LGC Geotechnical, Inc.

March 14, 2022 Project No. 21184-01

Mr. Ricardo Rivas
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11150 Santa Monica Boulevard, Suite 700
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Subject: ***Preliminary Geotechnical Evaluation for Proposed Self-Storage Redevelopment, 630 North Batavia Street, Orange, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed redevelopment of the property located at 630 North Batavia Street in Orange, 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,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

LGC Geotechnical has performed a geotechnical evaluation for the proposed self-storage buildings to be located at 630 North Batavia Street in Orange, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical design recommendations relative to the project.

1.1 Project Description

The location of the proposed self-storage facility is currently developed with two (2) existing one- and two-story industrial buildings and associated asphalt and concrete pavement of the Roseburrough Tool Company that are proposed to be demolished and replaced by two (2) one- and two-story self-storage buildings (see Figure 2 – Geotechnical Map). The existing topography at the site and surrounding area is nearly level, with site drainage via sheet flow toward the northwest corner of the site. The land adjacent to the north side and east side of the site is developed with railroad tracks and North Batavia Street, respectively. The land beyond the railroad tracks and street is developed with warehouse/industrial buildings. The land adjacent to the west of the site is located at 619 North Main Street and is developed with warehouse buildings with asphalt and concrete pavements, and two small structures that appear to be storage sheds that are located on the site's western property line. The land adjacent to the south of the site is developed with what appears to be four single-story retail/light industrial buildings with masonry construction along with asphalt and concrete pavement. The two buildings closest to the site's southern property line are located at 600 and 610 North Batavia Street. The building at 610 North Batavia Steet is located approximately 10 feet from the property line with asphalt pavement between the building and the property line, while the exterior wall of the building at 600 North Batavia Street appears to be located on the site's southern property line.

We understand that the proposed redevelopment of the site will include demolition of the existing buildings and improvements on the site for construction of at-grade, one-story, self-storage buildings around the perimeter of the site and an at-grade, two-story, self-storage building in the middle of the site. Parking and drive isles will be located between the interior and perimeter structures.

Preliminary building (dead plus live) loads were not provided at the time of this report. However, we have estimated the maximum wall and column (dead plus live) structural loads at 4 kips per lineal foot and 150 kips, respectively. Based on the preliminary grading plan proposed grades will not change significantly from existing grades.

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*

Our subsurface evaluation consisted of the excavation of four hollow-stem auger borings. The borings (HS-1 through HS-4) were excavated using a truck-mounted drill rig equipped with 6-inch-diameter hollow-stem augers with depths ranging from approximately 25 to 50 feet below existing grade. Infiltration borings (I-1 & I-2) were excavated to 5 feet below existing grade, east of the proposed building location. 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. Bulk samples were also collected and logged for laboratory testing at select depths. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. The borings were backfilled with cuttings.

The approximate locations of our subsurface explorations are provided on Figure 2. The boring logs are provided in Appendix B.

1.3 *Field Infiltration Testing*

Two field infiltration tests were performed in Borings I-1 and I-2 to an approximate depth of 5 feet below existing grade. The approximate location is shown on the Geotechnical Map (Figure 2). The borings for the infiltration tests were excavated using a drill rig equipped with 8-inch diameter hollow-stem augers. Estimation of the infiltration rate was accomplished in general accordance with the guidelines set forth by the County of Orange (2017). A 3-inch diameter perforated PVC pipe was placed in the borehole and the annulus was backfilled with gravel. The infiltration wells were pre-soaked prior to testing. At the completion of infiltration testing, the pipe was removed and backfilled with cuttings and tamped. Some settlement of the backfill should be expected.

In general, three-dimensional flow out of the test well (percolation), as observed in the field, is mathematically corrected to one-dimensional flow out of the bottom of the test well (infiltration). Infiltration testing was performed using relatively clean water, free of particulates, silt, etc. The results are presented in Appendix B and summarized below.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test No.	Approx. Depth Below Existing Grade (ft)	Observed Infiltration Rate* (in./hr.)	Measured Infiltration Rate** (in./hr.)
I-1	5	0.2	0.1
I-2	5	0.1	0.1

*Observed Infiltration Rates Do Not Include Factor of Safety.
**Measured Infiltration Rates Include a Factor of Safety of 2 in Order to Evaluate Feasibility.

The tested infiltration rates provided in this report are considered a general representation of the infiltration rates at the location of the proposed infiltration trench. Please note, the testing of infiltration rates is highly dependent upon the materials encountered at the point of testing (i.e., location and depth of testing). Varying subsurface conditions may exist outside of the test location which could alter the calculated infiltration rate. Please refer to Section 4.9.

1.4 <u>Laboratory Testing</u>

Representative bulk and driven samples were obtained for laboratory testing during our field evaluation. Laboratory testing included in-situ density and moisture content, Atterberg limits, expansion index, consolidation, collapse, R-value, grain size analysis for fines content, and corrosion (sulfate, chloride, pH and minimum resistivity). A summary of the laboratory test results is presented in Appendix C.

- Dry density of the samples collected ranged from approximately 96 pounds per cubic foot (pcf) to 128 pcf. Moisture contents ranged from approximately 1 percent to 24 percent.
- Atterberg Limit testing indicates that the Plasticity Index (PI) of the tested soils ranges from 6 to 10 and the soils are classified as low plasticity silts and clays.
- Two Expansion Index (EI) tests were performed, and the results indicated EI values ranging from 20 to 37, which are classified as having “Very Low” to “Low” expansion potential.
- Consolidation testing was performed on three samples. The plots are provided in Appendix C.
- One collapse-swell test was performed. The soil was found to have a swell of 0.03 percent. The result is provided in Appendix C.
- R-value testing was performed on one sample, the results indicate an R-value of 10.
- Grain size analysis for fines content (percent of particles by dry weight passing the #200 sieve) was performed on two samples. The fines content was found to range from 71 percent to 85 percent.
- Corrosion testing indicated a soluble sulfate content of approximately 0.067 percent (67 ppm), a chloride content of 20 parts per million (ppm), pH of 8.13, and a minimum resistivity of 2,590 ohm-centimeters. Based on Caltrans specifications, the soils are considered not corrosive.

Laboratory test results obtained from our field evaluation are provided in Appendix C.

2.0 GEOTECHNICAL CONDITIONS

2.1 Geologic Conditions

The site is located within the Peninsular Ranges Geomorphic Province, within the eastern boundary of the Los Angeles Sedimentary Basin. The Los Angeles Sedimentary Basin is a northwest-plunging synclinal sedimentary deposit that is bounded to the south of the subject site by the broadly uplifted costal mesa of Newport Beach. A channelized portion of the Santa Ana River passes approximately 0.75-miles to the west of the site. The river deposited widely dispersed sheet deposits prior to channelization.

2.2 Site-Specific Geology

The site is underlain by deposits of Quaternary-aged Old Fan Deposits (Morton & Miller, 2006). Where encountered, the upper approximately 10 feet of the alluvial fan soil was found to consist mostly of sandy silt to sandy clay, with lesser amounts of silty clay, and scattered silty sand. The soils in the approximately 10 feet were found to be moist and medium dense or medium stiff to very stiff in-place. Scattered roots were observed at a depth of approximately 7.5 feet in boring HS-2. Cobble-gravel-sand mixtures with lesser amounts of silt were encountered at depths between approximately 15 and 25 feet below the ground surface. These soils were found to be slightly moist to moist and medium dense to very dense in-place. Below depths of approximately 30 feet below the ground surface the encountered soils consisted of interbeds of silty sand, sandy silt, and silty clay, with scattered gravelly sands that were found to be slightly moist to wet, and medium dense to very dense or medium stiff to hard in-place. The approximate lateral extent of the earth units is presented on the Geotechnical Map (Figure 2), and the soils are described in the boring logs in Appendix B.

2.3 Geologic Structure

Geologic structure was not identified in the subject site geotechnical evaluation. The alluvial materials encountered are generally massive, but may include low angle bedding, typically dipping in a westerly direction.

2.4 Landslides and Rockfalls

The site and surrounding areas are nearly level, without any significant slopes. Therefore, due to the low topographic relief, the likelihood of landslides or rockfalls impacting the site is nil.

2.5 Groundwater

Groundwater was not encountered to the maximum depth explored (50 feet below the ground surface) during advancement of the deep borings at the subject site. The site is located approximately 0.75 miles east of the Santa Ana River, and the site is situated at an elevation approximately 50 feet higher than the riverbed. Based on information obtained from the

California Department of Water Resources, Water Data Library (DWR, 2021), there is a groundwater monitoring well located on the west side of the Santa Ana River, approximately 0.9 miles west of the site. The State Well number for the nearby well is 04S10W25G001S, and it is monitored by the Orange County Water District, where it is known locally as SAR-3/MP1. The well has been monitored from August of 1988, through October of 2021. The ground surface at the monitoring well is approximately 10 feet lower in elevation than the ground surface at the site. During the monitoring period the shallowest groundwater was detected approximately 58 feet below the ground surface at the monitoring location, while the deepest groundwater was detected at approximately 105 feet below the ground surface. The most recent monitoring information, from October of 2021, indicates that groundwater was at approximately 75 feet below the ground surface.

Groundwater and/or groundwater seepage conditions may occur in the future due to changes in land use and/or following periods of heavy rain. 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 landscape irrigation or precipitation especially during rainy seasons.

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 an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are 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, and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. A discussion of these secondary effects is provided in the following sections.

2.6.1 __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 rupture due to active faulting is not likely to occur onsite due to the absence of known active fault traces. 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 (Bray & Sancio, 2006). 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 is not located within a State of California Seismic Hazard Zone (CGS 2021) for liquefaction potential. Due to a lack of shallow groundwater (greater than 50 ft below ground surface); the site is not considered susceptible to liquefaction.

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

The site is located approximately 160 feet above sea level and is approximately 12.5 miles from the coast. Based on the elevation of the site, and the distance to the shore, there is a very low possibility of damage to the site during a large tsunami event.

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 33.7976 degrees north and longitude -117.8628 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 2. 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.

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.65 at a distance of approximately 13.67 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 6.6 at a distance of approximately 19.4 km from the site would contribute the most to this ground motion (USGS, 2008).

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 0.638g (SEAOC, 2021).

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)	1.382g	From SEAOC, 2021
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.491g	From SEAOC, 2021
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))	1.809	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.382	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	0.888g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.922g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.592g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.927	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.924	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 $T \le 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \ge T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

2.8 Rippability

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

2.9 Oversized Material

Generation of a surplus of oversized material (material greater than 8 inches in maximum dimension) is generally not anticipated during site grading. However, some oversized material may be encountered, which may result in excavation difficulty for narrow excavations. 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.

2.10 Expansive Soil Characteristics

Expansion Index (EI) test results indicate EI values range from 20 to 37 which are classified as exhibiting “Very Low” to “Low” expansion potential.

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 indicates that the site contains medium dense to dense, clayey and silty sands and very stiff to hard sandy silts and sandy clays to the maximum explored depth of approximately 50 feet below existing grade. The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- From a geotechnical perspective, the existing onsite soils are suitable material for use as general fill, provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- A static groundwater table was not encountered to the maximum explored depth of approximately 50 feet below existing ground surface. Historic high groundwater is estimated at approximately 58 feet or greater below existing grade.
- Based on the proposed layout, remedial grading will be required adjacent to property lines and existing buildings in portions of the site. Earthwork techniques such as slot cuts and/or temporary shoring will be required.
- The proposed development will likely be subjected to strong seismic ground shaking during its design life from one of the regional faults. The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation.
- The site is not located in a seismic hazard zone for liquefaction potential. Site soils are not considered susceptible to liquefaction due to lack of a groundwater table in the upper 50 feet.
- Soils exposed at the proposed foundation level are anticipated to have a “Low” expansion potential (Expansion Index not exceeding 50). This shall be confirmed at the completion of site earthwork.
- Excavation for foundations and underground improvements should be achievable with the appropriate equipment.
- The field percolation tests resulted in measured infiltration rates of approximately 0.1 inch per hour. These infiltration rates are based on feasibility factor of safety 2.0. Refer to Section 4.9.
- 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 City. 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, utility line construction and backfill, and construction of parking/driveway areas. We recommend that earthwork onsite be performed in accordance with the following recommendations, 2019 CBC/ City of Orange guidelines 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, engineered structures or improvements should be demolished and the area should be cleared of existing vegetation (shrubs, trees, grass, etc.), surface obstructions, existing debris and potentially compressible or otherwise unsuitable material. 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 cesspools or 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, cesspools 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, we recommend that removals 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.

Footings Adjacent to Property Lines and/or Existing Structures: Where extending the removals 5 feet beyond the proposed building is not possible due to constrains such as property lines and/or existing buildings, subsequent to the 5-foot vertical removal, the excavation along the property line edge of the proposed building may be backfilled to proposed bottom of footing with sand cement slurry or with the onsite fill materials, recompacted to at least 95 percent (instead of a minimum of 90 percent) relative compaction at near-optimum moisture content (per ASTM D1557) to the bottom of proposed footing. This zone is defined as the edge of the proposed building and extending a minimum of 5 horizontal feet (or width of the proposed footing if greater than 5 feet) from the property line into the building pad. Refer to Figure 4.

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 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 engineer 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.

