Appendix D:
Preliminary Geotechnical Evaluation

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July 31, 2018

Project No. 18078-01

Mr. Mike Tunney
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Subject: ***Preliminary Geotechnical Evaluation for Proposed Warehouse Building, Ashley Way, Colton, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed warehouse building to be located on Ashley Way, south of the intersection of East Cooley Drive and Ashley Way in Colton, 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 warehouse building to be located on Ashley Way, south of the intersection of East Cooley Drive and Ashley Way in Colton, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical design recommendations relative to the proposed development of the site.

1.1 Project Description and Background

The approximately 11-acre, roughly triangular-shaped site is bound to the north by Ashley Way, to the south by a drainage basin, to the east by the 215 Freeway and to the west by existing commercial building and parking of off East Cooley Drive. The site is relatively flat consisting of a previously graded building pad.

Based on the preliminary plans, the proposed development will include construction of an at-grade warehouse building, associated loading docks and parking areas. Per the project structural engineer, preliminary maximum dead plus live column and wall loads for the proposed structures are 87 kips and 7.4 kips per foot (HSA Associates, 2018). A preliminary grading plan was not available at the time of this report. However, proposed grades are not anticipated to significantly change from existing.

The recommendations given in this report are based on the layout and provided 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

A geotechnical field evaluation was performed by LGC Geotechnical consisting of four hollow-stem auger borings and five Cone Penetration Test (CPT) soundings.

The borings (HS-1 through HS-3 and I-1 through I-5), were excavated using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers with depths ranging from approximately 5 to 50 feet below existing grade. Borings I-1 through I-5 were drilled to 5 feet in depth for field percolation testing. 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 automatic 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 borings were capped with asphalt cold patch. At the completion of infiltration testing with I-1 through I-5, the installed pipe was removed and the resulting void backfilled with native soils.

The CPT soundings (CPT-1 through CPT-5) were pushed to depths ranging between approximately 47 to 60 feet below existing grade. Four of the five CPT soundings met refusal prior to the target depth of 60 feet below existing grade. The CPT soundings were pushed using an electronic cone penetrometer in general accordance with the current ASTM standards (ASTM D5778 and ASTM D3441). The CPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The interior of the cone penetrometer is instrumented with strain gauges that allow the simultaneous measurement of cone tip and friction sleeve resistance during penetration. The cone penetration assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.8 inches per second while the cone tip resistance and sleeve friction resistance are recorded at approximately every 2 inches and stored in digital form. A specially designed all-wheel drive 25-ton truck provides the required reaction weight for pushing the cone assembly.

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

1.3 Field Infiltration Testing

Five field percolation tests were performed to approximate depths of 5 feet below existing grade. The approximate locations are shown on Figure 2. Field percolation testing was performed in general accordance with the guidelines set forth by the County of San Bernardino (2013). For the falling head test, a 2-inch-diameter slotted PVC pipe was placed in the boreholes to a depth of approximately 5 feet below existing grade and the annulus was backfilled with gravel to the surface including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked per the County guidelines. The tests were performed with an average head (depth of water) of approximately 2 feet above the bottom of the proposed infiltration surfaces. Based on the County of San Bernardino methodology, the observed infiltration rate, summarized in Table 1, has normalized the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only. The measured infiltration rates are based on a factor of safety of 2.0 for feasibility. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. Refer to the discussion provided in Section 4.9.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Measured Infiltration Rate* (inch/hr)
I-1	2.1
I-2	0.8
I-3	2.4
I-4	2.2
I-5	1.1

*Based on a factor of safety of 2.0 for feasibility

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 moisture content, fines content, Atterberg Limits, consolidation, expansion index, laboratory compaction, R-Value and corrosion (sulfate, chloride, pH and minimum resistivity).

- Dry density of the samples collected ranged from approximately 92 pounds per cubic foot (pcf) to 127 pcf, with an average of 109 pcf. Field moisture contents ranged from approximately 1 percent to 27 percent, with an average of 6 percent.
- Nine fines content tests were performed and indicated a fines content (passing No. 200 sieve) ranging from approximately 22 to 96 percent. Based on the Unified Soils Classification System (USCS), five of the nine tested samples would be classified as "coarse-grained."
- A consolidation test was performed. The deformation versus vertical stress plot is provided in Appendix C.
- An Expansion Index (EI) test of a near-surface sample indicated an EI value of 3, corresponding to "Very Low" expansion potential.
- A laboratory compaction test resulted in a maximum dry density of 137.5 pcf at an optimum moisture content of 7.0 percent.
- An R-Value test of a near-surface sample resulted in an R-Value of 81.
- Corrosion testing indicated a soluble sulfate content of less than approximately 0.01 percent, a chloride content of 10 parts per million (ppm), pH of 8.4, and a minimum resistivity of 5,875 ohm-centimeters.

Laboratory test results obtained from our field evaluation are provided in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Geology

The site is located in the flood plain for the Santa Ana River. The regional geologic map of the area indicates the site is underlain by alluvial sand, gravel and clay of valley areas (Dibblee, 2003). The alluvial material is overlain by artificial fill soils.

In general, our borings and CPT soundings indicate that the site contains primarily medium dense to very dense sands with varying amounts of silt and gravel and layers of fine-grained silts and clays to the maximum explored depth of approximately 60 feet below existing grade. Blow counts of the hollow stem borings for sandy soils are very likely impacted by the presence of interbedded layers of fine-grained soils (i.e., silts and clays). CPT-2 and CPT-3 indicated more fine-grained soils in the upper approximate 25 feet below existing grade compared to the remaining borings and CPT soundings.

It should be noted that borings and CPT soundings are only representative of the location and time where/when they are performed and varying subsurface conditions may exist outside of the performed location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform and that soil is homogeneous within the project area. For details on the stratigraphy at the exploration locations, refer to Appendix B.

2.2 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, dipping in a westerly direction.

2.3 Landslides

Our research and field observations do not indicate the presence of landslides on the site or in the immediate vicinity. Review of regional geologic maps of the area do not indicate the presence of known or suspected landslides in the vicinity of the site (Dibblee, 2003).

2.4 Groundwater

Groundwater was not encountered during our subsurface evaluation to the maximum explored depth of approximately 50 feet below existing ground surface. For analysis purposes, historic high groundwater has been assumed at approximately 30 feet below existing grade.

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.

2.5 Faulting

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. Their purpose was to prevent the construction of urban developments across the trace of active faults. The result is the Alquist-Priolo Earthquake Fault Zoning Act, which 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 the Holocene Epoch (roughly the last 11,000 years). A potentially active fault is defined as any fault, which has had surface displacement during Quaternary time (last 1,600,000 years), but not within the Holocene. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault investigations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from the zone of previous ground rupture.

The subject site is not located within a State of California Fault Rupture Hazard Zone (CGS, 1977), nor is it located in County of San Bernardino Fault Rupture Hazard Map (Sbcounty.gov, 2018). The site is located in close proximity to Fault Rupture Hazard Zones for the San Bernardino section of the San Jacinto fault zone (CGS, 1977 & Sbcounty.gov, 2018). There are no known active or potentially active faults mapped on the site. The possibility of damage due to ground rupture, as a result of faulting, is considered very low since active faults are not known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching and shallow ground rupture, soil liquefaction, dynamic settlement, seiches and tsunamis. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. A discussion of these secondary effects is provided in the following sections.

2.5.1 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.

2.5.2 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 subject site is located in a zone of "Medium" liquefaction susceptibility according to the San Bernardino Geologic Hazards overlay (Sbcounty.gov, 2018). Based on the findings of our subsurface evaluation, the site contains sandy layers susceptible to liquefaction in the upper 50 feet. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008). Liquefaction analysis was based on the applicable seismic criteria (e.g., PGA_M from 2016 CBC) and the estimated historic high groundwater depth of 30 feet below existing grade. We estimate total seismic settlement due to liquefaction potential on the order of 1 to 2 inches. Differential seismic settlement may be estimated as 1-inch over a horizontal span of 40 feet. Refer to Appendix D.

2.5.3 <u>Lateral Spreading</u>

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.

Site sandy soils generally have a normalized clean sand tip resistance above 70. A normalized clean sand tip resistance of 70 corresponds to a blow count $(N_1)_{60}$ of at least 15. Soils with a corrected SPT $(N_1)_{60}$ blow count of 15 or greater are generally not considered susceptible to lateral spreading (Youd, Hansen, Bartlett, 2002). Due to the apparent density of site sandy soils, the potential for lateral spreading is considered low.

2.5.4 <u>Tsunamis and Seiches</u>

Based on the distance to open bodies of water, there is a low possibility of damage to the site during a large tsunami event.

2.6 <u>Seismic Design Parameters</u>

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

second. Representative site coordinates of latitude 34.0550 degrees north and longitude -117.3008 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class F modified due to site period to Site Class D are provided in Table 2 below.

Section 1803.5.12 of the 2016 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.908g (USGS, 2018).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 7.3 at a distance of approximately 4.1 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on 475-year average return period indicates that an earthquake magnitude of 7.2 at a distance of approximately 6.9 km from the site would contribute the most to this ground motion (USGS, 2008).

TABLE 2

Seismic Design Parameters

Selected Parameters from 2016 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class per Chapter 20 of ASCE 7	D*
Risk-Targeted Spectral Acceleration for Short Periods (S_S)**	2.361g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)**	1.080g
Site Coefficient F_a per Table 1613.3.3(1)	1.0
Site Coefficient F_v per Table 1613.3.3(2)	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_aS_S$]	2.361g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_vS_1$]	1.619g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.574g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	1.080g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	1.005
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	0.960

* Site Class F, seismic parameters only applicable for structure period ≤ 0.5 second, refer to discussion above.

** From USGS, 2018

2.7 ***Rippability***

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

2.8 ***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 E. If feasible, crushing oversized materials or exporting to an offsite location may be considered.

3.0 FINDINGS AND CONCLUSIONS

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

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

- In general, our borings and CPT soundings indicate that the site contains primarily medium dense to very dense sands with varying amounts of silt and gravel with layers of fine-grained silts and clays to the maximum explored depth of approximately 60 feet below existing grade. The near-surface 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 (not retaining wall backfill), provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- Groundwater was not encountered to the maximum explored depth of approximately 50 feet below existing ground surface. For analysis purposes, historic high groundwater has been assumed at approximately 30 feet below existing grade.
- 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 Earthquake Fault Rupture Hazard Zone and no faults were identified on the site during our site evaluation.
- Subsurface data indicates that the site contains sandy layers susceptible to liquefaction and liquefaction-induced settlement in the upper 50 feet. Total seismic settlements due to liquefaction potential are estimated to be on the order of 1 to 2 inches. Differential seismic settlement may be estimated as 1-inch over a horizontal span of 40 feet.
- Soils exposed at the proposed foundation level are anticipated to have a “Very Low” expansion potential (EI not exceeding 20). This shall be confirmed at the completion of site earthwork.
- Due to estimated dynamic settlement, proposed building foundation should consist of either a mat foundation or isolated pad footings interconnected with grade beams.
- Excavation for foundations and underground improvements should be achievable with the appropriate equipment.
- Five field percolation tests resulted in measured infiltration rates ranging from approximately 0.8 to 2.4 inches per hour. These infiltration rates are based on feasibility factor of safety 2.0. Refer to Section 4.9.

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 2016 California Building Code (CBC) requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level.” The “acceptable level” of risk is defined by the California Code of Regulations as “the level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

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

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of required earthwork removals, foundation construction and utility line/retaining wall construction and backfill. We recommend that earthwork onsite be performed in accordance with the following recommendations, 2016 CBC/City of Colton and the General Earthwork and Grading Specifications included in Appendix E. In case of conflict, the following recommendations shall supersede previous recommendations and those included as part of Appendix E.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill, engineered structures or improvements, the area should be cleared of existing vegetation (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 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 improvements. Building lines may be defined as the perimeter of the building proper, plus attached or adjacent foundation supported features, including canopies, elevators, or walls.

Retaining/Free-Standing Wall Structures: For planned retaining walls removals should extend a minimum of 5 feet below existing grade or 2 feet below proposed footings, whichever is greater. For minor structures such as free-standing and screen walls, the removals should extend at least 3 feet beneath the existing grade or 2 feet beneath the base of foundations, whichever is deeper.

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. 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

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter.

Based on our field evaluation, site soils upper approximate 5 feet are anticipated to be OSHA Type "C" soils (refer to the attached boring logs). Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person", required by OSHA standards, to evaluate soil conditions. Sandy soils are present and should be considered susceptible to caving. The contractor shall be responsible for providing the "competent person", required by OSHA standards, to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

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. 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.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

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

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

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

4.1.5 <u>Material for Fill</u>

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

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

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. Soils will likely require additional moisture conditioning in order to achieve the required compaction. Drying and/or mixing the very moist soils may also be required prior to reusing the materials in compacted fills. 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 compacted 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 Trench and Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other materials greater than 6 inches in diameter and organic matter. If trenches are shallow, or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 (per California Test Method [CTM] 217) or greater may be used to bed and shade the pipes. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least 90 percent relative compaction (per ASTM D1557).

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 3). 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.

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

Preliminary foundation recommendations are provided in the following sections. Provided that the remedial grading recommendations provided herein are implemented, structural mitigation using a stiffened foundation system may be used.

Foundation alternatives include a mat foundation or spread footings interconnected with grade beams. Due to liquefaction potential (Site Class "F") and dynamic settlement any isolated pad structural footings should be interconnected with grade beams.

Site soils are anticipated to be of Very Low expansion potential (EI of 20 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.

4.2.1 Preliminary Foundation Design Parameters

A stiffened foundation is recommended for support of the proposed building structure to reduce the effect of differential settlement due to potential dynamic settlement. Total dynamic settlement is estimated at 1 to 2 inches. Differential settlement may be estimated at 1-inch over 40 horizontal feet due to liquefaction potential (refer to Section 2.5.2).

For elastic design of a mat foundation supporting sustained concentrated loads, a modulus of vertical subgrade reaction (k) of 40 pounds per cubic inch (pounds per square inch per inch of deflection) may be used, provided the recommended earthwork is performed

4.2.2 Shallow Foundation Maintenance

Moisture conditioning of the subgrade soils is recommended prior to trenching the foundation. The subgrade moisture condition of the building pad soils should be maintained up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy.

Roots that extend near the vicinity of foundations can cause distress to foundations. Trees/large shrubs should not be planted 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.

