APPENDIX F

GEOTECHNICAL INVESTIGATION

July 28, 2022 Project No. 22086-01

Mr. Erik Johnson
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Subject: ***Preliminary Geotechnical Evaluation for Proposed Industrial Development, Evans Road, Menifee, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed industrial development to be located between Evans Road and Barnett Road in the City of Menifee, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed development.

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,

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1.0 INTRODUCTION

LGC Geotechnical has performed a geotechnical evaluation for the proposed industrial development to be located between Evans Road and Barnett Road, north of McLaughlin Road, in the City of Menifee, Riverside County, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical recommendations relative to the proposed development.

1.1 Project Description and Background

The approximately 14-acre, irregularly shaped site is bound by Evans Road on the west, by Barnett Road on the east, by a Southern California Edison easement along McLaughlin Road to the south, by vacant land to the north and the Romoland Flood Control channel to the northeast. The site is relatively flat and appears to have historically been used for dry farming of crops. Based on a review of aerial photos, a residence with a yard surrounded by trees was once located in the southeast corner of the site. It appears that the residence existed from the 1980's until it was demolished in the early 2000's, and the trees appear to have been removed by 2009. The Romoland Flood Control channel appears to have been constructed in 2014-2015.

We understand that the proposed development is to include grading and construction for two warehouse structures in the southern portion of the site and one in the central portion of the site. The proposed warehouses range from 74,780 square feet to 102,190 square feet (HPA, 2022). Each are presumed to be one-story, and each will include integral office space and associated parking, drive aisles and improvements. Proposed grades are not anticipated to change of significantly from existing. Preliminary building (dead plus live) loads were not provided at the time of this report. However, we have estimated the maximum column and wall structural (dead plus live) loads to be 150 kips and 4 kips per lineal foot, respectively.

The recommendations given in this report are based on the layout and estimated structural loads and grading information as indicated above. LGC Geotechnical should be provided with any updated project information, plans and/or any structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

1.2 Subsurface Exploration

In June of 2022 LGC Geotechnical performed a subsurface geotechnical evaluation of the subject site consisting of the excavation of 14 hollow-stem auger borings (8 borings for field percolation testing) in order to evaluate onsite geotechnical conditions.

The borings (HS-1 through HS-6 and I-1 through I-8) were excavated using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers with depths ranging from approximately 12 to 50 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound 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. After removal of the augers, the depth of the boring due to caving was measured and is noted on the boring logs. The borings were backfilled with cuttings. The approximate locations of our subsurface explorations are provided on our Geotechnical Map (Sheet 1). The boring logs are provided in Appendix B.

At the completion of excavation of the borings, I-1 through I-8, an infiltration well was constructed within each boring for testing as outlined in the “Field Percolation Testing” section below. At the completion of infiltration testing, the installed pipe was removed, and the resulting void backfilled with native soils. Please note that some settlement of the backfill may occur over time and the excavations should be topped off as needed.

1.3 Field Percolation Testing

Eight field percolation tests (I-1 through I-8) were performed in the approximate locations indicated on our Geotechnical Map (Sheet 1). Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Riverside (2011). A 3-inch diameter perforated PVC pipe was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils should be followed. If the water level does not meet that criterion, the procedure for fine-grained soils should be followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point. The pre-tests indicated the procedure for fine grained soils should be followed for the majority of the tests. The calculated infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., “Porchet Method”). The percolation tests were performed using relatively clean water free of particulates, silt, etc. The calculated infiltration rates are provided in Table 1 on the following page. Refer to the discussion provided in Section 4.9. Infiltration test data is provided in Appendix B.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Approximate Depth Below Existing Grade (feet)	Tested Infiltration Rate (inch/hr.) *	Reduced Infiltration Rate (inch/hr.) **
I-1	12	1.4	0.5
I-2	12	1.2	0.4
I-3	14	0.2	0.1
I-4	14	0.1	0.0
I-5	14	0.4	0.1
I-6	14	0.2	0.1
I-7	12	0.6	0.2
I-8	12	0.4	0.1

*Does not include a factor of safety
**Based on minimum Factor of Safety of 3.

1.4 Laboratory Testing

Representative bulk and driven samples were obtained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and dry density, Atterberg Limits, gradation/fines content, Expansion Index, consolidation, laboratory compaction, and corrosion characteristics (sulfate, chloride, pH and minimum resistivity). A summary of the laboratory test results is presented below with complete results provided in Appendix C.

- Dry density of the samples collected ranged from approximately 93 pounds per cubic foot (pcf) to 128 pcf, with an average of 113 pounds per cubic foot (pcf). Field moisture contents ranged from approximately 3 percent to 27 percent, with an average of 13 percent.
- Four Atterberg Limit (liquid limit and plastic limit) tests were performed. The Plasticity Index (PI) values ranged from 11 to 16.
- Nine fines content/gradation tests indicated a fines content (passing No. 200 sieve) ranging from approximately 29 to 58 percent. Based on the Unified Soils Classification System (USCS), six of the tested samples would be classified as "coarse-grained" and the remaining three samples would be classified as "fine-grained."
- Consolidation tests were performed. Swell at water inundation was about 1.5 percent or less. The deformation versus vertical stress plots are provided in Appendix C.
- Three Expansion Index (EI) tests indicated EI values ranging from 0 and 34, corresponding to "Very Low" to "Low" expansion potential.
- A laboratory compaction curve resulted in maximum dry density value of 133.5.0 pcf with optimum moisture content of 8.5.
- Corrosion testing indicated soluble sulfate content less than approximately 0.02 percent, chloride contents of 62 parts per million (ppm) and 126 ppm, pH values of 7.3 ad 7.7, and minimum resistivity values of 2,100 ohm-centimeters and 3,960 ohm-centimeters.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional and Local Geology

The site is located in the Northern Peninsular Range on the southern sector of the structural unit known as the Perris Block. The Perris Block is bounded on the northeast by the San Jacinto Fault Zone, on the southwest by the Elsinore Fault Zone, and the north by the Cucamonga Fault Zone. The southern boundary of the Perris Block is not as distinct but is believed to coincide with a complex group of faults trending southeast from the Murrieta, California area (Kennedy, 1977). The Peninsular Range is characterized by large Mesozoic age intrusive rock masses flanked by volcanic, metasedimentary and sedimentary rocks. Various thicknesses of colluvial and alluvial sediments derived from the erosion of the elevated portions of the region fill the low-lying areas.

Locally the site is within the Perris Valley. The Perris Valley is flanked by steep rocky hillsides composed of granitic and metasedimentary bedrock. The low-lying areas of the valley are gently sloping to generally flat and are typically underlain by old alluvial fan deposits.

2.2 Site-Specific Geology

Based on the Geologic Map of the Romoland Quadrangle (Morton, 2003), and our geologic field mapping, the subject site is underlain by undocumented agricultural fill, young alluvial fan deposits, and old alluvial fan deposits. The geologic units are summarized below, and their approximate lateral limits are depicted on the Geotechnical Map (Sheet 1). It should be noted that the relatively thin agricultural fill is not shown on the site plan and therefore there are areas delineated on the site plan that indicate alluvial deposits exposed at the surface that may be overlain by a thin cover of agricultural fill.

It should be noted that the excavated borings and trenches are only representative of the locations where they were excavated at the time in which they were performed, and varying subsurface conditions may exist outside of those location. In addition, subsurface conditions can change over time. The soil descriptions provided should not be construed to indicate that the subsurface profile is uniform, and that soil is homogeneous within the project area.

2.2.1 Artificial Fill - undocumented (not mapped)

Relatively thin undocumented agricultural fill from the tilling of the land for crop cultivation is not mapped on the site plan, as discussed above. However, it is interpreted as existing in the upper 1 to 2 feet. It is anticipated that the agricultural fill consists of the same material as the other near-surface soils encountered in the borings, and it is anticipated that the agricultural fill is dry and loose.

A residence was once located near the southeast corner of site, and undocumented fill may be associated with the former residence. It is not known if the residence was once serviced by an on-site sewage disposal system (septic system), however, the possibility of encountering a septic system is high given the rural setting. Typical septic systems consist of a septic tank connected to relatively shallow leach lines, or less commonly connected to a deep seepage pit (cesspit). Any existing fill associated with the former residence and associated septic system should be considered undocumented.

2.2.2 Quaternary Old Alluvial Fan Deposits (Map Symbol - Qof)

Quaternary old alluvial fan deposits were encountered to the maximum explored depth of approximately 51.5 feet below the ground surface. The upper approximately 2 to 4 feet were found to be porous, with root hairs. In general, the old alluvial fan deposits were found to consist mostly of silty sand to sandy silt that was dry to moist and medium dense to very dense or stiff to hard in-place. Scattered discontinuous beds of sandy clay and clayey sand, along with lenses of poorly graded sand were encountered.

2.3 Geologic Structure

Geologic structure was not identified in the subject site geotechnical evaluation. The alluvial materials encountered are generally massive, and bedding is likely nearly horizontal.

2.4 Landslides

The site is nearly flat. Our research and field observations do not indicate the presence of landslides on the site or in the immediate vicinity. Review of regional geologic maps of the area do not indicate the presence of known or suspected landslides at the site or in the vicinity of the site.

2.5 Groundwater

Groundwater was not encountered during our subsurface field evaluation to the maximum explored depth of approximately 50 feet below existing ground surface. Groundwater is anticipated to be greater than 50 feet below existing grade. Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local seepage or during rainy seasons. Groundwater conditions below the site may be variable, depending on numerous factors including seasonal rainfall, local irrigation and groundwater pumping, among others.

2.6 Faulting

California is located on the boundary between the Pacific and North American Lithospheric Plates. The average motion along this boundary is on the order of 50-mm/yr. in a right-lateral sense. The majority of the motion is expressed at the surface along the northwest trending San Andreas Fault Zone with lesser amounts of motion accommodated by sub-parallel faults located predominantly west of the San Andreas including the Elsinore, Newport-Inglewood, Rose Canyon, and Coronado Bank Faults. Within Southern California, a large bend in the San Andreas Fault north of the San Gabriel Mountains has resulted in a transfer of a portion of the right-lateral motion between the plates into left-lateral displacement and vertical uplift. Compression south and west of the bend has resulted in folding, left-lateral, reverse thrust faulting, and regional uplift creating the east-west trending Transverse Ranges and several east-west trending faults. Further south within the Los Angeles Basin, "blind thrust" faults are believed to have developed below the surface also as a result of this compression, which have resulted in earthquakes such as the 1994 Northridge event along faults with little to no surface expression.

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the construction of urban developments across the trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements. Special Publication 42 was most recently revised in 2018 (CGS, 2018). According to the State Geologist, an "active" fault is defined as one which has had surface displacement within Holocene time (roughly the last 11,700 years). Regulatory Earthquake Fault Zones have been delineated to encompass traces of known, Holocene-active faults to address hazards associated with surface fault rupture within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering-geologists can identify the locations of active faults and recommend setbacks from locations of possible surface fault rupture.

The subject site is not located within a State of California Fault Rupture Hazard Zone or a Riverside County Fault Zone (CGS, 2018 and County, 2022). The nearest Holocene-active faults to the site are faults in the Elsinore Fault Zone, located approximately 10 miles southwest of the site, and faults in the San Jacinto Fault Zone, located approximately 11 miles northeast of the site. The faults in both of these fault zones trend northwest-southeast, oblique to the site, and do not trend toward the site. Therefore, the possibility of damage due to ground rupture, as a result of faulting, is considered very low since active faults are not known to cross 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, dynamic settlement, seiches and tsunamis. 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.

2.6.1 Lurching and Shallow Ground Rupture

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are not likely to be significant where the thickness of soft sediments do not vary appreciably under structures. Ground cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

2.6.2 ***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 saturated, loose 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, depending on their plasticity and moisture content. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry loose sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

The site located in a zone of "Low" potential for liquefaction as delineated by the County of Riverside (County, 2022). Due to the depth of groundwater greater than 50 feet, the generally dense nature of the underlying soils, and the presence of cohesive soils, the potential for liquefaction and liquefaction-induced settlement is considered very low. The proposed development will primarily consist of compacted fill over dense alluvial fan deposits. These soils are not considered susceptible to dynamic settlement.

2.6.3 ***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 down-slope 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.

Due to the very low potential for liquefaction, the potential for lateral spreading is also considered very low.

2.6.4 ***Tsunamis and Seiches***

Based on the elevation of the site, with respect to sea level, the possibility of damage to the site during a large tsunami event is nil. Due to the lack of large, enclosed bodies of water close to the site, the possibility of damage from a seiche is nil.

2.7 ***Seismic Design Parameters***

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 33.7394 degrees north and longitude -117.1959 degrees west were utilized in our analyses. Please note that these coordinates are considered representative of the site for preliminary planning purposes only, however their applicability must be verified with respect to a desired specific location within the site. 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 2 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

Section 1803.5.12 of the 2019 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 0.550g (SEAOC, 2022). The design PGA may be taken as 0.367g (2/3 of PGA_M).

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.91 at a distance of 16.51 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.81 at a distance of 19.67 km from the site would contribute the most to this ground motion (USGS, 2014).

TABLE 2

Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**	**Notes/Exceptions**
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.410g	From SEAOC, 2022
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.522g	From SEAOC, 2022
F_a (per Table 1613.2.3(1))	1.0	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.778	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.410g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	0.928g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.940g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.619g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.940	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.619	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

2.8 ***Subsidence***

Subsidence is the settlement of the ground surface over large areas (typically on the order of square miles) typically due to the lowering of the groundwater table. Mitigation against such a large-scale groundwater drawdown cannot be mitigated on a site-specific level, but instead "requires regional cooperation among numerous agencies" and therefore is not a site-specific geotechnical consideration. The subject site is located within an area considered susceptible to subsidence as delineated by the County of Riverside (2022). The soils encountered in our field evaluation did not indicate the presence of soils susceptible to collapse or excessive settlement.

Based on the site geologic conditions, the potential for subsidence in the site development area is considered low.

2.9 ***Rippability***

In general, excavation for foundations and underground improvements should be achievable with the appropriate equipment.

2.10 ***Oversized Material***

Oversized material (material larger than 8 inches in maximum dimension) is not likely to be encountered or generated during site grading. However, if oversized material is encountered, recommendations are provided for appropriate handling of oversized materials in Appendix D. If feasible, crushing oversized materials or exporting to an offsite location may be considered.

3.0 FINDINGS AND CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed site development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the site design, grading, and construction.

The following is a summary of the primary geotechnical factors, which may affect future development of the site.

- In general, our subsurface evaluation primarily indicates that the site contains dense, clayey and silty sands and very stiff to hard sandy clays to the maximum explored depth of approximately 50 feet below existing grade. The near-surface undocumented/agricultural fill and loose and compressible natural alluvial soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was not encountered to the maximum explored depth of approximately 50 feet below existing ground surface. Historic high groundwater is anticipated to be greater than 50 feet below existing ground surface.
- The subject site is not located within an Alquist-Priolo Earthquake Fault Zone or a County of Riverside Fault Zone. No active faults are mapped on the site. No faults were identified on the site during our site evaluation. The proposed development will likely be subjected to strong seismic ground shaking during its design life from one of the regional faults.
- Due to a lack of groundwater in the upper 50 feet, the site is not considered susceptible to liquefaction and liquefaction-induced settlement. The soils at the site encountered in our borings were generally found to be dense and contained fines (e.g., silts and clays) and these soil types are generally not considered susceptible to seismically induced dry sand settlement.
- Site soils should be considered to have “Very Low” to “Low” expansion potential (EI not exceeding 50). This shall be confirmed at the completion of site earthwork. Mitigation measures are required for foundations and site improvements like concrete flatwork to minimize the impacts of expansive soils.
- Excavation for foundations and underground improvements should be achievable with the appropriate equipment.
- Field testing resulted in infiltration rates (based on required Factor of Safety of 3) ranging from essentially zero to 0.5 inches per hour. Site consists primarily of fine-grained soils; these soils have very low permeability and therefore have very low infiltration rates.
- The site contains soils with high fines content (i.e., silts and clay) that are not suitable for backfill of any site retaining walls. Therefore, select grading and stockpiling of native suitable sandy soils and/or import of sandy soils meeting project recommendations will be required.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the County. It is the responsibility of the builder to ensure these recommendations are provided to the appropriate parties.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2019 California Building Code (CBC) requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level.” The “acceptable level” of risk is defined by the California Code of Regulations as “the level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual exposed conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of required earthwork removals, foundation construction and utility line construction and backfill. We recommend that earthwork onsite be performed in accordance with the following recommendations, the City of Menifee, 2019 CBC and the General Earthwork and Grading Specifications included in Appendix D. In case of conflict, the following recommendations shall supersede previous recommendations and those included as part of Appendix D.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill, the area should be cleared of existing vegetation (shrubs, trees, grass, roots, etc.), surface obstructions, existing debris and potentially compressible or otherwise unsuitable material, including agricultural/undocumented fill. Debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed removal bottoms, should be replaced with suitable compacted fill material. Any abandoned utility lines should be completely removed and replaced with properly compacted fill.

If septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, if deep cesspits are encountered, they can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further grading.

4.1.2 Removal Depths and Limits

Building Structures: In order to provide a relatively uniform bearing condition for the planned structural improvements, removals should extend a minimum depth of 5 feet below existing grade or 2 feet below the proposed footings, whichever is greater. In general, the envelope for removals should extend laterally a minimum horizontal distance of 5 feet beyond the edges of the proposed building footprint. Deeper removals will likely be required for retaining wall footings for loading docks located at the front of the building structures.

Deeper removals may be required in the western portion of Building 1. Lower blow counts were encountered in Boring HS-6 at a depth of approximately 7.5 feet. Subgrade conditions should be evaluated during grading.

Retaining/Free-Standing Wall Structures: Removals should extend a minimum of 3 feet below existing grade, or 1-foot below proposed footings, whichever is greater. Deeper removals will likely be required for retaining wall footings for loading docks located at the front of the building structures.

Pavement and Hardscape Areas: Removals should extend to a depth of at least 2 feet below the existing grade. Removals in any design cut areas of the pavement may be reduced by the depth of the design cut but should not be less than 1-foot below the finished subgrade (i.e., below planned aggregate base/asphalt concrete). In general, the envelope for removals should extend laterally a minimum lateral distance of 2 feet beyond the edges of the proposed improvements.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above-noted minimum in order to obtain an acceptable subgrade including localized areas of undocumented fill. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal areas should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

We expect temporary excavation slopes up to 10 feet in height to be grossly stable at 1:1 (horizontal to vertical) inclinations or flatter, however, excavations must be performed in accordance with all Occupational Safety and Health Administration (OSHA) requirements. Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation, or 5 feet whichever is greater. The contractor will be responsible for providing the “competent person” required by Cal/OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain. Excavation safety and protection of off-site existing improvements during earthwork operations is the responsibility of the contractor.

In general, any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. Where needed, temporary shoring parameters can be provided, upon request.

4.1.4 <u>Removal Bottoms and Subgrade Preparation</u>

In general, removal bottom areas and any areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and re-compacted per project recommendations.

Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.5 <u>Material for Fill</u>

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill (i.e., non-retaining wall backfill), provided they are screened of organic materials, construction debris and any oversized material (8 inches in greatest dimension). Moisture conditioning of site soils should be anticipated as outlined in the section below.

Retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris and any material greater than 3 inches in maximum dimension. The site contains soils that are not suitable for retaining wall backfill due to their fines content and expansion potential, therefore select grading and stockpiling and/or import will be required by the contractor to obtain suitable retaining wall backfill soil.

From a geotechnical viewpoint, any required import soils should consist of clean, relatively granular soils of Very Low expansion potential (expansion index 20 or less based on ASTM D4829) and no particles larger than 3 inches in greatest dimension. Source samples of planned importation should be provided to the geotechnical consultant for laboratory testing a minimum of 3 working days prior to any planned importation for required laboratory testing.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

4.1.6 Fill Placement and Compaction

Material to be placed as fill should be brought to near-optimum moisture content (generally at about 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Significant moisture conditioning of site soils should be anticipated in order to achieve the required degree of compaction. Drying and/or mixing the very moist soils will be required prior to reusing the materials in compacted fills. Soils may also be present that will require additional moisture conditioning in order to achieve the required compaction. 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 not exceeding 8 inches in loose thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

Fill placed on any slopes greater than 5:1 (horizontal to vertical) should be properly keyed and benched into firm and competent soils as it is placed in lifts.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to a minimum of 95 percent relative compaction at or slightly above-optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction per ASTM D1557 at or slightly above-optimum moisture content.

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by the geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric to prevent the migration of fines into the rock backfill.

4.1.7 Trench and Retaining Wall Backfill and Compaction

Bedding material used within the pipe zone should conform to the requirements of the current Greenbook and the pipe manufacturer. Where applicable, sand having a sand equivalent (SE) of 20 or greater (per Caltrans Test Method [CTM] 217) may be used to bed and shade the pipes within the bedding zone. Sand backfill should be densified by jetting or flooding and then tamped to ensure adequate compaction. Bedding sand should be from a natural source, manufactured sand from recycled material is not suitable for jetting. The onsite soils may generally be considered suitable as trench backfill (zone defined as 12 inches above the pipe to subgrade), provided the soils are screened of rocks greater than 6 inches in maximum dimension, construction debris and organic material. Trench backfill should be compacted in uniform lifts (as outlined above in Section “Material for Fill”) by mechanical means to at least 90 percent relative compaction (per ASTM D1557). If gap-graded rock is used for trench backfill, refer to above Section 4.1.6.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, flowable fill such as sand-cement slurry may be substituted for compacted backfill. The slurry should contain about one sack of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Sand cement slurry placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

Any required retaining wall backfill should consist of predominately granular, sandy soils outlined in Section 4.1.5. The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (Refer to Figure 2). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to a minimum of 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted. If gap-graded rock is used for retaining wall backfill, refer to above Section 4.1.6.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.8 Shrinkage and Subsidence

Allowance in the earthwork volumes budget should be made for an estimated ±5 percent reduction in volume of near-surface (upper approximate 5 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence, due to earthwork operations, is expected to be on the order of 0.1-foot. These values are estimates only and exclude losses due to removal of any vegetation or debris. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor and accuracy of the topographic survey.

4.2 Preliminary Foundation Design Parameters

Site soils are anticipated to be of Very Low to Low expansion potential (EI of 50 or less per ASTM D4829). However, this must be verified based on as-graded conditions. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork. Recommended soil bearing and estimated static settlement are provided in Section 4.3. If requested, geotechnical foundation parameters for post-tensioned slabs will be provided.

In consideration of site expansive soils, the following preliminary recommendations may be used:

- Moisture-condition (presoak) slab subgrade to 100% of optimum moisture content to a minimum depth of 12 inches prior to trenching.

Slab thickness and reinforcement should be determined by the structural engineer.

For elastic design of a foundation supporting sustained concentrated loads, a modulus of vertical subgrade reaction (k) of 100 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed foundation using the following formula:

$k = 100[(B+1)/2B]^2$
k = modulus of vertical subgrade reaction, pounds per cubic inch (pci)
B = foundation width (feet)

4.2.1 Slab Underlayment Guidelines

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand, or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.2.2 Shallow Foundation Maintenance

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation, should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. These recommendations should be provided to future owners and property management personnel.

4.3 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, the following minimum footing widths and embedments for isolated spread and continuous wall footings are recommended for the corresponding allowable bearing pressures.

TABLE 3

Allowable Soil Bearing Pressures

Allowable Static Bearing Pressure (psf)	Minimum Footing Width (feet)	Minimum Footing Embedment* (feet)
3,000	3	2
2,500	2	2
2,000	2	1.5
1,500	1.5	1.0

*Refers to minimum depth to the bottom of the footing below lowest adjacent finish grade.

These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5 horizontal feet to 1-foot vertical) conditions only. Bearing values indicated above are for total dead loads and 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 loading.

Soil settlement is a function of footing dimensions and applied soil bearing pressure. In utilizing the above-mentioned allowable bearing capacity, assumed structural loads, and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be on the order of 1-inch or less. Differential settlement should be anticipated between nearby columns or walls where a large differential loading condition exists. Settlement estimates should be evaluated by LGC Geotechnical when foundation plans are available.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.30 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for lateral resistance. This passive pressure is applicable for level (ground slope equal to or flatter than 5 horizontal feet to 1-foot vertical) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt concrete. The provided allowable passive pressure is based on a factor of safety of 1.5 and may be increased by one-third for short duration wind or seismic loading.

4.4 Lateral Earth Pressures for Retaining Walls

The following preliminary lateral earth pressures may be used for retaining walls 10 feet or less. Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design.

The following lateral earth pressures are presented on Table 4 below for approved select granular soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and Very Low expansion potential (EI of 20 or less per ASTM D4829). The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria.

TABLE 4

Lateral Earth Pressures – Sandy Backfill

Conditions	**Equivalent Fluid Unit Weight (pcf)**
	Level Backfill
	Approved Soils
Active	35
At-Rest	55

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 earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Figure 2). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. 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 consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed basement/retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist vehicular traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.3 and 0.5 may be used for the active and at-rest conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The retaining/basement wall designer should contact the geotechnical consultant for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 10 pcf for a level backfill condition. This increment should be applied in addition to the provided static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 2019 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. The provided seismic lateral earth pressure should not be used for retaining walls exceeding 12 feet in height. If a retaining wall greater than 12 feet in height is proposed or a retaining wall with a sloping backfill condition, the retaining wall designer should contact the geotechnical consultant for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures.
Seismic lateral earth pressures are estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.5 Preliminary Pavement Sections

The following preliminary minimum asphalt concrete (AC) pavement sections are provided in Table 5 based on an assumed R-value of 20 for Traffic Indices (TI) of 5 through 7. These recommendations should be confirmed with R-value testing of representative near-surface soils at the completion of earthwork. Final street sections should be confirmed by the project civil engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 5

Paving Section Options

Assumed Traffic Index	5.0	6.0	7.0
R -Value Subgrade	20	20	20
AC Thickness	4.0 inches	4.0 inches	5.0 inches
Aggregate Base Thickness	5.0 inches	8.5 inches	10.0 inches

The provided preliminary Portland Cement concrete pavement section is based on the guidelines of the American Concrete Institute (ACI 330R-08). For the final design section, we recommend a traffic study be performed as LGC Geotechnical does not perform traffic engineering. Traffic study should include the design vehicle (number of axles and load per axle) and estimated number of daily repetitions/trips. Based on an assumed Traffic Category C with an assumed Average Daily Truck Traffic (ADTT) of 30, we recommend a preliminary section of a minimum of 6 inches of concrete over 4 inches of compacted aggregate base over compacted subgrade. The concrete should have a minimum compressive strength of 4,000 psi and a minimum flexural strength of 550 psi at the time the pavement is subjected to traffic. Steel reinforcement is not required (ACI, 2013). This pavement section assumes that edge restraints like a curb and gutter will be provided. To reduce the potential (but not eliminate) for cracking, paving should provide control joints at regular intervals not exceeding 10 feet in each direction. Decreasing the spacing of these joints will further reduce, but not eliminate the potential for unsightly cracking. Preliminary pavement section is based on a 20-year design. Truck loading is defined one 16-kip axle and two 32-kip tandem axles (80 kips). Alternate section(s) may be provided based on anticipated specific traffic loadings and repetitions provided by others. LGC Geotechnical does not perform traffic engineering and determination of traffic loading is not the purview of the geotechnical consultant.

The thicknesses shown are for minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous section "Site Earthwork" and the related sub-sections of this report.

4.6 Soil Corrosivity

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing indicated soluble sulfate content less than approximately 0.02 percent, chloride contents of 62 parts per million (ppm) and 126 ppm, pH values of 7.3 ad 7.7, and minimum resistivity values of 2,100 ohm-centimeters and 3,960 ohm-centimeters. . Based on Caltrans Corrosion Guidelines (2021), 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 1,500 ppm (0.15 percent) or greater.

Based on laboratory sulfate test results, the near-surface soils have an exposure class of "S0" per ACI 318-14, Table 19.3.1.1 with respect to sulfates. This must be verified based on as-graded conditions.

4.7 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, etc.) has a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 6. These guidelines will reduce the potential for irregular cracking and promote cracking along control joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement and control joints will further reduce cosmetic distress. Please note that where tile is planned to be placed over concrete the architect must take special care to ensure that control joints are carried up through the tile from the concrete. The concrete flatwork will move over time, the architect and designer must make provisions for this movement in both design and construction.

TABLE 6

Nonstructural Concrete Flatwork for Low Expansion Potential

	Flatwork	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	City/Agency Standard
Reinforcement	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	City/Agency Standard

4.8 Surface Drainage and Landscaping

4.8.1 Precise Grading

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed structures be sloped away from the proposed structures and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building geometry necessitates that drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.8.2 Landscaping

Planters adjacent to a building or structure should be avoided wherever possible or be properly designed (e.g., lined with a membrane), to reduce the penetration of water into the adjacent footing subgrades and thereby reduce moisture-related damage to the foundation. Planting areas at grade should be provided with appropriate positive drainage. Wherever possible, exposed soil areas should be above adjacent paved grades to facilitate drainage. Planters should not be depressed below adjacent paved grades unless provisions for drainage, such as multiple depressed area drains, are constructed. Adequate drainage gradients, devices, and curbing should be provided to prevent runoff from adjacent pavement or walks into the planting areas. Irrigation methods should promote uniformity of moisture in planters and beneath adjacent concrete flatwork. Overwatering and underwatering of landscape areas must be avoided. Irrigation levels should be kept to the absolute minimum level necessary to maintain healthy plant life.

Area drain inlets should be maintained and kept clear of debris in order to properly function. Owners and property management personnel should also be made aware that excessive irrigation of neighboring properties can cause seepage and moisture conditions. Owners and property management personnel should be furnished with these recommendations communicating the importance of maintaining positive drainage away from structures, towards streets, when they design their improvements.

The impact of heavy irrigation or inadequate runoff gradients can create perched water conditions. This may result in seepage or shallow groundwater conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to the change in moisture content of foundation soils, which may cause distress to a structure and associated improvements, moisture content of the soils surrounding the structure should be kept as relatively constant as possible.

4.9 Subsurface Water Infiltration

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentional infiltration below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement. From a geotechnical perspective, generally we do not recommend that water be intentionally infiltrated. If required, we recommend the absolute minimum amount of water be infiltrated into site soils.

The following should be considered in the design of the required infiltration system:

- Please note that the infiltration values reported herein are for native materials (not for compacted fill).
- An adequate setback distance between the proposed infiltration system and adjacent private property should be maintained.
- An adequate setback distance between the proposed infiltration system and proposed building structures should be maintained.
- Any designed infiltration system will require routine periodic maintenance.
- As with any systems that are designed to concentrate the surface flow and direct the water into the subsurface soils, some type of nuisance water and/or other water-related issues should be expected.

LGC Geotechnical should be provided with updated details for the infiltration system when available for geotechnical review.

4.10 Pre-Construction Documentation and Construction Monitoring

A program of documentation and monitoring be devised and put into practice before the onset of any groundwork. LGC Geotechnical can perform these services at your request. This should include, but not necessarily be limited to, detailed documentation of the existing improvements, buildings, and utilities around the area of proposed excavation, with particular attention to any distress that is already present prior to the start of work. At the completion of construction, we recommend that the adjacent properties be re-documented to confirm their condition after potentially damaging activities are completed. In the event of future claims, any post-construction damage may be attributed to other causes.

4.11 *Geotechnical Plan Review*

Project plans (e.g., grading, foundation, etc.) and any other improvement plans, and final project drawings should be reviewed by this office prior to construction to verify that our geotechnical recommendations, provided herein, have been appropriately incorporated. Additional or modified geotechnical recommendations may be required based on the proposed design.

4.12 *Geotechnical Observation and Testing During Construction*

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. Geotechnical observation and testing is required per Section 1705 of the 2019 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc.);
- During retaining wall/utility trench backfill and compaction;
- Preparation of subgrade and placement of aggregate base;
- After excavation of structural footings (e.g., building, retaining/free standing wall, etc.) and prior to placing reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 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 than 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. Therefore, 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.

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, and should not be relied upon after a period of 3 years.

FIGURE 1
Site Location Map

LGC Geotechnical, Inc.