Existing, off-site improvements and building structures are present adjacent to portions of the site property lines. 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.

The potential for impacting the existing improvements and adjacent properties may be reduced by performing excavations within 5 lateral feet of the existing off-site improvements using narrow "A-B-C" slot cuts. "A-B-C" slot cuts are defined as excavations perpendicular to sensitive property boundaries that are divided into multiple "slots" of equal width. If slots are labeled A, B, C, A, B, C, etc., then "A" slots should be excavated at the same time but must be backfilled before "B" slots can be excavated, etc. Slot cuts should be no wider than 12 feet and no deeper than 5 feet. Where proposed excavations are adjacent to adjacent building structures (and within 5 horizontal feet), slot cuts should be no wider than 5 feet and no deeper than 5 feet. Slot cuts should be backfilled immediately with properly placed compacted fill (per Section 4.1.6) or cement slurry to finish grade prior to excavation of adjacent slots. Due to the presence of sands at the site which are susceptible to caving, narrower slot cuts may be required. This should be further evaluated during grading. Protection of the existing improvements during grading is the responsibility of the contractor.

4.1.4 Removal Bottoms and Subgrade Preparation

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 Material for Fill

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, therefore select grading and stockpiling and/or import will be required by the contractor for obtaining 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.

The placement of concrete or masonry demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 1-inch in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned street areas (i.e., not within building pad areas).

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). Moisture conditioning of site soils should be anticipated in order to achieve the required degree of 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 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.

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 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 <u>Trench and Retaining Wall Backfill and Compaction</u>

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 4). 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 to 10 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 Recommendations

Site soils are anticipated to be of 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.

Please note that building structures are proposed adjacent to existing building structures in portions of the site. Deepened and/or widening of footings may be prudent in these areas in order to reduce the surcharge on the adjacent existing structure.

4.2.1 Preliminary Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2019 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 25
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer.
- Moisture condition subgrade soils to 100 % of optimum moisture content to a depth of 12 inches prior to trenching for footings.

4.2.2 Provisional Post-Tensioned Foundation Design Parameters

The geotechnical parameters provided herein may be used for post-tensioned slab foundations with a deepened perimeter footing or a post-tensioned mat slab. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2019 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method.

Our design parameters are based on our experience with similar projects, test results onsite, and the anticipated nature of the soil (with respect to expansion potential). Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners and/or property maintenance personnel not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

TABLE 3

Preliminary Post-Tensioned Foundation Design Parameters

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	Low[1]	Low[1]
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	9.0 feet 0.25 inch	9.0 feet 0.30 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	5.5 feet 0.55 inch	5.5 feet 0.66 inch

1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Moisture condition to 100 % of optimum moisture content to a depth of 12 inches prior to trenching.

4.2.3 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. The builder should provide these recommendations to future owners and property management personnel.

4.2.4 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.

4.3 *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 4

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

*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 0.35 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 wall structures 10 feet or less in height. 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 5 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 5

Lateral Earth Pressures – Sandy Backfill

Conditions	**Equivalent Fluid Unit Weight (pcf)**
	Level Backfill
	Approved Granular 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, refer to Figure 3. 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 retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, basement/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 0.5 and 0.30 may be used for at-rest and active conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The structural designer should contact the geotechnical engineer 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. This increment should be applied in addition to the provided static lateral earth pressure using a "normal" 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. This seismic lateral earth pressure is 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 6 below. An R-Value of 10 was utilized for preliminary calculations. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Determination of the Traffic Index (TI) is not the purview of the geotechnical consultant. Final pavement sections should be confirmed by the project civil/transportation engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 6

Paving Section Options

Pavement Area	**Assumed Traffic Index***	**Section Thickness (inches)**	
		Asphalt Concrete	**Aggregate Base**
Auto Parking	4.5	4.0	5.5
Circulation Drives (little to no truck traffic)	5.0	4.0	7.5
Truck Driveways	6.0	4.0	11.0

*Determination of the Traffic Index is not the purview of the geotechnical consultant

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 20, 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 <u>minimum</u> 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 a soluble sulfate content of approximately 0.067 percent, a chloride content of 20 parts per million (ppm), pH of 8.13, and a minimum resistivity of 2,590 ohm-centimeters. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the sulfate concentration is 2,000 ppm (0.2 percent) or greater, the chloride concentration is 500 ppm or greater, the pH is 5.5 or less, or the minimum resistivity is equal to or less than 1,500 om-cm.

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 (ACI, 2014). 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 7 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement and construction 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 construction joints are carried up through the tile from the concrete. The concrete flatwork will move over time, the architect and builder must make provisions for this movement in both design and construction.

TABLE 7

Nonstructural Concrete Flatwork

	Flatwork	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 inches	City/Agency Standard
Presoak	Wet down prior to placing	City/Agency Standard
Minimum Reinforcement	No. 3 rebar at 24 inches on centers	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

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 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.

If it is determined that water must be infiltrated due to regulatory requirements, we recommend the absolute minimum amount of water be infiltrated and that the infiltration areas not be located near settlement-sensitive existing /proposed improvements, retaining wall structures, property lines, or any slopes. We recommend the design of any infiltration system include at least one redundancy or overflow system. It may be prudent to provide an overflow system connected directly to a storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration with time and/or very high flow volumes.

As with all systems that are designed to concentrate surface flow and direct the water into the subsurface soils, some minor settlement, nuisance type localized saturation and/or other water related issues should be expected. Due to variability in geologic and hydraulic conductivity characteristics, these effects may be experienced at the onsite location and/or potentially at other locations beyond the physical limits of the subject site. Infiltrated water may enter underground utility pipe zones or flow along heterogeneous soil layers or geologic structure and migrate laterally impacting other improvements which may be located far away or at an elevation much lower than the infiltration source.

Adequate distances should be maintained between infiltration locations and structures. The invert of any storm water infiltration system should be set back a minimum of 15 feet from building structures and outside a 1:1 plane drawn up from the bottom of adjacent foundations.

Observed infiltration rates (no factor of safety) of 0.1 and 0.2 inches per hour were obtained from field infiltration testing. The design infiltration rate is determined by dividing the observed infiltration rate by a series of safety factors for site suitability and design considerations that are the purview of both the geotechnical consultant and designer of the infiltration system. The following geotechnical factors of safety provided in Table 8 can be used to determine any required design infiltration rate.

TABLE 8

Geotechnical Factors of Safety for Design Infiltration Rate

Geotechnical Reduction Factors	
Consideration	**F.S.**
Soil Assessment Methods (RF_t)	2
Site Variability (RF_v)	1
Long-term Siltation & Maintenance (RF_s)	Per Infiltration Designer
Calculated Design F.S.	Per Infiltration Designer
Combined F.S.= RF_t x RF_t x RF_s	**TBD**

These values are for native materials only and are not to be utilized for compacted fill. Infiltration shall not be permitted directly on or into compacted fill soils. The infiltration values provided are based on clean water and this requires the removal of trash, debris, soil particles, etc., and on-going maintenance. Over time, siltation, plugging, and clogging of the system may reduce the infiltration rate and subsequently reduce the effectiveness of the infiltration system. It should be noted that methods to prevent this shall be the sole responsibility of the infiltration designer and are not the purview of the geotechnical consultant. If adequate measures cannot be incorporated into the design and maintenance of the system, then the infiltration rates may need to be further reduced. These and other factors should be considered in selecting a design infiltration rate.

4.10 Pre-Construction Documentation and Construction Monitoring

It is recommended that 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. Subsequent readings should be scheduled consistent with the program of work.

4.11 Geotechnical Plan Review

Grading and foundation 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 layout.

4.12 Footing/Foundation Excavations

Footing/foundation excavation bottoms should be firm, relatively unyielding, and free of loose material. Footing/foundation excavations should be observed and accepted by the geotechnical consultant prior to placement of steel reinforcement.

Because of the sandy nature of some of the on-site soils, the materials at the base of foundations may become loosened and disturbed after excavating and subsequently drying out. It may be required immediately prior to placing reinforcing steel, the base of foundations be moistened and compacted.

4.13 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 utility trench and retaining wall backfill and compaction;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation 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.

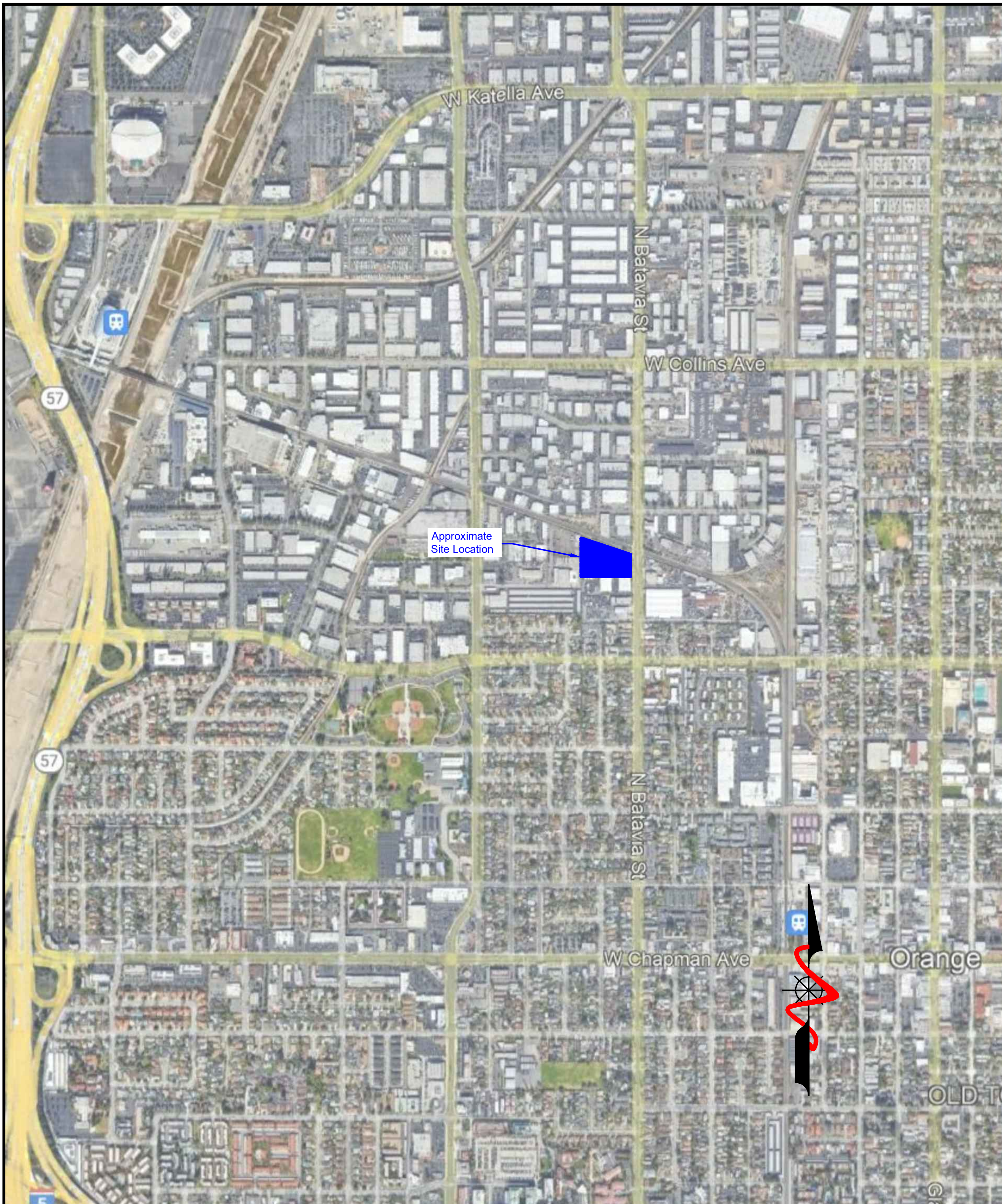


FIGURE 1
Site Location Map

LGC Geotechnical, Inc.

