LGC Geotechnical, Inc.

June 14, 2019

Project No. 19063-01

Mr. Jeremy Ogulnick
RHW Holdings
240 Newport Center Drive, Suite 200
Newport Beach, CA 92660

Subject: ***Geotechnical EIR/Due-Diligence Level Report for the Proposed Mixed-Use Development, The Bowery, 2300 Red Hill Avenue, Santa Ana, California***

In accordance with your request, LGC Geotechnical, Inc. is providing this geotechnical EIR/due-diligence level report for the proposed mixed-use development located at 2300 Red Hill Avenue in the City of Santa Ana, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to confirm that the site can be developed from a geotechnical perspective.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
RYAN L. DOUGLAS
No. 84840
CIVIL
STATE OF CALIFORNIA

Ryan Douglas, RCE 84840
Project Engineer

Dennis Boratynec, GE 2770
Vice President

REGISTERED PROFESSIONAL ENGINEER
DENNIS BORATYNEC
No. 2770
GEOTECHNICAL
STATE OF CALIFORNIA

RLD/DJB/CNJ/aca

Distribution: (4) Addressee (3 wet signed copies for agency submittal and 1 electronic copy)
(1) EPD Solutions, Inc. (electronic copy)
Attn: Mr. Jeremy Krout

TABLE OF CONTENTS

LIST OF TABLES, ILLUSTRATIONS, & APPENDICES

Tables

Figures

Appendices

1.0 <u>INTRODUCTION</u>

1.1 <u>Purpose and Scope of Services</u>

This report presents the results of our geotechnical EIR/due-diligence level report for the proposed mixed-use development located at 2300 Red Hill Avenue in the City of Santa Ana, California (see Site Location Map, Figure 1). The conceptual site plan by Architects Orange (AO, 2019) and site topo from Fuscoe Engineering (Fuscoe, 2019) was utilized as a base map for our Geotechnical Exploration Location Map (Figure 2).

The purpose of our study was to evaluate the existing onsite geotechnical conditions and to confirm that the site can be developed from a geotechnical perspective. As part of this report, we have: 1) reviewed available geotechnical reports, geologic maps, and satellite images pertinent to the site (Appendix A); 2) performed a limited subsurface geotechnical evaluation of the site consisting of the excavation of five small-diameter borings ranging in depth from approximately 5 to 50 feet below existing ground surface; 3) performed two field infiltration tests; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this EIR/due-diligence level geotechnical evaluation report presenting our findings, conclusions and preliminary recommendations as it relates to the proposed mixed-use development.

The findings and conclusions presented herein should be considered preliminary and will need to be confirmed as part of a grading plan review report to be provided at a later date. It should be noted that LGC Geotechnical does not provide environmental consulting services.

1.2 <u>Project Description</u>

Based on the provided information and conceptual site plans by Architects Orange (AO, 2019), the proposed mixed-use development will consist of three parking structures along with at-grade retail and residential buildings. Parking Structure "A" will have 7 stories, Parking Structure "B" will have 6 stories and Parking Structure "C" will have 6-stories. The residential buildings are up to 7 stories high and amenity areas are planned within the residential and retail areas. Parking structures are anticipated to be reinforced concrete and the retail and residential structures are anticipated to be wood-framed.

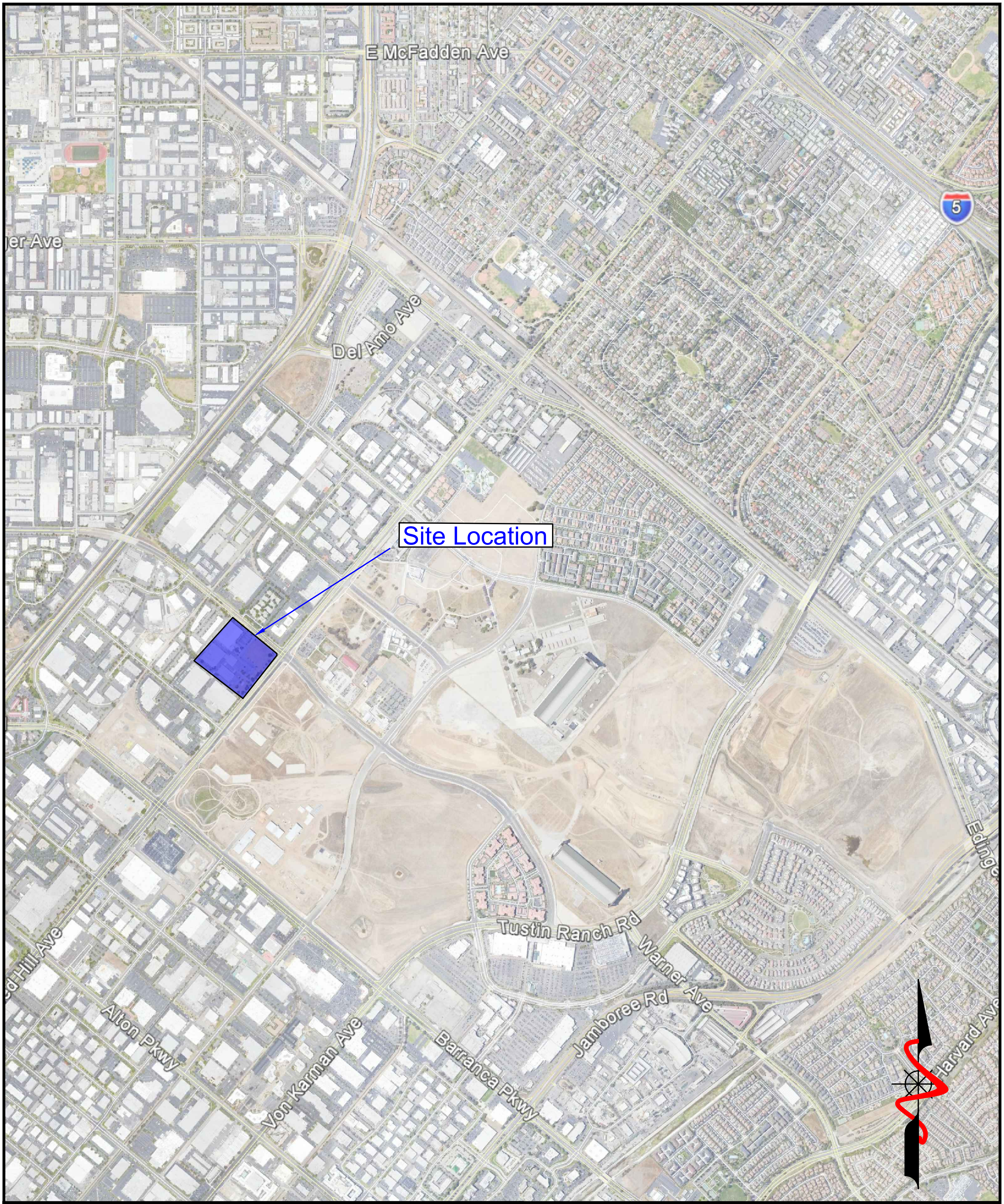
We anticipate finish grades will not vary significantly (± 4 feet) from current grade. Two on-grade swimming pools and spas are proposed in the courtyard areas and one rooftop swimming pool, spa and deck amenity area is proposed on the roof of Parking Structure "A". A number of courtyards with lounges and other amenities are also proposed. A retail plaza with multiple retail buildings and on-grade parking is proposed in the south eastern portion of the site adjacent to Redhill Avenue. Presented in Table 1 is a summary of our <u>estimated</u> structural (dead plus live) loads for the proposed 7-story mixed-use residential/retail structures and the proposed 7-story parking structures. Please note that structural loads and a preliminary grading plan were not provided to us at the time of this report.

<u>TABLE 1</u>

<u>Estimated Structural Loads</u>

Planned Structure	Column Loads (kips)	Wall Loads (kip/ft)
7-Story Parking Structure(s)	1200	-
7-Story Mixed-Use Structures	200	10

The preliminary recommendations given in this report are based upon the proposed layout and estimated structural loading information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and the actual structural loads when they become available, in order to either confirm or modify the recommendations provided herein. This may include but is not limited to additional subsurface borings/CPTs, laboratory testing and analysis to provide a design level geotechnical report.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	EPD - The Bowery, Santa Ana
PROJECT NO.	19063-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	June 2019

1.3 *Existing Conditions*

The relatively flat site is approximately 15 acres and is bound in the north easterly direction by Warner Avenue, in the south easterly direction by Red Hill Avenue and in the south westerly and north westerly directions by existing commercial/industrial buildings (see Figure 1 – Site Location Map). The site currently consists of three large existing industrial buildings, associated at-grade parking and drive aisles, and turf covered open space. Existing elevations range from approximately 57 to 65 feet above mean sea level (msl). In general, the site drains from north to south.

1.4 *Previous Site Geotechnical Information*

Review of historic aerial photographs indicate that in 1946 the site was agricultural land with a small residence in the eastern corner of the property. Red Hill and Warner Avenue are constructed in their present locations. Army barracks related to the Naval Reservation are visible across Red Hill Avenue. By 1980, the Ricoh Electronics Building and its parking lot at 2300 Red Hill Avenue was constructed. Adjacent buildings to the northwest and southwest of the site were constructed. By 1994, the site was constructed to its present condition (Historic Aerials, 2019).

Review of previous geotechnical compaction reports indicates much of the site fill soils have been geotechnically observed and tested. Documentation consists of the following reports:

In 1979, G.A. Nicoll & Associates (Nicoll) performed geotechnical observation and testing for the Ricoh Electronics Building at 2300 Red Hill Avenue (Nicoll, 1979a and 1979b). Geotechnical observation was primarily for the over-excavation of the building pad, underground structures, and associated utilities. After the area was cleared of vegetation and debris, soils within the building pad were excavated to depths of approximately 5 feet below existing grade. Fill was placed in approximate 8-inch thick lifts and compacted with heavy compaction equipment. Where tested using ASTM Test Method D1556, compaction soils were found to meet the project requirement of at least 90 percent relative compaction, as determined by ASTM Test Method D1557. Nicoll concluded, based on their observation and testing, that fill soils were compacted to at least the minimum required relative compaction.

In 1981, Nicoll performed geotechnical observation and testing for the REZ Toner Building at 2310 Red Hill Avenue (Nicoll, 1981). Geotechnical observation was primarily for the over-excavation of the building pad. After the area was cleared of vegetation and debris, soils within the building footprint and mechanical pit areas were excavated. Fill was placed in approximate 8-inch thick lifts and compacted with heavy compaction equipment. Where tested using ASTM Test Method D1556, soils were found to meet the project requirement of at least 90 percent relative compaction, as determined by ASTM Test Method D1557. Up to approximately 16.5 feet of artificial fill was placed in the mechanical pit area and up to approximately 9 feet of fill was placed in the building pad area. Fill was placed up to 15 feet beyond the limit of the building foundations. Nicoll concluded, based on their observation and testing, that fill soils were compacted to at least the minimum required relative compaction.

In 1989, Lotus Consulting Engineers (Lotus) performed geotechnical observation and testing for the removal of an underground storage tank located between 2310 and 2320 Red Hill Avenue. The tank was removed, approximately 10 feet of crushed miscellaneous base followed by approximately 5 feet of onsite soils were placed in approximate 8-inch thick lists and compacted with heavy compaction equipment. Where tested, soils were found to meet the project requirement of at least 92 percent and 90 percent relative compaction for the crushed miscellaneous base and onsite soil, respectively, as determined by ASTM Test Method D1557. Lotus concluded, based on their observation and testing, that fill soils were compacted to at least the minimum required relative compaction.

In 1990, Nicoll performed geotechnical observation and testing for an addition to the Thermal Paper Plant at 2320 Red Hill Avenue (Nicoll, 1990). Geotechnical observation was primarily for the over-excavation of the building pad for a maintenance building addition. After the area was cleared of vegetation and debris, upper soils within the building addition pad were excavated approximately 10 to 13 feet below existing grade. The bottom was stabilized with approximately 24 inches of gravel prior to fill placement. Fill was placed in approximate 8-inch thick lifts and compacted with heavy compaction equipment. Where tested using ASTM Test Method D1556, soils were found to meet the project requirement of at least 90 percent relative compaction, as determined by ASTM Test Method D1557. Up to approximately 13 feet of artificial fill was placed and fill was placed up to 5 feet beyond the limit of the building foundations. Nicoll concluded, based on their observation and testing, that fill soils were compacted to at least the minimum required relative compaction.

