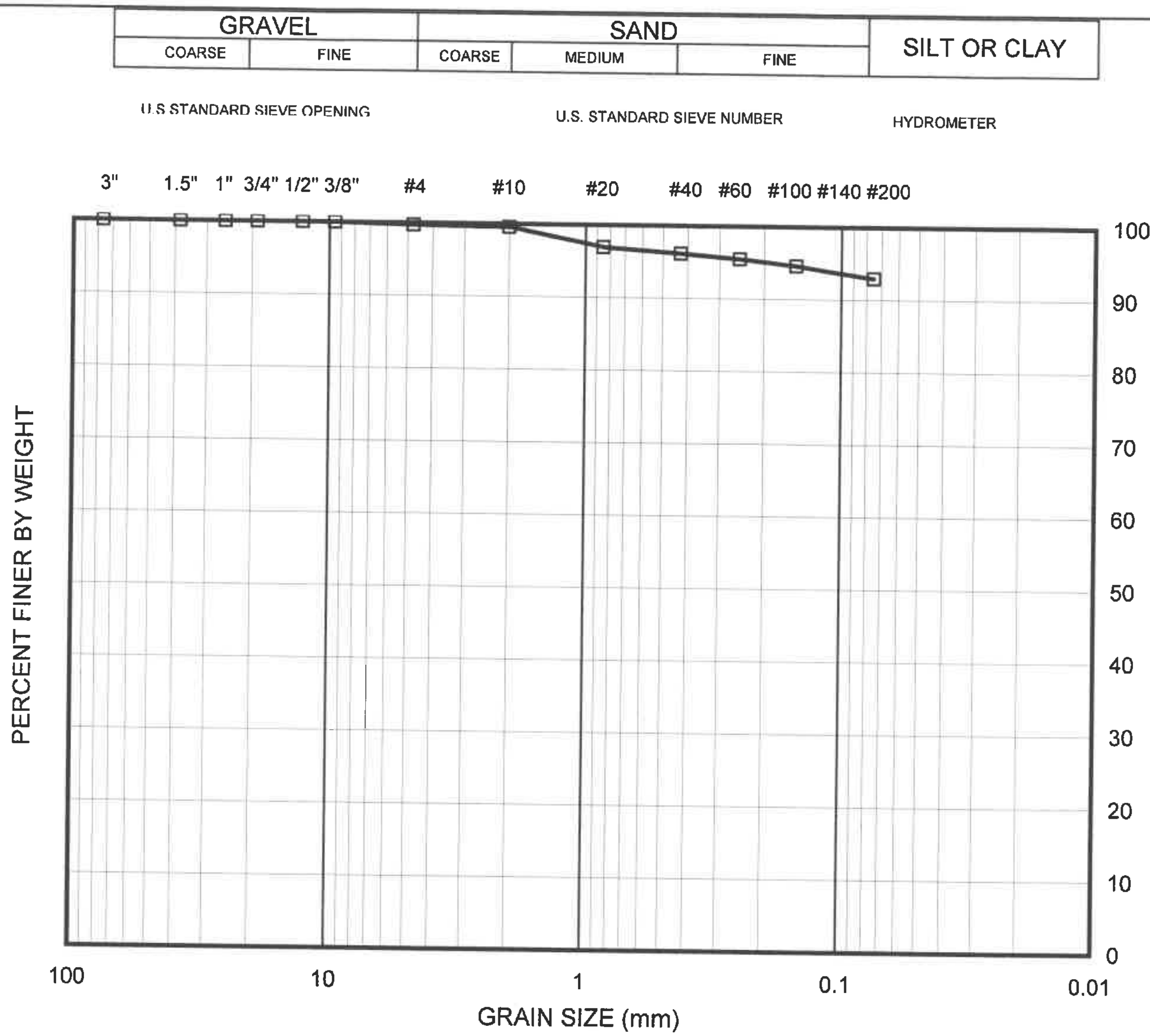
Project Name:
Shopoff Anaheim 2.

Client Job No.: 184014-00
Client Name: LGC Valley, Inc.
EGLAB Project No.: 18-059-014

GRAIN SIZE DISTRIBUTION CURVE
(ASTM D422)