PROJECT NAME	SCIND Batavia Point, LLC
PROJECT NO.	21184-01
ENG. / GEOL.	KMS/KBC
SCALE	Not to Scale
DATE	March 2022

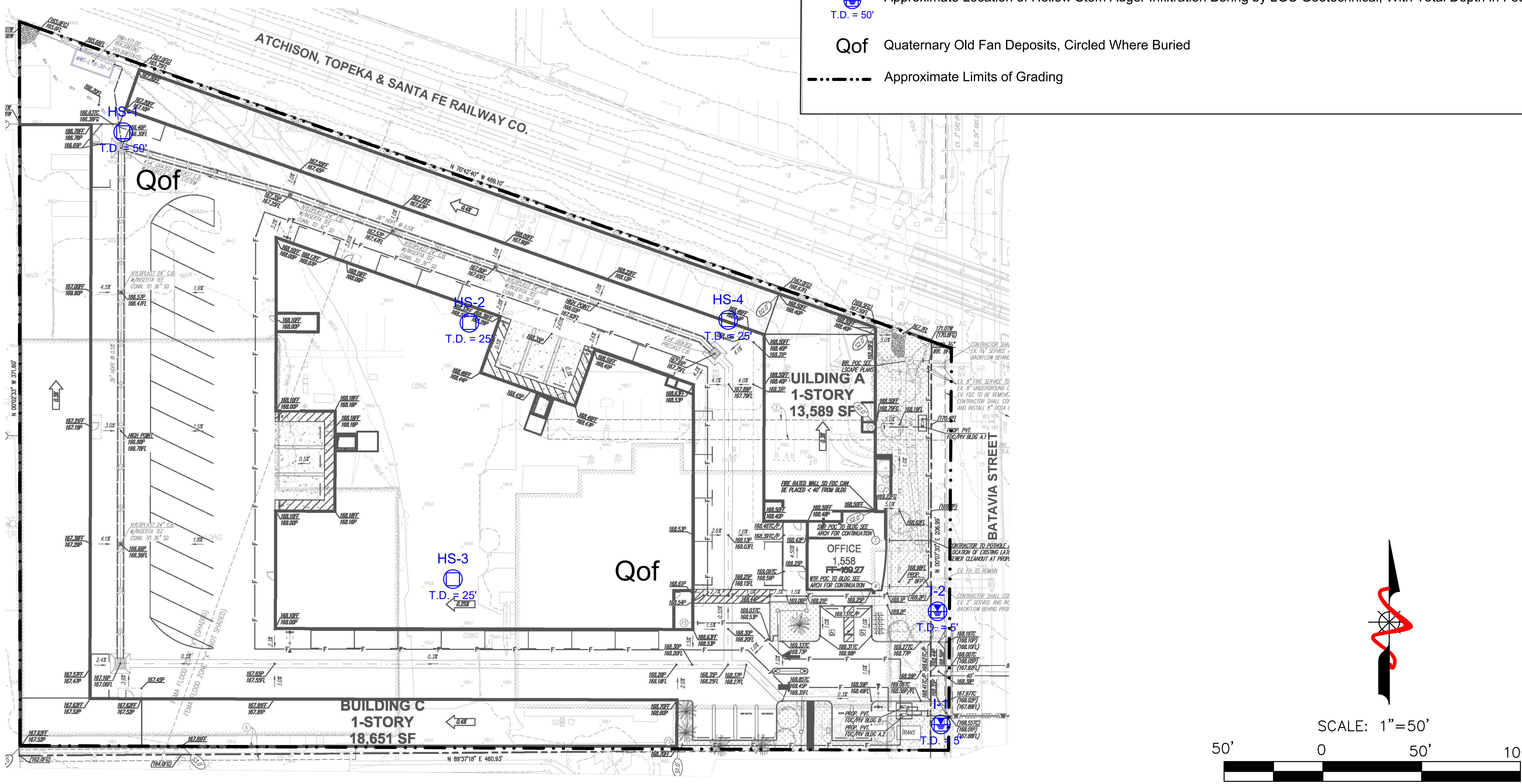
LEGEND

HS-1 T.D. = 50' — Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet

I-1 T.D. = 50' — Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet

Qof — Quaternary Old Fan Deposits, Circled Where Buried

Approximate Limits of Grading



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

FIGURE 2
Geotechnical Map

PROJECT NAME	SCIND Batavia Point, LLC
PROJECT NO.	21184-01
ENG. / GEOL.	KMS/KBC
SCALE	1" = 50'
DATE	March 2022

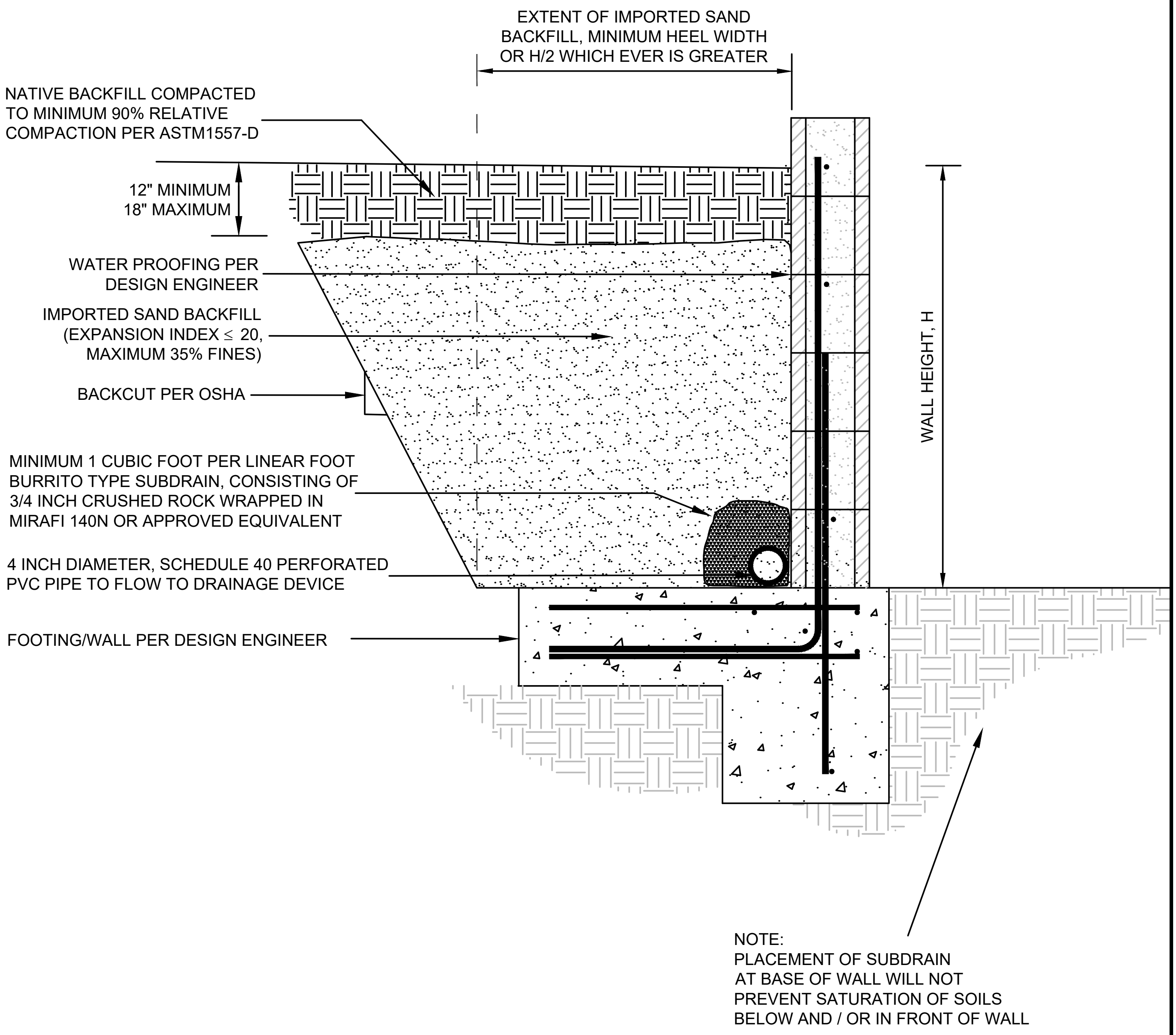


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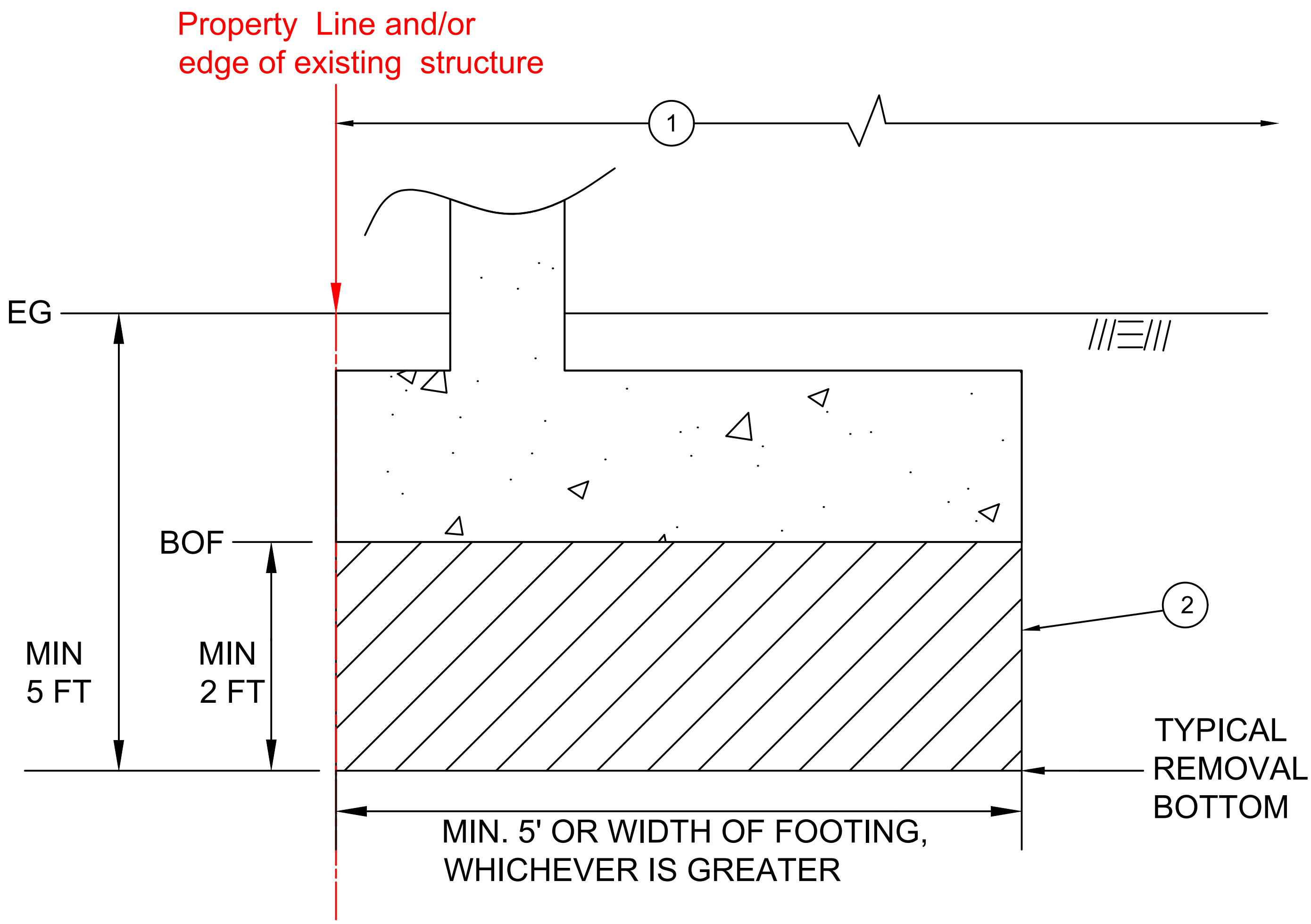
FIGURE 3

Retaining Wall Backfill and Subdrain Detail Level Backfill Approved Import Sand

PROJECT NAME	SCIND Batavia Point, LLC
PROJECT NO.	21184-01
ENG. / GEOL.	KMS/KBC
SCALE	Not to Scale
DATE	March 2022

EARTHWORK REMOVALS FOR FOOTINGS ADJACENT TO PROPERTY LINES AND/OR EXISTING STRUCTURES



1. EXCAVATIONS 4' OR DEEPER WITHIN 5' HORIZONTAL FEET OF PROPERTY LINE AND/OR EXISTING BUILDING STRUCTURES SHOULD BE MADE IN "A-B-C" SLOT CUTS OR TEMPORARY SHORING SHOULD BE USED

2. REPLACE REMOVED UNSUITABLE SOIL BELOW PROPOSED FOOTINGS WITH SAND CEMENT SLURRY OF SOIL COMPACTED TO A MINIMUM OF 95% RELATIVE COMPACTION

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FIGURE 4

PROJECT NAME	*SCIND Batavia Point, LLC*
PROJECT NO.	*21184-01*
ENG / GEO	*KMS/KBC*
SCALE	*Not to Scale*
DATE	*March 2022*

Appendix A
References

APPENDIX A

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Appendix B
Boring Logs and Field Infiltration Data

Geotechnical Boring Log Borehole LGC-HS-1

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~159' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B						@ 0' - 6" of concrete pavement	
			R-1	7 10 12	118.5	14.1	CL-ML	@ 2.5' - Sandy SILT to Silty CLAY: brown, moist, very stiff	
155	5		SPT-1	2 4 4		14.8		@ 5' - Sandy SILT to Silty CLAY: brown, moist, medium stiff	
			R-2	5 8 8	116.7	13.9		@ 7.5' - Silty CLAY: yellowish brown, moist, very stiff	CN
150	10		SPT-2	2 3 4		16.0		@ 10' - Sandy SILT to Silty CLAY: moist to very moist, medium stiff, low plasticity	
145	15		R-3	8 20 22		3.5	GM	@ 15' - Silty Gravel: grayish brown, slightly moist, medium dense, sample disturbed	
140	20		SPT-3	20 26 24		3.3	SP	@ 20' - SAND w/ Fine to Coarse Gravel: brown & gray, slightly moist, dense to very dense	
135	25		R-4	14 50/6"	120.4	1.3	GP	@ 25' - SAND w/ Fine to Coarse Gravel: brown & gray, slightly moist, very dense	
130	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