1.5 <u>Subsurface Exploration</u>

A geotechnical field evaluation was performed by LGC Geotechnical. This program consisted of drilling and sampling five small-diameter borings.

The borings were drilled by CalPac Drilling under subcontract to LGC Geotechnical. The depth of the borings ranged from approximately 5 to 50 feet below existing grade. The upper approximate 5 feet were hand-augered due to potential utility line conflicts. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were performed using a B-61 truck-mounted drill rig equipped with 6-inch and 8-inch diameter hollow-stem augers. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch tall brass rings. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged for laboratory testing at select depths. At the completion of drilling, the borings were backfilled with cement bentonite and the surface was replaced with asphalt cold-patch.

Infiltration testing was performed within two of the borings (I-1 through I-2) to depths of 5 feet below existing grade. An LGC Geotechnical staff geologist installed standpipes, backfilled the borings with crushed rock and pre-soaked the infiltration holes prior to testing. Infiltration testing was performed per the County of Orange testing guidelines. The locations were subsequently backfilled with native soils at the completion of testing.

Boring logs are presented in Appendix B and their approximate locations are depicted on Figure 2.

1.6 Laboratory Testing

Representative driven and bulk samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ unit weight and moisture content, fines content, Atterberg Limits (liquid limit and plastic limit), consolidation, laboratory compaction, expansion index, and corrosion (sulfate, chloride, pH, and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 91 pounds per cubic foot (pcf) to 126 pcf, with an average of 108 pcf. Field moisture contents ranged from approximately 14 percent to 36 percent, with an average of 22 percent.
- Four fines content (percent passing No. 200 sieve) tests ranging from approximately 47 percent to 93 percent. Based on the Unified Soils Classification System (USCS), three of the tested samples would be classified as "fine-grained" and one sample would be classified as "coarse-grained."
- Two Atterberg Limit (liquid limit and plastic limit) tests were performed. Results indicated Plasticity Index values ranging from 11 to 32.
- Two consolidation tests were performed. The stress vs. deformation plots are provided in Appendix C.
- One laboratory compaction test of a near surface sample indicated a maximum dry density of 122.5 pcf with an optimum moisture content of 12.0 percent.
- Two Expansion Index (EI) tests were performed. Results were EI values of 25 and 44, corresponding to "Low" expansion potential.
- Corrosion testing indicated soluble sulfate contents of approximately 0.1 percent or less, chloride content of 100 parts per million (ppm), pH value of 7.5, and minimum resistivity value of 515 ohm-cm.

A summary of the laboratory test results is presented in Appendix C.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is generally located within the Peninsular Ranges Geomorphic Province of California, more specifically at the eastern edge of the Los Angeles Sedimentary Basin. The Los Angeles Basin is a northwest-plunging synclinal sedimentary deposit that is bounded to the south of the subject site by the broadly uplifted coastal mesa of Newport Beach and the San Joaquin Hills, to the north by the foothills of the Santa Ana mountain range. The site is located on young alluvial fan materials that include previous floodplain deposits. A channelized portion of the Peters Canyon Creek passes approximately two miles away from the site to the east. The creek drains into Upper Newport Bay located south of the site (Morton, 2004 & CDMG, 2001b).

2.2 Site-Specific Geology

The subject site covers a rectangular-shaped parcel on relatively flat alluvial flood plains, typical of the Los Angeles Basin. Based on our subsurface exploration and review of pertinent geologic literature and maps, the site is generally underlain by older artificial fill soils and Quaternary-aged young alluvial fan deposits.

It should be noted that geotechnical explorations are only representative of the location where they are performed and varying subsurface conditions may exist outside of each location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform and that soil is homogeneous within the project area. A brief description of the materials encountered during drilling is presented in the following section, and the approximate boring locations are depicted on the Geotechnical Exploration Location Map (Figure 2). For details on the stratigraphy at the exploration locations, refer to the boring and test pit logs provided in Appendix B

2.2.1 Older Artificial Fill (Map Symbol – afo)

Older artificial fill was observed in the field explorations up to 7.5 feet below existing grade in borings HS-1 though HS-3. The fill was observed to consist of slightly moist to moist clays and silts with variable amounts of sand.

2.2.2 Quaternary Young Alluvial Fan Deposits (Map Symbol – Qyf)

Quaternary young alluvial fan deposits were observed underlying the older artificial fill. Where observed, the alluvial materials generally consisted of moist to wet, medium stiff to hard clays with variable sand content, as well as loose to medium dense, moist to wet clayey and silty sands to the maximum explored depth of approximately 50 feet below existing grade.

2.3 Landslides and Slope Stability

Document research and field observations do not indicate the presence of landslides on the site or in the immediate vicinity (Morton, 2004). Review of the Seismic Hazards Zone Map (CDMG, 2002b) and the Seismic Hazard Zone Report (CDMG, 2001a) for the Tustin 7.5 Minute Quadrangle indicates that the site is not located within a mapped area considered potentially susceptible to seismically-induced slope instability.

2.4 Groundwater

The measured depth of groundwater in our borings ranged from approximately 24 to 33 feet below existing grade. Historic high groundwater is estimated to be about 10 feet below existing grade (CDMG, 2001a).

It should be noted that higher localized and seasonal perched groundwater conditions may accumulate below the surface, and should be expected throughout the design life of the proposed improvements. In general, groundwater conditions below any given site may vary over time depending on numerous factors including seasonal rainfall and local irrigation among others.

2.5 Field Infiltration Testing

One field percolation test was performed in the area of the proposed infiltration trench, as directed by the project civil engineer, and the location is depicted on Figure 2 – Geotechnical Exploration Location Map. Test well installation consisted of placing a 3-inch diameter perforated PVC pipe in the excavated borehole and backfilling the annulus with crushed rock including the placement of approximately 2 inches of crushed rock at the bottom of the borehole. The infiltration test well was presoaked the day of installation and testing took place within 24 hours of presoaking. During the pre-test the water level was observed to drop less than 6 inches in 25 minutes for two consecutive readings. Therefore, the test procedure for fine-grained soils or “slow test” was followed. Test well installation and the estimation of infiltration rates were accomplished in general accordance with the guidelines set forth by the County of Orange (2013). In general, three-dimensional flow out of the test well (*percolation*), as observed in the field, is mathematically reduced to one-dimensional flow out of the bottom of the test well (*infiltration*). Infiltration tests are performed using relatively clean water, free of particulates, silt, etc. The results of our recent field infiltration testing are presented in Appendix D and summarized below.

TABLE 2

Summary of Field Infiltration Testing

Infiltration Test Identification	Approx. Depth Below Existing Grade (ft)	Observed Infiltration Rate* (in./hr.)	Measured Infiltration Rate** (in./hr.)
I-1	5	0.3	0.15

*Observed Infiltration Rates Do Not Include Factor of Safety.

**Measured Infiltration Rates Include a Factor of Safety of 2 in Order to Evaluate Feasibility.

The tested infiltration rates provided in this report are considered a general representation of the infiltration rates at the location of the proposed infiltration trench. Please note, the testing of infiltration rates is highly dependent upon the materials encountered at the point of testing (i.e. location and depth of testing). Varying subsurface conditions may exist outside of the test location which could alter the calculated infiltration rate.

2.6 <u>Seismic Design Parameters</u>

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 CBC. Since the site contains soils that are susceptible to liquefaction (refer to above Section "Liquefaction and Dynamic Settlement"), ASCE 7 which has been adopted by the CBC requires that site soils be assigned Site Class "F" and a site-specific response spectrum be performed. However, in accordance with Section 20.3.1 of ASCE 7, if the fundamental periods of vibration of the planned structure are equal to or less than 0.5 second, a site-specific response spectrum is not required and ASCE 7/2016 CBC site class and seismic parameters may be used in lieu of a site-specific response spectrum. **It should be noted that the seismic parameters provided herein are not applicable for any structure having a fundamental period of vibration greater than 0.5 second. Should the structural engineer determine that any of the proposed structures have a fundamental period of vibration greater than 0.5 seconds, a site-specific response spectrum will have to be prepared.**

Representative site coordinates of latitude 33.7099 degrees north and longitude -117.8395 degrees west 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 F modified to Site Class D for building structures with a period of vibration equal to or less than 0.5 second are provided in Table 3 on the following page.

TABLE 3

Seismic Design Parameters for Structures with a Period of Vibration ≤ 0.5 Second

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)**	1.508g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)**	0.558g
Site Coefficient F_a per Table 1613.3.3(1)	1.000
Site Coefficient F_v per Table 1613.3.3(2)	1.500
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_a S_S$]	1.508g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_v S_1$]	0.837g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3) S_{MS}$]	1.006g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3) S_{M1}$]	0.558g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	1.001
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	1.034

* Site is Class F, seismic parameters provided herein are only applicable for structure period ≤ 0.5 second, refer to discussion above.
** From SEAOC, 2019

Section 1803.5.12 of the 2016 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.565g (SEAOC, 2019).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.9 at a distance of approximately 3.1 km from the site would contribute the most to this ground motion (USGS, 2008).

2.7 Faulting

The subject site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CGS, 2018). A fault is considered "active" if evidence of surface rupture in Holocene time (the last approximately 11,000 years) is present. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site. The closest known active faults are associated with the San Joaquin Hills Fault, located approximately 1.5 miles from the site; and the Newport-Inglewood Fault Zone, approximately 8.4 miles southwest of the site; and the Elsinore Fault Zone, approximately 13.2 miles northeast of the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching and shallow ground rupture, soil liquefaction, and dynamic settlement. 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 onsite geology. A discussion of these secondary effects is provided in the following sections.

2.7.1 Liquefaction and Dynamic Settlement

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 coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that loose, saturated, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 2001b), the site is located within a liquefaction hazard zone. In general, site soils consist of medium to high plasticity clays and silts and are not susceptible to liquefaction (Bray & Sancio, 2006). However, based on our field data, relatively isolated loose to medium dense sand layers, generally located approximately 40 to 50 feet below existing grade, are considered susceptible to liquefaction. The recent encountered in-situ groundwater depth of 25 feet below existing grade and historic high groundwater depth of 10 feet below existing grade were both used in the liquefaction analysis. The liquefaction evaluation was performed using data from boring HS-2. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and based on the seismic criteria of the 2016 California Building Code (CBC) and historic high groundwater depth. Liquefaction induced settlement was estimated using the PGA_M per the 2016 CBC and a moment magnitude of 6.9 (USGS, 2008).

Based on the PGA_M and our preliminary liquefaction analysis, seismic settlement potential in the upper approximate 50 feet is estimated to be on the order of 2 inches or less. Differential seismic settlement can be estimated as 1-inch over a horizontal span of about 40 feet. Seismically induced settlements were estimated by the procedure outlined by Tokimatsu and Seed (1987). Liquefaction calculations are provided in Appendix E.

2.7.2 Lateral Spreading

Lateral spreading is a type of liquefaction induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Site soils are generally medium to high plasticity clays and silts and are not susceptible to liquefaction (Bray & Sancio, 2006). However, relatively isolated loose to medium dense sand layers are present and considered susceptible to liquefaction. These isolated layers were generally encountered at depths greater than 40 feet below existing grade. Based on the relatively flat topography of the site, lack of a free face nearby and general lack of potentially liquefiable layers in the upper 40 feet, the potential for lateral spreading is considered low.