PROJECT NAME	Lovett Industrial- Evans Rd
PROJECT NO.	22086-01
ENG. / GEOL.	BTZ/KBC
SCALE	Not to Scale
DATE	July 2022

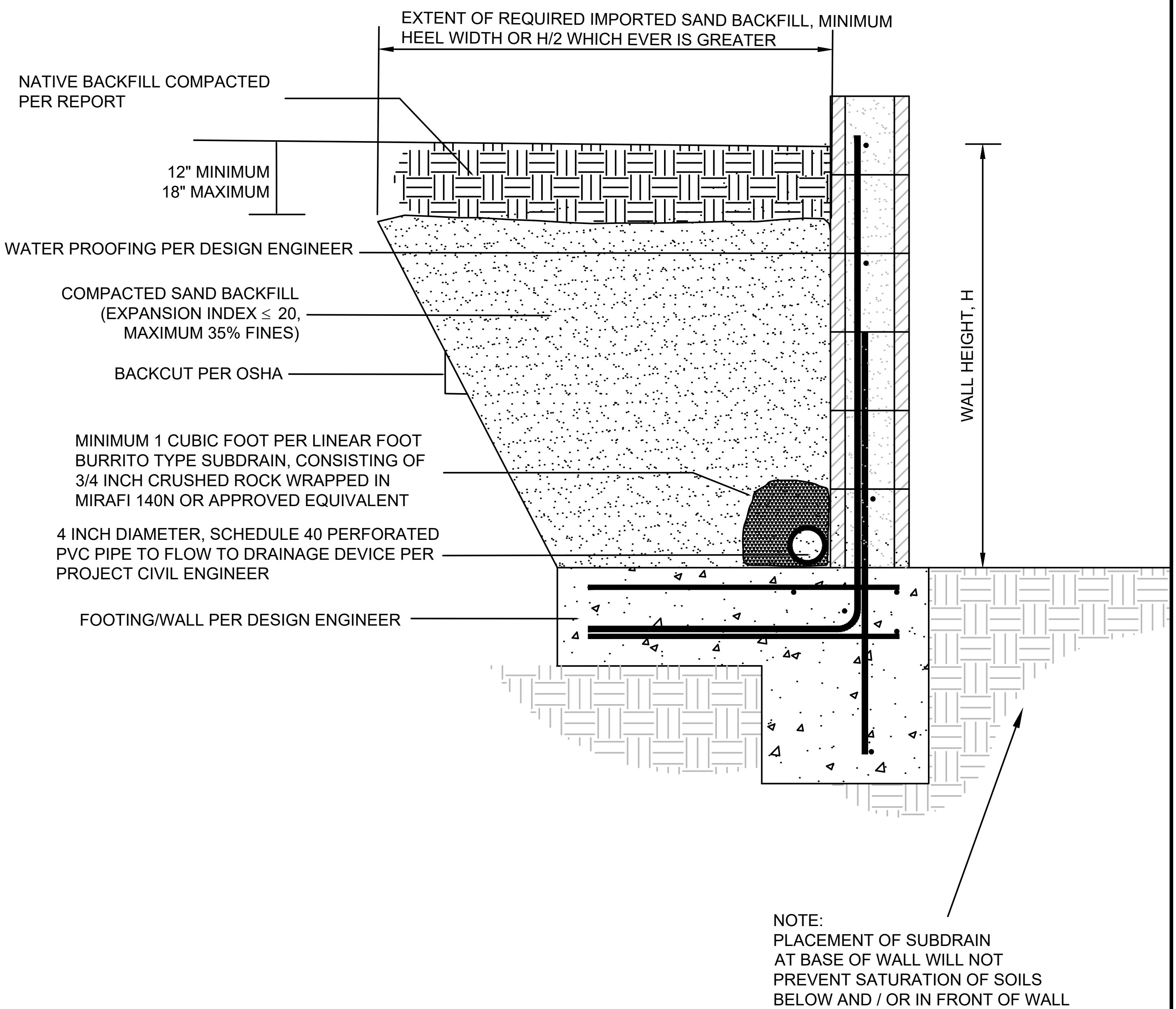

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FIGURE 2
Retaining Wall Backfill Detail

PROJECT NAME	Lovett Industrial- Evans Rd
PROJECT NO.	22086-01
ENG. / GEOL.	BTZ/KBC
SCALE	Not to Scale
DATE	July 2022

Appendix A
References

APPENDIX A

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Appendix B
Boring Logs & Field Infiltration Data

Geotechnical Boring Log Borehole HS-1

Date: 6/13/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett Industrial - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1424' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Logged By RNP
Sampled By RNP
Checked By BTZ

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 1.5' - Undocumented Fill (Afu)** @ 0' - Dead dry grass, vegetation **@ 1.5 to T.D. - Old Alluvial Fan Deposit (Qof)**	CR EI
			R-1	5 10 15	120.4	7.2	SM	@ 2.5' - Silty SAND: dark reddish brown, moist to moist, loose to medium dense, trace of fine gravel and coarse sand, porous with minor roots	
1420									
	5		SPT-1	6 8 8		12.4	ML	@ 5' - Sandy SILT: olive brown to dark reddish brown, moist, very stiff	SA
			R-2	9 24 41	116.2	15.1		@ 7.5' - Sandy SILT: yellowish brown, moist, hard	
1415									
	10		SPT-2	10 11 16		8.8	SM	@ 10' - Silty SAND: olive brown, moist, dense	#200
1410									
	15		R-3	10 30 50/4"	116.5	4.3		@ 15' - Silty SAND: light brown, slightly moist, very dense	
1405									
	20		SPT-3	9 13 14		2.8		@ 20' - Silty SAND: light gray, slightly moist, dense, few fine gravel	
1400									
	25		R-4	16 22 44	111.1	3.2		@ 25' - Silty SAND: light gray, slightly moist,very dense, few fine gravel	
1395									
	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: 6/16/2022

Geotechnical Boring Log Borehole HS-1

Date: 6/13/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett Industrial - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1424' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	11 14 15		3.0	SM	@ 30' - Silty SAND with trace of Gravel: light gray, slightly moist, dense, medium to coarse grained	
1390									
	35		R-5	7 24 34	108.7	2.9	SP	@ 35' - SAND with trace of Gravel: light gray to brown, slightly moist, dense	
1385									
	40		SPT-5	13 16 23		6.9	SM	@ 40' - Silty SAND with trace of Gravel: light gray to yellow brown, moist, dense	
1380									
	45		R-6	12 22 31	97.7	27.0	SM/ML	@ 45' - Silty SAND to Sandy SILT: yellowish brown, moist, dense to hard	
1375									
	50		SPT-6	8 20 22		9.2	SM	@ 50' - Silty SAND: light brown, moist, very dense	
1370								Total Depth = 51.5' Groundwater Not Encountered Caving = Approximately 40' after removing auger Backfilled with Cuttings on 6/13/2022	
	55								
1365									
	60								

LGC Geotechnical, Inc.

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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: 6/13/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett Industrial - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1424' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@ 0 to 1.5' - <u>Undocumented Fill (Afu)</u>** @ 0' - 1 foot tall dry weeds	
			R-1	18 27 41	125.7	8.2	SM	**@ 1.5 to T.D. - <u>Old Alluvial Fan Deposit (Qof)</u>** @ 2.5' - Silty SAND: dark brown, dry to slightly moist, medium dense to dense, trace of fine gravel, scattered root hairs	
1420	5		SPT-1	13 22 26		10.3		@ 5' - Silty SAND: orange brown, dry to slightly moist, medium dense	#200
			R-2	15 25 35	112.4	14.6	ML	@ 7.5' - Sandy SILT: olive brown, moist, hard	AL CN
1415	10		SPT-2	4 12 15		13.1	SM	@10' - Silty fine SAND: brown, slightly moist to moist, medium dense	
1410	15		R-3	16 23 20	109.7	5.1	SP-SM	@ 15' - SAND with Silt: reddish brown, slightly moist to moist, medium dense, little gravel, medium to coarse grained	
1405	20		SPT-3	3 6 17		23.7	ML	@ 20' - Sandy SILT: brown, moist, very stiff	
1400	25		R-4	18 28 33	118.7	4.3	SP-SM	@ 25' SAND with Silt: light gray to light brown, slightly moist, dense	
1395	30							Total Depth = 26.5' Groundwater Not Encountered Backfilled with Cuttings on 6/13/2022	

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

Last Edited: 6/16/2022

Geotechnical Boring Log Borehole HS-3

Date: 6/13/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett Industrial - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1423' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@ 0 to 1.5' - Undocumented Fill (Afu)** @ 0' - 1 foot tall weeds	EI
1420			R-1	15 23 24	117.8	6.3	SM	**@ 1.5 to T.D. - Old Alluvial Fan Deposit (Qof)** @ 2.5' - Silty SAND: brown to dark brown, dry to slightly moist, dense, porous with root hairs	
	5	B-1	SPT-1	8 7 6		8.3		@ 5' - Silty SAND with trace of Gravel: brown, slightly moist, medium dense	#200
1415			R-2	10 11 22	101.2	19.4	MH	@ 7.5' - Sandy SILT: olive brown, moist, very stiff	AL CN
	10		SPT-2	5 5 8		22.7	ML	@ 10' - Sandy SILT: brown, moist, very stiff	
1410									
	15		R-3	9 7 4	103.3	10.8	SM	@ 15' - Silty SAND with trace of Gravel: brown to olive brown, slightly moist to moist, loose	
1405									
	20		SPT-3	5 10 16		15.4	ML	@ 20' - Sandy SILT: dark brown, moist, very stiff	
1400									
	25		R-4	50/3"	112.8	14.9	SM	@ 25' - Silty SAND: olive brown, moist, very dense	
1395								Total Depth = 26.5' Groundwater Not Encountered Caving = Approximately 22' after removing auger Backfilled with Cuttings on 6/13/2022	
	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: 6/16/2022

Geotechnical Boring Log Borehole HS-4

Date: 6/22/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1425' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By JMN
Sampled By JMN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@ 0 to 1.5' - <u>Undocumented Fill (Afu)</u>** @ 0' - Grass with Silty SAND - light brown, dry	
			SPT-1	18 26 38		5.2	SM	**@ 1.5 to T.D. - <u>Old Alluvial Fan Deposits (Qof)</u>** @ 2.5' - Silty SAND: red yellow, slightly moist, very dense	MD #200
1420	5	B-1	R-1	28 50/5"	112.8	14.9		@ 5' - Silty SAND: red yellow, moist, very dense	
			SPT-2	8 10 8		15.2	ML	@ 7.5' - Sandy SILT: light yellow orange, very moist, very stiff	
1415	10		R-2	8 13 13	103.9	24.7	CL	@ 10' - CLAY with Sand: dull yellow orange, very moist, very stiff	AL CN
1410	15		SPT-3	3 6 9		25.5	ML	@ 15' - Sandy SILT: dull yellow orange, very moist, very stiff	
1405	20		R-3	9 15 24	112.2	5.2	SM	@ 20' - Silty SAND and Gravel: slightly moist, dense	
1400	25		SPT-4	6 9 15		5.6		@ 25' - Silty SAND: light gray, slightly moist, dense	
	30								

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

Last Edited: 7/11/2022

Geotechnical Boring Log Borehole HS-4

Date: 6/22/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1425' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By JMN
Sampled By JMN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-4	23 31 50/4"	120.7	11.1	SM	@ 30' - Silty SAND: bright yellowish brown, moist, very dense	
1390	35		SPT-5	9 14 21		19.8	ML	@ 35' - Sandy SILT: yellow brown, moist, hard	
1385	40		R-5	11 31 42	107.5	21.7		@ 40' - Sandy SILT: yellow brown, moist, hard, small amounts of fine gravel	
1380	45		SPT-6	6 17 22		13.7	SM	@ 45' - Silty SAND: dull yellow brown, moist, dense	
1375	50		R-6	17 50/6"	123.3	11.6		@ 50' - Silty SAND: light yellow orange, moist, very dense	
1370	55							Total Depth = 51' Groundwater Not Encountered Caving = Approximately 46' after removing auger Backfilled with Cuttings on 6/22/2022	
	60								

LGC Geotechnical, Inc.

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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-5

Date: 6/22/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1425' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By JMN
Sampled By JMN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@ 0 to 1.5' - Undocumented Fill (Afu)** @ 0' - Tall grass/flowers	
			R-1	6 12 17	127.6	10.5	SM/ML	**@ 0 to T.D. - Old Alluvial Fan Deposits (Qof)** @ 2.5' - Silty SAND to Sandy SILT: dark brown, moist, medium dense/very stiff	
1420	5	B-1	SPT-1	5 8 11		12.6	SM	@ 5' - Silty SAND: bright yellowish brown, moist, medium dense	
			R-2	5 12 18	124.6	11.2	CL	@ 7.5' - CLAY with Sand: dark yellowish brown, moist, very stiff	CN
1415	10		SPT-2	7 6 7		17.0		@ 10' - Sandy CLAY: dull yellowish brown, moist, very stiff	#200
1410	15		R-3	5 13 19	99.9	27.4	ML	@ 15' - Sandy SILT: dull yellowish brown, very moist, very stiff	
1405	20		SPT-3	6 9 9		20.2	SC	@ 20' - Clayey SAND: dull yellow orange, moist, medium dense	#200
1400	25		R-4	7 8 32	93.4	26.0	ML	@ 25' - Sandy SILT: dull yellow orange, very moist, hard	SA
	30								

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

Last Edited: 7/12/2022

Geotechnical Boring Log Borehole HS-5

Date: 6/22/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1425' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Logged By JMN
Sampled By JMN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	8 10 16		15.4	SM	@ 30' - Silty SAND: yellow orange, moist, medium dense	
1390	35		R-5	16 27 32	116.4	4.9	SP-SM	@ 35' - SAND with Silt: slightly moist, dense	
1385	40		SPT-5	13 15 16		7.7	SM	@ 40' - Silty SAND: dull yellow orange to light gray, moist, dense	
1380	45		R-6	21 31 38	117.8	5.4	SP-SM	@ 45' - SAND with Silt: light gray with white, dry to slightly moist, very dense	
1375	50		SPT-6	10 12 19		28.2	CL	@ 50' - Sandy CLAY: dull yellowish brown, very moist, hard	
1370	55							Total Depth = 51.5' Groundwater Not Encountered Caving = Approximately 45' after removing auger Backfilled with Cuttings on 6/22/2022	
	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

Geotechnical Boring Log Borehole HS-6

Date: 6/22/2022	**Drilling Company:** Cal Pac Drilling
Project Name: Lovett - Evans Rd.	**Type of Rig:** Truck Mounted
Project Number: 22086-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~1423' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By JMN
Sampled By JMN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@ 0 to 1.5' - Undocumented Fill (Afu)** @ 0' - Grass/top soil, silty sand, dry	CR EI
1420			SPT-1	4 4 8		11.8	SM	**@ 1.5 to T.D. - Old Alluvial Fan Deposits (Qof)** @ 2.5' - Silty SAND: dull yellowish brown, moist, very loose	
	5	B-1	R-1	9 16 20	120.3	13.7	CL	@ 5' - Sandy CLAY: dull yellowish brown to brown, slightly moist, very stiff	AL CN
1415			SPT-2	4 3 3		11.4	SM	@ 7.5' - Silty SAND: reddish brown, moist, loose	SA
	10		R-2	4 9 12	121.5	11.9		@ 10' - Silty SAND: dull yellow orange, moist, medium dense	
1410									
	15		SPT-3	4 8 16		20.4	ML	@ 15' - Sandy SILT: dull yellow orange, very moist, very stiff	
1405									
	20		R-3	4 18 19	91.9	31.4		@ 20' - Sandy SILT: dull yellow orange, very moist to wet, very stiff	
1400									
	25		SPT-4	7 9 14		16.1		@ 25' - Sandy SILT: dull yellow orange to light gray, moist, very stiff	
1395								Total Depth = 26.5' Groundwater Not Encountered Caving = Approximately 23' after removing auger Backfilled with Cuttings on 6/22/2022	
	30								

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

Last Edited: 7/12/2022

Infiltration Test Data Sheet

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/14/2022
Boring Number: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	12
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:35	9:00	25.0	8.08	9.35	1.27	Yes
2	9:01	9:26	25.0	7.35	8.75	1.40	Yes

*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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:28	9:42	14.0	6.13	7.05	0.92	1.4
2	9:43	9:54	11.0	6.70	7.35	0.65	1.4
3	9:55	10:05	10.0	5.75	6.38	0.63	1.2
4	10:05	10:17	12.0	6.38	7.25	0.87	1.6
5	10:19	10:29	10.0	6.95	7.55	0.60	1.5
6	10:29	10:39	10.0	7.55	8.05	0.50	1.4
7							
8							
9							
10							
11							
12							

Tested Infiltration Rate (No Factor of Safety)	**1.4**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.5**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/14/2022
Boring Number: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	12
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:39	9:04	25.0	5.90	6.65	0.75	Yes
2	9:05	9:30	25.0	4.85	5.73	0.88	Yes

*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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:31	9:45	14.0	3.10	3.50	0.40	0.4
2	9:45	9:57	12.0	3.50	5.00	1.50	1.9
3	9:59	10:09	10.0	3.20	4.50	1.30	1.9
4	10:09	10:19	10.0	3.08	4.40	1.32	1.9
5	10:21	10:31	10.0	2.70	2.92	0.22	0.3
6	10:31	10:41	10.0	2.92	3.80	0.88	1.2
7							
8							
9							
10							
11							
12							

Tested Infiltration Rate (No Factor of Safety)	**1.2**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.4**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012
Spreadsheet Revised on: 10/30/2019

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/14/2022
Boring Number: I-3

Test hole dimensions (if circular)	
Boring Depth (feet)*:	14
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:47	9:12	25.0	8.80	9.40	0.60	Yes
2	9:13	9:38	25.0	7.50	7.80	0.30	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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:40	10:12	32.0	5.65	6.35	0.70	0.3
2	10:14	10:44	30.0	3.50	5.55	2.05	0.9
3	10:44	11:14	30.0	5.55	6.20	0.65	0.3
4	11:14	11:44	30.0	6.20	6.80	0.60	0.3
5	11:44	12:14	30.0	6.80	7.28	0.48	0.3
6	12:15	12:45	30.0	6.40	6.95	0.55	0.3
7	12:45	13:15	30.0	6.95	7.41	0.46	0.3
8	13:16	13:46	30.0	6.60	7.13	0.53	0.3
9	13:46	14:16	30.0	7.13	7.55	0.42	0.2
10	14:16	14:46	30.0	7.55	7.94	0.39	0.2
11	14:46	15:16	30.0	7.94	8.25	0.31	0.2
12	15:16	15:46	30.0	8.25	8.60	0.35	0.2