Geotechnical Boring Log Borehole LGC-HS-1

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~159' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	5 11 5		24.3	CL	@ 30' - Silty CLAY w/ trace Gravel: brown, moist to very moist, very stiff, low plasticity	
125	35		R-5	12 14 13	103.8	22.1		@ 35' - Lean CLAY w/ trace Gravel: yellowish brown to brown, moist to very moist, stiff	AL
120	40		SPT-5	3 4 4		23.2		@ 40' - Silty CLAY w/ trace of Gravel: brown, moist to very moist, medium stiff, low plasticity	
115	45		R-6	21 50/3"	109.9	4.6	SM	@ 45' - Silty SAND w/ Gravel: gray to brown, slightly moist to moist, very dense	
110	50		SPT-6	11 50/6"		4.5		@ 50' - SAND: gray to brown, slightly moist to moist, medium dense to dense	
105	55							Total Depth = 51' Groundwater Not Encountered Backfilled with Cuttings on 10/26/2021	
100	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 LGC-HS-2

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~161' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
160	0							@ 0' - 10" Asphalt, two layers of asphalt	
			R-1	7 8 9	107.4	10.6	CL	@ 2.5' - Silty CLAY: brown, slightly moist, stiff, low plasticity	
155	5		SPT-1	6 5 5		11.6	CL-ML	@ 5' - Sandy SILT to Sandy CLAY: dusky brown, slightly moist to moist, stiff, low plasticity	-#200
			R-2	5 9 9	109.1	7.8		@ 7.5' - Silty CLAY: light brown to brown, slightly moist to moist, loose, scattered roots	CO
150	10		SPT-2	2 3 4		12.8		@ 10' - Sandy SILT to Sandy CLAY: dusky brown, slightly moist to moist, medium stiff	-#200
145	15		R-3	14 37 50/4"		1.0	GP	@ 15' - Poorly Graded Gravel: gray slightly moist, very dense, sample disturbed	
140	20		SPT-3	20 50/3"		1.7		@ 20' - Poorly Graded Gravel w/ Silty SAND: dusky gray, slightly moist, very dense	
135	25		R-4	17 50/6"		1.6	SP	@ 25' - Poorly Graded SAND w/ Gravel: gray to light brown, slightly moist, very dense	
	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

Geotechnical Boring Log Borehole LGC-HS-2

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~161' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
130	30		SPT-4	5 3 5		9.6	CL-ML	@ 30' - Silty CLAY w/ Fine Gravel: yellowish brown, slightly moist to moist, medium stiff	AL
125	35		R-5	7 9 12	96.4	19.3	CL	@ 35' - Silty CLAY: brown, moist, stiff, low plasticity	
120	40		SPT-5	2 5 27		10.1	CL-ML	@ 40' - Sandy SILT to Sandy CLAY: brown to dark brown, moist, hard, low plasticity	
115	45		R-6	20 50/3"	128.5	3.5	GP	@ 45' - Poorly Graded Sandy Gravel: reddish brown, slightly moist, very dense	
110	50		SPT-6	12 45 35		3.6	SP	@ 50' - Poor Graded Gravelly SAND: gray to brown, slightly moist, very dense	
105	55							Total Depth = 51.5' Groundwater Not Encountered Backfilled with Cuttings on 10/26/2021	
	60								

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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

Geotechnical Boring Log Borehole LGC-HS-3

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~161' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
160	0							@ 0' - 6.5" Asphalt	
			R-1	5 9 11	116.2	15.6	CL-ML	@ 2.5' -Sandy SILT to Sandy CLAY: brown to reddish brown, moist, stiff, low plasticity	
155	5		SPT-1	3 3 3		14.2		@ 5' - Sandy SILT to Sandy CLAY: dark brown, moist, medium stiff, low plasticity	
			R-2	5 7 7				@ 7.5' - Silt CLAY: reddish brown to yellowish brown, moist to very moist, medium stiff, low plasticity	CN
150	10		SPT-2	3 3 3		22.7	CL	@ 10' - Silty CLAY: dusky brown, moist to very moist, medium stiff, low plasticity	
145	15		R-3	9 19 40	110.9	3.0	SP	@ 15' - Poorly Graded SAND: brown to dark gray, slightly moist, dense	
140	20		SPT-3	14 47 24		1.9	GP	@ 20' -Sandy Poorly Graded Gravel: dusky gray, dry to slightly moist, very dense	
135	25		R-4	42 50/3"	116.6	2.5		@ 25' - Sandy Poorly Graded Gravel: gray to brown, moist, very dense	
								Total Depth = 26' Groundwater Not Encountered Backfilled with Cuttings on 10/26/2021	
	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: 12/1/2021

Geotechnical Boring Log Borehole LGC-HS-4

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~161' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
160	0	B-2						@ 0' - 4" Asphalt	
			R-1	5 7 15	114.1	16.3	ML	@ 2.5' - Sandy SILT w/ trace Gravel: olive to gray, moist, stiff	
155	5		SPT-1	5 5 9		14.5	SM	@ 5' - Silty SAND: olive gray, very moist, medium dense	
			R-2	5 6 10	115.7	12.8	CL-ML	@ 7.5' - Silty CLAY: olive gray, moist, stiff	CN
150	10		SPT-2	3 4 5		13.7		@ 10' - Sandy SILT to Sandy CLAY: grayish brown, moist, stiff	
145	15		R-3	9 21 32	115.9	3.9	GP	@ 15' - Sandy Poorly Graded Gravel w/ SILT: grayish brown, slightly moist, dense	
140	20		SPT-3	12 24 23	R-1	1.2		@ 20' - Sandy Poorly Graded Gravel w/ SILT, grayish brown, slightly moist, dense	
135	25		R-4	18 22 24	115.5	4.4	SP-SM	@ 25' - SAND to Silty SAND w/ Gravel: grayish brown to reddish brown, slightly moist, dense	
	30							Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 10/26/2021	

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 LGC-I-1

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~165' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0								
			R-1				CL-ML	@ 3.5' - Sandy SILT to Sandy CLAY: brown, slightly moist to moist, stiff	
160	5							Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 10/26/2021	
155	10								
150	15								
145	20								
140	25								
	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: 12/3/2021

Geotechnical Boring Log Borehole LGC-I-2

Date: 10/26/2021	**Drilling Company:** Cal Pack Drilling
Project Name: SCIND Batavia Point	**Type of Rig:** Truck Mounted
Project Number: 21184-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~166' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KBC

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
165	0								
			R-1				CL-ML	@ 3.5' - Sandy SILT to Sandy CLAY: brown, slightly moist to moist, stiff	
160	5							Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 10/26/2021	
155	10								
150	15								
145	20								
140	25								
	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: 12/3/2021

Infiltration Test Data Sheet

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

Project Name: SCIND Batavia Point
Project Number: 21189-01
Date: 10/27/2021
Boring Number: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5
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	8:37	9:02	25.0	2.61	2.73	0.12	No
2	9:04	9:29	25.0	2.73	2.86	0.13	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, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:31	10:01	30.0	2.65	2.78	0.13	0.2
2	10:02	10:32	30.0	2.60	2.70	0.10	0.2
3	10:33	11:03	30.0	2.70	2.83	0.13	0.2
4	11:04	11:34	30.0	2.70	2.83	0.13	0.2
5	11:36	12:06	30.0	2.73	2.85	0.12	0.2
6	12:09	12:39	30.0	2.45	2.52	0.07	0.1
7	12:39	13:09	30.0	2.52	2.63	0.11	0.2
8	13:09	13:39	30.0	2.63	2.74	0.11	0.2
9	13:40	14:10	30.0	2.61	2.70	0.09	0.1
10	14:10	14:40	30.0	2.70	2.81	0.11	0.2
11	14:41	15:11	30.0	2.61	2.72	0.11	0.2
12	15:13	15:43	30.0	2.58	2.68	0.10	0.2

Tested Infiltration Rate (No Factor of Safety)	**0.2**
Minimum Factor of Safety	**2.0**
Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: SCIND Batavia Point
Project Number: 21189-01
Date: 10/27/2021
Boring Number: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5
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	8:38	9:03	25.0	2.70	2.75	0.05	No
2	9:04	9:29	25.0	2.69	2.72	0.03	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, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:31	10:01	30.0	2.55	2.58	0.03	0.0
2	10:02	10:32	30.0	2.49	2.52	0.03	0.0
3	10:32	11:02	30.0	2.52	2.56	0.04	0.1
4	11:04	11:34	30.0	2.48	2.52	0.04	0.1
5	11:36	12:06	30.0	2.52	2.60	0.08	0.1
6	12:09	12:39	30.0	2.43	2.49	0.06	0.1
7	12:39	13:09	30.0	2.44	2.52	0.08	0.1
8	13:09	13:39	30.0	2.52	2.59	0.07	0.1
9	13:40	14:10	30.0	2.51	2.59	0.08	0.1
10	14:10	14:40	30.0	2.59	2.65	0.06	0.1
11	14:41	15:11	30.0	2.47	2.55	0.08	0.1
12	15:13	15:43	30.0	2.49	2.56	0.07	0.1

Tested Infiltration Rate (No Factor of Safety)	**0.1**
Minimum Factor of Safety	**2.0**
Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. 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 relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring and/or trench logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the following table.

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 0-5 ft	37	Low
HS-4 @ 0-5 ft	20	Very Low

* ASTM D4829

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

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-1 @ 35 ft	27	18	9	CL
HS-2 @ 30 ft	23	17	6	CL-ML
HS-4 @ 0-5 ft	28	18	10	CL

APPENDIX C (Cont'd)

Laboratory Testing Procedures and Test Results

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-2 @ 5'	Brown Sandy Silt	71
HS-2 @ 10'	Brown Sandy Silt	85

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented in the following table.

Sample Location	Chloride Content, ppm
HS-4 @ 0-5 ft	20

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. As a result of a decrease in resistivity, the potential for corrosion increases. The results are presented in the following table.

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-4 @ 0-5 ft	8.13	2590

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the following table.

Sample Location	Sulfate Content (%)
HS-4 @ 0-5 ft	.0067

*Based on ACI 318R-19, Table 19.3.1.1

Laboratory Testing Procedures and Test Results

Hydro-consolidation: Hydro-consolidation tests (collapse) were performed on selected, relatively undisturbed ring samples (ASTM D4546). Samples were placed in a consolidometer and a load approximately equal to the in-situ overburden pressure was applied. Water was then added to the sample and the percent hydro-consolidation under the applied load was measured. The percent for the load was calculated as the ratio of the amount of vertical deformation to the original sample height. The percent hydro-consolidation results are presented in the following table.

Sample Location	**Percent Hydro-consolidation**
HS-2 @ 7.5 ft	-0.03

Note: Positive values of hydro-consolidation represent collapse of the soil structure, while negative values represent heave (or swelling) or the soil structure.

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 plots are provided in this Appendix.

R-Value: The resistance R-value was determined by the ASTM D2844 for base, subbase, and basement soils. The samples were prepared and exudation pressure and R-value were determined. The graphically determined R-values at exudation pressure of 300 psi are reported in this appendix. These results were used for pavement design purposes. The results of these tests are presented in the following table.

Sample Location	**R-Value**
HS-1 @ 0-5 ft	10

ATTERBERG LIMITS
ASTM D 4318

Project Name:	Orange	Tested By:	Y. Nguyen	Date:	11/22/21
Project No. :	21184-01	Input By:	J. Ward	Date:	12/03/21
Boring No.:	HS-1	Checked By:	J. Ward		
Sample No.:	R-5	Depth (ft.)	35.0		
Soil Identification:	Yellowish brown lean clay (CL)				

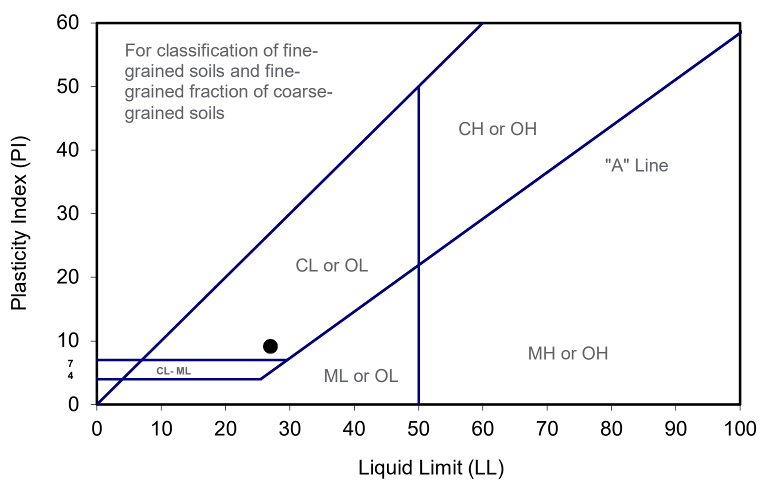
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	26	19	
Wet Wt. of Soil + Cont. (g)	9.85	9.89	19.21	19.91	20.77	
Dry Wt. of Soil + Cont. (g)	8.52	8.55	15.51	15.95	16.49	
Wt. of Container (g)	1.07	1.07	1.06	1.07	1.02	
Moisture Content (%) [Wn]	17.85	17.91	25.61	26.61	27.67	

Liquid Limit	**27**
Plastic Limit	**18**
Plasticity Index	**9**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 5.11