2.8 Expansion Potential

Based on the results of our recent laboratory testing and previous background reports, site soils are anticipated to have a "Low" to "High" expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and understanding of the proposed redevelopment, it is our opinion that the proposed development is feasible from a geotechnical standpoint. A summary of our conclusions are as follows:

- The field explorations generally indicate medium stiff to hard fine-grained clays interbedded with layers of medium dense sands with varying fines content to the maximum explored depth of approximately 50 feet below existing grade.
- Approximately 7.5 feet of previously placed undocumented artificial fill over native alluvial fan deposits was observed during this subsurface evaluation. Previous reports indicate that older artificial fill may be present up to approximately 17 feet below existing grade. Older artificial fill soils are primarily clays and silts with variable amounts of sand. Native alluvial fan deposits are primarily medium stiff to hard clays with variable sand content, as well as loose to medium dense clayey and silty sands to the maximum explored depth of approximately 50 feet below exiting grade.
- The near-surface soils are generally loose, dry and collapsible and are not suitable for the planned improvements in their present condition; removal and recompaction will be required.
- Groundwater was encountered during our recent subsurface evaluation at depths ranging from approximately 24 to 33 feet below existing ground surface. Historic high groundwater for the site is about 10 feet below existing ground surface (CDMG, 2001a).
- The site is located within a State of California Seismic Hazard Zone for liquefaction potential (CDMG, 2001b). In general, site soils generally consist of medium to high plasticity clays and silts and are not susceptible to liquefaction (Bray & Sancio, 2006). However, based on our field data, relatively isolated loose to medium dense sand layers generally located approximately 40 to 50 feet below existing grade are considered susceptible to liquefaction. Based on the PGA_M and our preliminary liquefaction analysis, seismic settlement potential in the upper approximate 50 feet is estimated to be on the order of 2 inches or less. Differential seismic settlement can be estimated as 1-inch over a horizontal span of about 40 feet.
- Based on the relatively flat topography of the site, lack of a free face nearby and general lack of potentially liquefiable layers in the upper 40 feet, the potential for lateral spreading is considered low.
- Due to site liquefaction potential, a site-specific response spectrum (not provided herein) will be required for any proposed structure with a fundamental period of vibration greater than 0.5 second.
- The proposed development will likely be subjected to strong seismic ground shaking during its design life. The site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CGS, 2018).
- Site contains soils with a “Low” to “High” expansion potential. Mitigation measures will be required for building foundations and flatwork.
- Based on Caltrans Corrosion Guidelines (2015), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2

percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria (Caltrans, 2015).

- Due to the upper approximate 20 feet of the site consisting of fine-grained clays, presence of shallow groundwater and site liquefaction potential, intentional infiltration of storm water is not considered recommended.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that the sandy and silty earth materials generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.
- Site contains clayey soils with high fines content and expansion potential that are not suitable for backfill of retaining walls. Therefore, import of sandy soils meeting project recommendations will be required.

4.0 RECOMMENDATIONS

A design-level geotechnical report based on the project grading and foundation plans should be prepared in order to provide design-level geotechnical recommendations (as necessary) for the proposed development. Additional field work and laboratory testing will likely be required. Additional and/or modified geotechnical recommendations will also likely be required.

Based on our preliminary EIR/due-diligence level study, the following is a summary of our preliminary geotechnical recommendations.

- From a geotechnical perspective, the site is feasible for construction of the proposed mixed-use residential development, provided that the recommendations of the geotechnical consultant are followed and the grading is performed in general accordance with applicable plans, codes, and City of Santa Ana requirements. The site was evaluated to the 2016 California Building Code (CBC) standard.
- All undocumented fill and unsuitable soft alluvial deposits shall be removed to suitable competent native materials prior to placement of proposed artificial fill. Recommendations for removal and recompaction and removal depths will be provided in a subsequent comprehensive report.
- Native alluvial soils are generally considered fine grained, overconsolidated and moderately compressible clay soils that are anticipated to consolidate when building loads are applied. From a geotechnical perspective, onsite soils are anticipated to be suitable for use as general compacted fill (not retaining wall backfill) provided they are screened of organic materials, construction debris and any oversized material (8 inches in greatest dimension). It should be emphasized that soils in the upper approximate 10 to 20 feet above groundwater are generally well above optimum moisture content and will require significant moisture conditioning (drying back) to achieve adequate compaction.
- Due to the high moisture contents, soft and yielding soils near removal bottoms may be exposed to wet and pumping conditions. Crushed rock may be paced over the soft wet removal bottom soils to stabilize the subgrade and provide a base for the compaction of the required fill.
- Ideally, import soils should consist of non-corrosive (negligible sulfates and low chlorides) and predominantly granular soils with an Expansion Index (EI) of 20 or less. However, the minimum criteria for import soils may be changed by you. Potentially acceptable import soils should; therefore, not exceed sulfate levels of 1 percent by weight, chloride levels of 500 ppm or Expansion Indices of 130. It should be noted that increasing the minimum criteria for import soils such as sulfate content and Expansion Index would adversely affect the design of the foundations. Higher concrete compressive strengths and thicker more rigid slab sections would be needed to mitigate against expansive/corrosion imported soils.
- Due to the proximity of the proposed improvements to the property line in portions of the site, temporary shoring or "A-B-C" slot cuts may be required to achieve required earthwork removal and recompaction.
- Preliminary settlement estimates, <u>based on our estimated building loads</u>, are on the order of 2 to 4 inches for the parking structures and 1 to 2 inches for the apartment buildings. Please note that these are very preliminary estimates based on our estimated building loads. These assumptions must be verified based on additional subsurface work such as borings/CPTs and laboratory testing and re-

evaluated based on actual building/structure loads from the structural engineer. Please note the above settlement estimates are very preliminary and do not take into account earthwork removals, ground improvement (e.g., geopiers) or deep foundation systems.

- The proposed parking structures will likely have to be supported on ground improvement (e.g., geopiers) or a deep foundation system (piles). This is a result of the anticipated column loads and the presence of fine-grained relatively compressible clay soils in the upper approximate 10 to 30 feet, which would result in long-term settlement beyond tolerable limits. However, due to the presence of isolated sandy layers to approximately 45 feet below existing ground surface and lack of a "bearing stratum" at depth, deep foundations are likely not feasible. We recommend the parking structures be supported on ground improvement such as Rammed Aggregate Piers (RAP), also known as geopiers. Additional field work, laboratory testing and analysis will have to be performed once actual building loads are known to further evaluate and confirm this.
- The proposed 5 and 7-story apartments will likely be supported on a rigid shallow conventional foundation system designed to resist site expansive soils and anticipated long-term static settlement provided recommended earthwork removal and re-compaction is performed. Foundation design should be based on the expansion index of the site soils. Additional field work, laboratory testing and analysis will have to be performed once actual building loads are known to further evaluate and confirm this.
- Due to potential elevated pedestrian walkway structures between the parking structure and apartment buildings, total and differential settlement of the proposed structures may have to be considered in order to maintain level transitions between the structures.
- Based on laboratory sulfate test results, the near surface soils are anticipated to be designated to a class "S1" per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the "S1" sulfate classification.
- Geotechnical stability and integrity of the project site is reliant upon appropriate handling of surface water. Due to the low infiltration rate, shallow groundwater and site liquefaction potential, we strongly recommend against the intentional infiltration of storm water.
- Additional geotechnical subsurface evaluation must be performed in order to provide design-level geotechnical recommendations for the proposed development. Based on our preliminary study, we recommend performing additional field work consisting of Cone Penetration Test (CPT) soundings and additional borings or test pits. Further evaluation of required earthwork removals, soil settlement due to static building loads and liquefaction must be performed. Based on the results of our field evaluation and laboratory testing, updated and/or amended geotechnical conclusions and recommendations may be required.
- Final design level recommendations utilizing the site grading plans and structure loads should be provided as part of a comprehensive geotechnical report. Additional field work and laboratory testing, as mentioned above, should be anticipated.

5.0 LIMITATIONS

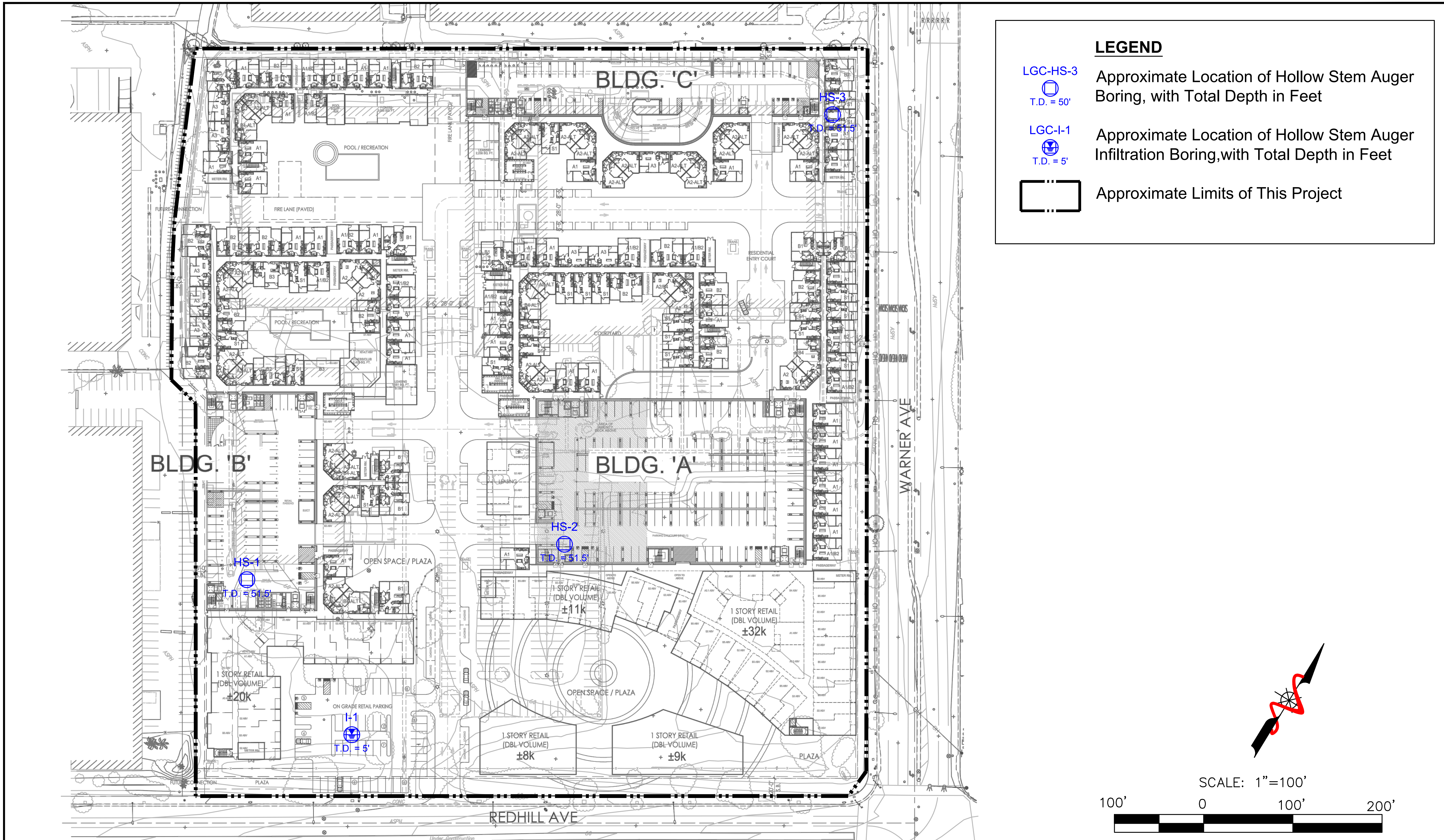
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during construction.

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 other consultants and incorporated into the plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site 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. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

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. Therefore, this report is subject to review and modification.



LGC Geotechnical, Inc.