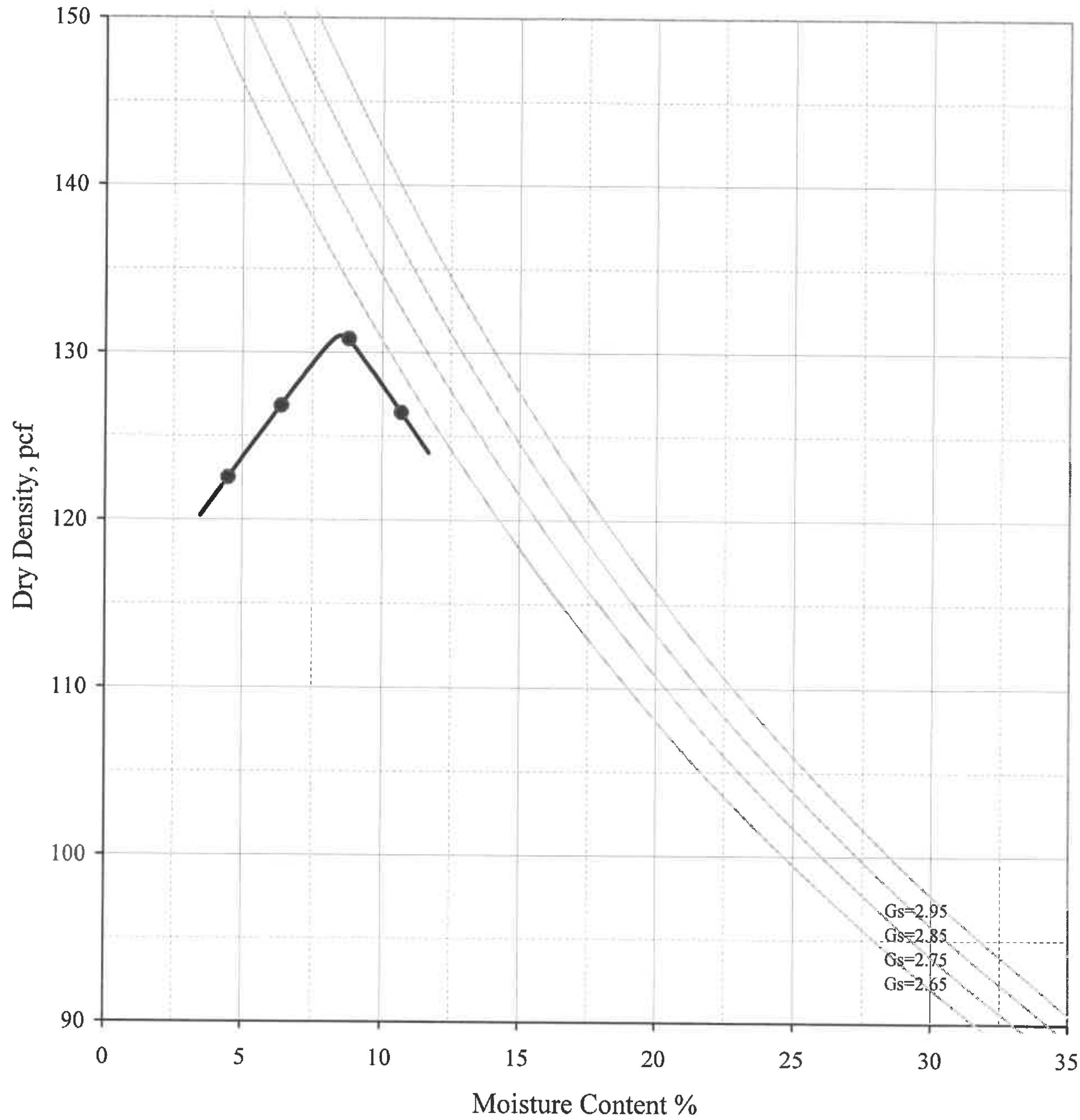
06/18/18

FIGURE



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-4	17	42.5	Bag	CL	N/A	N/A

EGLAB, INC.	Project Name: Shopoff Anaheim 2. Client Job No.: 184014-00 Client Name: LGC Valley, Inc. EGLAB Project No.: 18-059-014
06/18/18	**GRAIN SIZE DISTRIBUTION CURVE** (ASTM D422) FIGURE



Maximum Dry Density = 131 pcf

Optimum Moisture Content = 8.5 %

Boring No: B-1

Sample: A

Depth : 2.5-5 feet

Description : SC

EGLAB, INC.

Project Name:	Shopoff Anaheim 2.
Client Name:	LGC Valley, Inc.
Job No.:	184014-00
EGLAB Project No.:	18-059-014
Date : Jun-18	Figure C-16

Modified Proctor

(ASTM D1557)

APPENDIX D

Infiltration Boring Logs

Infiltration Boring Log I-1

Date: March 14, 2019	**Page: 1 of 1**
Project Name: Lincoln at Euclid	**Project Number: 184014-00**
Drilling Company: Baja Explorations	**Type of Rig: Hollow Stem Auger**
Drive Weight: 140 lbs.	**Drop: 30"** **Hole Dia: 8"**
Elevation of Top of Hole: 131 Feet	**Hole Location: See Map**

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
131	0						SC-SM	@0-3.5"- Asphalt **Young Alluvial Fan Deposits (Qyf):** @3.5" clayey silty fine to medium SAND; medium brown, moist, 10% cobbles	
126	5							@5' becomes dark brown	
							CL	@7.5' very fine to fine sandy silty CLAY, medium brown, moist	
							ML	@9' fine sandy clayey SILT; medium brown, moist	
121	10							**Total Depth = 10 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
116	15								
111	20								
106	25								
101	30								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

LGC VALLEY, INC.
THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED

Infiltration Boring Log I-2

Date: March 14, 2019	**Page: 1 of 1**
Project Name: Lincoln at Euclid	**Project Number: 184014-00**
Drilling Company: Baja Explorations	**Type of Rig: Hollow Stem Auger**
Drive Weight: 140 lbs.	**Drop: 30"** **Hole Dia: 8"**
Elevation of Top of Hole: 134 Feet	**Hole Location: See Map**

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
134	0						ML	**<u>Young Alluvial Fan Deposits (Qyf):</u>** @0' fine to medium sandy SILT; dark brown, moist @5' increase in clay and silt	
129	5								
							CL	@7.5' fine sandy silty CLAY; medium brown, moist @10' some cobbles present 15%	
124	10							**Total Depth = 10 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
119	15								
114	20								
109	25								
104	30								

LGC

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Infiltration Boring Log I-3

Date: March 14, 2019	**Page: 1 of 1**
Project Name: Lincoln at Euclid	**Project Number: 184014-00**
Drilling Company: Baja Explorations	**Type of Rig: Hollow Stem Auger**
Drive Weight: 140 lbs.	**Drop: 30"** **Hole Dia: 8"**
Elevation of Top of Hole: 133 Feet	**Hole Location: See Map**

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
133	0						ML	<u>**Young Alluvial Fan Deposits (Qyf):**</u> @0’ fine to medium sandy SILT; dark brown, moist; organics present @7.5’ increase in clay content	
128	5								
123	10							**Total Depth = 10 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
118	15								
113	20								
108	25								
103	30								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

Infiltration Boring Log I-4

Date: March 14, 2019	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 132 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
132	0						CL	**<u>Young Alluvial Fan Deposits (Qyf):</u>** @0’ fine sandy silty CLAY; dark brown, moist; organics present	
127	5						SC-SM	@5’ clayey silty fine to medium SAND; dark gray to dark brown @7.5’ clayey silty fine SAND; medium brown, moist	
122	10							@10’ silty fine to medium SAND; light brown, damp **Total Depth = 10 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
117	15								
112	20								
107	25								
102	30								

LGC

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Infiltration Boring Log I-5

Date: March 14, 2019	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 130 Feet	Hole Location: See Map

Logged By: LF
Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
130	0						SM	**Young Alluvial Fan Deposits (Qyf):** @0' silty fine to medium SAND; dark gray to dark brown, moist	
125	5						SC-SM	@5' clayey silty fine to medium SAND; dark brown, moist	
							SM	@7.5' silty fine to medium SAND; medium brown, moist	
							ML	@9' fine to medium sandy SILT; dark brown, moist	
120	10								
115	15							**Total Depth = 10 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
110	20								
105	25								
100	30								

LGC

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Infiltration Boring Log E-1

Date: March 14, 2019	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 132 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
132	0						SC-SM	@0-3.5" Asphalt **Young Alluvial Fan Deposits (Qyf):** @3.5" clayey silty fine to medium SAND; medium brown, moist; 10% cobbles	
127	5							@2.5' 20% cobbles	
							CL	@7.5' very fine sandy silty CLAY; medium brown, moist; few cobbles	
122	10								
							ML	@12.5' clayey SILT; medium brown, moist	
117	15								
							SC-SM	@17.5' clayey silty fine SAND; medium brown, moist @20' silty fine to medium SAND; medium brown, moist	
112	20							**Total Depth = 20 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
107	25								
102	30								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

Infiltration Boring Log E-2

Date: March 14, 2019	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 133 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
133	0						ML	**<u>Young Alluvial Fan Deposits (Qyf):</u>** @0' fine to medium sandy SILT; dark brown, moist; some gravel present; organics present @2.5' fine to medium sandy clayey SILT; dark brown, moist	
128	5								
123	10						CL	@12.5' fine sandy silty CLAY; dark brown, moist	
118	15						ML	@15' fine sandy clayey SILT; dark brown, moist	
							SM	@17.5' silty fine to medium SAND; light brown, damp to moist	
113	20							**Total Depth = 20 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
108	25								
103	30								

LGC

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Infiltration Boring Log E-3

Date: March 14, 2019	Page: 1 of 1
Project Name: Lincoln at Euclid	Project Number: 184014-00
Drilling Company: Baja Explorations	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 131 Feet	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: LF Sampled By: LF	Type of Test
131	0						SM	**Young Alluvial Fan Deposits (Qyf):** @0' silty fine to medium SAND; medium brown, damp; some cobbles present	
126	5						SC-SM	@5' clayey silty fine to medium SAND; medium brown, moist	
121	10						ML	@10' clayey fine to medium sandy SILT; dark brown, moist	
116	15							@15' fine to medium sandy SILT; light brown, damp	
							SM	@19' silty fine to medium SAND; light brown, dry to damp	
111	20							**Total Depth = 20 Feet** **No Groundwater Encountered** **Backfilled with Native Soil** **on March 15, 2019**	
106	25								
101	30								

