LGC Geotechnical, Inc.

March 12, 2018

Project No. 18012-01

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Subject: ***Preliminary Geotechnical Evaluation and Design Recommendations for Proposed Residential Development, Former Glenelder Elementary School Site, Hacienda Heights, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed residential development located at the former Glenelder Elementary school site located in the City of Hacienda Heights, County of Los Angeles, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed residential 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 Submitted,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our preliminary geotechnical evaluation for the proposed approximately 10-acre residential development located at the former Glenelder Elementary School site in the City of Hacienda Heights, California. Refer to the Site Location Map (Figure 1).

The purpose of our study was to provide a preliminary geotechnical evaluation relative to the proposed residential development. As part of our scope of work, we have: 1) reviewed available geotechnical background information including in-house regional geologic maps and published geotechnical literature pertinent to the site (Appendix A); 2) performed a limited subsurface geotechnical evaluation of the site consisting of the excavation of five small-diameter borings ranging in depth from approximately 5 to 25 feet below existing ground surface and six cone penetration test (CPT) soundings ranging in depth from 22 to 33 feet below existing ground surface; 3) performed two field percolation tests; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this preliminary geotechnical summary report presenting our findings, preliminary conclusions and recommendations for the development of the proposed project.

1.2 Project Description

The site is bound to the north by Folger Street, to the east by Glenelder Avenue, to the south by existing residential units located on Denley Street and to the west by Hinnen Avenue. The site is currently a decommissioned elementary school operated and maintained by the Hacienda La Puente Unified School District (HLPUSD).

Based on concept site studies (WHA, 2018), the proposed improvements include the construction of single family residential units. Two internal streets are currently proposed throughout the site. Design cuts and fills (not including required remedial grading) are anticipated to be on the order of 2 to 5 feet. The proposed building structures are anticipated to be relatively light-weight at-grade structures with maximum column and wall loads of approximately 30 kips and 2 kips per linear foot, respectively.

The recommendations given in this report are based upon the estimated structural loading, grading and layout information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and any changes to structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

1.3 Existing Conditions

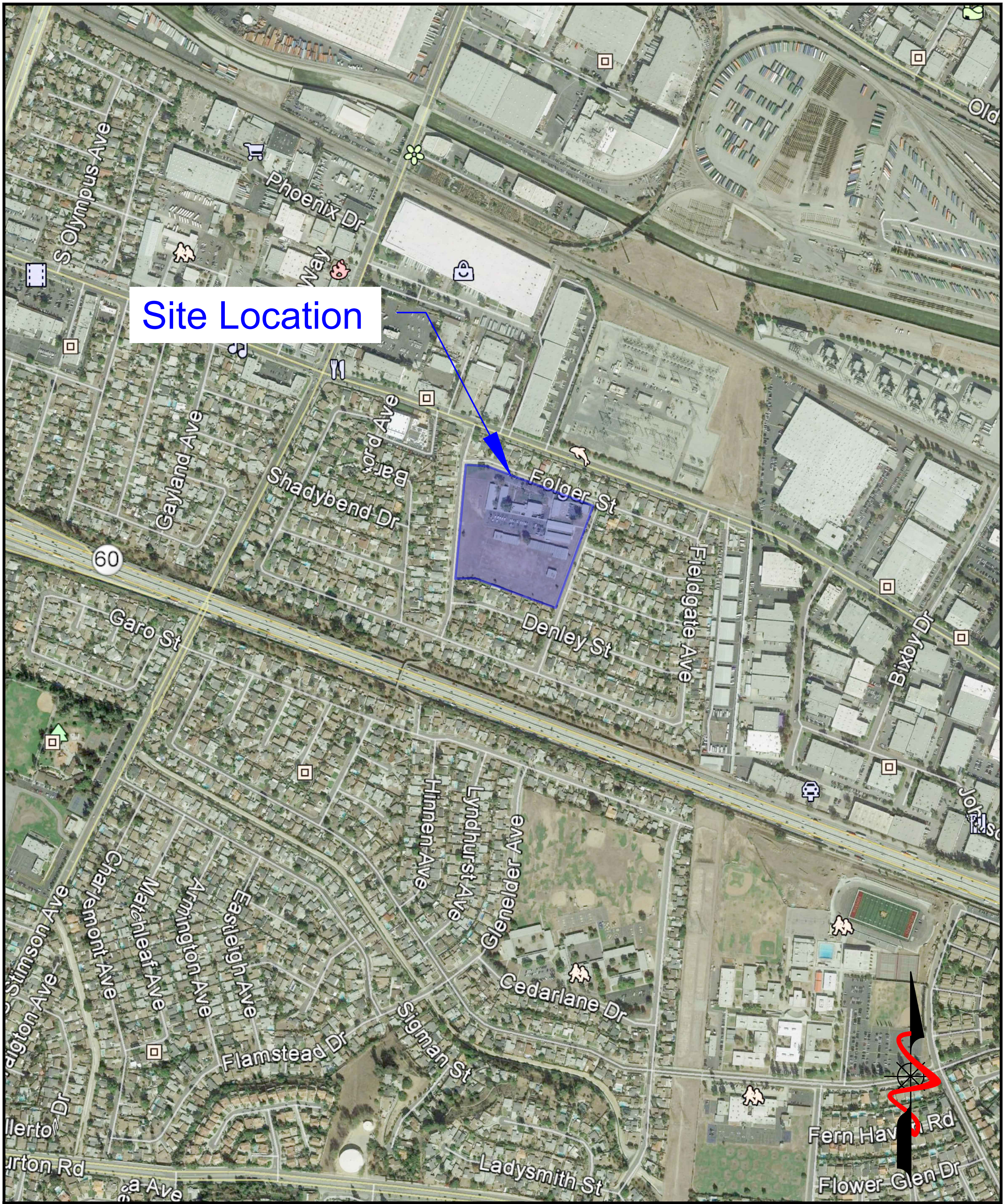
The “Glenelder” site is a slightly irregular-shaped site operated and maintained by the HLPUSD. The current topography consists of a relatively flat pad for the majority of the site. According to the original grading plans (Kristner, 1957) the site was positioned on a gently sloped natural grade originally consisting of cuts and fills up to approximately 2 feet. The southern property line appears to be shared with existing fence and wall structures separating the subject site and residential lots. The western, northern and eastern property lines are bordered by Hinnen Avenue, Folger Street and Glenelder Avenue, respectively.