LGC Geotechnical, Inc.
131 Calle Iglesia, Ste. 200
San Clemente, CA 92672
TEL (949) 369-6141 FAX (949) 369-6142

FIGURE 2
Geotechnical Exploration Location Map

PROJECT NAME	The Bowery - Santa Ana
PROJECT NO.	19063-01
ENG. / GEOL.	RLD / KTM
SCALE	1" = 100'
DATE	June, 2019

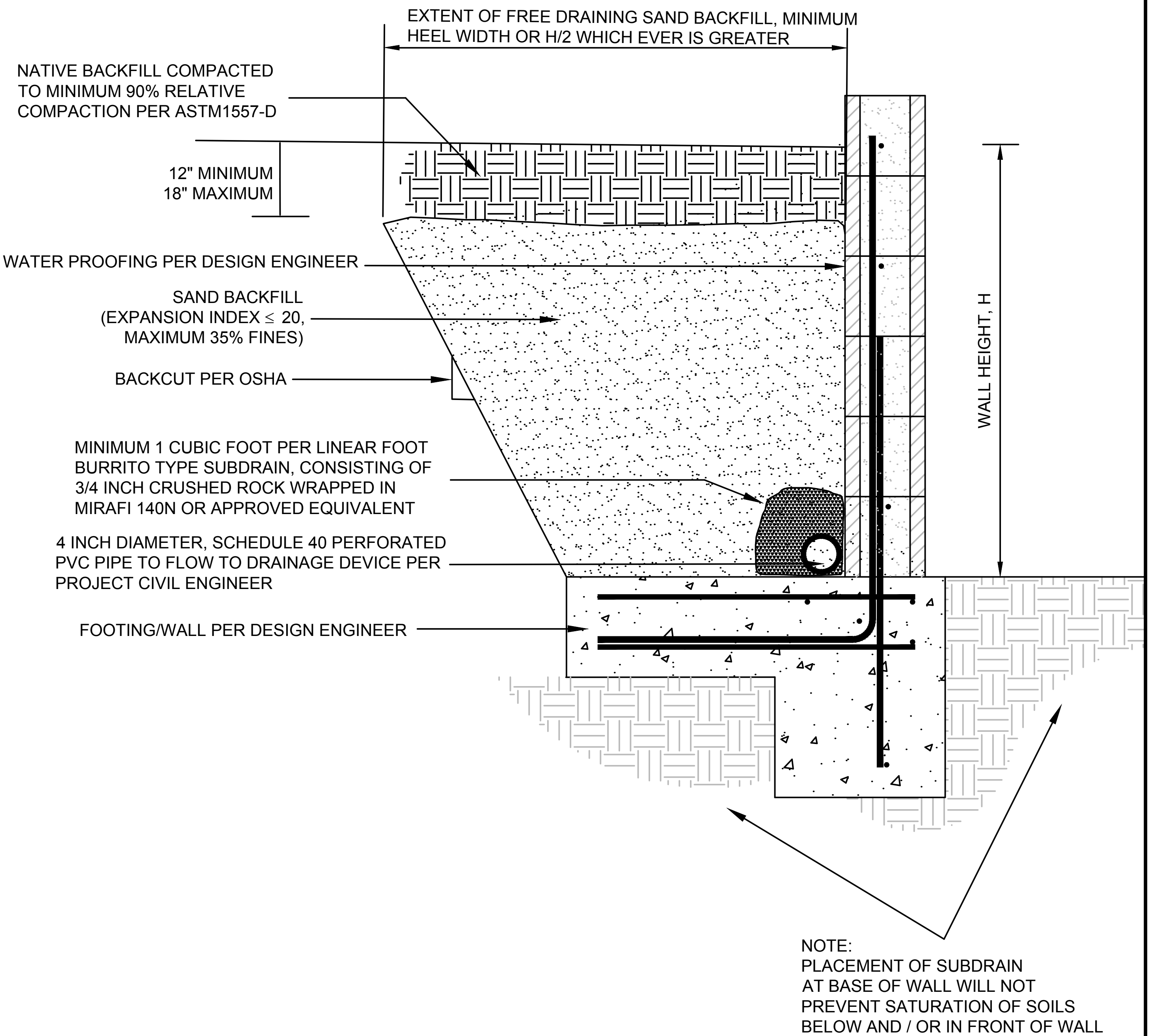


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FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	EPD - The Bowery, Santa Ana
PROJECT NO.	19063-01
ENG.	RLD
SCALE	Not to Scale
DATE	June 2019

Appendix A
References

APPENDIX A

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Appendix B
Boring Logs

Geotechnical Boring Log Borehole HS-1

Date: 5/7/2019	**Drilling Company:** Cal Pac Drilling
Project Name: The Bowery	**Type of Rig:** B-61
Project Number: 19063-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~59' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to 5' - <u>Older Artificial Fill (afo)</u>**	
								@0' Approximately 5 inches AC over 5 inches AB.	
		B-1					CL	@1' - CLAY with Sand: brown, moist, stiff	CR
55								**@5' to T.D. - <u>Quaternary Young Alluvial Fan Deposits (Qyf)</u>**	
	5		R-1	4 4 6	115.2	14.2	CL	@5' - Sandy CLAY: brown, moist, stiff; scattered pebbles	
			R-2	3 5 9	113.0	18.1	SC	@7.5' - Clayey SAND: brown, very moist, medium dense	
50	10		R-3	2 4 9	109.2	19.0	ML	@10' - SILT with Sand: Gray-brown, very moist, stiff; slight chemical odor	
45	15		SPT-1	1 1 3		25.8		@15' - Sandy SILT: gray with red-orange, very moist, medium stiff	
40	20		R-4	3 4 6	100.1	26.5	CL	@20' - CLAY: gray and red-orange, very moist, stiff	
35								@24' - Groundwater encountered	
	25		SPT-2	1 2 4		20.8		@25' - CLAY: brown, very moist to wet, stiff	
30	30								

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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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:

B	BULK SAMPLE
R	RING SAMPLE (CA Modified Sampler)
G	GRAB SAMPLE
SPT	STANDARD PENETRATION TEST SAMPLE
▽	GROUNDWATER TABLE

TEST TYPES:

DS	DIRECT SHEAR
MD	MAXIMUM DENSITY
SA	SIEVE ANALYSIS
S&H	SIEVE AND HYDROMETER
EI	EXPANSION INDEX
CN	CONSOLIDATION
CR	CORROSION
AL	ATTERBERG LIMITS
CO	COLLAPSE/SWELL
RV	R-VALUE
-#200	% PASSING # 200 SIEVE

Last Edited: 5/29/2019

Geotechnical Boring Log Borehole HS-1

Date: 5/7/2019	**Drilling Company:** Cal Pac Drilling
Project Name: The Bowery	**Type of Rig:** B-61
Project Number: 19063-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~59' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-5	10 11 15	112.9	18.2	CL	@30' - CLAY: brown and light brown, very moist, very stiff	
25									
	35		SPT-3	5 5 9		19.8		@35' - Sandy CLAY: light pinkish brown, very moist, very stiff	
20									
	40		R-6	7 9 17	111.7	18.8	ML	@40' - Sandy SILT: gray-brown and gray mottled, very moist, very stiff	
15									
	45		SPT-4	3 5 6		27.1	SC	@45' - Clayey SAND: brown, wet, medium dense	
10									
	50		R-7	6 8 10	103.7	25.6	ML	@50' - Sandy SILT: reddish brown and gray, wet, very stiff	
5								Total Depth = 51.5' Groundwater Encountered at Approximately 24' Backfilled with Cement Bentonite and Capped with AC Cold-Patch 5/7/2019	
	55								
	60								

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SAMPLE TYPES:
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE
GROUNDWATER TABLE

TEST TYPES:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 5/7/2019	**Drilling Company:** Cal Pac Drilling
Project Name: The Bowery	**Type of Rig:** B-61
Project Number: 19063-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~62' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						**@0' to 5' - <u>Older Artificial Fill (afo)</u>**	MD EI CR
								@0' - Approximately 5 inches AC over 5 inches AB	
60							CL	@1' - Below is CLAY with Sand: brown, moist, stiff	
	5		R-1	5 9 13	119.0	15.6	ML	@5' - SILT with SAND: brown, moist, very stiff	
								@7.5' to T.D. - <u>Quaternary Young Alluvial Fan Deposit (Qyf)</u>	
55		B-2	R-2	3 4 5	114.7	17.8	SC	@7.5' - Clayey SAND: brown, very moist, loose; micaceous	CN -#200
	10		R-3	4 3 3	91.4	30.8	CH	@10' - Fat CLAY: dark yellowish brown, very moist, medium stiff	CN AL
50									
	15		SPT-1	1 2 3		29.1		@15' - Fat CLAY: dark gray with some brown mottle, very moist, medium stiff	-#200
45									
	20		R-4	3 4 6	93.0	29.2	CL	@20' - CLAY: olive gray and brown mottled, very moist, stiff	
40									
	25		SPT-2	1 1 4		22.7		@25' - CLAY: brown, reddish brown, and gray mottled, very moist, medium stiff	AL -#200
35									
	30								

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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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 5/29/2019

Geotechnical Boring Log Borehole HS-2

Date: 5/7/2019	**Drilling Company:** Cal Pac Drilling
Project Name: The Bowery	**Type of Rig:** B-61
Project Number: 19063-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~62' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-5	7 9 10	103.0	24.3	CL	@30' - CLAY: brown and light brown mottled, very moist, very stiff	
30									
								@33.5' - Groundwater encountered	
	35		SPT-3	6 7 9		17.8		@35' - Sandy CLAY: reddish brown, moist to very moist, very stiff	AL -#200
25									
	40		R-6	6 10 11	113.5	17.4	SM	@40' - Silty SAND: brown and gray mottled, moist to very moist, very stiff	
20									
	45		SPT-4	5 9 13		32.0	SC	@45' - Clayey SAND: brown, wet, medium dense	
15									
	50		R-7	5 10 12	112.9	16.8	CL	@50' - Sandy CLAY: brown to reddish brown, moist, very stiff	
10								Total Depth = 51.5' Groundwater Encountered at Approximately 33.5' Backfilled with Cement Bentonite and Capped with AC Cold Patch 5/7/2019	
	55								
5									
	60								

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SAMPLE TYPES:
- B BULK SAMPLE
- R RING SAMPLE (CA Modified Sampler)
- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- ▽ GROUNDWATER TABLE

TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 5/7/2019	**Drilling Company:** Cal Pac Drilling
Project Name: The Bowery	**Type of Rig:** B-61
Project Number: 19063-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~62' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to 5' - Older Artificial Fill (afo)**	EI
								@0' - Approximately 4 inches AC over 5 inches AB	
60			B-1				SC	@1' - Clayey SAND: brown, slightly moist, stiff	
								@5' to T.D. - Quaternary Young Alluvial Fan Deposits (Qya)	
	5		R-1	6 8 12	120.0	13.8	SC	@5' - Clayey SAND: brown and gray mottled, moist, medium dense	
55			R-2	4 4 6	93.3	31.0	CL	@7.5' - Sandy CLAY: brown and gray mottled, very moist, stiff	
	10		R-3	8 8 7	99.4	22.4	SC	@10' - Clayey SAND: brown and gray mottled, very moist, medium dense	
50									
	15		SPT-1	1 2 3		17.0	CL	@15' - CLAY: gray and brown mottled, moist, medium stiff; micaceous	
45									
	20		R-4	3 4 7	99.5	25.9		@20' - CLAY: olive gray and brown mottled, very moist, stiff	CN AL
40									
	25	▽	SPT-2	2 1 2		35.7		@25' - CLAY: brown and gray mottled, wet, medium stiff; groundwater encountered	
35									
	30								

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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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:

B	BULK SAMPLE
R	RING SAMPLE (CA Modified Sampler)
G	GRAB SAMPLE
SPT	STANDARD PENETRATION TEST SAMPLE
▽	GROUNDWATER TABLE

TEST TYPES:

DS	DIRECT SHEAR
MD	MAXIMUM DENSITY
SA	SIEVE ANALYSIS
S&H	SIEVE AND HYDROMETER
EI	EXPANSION INDEX
CN	CONSOLIDATION
CR	CORROSION
AL	ATTERBERG LIMITS
CO	COLLAPSE/SWELL
RV	R-VALUE
-#200	% PASSING # 200 SIEVE

Last Edited: 5/29/2019

Geotechnical Boring Log Borehole HS-3

Date: 5/7/2019	**Drilling Company:** Cal Pac Drilling
Project Name: The Bowery	**Type of Rig:** B-61
Project Number: 19063-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~62' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By ARN
Sampled By ARN
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-5	6 8 10	116.7	17.3	CL	@30' - CLAY: reddish brown, moist to very moist, very stiff	.
30									
	35		SPT-3	5 9 9		17.4		@35' - CLAY with Sand: brown with tan mottling, moist to very moist, very stiff	.
25									
	40		R-6	10 20 19	126.1	14.0		@40' - Sandy CLAY with some Gravel: brown, moist, hard; scattered coarse grained sand	.
20									
	45		SPT-4	4 10 11		19.1	SC	@45' - Clayey SAND: brown, wet, medium dense	.
15									
	50		R-7	5 9 13				@50' - No Recovery	.
10								Total Depth = 51.5' Groundwater Encountered at Approximately 25' Backfilled with Cement Bentonite and Capped with AC Cold-Patch on 5/7/2019	
	55								
5									
	60								

LGC Geotechnical, 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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
B BULK SAMPLE
R RING SAMPLE (CA Modified Sampler)
G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
SA SIEVE ANALYSIS
S&H SIEVE AND HYDROMETER
EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory 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 a table summarizing the test results.