LGC

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LGC VALLEY, INC.
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APPENDIX E

Infiltration Test Data Sheets

Project Name: Lincoln At Euclid | **Test Hole Number:** I-1
Project Number: 184014-01 | **Elevation at Test Hole Bottom:** 121 Feet
Date Excavated: 3/14/2019 | **Geologic Unit:** Young Alluvial Deposits (Qyf)
Tested by: LF | **Date:** 3/15/2019 **Test Hole Size/Shape:** 8-in/round
Test Hole Depth: 10 Feet | **Soil Type:** clayey SILT

Test Hole Pre-Soaked on 3/14/2018 | 2-inches of gravel in bottom

TEST PERIOD

Time	Δt	H_o	H_f	ΔH
9:23 9:45	25	88.79	86.39	2.40
9:45 10:10	25	86.39	77.98	8.40
7:30 8:00	30	106.19	94.79	11.40
8:00 8:30	30	94.79	88.79	6.00
8:30 9:00	30	88.79	84.59	4.20
9:00 9:30	30	92.99	88.19	4.80
9:30 10:00	30	88.19	83.99	4.20
10:00 10:30	30	83.99	80.38	3.60
10:30 11:00	30	80.38	76.78	3.60
11:00 11:30	30	89.99	85.79	4.20
11:30 12:00	30	85.79	82.79	3.00
12:00 12:30	30	82.79	79.78	3.00
12:30 1:00	30	88.19	85.19	3.00
1:00 1:30	30	85.19	82.18	3.00

$$\boldsymbol{Infiltration\ Rate} = \frac{\Delta H\ 60\ r}{\Delta t\ (r+2H_{ave})} = \frac{(3.00\ in)\left(60\frac{min}{hr}\right)(4\ in)}{(30\ min)\ (4\ in+2(\frac{85.19+82.18}{2}\ in))} = \mathbf{0.14}\ \boldsymbol{in/hr}$$

* Δt= Time Interval (min), H_o= Initial Water Level (in), H_f= Final Water Level (in),
ΔH= Change in Water Level (in), H_{ave}= Average Head Height Over Time Interval, r= Radius of Test Hole

$$\boldsymbol{Factor\ of\ Safety} = \mathbf{2.25} \qquad \boldsymbol{Adjusted\ Infiltraton\ Rate} = \frac{0.14\ in/hr}{2.25} = \mathbf{0.06}\ \boldsymbol{in/hr}$$

Project Name: Lincoln At Euclid
Test Hole Number: I-2
Project Number: 184014-01
Elevation at Test Hole Bottom: 124 Feet
Date Excavated: 3/14/2019
Geologic Unit: Undocumented Artificial Fill (Afu)
Tested by: LF
Date: 3/15/2019 **Test Hole Size/Shape:** 8-in/round
Test Hole Depth: 10 Feet
Soil Type: clayey SILT

Test Hole Pre-Soaked on 3/14/2018

2-inches of gravel in bottom

TEST PERIOD

Time	Δt	H_o	H_f	ΔH
0:00 – 0:25	25	92.39	88.19	4.20
0:25 – 0:50	25	88.19	85.79	2.40
7:45 – 8:15	30	97.79	91.79	6.00
8:15 – 8:45	30	91.79	88.79	3.00
8:45 – 9:15	30	88.79	85.79	3.00
9:15 – 9:45	30	91.19	88.19	3.00
9:45 – 10:15	30	88.19	85.19	3.00
10:15 – 10:45	30	85.19	82.79	2.40
10:45 – 11:15	30	82.79	80.38	2.40
11:15 – 11:45	30	80.38	77.98	2.40
11:45 – 12:15	30	83.99	81.58	2.40
12:15 – 12:45	30	88.79	86.39	2.40
12:45 – 1:15	30	86.39	82.79	3.60
1:15 – 1:45	30	82.79	80.38	2.40

$$\boldsymbol{Infiltration\ Rate} = \frac{\Delta H\ 60\ r}{\Delta t\ (r+2H_{ave})} = \frac{(2.40\ in)\left(60\frac{min}{hr}\right)(4\ in)}{(30\ min)\ (4\ in+2(\frac{82.79+80.38}{2}\ in))} = \mathbf{0.11\ in/hr}$$

* Δt= Time Interval (min), H_o= Initial Water Level (in), H_f= Final Water Level (in), ΔH= Change in Water Level (in), H_{ave}= Average Head Height Over Time Interval, r= Radius of Test Hole

$$\boldsymbol{Factor\ of\ Safety = 2.25} \quad \boldsymbol{Adjusted\ Infiltraton\ Rate} = \frac{0.11\ in/hr}{2.25} = \mathbf{0.05\ in/hr}$$

Project Name:	Lincoln At Euclid	**Test Hole Number:**	I-3
Project Number:	184014-01	**Elevation at Test Hole Bottom:**	123 Feet
Date Excavated:	3/14/2019	**Geologic Unit:**	Undocumented Artificial Fill (Afu)
Tested by:	LF	**Date:** 3/15/2019 **Test Hole Size/Shape:**	8-in/round
Test Hole Depth:	10 Feet	**Soil Type:**	sandy CLAY

Test Hole Pre-Soaked on 3/14/2018 | 2-inches of gravel in bottom

TEST PERIOD

Time	Δt	H_o	H_f	ΔH
0:00 0:25	25	95.39	93.59	1.80
0:25 0:50	25	93.59	92.39	1.20
7:40 8:10	30	93.59	91.79	1.80
8:10 8:40	30	91.79	91.19	0.60
8:40 9:10	30	91.19	89.99	1.20
9:10 9:40	30	89.99	89.39	0.60
9:40 10:10	30	92.99	92.39	0.60
10:10 10:40	30	92.39	91.31	1.08
10:40 11:10	30	91.31	91.19	0.12
11:10 11:40	30	91.19	90.59	0.60
11:40 12:10	30	90.59	89.99	0.60
12:10 12:40	30	89.99	89.39	0.60
12:40 1:10	30	89.39	88.79	0.60
1:10 1:40	30	88.79	88.19	0.60

$$\boldsymbol{Infiltration\ Rate} = \frac{\Delta H\ 60\ r}{\Delta t\ (r+2H_{ave})} = \frac{(\mathbf{0.60}\ \boldsymbol{in})\left(\mathbf{60}\frac{\boldsymbol{min}}{\boldsymbol{hr}}\right)(\mathbf{4}\ \boldsymbol{in})}{(\mathbf{30}\ \boldsymbol{min})\ (\mathbf{4}\ \boldsymbol{in}+2(\frac{88.79+88.19}{2}\ \boldsymbol{in}))} = \mathbf{0.03}\ \boldsymbol{in/hr}$$

* Δt= Time Interval (min), H_o= Initial Water Level (in), H_f= Final Water Level (in),
ΔH= Change in Water Level (in), H_{ave}= Average Head Height Over Time Interval, r= Radius of Test Hole

$$\boldsymbol{Factor\ of\ Safety} = \mathbf{2.25} \quad \boldsymbol{Adjusted\ Infiltraton\ Rate} = \frac{0.03\ in/hr}{2.25} = \mathbf{0.01}\ \boldsymbol{in/hr}$$