Existing improvements consist of seven single-story school buildings, an asphalt concrete parking lot and entry drive way, sports courts currently used for parking, park play apparatus, a grass field and miscellaneous landscaping.

The site has minor relief, with the highest being the southeastern corner of the site at an approximate elevation of 366 feet and the lowest being the northwestern corner of the site at an approximate elevation of 351 feet. The site gently slopes gently from south to north.

1.4 __Background__

Review of historical aerials indicates that the elementary school and seven associated buildings were constructed after 1953, but prior to 1963 (Historic Aerials, 2017). The aerial photos from 1948 and 1953 indicates the site was previously agricultural. According to the Executive Listing (Tierra, 2017), the subject school site is currently operating as IT support as well as a location for a Head Start program. There are currently no tenants occupying the school.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Lennar - Glenelder
PROJECT NO.	18012-01
ENG. / GEOL.	RLD/KTM
SCALE	NTS
DATE	March 2018

1.5 Subsurface Geotechnical Evaluation

LGC Geotechnical performed a subsurface geotechnical evaluation of the site consisting of the excavation of five hollow-stem auger borings and six CPT soundings to evaluate onsite geotechnical conditions.

Five hollow-stem borings (HS-1 through HS-3 & I-1 through I-2) were drilled to depths ranging from approximately 5 to 50 feet below existing grade. An LGC Geotechnical staff engineer observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated by 2R Drilling, Inc. under subcontract to LGC Geotechnical using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples of the near-surface soils were also collected and logged at select borings for laboratory testing. At the completion of drilling, the borings were backfilled with the native soil cuttings and tamped. Some settlement of the backfill soils may occur over time.

Six CPT soundings (CPT-1 through CPT-6) were pushed to depths ranging between approximately 22 to 33 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. All CPTs were performed by Middle Earth Geo Testing, Inc. using a 25-ton all-wheel drive CPT rig.

Infiltration testing was performed within two of the borings (I-1 and I-2) to depths of 5 and 10 feet below existing grade. An LGC Geotechnical staff engineer installed standpipes, backfilled the borings with crushed rock and pre-soaked the infiltration holes prior to testing. Infiltration testing was performed per the County of Los Angeles testing guidelines. The locations were subsequently backfilled with native soils at the completion of testing.

The approximate locations of our subsurface explorations are provided on the Geotechnical Exploration Location Map (Figure 2). The boring and CPT logs are provided in Appendix B.

1.6 Laboratory Testing

Representative bulk and driven (relatively undisturbed) samples were obtained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, Atterberg Limits, fines content, expansion index, consolidation, laboratory compaction, direct shear, R-value and corrosion (sulfate, chloride, pH and minimum resistivity).

The following is a summary of the laboratory test results:

- Dry density of the samples collected ranged from approximately 100 pounds per cubic foot (pcf) to 132 pcf, with an average of 116 pcf. Field moisture contents ranged from approximately 2 to 28 percent, with an average of 12 percent.
- Two fines content tests were performed and indicated a fines content (passing No. 200 sieve) of approximately 6 and 8 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as "coarse-grained."
- One Atterberg Limit (liquid limit and plastic limit) test was performed. Results indicated a Plasticity Index (PI) value of 5.
- One direct shear test was performed. The plot is provided in Appendix C.
- One consolidation test was performed. The load versus deformation plot is provided in Appendix C.
- One swell/collapse test was performed. The plot is provided in Appendix C.
- One laboratory compaction test of a near surface sample indicated a maximum dry density of 121.0 pcf with an optimum moisture content of 13.0 percent.
- Expansion potential testing indicated an expansion index value of 60, corresponding to "Medium" expansion potential.
- One R-value test was performed. Results indicate an R-value of 12.
- Corrosion testing indicated soluble sulfate contents of approximately 0.017 percent, a chloride content of 66 parts per million (ppm), pH of 7.0, and a minimum resistivity of 1,000 ohm-centimeters.