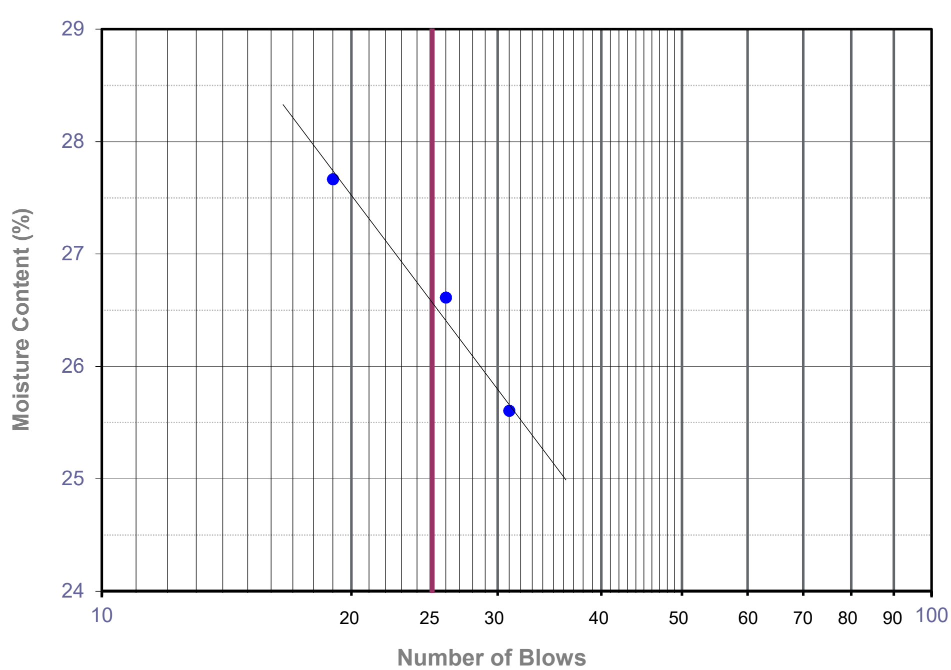
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



ATTERBERG LIMITS
ASTM D 4318

Project Name:	Orange	Tested By:	Y. Nguyen	Date:	11/22/21
Project No. :	21184-01	Input By:	J. Ward	Date:	12/03/21
Boring No.:	HS-2	Checked By:	J. Ward		
Sample No.:	SPT-4	Depth (ft.)	30.0		
Soil Identification:	Yellowish brown silty clay (CL-ML)				

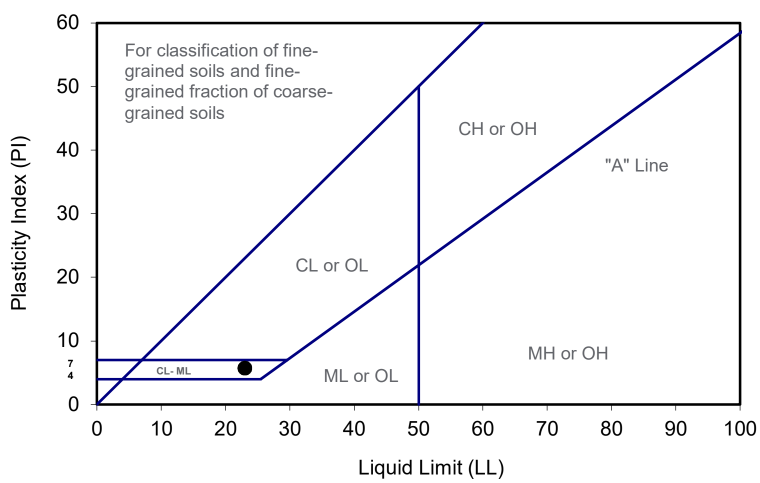
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	23	17	
Wet Wt. of Soil + Cont. (g)	10.02	10.21	22.06	21.22	19.10	
Dry Wt. of Soil + Cont. (g)	8.69	8.86	18.18	17.42	15.62	
Wt. of Container (g)	1.02	1.05	1.02	1.06	1.04	
Moisture Content (%) [Wn]	17.34	17.29	22.61	23.23	23.87	

Liquid Limit	**23**
Plastic Limit	**17**
Plasticity Index	**6**
Classification	**CL-ML**

PI at "A" - Line = 0.73(LL-20) | 2.19

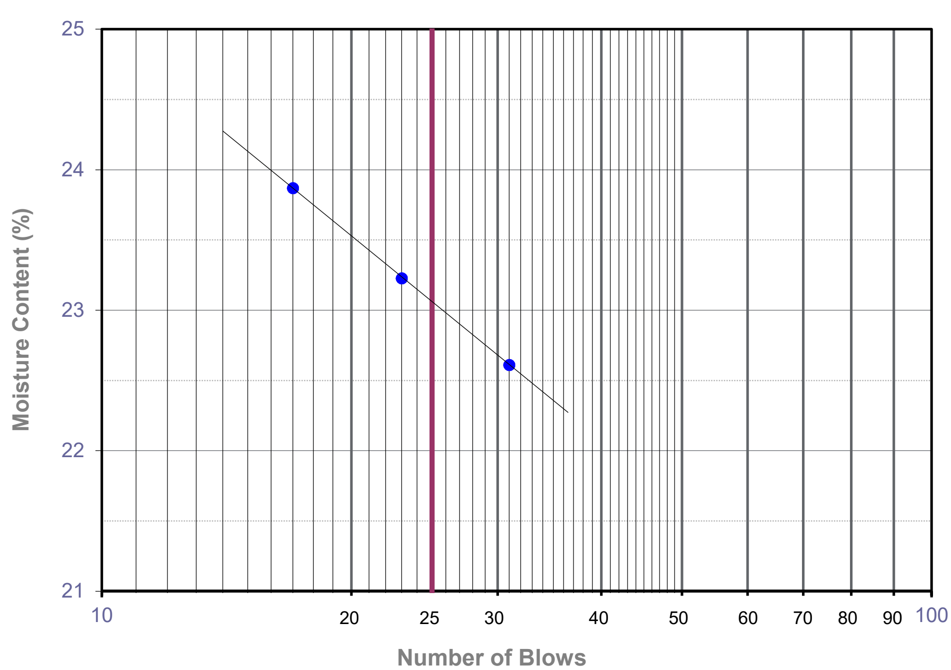
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



ATTERBERG LIMITS
ASTM D 4318

Project Name:	Orange	Tested By:	Y. Nguyen	Date:	11/22/21
Project No. :	21184-01	Input By:	J. Ward	Date:	12/03/21
Boring No.:	HS-4	Checked By:	J. Ward		
Sample No.:	B-1	Depth (ft.)	0-5		
Soil Identification:	Olive gray clayey sand (SC)				

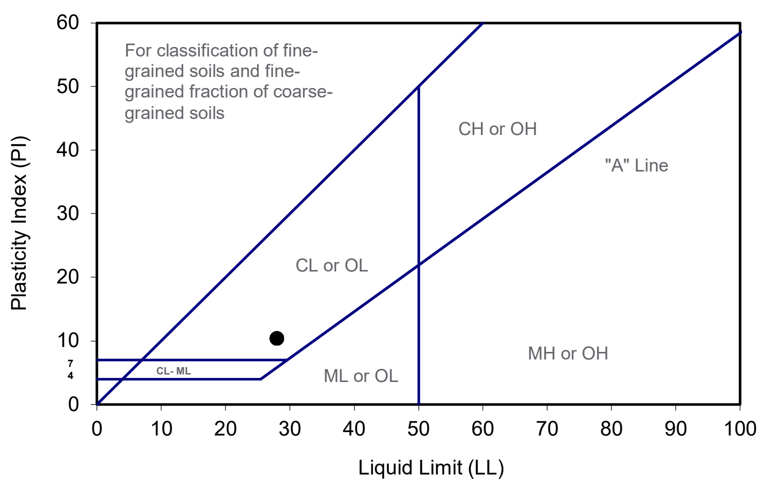
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			34	27	20	
Wet Wt. of Soil + Cont. (g)	9.90	10.03	18.33	18.36	21.54	
Dry Wt. of Soil + Cont. (g)	8.58	8.68	14.74	14.66	16.94	
Wt. of Container (g)	1.03	1.07	1.05	1.02	1.00	
Moisture Content (%) [Wn]	17.48	17.74	26.22	27.13	28.86	

Liquid Limit	**28**
Plastic Limit	**18**
Plasticity Index	**10**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 5.84

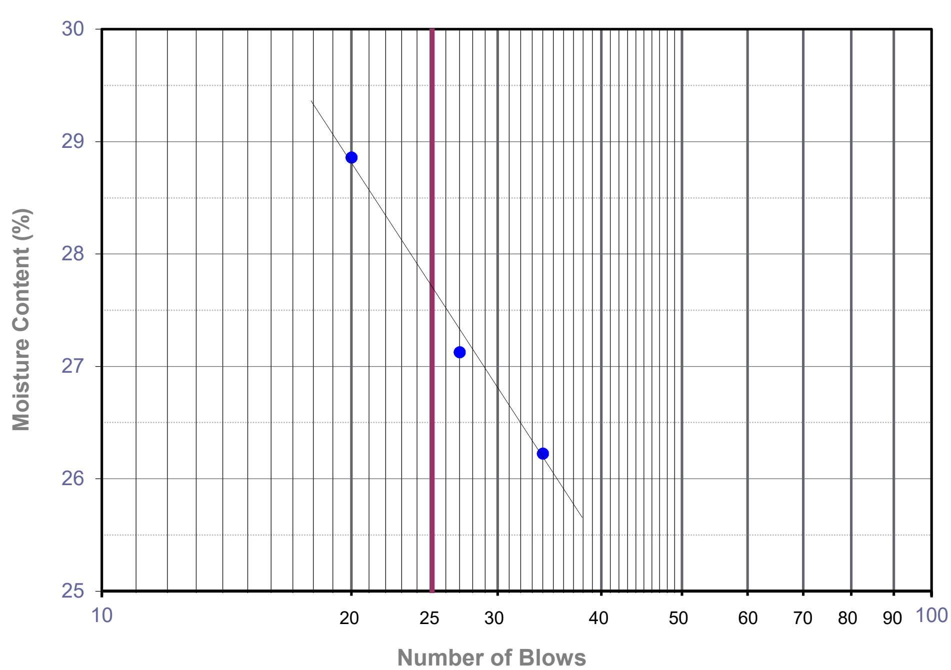
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Orange — Tested By : G. Bathala — Date: 11/13/21

Project No. : 21184-01 — Checked By: J. Ward — Date: 12/03/21

Boring No.	HS-4			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive gray SC			
Wet Weight of Soil + Container (g)	85.52			
Dry Weight of Soil + Container (g)	84.43			
Weight of Container (g)	38.48			
Moisture Content (%)	2.37			
Weight of Soaked Soil (g)	100.48			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	0			
Crucible No.	12			
Furnace Temperature (°C)	860			
Time In / Time Out	15:45/16:30			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	20.7505			
Wt. of Crucible (g)	20.7489			
Wt. of Residue (g) (A)	0.0016			
PPM of Sulfate (A) x 41150	65.84			
PPM of Sulfate, Dry Weight Basis	**67**			

CHLORIDE CONTENT, DOT California Test 422

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

pH TEST, DOT California Test 643

pH Value	**8.13**			
Temperature °C	20.3			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Orange	Tested By :	A. Santos Date: 11/23/21
Project No. :	21184-01	Checked By:	J. Ward Date: 12/03/21
Boring No.:	HS-4	Depth (ft.) :	0-5
Sample No. :	B-1		

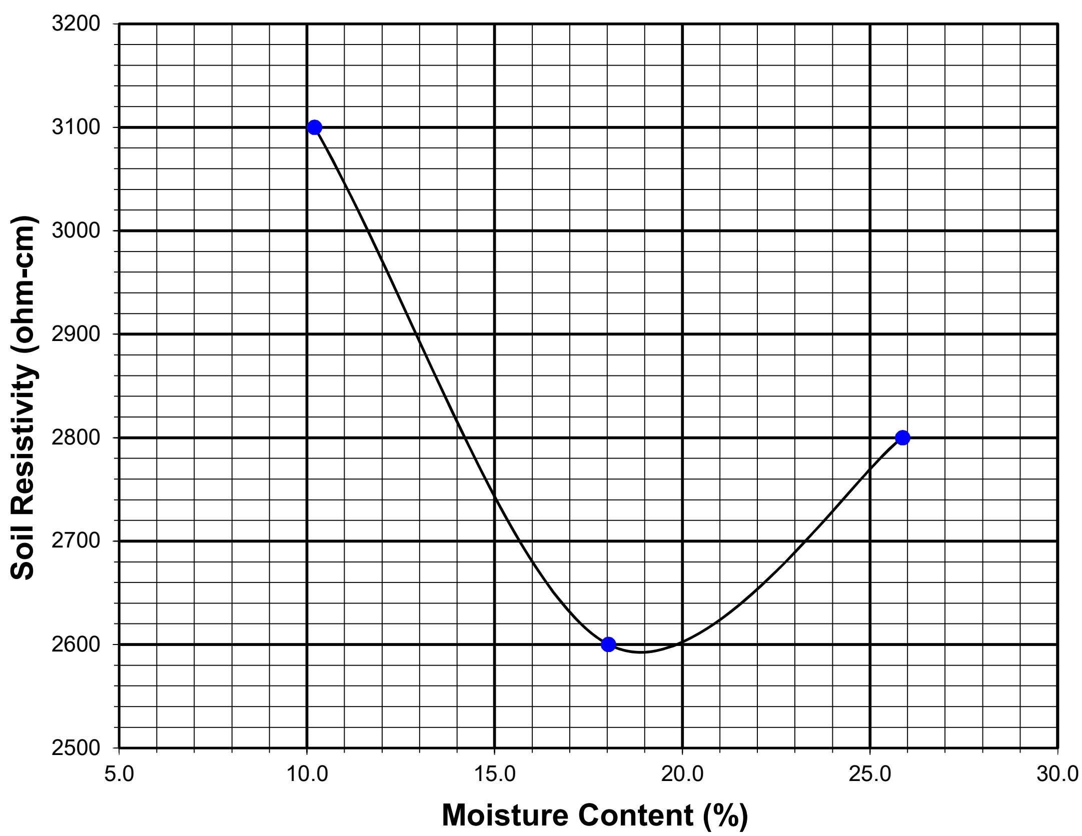
Soil Identification:* Olive gray SC

*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	10.20	3100	3100
2	20	18.04	2600	2600
3	30	25.87	2800	2800
4				
5				