Project Name:	Lincoln At Euclid	**Test Hole Number:**	I-4
Project Number:	184014-01	**Elevation at Test Hole Bottom:**	122 Feet
Date Excavated:	3/14/2019	**Geologic Unit:**	Undocumented Artificial Fill (Afu)
Tested by:	LF	**Date:** 3/14/2019 **Test Hole Size/Shape:**	8-in/round
Test Hole Depth:	10 Feet	**Soil Type:**	silty SAND

Test Hole Pre-Soaked on 3/14/2018

2-inches of gravel in bottom

TEST PERIOD

Time	Δt	H_o	H_f	ΔH
0:06 0:31	25	108.00	79.78	28.21
0:31 0:57	25	104.39	79.18	25.21
1:03 1:13	10	100.79	87.59	13.21
1:14 1:24	10	97.19	86.39	10.80
1:24 1:34	10	86.39	77.98	8.40
1:34 1:44	10	83.99	77.98	6.00
1:44 1:54	10	83.99	77.98	6.00
1:54 2:04	10	77.98	70.78	7.20

$$\boldsymbol{Infiltration\ Rate} = \frac{\Delta H\ 60\ r}{\Delta t\ (r+2H_{ave})} = \frac{(7.20\ in)\left(60\frac{min}{hr}\right)(4\ in)}{(10\ min)\ (4\ in+2(\frac{77.98+70.78}{2}\ in))} = \mathbf{1.13\ in/hr}$$

* Δt= Time Interval (min), H_o= Initial Water Level (in), H_f= Final Water Level (in),
ΔH= Change in Water Level (in), H_{ave}= Average Head Height Over Time Interval, r= Radius of Test Hole

$$\boldsymbol{Factor\ of\ Safety} = \mathbf{2.25} \quad \boldsymbol{Adjusted\ Infiltraton\ Rate} = \frac{1.13\ in/hr}{2.25} = \mathbf{0.50\ in/hr}$$

Project Name:	Lincoln At Euclid	**Test Hole Number:**	I-5
Project Number:	184014-01	**Elevation at Test Hole Bottom:**	120 Feet
Date Excavated:	3/14/2019	**Geologic Unit:**	Undocumented Artificial Fill (Afu)
Tested by:	LF	**Date:** 3/14/2019 **Test Hole Size/Shape:**	8-in/round
Test Hole Depth:	10 Feet	**Soil Type:**	silty SAND

Test Hole Pre-Soaked on 3/14/2018

2-inches of gravel in bottom

TEST PERIOD

Time	Δt	H_o	H_f	ΔH
0:18 – 0:43	25	103.19	88.79	14.41
0:44 – 1:09	25	98.39	86.39	12.00
1:10 – 1:20	10	99.59	93.59	6.00
1:20 – 1:30	10	93.59	88.79	4.80
1:30 – 1:40	10	96.59	92.39	4.20
1:40 – 1:50	10	92.39	88.79	3.60
9:22 – 2:00	10	93.59	88.79	4.80
2:00 – 2:10	10	88.79	86.39	2.40

$$\boldsymbol{Infiltration\ Rate} = \frac{\Delta H\ 60\ r}{\Delta t\ (r+2H_{ave})} = \frac{(7.20\ in)\left(60\frac{min}{hr}\right)(4\ in)}{(20\ min)\ (4\ in+2(\frac{93.59+86.39}{2}\ in))} = \mathbf{0.47}\ \boldsymbol{in/hr}$$

* Δt= Time Interval (min), H_o= Initial Water Level (in), H_f= Final Water Level (in), ΔH= Change in Water Level (in), H_{ave}= Average Head Height Over Time Interval, r= Radius of Test Hole

$$\boldsymbol{Factor\ of\ Safety} = \mathbf{2.25} \qquad \boldsymbol{Adjusted\ Infiltraton\ Rate} = \frac{0.47\ in/hr}{2.25} = \mathbf{0.21}\ \boldsymbol{in/hr}$$

APPENDIX F

Worksheet H: Factor of Safety and Design Infiltration Rate
and
Worksheet I: Summary of Groundwater-Related Feasibility Criteria

Worksheet H: Factor of Safety and Design Infiltration Rate Worksheet For

Western Facility

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25	1	0.25
		Predominant soil texture	0.25	2	0.50
		Site soil variability	0.25	1	0.25
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.25
B	Design	Tributary area size	0.25	1	0.25
		Level of pretreatment/ expected sediment loads	0.25	1	0.25
		Redundancy	0.25	1	0.25
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_B = \Sigma p$			1

Combined Safety Factor, $S_{TOT} = S_A \times S_B$	2.25
Measured Infiltration Rate, inch/hr, K_M (corrected for test-specific bias)	0.80 in/hour
Design Infiltration Rate, in/hr, $K_{DESIGN} = S_{TOT}$ / K_M	0.36 in/hour

Supporting Data

Briefly describe infiltration test and provide reference to test forms:

Two tests were conducted using the Percolation Test Procedure within the location of the proposed facility on the west of the site (I-4 and I-5). The measured infiltration rates of the two borings were averaged and divided by the factor of safety of 2.25 to come up with a Design Infiltration Rate of 0.36 in/hour. Infiltration is feasible in this location.

See Section 3 of this report for description of test method. See Geotechnical Map for location of tests. See Appendix D for logs of material tested. See Appendix E for the infiltration test data sheets.

Note: The minimum combined adjustment factor shall not be less than 2.0 and the maximum combined adjustment factor shall not exceed 9.0.

Worksheet H: Factor of Safety and Design Infiltration Rate Worksheet For

Central Facility

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25	1	0.25
		Predominant soil texture	0.25	2	0.50
		Site soil variability	0.25	1	0.25
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.25
B	Design	Tributary area size	0.25	1	0.25
		Level of pretreatment/ expected sediment loads	0.25	1	0.25
		Redundancy	0.25	1	0.25
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_B = \Sigma p$			1

Combined Safety Factor, $S_{TOT} = S_A \times S_B$	2.25
Measured Infiltration Rate, inch/hr, K_M (corrected for test-specific bias)	0.07 in/hour
Design Infiltration Rate, in/hr, $K_{DESIGN} = S_{TOT} / K_M$	0.03 in/hour

Supporting Data

Briefly describe infiltration test and provide reference to test forms:

Two tests were conducted using the Percolation Test Procedure within the location of the proposed facility in the central portion of the site (I-2 and I-3). The measured infiltration rates of the two borings were averaged and divided by the factor of safety of 2.25 to come up with a Design Infiltration Rate of 0.03 in/hour. Infiltration rates in this location do not meet the minimum threshold of 0.3 in/hour to be considered feasible for infiltration according to Section VII.2 (Orange County, 2013).

See Section 3 of this report for description of test method. See Geotechnical Map for location of tests. See Appendix D for logs of material tested. See Appendix E for the infiltration test data sheets.