4.2.3 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, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 18 inches below lowest adjacent ground surface. This value may be increased by 500 psf for each additional foot of embedment and by 300 psf for each additional foot of foundation width to a maximum value of 3,000 psf. A mat foundation a minimum of 8 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. 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 are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads). The increase of bearing capacity is based on a reduced factor of safety (seismic factor of safety equal to three-fourths of the static factor of safety) for short duration 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 230 psf per foot of depth (or pcf) to a maximum of 2,300 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. This increase is based on a reduced factor of safety for short duration 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. 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 3 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 3

Lateral Earth Pressures – Sandy Backfill

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

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed, 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 0.46 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 2016 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 provisional minimum asphalt concrete (AC) pavement sections are provided in Table 4 based on an R-value of 40 for Traffic Indices (TI) of 5 through 7. These recommendations should be confirmed with R-value testing of representative near-surface soils at the completion of earthwork. Final pavement sections should be confirmed by the project civil engineer based upon the final design Traffic Index. Determination of the TI is not the purview of the geotechnical consultant. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 4

Paving Section Options

Assumed Traffic Index	5.0	6.0	7.0
R -Value Subgrade	40	40	40
AC Thickness	4.0 inches	4.0 inches	4.0 inches
Aggregate Base Thickness	4.0 inches	5.0 inches	7.0 inches

For preliminary planning purposes, a Portland Cement concrete pavement section may consist of a minimum of 6 inches of concrete (reinforced with No. 3 rebar at 24 inches on-center each way) over 4 inches of compacted aggregate base over compacted subgrade soils. The concrete should have a minimum compressive strength of 4,000 psi at the time the pavement is subjected to traffic. To reduce the potential (but not eliminate) for cracking, concrete paving should include control joints at regular intervals not exceeding 12 feet in each direction. The recommended concrete section provided above is based on an approximate Traffic Index of 5.5. The thickness of the section should be thickened for increased heavy truck loading conditions based on the anticipated traffic volume.

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

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

4.6 Soil Corrosivity

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

Corrosion testing indicated a soluble sulfate content of less than approximately 0.01 percent, a chloride content of 10 ppm, pH of 8.4, and a minimum resistivity of 5,875 ohm-centimeters. Based on Caltrans Corrosion Guidelines (2015), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2 percent) or greater.

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

4.7 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, etc.) has a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 5. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement 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 5

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 residences be sloped away from the proposed building 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.

Geotechnical stability and integrity of the project site is reliant upon appropriate handling of surface water. Due to site liquefaction potential, the intentional infiltration of storm water is not recommended.

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 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 2016 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 <u>LIMITATIONS</u>

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.

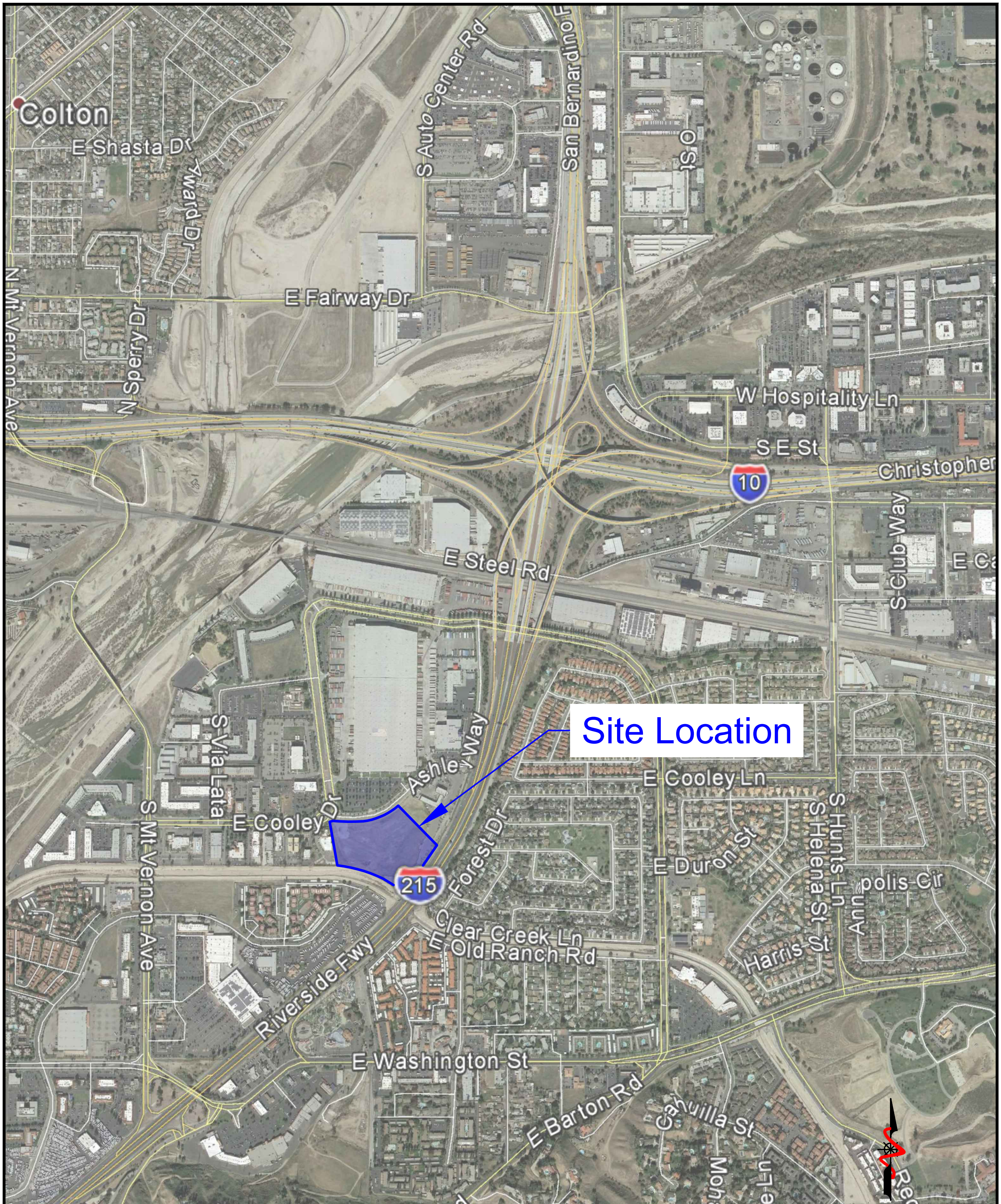

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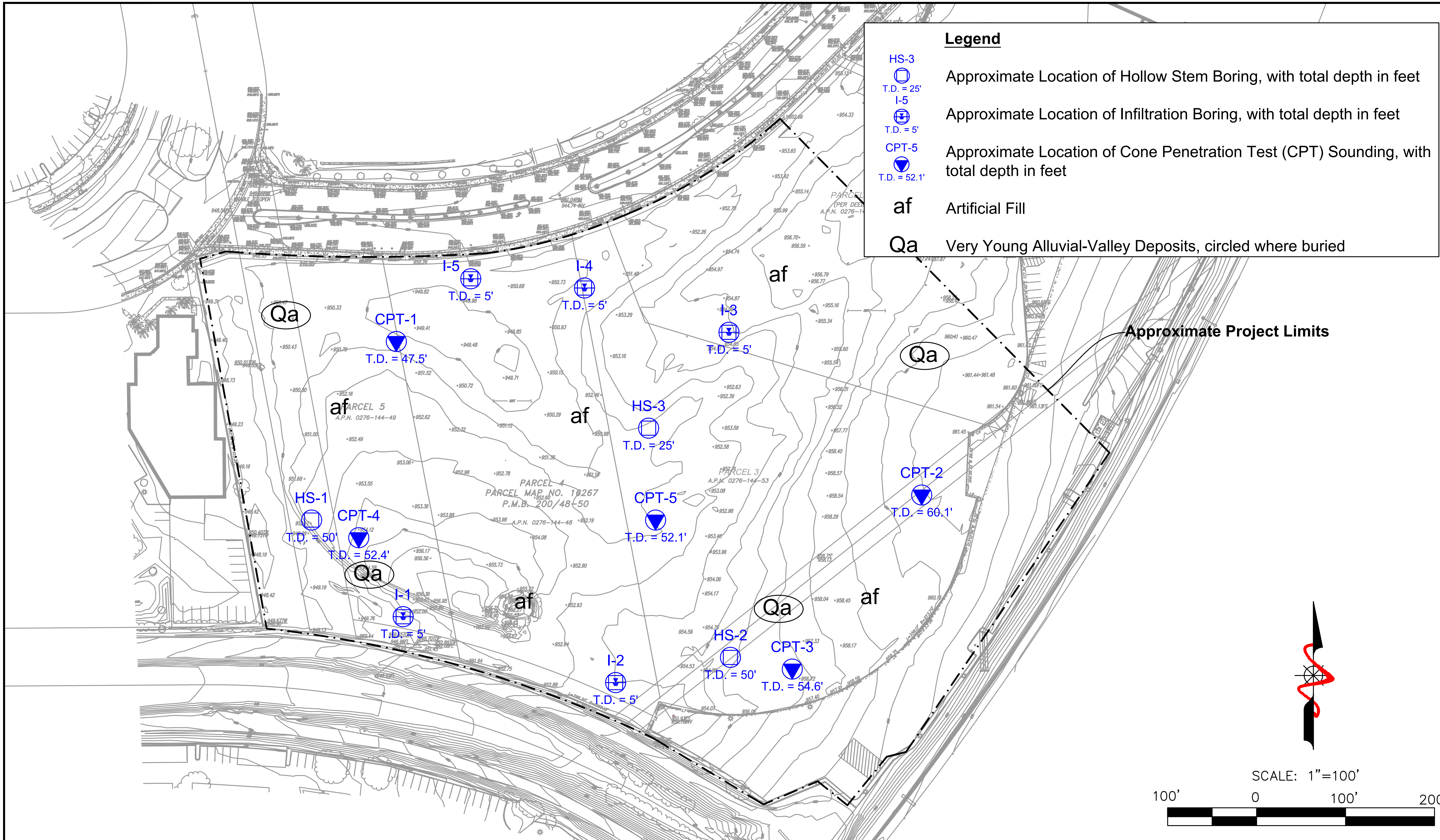
FIGURE 1
Site Location Map

PROJECT NAME	HPI-Ashley Way
PROJECT NO.	18078-01
ENG. / GEOL.	BTZ/KBC
SCALE	Not to Scale
DATE	July 2018



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

FIGURE 2
Boring Location Map

PROJECT NAME	HPI-Ashley Way
PROJECT NO.	18078-01
ENG. / GEOL.	BTZ/KBC
SCALE	1" = 100'
DATE	July 2018

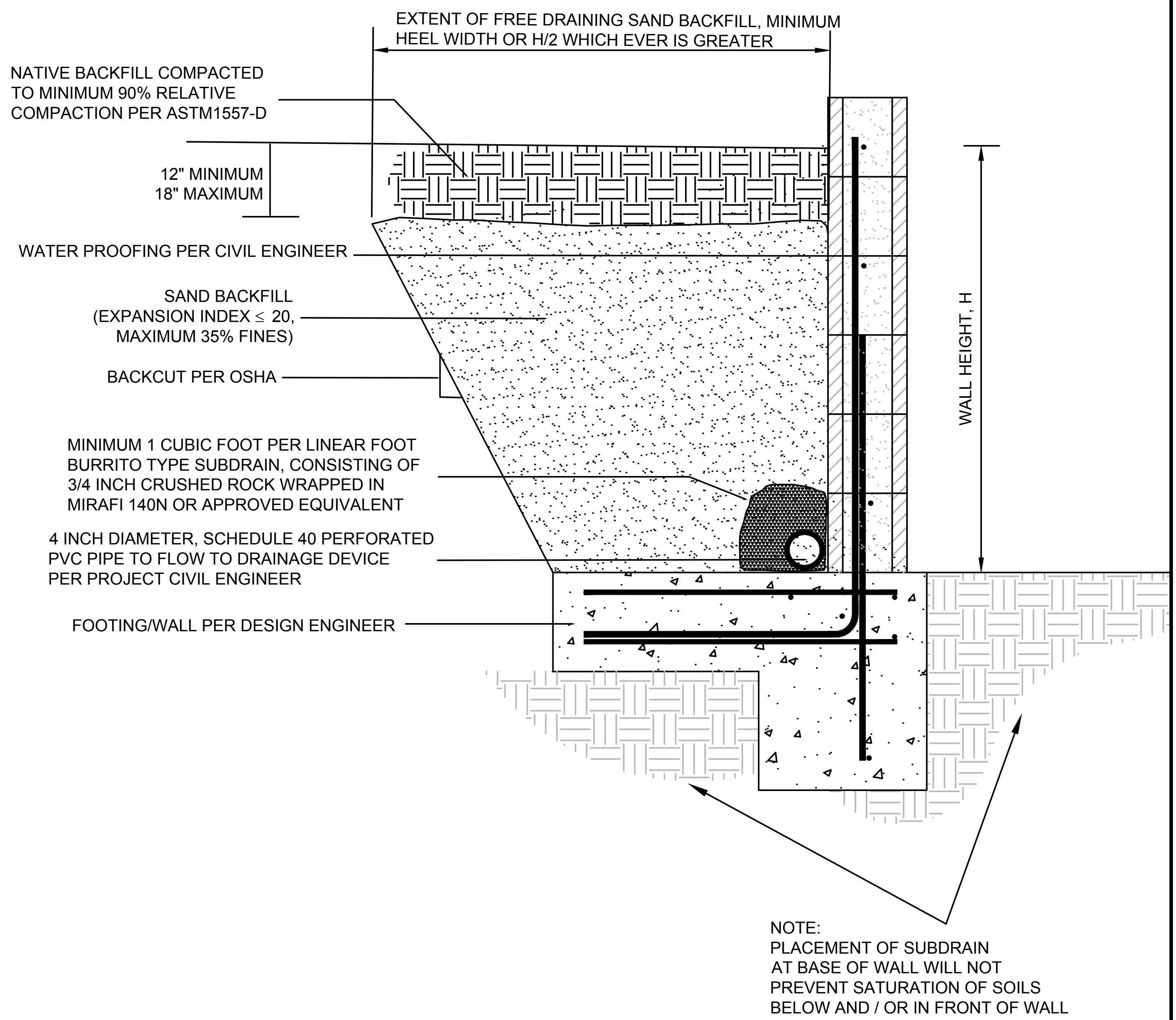


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FIGURE 3

Retaining Wall Backfill Detail

PROJECT NAME	Ashley Way
PROJECT NO.	18078-01
ENG. / GEOL.	BTZ/KBC
SCALE	Not to Scale
DATE	July 2018