Tested Infiltration Rate (No Factor of Safety)	**0.2**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019

Infiltration Test Data Sheet

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/14/2022
Boring Number: I-4

Test hole dimensions (if circular)	
Boring Depth (feet)*:	14
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:47	9:12	25.0	5.74	6.23	0.49	No
2	9:17	9:42	25.0	6.03	6.49	0.46	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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:56	10:26	30.0	6.62	6.93	0.31	0.2
2	10:27	10:57	30.0	6.93	7.28	0.35	0.2
3	10:58	11:28	30.0	6.00	6.40	0.40	0.2
4	11:28	11:58	30.0	6.40	6.75	0.35	0.2
5	11:58	12:28	30.0	6.75	7.03	0.28	0.2
6	12:29	12:59	30.0	6.10	6.51	0.41	0.2
7	12:59	13:29	30.0	6.51	6.80	0.29	0.2
8	13:29	13:59	30.0	6.80	7.10	0.30	0.2
9	13:59	14:29	30.0	7.10	7.35	0.25	0.1
10	14:29	14:59	30.0	7.35	7.55	0.20	0.1
11	14:59	15:29	30.0	7.55	7.79	0.24	0.1
12	15:29	15:59	30.0	7.79	8.00	0.21	0.1

Tested Infiltration Rate (No Factor of Safety)	**0.1**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.0**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019

Infiltration Test Data Sheet

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/14/2022
Boring Number: I-5

Test hole dimensions (if circular)	
Boring Depth (feet)*:	14
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:53	9:18	25.0	8.92	9.76	0.84	Yes
2	9:21	9:46	25.0	9.37	9.81	0.44	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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:59	10:29	30.0	9.06	9.61	0.55	0.5
2	10:31	11:01	30.0	8.93	9.56	0.63	0.5
3	11:08	11:38	30.0	8.53	9.21	0.68	0.5
4	11:40	12:13	33.0	8.22	8.85	0.63	0.4
5	12:12	12:42	30.0	8.44	8.88	0.44	0.3
6	12:44	13:14	30.0	8.10	8.79	0.69	0.5
7	13:17	13:47	30.0	8.38	9.02	0.64	0.5
8	13:50	14:20	30.0	9.10	9.63	0.53	0.4
9	14:22	14:52	30.0	9.64	10.15	0.51	0.5
10	14:55	15:25	30.0	8.64	9.19	0.55	0.4
11	15:27	15:57	30.0	8.53	9.05	0.52	0.4
12	15:59	16:29	30.0	8.41	8.90	0.49	0.4

Tested Infiltration Rate (No Factor of Safety)	**0.4**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012
Spreadsheet Revised on: 10/30/2019

Infiltration Test Data Sheet

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/14/2022
Boring Number: I-6

Test hole dimensions (if circular)	
Boring Depth (feet)*:	14
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:57	9:22	25.0	6.22	6.84	0.62	Yes
2	9:24	9:49	25.0	6.02	6.48	0.46	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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	10:01	10:31	30.0	6.49	6.93	0.44	0.2
2	10:34	11:04	30.0	6.95	7.30	0.35	0.2
3	11:10	11:40	30.0	7.31	7.64	0.33	0.2
4	11:43	12:13	30.0	7.66	7.95	0.29	0.2
5	12:15	12:45	30.0	7.95	8.29	0.34	0.2
6	12:48	13:18	30.0	8.21	8.52	0.31	0.2
7	13:20	13:50	30.0	8.25	8.53	0.28	0.2
8	13:52	14:22	30.0	8.18	8.51	0.33	0.2
9	14:24	14:54	30.0	8.15	8.40	0.25	0.2
10	14:56	15:26	30.0	8.27	8.54	0.27	0.2
11	15:28	15:58	30.0	8.31	8.57	0.26	0.2
12	16:00	16:30	30.0	8.30	8.57	0.27	0.2

Tested Infiltration Rate (No Factor of Safety)	**0.2**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/23/2022
Boring Number: I-7

Test hole dimensions (if circular)	
Boring Depth (feet)*:	12
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:43	9:08	25.0	5.78	6.86	1.08	Yes
2	9:09	9:34	25.0	5.40	6.10	0.70	Yes

*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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:36	9:46	10.0	5.52	5.84	0.32	0.6
2	9:46	9:56	10.0	5.84	6.11	0.27	0.5
3	9:56	10:06	10.0	5.56	5.82	0.26	0.5
4	10:06	10:16	10.0	5.82	6.10	0.28	0.5
5	10:17	10:27	10.0	5.25	5.55	0.30	0.5
6	10:27	10:37	10.0	5.55	5.85	0.30	0.6
7							
8							
9							
10							
11							
12							

Tested Infiltration Rate (No Factor of Safety)	**0.6**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.2**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012
Spreadsheet Revised on: 10/30/2019

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lovett Industrial - Evans Rd.
Project Number: 22086-01
Date: 6/23/2022
Boring Number: I-8

Test hole dimensions (if circular)	
Boring Depth (feet)*:	12
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	
Pit Length (feet):	
Pit Breadth (feet):	

*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	8:41	9:06	25.0	4.70	5.42	0.72	Yes
2	9:07	9:32	25.0	4.41	5.10	0.69	Yes

*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, t (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, D (feet)	Tested Infiltration Rate(in/hr)
1	9:33	9:43	10.0	4.63	4.85	0.22	0.4
2	9:43	9:53	10.0	4.85	5.12	0.27	0.5
3	9:53	10:03	10.0	5.12	5.40	0.28	0.5
4	10:04	10:14	10.0	4.80	5.03	0.23	0.4
5	10:14	10:24	10.0	5.03	5.26	0.23	0.4
6	10:24	10:34	10.0	5.26	5.47	0.21	0.4
7							
8							
9							
10							
11							
12							

Tested Infiltration Rate (No Factor of Safety)	**0.4**
MinimumFactor of Safety	**3.0**
Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Riverside County 09/2012

Spreadsheet Revised on: 10/30/2019

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 in the boring logs.

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).

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 5 ft	Sandy Silt	51
HS-1 @ 10 ft	Silty Sand	29
HS-2 @ 5 ft	Silty Sand	49
HS-3 @ 5 ft	Silty Sand	46
HS-4 @ 2.5 ft	Silty Sand	45
HS-5 @ 10 ft	Sandy Clay	58
HS-5 @ 20 ft	Clayey Sand	36
HS-5 @ 25 ft	Sandy Silt	50
HS-6 @ 7.5 ft	Silty Sand	42

Laboratory Test Results

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 @ 7.5 ft	42	31	11	ML
HS-3 @ 7.5 ft	51	38	13	MH
HS-4 @ 10 ft	36	24	12	CL
HS-6 @ 5 ft	32	16	16	CL

Expansion Index: The expansion potential of selected representative samples was evaluated by the Expansion Index Test per ASTM D4829. The results are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 1-5 ft	6	Very Low
HS-3 @ 0-5 ft	0	Very Low
HS-6 @ 0-5 ft	34	Low

* Per ASTM D4829

Consolidation: Consolidation tests were performed per ASTM D2435. Samples (2.4 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 provided in this Appendix.

Laboratory Test Results

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-4 @ 0-5 ft	Light Brown Silty Sand	133.5	8.5

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

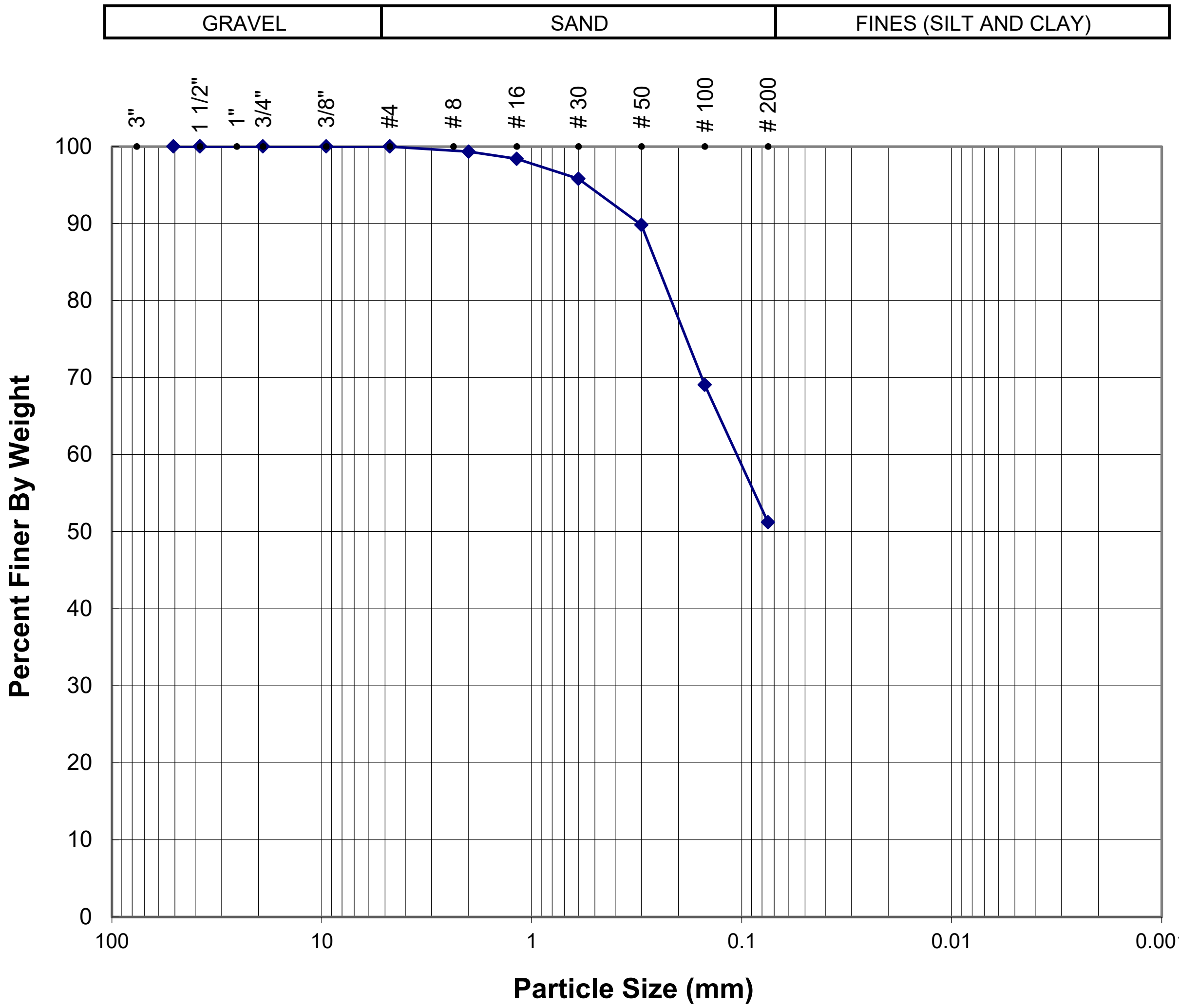
Sample Location	Sulfate Content (ppm)	Sulfate Content (%)
HS-1 @ 1-5 ft	133	< 0.02
HS-6 @ 0-5 ft	181	< 0.02

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

Sample Location	Chloride Content (ppm)
HS-1 @ 1-5 ft	62
HS-6 @ 0-5 ft	126

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-1 @ 1-5 ft	7.3	3,960
HS-6 @ 0-5 ft	7.7	2,100



Location:	Sample No.:	Depth (ft.)	Soil Type	Gravel (%)	Sand (%)	Fines (%)
HS-1	**SPT-1**	**5**	**ML**	**0**	**49**	**51**

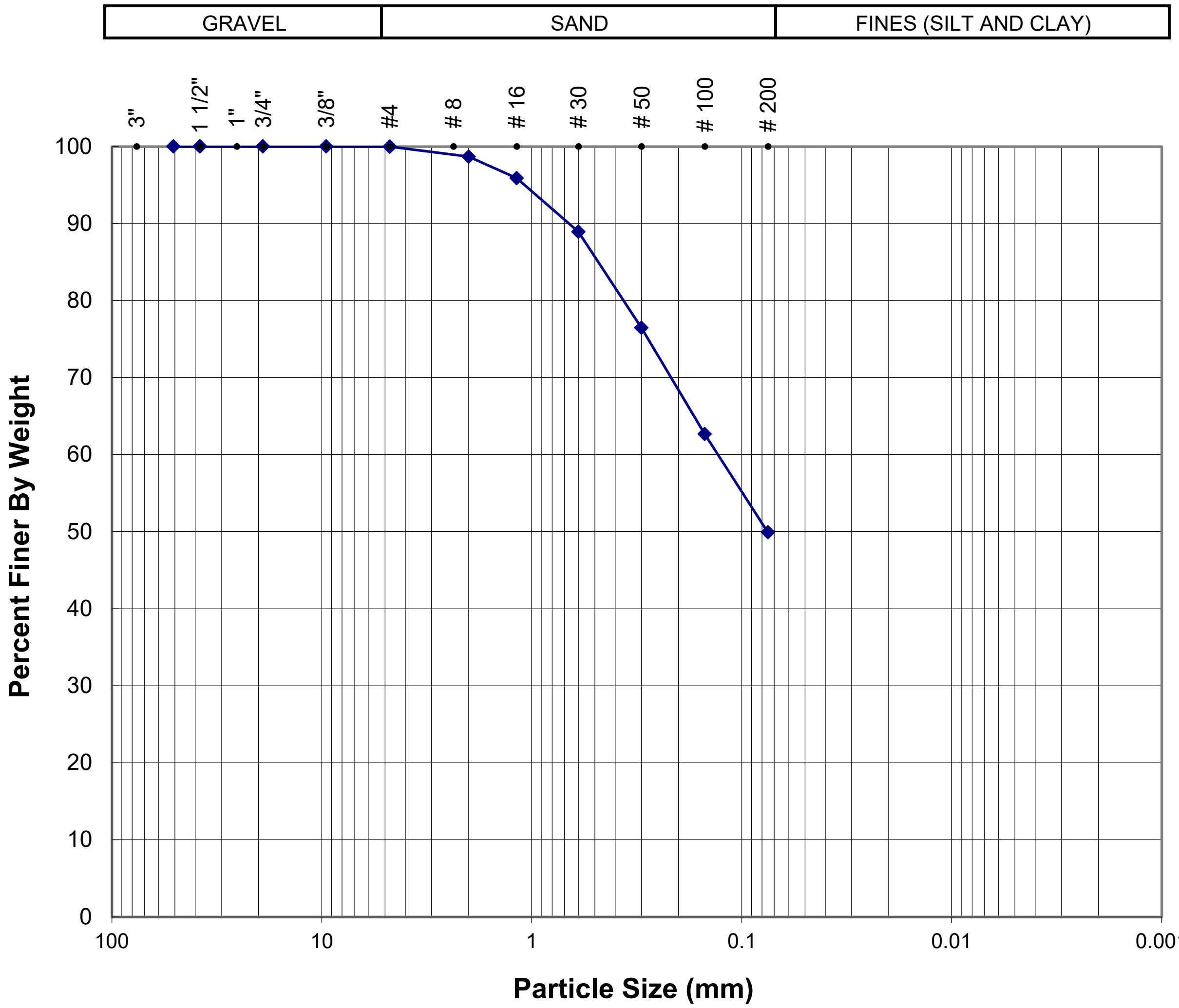
Sample Description: Brown Sandy Silt

LGC Geotechnical, Inc.