Worksheet H: Factor of Safety and Design Infiltration Rate Worksheet For

Eastern Facility

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25	1	0.25
		Predominant soil texture	0.25	2	0.50
		Site soil variability	0.25	1	0.25
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.25
B	Design	Tributary area size	0.25	1	0.25
		Level of pretreatment/ expected sediment loads	0.25	1	0.25
		Redundancy	0.25	1	0.25
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_B = \Sigma p$			1

Combined Safety Factor, $S_{TOT} = S_A \times S_B$	2.25
Measured Infiltration Rate, inch/hr, K_M (corrected for test-specific bias)	0.14 in/hour
Design Infiltration Rate, in/hr, $K_{DESIGN} = S_{TOT} / K_M$	0.06 in/hour

Supporting Data

Briefly describe infiltration test and provide reference to test forms:

One test was conducted using the Percolation Test Procedure within the location of the proposed facility in the eastern portion of the site (I-1). The measured infiltration rate was divided by the factor of safety of 2.25 to come up with a Design Infiltration Rate of 0.06 in/hour. Infiltration rates in this location do not meet the minimum threshold of 0.3 in/hour to be considered feasible for infiltration according to Section VII.2 (Orange County, 2013).

See Section 3 of this report for description of test method. See Geotechnical Map for location of tests. See Appendix D for logs of material tested. See Appendix E for the infiltration test data sheets.

Worksheet I: Summary of Groundwater-related Feasibility Criteria

1	Is project large or small? (as defined by Table VIII.2) circle one	Large		Small
2	What is the tributary area to the BMP?	A, B, C	1.28, 2.79, 1.46	acres
3	What type of BMP is proposed?	Underground infiltration chambers/ Subsurface infiltration galleries		
4	What is the infiltrating surface area of the proposed BMP?	A_{BMP}	76,262	sq-ft
5	What land use activities are present in the tributary area (list all) The proposed project is a multifamily development with 101 townhomes.			
6	What land use-based risk category is applicable?	L	M	H
7	If M or H, what pretreatment and source isolation BMPs have been considered and are proposed (describe all): N/A			
8	What minimum separation to mounded seasonally high groundwater applies to the proposed BMP? See Section VIII.2 (circle one)	5 ft		100 ft
9	Provide rationale for selection of applicable minimum separation to seasonally high mounded groundwater: Given that seasonally high groundwater is approximately 100 feet below ground surface, and no shallow perched water was encountered in any of our borings, groundwater mounding does not provide a constraint to infiltration at this site.			
10	What is separation from the infiltrating surface to seasonally high groundwater?	SHGWT	100+	ft
11	What is separation from the infiltrating surface to mounded seasonally high groundwater?	Mounded SHGWT	90	ft
12	Describe assumptions and methods used for mounding analysis: Storm water infiltration and recharge to the underlying groundwater table could lead to groundwater mounding when a shallow groundwater table is present (less than 10 feet); however, groundwater is approximately 100 feet below the ground surface, so mounding is not anticipated under these conditions.			
13	Is the site within a plume protection boundary (See Figure	Y	N	N/A

Worksheet I: Summary of Groundwater-related Feasibility Criteria

	VIII.2)?	
14	Is the site within a selenium source area or other natural plume area (See Figure VIII.2)?	Y **N** N/A
15	Is the site within 250 feet of a contaminated site?	**Y** N N/A
16	If site-specific study has been prepared, provide citation and briefly summarize relevant findings: The Site Specific Study is attached as Appendix G of this report. **Conclusions** • Detected concentrations of contaminants in on-Site soil samples do not indicate that storm water infiltration will cause adverse impacts to groundwater quality. • Although detected concentrations of PCE in soil vapor exceed the conservative residential screening level cancer risk threshold for indoor air, infiltration of surface water through the relatively low concentrations of PCE in soil vapor are unlikely to cause significant impacts to groundwater (Roux Associates, Inc., 2019).	
17	Is the site within 100 feet of a water supply well, spring, septic system?	Y **N** N/A
18	Is infiltration feasible on the site relative to groundwater-related criteria?	**Y** N
Provide rationale for feasibility determination: All criterion listed on this worksheet indicate that infiltration is feasible from a groundwater feasibility standpoint.		

Note: if a single criterion or group of criteria would render infiltration infeasible, it is not necessary to evaluate every question in this worksheet.

APPENDIX G

<u>Site Specific Study Assessing Impact of Infiltration BMPs</u>
<u>By Roux Associates, Inc.</u>

ROUX
ROUX ASSOCIATES, INC.
Environmental Consulting
& Management

April 4, 2019

Mr. James O'Malley
SLF-West Lincoln, LLC
2 Park Plaza, Suite 700
Irvine, CA 92614

Re: Site Specific Study Assessing Impact of Proposed Infiltration BMPs Within 250 Feet of a Contaminated Site
1631 and 1699 West Lincoln Avenue
Anaheim, California
Roux Project No. 3224.0003L002

Dear Mr. O'Malley,

Roux Associates, Inc. (Roux Associates) on behalf of SLF-West Lincoln, LLC (SLF) has prepared this site-specific study (Study) for two parcels located at 1631 and 1699 West Lincoln Avenue in the City of Anaheim, California (Site; Figures 1 and 2). The objective of this Study is to address the prohibition of infiltration within 250 feet of contaminated sites unless a site-specific study such as this demonstrates that the proposed infiltration would not adversely impact groundwater conditions (TGD, 2013[1]).

General Background

The Site consists of two irregular-shaped adjoining parcels situated within a triangular city block. The city block is bound by West Lincoln Avenue to the south, South Euclid Street to the west, and by a Southern Pacific Railroad (SPRR) easement to the northeast (Figure 2). The Site lies at an elevation of approximately 125 feet above mean sea level with local relief sloping gently to the west-southwest. The parcel situated at 1631 West Lincoln Avenue is approximately 2 acres in size and is currently occupied by a cement manufacturing company. The parcel at 1699 West Lincoln Avenue approximately 4 acres in size and is currently vacant. Both properties are currently owned by Anastasi Development Company, LLC (Anastasi). SLF-West Lincoln, LLC proposes to purchase the Site and construct multiple residential units with stormwater infiltration best management practices (BMPs) as shown on Figure 3.

Previous Environmental Investigations

The first known subsurface assessment at the Site was a leaking underground storage tank (LUST) investigation conducted by FREY Environmental, Inc. (FREY) in 2005. The investigation targeted a former 10,000-gallon diesel underground storage tank (UST) at 1631 West Lincoln Avenue and included the collection and analysis of samples from four soil borings

[1] Orange County, 2013, Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs), Appendices VII and VIII, dated December 20, 2013.

3224.0003L.107/LR

at five-foot intervals in the vicinity of the former UST to depths of 40 feet bgs. Soil samples were analyzed for TPH and fuel-related volatile organic compounds (VOCs).

In July 2018 Environmental Management Strategies (EMS) prepared a Phase I Environmental Site Assessment (ESA) for 1631 and 1699 West Lincoln Avenue on behalf of SLF which was updated and reissued in February 2019. Based on the findings of the Phase I ESA, EMS identified several recognized environmental conditions (RECs) for off-Site adjacent properties, although none were identified within the Site boundary. An off-Site REC of specific concern was past dry-cleaning operations reportedly performed at 1683 West Lincoln Avenue using the chemical tetrachloroethene (PCE).