Appendix A
References

APPENDIX A

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Appendix B
Boring/CPT Logs & Field Infiltration Data

Geotechnical Boring Log Borehole HS-1

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~953' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By BTZ DESCRIPTION	Type of Test
	0							**@0' to 10' Artificial Fill (af):** @0' Silty SAND with gravel: brown, dry; grass rootlets, trash debris	
950		B-1	R-1	17 33 50/4"	113.2	7.1	SM	@2.5' Gravelly Silty SAND: medium brown, moist, very dense; well rounded gravel > 1/8 inch	RV
	5		SPT-1	9 10 24		9.3		@5' Silty SAND: medium brown, moist, dense; traces of gravel	
945			R-2	17 20 20	117.0	3.6		@7.5' Silty SAND: medium brown, slightly moist, dense	
								@10' to T.D. Very Young Alluvial-Valley Deposits (Qa):	
	10		SPT-2	3 3 4		7.0		@10' Silty SAND: medium brown, moist, loose; trace gravel	-200
940									
	15		R-3	5 7 8	113.8	2.8	SP	@15' SAND: gray to medium brown, slightly moist, medium dense; medium grained, subangular sand grains; trace gravel	
935									
	20		SPT-3	2 2 3			CL-ML	@20' Sandy Silty CLAY: olive brown, moist, medium stiff; micaceous	-200 AL
930									
	25		R-4	5 10 16	104.1	2.9	SP	@25' SAND with gravel: gray to brown, slightly moist, medium dense	
925									
	30								

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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

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

Last Edited: 6/12/2018

Geotechnical Boring Log Borehole HS-1

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~953' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	8 12 15		7.3	SP-SM	@30' SAND with silt: medium brown and gray, moist, dense @30.5' ~2 inch CLAY layer	
920									
	35		R-5	7 11 16	96.6	26.9	SM-ML	@35' Silty SAND interbedded with Sandy SILT: gray and brown, very moist, medium dense/very stiff; fine sand	-200
915									
	40		SPT-5	14 16 21		7.4	SP	@40' SAND: gray and brown, moist, dense; angular grains; trace gravel	
910									
	45		R-6	5 16 50/4"	112.0	1.4	SM	@45' Silty SAND interbedded with Silty-Clayey SAND: gray to brown, dry to slightly moist, very dense	
905									
	50		SPT-6	22 28 33		5.2	SP-SM	@50' SAND with silt and gravel: gray to brown, slightly moist, very dense	
900								Total Depth = 51.5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
	55								
895									
	60								

Geotechnical, Inc.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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

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

Geotechnical Boring Log Borehole HS-2

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~955' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to 7.5' Artificial Fill (af):**	
							SM	@0' Silty SAND to Sandy SILT with gravel: light brown, dry	
		B-1	R-1	24 27 22	126.6	7.6	SC-SM	@2.5' Silty, Clayey SAND with gravel: dark brown, moist, medium dense; subangular grains	EI CR MD
950	5		SPT-1	9 8 6		1.6		@5' same as above; dry to slightly moist, gravel <1/2 inch	
								@7.5' to T.D. Very Young Alluvial-Valley Deposits (Qa)	
			R-2	3 4 4	103.6	11.6	SC-SM	@7.5' Silty Clayey SAND: olive brown, moist, loose; some white mineralized root casts	CN -200
945	10		SPT-2	3 2 3		9.6	SM	@10' Silty SAND: brown, moist, loose	-200
940	15		R-3	4 6 7	92.3	9.5		@15' Silty SAND: medium brown, moist, medium dense	-200
935	20		SPT-3	6 5 9		3.2	SP	@20' SAND: medium brown, slightly moist, medium dense; generally coarse grained	
930	25		R-4	6 12 25	104.7	2.5		@25' SAND: medium brown to gray, slightly moist, dense	
	30								

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

Last Edited: 6/12/2018

Geotechnical Boring Log Borehole HS-2

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~955' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	8 10 14			SP	@30' SAND: gray, slightly moist, dense	
920	35		R-5	17 28 33	111.3	3.9	SP-SM	@35' SAND transitioning to Silty SAND: gray, slightly moist, dense; medium grained	
915	40		SPT-5	10 14 16		2.9	SP	@40' SAND: gray and brown, slightly moist, dense; ~2 inch silt layer present, slightly moist	
910	45		R-6	16 24 42	99.0	2.1		@45' SAND: gray, slightly moist, very dense	
905	50		SPT-6	50/4"		1.2		@50' SAND with trace gravel: gray, dry to slightly moist, very dense	
								Total Depth = 51.5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
900	55								
	60								

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- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~952' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to TD <u>Very Young Alluvial-Valley Deposits (Qa)</u>:**	
950								@0' Gravelly Silty SAND: light brown, dry; scattered cobbles ~6 inch	
		B-1	SPT-1	6 4 5		5.0	SP-SM	@2.5' SAND with silt: brown, slightly moist, medium dense; trace gravel	
	5		R-1	5 6 12	108.1	11.5	SM	@5' Silty SAND: brown, moist, medium dense; some coarse sand, root casts	
945			SPT-2	2 3 3		20.7	CL	@7.5' Clay: pale olive, very moist, medium stiff to stiff; micaceous	-200 AL
	10		R-2	4 10 11	99.5	3.0	SP	@10' SAND: gray and brown, slightly moist, medium dense	
940									
	15		SPT-3	3 3 4		20.7	ML	@15' Sandy SILT: medium to dark brown, moist, stiff	-200
935									
	20		R-3	11 18 22	107.5	1.8	SM	@20' Silty SAND with gravel: gray and brown, dry to slightly moist, dense; scattered gravel >1 inch	
930									
	25		SPT-4	7 11 9		5.3		@25' Silty SAND: gray and brown, slightly moist, medium dense; mostly coarse grained	-200
925								Total Depth = 26.5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
	30								

Geotechnical, Inc.

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G GRAB SAMPLE
SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

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

Last Edited: 6/12/2018

Geotechnical Boring Log Borehole I-1

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~951' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
950	0							**@0' to TD - <u>Very Young Alluvial-Valley Deposits (Qa):</u>** @0' Silty SAND with gravel: light brown, very dry to dry; scattered cobbles ~6 inch; grass	
			SPT-1	5 5 4		1.9	SM	@2.5' Silty SAND with gravel: light brown, slightly moist, medium dense	
	5							@5' Silty SAND with gravel: light brown, slightly moist, medium dense	
945								Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
940	10								
935	15								
930	20								
925	25								
	30								

Last Edited: 6/12/2018

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- CN CONSOLIDATION
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- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-2

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~954' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to TD - Very Young Alluvial-Valley Deposits (Qa):** @0' Silty SAND to Sandy SILT: light brown, very dry to dry, loose; scattered cobbles ~1/2 ft; grass	
			R-1	6 9 10	105.5	4.4	SM-ML	@2.5' Silty SAND to Sandy SILT: light brown, slightly moist, medium dense/stiff	
950	5							@5' Silty SAND to Sandy SILT: light brown, slightly moist	
945	10							Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
940	15								
935	20								
930	25								
925	30								

Geotechnical, Inc.

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- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 6/12/2018

Geotechnical Boring Log Borehole I-3

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~954' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By ARN Sampled By ARN Checked By BTZ DESCRIPTION	Type of Test
	0							**@0' to TD - <u>Very Young Alluvial-Valley Deposits (Qa):</u>** @0' Silty SAND with gravel: light brown, dry; scattered cobbles, grass	
			R-1	9 11 15	120.0	6.2	SM	@2.5' Silty SAND: reddish brown, slightly moist, medium dense	
950	5							@5' same as above; increased sand content, darker brown	
								Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
945	10								
940	15								
935	20								
930	25								
925	30								

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

Last Edited: 6/12/2018

Geotechnical Boring Log Borehole I-4

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~951' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
950	0							**@0' to TD - <u>Very Young Alluvial-Valley Deposits (Qa):</u>**	
							SM	@0' Silty SAND with gravel: light brown, dry, loose; scattered cobbles, grass	
			SPT-1	7 9 11		2.2		@1' Silty SAND: light brown	
							SP	@2.5' SAND: gray, slightly moist, medium dense; trace gravel	
	5							@5' same as above	
945								Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
	10								
940									
	15								
935									
	20								
930									
	25								
925									
	30								

Geotechnical, Inc.

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SAMPLE TYPES:
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SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

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

Last Edited: 6/12/2018

Geotechnical Boring Log Borehole I-5

Date: 6/4/2018	**Drilling Company:** 2R
Project Name: Howard-Ashley Way	**Type of Rig:** CME-75
Project Number: 18078-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~951' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By ARN
Sampled By ARN
Checked By BTZ

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
950	0							**@0' to TD - Very Young Alluvial-Valley Deposits (Qa):** @0' Silty SAND with gravel: light brown, dry; grass roots, refuse	
			R-1	13 18 14	120.5	7.5	SM-ML	@2.5' Sandy Silt to Silty SAND: medium brown, slightly moist to moist, medium dense/stiff; caliche root casts	
	5							@5' same as above	
945								Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 6/4/2018	
	10								
940									
	15								
935									
	20								
930									
	25								
925									
	30								

Last Edited: 6/12/2018

Geotechnical, Inc.

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- GROUNDWATER TABLE

TEST TYPES:
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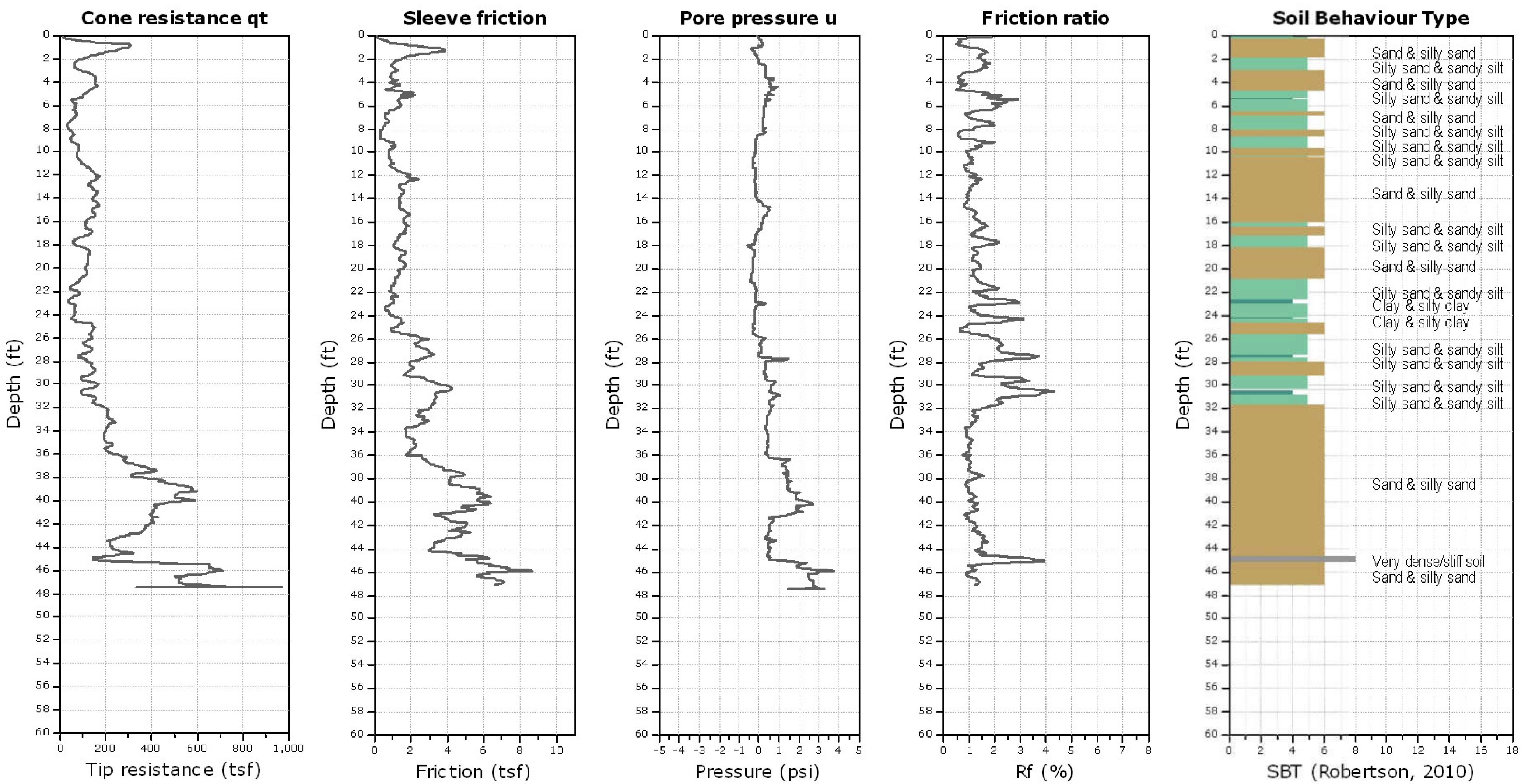
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

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

CPT-1

Project: **LGC Geotechnical, Inc.**
Location: **E. Cooley Drive & Ashley Way Colton, CA**

Total depth: 47.51 ft, Date: 6/26/2018
Cone Type: Vertek



CPeT-IT v.2.0.1.55 - CPTU data presentation & interpretation software - Report created on: 6/27/2018, 10:37:42 AM
Project file: C:\LGCColton6-18\Plot Data\Plots.cpt