PARTICLE SIZE ANALYSIS
(ASTM D 422)

Project Number: 22086-01
Date: Jul-22

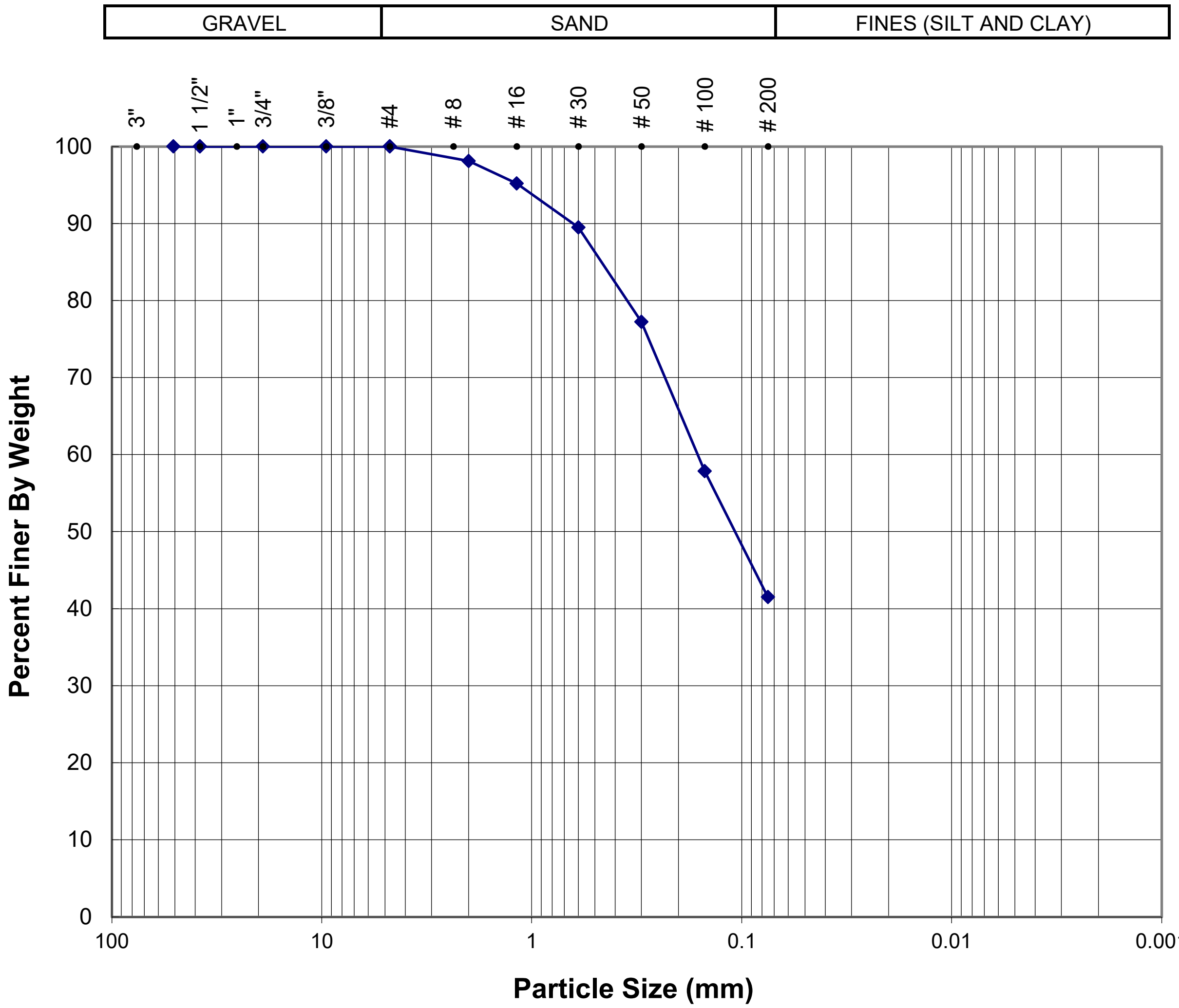
Evans Road



Location:	Sample No.:	Depth (ft.)	Soil Type	Gravel (%)	Sand (%)	Fines (%)
HS-5	**R-4**	**25**	**ML**	**0**	**50**	**50**

Sample Description: Reddish Brown Sandy Silt

LGC Geotechnical, Inc.	**PARTICLE SIZE ANALYSIS** (ASTM D 422)	**Project Number:** 22086-01 **Date:** Jul-22 **Evans Road**

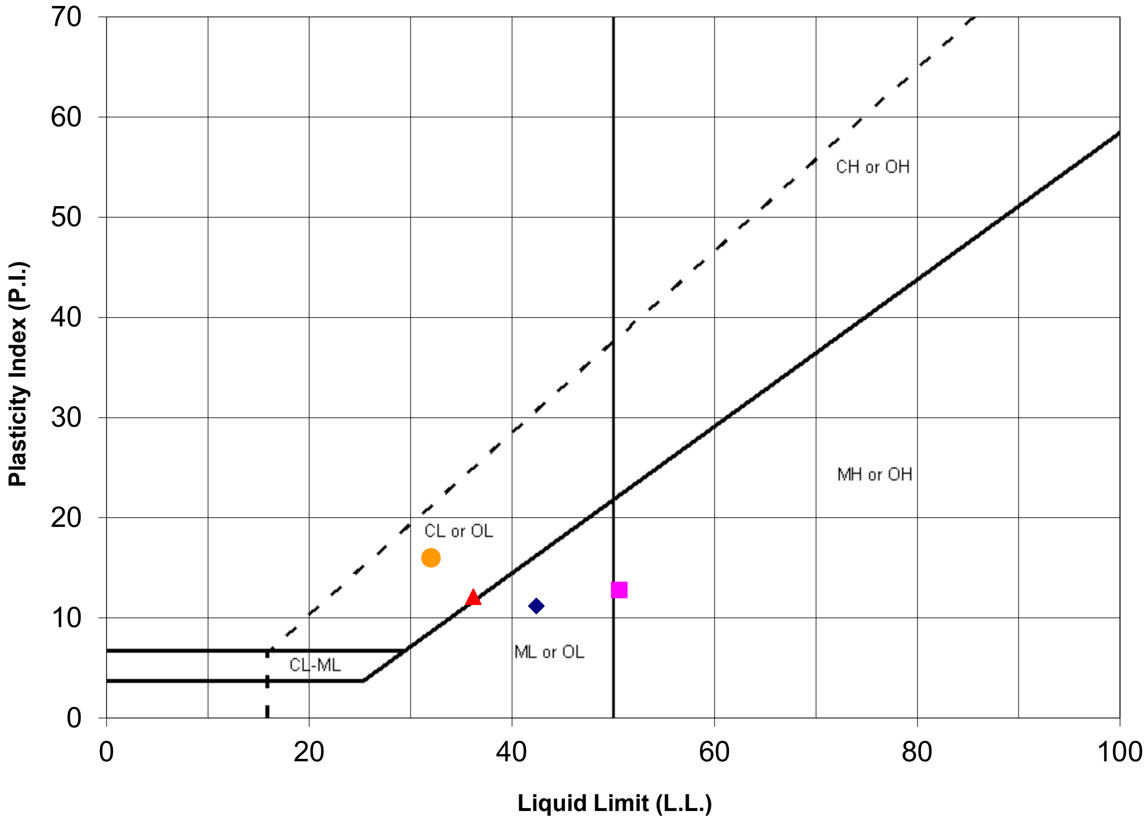


Location:	Sample No.:	Depth (ft.)	Soil Type	Gravel (%)	Sand (%)	Fines (%)
HS-6	**SPT-2**	**7.5**	**SM**	**0**	**58**	**42**

Sample Description: Reddish Brown Silty Sand

LGC Geotechnical, Inc.	**PARTICLE SIZE ANALYSIS** (ASTM D 422)	**Project Number:** 22086-01 **Date:** Jul-22 **Evans Road**

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	R-2	7.5'	-	42	31	11	ML
■	HS-3	R-2	7.5'	-	51	38	13	MH
▲	HS-4	R-2	10'	-	36	24	12	CL
●	HS-6	R-1	5'	-	32	16	16	CL

LGC Geotechnical, Inc.

ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 22086-01
Date: Jul-22

Evans Road

Location	Sample No.	Depth (ft)	Molding Moisture Content (%)	Initial Dry Density (pcf)	Final Moisture Content (%)	Expansion Index	Expansion Classification[1]
HS-1	B-1	1-5'	6.9	121.4	13.5	6	Very Low
HS-3	B-1	0-5'	8.2	115.9	13.6	0	Very Low
HS-6	B-1	0-5'	8.6	115.0	17.7	34	Low

LGC Geotechnical, Inc.

EXPANSION INDEX
(ASTM D 4829)

Project Number: 22086-01
Date: Jul-22

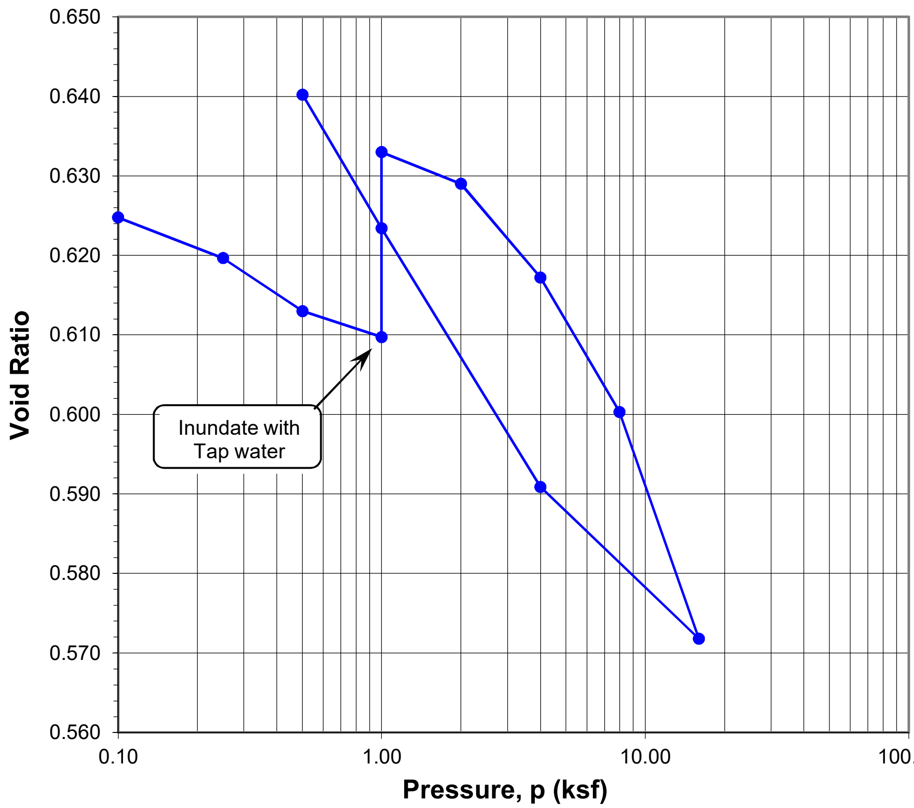
Evans Road

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Lovett Industrial - Evans Road	Tested By: G. Bathala	Date: 06/29/22
Project No.:	22086-01	Checked By: J. Ward	Date: 07/22/22
Boring No.:	HS-2	Depth (ft.): 7.5	
Sample No.:	R-2	Sample Type:	Ring
Soil Identification:	Light olive brown lean clay (CL)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	197.84
Weight of Ring (g):	45.29
Height after consol. (in.):	1.0092
Before Test	
Wt. of Wet Sample+Cont. (g):	171.28
Wt. of Dry Sample+Cont. (g):	150.95
Weight of Container (g):	52.01
Initial Moisture Content (%)	20.5
Initial Dry Density (pcf)	105.2
Initial Saturation (%):	90
Initial Vertical Reading (in.)	0.0671
After Test	
Wt. of Wet Sample+Cont. (g):	271.24
Wt. of Dry Sample+Cont. (g):	240.66
Weight of Container (g):	71.41
Final Moisture Content (%)	24.67
Final Dry Density (pcf):	102.2
Final Saturation (%):	100
Final Vertical Reading (in.)	0.0631
Specific Gravity (assumed):	2.74
Water Density (pcf):	62.43

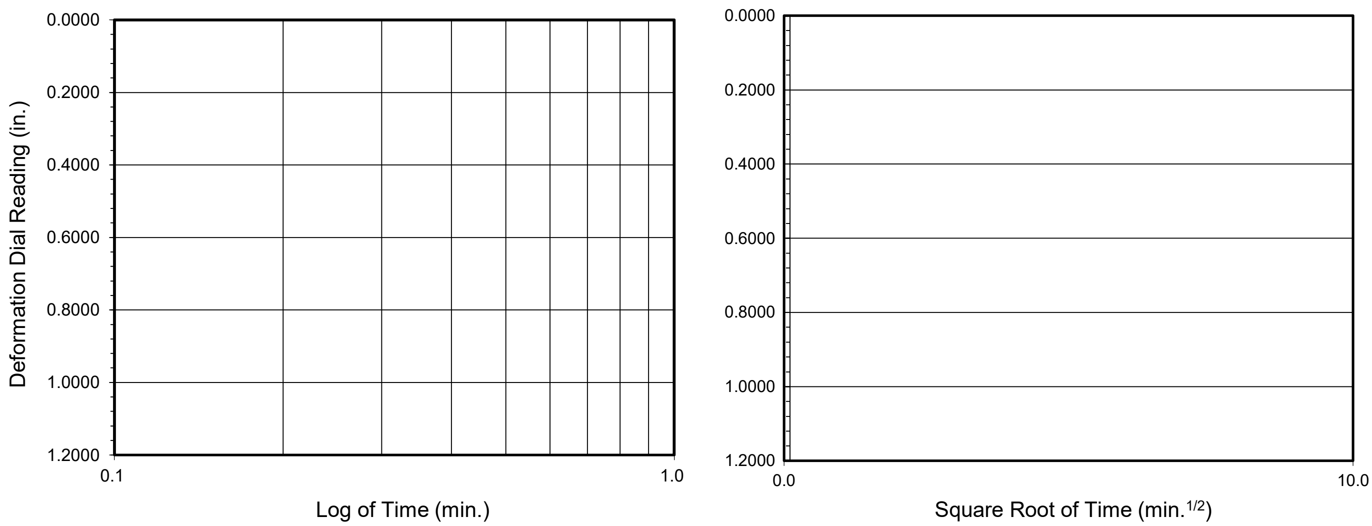


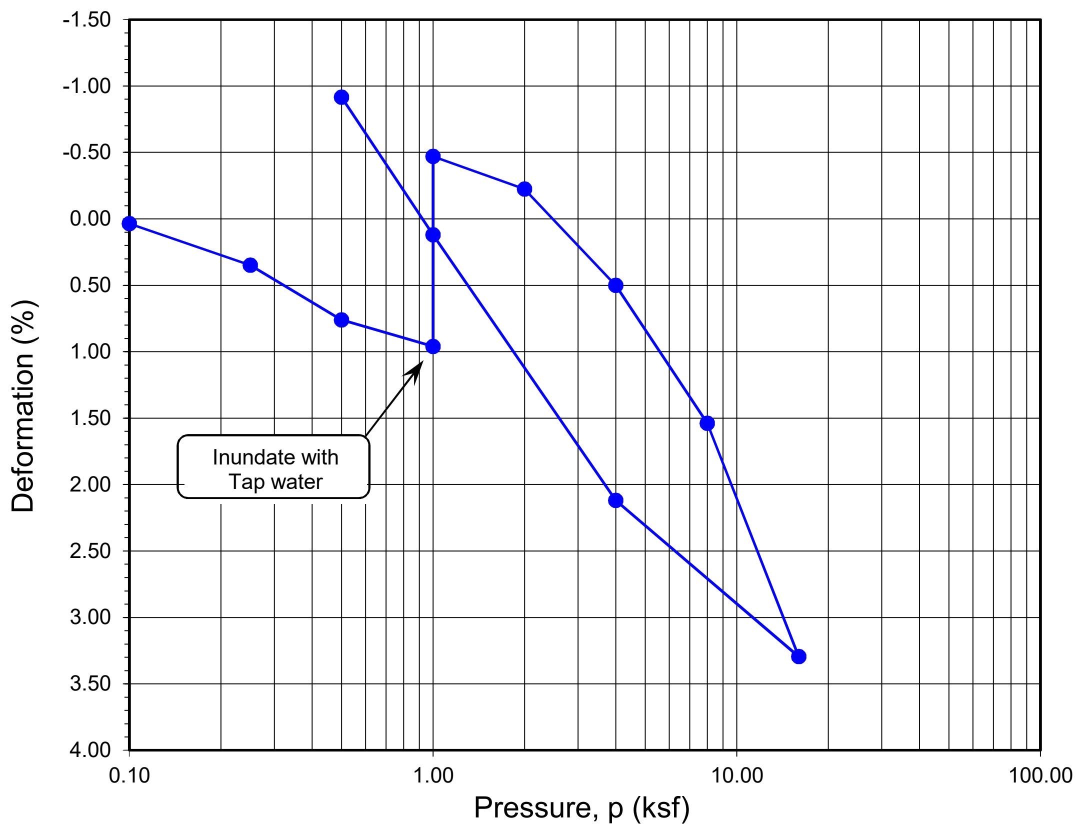
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0675	0.9997	0.00	0.03	0.625	0.03
0.25	0.0708	0.9963	0.02	0.37	0.620	0.35
0.50	0.0756	0.9915	0.09	0.85	0.613	0.76
1.00	0.0791	0.9880	0.24	1.20	0.610	0.96
1.00	0.0648	1.0023	0.24	-0.23	0.633	-0.47
2.00	0.0688	0.9984	0.39	0.17	0.629	-0.22
4.00	0.0774	0.9897	0.53	1.03	0.617	0.50
8.00	0.0891	0.9780	0.66	2.20	0.600	1.54
16.00	0.1082	0.9590	0.81	4.11	0.572	3.30
4.00	0.0949	0.9722	0.66	2.78	0.591	2.12
1.00	0.0738	0.9933	0.55	0.67	0.623	0.12
0.50	0.0631	1.0041	0.51	-0.41	0.640	-0.92

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**	**20.5**	**24.7**	**105.2**	**102.2**	**0.625**	**0.640**	**90**	**100**

Soil Identification: Light olive brown lean clay (CL)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 22086-01