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

Grain Size Distribution/Fines Content: 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 (ASTM D1140). Where applicable, 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 D6913 (sieve) or ASTM D422 (sieve and hydrometer).

Sample Location	Description	% Passing # 200 Sieve
HS-2 @ 7.5 ft	Clayey Sand	47
HS-2 @ 15 ft	Clay	93
HS-2 @ 25 ft	Clay	88
HS-2 @ 35 ft	Sandy Clay	60

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined per ASTM D4318 for engineering classification of fine-grained material and presented in the table below. The USCS soil classification indicated in the table below is based on the portion of sample passing the No. 40 sieve and may not necessarily be representative of the entire sample. The plots are provided in this Appendix.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-2 @ 10 ft	55	23	32	CH
HS-2 @ 25 ft	36	19	17	CL
HS-2 @ 35 ft	32	12	20	CL
HS-3 @ 20 ft	33	22	11	CL

APPENDIX C (Cont'd)

Laboratory Test Results

Consolidation: Three consolidation tests were performed per ASTM D2435. A sample (2.4 inches in diameter and 1 inch in height) was placed in a consolidometer and increasing loads were applied. The sample was 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 provided in this Appendix.

Laboratory Compaction: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of this tests are presented in the table below.

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2 @ 0-5 ft	Brown clay with sand	122.5	12.0

Expansion Index: The expansion potential of select representative samples were evaluated by the Expansion Index Test per ASTM D4829.

Sample Location	Expansion Index	Expansion Potential*
HS-2 @ 0-5 ft	44	Low
HS-2 @ 0-5 ft	25	Low

* Per ASTM D4829

Soluble Sulfates: The soluble sulfate contents of a selected sample was determined by standard geochemical methods (CTM 417). The test results are presented in the table below.

Sample Location	Sulfate Content (%)
HS-1 @ 2-5 ft	0.06
HS-2 @ 0-5 ft	0.1

Chloride Content: Chloride content was tested per CTM 422. The results are presented below.

Sample Location	Chloride Content (ppm)
HS-2 @ 0-5 ft	100

Laboratory Test Results

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-2 @ 0-5 ft	7.5	515

ATTERBERG LIMITS

ASTM D 4318

Project Name:	Santa Ana	Tested By:	R. Manning	Date:	05/17/19
Project No. :	19063-01	Input By:	G. Bathala	Date:	05/24/19
Boring No.:	HS-2	Checked By:	J. Ward		
Sample No.:	R-3	Depth (ft.)	10.0		
Soil Identification:	Dark yellowish brown fat clay (CH)				

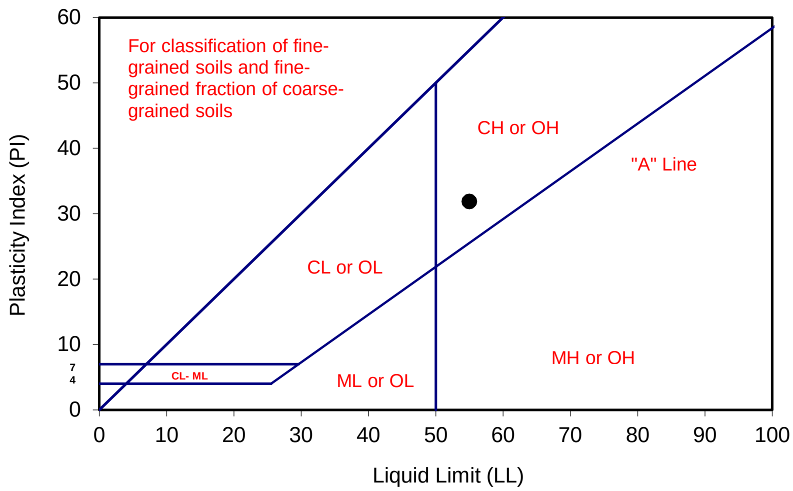
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	27	19	
Wet Wt. of Soil + Cont. (g)	18.81	18.15	23.51	23.29	23.87	
Dry Wt. of Soil + Cont. (g)	17.40	16.82	20.16	19.84	20.17	
Wt. of Container (g)	11.30	11.06	13.70	13.48	13.64	
Moisture Content (%) [Wn]	23.11	23.09	51.86	54.25	56.66	

Liquid Limit	**55**
Plastic Limit	**23**
Plasticity Index	**32**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 25.55

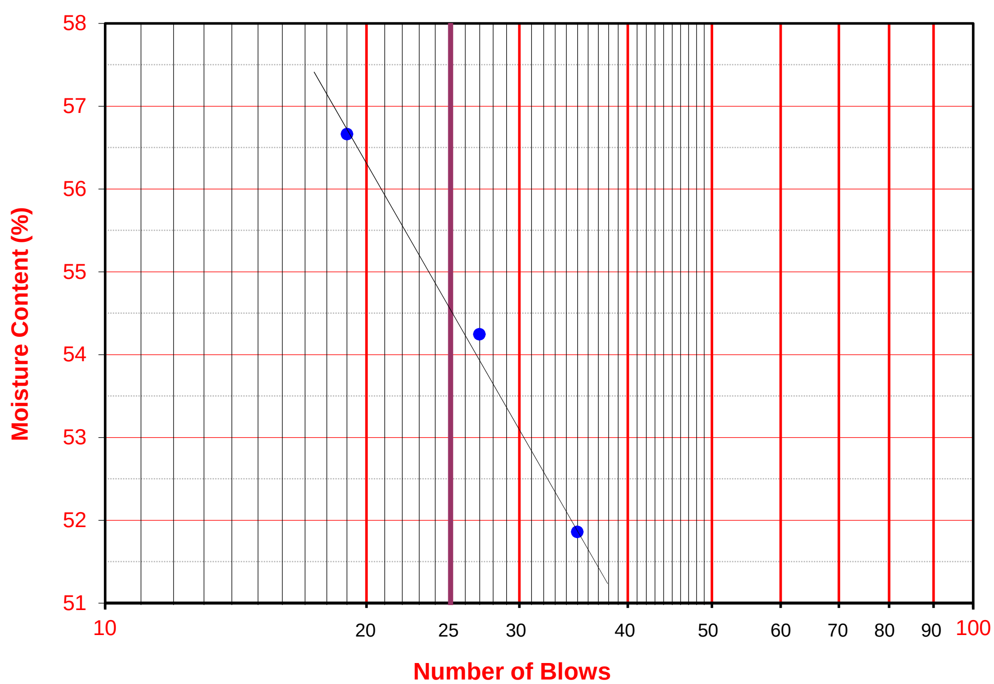
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

☐	Wet Preparation Multipoint - Wet
X	Dry Preparation Multipoint - Dry
X	Procedure A Multipoint Test
☐	Procedure B One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	Santa Ana	Tested By:	R. Manning	Date:	05/16/19
Project No. :	19063-01	Input By:	G. Bathala	Date:	05/24/19
Boring No.:	HS-3	Checked By:	J. Ward		
Sample No.:	R-4	Depth (ft.)	20.0		
Soil Identification:	Olive gray and brown clay (CL)				

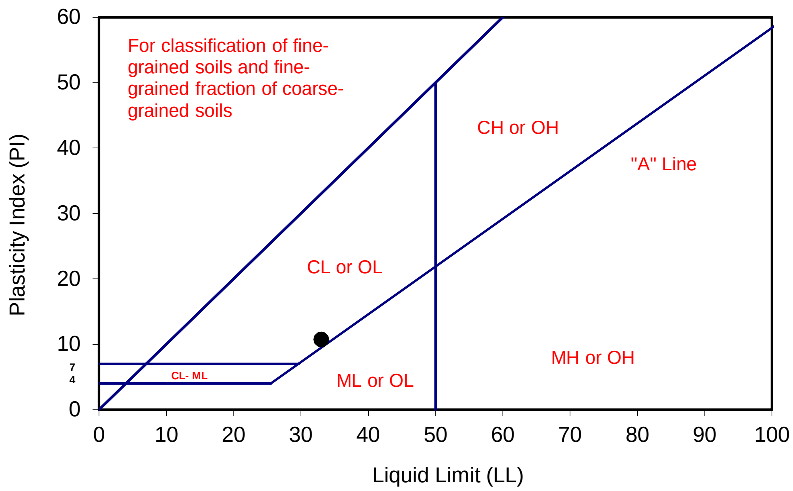
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	25	17	
Wet Wt. of Soil + Cont. (g)	19.64	19.57	23.88	24.13	23.91	
Dry Wt. of Soil + Cont. (g)	18.12	18.05	21.40	21.47	21.22	
Wt. of Container (g)	11.30	11.21	13.63	13.42	13.36	
Moisture Content (%) [Wn]	22.29	22.22	31.92	33.04	34.22	

Liquid Limit	**33**
Plastic Limit	**22**
Plasticity Index	**11**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 9.49

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



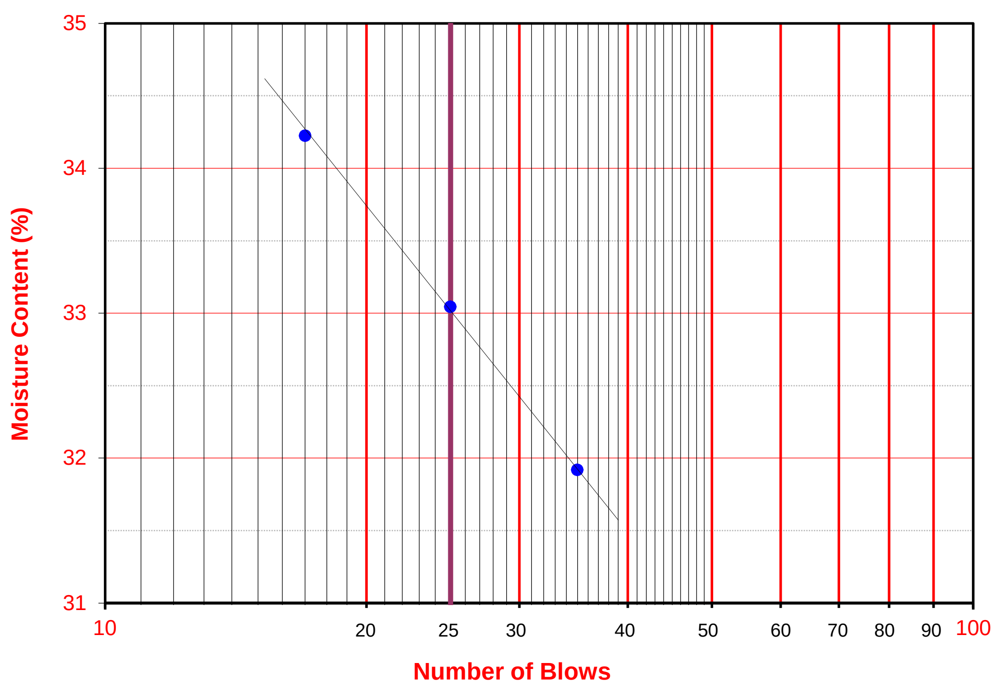
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