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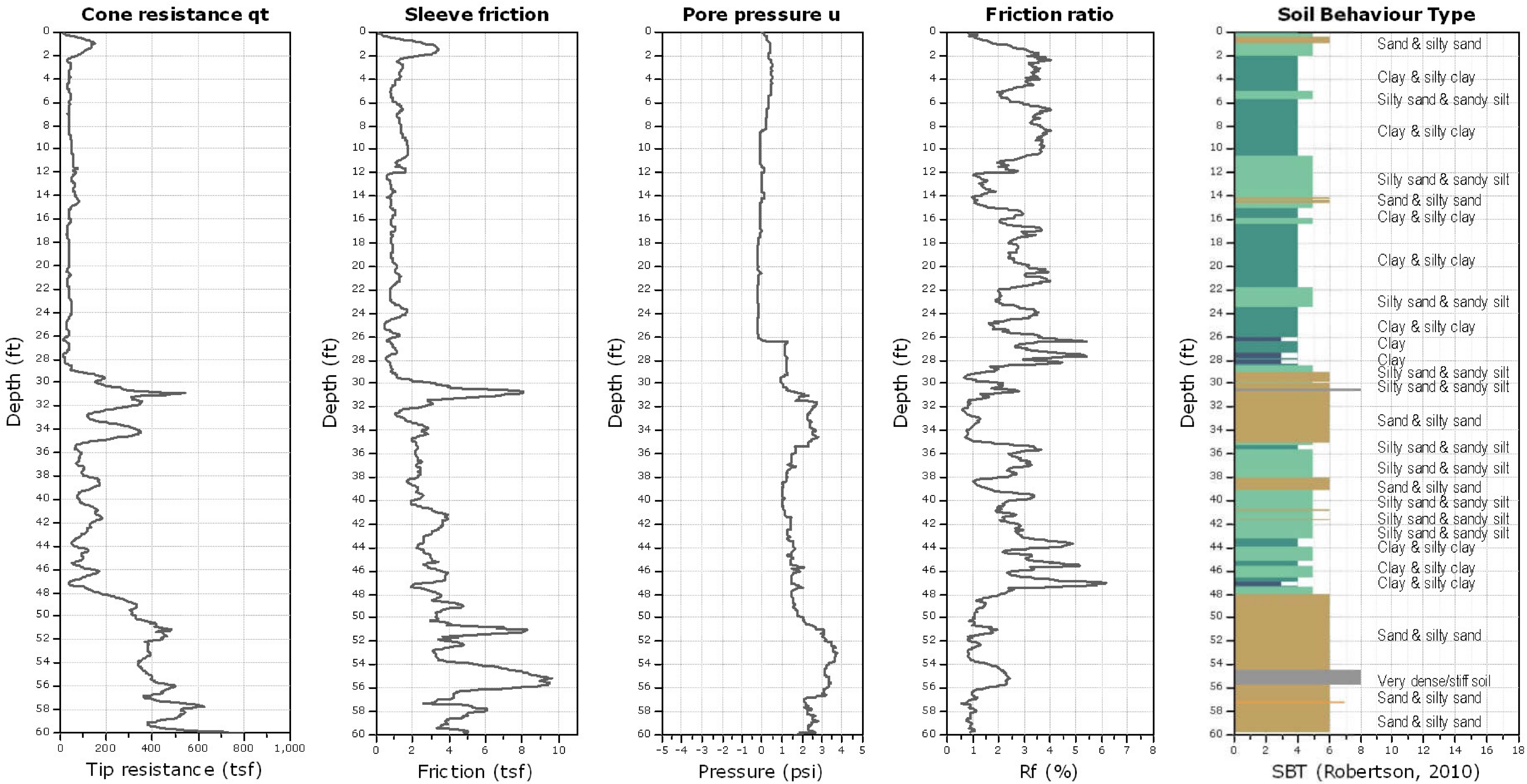
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

Project: LGC Geotechnical, Inc.
Location: E. Cooley Drive & Ashley Way Colton, CA

CPT-2
Total depth: 60.17 ft, Date: 6/26/2018
Cone Type: Vertek



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Project file: C:\LGCColton6-18\Plot Data\Plots.cpt

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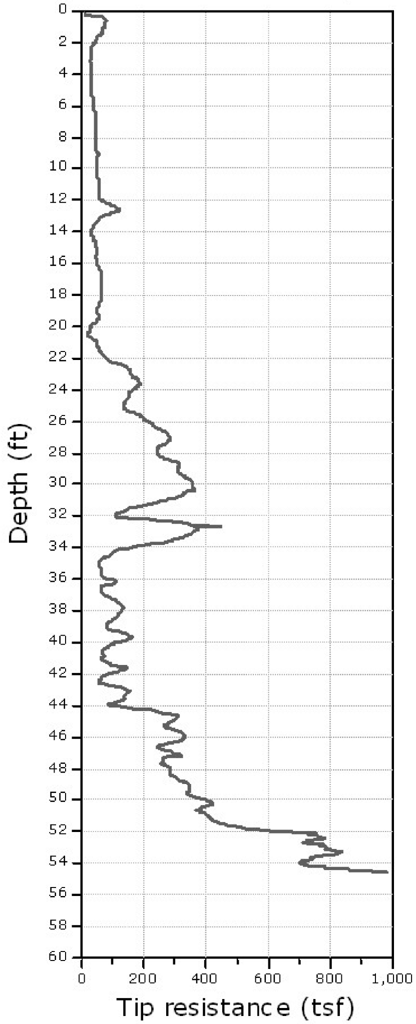
www.kehoetesting.com

CPT-3

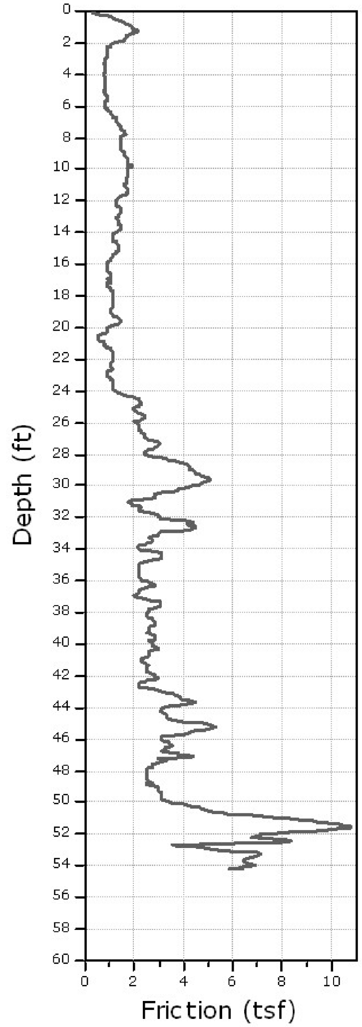
Project: LGC Geotechnical, Inc.
Location: E. Cooley Drive & Ashley Way Colton, CA

Total depth: 54.60 ft, Date: 6/26/2018
Cone Type: Vertek

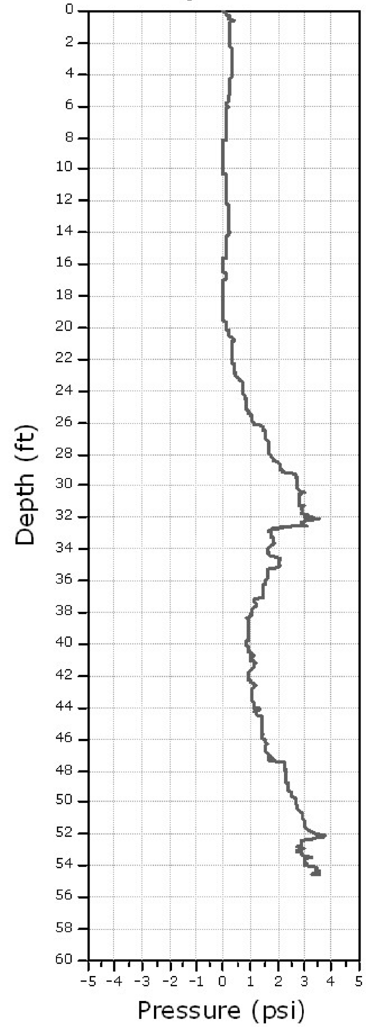
Cone resistance qt



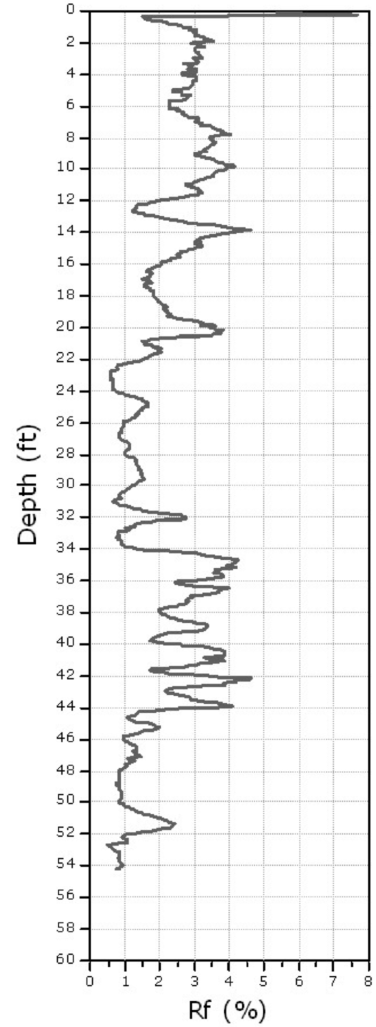
Sleeve friction



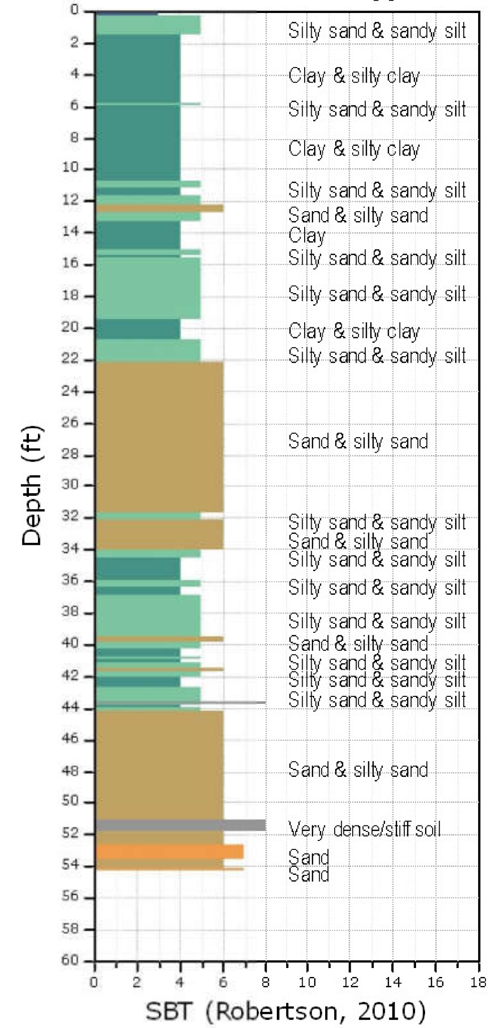
Pore pressure u



Friction ratio



Soil Behaviour Type



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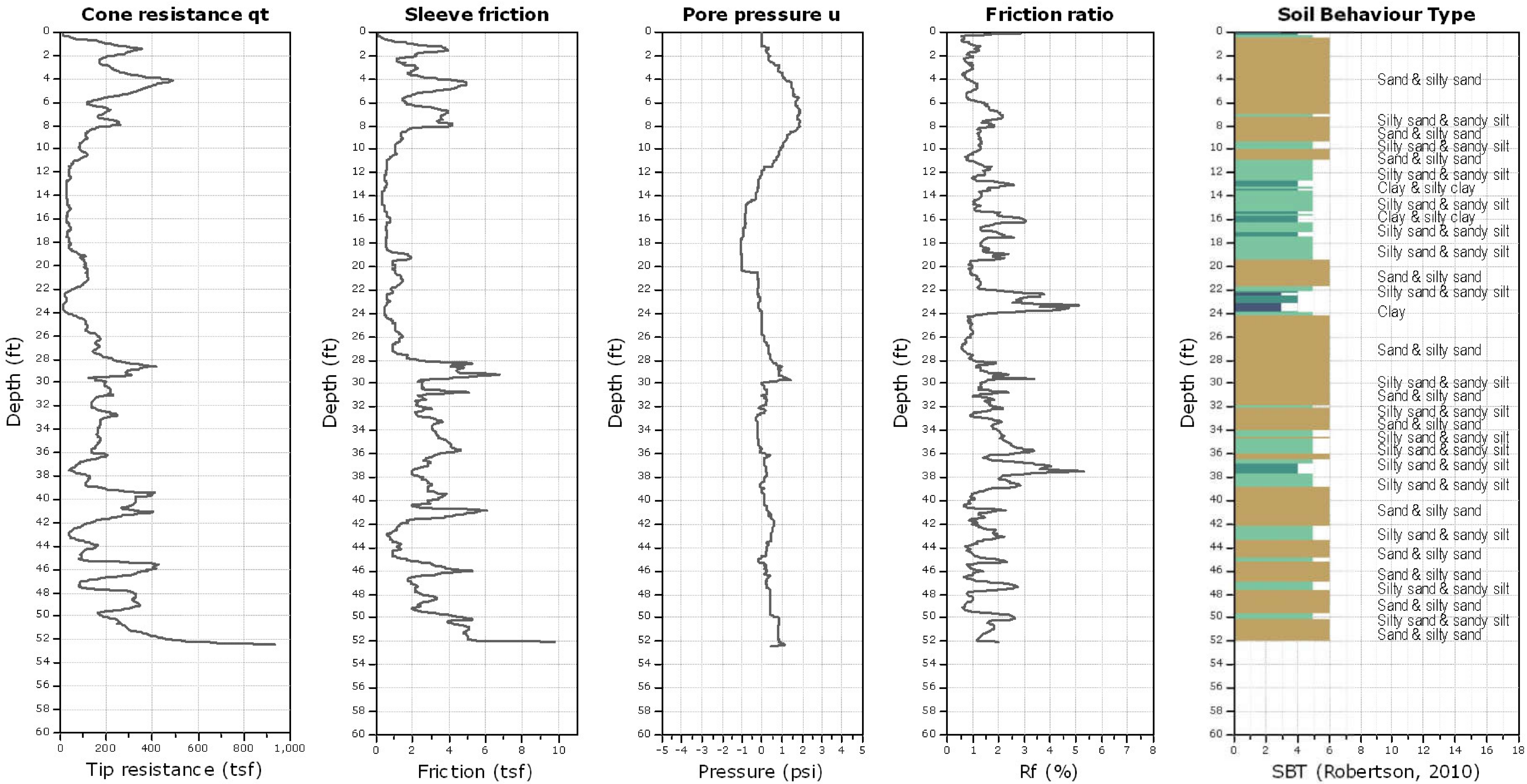
Project: LGC Geotechnical, Inc.
Location: E. Cooley Drive & Ashley Way Colton, CA