Moisture Content (%) (MCi)	2.37
Wet Wt. of Soil + Cont. (g)	85.52
Dry Wt. of Soil + Cont. (g)	84.43
Wt. of Container (g)	38.48
Container No.	
Initial Soil Wt. (g) (Wt)	130.70
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	
2590	**19.0**	**67**	**20**	**8.13**	**20.3**



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

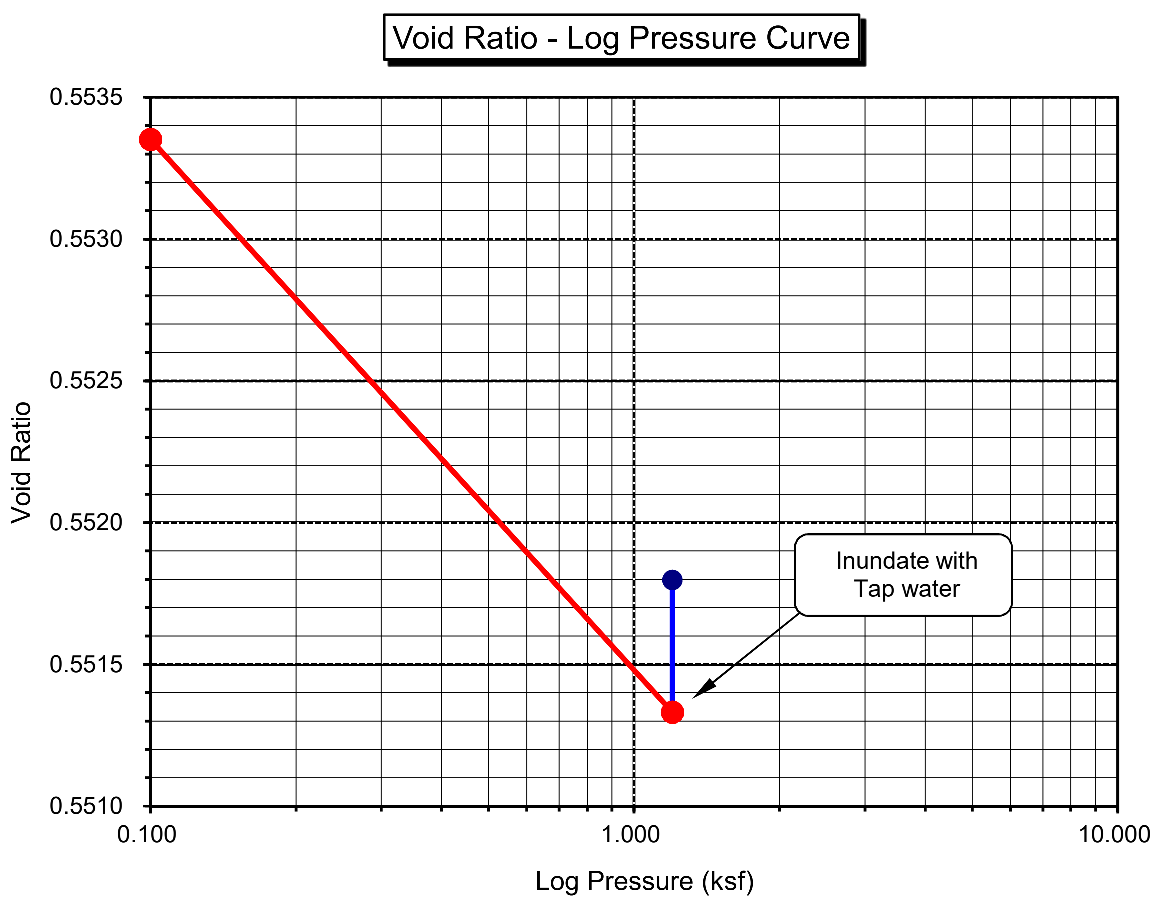
Project Name:	Orange	Tested By: G. Bathala	Date: 11/17/21
Project No.:	21184-01	Checked By: J. Ward	Date: 12/03/21
Boring No.:	HS-2	Sample Type: Ring	
Sample No.:	R-2	Depth (ft.) 7.5	
Sample Description:	Brown silty clay (CL-ML)		

Initial Dry Density (pcf):	108.5
Initial Moisture (%):	7.80
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2532
Diameter(in):	2.415

Final Dry Density (pcf):	108.6
Final Moisture (%) :	17.7
Initial Void Ratio:	0.5538
Specific Gravity(assumed):	2.70
Initial Saturation (%)	38.0

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2529	0.9997	0.00	-0.03	0.5534	-0.03
1.200	0.2484	0.9952	0.32	-0.48	0.5513	-0.16
H2O	0.2487	0.9955	0.32	-0.45	0.5518	-0.13

Percent Swell (+) / Settlement (-) After Inundation = 0.03



ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: Orange

Project No.: 21184-01

Boring No.: HS-1

Sample No.: R-2

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

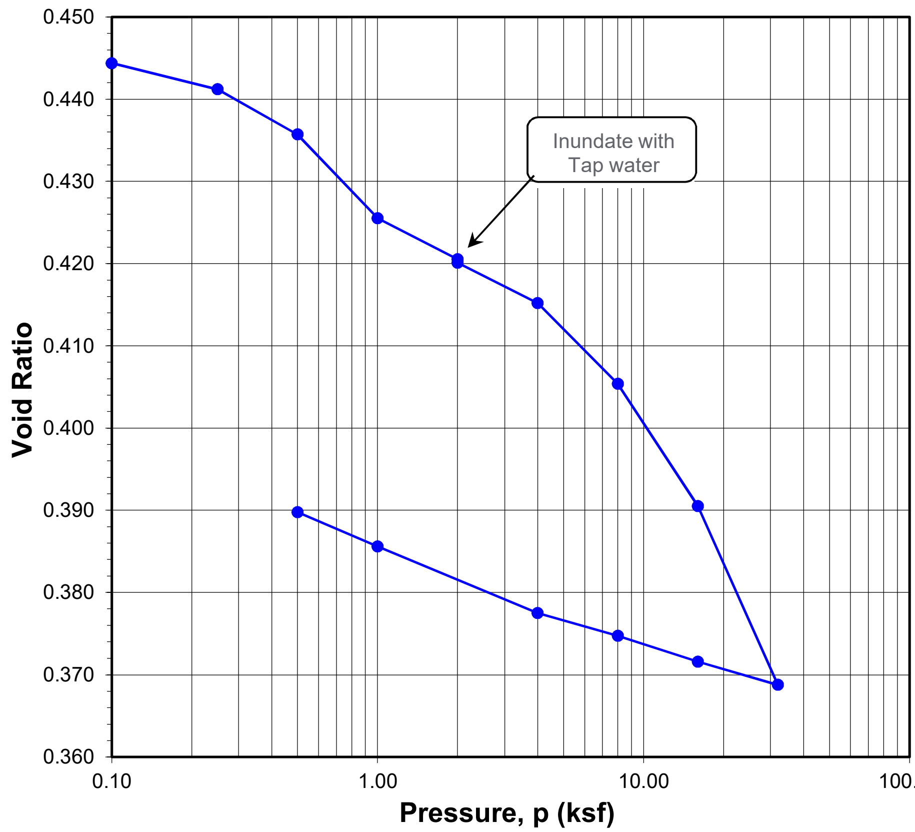
Tested By: G. Bathala Date: 11/12/21

Checked By: J. Ward Date: 12/02/21

Depth (ft.): 7.5

Sample Type: Ring

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	205.29
Weight of Ring (g)	45.51
Height after consol. (in.)	0.9622
Before Test	
Wt.Wet Sample+Cont. (g)	189.98
Wt.of Dry Sample+Cont. (g)	171.51
Weight of Container (g)	38.30
Initial Moisture Content (%)	13.9
Initial Dry Density (pcf)	116.7
Initial Saturation (%)	84
Initial Vertical Reading (in.)	0.2837
After Test	
Wt.of Wet Sample+Cont. (g)	262.89
Wt. of Dry Sample+Cont. (g)	243.88
Weight of Container (g)	59.03
Final Moisture Content (%)	13.64
Final Dry Density (pcf)	120.4
Final Saturation (%)	92
Final Vertical Reading (in.)	0.2405
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