The second subsurface investigation at the Site was a Phase II investigation performed in June and July 2018 by EMS. During the investigation, soil samples were collected from eight on-Site and 11 off-Site locations at depths of 2 and 5 feet bgs. Soil vapor samples were obtained from each of these locations at a sample depth of 5 feet bgs (two on-Site and four off-Site locations also had soil vapor samples collected at 15 feet bgs). A total of 16 on-Site and 22 off-Site soil samples were analyzed for California Assessment Manual (CAM) metals, total petroleum hydrocarbons (TPH) as gasoline (TPH-g), TPH as diesel (TPH-d), TPH as motor oil (TPH-o), and VOCs. Additionally, soil samples from depths of 2 and 5 feet bgs were analyzed for organochlorine pesticides (OCPs) from three on-Site and one off-Site soil borings. A total of nine on-Site and 13 off-Site soil vapor samples were analyzed for VOCs.

The third and latest subsurface investigation was performed by Roux Associates in January 2019 to fill data gaps from the previous EMS investigation. During this investigation soil and soil vapor samples were collected from three on-Site boring locations. Discrete soil samples were collected from each location at nominal depths of 0.5 and 1.5 bgs. Nested soil vapor probes were installed at each location with sample depths of 5, 15, and 30 feet bgs. A total of six soil samples were analyzed for lead, arsenic, and OCPs and a total of nine soil vapor samples were analyzed for VOCs. Roux Associates prepared a Preliminary Endangerment Assessment Equivalent Report (PEA-E Report) documenting the 2019 investigation and summarizing the previous subsurface and Phase I ESA investigations for the Site.[2]

<u>Subsurface Soil Investigation Summary and Findings</u>

<u>1631 West Lincoln Avenue</u> – A summary of the soil sample analytical results for the three subsurface investigations performed to date at this location is below:

1. FREY 2005 - Analytical results for soil samples collected from borings FB1 through FB4, installed in the area of the previous LUST were reported as not detected. The soil samples were collected at five-foot intervals from five to 40-feet bgs from each boring and analyzed for TPH-g and TPH-d using EPA Method 8015M as well as benzene, toluene, ethylbenzene, xylenes, fuel oxygenates, and ethanol using EPA Method 8260B. Analytical results for each parameter were reported as not detected above laboratory detection limits. Case closure was provided by the oversight agency due to the apparent limited vertical and lateral extent of impacted soil.

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[2] Roux Associates, Preliminary Endangerment Assessment Equivalent Report, Lincoln Avenue Assemblage, 1631 & 1699 West Lincoln Avenue, Anaheim, California. April 4, 2019

2. EMS 2018 - Six soil samples were collected from borings SLF-13, SLF-14 and SLF-21 at depths of 2 and 5 feet bgs during a June/July 2018 investigation; were analyzed for TPH-g, TPH-d, TPH-o, VOCs, and CAM metals; and the analytical results were below residential screening levels or expected background concentrations.

3. Roux Associates 2019– Two soil samples were collected from boring SVR-3 at depths of 0.5 and 1.5 feet bgs were analyzed for lead, arsenic, and OCPs. The analytical results were below residential screening levels or expected background concentrations.

Based on the above information soil contamination at 1631 West Lincoln Avenue is not considered a concern.

<u>1699 West Lincoln Avenue</u> – A summary of the soil sample analytical results for the two investigations performed to date at this location is below:

1. EMS 2018 – Ten soil samples from borings SLF-1 through SLF-5 at depths of 2 and 5 feet bgs were analyzed for TPH-g, TPH-d, TPH-o, VOCs, and CAM metals, and the analytical results were below residential screening levels or expected background concentrations. Four soil samples from borings SLF-1and SLF-4 were analyzed for OCPs and the analytical results were all reported as not detected.

2. Roux Associates 2019– Four soil samples collected from borings SVR-1 and SVR-2 at depths of 0.5 and 1.5 feet bgs were analyzed for lead, arsenic, and OCPs. The analytical results were below residential screening levels or expected background concentrations.

Based on the above summary, soil contamination at 1699 West Lincoln Avenue is not considered a concern.

<u>Off-Site Assessment Within 250 Feet of Proposed Infiltration BMPs</u> – A summary of the soil sample analytical results for the investigation performed at off-Site locations within 250 feet of the proposed infiltration BMPs is below:

In June/July 2018 EMS collected 20 soil samples from off-Site borings SLF-6 , SLF-7, SLF-9 through SLF-12, SLF-17, SLF-19, and SLF-20 at depths of 2 and 5 feet bgs and analyzed them for TPH-g, TPH-d, TPH-o, VOCs, and CAM metals. The off-Site soil sample analytical results were below residential screening levels or expected background concentrations. However, at the off-Site property 1683 West Lincoln, soil samples collected from borings SLF-9, SLF-19, and SLF-20 were reported with detections of the VOC tetrachloroethene (PCE) in at least one soil sample from each of the three borings with detections ranging from 6.0 micrograms per kilogram (µg/kg) to 66µg/kg. As mentioned previously, these detections are below residential screening levels, however their presence in soil appears to indicate a nearby source of PCE contamination in the subsurface at this property.

Subsurface Soil Vapor Investigation Summary and Findings

1631 and 1699 West Lincoln Avenue – A summary of the two soil vapor investigations performed at these two on-Site locations is below:

1. EMS 2018 – Soil vapor samples from collected from locations SLF-1 through SLF-5 and SLF-13, SLF-14 and SLF-21 at depths 5 feet bgs and from SLF-13 and SLF-21 from 15 feet bgs were analyzed for VOCs. With the exception of one sample result (PCE at 680 micrograms per cubic meter [μg/m^3]) the analytical results were below converted residential indoor air screening levels.

2. Roux Associates 2019– Soil vapor samples from collected from locations SVR-1 through SVR-3 at depths 5, 15, and 30 feet bgs were analyzed for VOCs. Analytical results for one of the samples collected form 15 feet bgs and two of the samples collected from 30 feet bgs exceeded the converted residential indoor air screening levels.

Although soil vapor concentrations at these two on-Site properties exceed conservative residential cancer risk screening level threshold of 1E-06, they are within the range of acceptability (1E-06 to 1E-04) as established in National Contingency Plan (NCP). Also, it is apparent that the off-Site property located at 1683 West Lincoln Avenue is the source of the soil vapor impact.

Soil Vapor Contamination Within 250 Feet of Proposed Infiltration BMPs – A summary of the soil vapor sample analytical results for the investigation performed at off-Site locations within 250 feet of the proposed infiltration BMPs is below:

In June/July 2018 EMS collected soil vapor samples from locations SLF-6 , SLF-7, SLF-9 through SLF-12, SLF-17, SLF-19, and SLF-20 at depths of 5-feet bgs from each location and additionally from depths of 15 feet bgs from SLF-10, SLF-11, SLF-17, SLF-19, and SLF-20. Each soil vapor sample was analyzed for VOCs. PCE concentrations in eleven of the fourteen samples analyzed exceeded the converted residential indoor air cancer risk screening level for cancer risk.

The detected PCE concentrations were highest in samples collected from locations adjacent to the former dry-cleaning operations reportedly performed at 1683 West Lincoln Avenue and generally decreased as distance from the 1683 property increased, which appears to indicate a source of PCE contamination is present in the subsurface at this property.