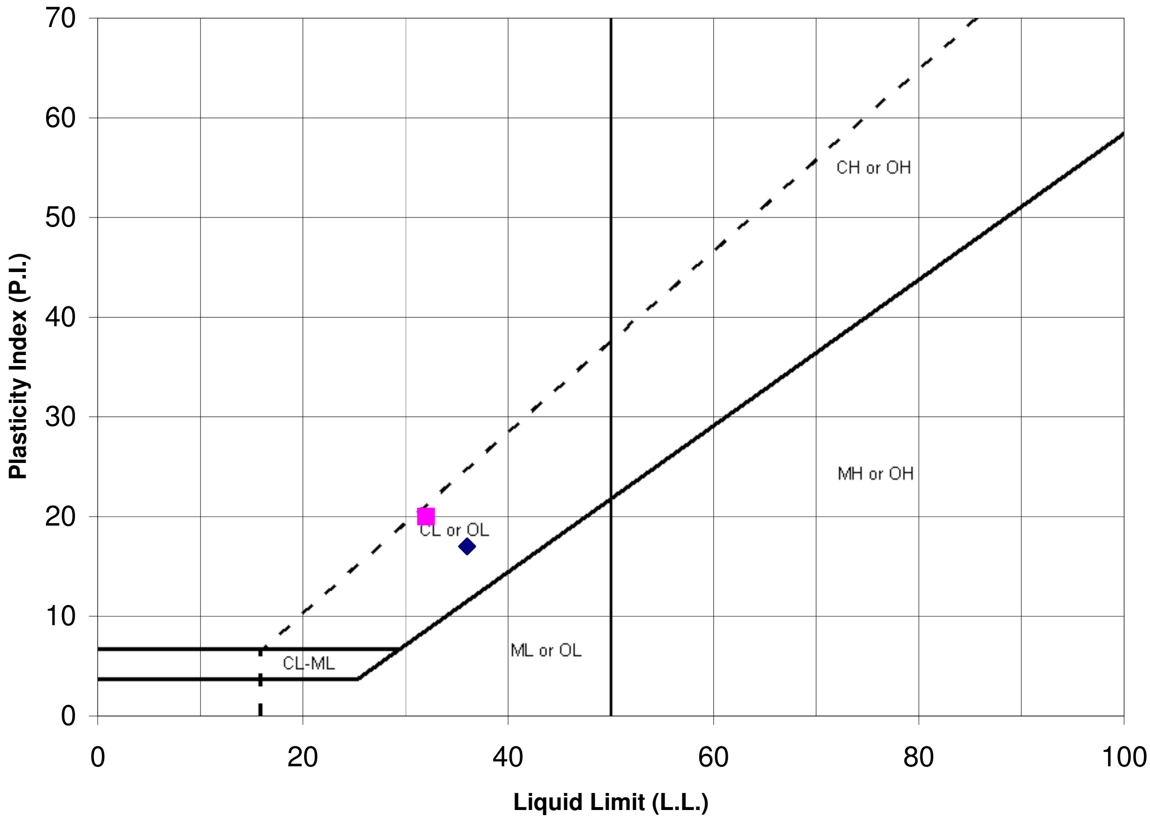
X Dry Preparation
Multipoint - Dry

X Procedure A
Multipoint Test

☐ Procedure B
One-point Test



PLASTICITY CHART - CLASSIFICATION OF FINE-GRAINED SOILS



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	**HS-2**	**SPT-2**	**25**	**-**	**36**	**19**	**17**	**CL**
■	**HS-2**	**SPT-3**	**35**	**-**	**32**	**12**	**20**	**CL**

LGC Geotechnical, Inc.

ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 19063-01
Date: May-19

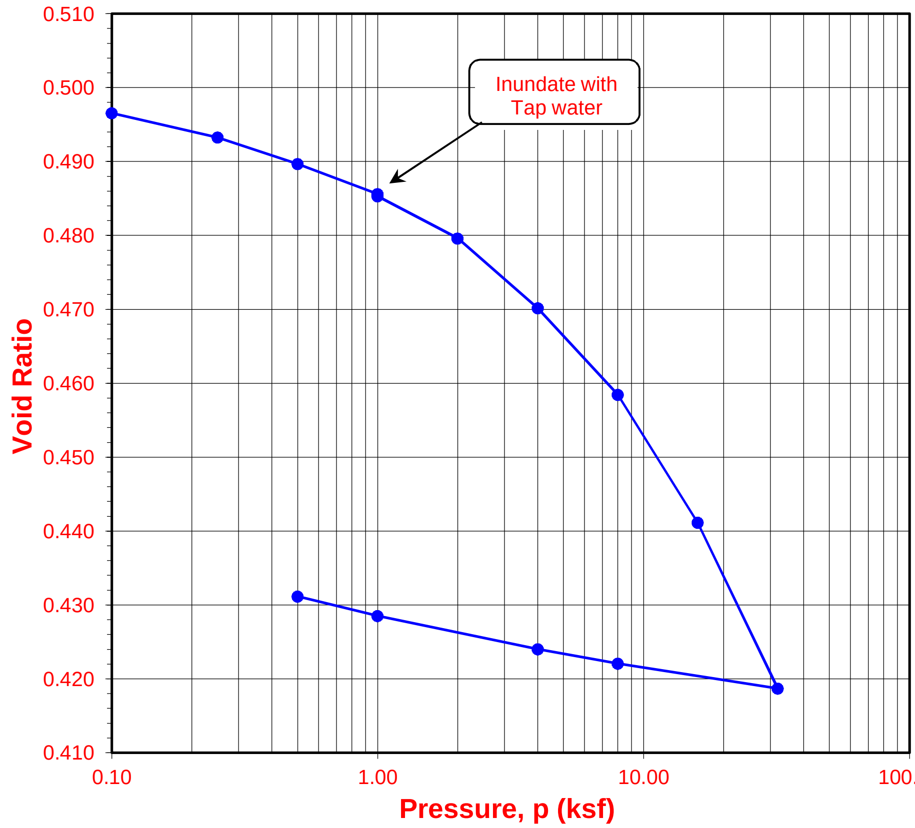
The Bowery - Santa Ana

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Santa Ana	Tested By:	G. Bathala	Date:	05/16/19
Project No.:	19063-01	Checked By:	J. Ward	Date:	05/30/19
Boring No.:	HS-2	Depth (ft.):	7.5		
Sample No.:	R-2	Sample Type:	Ring		
Soil Identification:	Brown clayey sand (SC)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	208.61
Weight of Ring (g)	42.93
Height after consol. (in.)	0.9539
Before Test	
Wt.Wet Sample+Cont. (g)	485.28
Wt.of Dry Sample+Cont. (g)	436.98
Weight of Container (g)	166.26
Initial Moisture Content (%)	17.8
Initial Dry Density (pcf)	116.9
Initial Saturation (%)	100
Initial Vertical Reading (in.)	0.3189
After Test	
Wt.of Wet Sample+Cont. (g)	274.60
Wt. of Dry Sample+Cont. (g)	258.15
Weight of Container (g)	69.53
Final Moisture Content (%)	11.29
Final Dry Density (pcf)	127.0
Final Saturation (%)	83
Final Vertical Reading (in.)	0.2652
Specific Gravity (assumed)	2.81
Water Density (pcf)	62.43

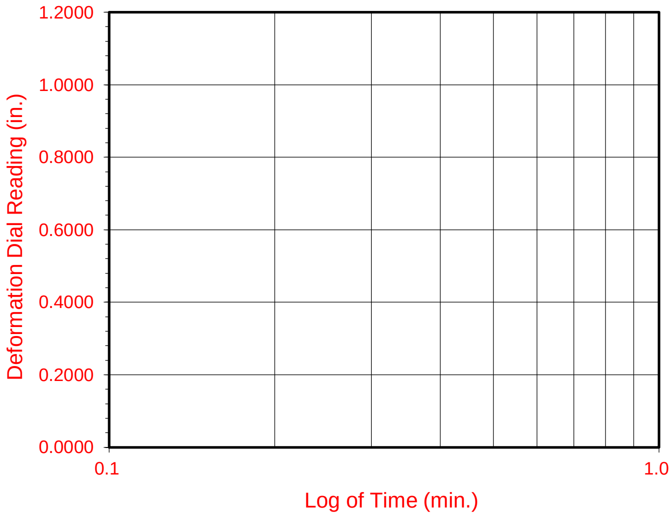


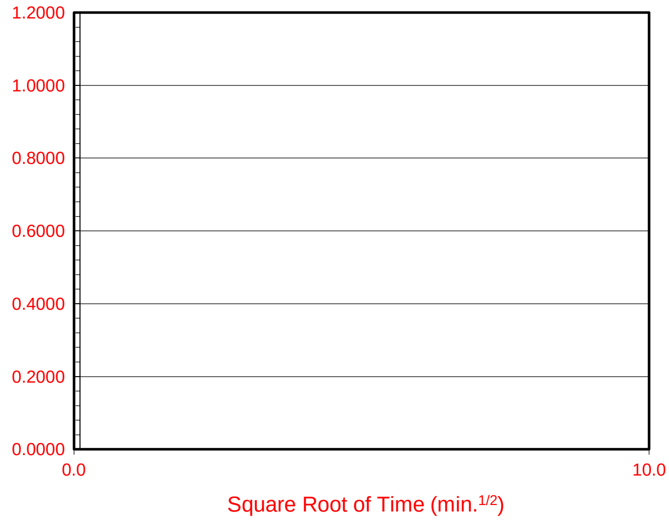
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3164	0.9975	0.00	0.25	0.497	0.25
0.25	0.3138	0.9949	0.04	0.51	0.493	0.47
0.50	0.3109	0.9920	0.09	0.80	0.490	0.71
1.00	0.3071	0.9882	0.20	1.18	0.486	0.98
1.00	0.3069	0.9880	0.20	1.20	0.485	1.00
2.00	0.3013	0.9824	0.38	1.76	0.480	1.38
4.00	0.2929	0.9740	0.59	2.60	0.470	2.01
8.00	0.2825	0.9636	0.85	3.64	0.458	2.79
16.00	0.2684	0.9495	1.11	5.06	0.441	3.95
32.00	0.2505	0.9316	1.40	6.84	0.419	5.44
8.00	0.2555	0.9366	1.13	6.35	0.422	5.22
4.00	0.2582	0.9393	0.99	6.08	0.424	5.09
1.00	0.2630	0.9441	0.81	5.59	0.429	4.78
0.50	0.2652	0.9463	0.76	5.37	0.431	4.61

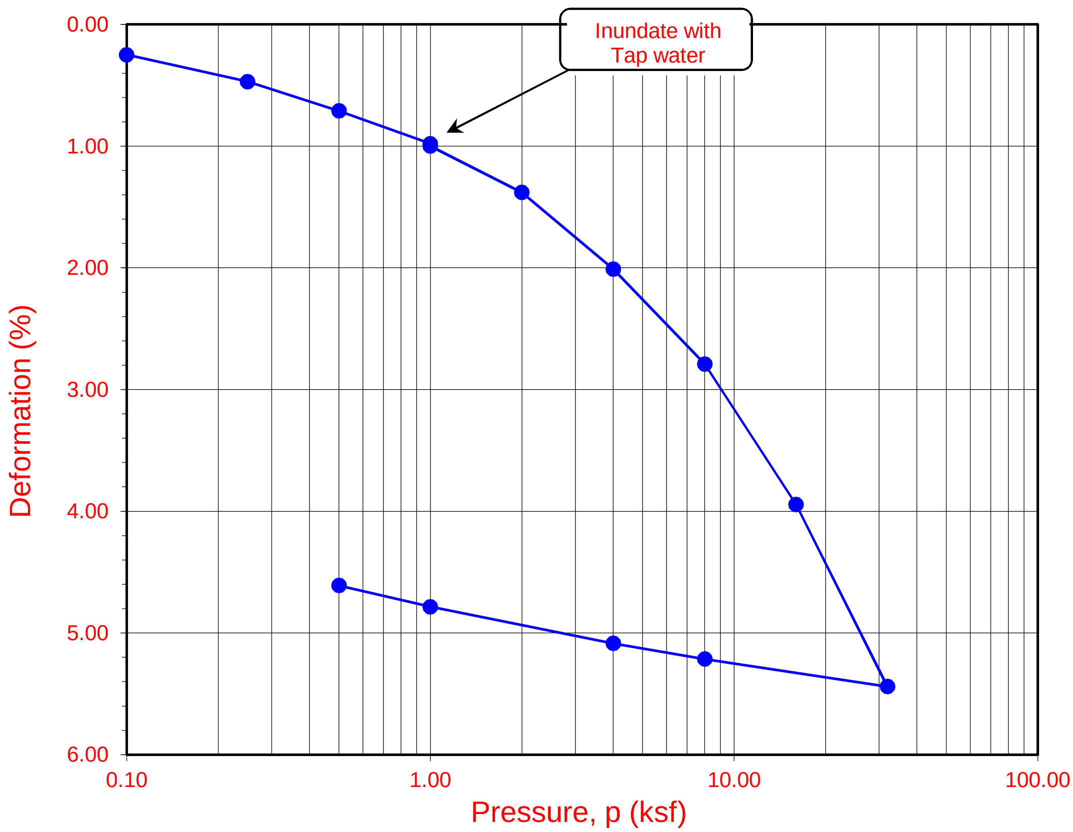
Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-2	**R-2**	**7.5**	**17.8**	**11.3**	**116.9**	**127.0**	**0.500**	**0.431**	**100**	**83**