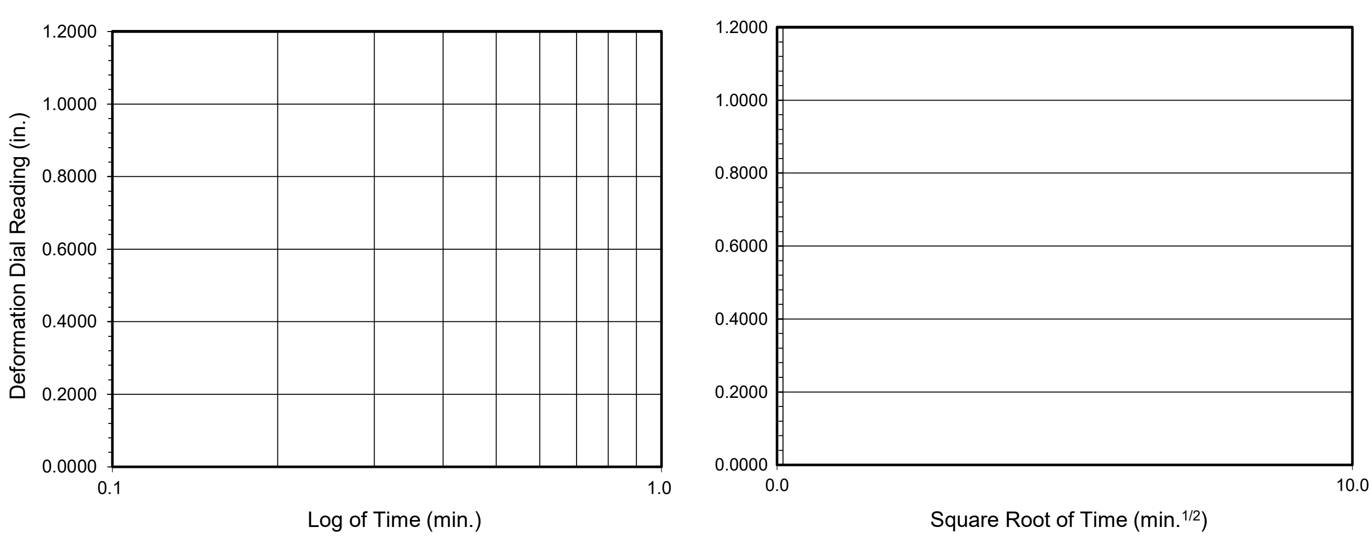


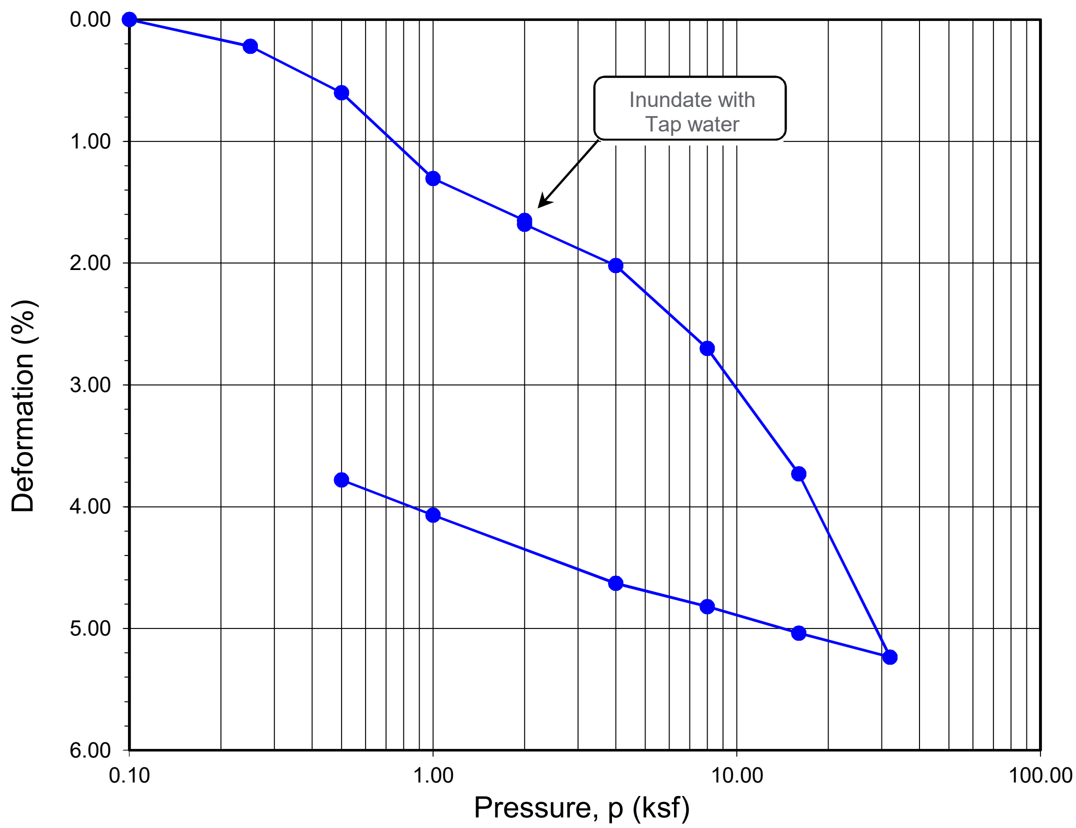
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.2837	1.0000	0.00	0.00	0.444	0.00
0.25	0.2810	0.9973	0.05	0.27	0.441	0.22
0.50	0.2766	0.9929	0.11	0.71	0.436	0.60
1.00	0.2688	0.9851	0.19	1.50	0.426	1.31
2.00	0.2642	0.9805	0.30	1.95	0.421	1.65
2.00	0.2639	0.9802	0.30	1.98	0.420	1.68
4.00	0.2591	0.9754	0.44	2.46	0.415	2.02
8.00	0.2502	0.9665	0.65	3.35	0.405	2.70
16.00	0.2373	0.9536	0.91	4.64	0.391	3.73
32.00	0.2197	0.9360	1.17	6.40	0.369	5.23
16.00	0.2227	0.9390	1.06	6.10	0.372	5.04
8.00	0.2265	0.9428	0.90	5.72	0.375	4.82
4.00	0.2298	0.9461	0.76	5.39	0.378	4.63
1.00	0.2371	0.9534	0.59	4.66	0.386	4.07
0.50	0.2405	0.9568	0.54	4.32	0.390	3.78

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-1	**R-2**	**7.5**	**13.9**	**13.6**	**116.7**	**120.4**	**0.444**	**0.390**	**84**	**92**

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

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 21184-01

Orange

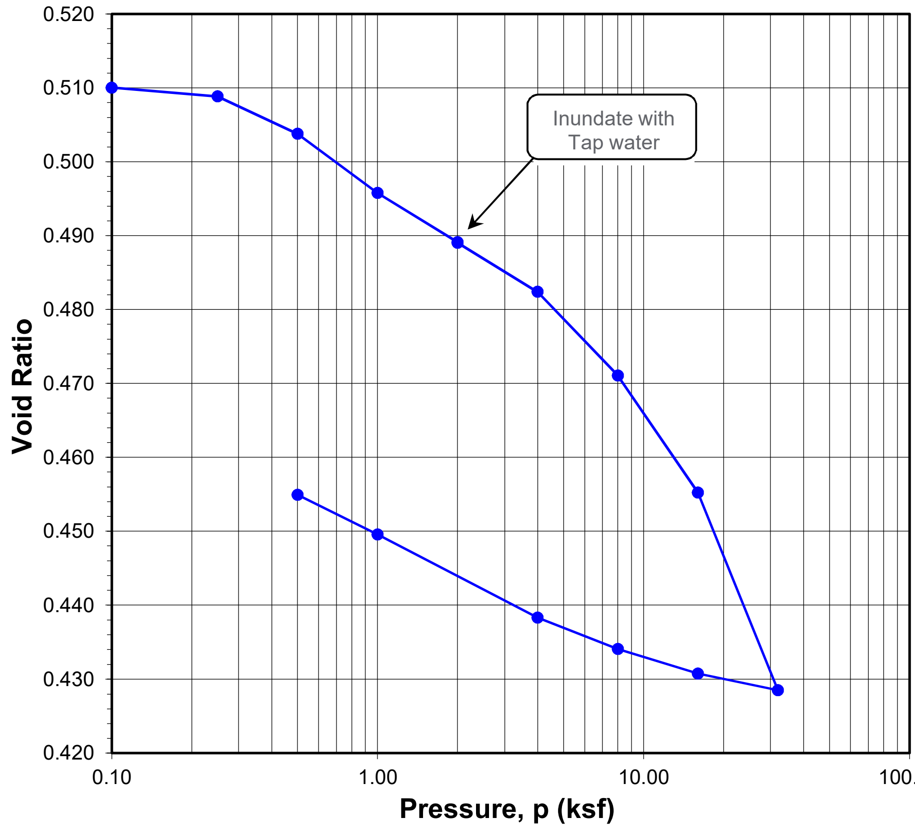
12-21

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Orange	Tested By: G. Bathala	Date:	11/12/21
Project No.:	21184-01	Checked By: J. Ward	Date:	12/02/21
Boring No.:	HS-3	Depth (ft.): 7.5		
Sample No.:	R-2	Sample Type:	Ring	
Soil Identification:	Yellowish brown silty clay (CL-ML)			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	199.79
Weight of Ring (g)	45.68
Height after consol. (in.)	0.9635
Before Test	
Wt.Wet Sample+Cont. (g)	189.28
Wt.of Dry Sample+Cont. (g)	170.02
Weight of Container (g)	40.04
Initial Moisture Content (%)	14.8
Initial Dry Density (pcf)	111.6
Initial Saturation (%)	78
Initial Vertical Reading (in.)	0.3242
After Test	
Wt.of Wet Sample+Cont. (g)	237.40
Wt. of Dry Sample+Cont. (g)	217.18
Weight of Container (g)	36.72
Final Moisture Content (%)	15.00
Final Dry Density (pcf)	116.3
Final Saturation (%)	90
Final Vertical Reading (in.)	0.2827
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

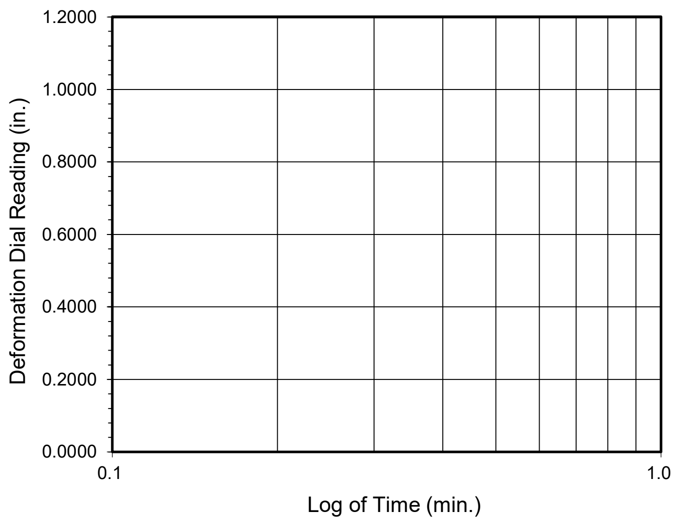


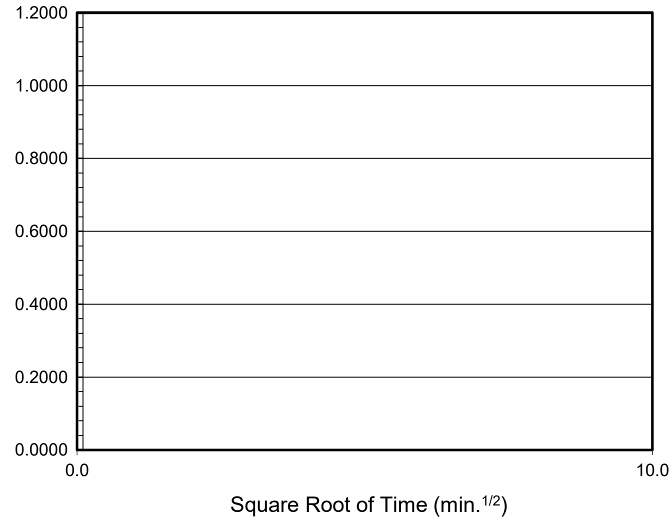
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3242	1.0000	0.00	0.00	0.510	0.00
0.25	0.3231	0.9989	0.03	0.11	0.509	0.08
0.50	0.3195	0.9953	0.06	0.48	0.504	0.42
1.00	0.3136	0.9894	0.12	1.07	0.496	0.95
2.00	0.3082	0.9840	0.22	1.61	0.489	1.39
2.00	0.3081	0.9839	0.22	1.61	0.489	1.39
4.00	0.3024	0.9782	0.35	2.18	0.482	1.83
8.00	0.2931	0.9689	0.53	3.11	0.471	2.58
16.00	0.2805	0.9563	0.74	4.37	0.455	3.63
32.00	0.2604	0.9362	0.98	6.38	0.429	5.40
16.00	0.2631	0.9389	0.86	6.11	0.431	5.25
8.00	0.2665	0.9423	0.74	5.77	0.434	5.03
4.00	0.2702	0.9460	0.65	5.40	0.438	4.75
1.00	0.2789	0.9547	0.53	4.54	0.450	4.01
0.50	0.2827	0.9585	0.50	4.15	0.455	3.65

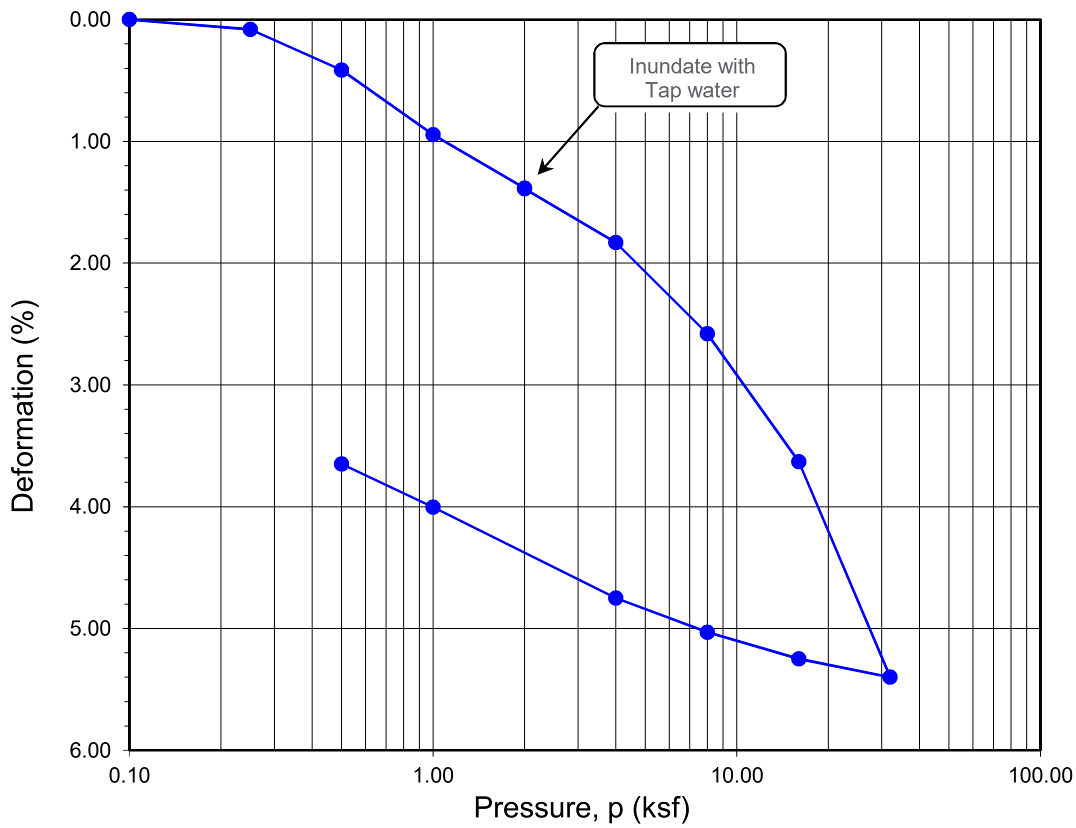
Time Readings

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-3	**R-2**	**7.5**	**14.8**	**15.0**	**111.6**	**116.3**	**0.510**	**0.455**	**78**	**90**