Lovett Industrial - Evans Road

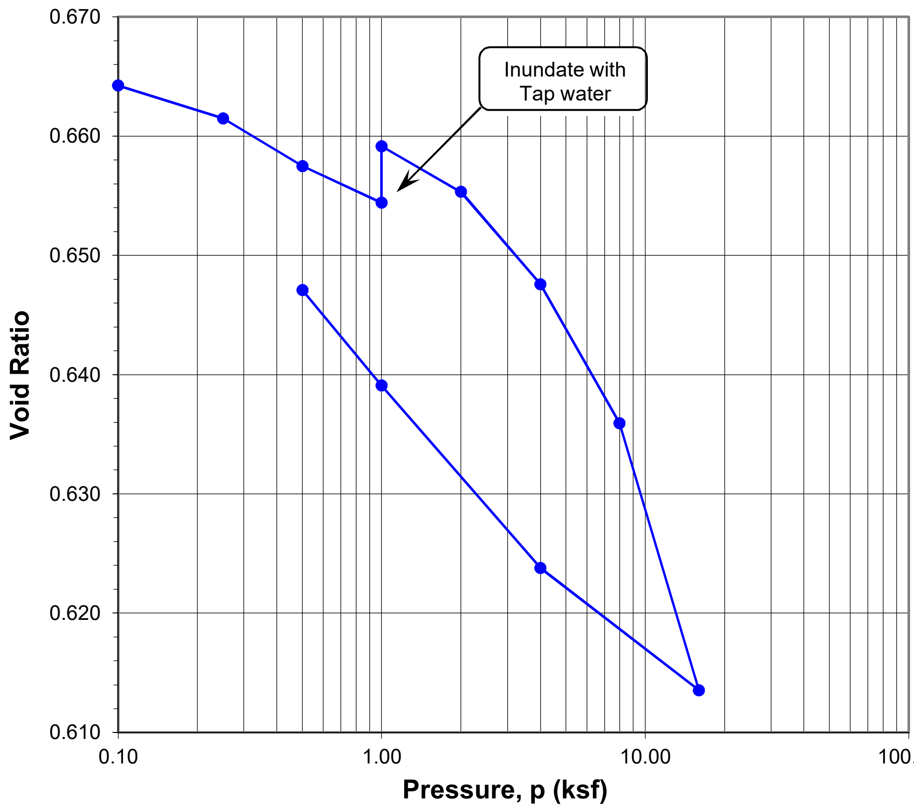
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Lovett Industrial - Evans Road	Tested By: G. Bathala	Date: 06/29/22
Project No.:	22086-01	Checked By: J. Ward	Date: 07/22/22
Boring No.:	HS-3	Depth (ft.): 7.5	
Sample No.:	R-2	Sample Type:	Ring
Soil Identification:	Olive yellow silty clay (CL-ML)		

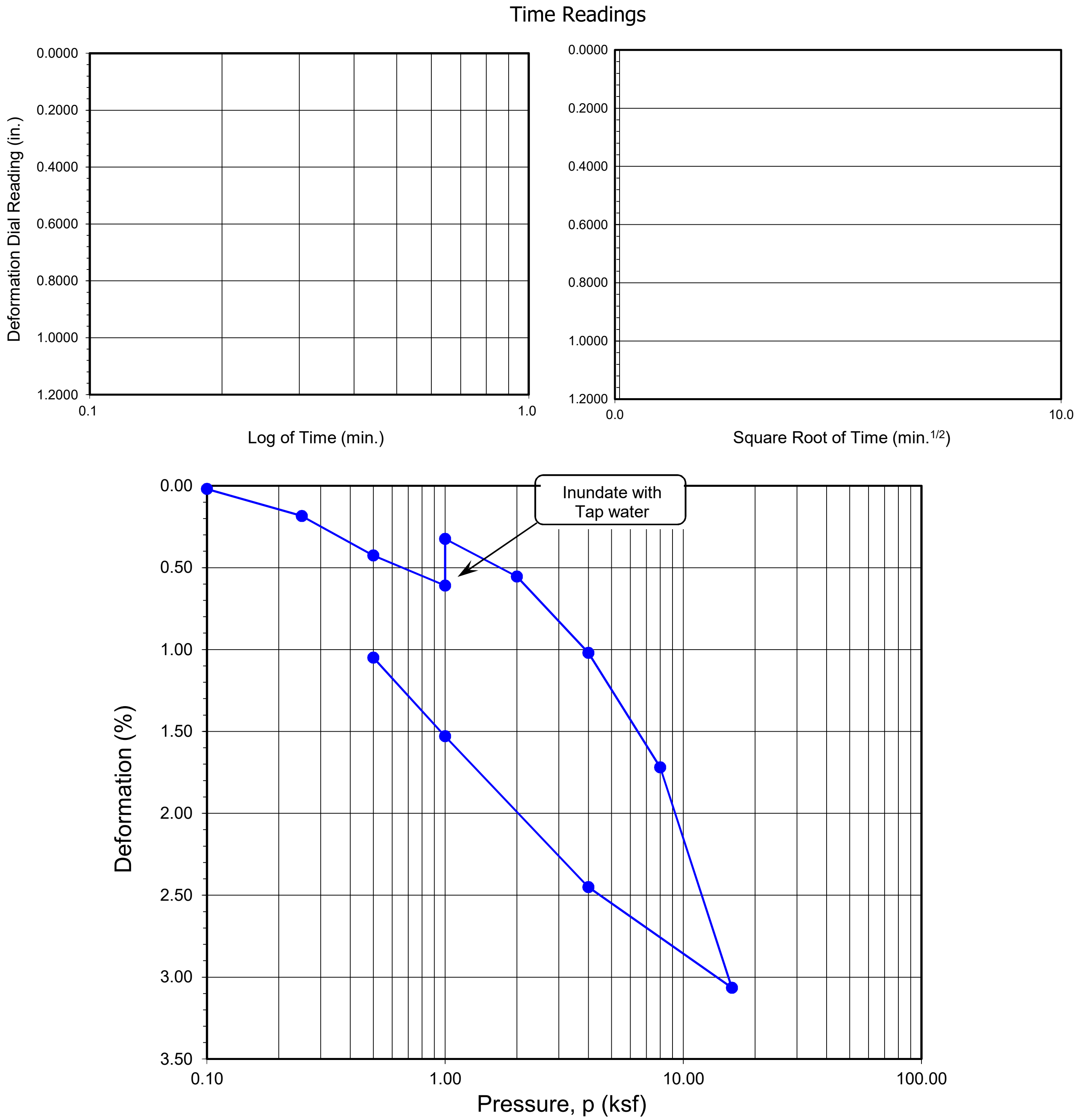
Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	185.26
Weight of Ring (g):	42.18
Height after consol. (in.):	0.9895
Before Test	
Wt. of Wet Sample+Cont. (g):	184.03
Wt. of Dry Sample+Cont. (g):	165.69
Weight of Container (g):	60.94
Initial Moisture Content (%)	17.5
Initial Dry Density (pcf)	101.3
Initial Saturation (%):	71
Initial Vertical Reading (in.)	0.0523
After Test	
Wt. of Wet Sample+Cont. (g):	249.16
Wt. of Dry Sample+Cont. (g):	219.85
Weight of Container (g):	57.94
Final Moisture Content (%)	24.48
Final Dry Density (pcf):	100.6
Final Saturation (%):	98
Final Vertical Reading (in.)	0.0660
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0525	0.9998	0.00	0.02	0.664	0.02
0.25	0.0546	0.9978	0.04	0.22	0.661	0.18
0.50	0.0576	0.9948	0.10	0.52	0.657	0.42
1.00	0.0601	0.9922	0.17	0.78	0.654	0.61
1.00	0.0573	0.9951	0.17	0.50	0.659	0.33
2.00	0.0604	0.9920	0.25	0.81	0.655	0.56
4.00	0.0660	0.9863	0.35	1.37	0.648	1.02
8.00	0.0742	0.9781	0.47	2.19	0.636	1.72
16.00	0.0893	0.9631	0.63	3.70	0.614	3.07
4.00	0.0815	0.9708	0.47	2.92	0.624	2.45
1.00	0.0712	0.9811	0.36	1.89	0.639	1.53
0.50	0.0660	0.9863	0.32	1.37	0.647	1.05

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-2**	**7.5**	**17.5**	**24.5**	**101.3**	**100.6**	**0.665**	**0.647**	**71**	**98**

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

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 22086-01

Lovett Industrial - Evans Road

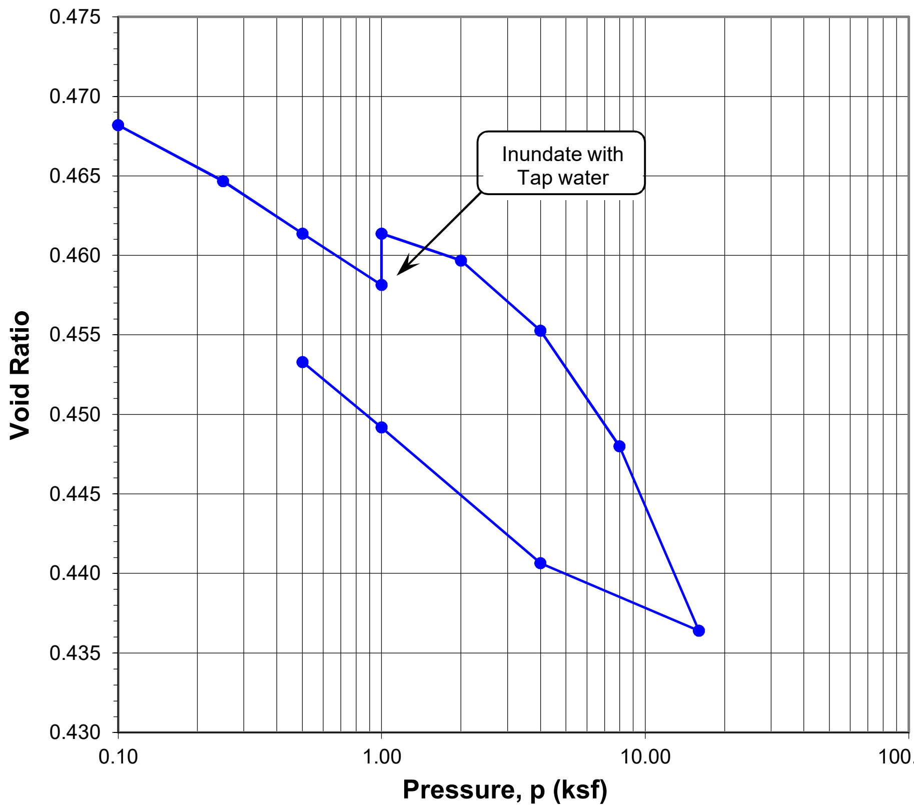
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Lovett Industrial - Evans Road	Tested By: G. Bathala	Date: 06/29/22
Project No.:	22086-01	Checked By: J. Ward	Date: 07/22/22
Boring No.:	HS-4	Depth (ft.): 10.0	
Sample No.:	R-2	Sample Type:	Ring
Soil Identification:	Light olive brown lean clay with sand (CL)s		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	201.22
Weight of Ring (g):	43.38
Height after consol. (in.):	0.9895
Before Test	
Wt. of Wet Sample+Cont. (g):	174.34
Wt. of Dry Sample+Cont. (g):	159.36
Weight of Container (g):	55.17
Initial Moisture Content (%)	14.4
Initial Dry Density (pcf)	114.8
Initial Saturation (%):	83
Initial Vertical Reading (in.)	0.0901
After Test	
Wt. of Wet Sample+Cont. (g):	259.78
Wt. of Dry Sample+Cont. (g):	237.44
Weight of Container (g):	58.02
Final Moisture Content (%)	16.42
Final Dry Density (pcf):	114.3
Final Saturation (%):	93
Final Vertical Reading (in.)	0.1036
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

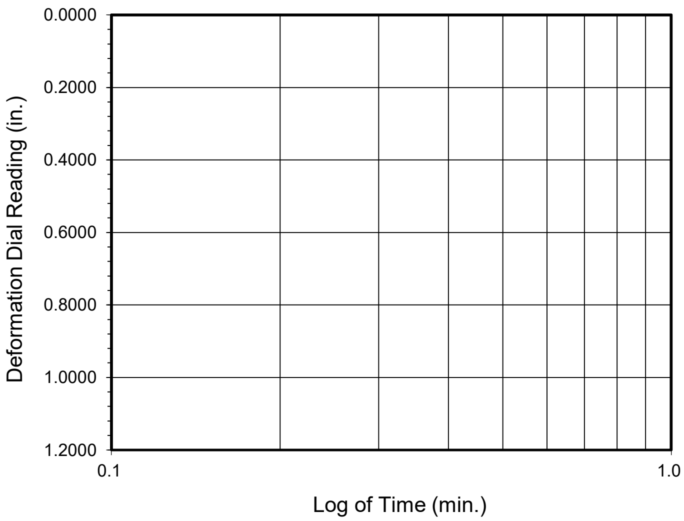


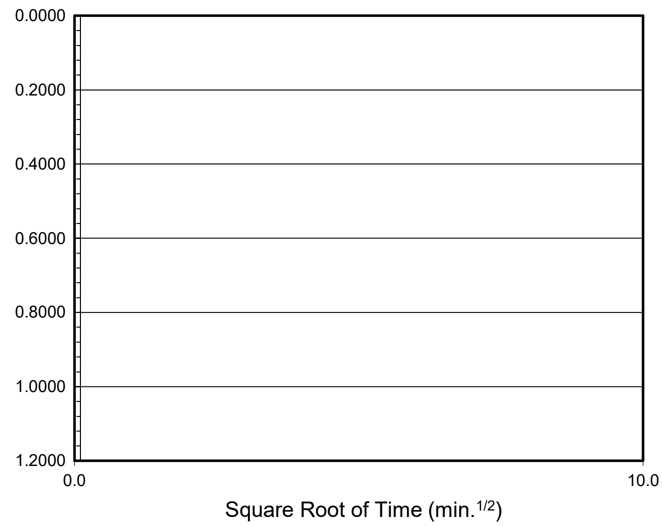
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.0905	0.9997	0.00	0.03	0.468	0.03
0.25	0.0934	0.9968	0.05	0.32	0.465	0.27
0.50	0.0962	0.9939	0.11	0.61	0.461	0.50
1.00	0.0992	0.9909	0.19	0.91	0.458	0.72
1.00	0.0970	0.9931	0.19	0.69	0.461	0.50
2.00	0.0992	0.9910	0.29	0.91	0.460	0.62
4.00	0.1034	0.9868	0.41	1.33	0.455	0.91
8.00	0.1096	0.9805	0.54	1.95	0.448	1.41
16.00	0.1190	0.9711	0.69	2.89	0.436	2.20
4.00	0.1143	0.9758	0.51	2.42	0.441	1.91
1.00	0.1070	0.9831	0.36	1.69	0.449	1.33
0.50	0.1036	0.9865	0.30	1.35	0.453	1.05

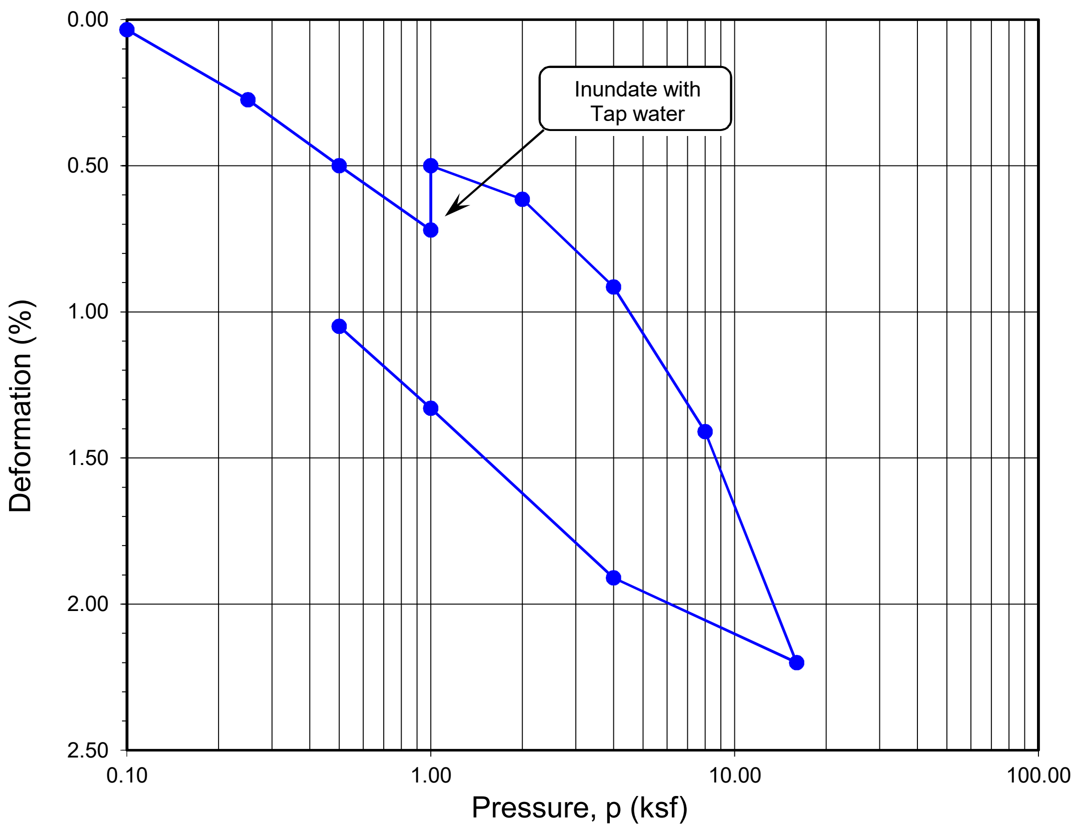
Time Readings

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-4	**R-2**	**10**	**14.4**	**16.4**	**114.8**	**114.3**	**0.469**	**0.453**	**83**	**93**