CPT-4
Total depth: 52.43 ft, Date: 6/26/2018
Cone Type: Vertek



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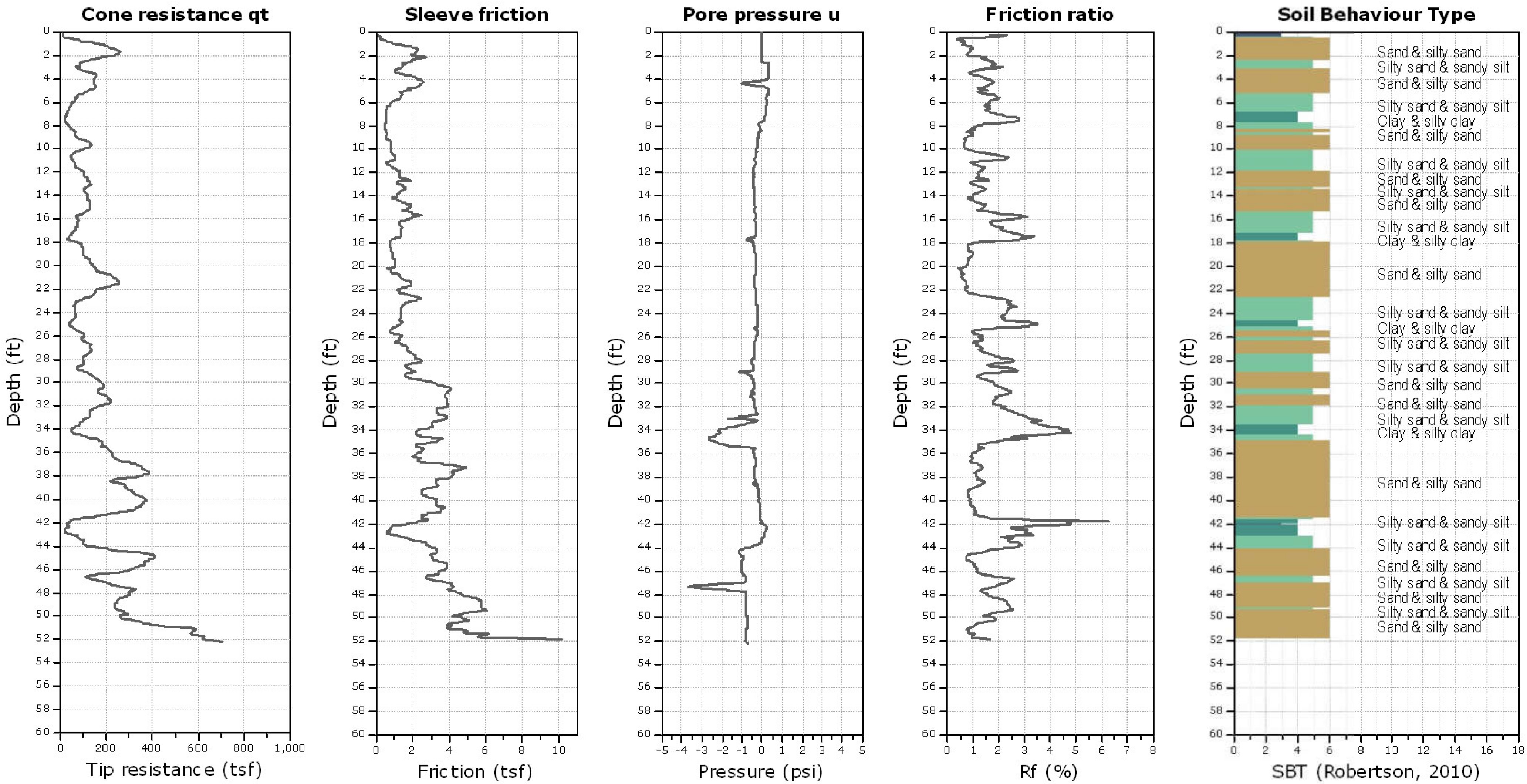
CPT-5

Project: LGC Geotechnical, Inc.
Location: E. Cooley Drive & Ashley Way Colton, CA

Total depth: 52.17 ft, Date: 6/26/2018
Cone Type: Vertek



Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name:	Howard-Ashley Way
Project Number:	18078-01
Date:	6/5/2018
Boring Number:	I-1

Test hole dimensions (if circular)

Boring Depth (feet)*:	4.79
Boring Diameter (inches):	8
Pipe Diameter (inches):	2

*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	12:20	12:45	25.0	3.13	4.40	1.28	yes
2	0:48	1:13	25.0	3.18	4.40	1.23	yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	1:14	1:24	10.0	3.10	3.90	0.80	6.6
2	1:25	1:35	10.0	2.80	3.55	0.75	5.1
3	1:37	1:47	10.0	2.85	3.53	0.68	4.6
4	1:49	1:59	10.0	2.90	3.53	0.63	4.3
5	2:00	2:10	10.0	2.90	3.54	0.64	4.4
6	2:12	2:22	10.0	2.95	3.54	0.59	4.1
7							
8							
9							
10							
11							
12							

Calculated Infiltration Rate (No factors of safety)	**4.1**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**2.1**

Sketch:

Notes:

Based on Guidelines from: San Bernardino 06/21/2013

Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

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

Project Name: Howard-Ashley Way
Project Number: 18078-01
Date: 6/5/2018
Boring Number: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5.29
Boring Diameter (inches):	8
Pipe Diameter (inches):	2

*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	12:28	12:53	25.0	3.60	4.15	0.55	yes
2	0:54	1:19	25.0	3.35	3.93	0.58	yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	1:20	1:30	10.0	3.50	3.73	0.23	1.5
2	1:31	1:41	10.0	3.43	3.65	0.23	1.4
3	1:42	1:52	10.0	3.20	3.43	0.23	1.3
4	1:52	2:02	10.0	3.43	3.70	0.28	1.7
5	2:03	2:13	10.0	3.43	3.68	0.25	1.6
6	2:14	2:24	10.0	3.68	3.90	0.23	1.6
7							
8							
9							
10							
11							
12							

Calculated Infiltration Rate (No factors of safety)	**1.6**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**0.8**

Sketch:

Notes:

Based on Guidelines from: San Bernardino 06/21/2013
Spreadsheet Revised on: 10/26/2016

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Howard-Ashley Way
Project Number: 18078-01
Date: 6/5/2018
Boring Number: I-3

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5.33
Boring Diameter (inches):	8
Pipe Diameter (inches):	2

*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	9:16	9:41	25.0	3.57	4.82	1.25	yes
2	9:43	10:08	25.0	3.62	4.82	1.20	yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	10:09	10:19	10.0	3.12	4.42	1.30	9.0
2	10:20	10:30	10.0	3.39	4.22	0.83	5.8
3	10:31	10:41	10.0	3.34	4.07	0.73	4.9
4	10:41	10:51	10.0	3.14	3.84	0.70	4.2
5	10:51	11:01	10.0	3.09	3.84	0.75	4.4
6	11:02	11:12	10.0	3.07	3.87	0.80	4.7
7							
8							
9							
10							
11							
12							

Calculated Infiltration Rate (No factors of safety)	**4.7**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**2.4**

Sketch:

Notes:

Based on Guidelines from: San Bernardino 06/21/2013

Spreadsheet Revised on: 10/26/2016

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Howard-Ashley Way
Project Number: 18078-01
Date: 6/5/2018
Boring Number: I-4

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5.04
Boring Diameter (inches):	8
Pipe Diameter (inches):	2

*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	9:03	9:28	25.0	3.45	4.65	1.20	yes
2	9:29	9:53	24.0	3.30	4.65	1.35	yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	9:54	10:04	10.0	3.25	3.95	0.70	5.2
2	10:05	10:15	10.0	3.35	4.00	0.65	5.1
3	10:16	10:26	10.0	3.08	3.75	0.68	4.5
4	10:27	10:37	10.0	2.95	3.65	0.70	4.4
5	10:38	10:48	10.0	3.35	3.90	0.55	4.2
6	10:49	10:59	10.0	3.15	3.78	0.63	4.3
7							
8							
9							
10							
11							
12							

Calculated Infiltration Rate (No factors of safety)	**4.3**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**2.2**

Sketch:

Notes:

Based on Guidelines from: San Bernardino 06/21/2013

Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Howard-Ashley Way
Project Number: 18078-01
Date: 6/5/2018
Boring Number: I-5

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5.41
Boring Diameter (inches):	8
Pipe Diameter (inches):	2

*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	8:55	9:20	25.0	3.52	4.32	0.80	yes
2	9:23	9:48	25.0	3.52	4.22	0.70	yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	9:49	9:59	10.0	3.32	3.72	0.40	2.3
2	10:00	10:12	12.0	3.72	4.07	0.35	2.1
3	10:13	10:23	10.0	3.42	3.72	0.30	1.8
4	10:24	10:34	10.0	3.52	5.22	1.70	16.9
5	10:35	10:45	10.0	3.29	3.64	0.35	2.0
6	10:45	10:55	10.0	3.64	3.97	0.33	2.2
7							
8							
9							
10							
11							
12							

Calculated Infiltration Rate (No factors of safety)	**2.2**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**1.1**

Sketch:

Notes:

Based on Guidelines from: San Bernardino 06/21/2013
Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on driven samples obtained from the test borings. The results of these tests are presented in the boring logs.

Fines Content: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve).

Sample Location	**Description**	**% Passing # 200 Sieve**
HS-1 @ 10 ft	Silty Sand	31
HS-1 @ 20 ft	Sandy Silty Clay	52
HS-1 @ 35 ft	Sandy Silt	90
HS-2 @ 7.5 ft	Silty Clayey Sand	37
HS-2 @ 10 ft	Silty Sand	44
HS-2 @ 15 ft	Silty Sand	39
HS-3 @ 7.5 ft	Clay	96
HS-3 @ 15 ft	Sandy Silt	68
HS-3 @ 25 ft	Silty Sand	22

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

Sample Location	**Liquid Limit (%)**	**Plastic Limit (%)**	**Plasticity Index (%)**	**USCS Soil Classification**
HS-1 @ 20 ft	28	21	7	CL-ML
HS-3 @ 7.5	42	23	19	CL

Laboratory Test Results

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

Sample Location	Expansion Index	Expansion Potential*
HS-2 @ 3-6 ft	3	Very Low

* Per ASTM D4829

Consolidation: Consolidation tests were performed per ASTM D2435. Samples (2.4 inches in diameter and 1 inch in height) were placed in a consolidometer and increasing loads were applied. The samples were allowed to consolidate under "double drainage" and total deformation for each loading step was recorded. The percent consolidation for each load step was recorded as the ratio of the amount of vertical compression to the original sample height. The consolidation pressure curve is provided in this Appendix.

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

Sample Location	Sample Description	Maximum Dry Density (pcf)*	Optimum Moisture Content (%)*
HS-1 @ 3-5 ft	Dark Brown Silty-Clayey Sand	137.5	7.0

*Includes Rock Correction Factor

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

Sample Location	Sulfate Content, %
HS-2 @ 3-6 ft	< 0.01

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

Sample Location	Chloride Content, ppm
HS-2 @ 3-6 ft	10

Laboratory Test Results

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-2 @ 3-6 ft	8.4	5,875

R-value Test: R-value test was performed in general accordance with California Test Method 301. The plot is attached.

Sample No.	R-Value
HS-1 @ 3-5 ft	81

ATTERBERG LIMITS

ASTM D 4318

Project Name:	Colton	Tested By:	R. Manning	Date:	06/27/18
Project No. :	18078-01	Input By:	J. Ward	Date:	07/12/18
Boring No.:	HS-1	Checked By:	J. Ward		
Sample No.:	SPT-3	Depth (ft.)	20.0		

Soil Identification: Olive brown sandy silty clay s(CL-ML)

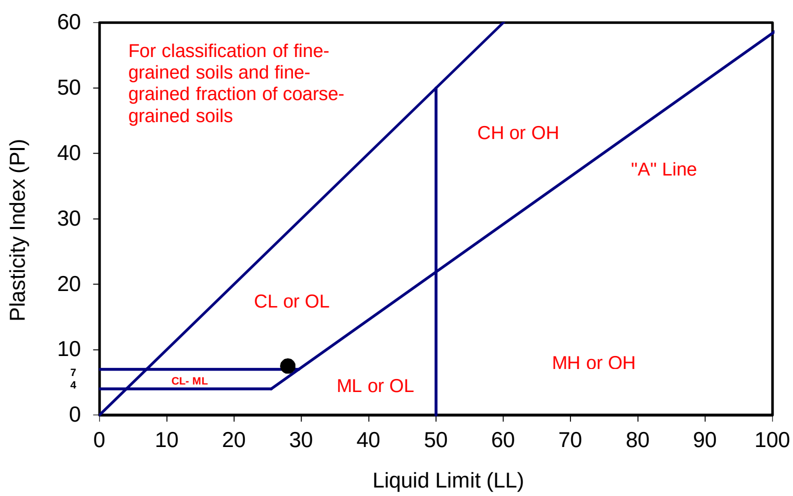
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			35	24	17	
Wet Wt. of Soil + Cont. (g)	18.70	19.24	29.21	27.86	28.92	
Dry Wt. of Soil + Cont. (g)	17.43	17.98	25.92	24.77	25.49	
Wt. of Container (g)	11.30	11.77	13.56	13.55	13.54	
Moisture Content (%) [Wn]	20.72	20.29	26.62	27.54	28.70	

Liquid Limit	**28**
Plastic Limit	**21**
Plasticity Index	**7**
Classification	**CL-ML**