Soil Identification: Brown clayey sand (SC)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 19063-01

Santa Ana

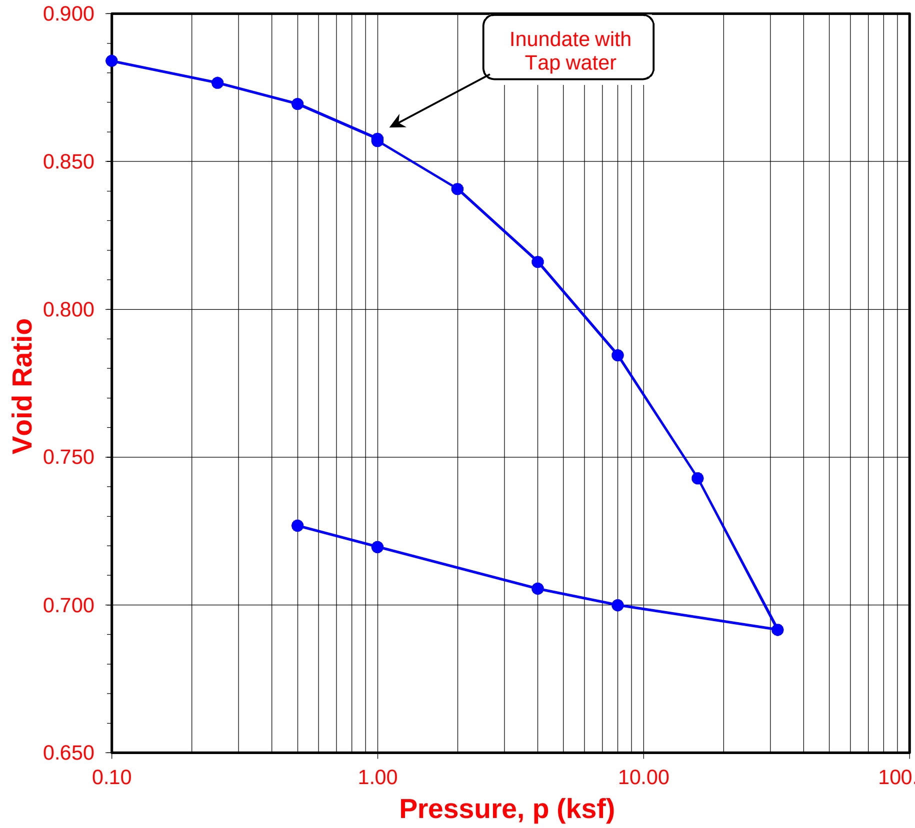
05-19

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Santa Ana	Tested By: G. Bathala	Date: 05/16/19
Project No.:	19063-01	Checked By: J. Ward	Date: 05/30/19
Boring No.:	HS-2	Depth (ft.):	10.0
Sample No.:	R-3	Sample Type:	Ring
Soil Identification:	Dark yellowish brown fat clay (CH)		

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	196.83
Weight of Ring (g)	45.82
Height after consol. (in.)	0.9128
Before Test	
Wt.Wet Sample+Cont. (g)	335.46
Wt.of Dry Sample+Cont. (g)	265.78
Weight of Container (g)	39.40
Initial Moisture Content (%)	30.8
Initial Dry Density (pcf)	96.0
Initial Saturation (%)	100
Initial Vertical Reading (in.)	0.3011
After Test	
Wt.of Wet Sample+Cont. (g)	254.81
Wt. of Dry Sample+Cont. (g)	229.91
Weight of Container (g)	65.66
Final Moisture Content (%)	21.03
Final Dry Density (pcf)	107.9
Final Saturation (%)	89
Final Vertical Reading (in.)	0.2046
Specific Gravity (assumed)	2.91
Water Density (pcf)	62.43

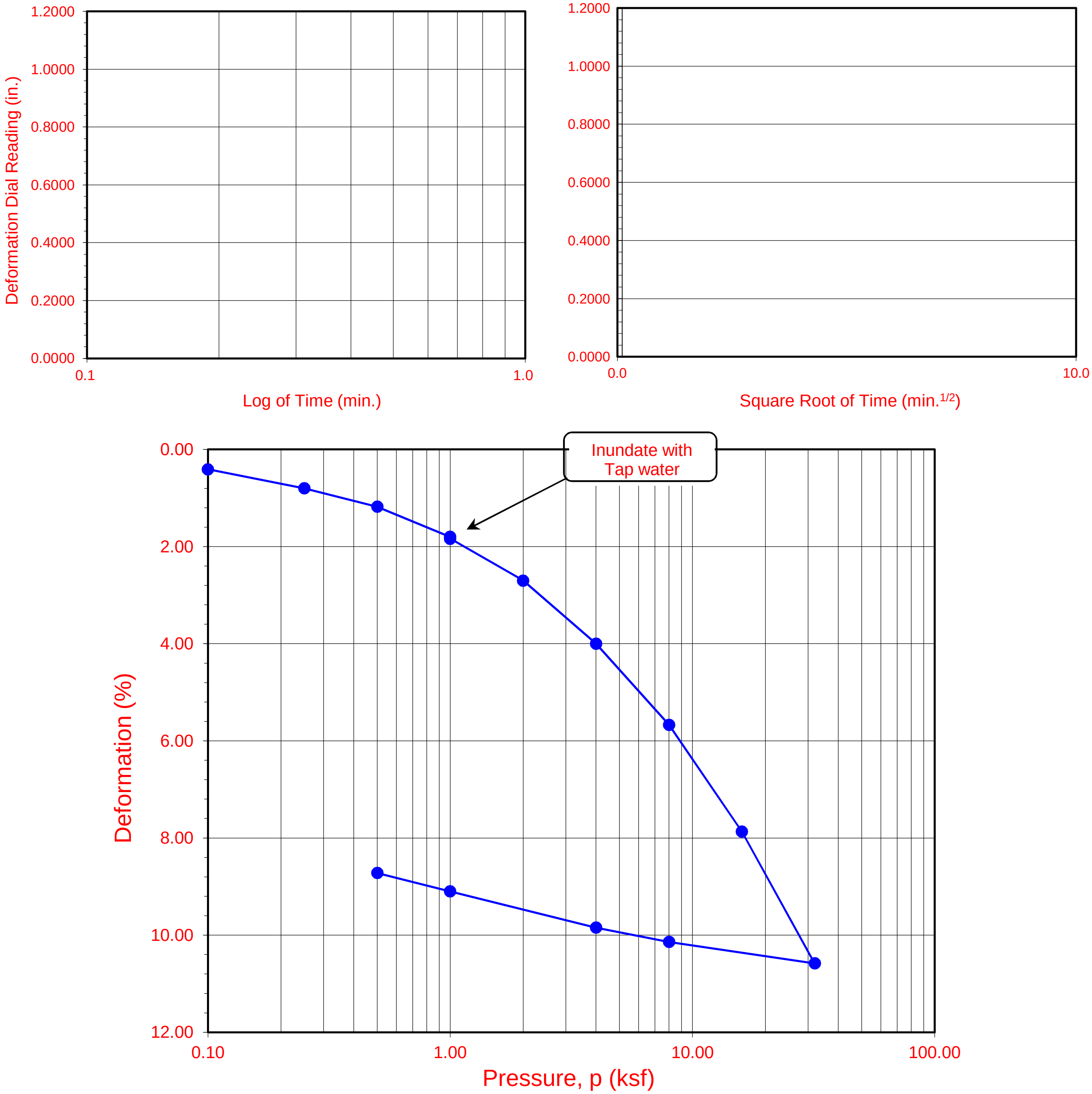


Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2970	0.9959	0.00	0.41	0.884	0.41
0.25	0.2920	0.9909	0.11	0.91	0.877	0.80
0.50	0.2868	0.9857	0.25	1.43	0.869	1.18
1.00	0.2790	0.9779	0.41	2.21	0.858	1.80
1.00	0.2786	0.9775	0.41	2.25	0.857	1.84
2.00	0.2679	0.9668	0.62	3.32	0.841	2.70
4.00	0.2529	0.9518	0.82	4.82	0.816	4.00
8.00	0.2341	0.9330	1.03	6.70	0.785	5.67
16.00	0.2095	0.9084	1.29	9.16	0.743	7.87
32.00	0.1795	0.8784	1.58	12.16	0.692	10.58
8.00	0.1860	0.8849	1.37	11.51	0.700	10.14
4.00	0.1902	0.8891	1.25	11.10	0.706	9.85
1.00	0.1998	0.8987	1.03	10.13	0.720	9.10
0.50	0.2046	0.9035	0.93	9.65	0.727	8.72

Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-2	**R-3**	**10.0**	**30.8**	**21.0**	**96.0**	**107.9**	**0.892**	**0.727**	**100**	**89**

Soil Identification: Dark yellowish brown fat clay (CH)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 19063-01

Santa Ana

05-19

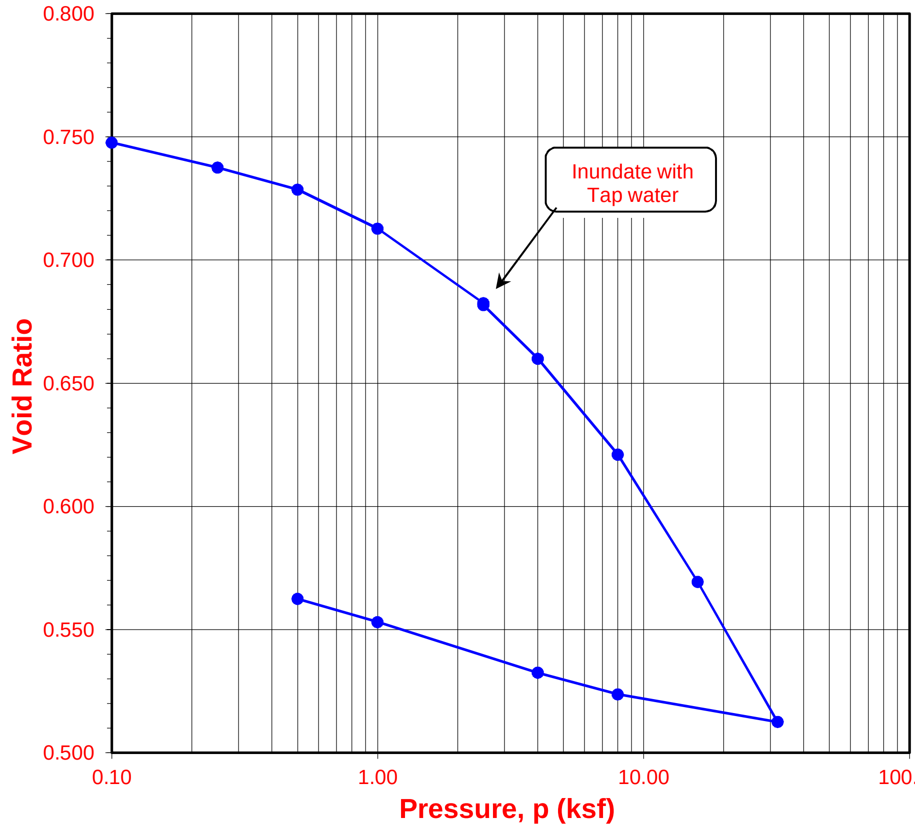
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: Santa Ana
Project No.: 19063-01
Boring No.: HS-3
Sample No.: R-4
Soil Identification: Olive gray and brown clay (CL)

Tested By: G. Bathala Date: 05/22/19
Checked By: J. Ward Date: 06/05/19
Depth (ft.): 20.0
Sample Type: Ring