Soil Identification: Light olive brown lean clay with sand (CL)s

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 22086-01

Lovett Industrial - Evans Road

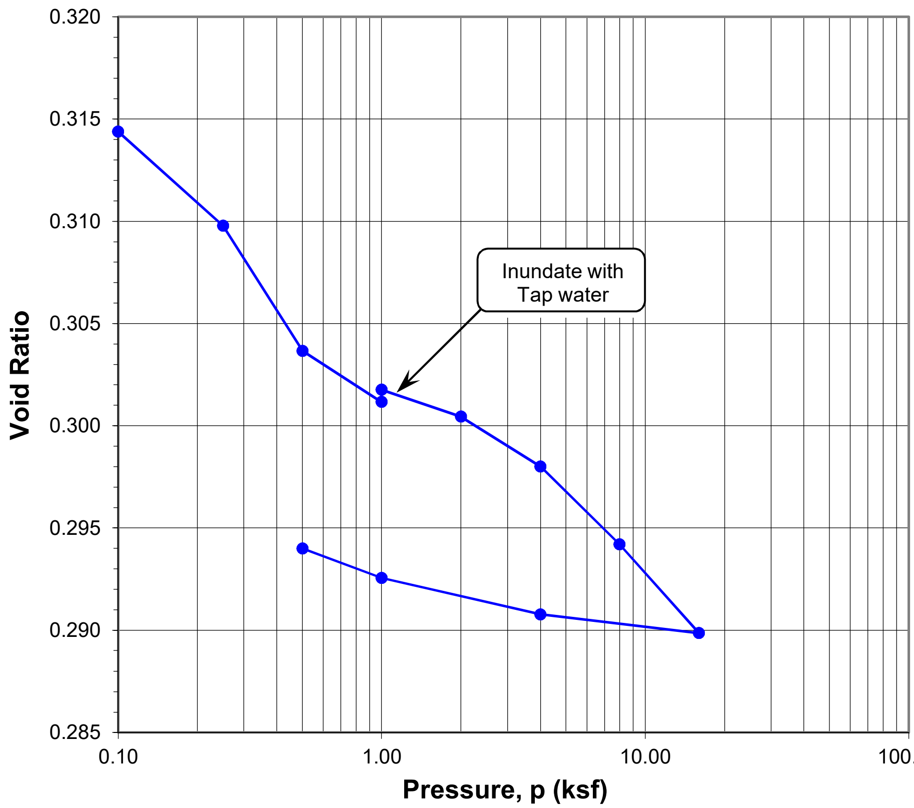
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Lovett Industrial - Evans Road	Tested By: G. Bathala	Date: 06/29/22
Project No.:	22086-01	Checked By: J. Ward	Date: 07/22/22
Boring No.:	HS-5	Depth (ft.): 7.5	
Sample No.:	R-2	Sample Type:	Ring
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	214.14
Weight of Ring (g):	43.42
Height after consol. (in.):	0.9842
Before Test	
Wt. of Wet Sample+Cont. (g):	191.47
Wt. of Dry Sample+Cont. (g):	179.17
Weight of Container (g):	64.76
Initial Moisture Content (%)	10.8
Initial Dry Density (pcf)	128.2
Initial Saturation (%):	92
Initial Vertical Reading (in.)	0.1371
After Test	
Wt. of Wet Sample+Cont. (g):	266.57
Wt. of Dry Sample+Cont. (g):	249.78
Weight of Container (g):	52.53
Final Moisture Content (%)	10.91
Final Dry Density (pcf):	130.0
Final Saturation (%):	99
Final Vertical Reading (in.)	0.1564
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

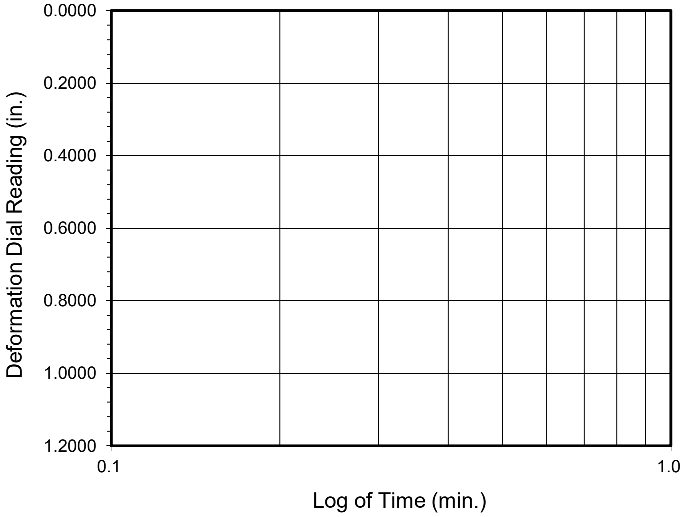


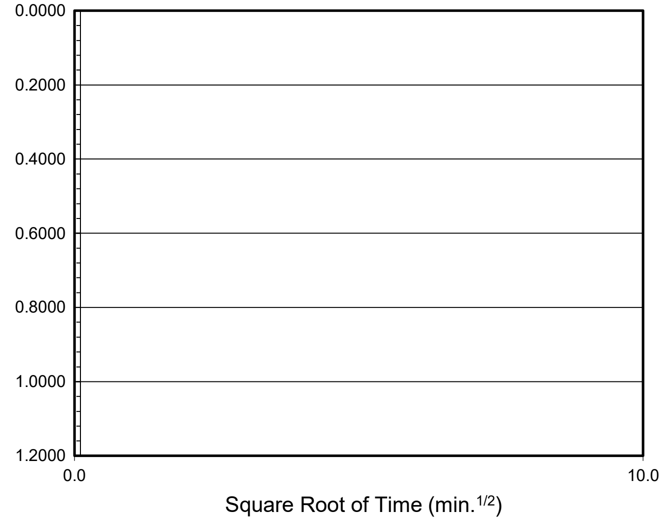
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.1375	0.9997	0.00	0.03	0.314	0.03
0.25	0.1416	0.9956	0.06	0.45	0.310	0.39
0.50	0.1470	0.9901	0.14	0.99	0.304	0.85
1.00	0.1499	0.9872	0.24	1.28	0.301	1.04
1.00	0.1495	0.9877	0.24	1.24	0.302	0.99
2.00	0.1515	0.9857	0.34	1.44	0.300	1.10
4.00	0.1544	0.9827	0.45	1.73	0.298	1.28
8.00	0.1586	0.9785	0.58	2.15	0.294	1.57
16.00	0.1634	0.9737	0.73	2.63	0.290	1.90
4.00	0.1610	0.9761	0.56	2.39	0.291	1.83
1.00	0.1581	0.9791	0.40	2.10	0.293	1.70
0.50	0.1564	0.9808	0.34	1.93	0.294	1.59

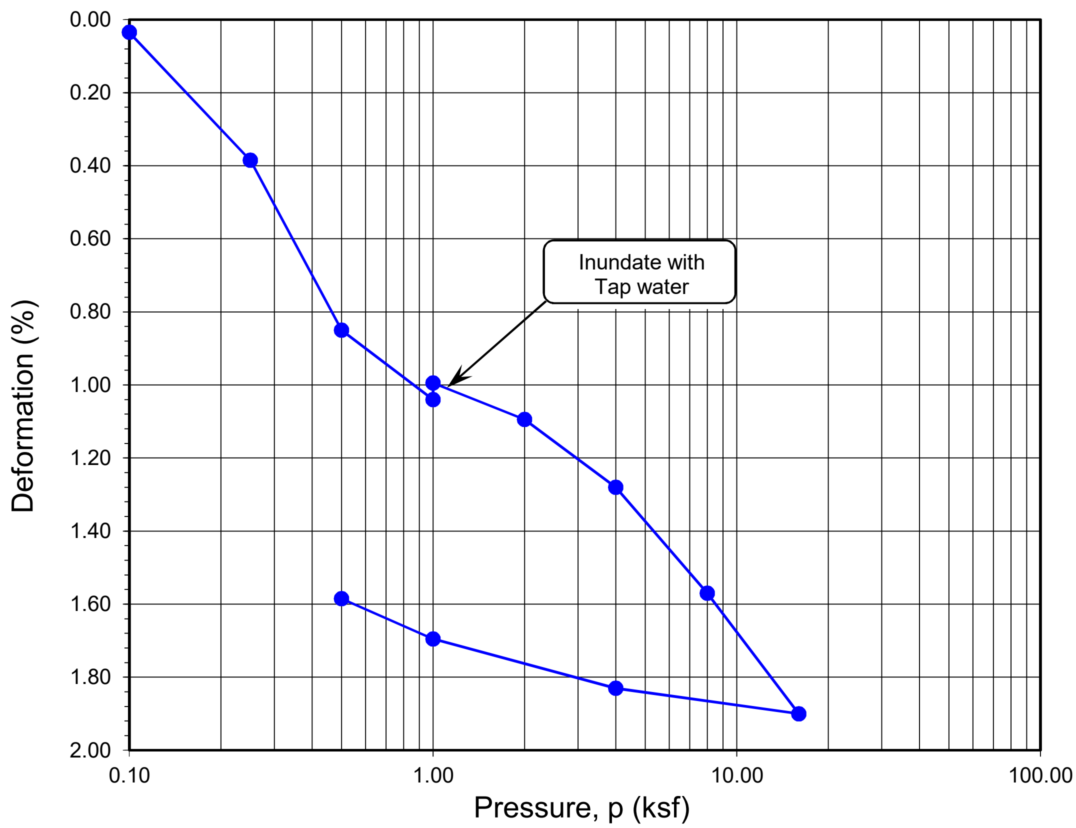
Time Readings

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-5	**R-2**	**7.5**	**10.8**	**10.9**	**128.2**	**130.0**	**0.315**	**0.294**	**92**	**99**

Soil Identification: Dark yellowish brown lean clay with sand (CL)s

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 22086-01

Lovett Industrial - Evans Road

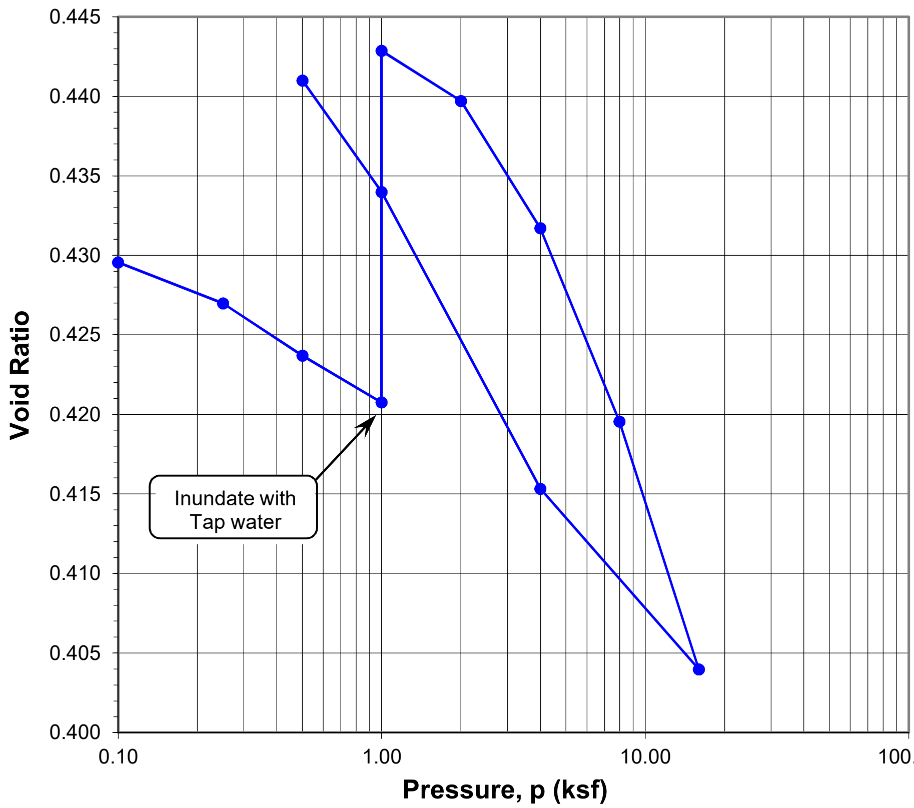
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Lovett Industrial - Evans Road	Tested By: G. Bathala	Date: 06/29/22
Project No.:	22086-01	Checked By: J. Ward	Date: 07/22/22
Boring No.:	HS-6	Depth (ft.): 5.0	
Sample No.:	R-1	Sample Type:	Ring
Soil Identification:	Yellowish brown lean clay (CL)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	209.75
Weight of Ring (g):	45.08
Height after consol. (in.):	1.0077
Before Test	
Wt. of Wet Sample+Cont. (g):	199.83
Wt. of Dry Sample+Cont. (g):	182.85
Weight of Container (g):	58.50
Initial Moisture Content (%)	13.7
Initial Dry Density (pcf)	120.5
Initial Saturation (%):	88
Initial Vertical Reading (in.)	0.0945
After Test	
Wt. of Wet Sample+Cont. (g):	264.84
Wt. of Dry Sample+Cont. (g):	241.21
Weight of Container (g):	52.64
Final Moisture Content (%)	16.47
Final Dry Density (pcf):	118.4
Final Saturation (%):	100
Final Vertical Reading (in.)	0.0896
Specific Gravity (assumed):	2.76
Water Density (pcf):	62.43