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

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 21184-01

Orange

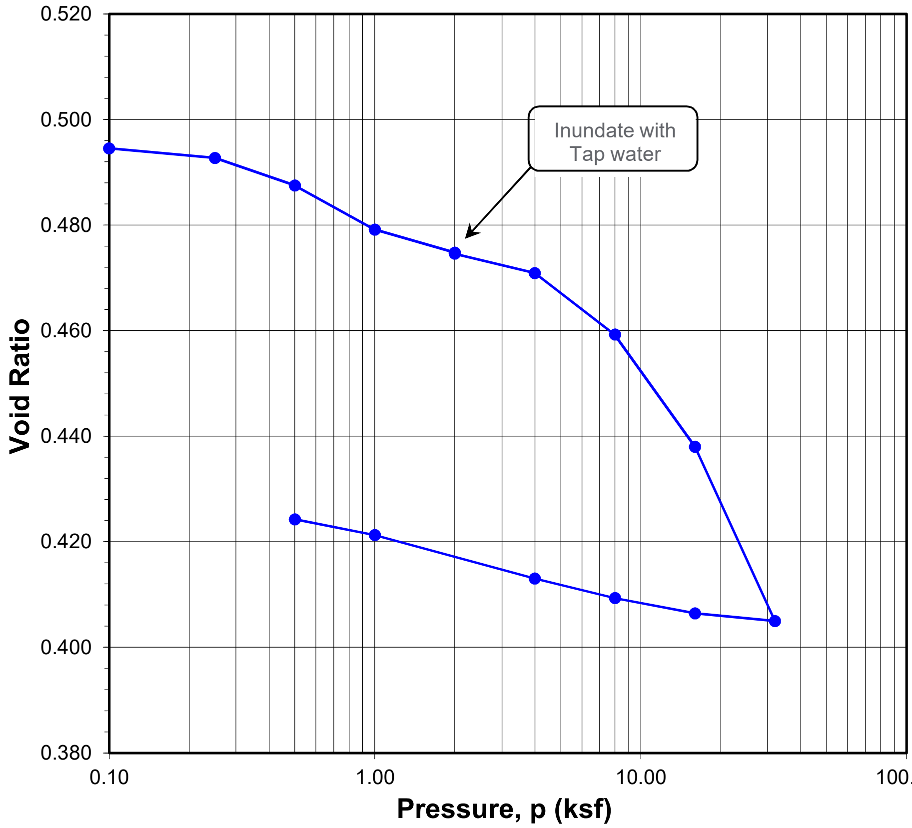
12-21

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Orange	Tested By:	G. Bathala	Date:	11/12/21
Project No.:	21184-01	Checked By:	J. Ward	Date:	12/02/21
Boring No.:	HS-4	Depth (ft.):	7.5		
Sample No.:	R-2	Sample Type:	Ring		
Soil Identification:	Olive gray silty clay (CL-ML)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	198.13
Weight of Ring (g)	45.14
Height after consol. (in.)	0.9529
Before Test	
Wt.Wet Sample+Cont. (g)	224.90
Wt.of Dry Sample+Cont. (g)	206.06
Weight of Container (g)	59.14
Initial Moisture Content (%)	12.8
Initial Dry Density (pcf)	112.8
Initial Saturation (%)	70
Initial Vertical Reading (in.)	0.3105
After Test	
Wt.of Wet Sample+Cont. (g)	259.47
Wt. of Dry Sample+Cont. (g)	242.14
Weight of Container (g)	63.85
Final Moisture Content (%)	13.02
Final Dry Density (pcf)	116.2
Final Saturation (%)	78
Final Vertical Reading (in.)	0.2586
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

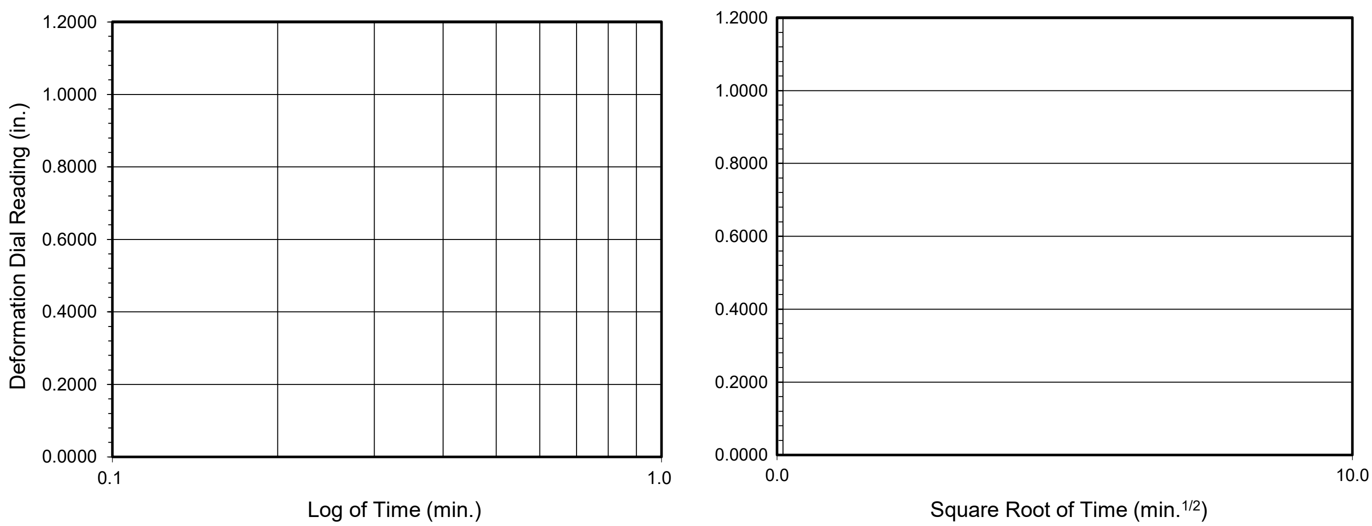


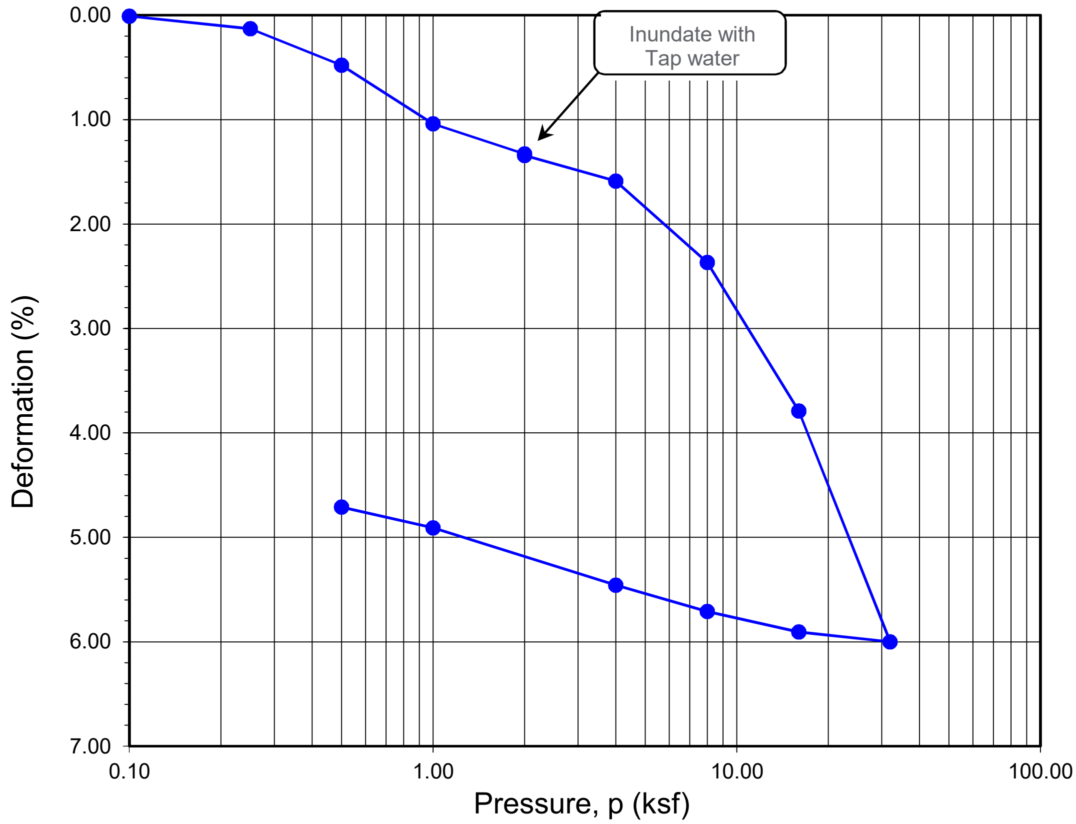
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3104	0.9999	0.00	0.01	0.495	0.01
0.25	0.3089	0.9984	0.03	0.16	0.493	0.13
0.50	0.3045	0.9940	0.12	0.60	0.488	0.48
1.00	0.2971	0.9866	0.30	1.34	0.479	1.04
2.00	0.2927	0.9822	0.45	1.78	0.475	1.33
2.00	0.2926	0.9821	0.45	1.80	0.475	1.35
4.00	0.2882	0.9777	0.64	2.23	0.471	1.59
8.00	0.2787	0.9682	0.81	3.18	0.459	2.37
16.00	0.2627	0.9522	0.99	4.78	0.438	3.79
32.00	0.2384	0.9279	1.21	7.21	0.405	6.00
16.00	0.2408	0.9303	1.07	6.98	0.406	5.91
8.00	0.2439	0.9334	0.95	6.66	0.409	5.71
4.00	0.2475	0.9370	0.84	6.30	0.413	5.46
1.00	0.2555	0.9450	0.59	5.50	0.421	4.91
0.50	0.2586	0.9481	0.48	5.19	0.424	4.71

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**	**7.5**	**12.8**	**13.0**	**112.8**	**116.2**	**0.495**	**0.424**	**70**	**78**

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

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 21184-01

Orange

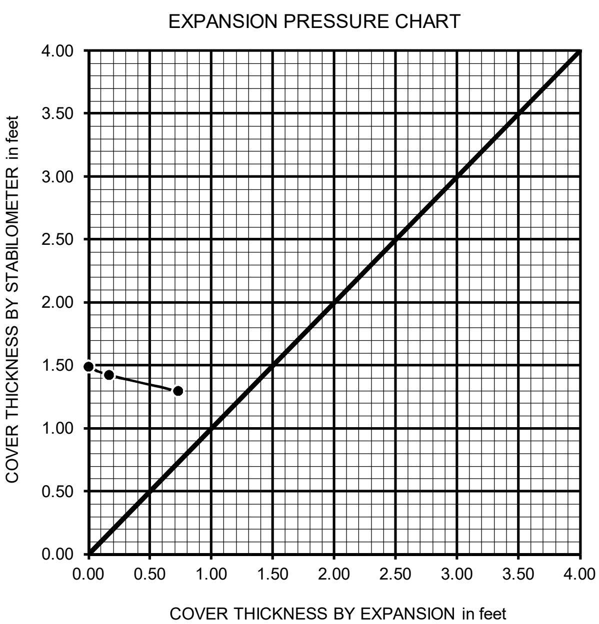
12-21

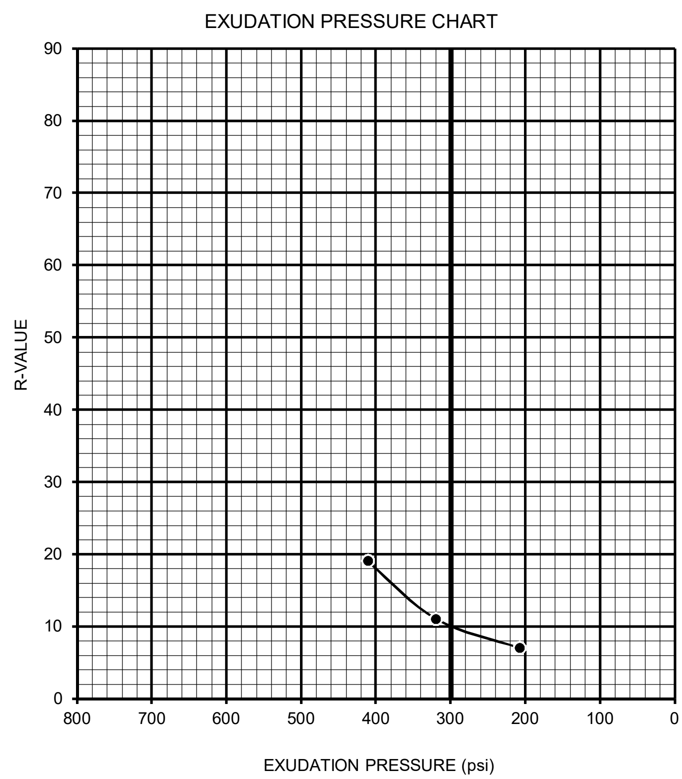
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Orange	PROJECT NUMBER:	21184-01
BORING NUMBER:	HS-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Strong brown sandy lean clay s(CL)	DATE COMPLETED:	11/16/2021

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	14.5	15.4	16.7
HEIGHT OF SAMPLE, Inches	2.48	2.52	2.57
DRY DENSITY, pcf	120.6	118.8	115.4
COMPACTOR PRESSURE, psi	110	80	60
EXUDATION PRESSURE, psi	411	319	207
EXPANSION, Inches x 10exp-4	22	5	0
STABILITY Ph 2,000 lbs (160 psi)	116	132	140
TURNS DISPLACEMENT	4.15	4.35	4.55
R-VALUE UNCORRECTED	19	11	7
R-VALUE CORRECTED	19	11	7

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.30	1.42	1.49
EXPANSION PRESSURE THICKNESS, ft.	0.73	0.17	0.00





R-VALUE BY EXPANSION:	25
R-VALUE BY EXUDATION:	10
EQUILIBRIUM R-VALUE:	10

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.