Groundwater Conditions

Nearby properties on Geotracker and Envirostor have reported depths to groundwater ranging between approximately 62 and 102 feet below ground surface (bgs). The varying depths of groundwater are likely due to the presence of discontinuous perched aquifers.

According to information from an investigation occurring at a site west of Euclid Avenue and approximately 300 feet west of the Site (Euclid Way Industrial Park), groundwater is present at depths of approximately 80 to 85 feet bgs and the groundwater flow direction is east-southeast under a relatively flat gradient of approximately 0.001 to 0.002 ft/ft.[3]

Summary

Concentrations of contaminants in soil at on-Site locations and off-Site locations within 250 feet of proposed BMPs are below residential screening levels or expected background concentrations. However, at the off-Site property 1683 West Lincoln, detections of PCE in soil samples ranged from 6.0 µg/kg to 66µg/kg. These detections are below residential screening levels, however their presence in soil appears to indicate a nearby source of PCE contamination in the subsurface at this property.

Estimated cancer risk associated with soil vapor concentrations detected at the two on-Site properties are within the range of acceptability (1E-06 to 1E-04) as established in the NCP but exceed conservative screening level threshold of 1E-06. It is apparent that the off-Site property located at 1683 West Lincoln Avenue is the probable source of the PCE detections in soil vapor.

Nearby properties on Geotracker and Envirostor have reported depths to groundwater ranging between approximately 62 and 102 feet below ground surface (bgs). According to information from a nearby investigation occurring across Euclid Avenue, groundwater is present at depths of approximately 80 to 85 feet bgs and flows to the east-southeast.

Conclusions

- Detected concentrations of contaminants in on-Site soil samples do not indicate that storm water infiltration will cause adverse impacts to groundwater quality.

- Although detected concentrations of PCE in soil vapor exceed the conservative residential screening level cancer risk threshold for indoor air, infiltration of surface water through the relatively low concentrations of PCE in soil vapor are unlikely to cause significant impacts to groundwater.

- There is no evidence of groundwater impacts within 250 feet of the proposed infiltration BMPs.

Based on the above conclusions it is unlikely that the proposed stormwater infiltration BMP locations will cause adverse impacts to groundwater quality.

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[3] Centec Engineering, Inc. (Centec, 2017), Vapor Intrusion Risk Mitigation and Groundwater Monitoring Report, Euclid Way Industrial Park, 231-307 North Euclid Way. May 30.

Sincerely,

ROUX ASSOCIATES, INC.

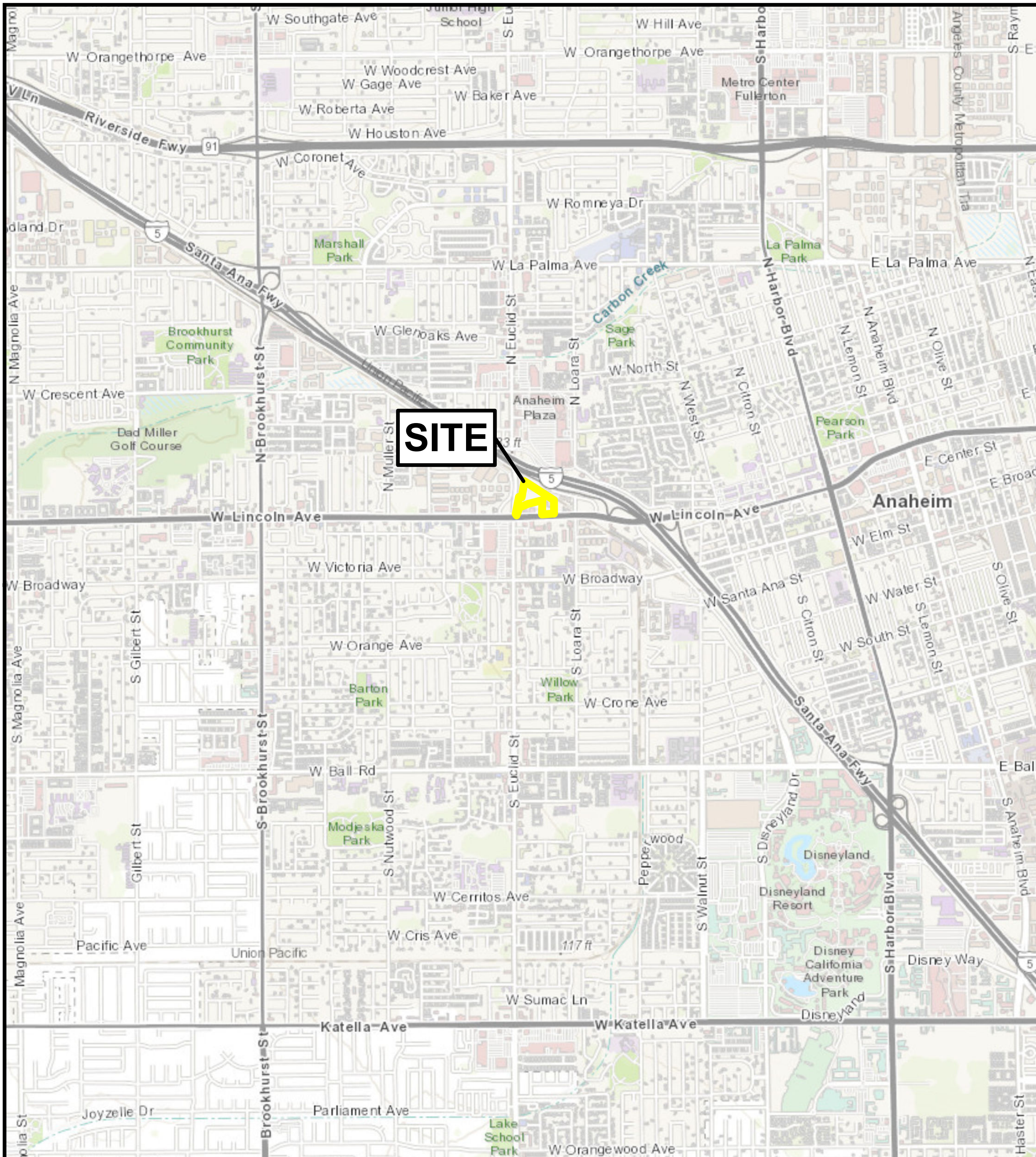
David J. DeVries, P.G., C.Hg.	April 4, 2019	
Senior Hydrogeologist	Date	Signature
Chris Rose, P.E.	April 4, 2019	
Senior Engineer	Date	Signature

PROFESSIONAL GEOLOGIST
DAVID JOHN DeVRIES
CERTIFIED HYDROGEOLOGIST
No. 594
Exp 4/30/19
STATE OF CALIFORNIA

Attachments:

Figure 1 – Site Location Map
Figure 2 – Site Plan
Figure 3 - Proposed Infiltration BMP Locations

FIGURES



0.25 0 0.25 0.5
Miles

N

Title:

SITE LOCATION MAP

1631 AND 1699 WEST LINCOLN AVENUE
ANAHEIM, CALIFORNIA

Prepared For: SLF-West Lincoln, LLC

ROUX

Compiled by: MAE	Date: 1/14/2019	FIGURE
Prepared by: MAE	Scale: 1:35,000	1
Project Mgr: DD	Office: LA	
File No: F(AP)	Project:3224.0003L000	