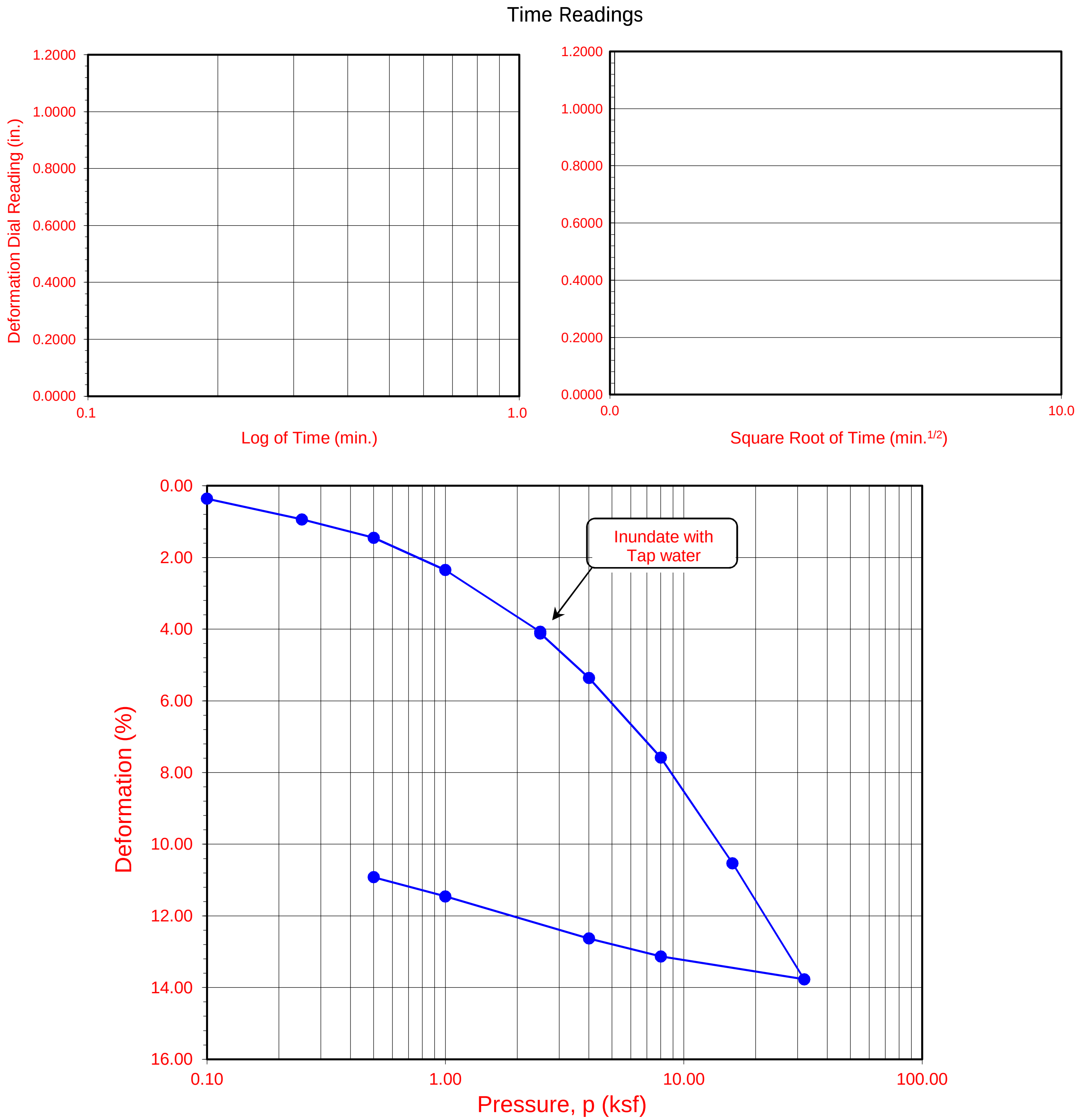
Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	198.04
Weight of Ring (g)	46.12
Height after consol. (in.)	0.8908
Before Test	
Wt.Wet Sample+Cont. (g)	336.15
Wt.of Dry Sample+Cont. (g)	275.03
Weight of Container (g)	38.86
Initial Moisture Content (%)	25.9
Initial Dry Density (pcf)	100.4
Initial Saturation (%)	97
Initial Vertical Reading (in.)	0.3047
After Test	
Wt.of Wet Sample+Cont. (g)	228.04
Wt. of Dry Sample+Cont. (g)	203.37
Weight of Container (g)	38.08
Final Moisture Content (%)	20.70
Final Dry Density (pcf)	111.3
Final Saturation (%)	100
Final Vertical Reading (in.)	0.1905
Specific Gravity (assumed)	2.82
Water Density (pcf)	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3011	0.9964	0.00	0.36	0.748	0.36
0.25	0.2950	0.9903	0.03	0.97	0.738	0.94
0.50	0.2896	0.9849	0.06	1.51	0.729	1.45
1.00	0.2800	0.9753	0.12	2.47	0.713	2.35
2.50	0.2615	0.9568	0.25	4.33	0.683	4.08
2.50	0.2610	0.9563	0.25	4.37	0.682	4.12
4.00	0.2476	0.9429	0.35	5.71	0.660	5.36
8.00	0.2236	0.9189	0.53	8.11	0.621	7.58
16.00	0.1920	0.8873	0.74	11.27	0.569	10.53
32.00	0.1572	0.8525	0.98	14.75	0.513	13.77
8.00	0.1660	0.8613	0.74	13.87	0.524	13.13
4.00	0.1719	0.8672	0.65	13.28	0.532	12.63
1.00	0.1849	0.8802	0.53	11.99	0.553	11.46
0.50	0.1905	0.8858	0.50	11.42	0.562	10.92

Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-3	**R-4**	**20.0**	**25.9**	**20.7**	**100.4**	**111.3**	**0.754**	**0.562**	**97**	**100**

Soil Identification: Olive gray and brown clay (CL)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 19063-01

Santa Ana

06-19

Appendix D
Infiltration Test Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc
131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: The Bowery - Santa Ana
Project Number: 19063-01
Date: 5/8/2019
Boring Number: I-1

Test hole dimensions (if circular)

Boring Depth (feet)*: 5
Boring Diameter (inches): 8
Pipe Diameter (inches): 2.75

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet):
Pit Length (feet):
Pit Breadth (feet):

Minimum test Head (D_o):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 3.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

*Pre-Test (Sandy Soil Criteria)**

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	7:56	8:21	25.0	2.98	3.11	0.13	no
2	8:21	8:46	25.0	3.11	3.25	0.14	no

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	8:47	9:17	30.0	2.91	3.05	0.14	0.26
2	9:17	9:46	29.0	3.05	3.2	0.15	0.30
3	9:47	10:17	30.0	2.88	3.03	0.15	0.27
4	10:17	10:47	30.0	3.03	3.19	0.16	0.31
5	10:48	11:18	30.0	2.95	3.1	0.15	0.28
6	11:18	11:48	30.0	2.96	3.12	0.16	0.30
7	11:48	12:18	30.0	3.12	3.28	0.16	0.33
8	12:19	12:49	30.0	2.86	3.02	0.16	0.29
9	12:49	13:19	30.0	3.02	3.19	0.17	0.33
10	13:19	13:49	30.0	2.95	3.11	0.16	0.30
11	13:50	14:20	30.0	2.97	3.13	0.16	0.30
12	14:21	14:51	30.0	2.94	3.09	0.15	0.28

Calculated Infiltration Rate (No factors of safety) **0.28**
Factor of Safety **2.0**
Calculated Infiltration Rate (With Factor of Safety) **0.14**

Sketch:

Notes:

Based on Guidelines from: Orange County 12/20/2013
Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Appendix E
Liquefaction Analysis

LIQUEFACTION EVALUATION

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997
and *Evaluation of Settlments in Sand due to Earthquake Shaking*, Tokimatsu and Seed, 1987

Seismic Event
Moment Magnitude 6.9
Peak Ground Acceleration 0.57 g

Profile Constants
Total Unit Weight (lb/ft^3) 120
Unit Weight of Water (lbs/ft 62.4

Depth to GWT
During Investigation (ft) 25
During Design Event (ft) 10

Project Name The Bowery - Santa Ana
Project Number 19063-01
Boring HS-2

Determination of Cyclic Resitance Ratio

Sampling Data		Blow Count		Thickness	During Investigation			Sampling Correction Factors											
Depth (ft)	Depth (m)	SPT	Rings	(ft)	Total Stress (psf)	Pore Pressure (psf)	Effective Stress (psf)	Sampler Diameter	SPT N_m	Overburden C_N	Energy C_E	Borehole C_B	Rod Length C_R	Sampler Type C_S	$(N_1)_{60}$	Fines Content	$(N_1)_{60cs}$	K_σ	$CRR_{7.5}$
5	1.5		22	6.25	720	0	720	0.62	13.64	1.70	1.33	1.00	0.75	1.00	23.17	80	32.80	1.000	SPT >30 NF
7.5	2.3		9	2.5	1020	0	1020	0.62	5.58	1.43	1.33	1.00	0.75	1.00	7.96	47	14.56	1.000	0.157
10	3.0		6	3.75	1320	0	1320	0.62	3.72	1.26	1.33	1.00	0.75	1.00	4.67	93	10.60	1.000	0.115
15	4.6	5		5	1920	0	1920	1.00	5.00	1.04	1.33	1.00	0.85	1.10	6.48	93	12.78	1.000	0.138
20	6.1		10	5	2520	0	2520	0.62	6.20	0.91	1.33	1.00	0.95	1.00	7.13	88	13.56	0.964	0.141
25	7.6	5		5	3120	0	3120	1.00	5.00	0.82	1.33	1.00	0.95	1.10	5.68	88	11.82	0.924	0.118
30	9.1		19	5	3720	312	3408	0.62	11.78	0.78	1.33	1.00	0.95	1.00	11.65	88	18.98	0.907	0.186
35	10.7	16		5	4320	624	3696	1.00	16.00	0.75	1.33	1.00	1.00	1.10	17.59	60	26.11	0.890	0.269
40	12.2		21	5	4920	936	3984	0.62	13.02	0.72	1.33	1.00	1.00	1.00	12.54	35	19.98	0.875	0.189
45	13.7	22		5	5520	1248	4272	1.00	22.00	0.70	1.33	1.00	1.00	1.10	22.50	35	31.91	0.860	SPT >30 NF
50	15.2		22	2.5	6120	1560	4560	0.62	13.64	0.68	1.33	1.00	1.00	1.00	12.28	60	19.73	0.846	0.180

Determination of Cyclic Stress Ratio

Sampling Data		Blow Count			During Design Event							
Depth (ft)	Depth (m)	SPT	Rings	Thickness	Total Stress (psf)	Pore Pressure (psf)	Effective Stress (psf)	r_d	CSR	MSF	FS	Depth
5	1.52		22	6.25	600	0	600	0.99024	0.366883	1.238	Above GWT	5
7.5	2.29		9	2.5	900	0	900	0.98456	0.36478	1.238	Above GWT	7.5
10	3.05		6	3.75	1200	0	1200	0.97914	0.362772	1.238	Bray Clay	10
15	4.57	5		5	1800	312	1488	0.96856	0.434094	1.238	Bray Clay	15
20	6.10		10	5	2400	624	1776	0.9569	0.479095	1.238	Bray Clay	20
25	7.62	5		5	3000	936	2064	0.94183	0.507194	1.238	Bray Clay	25
30	9.14		19	5	3600	1248	2352	0.92058	0.522052	1.238	Bray Clay	30
35	10.67	16		5	4200	1560	2640	0.89062	0.524959	1.238	Bray Clay	35
40	12.19		21	5	4800	1872	2928	0.85103	0.516899	1.238	0.45	40
45	13.72	22		5	5400	2184	3216	0.80363	0.499945	1.238	Corr. SPT>30	45
50	15.24		22	2.5	6000	2496	3504	0.75271	0.477535	1.238	0.47	50

Liquefaction-Induced Settlement Analysis

Depth	Vol. Strain (%) SP117 Fig7.11	Settlement (in.)
5.0		
7.5		
10.0		
15.0		
20.0		
25.0		
30.0		
35.0		
40.0	1.60	1.0
45.0		
50.0	1.60	0.5
	Total =	**1.4**

6/14/2019