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

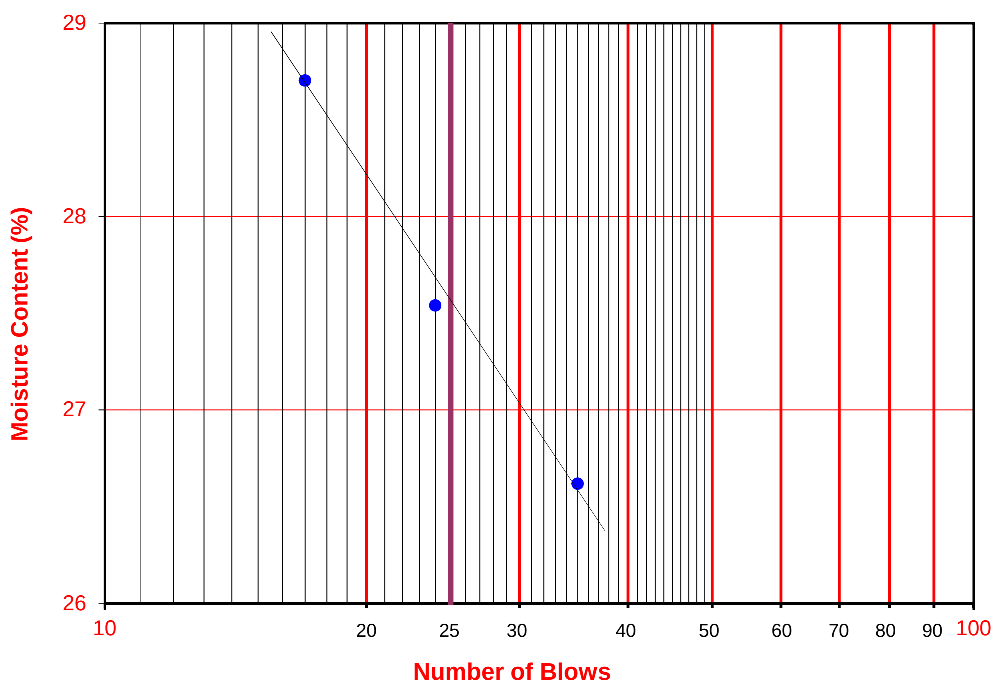
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



ATTERBERG LIMITS

ASTM D 4318

Project Name:	Colton	Tested By:	A. Santos	Date:	06/26/18
Project No. :	18078-01	Input By:	J. Ward	Date:	07/12/18
Boring No.:	HS-3	Checked By:	J. Ward		
Sample No.:	SPT-2	Depth (ft.)	7.5		
Soil Identification:	Pale olive lean clay (CL)				

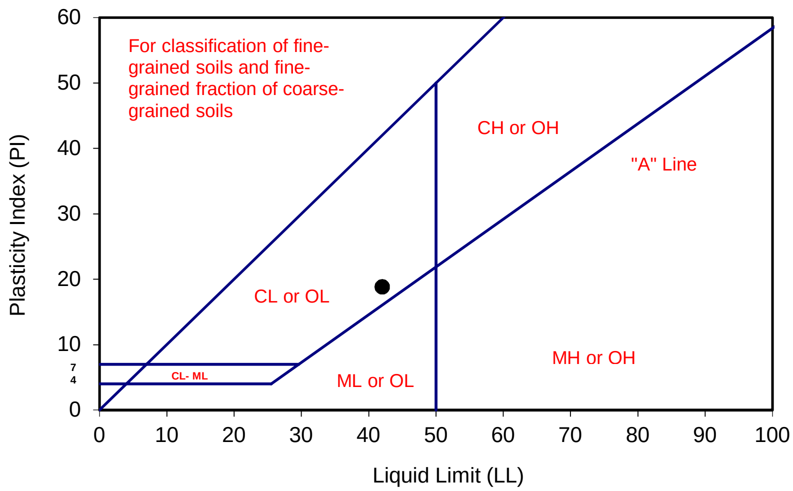
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			30	23	17	
Wet Wt. of Soil + Cont. (g)	9.02	9.32	17.16	20.24	17.60	
Dry Wt. of Soil + Cont. (g)	7.53	7.76	12.49	14.56	12.60	
Wt. of Container (g)	1.07	1.05	1.06	1.04	1.09	
Moisture Content (%) [Wn]	23.07	23.25	40.86	42.01	43.44	

Liquid Limit	**42**
Plastic Limit	**23**
Plasticity Index	**19**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 16.06

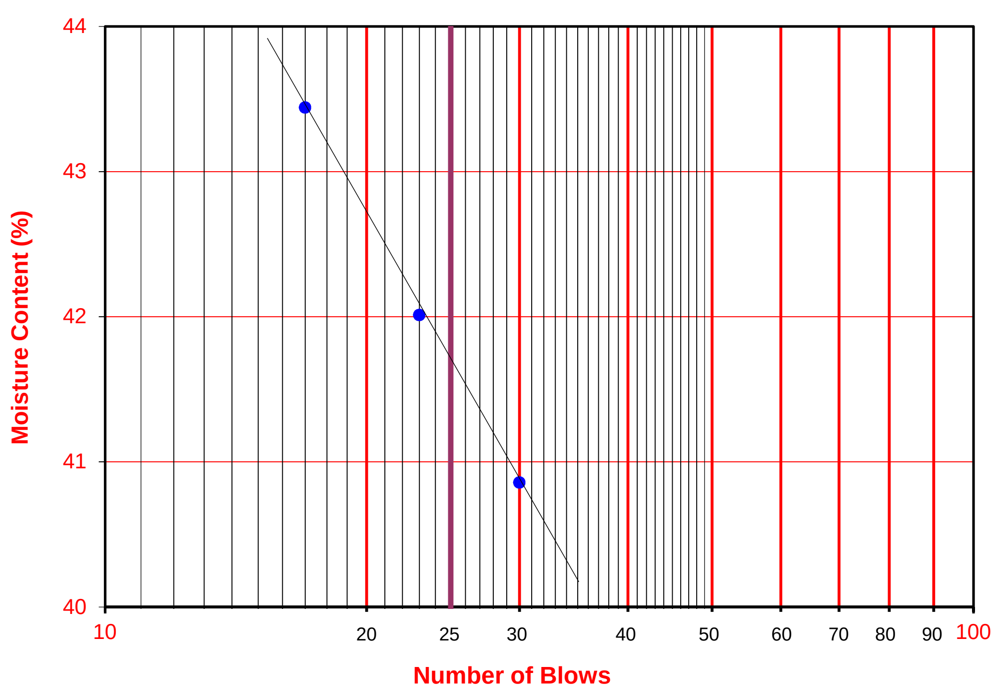
One - Point Liquid Limit Calculation

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

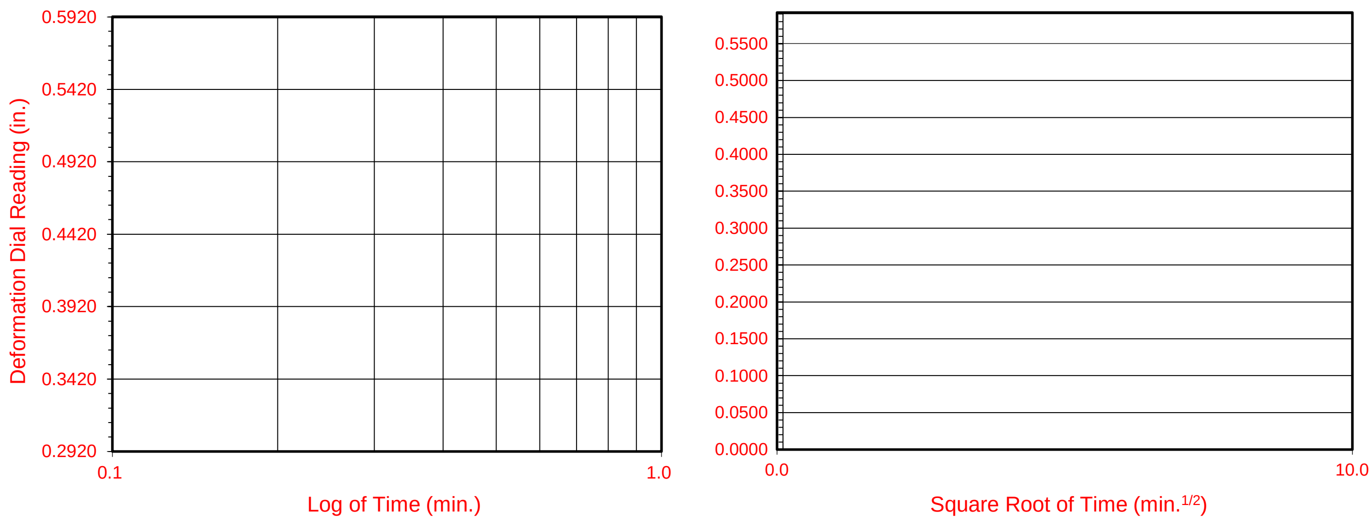


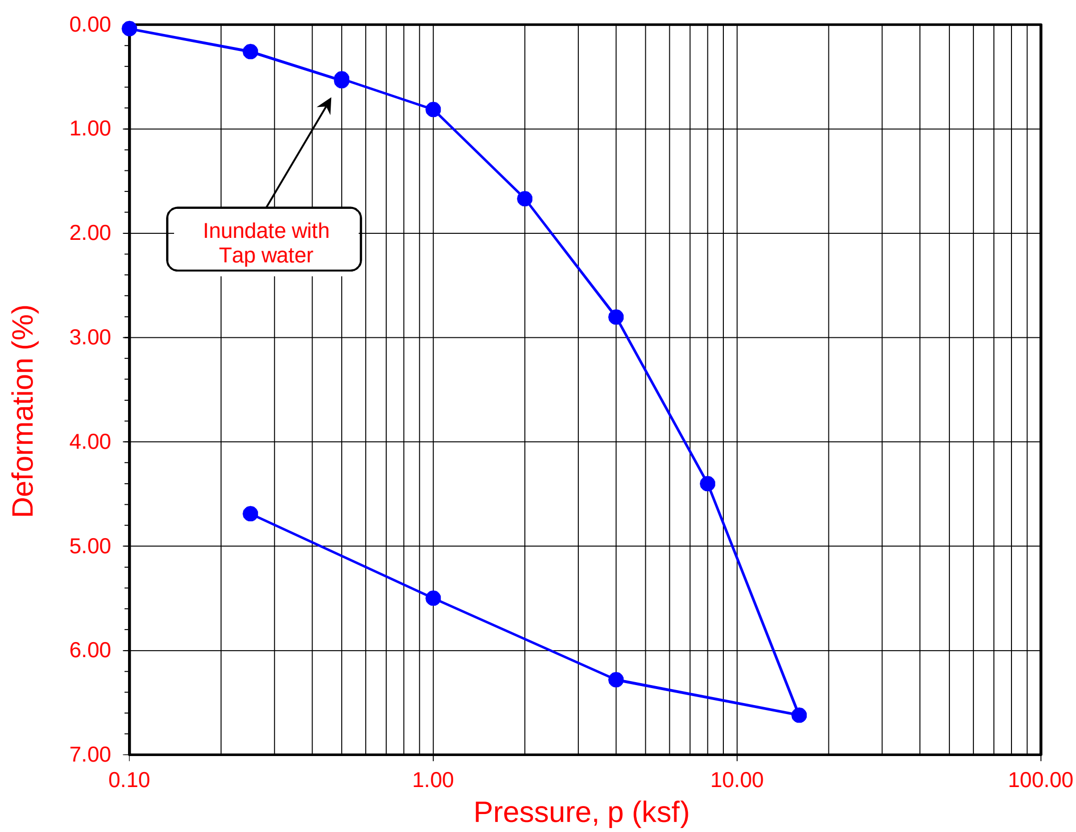
PROCEDURES USED

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



Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-2	**R-2**	**7.5**	**11.6**	**21.5**	**103.6**	**103.7**	**0.627**	**0.551**	**50**	**93**

Soil Identification: Olive brown silty, clayey sand (SC-SM)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 18078-01

Colton

07-18

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Colton	Tested By:	S. Felter	Date:	06/25/18
Project No.:	18078-01	Checked By:	J. Ward	Date:	07/12/18
Location:	HS-2	Depth (ft.):	3-6		
Sample No.:	B-1				
Soil Identification:	Dark brown silty, clayey sand (SC-SM)				

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0030
Wt. Comp. Soil + Mold (g)	600.30	447.43
Wt. of Mold (g)	180.90	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	842.70	628.33
Dry Wt. of Soil + Cont. (g)	780.30	569.23
Wt. of Container (g)	0.00	180.90
Moisture Content (%)	8.00	15.22
Wet Density (pcf)	126.5	134.6
Dry Density (pcf)	117.1	116.8
Void Ratio	0.439	0.444
Total Porosity	0.305	0.307
Pore Volume (cc)	63.2	63.8
Degree of Saturation (%) [S_{meas}]	**49.2**	92.6

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
06/25/18	11:10	1.0	0	0.0190
06/25/18	11:20	1.0	10	0.0190
Add Distilled Water to the Specimen				
06/25/18	14:02	1.0	162	0.0220
06/26/18	6:45	1.0	1165	0.0220
06/26/18	7:55	1.0	1235	0.0220

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**3**

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Colton	Tested By:	O. Figueroa	Date:	06/27/18
Project No.:	18078-01	Input By:	J. Ward	Date:	07/12/18
Boring No.:	HS-2	Depth (ft.):	3-6		
Sample No.:	B-1				
Soil Identification:	Dark brown silty, clayey sand (SC-SM)				

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

			Scalp Fraction (%)		
Preparation Method:	X	Moist	#3/4		Rammer Weight (lb.) = 10.0
		Dry	#3/8		Height of Drop (in.) = 18.0
Compaction Method	X	Mechanical Ram	#4	9.5	Mold Volume (ft³) 0.03330
		Manual Ram			

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3916	4038	3983			
Weight of Mold (g)	1848	1848	1848			
Net Weight of Soil (g)	2068	2190	2135			
Wet Weight of Soil + Cont. (g)	446.1	455.6	468.8			
Dry Weight of Soil + Cont. (g)	427.2	427.0	430.1			
Weight of Container (g)	39.5	39.1	39.1			
Moisture Content (%)	4.87	7.37	9.90			
Wet Density (pcf)	136.9	145.0	141.3			
Dry Density (pcf)	130.5	135.0	128.6			

Maximum Dry Density (pcf)	135.0	**Optimum Moisture Content (%)**	7.5
Corrected Dry Density (pcf)	137.5	**Corrected Moisture Content (%)**	7.0