S:\Los Angeles\Clients\Shopoff\Shopoff Advisors - Anaheim 3224.0003L000\05Workables\Report\figures\GIS\Figure 1 - Site Location Map.mxd

LEGEND

(red line)	APPROXIMATELY OUTLINE OF PARCELS CURRENTLY UNDER CONSIDERATION (1631 AND 1699)
(dashed line)	APPROXIMATE OUTLINE OF PREVIOUSLY CONSIDERED PARCELS
1695	ADDRESS(ES) ASSOCIATED WITH PARCELS

N

100' 0' 100'

Title:

SITE PLAN

1631 AND 1699 WEST LINCOLN AVENUE
ANAHEIM, CALIFORNIA

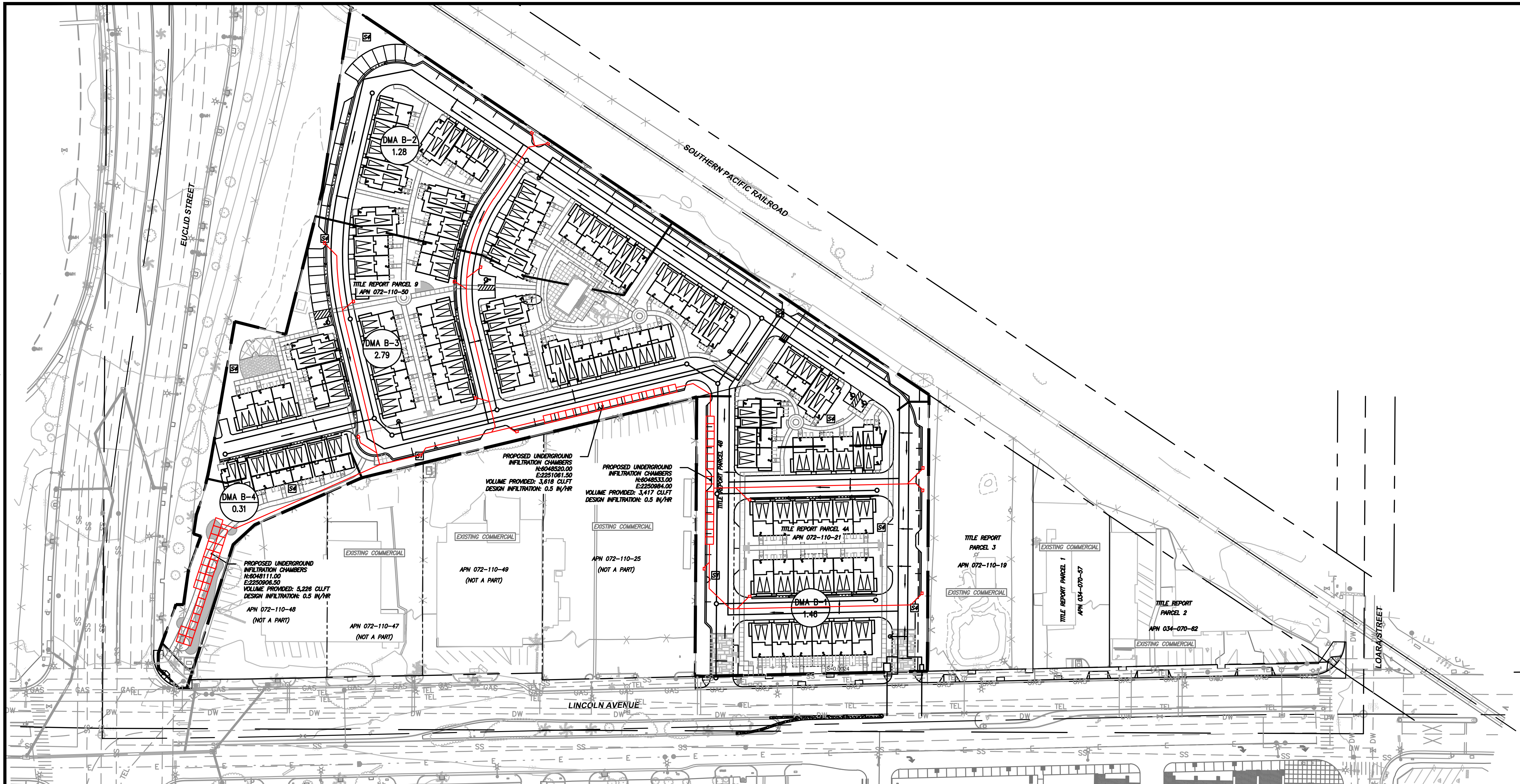
Prepared for:

SLF-WEST LINCOLN, LLC

ROUX

Compiled by: M.A.E.	Date: 15JAN19	FIGURE
Prepared by: A.T.	Scale: 1" = 100'	2
Project Mgr: D.D.	Project: 3224.0003L000	
File: FIGURE2-SITE PLAN.DWG		

S:\LOS ANGELES\CLIENTS\SHOPOFF\SLF-WEST LINCOLN - ANAHEIM 3224.0003L000\05WORKABLES\REPORT\FIGURES\CAD\FIGURE2-SITE PLAN.DWG



LEGEND

RIGHT OF WAY	STORM DRAIN LINE
PROPERTY LINE	STORM DRAIN INLET
STREET CENTERLINE	E-1 XX.XX –SUBAREA DESTINATION –SUBAREA ACREAGE
SUBAREA BOUNDARY	FLOW DIRECTION
FLOW PATH	

S1 STORM DRAIN MESSAGE AND SIGNAGE

S3 TRASH AND WASTE STORAGE

S4 EFFICIENT IRRIGATION

N

100' 0 100'

Title:

PROPOSED INFILTRATION BMP LOCATIONS

1631 AND 1699 WEST LINCOLN AVENUE
ANAHEIM, CALIFORNIA

Prepared for:

SLF-WEST LINCOLN, LLC

ROUX

Compiled by: A.T.	Date: 28MAR19	FIGURE
Prepared by: A.T.	Scale: 1" = 100'	3
Project Mgr: D.D.	Project: 3224.0003L000	
File: BMP EXHIBIT.DWG		

\\NY.ROUXINC.COM\SHARED\LOS ANGELES\CLIENTS\SHOPOFF\SLF-WEST LINCOLN - ANAHEIM 3224.0003L000\05WORKABLES\SITE-SPECIFIC STUDY (INFILTRATION SUPPORT)\FIGURES\FIGURE 3 FILES\FIG 3 CAD FILES\BMP EXHIBIT.DWG

APPENDIX H

General Earthwork and Grading Specifications for Rough Grading

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 a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant 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.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent bas

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 project 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 "equipment" 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 personnel will be available for observation and testing. 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. It is the contractor's sole responsibility to provide proper fill compaction.

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 10 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. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

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 from oversize material and the working surface is reasonably uniform, flat, and free from 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 from 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 D1557-91).

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 D1557-91). 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 back-rolling of slopes with sheeps-foot 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 D1557-91.

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

7.2 All bedding and backfill of utility trenches shall be done 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 maximum from 1 foot above the top of the conduit to the surface.

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

7.4 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.

7.5 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.