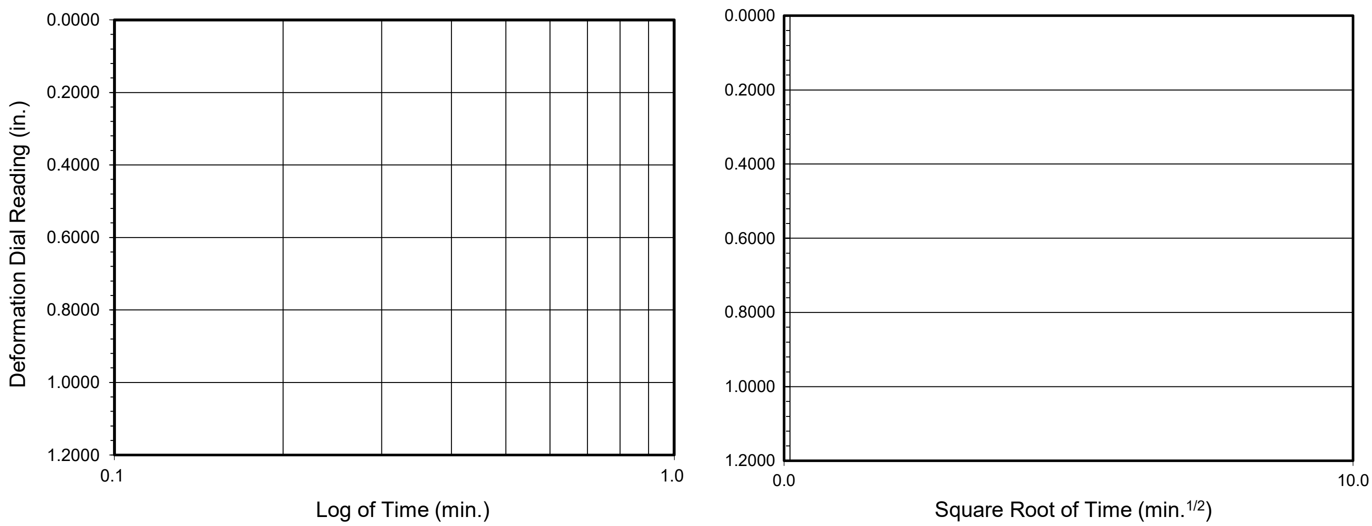


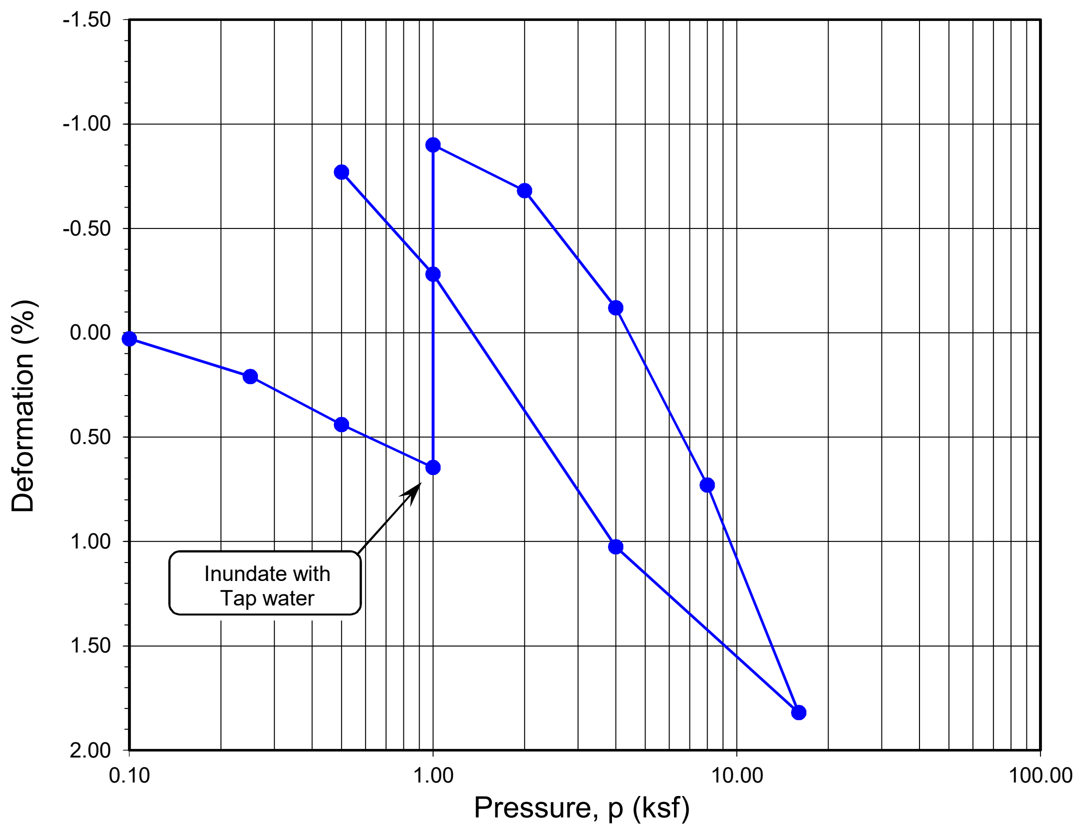
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0948	0.9997	0.00	0.03	0.430	0.03
0.25	0.0970	0.9975	0.04	0.25	0.427	0.21
0.50	0.0998	0.9947	0.09	0.53	0.424	0.44
1.00	0.1026	0.9920	0.16	0.81	0.421	0.65
1.00	0.0871	1.0074	0.16	-0.74	0.443	-0.90
2.00	0.0901	1.0044	0.24	-0.44	0.440	-0.68
4.00	0.0968	0.9977	0.35	0.23	0.432	-0.12
8.00	0.1066	0.9879	0.48	1.21	0.420	0.73
16.00	0.1190	0.9755	0.63	2.45	0.404	1.82
4.00	0.1094	0.9852	0.46	1.49	0.415	1.03
1.00	0.0950	0.9995	0.33	0.05	0.434	-0.28
0.50	0.0896	1.0049	0.28	-0.49	0.441	-0.77

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-6	**R-1**	**5**	**13.7**	**16.5**	**120.5**	**118.4**	**0.430**	**0.441**	**88**	**100**

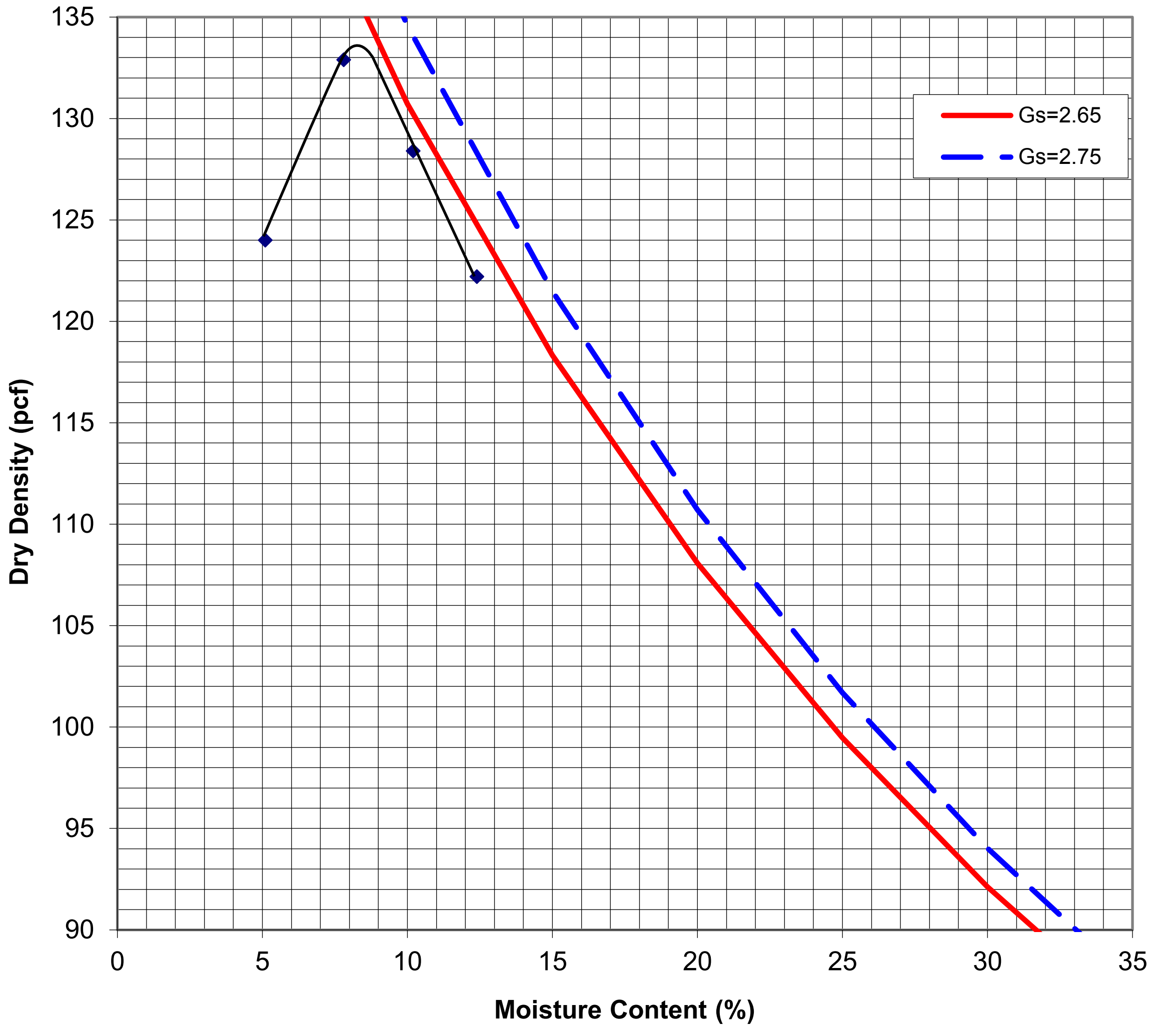
Soil Identification: Yellowish brown lean clay (CL)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 22086-01

Lovett Industrial - Evans Road

07-22



Location:	Sample No.:	Depth (ft)	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-4	**B-1**	**0-5'**	**Light brown silty SAND (SM)**	**133.5**	**8.5**

LGC Geotechnical, Inc.	**LABORATORY COMPACTION** (ASTM D 1557)	**Project Number:** 22086-01 **Date:** Jul-22 **Evans Road**

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	Lovett Industrial - Evans Road	Tested By :	G. Berdy	Date:	06/29/22
Project No. :	22086-01	Checked By:	J. Ward	Date:	07/22/22

Boring No.	HS-1	HS-6		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Brown SM	Brown SM		
Wet Weight of Soil + Container (g)	196.84	169.34		
Dry Weight of Soil + Container (g)	192.09	164.59		
Weight of Container (g)	65.20	58.31		
Moisture Content (%)	3.74	4.47		
Weight of Soaked Soil (g)	100.15	100.15		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	61	94		
Crucible No.	2	3		
Furnace Temperature (°C)	860	860		
Time In / Time Out	8:00/8:45	8:00/8:45		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	28.7116	24.5192		
Wt. of Crucible (g)	28.7085	24.5150		
Wt. of Residue (g) (A)	0.0031	0.0042		
PPM of Sulfate (A) x 41150	127.56	172.83		
PPM of Sulfate, Dry Weight Basis	**133**	**181**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.5	0.8		
PPM of Chloride (C -0.2) * 100 * 30 / B	60	120		
PPM of Chloride, Dry Wt. Basis	**62**	**126**		

pH TEST, DOT California Test 643

pH Value	**7.27**	**7.73**		
Temperature °C	21.0	21.1		

SOIL RESISTIVITY TEST

DOT CA TEST 643

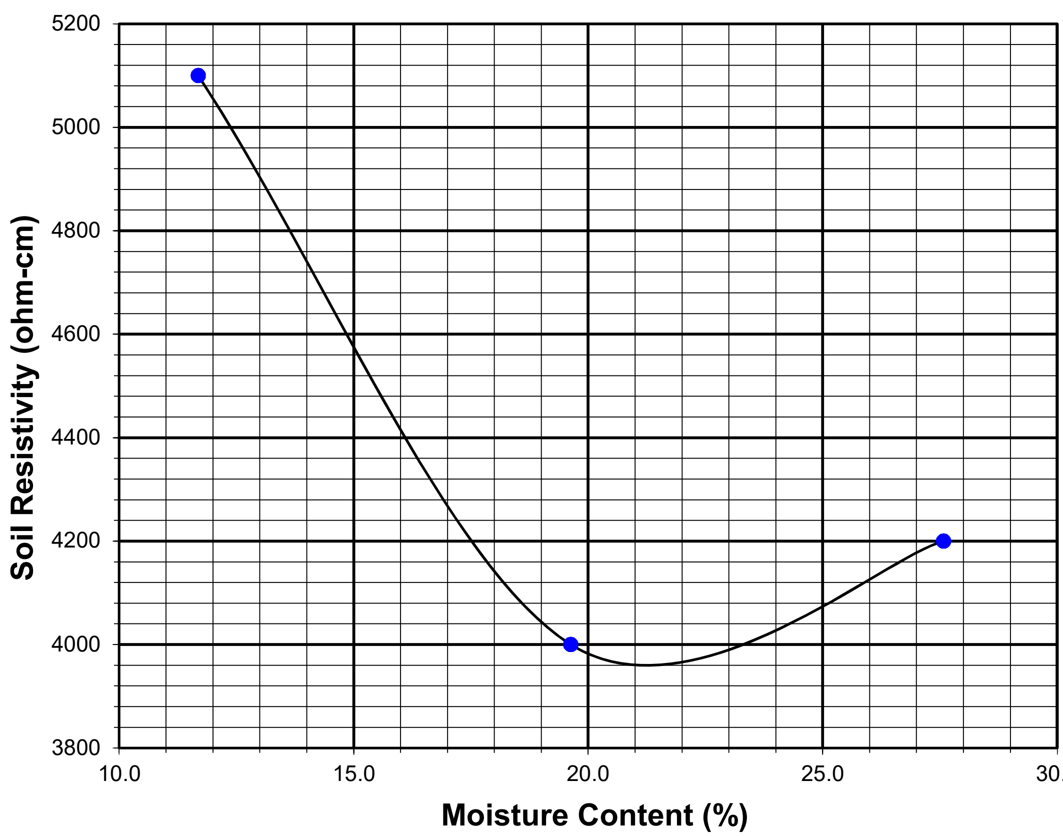
Project Name:	Lovett Industrial - Evans Road	Tested By :	A. Santos Date: 07/07/22
Project No. :	22086-01	Checked By:	J. Ward Date: 07/22/22
Boring No.:	HS-1	Depth (ft.) :	0-5
Sample No. :	B-1		
Soil Identification:*	Brown SM		

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	11.69	5100	5100
2	20	19.63	4000	4000
3	30	27.57	4200	4200
4				
5				

Moisture Content (%) (MCi)	3.74
Wet Wt. of Soil + Cont. (g)	196.84
Dry Wt. of Soil + Cont. (g)	192.09
Wt. of Container (g)	65.20
Container No.	
Initial Soil Wt. (g) (Wt)	130.60
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
3960	**21.2**	**133**	**62**	**7.27**	**21.0**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Lovett Industrial - Evans Road	Tested By :	A. Santos Date: 07/07/22
Project No. :	22086-01	Checked By:	J. Ward Date: 07/22/22
Boring No.:	HS-6	Depth (ft.) :	0-5
Sample No. :	B-1		

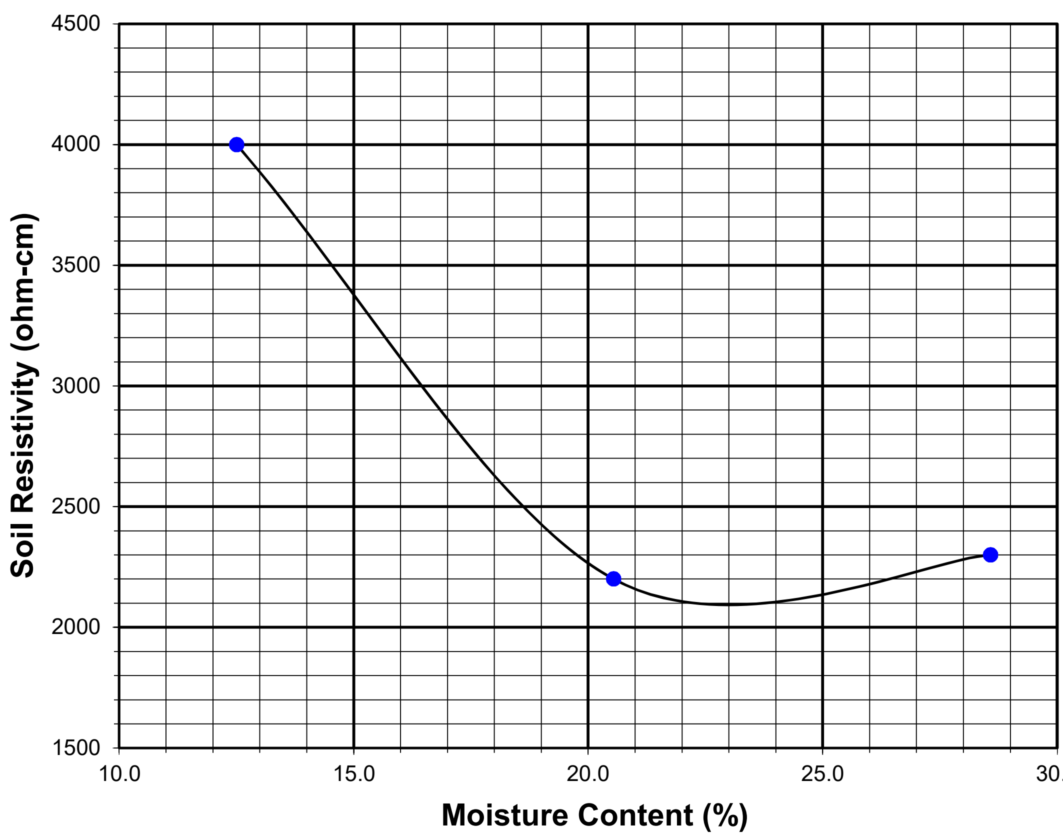
Soil Identification:* Brown SM

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	12.51	4000	4000
2	20	20.54	2200	2200
3	30	28.58	2300	2300
4				
5				

Moisture Content (%) (MCi)	4.47
Wet Wt. of Soil + Cont. (g)	169.34
Dry Wt. of Soil + Cont. (g)	164.59
Wt. of Container (g)	58.31
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
2100	**23.0**	**181**	**126**	**7.73**	**21.1**



Appendix D
General Earthwork and Grading Specifications

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 basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the 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). 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 over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations 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 over-excavated 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 over-excavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 *Import*

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

4.0 *Fill Placement and Compaction*

4.1 *Fill Layers*

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

4.2 *Fill Moisture Conditioning*

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

4.3 *Compaction of Fill*

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

4.4 *Compaction of Fill Slopes*

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

4.5 *Compaction Testing*

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 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.

LGC Geotechnical, Inc.

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

Sheet 1
Geotechnical Map

CLIENT:
Lovett Industrial
1730 East Holly Avenue, #808
El Segundo, California 90245

CIVIL ENGINEER:
Thienes Engineering, Inc.
14349 Firestone Blvd.
La Mirada, CA 90638

PROJECT NAME	Lovett Industrial - Evans Road
PROJECT NO.	22086-01
ENG. / GEOL.	BTZ / KBC
SCALE	1" = 40'
DATE	July 2022

SHEET
1 of 1