X **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

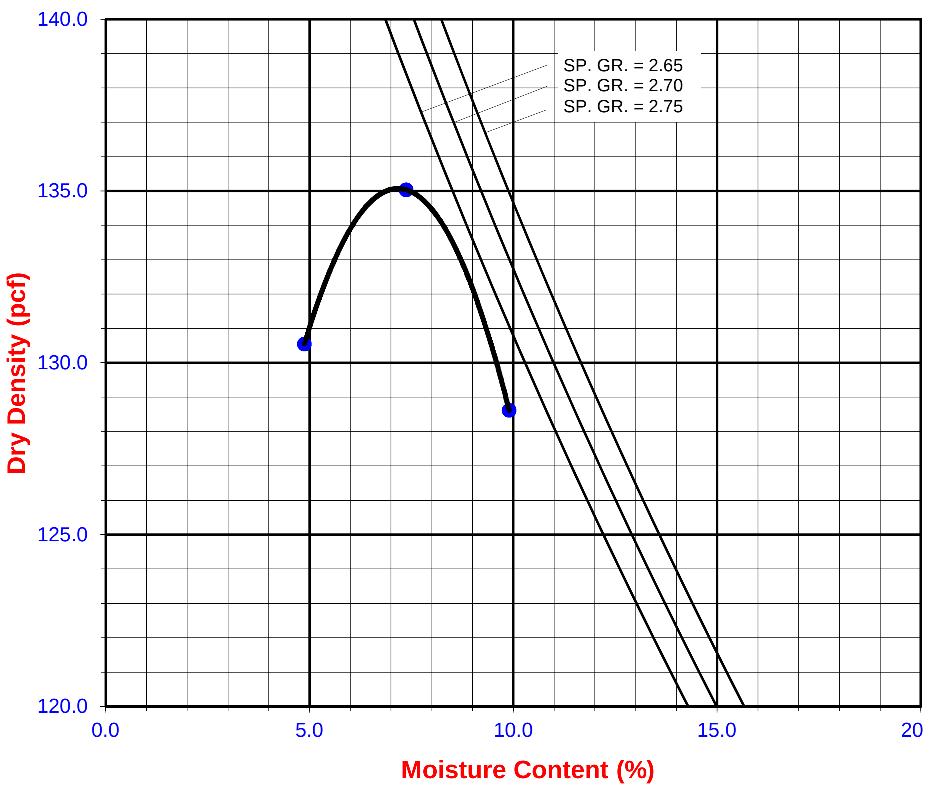
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Colton	Tested By :	A. Santos	Date:	06/27/18
Project No. :	18078-01	Data Input By:	J. Ward	Date:	07/12/18
Boring No.:	HS-2	Depth (ft.) :	3-6		
Sample No. :	B-1				

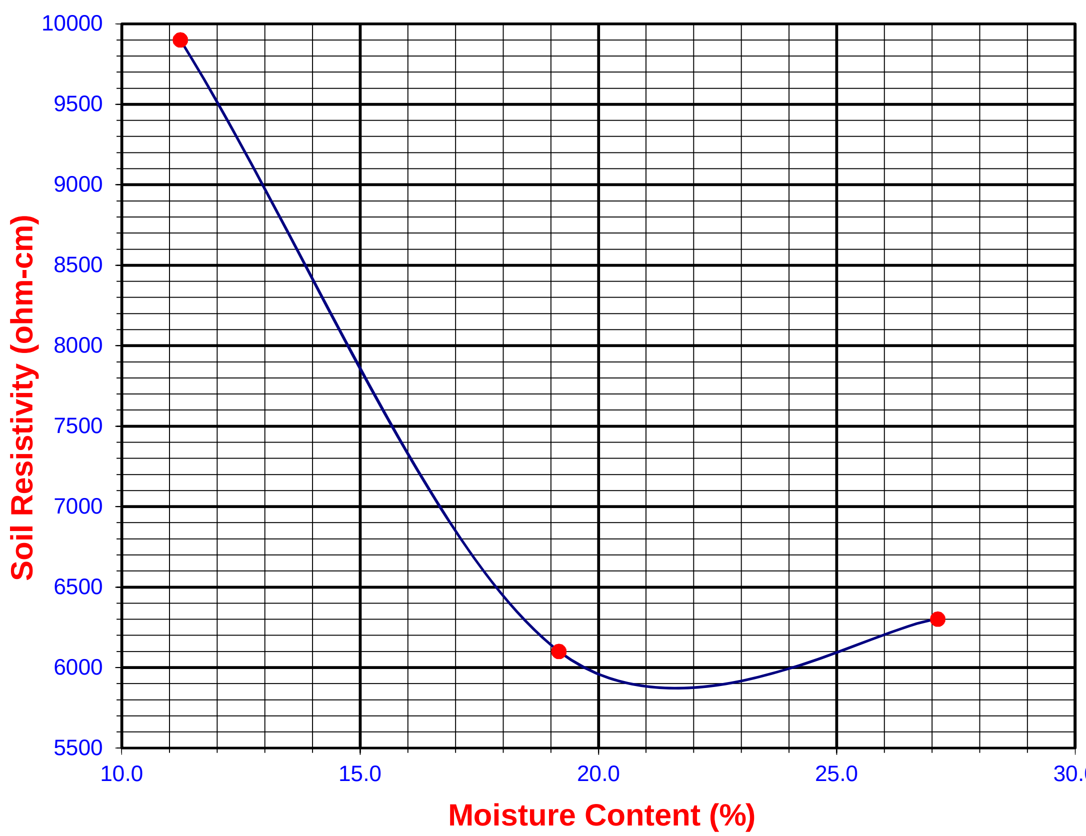
Soil Identification:* Dark brown SC-SM

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	11.23	9900	9900
2	20	19.17	6100	6100
3	30	27.12	6300	6300
4				
5				

Moisture Content (%) (MCi)	3.28
Wet Wt. of Soil + Cont. (g)	167.09
Dry Wt. of Soil + Cont. (g)	163.02
Wt. of Container (g)	39.04
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
5875	**21.6**	**98**	**10**	**8.42**	**22.8**



R-VALUE TEST RESULTS

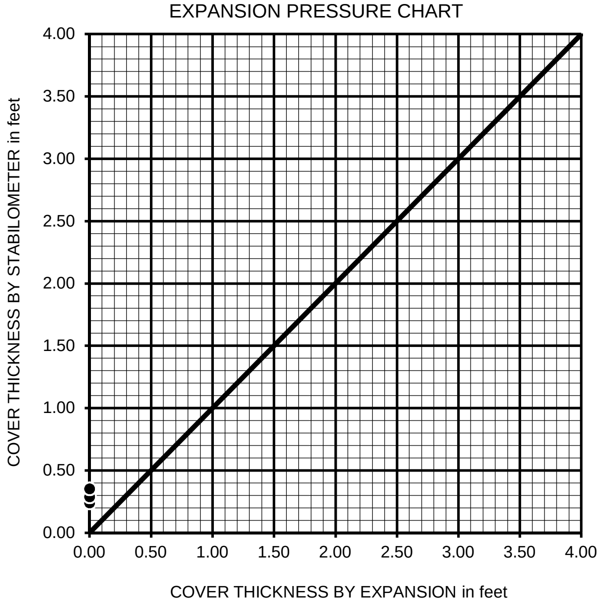
DOT CA Test 301

PROJECT NAME:	Colton	PROJECT NUMBER:	18078-01
BORING NUMBER:	HS-1	DEPTH (FT.):	3-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	S. Felter
SAMPLE DESCRIPTION:	Brown silty sand with gravel (SM)g	DATE COMPLETED:	6/26/2018

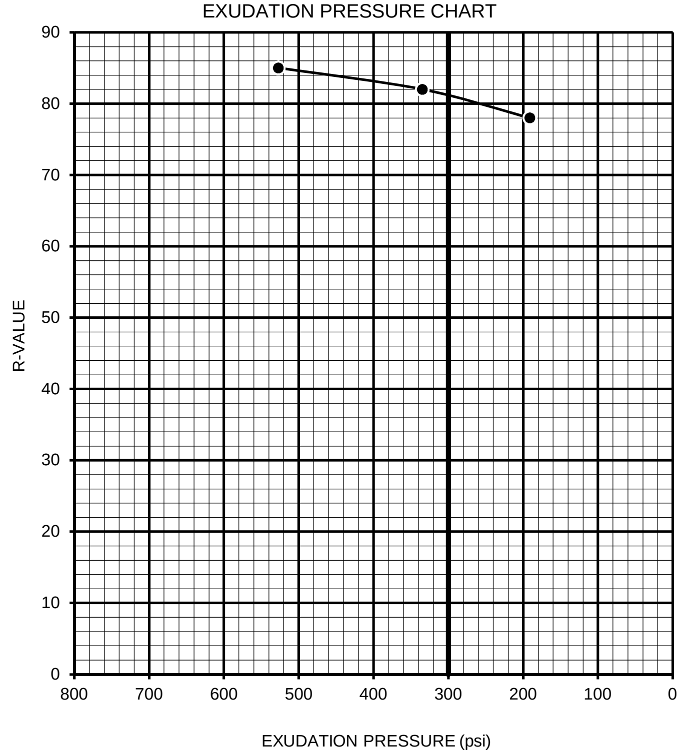
TEST SPECIMEN	**a**	**b**	**c**
MOISTURE AT COMPACTION %	7.0	7.4	7.9
HEIGHT OF SAMPLE, Inches	2.54	2.49	2.53
DRY DENSITY, pcf	129.2	132.0	129.3
COMPACTOR PRESSURE, psi	350	350	350
EXUDATION PRESSURE, psi	527	335	191
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	15	19	22
TURNS DISPLACEMENT	4.19	4.20	4.51
R-VALUE UNCORRECTED	85	82	78
R-VALUE CORRECTED	85	82	78

DESIGN CALCULATION DATA	**a**	**b**	**c**
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.24	0.29	0.35
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	81
EQUILIBRIUM R-VALUE:	81

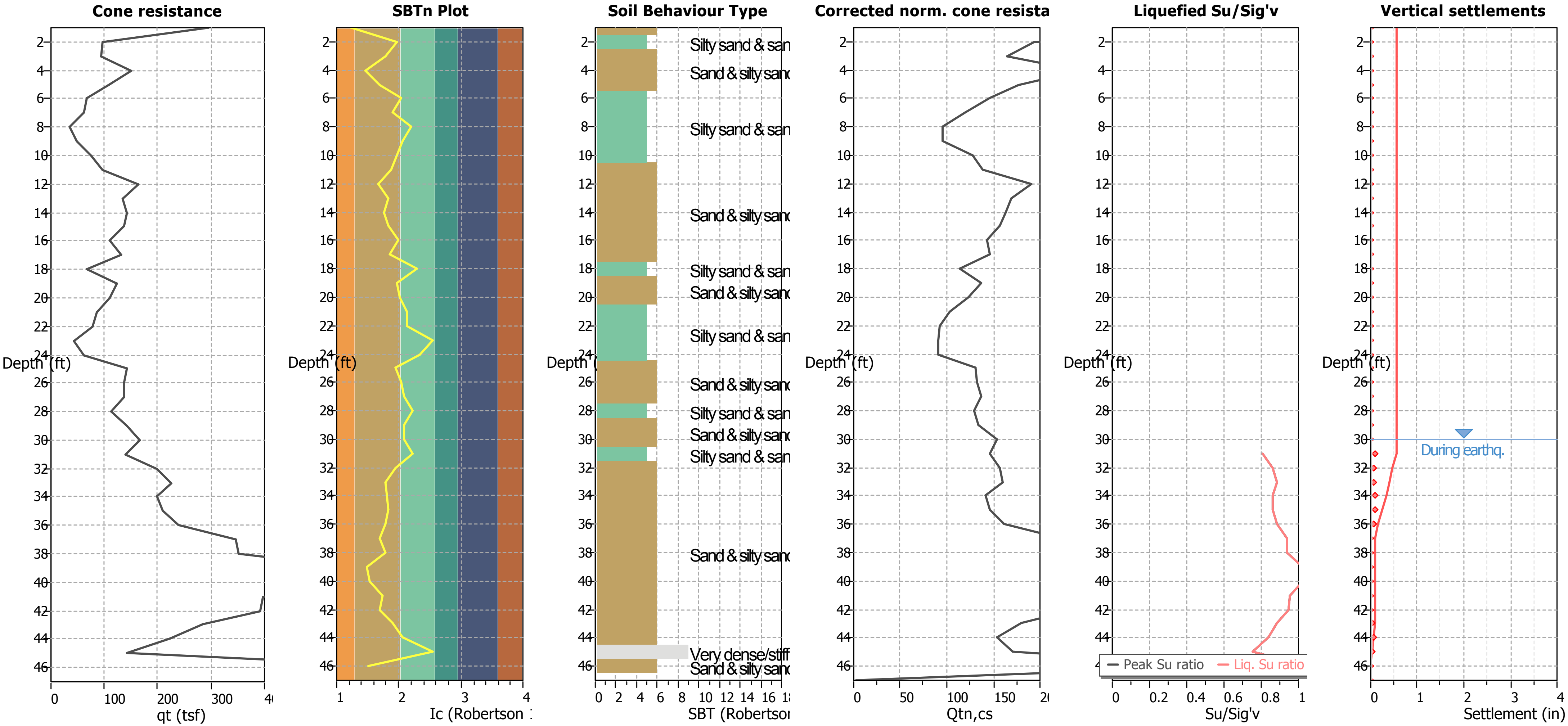
(FREE WATER DRAIN)

Appendix D
Liquefaction Analysis

LGC Geotechnical, Inc.

Project: 18078-01
Location: Colton, CA

CPT: CPT-01
Total depth: 47.00 ft

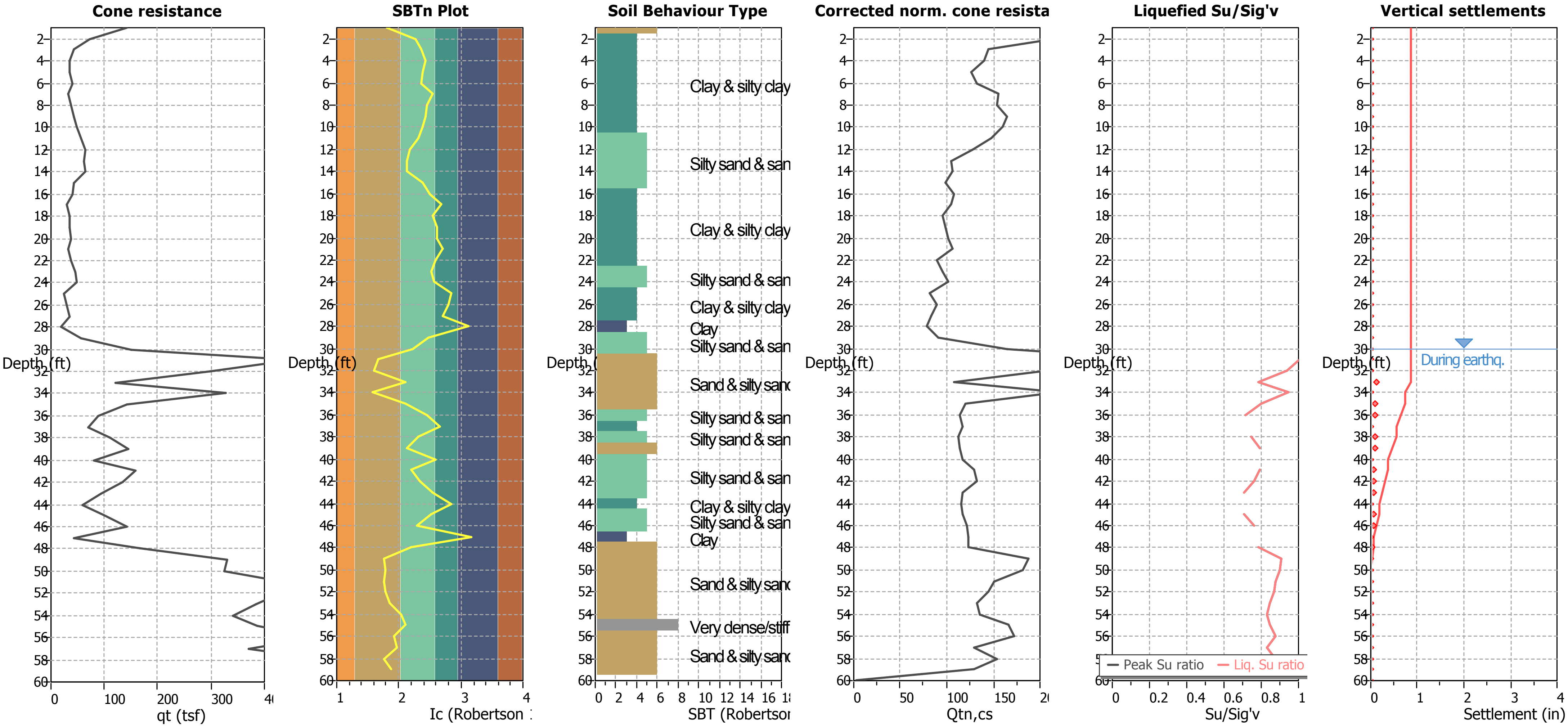


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	100.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

LGC Geotechnical, Inc.

Project: 18078-01
Location: Colton, CA

CPT: CPT-02
Total depth: 60.00 ft

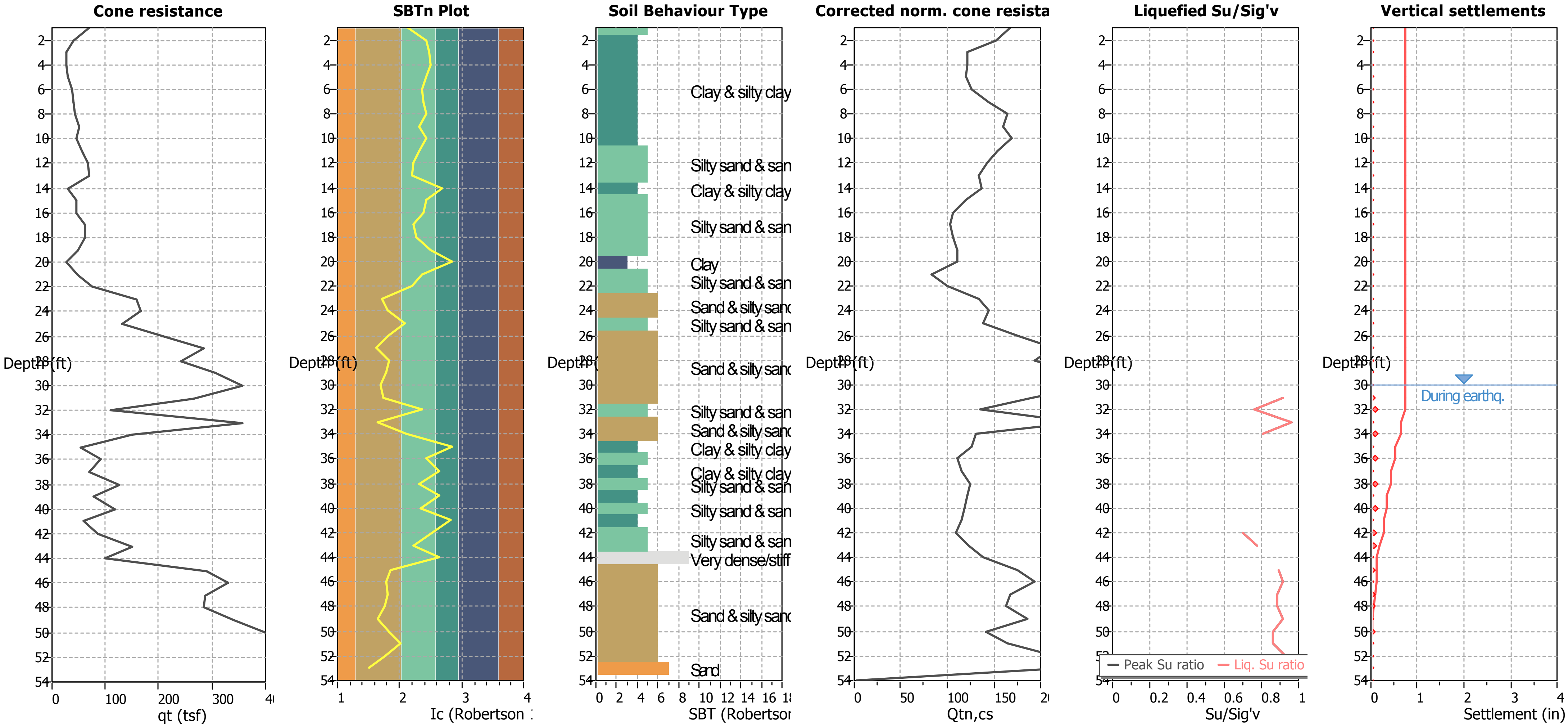


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	100.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

LGC Geotechnical, Inc.

Project: 18078-01
Location: Colton, CA

CPT: CPT-03
Total depth: 54.00 ft



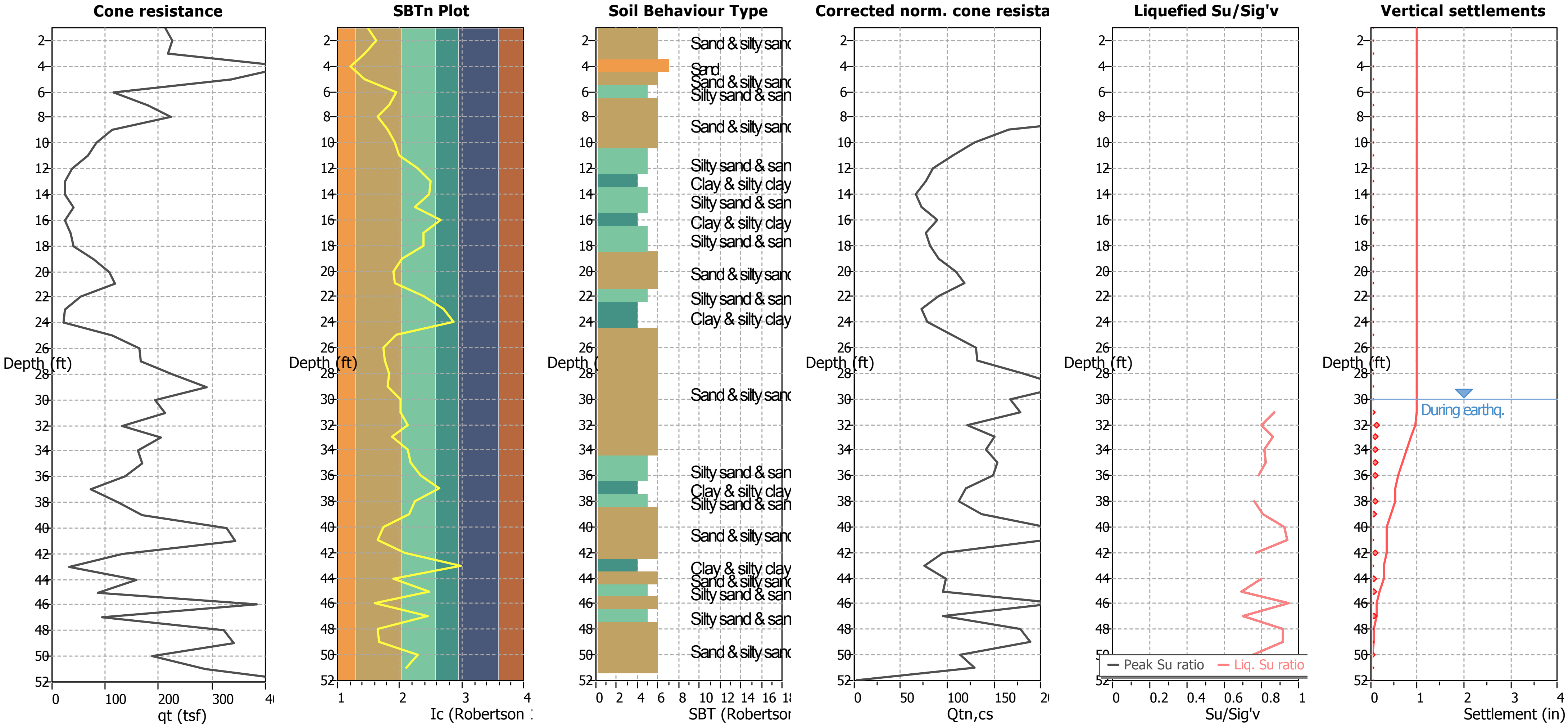
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	100.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

CPeT-IT v.2.2.1.8 - CPTU data presentation & interpretation software - Report created on: 7/24/2018, 7:48:07 AM
Project file: Z:\2018\18078-01 HIP - Ashley Way\Engineering\cpt data\liquefaction_GWT@30'_MCE_ev.clq

LGC Geotechnical, Inc.

Project: 18078-01
Location: Colton, CA

CPT: CPT-04
Total depth: 52.00 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	100.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

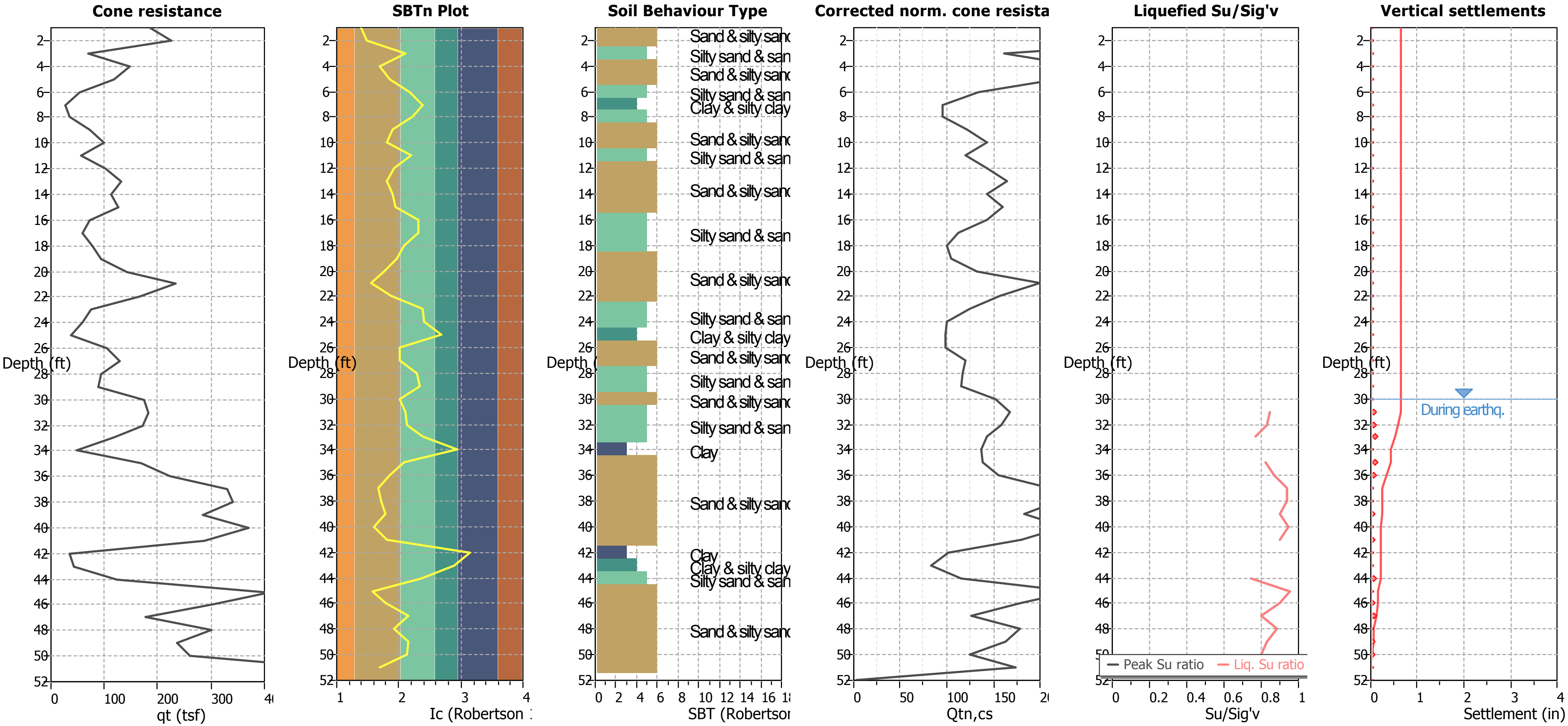
LGC Geotechnical, Inc.

Project: 18078-01

Location: Colton, CA

CPT: CPT-05

Total depth: 52.00 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	100.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.91	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Appendix E
General Earthwork & 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. 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 Oversized

Oversized 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 oversized material is completely surrounded by compacted or densified fill. Oversized 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 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.