A summary of the laboratory test results is presented in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Geologic Conditions

The subject site is located within the southeastern portion of the San Gabriel Valley, within the Peninsular Ranges Geomorphic Province. It is located in a broad alluvial valley that is several miles north of the northwest-trending Whittier Hills that are bounded by the Whittier Hills Fault. The southwest-trending San Jose Hills, located to the northeast of the site, are bound by the Walnut Creek Fault. The region has a complex geologic history influenced by periods of uplift, folding, faulting, and alluvial deposition; however, no faults are known to transect the site.

The site is located on a laterally extensive young alluvial fan deposit interpreted to be approximately middle Holocene age (Morton & Miller, 2003). It is about a quarter-mile from the channelized San Jose Creek, a west-flowing drainage that joins the San Gabriel River several miles downstream.

2.2 Generalized Subsurface Conditions

The field explorations (borings and CPTs) indicate site soils primarily consist of fine-grained very stiff silts with varying amounts of medium dense sand and gravelly sand layers to the maximum explored depth of approximately 50 feet below ground.

It should be noted that borings and CPTs 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.

2.3 Groundwater

Groundwater was encountered in one of our borings (HS-1) at a depth of approximately 41.5 feet below existing grade. Historic high groundwater is estimated to be about 15 feet below existing grade (CDMG, 1998).

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 due to local seepage caused by irrigation and/or recent precipitation. Local perched groundwater conditions or surface seepage may develop once site development is completed.

2.4 Field Infiltration Testing

Two field percolation tests were performed in Borings I-1 and I-2 to approximate depths of 5 and 10 feet below existing grade, respectively. The approximate locations are shown on the Geotechnical Exploration Location Map (Figure 2). The borings for the infiltration tests were excavated using a drill rig equipped with 8-inch diameter hollow-stem augers. Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Los Angeles (2017). A 3-inch diameter perforated PVC pipe was placed in the borehole and the annulus was backfilled with gravel. The infiltration wells were pre-soaked 1 hour prior to testing. During the pre-test, water was added to the boring and was observed for 10 and 30 minutes. Water remained in the boring after 30 minutes in each of the two percolation test borings; therefore, the procedure for fine-grained soils was followed. One reading was recorded every 30 minutes from a fixed reference point until 8 readings were recorded. At the completion of infiltration testing, the pipe was removed and backfilled with cuttings and tamped. Some settlement of the backfill should be expected.

Based on the County of Los Angeles testing guidelines, the raw infiltration is calculated by dividing the volume of water discharged by the surface area of the test section (including sidewalls plus the bottom of the boring), in a given amount of time. The average of the stabilized infiltration rate over the last three consecutive readings is the measured infiltration rate. The measured infiltration rates are provided in Table 1 below. Please note that the values provided in Table 1 <u>do not include reduction factors</u> for the test procedure, site variability and long-term siltation plugging that are required for the design infiltration rate, refer to Table 5 located in Section 4.7. Infiltration tests were performed using relatively clean water free of particulates, silt, etc. Refer to the infiltration test data provided in Appendix D.

TABLE 1

Summary of Field Percolation Testing

Infiltration Test Location	Measured Infiltration Rate* (inch/hr)
I-1	0.0
I-2	0.6

*Does Not Include Required Reduction Factors, refer to Table 5, Section 4.7.

2.5 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 California Building Code (CBC). 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 34.0056 degrees north and longitude -117.9533 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 2 on the following page.

TABLE 2

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

Selected Parameters from 2016 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class per Chapter 20 of ASCE 7	D*
Risk-Targeted Spectral Acceleration for Short Periods (S_S)**	2.189g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)**	0.777g
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: $S_{MS} = F_aS_S$]	2.189g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_vS_1$]	1.165g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.460g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.777g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	0.955
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	0.978

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

** From USGS, 2018

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 0.821g (USGS, 2017).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.79 at a distance of approximately 11.7 km from the site would contribute the most to this ground motion (USGS, 2008).

2.6 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, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. 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 evaluations 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 an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site (CDMG, 1991).

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, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. Some of the major active nearby faults that could produce these secondary effects include the Whittier, Puente Hills, and San Andreas Faults, among others (CGS, 2018). A discussion of these secondary effects is provided in the following sections.

2.6.1 Liquefaction and Dynamic Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that 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.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1998), the site is located within a liquefaction hazard zone. Subsurface field data indicates that the site contains generally thin sandy layers susceptible to liquefaction interfingered with fine-grained non-liquefiable soils and very dense sands. The recent explored groundwater elevation of 41.5 feet below existing grade and historic high groundwater elevation of 15 feet below existing grade were both used in the liquefaction analysis. The liquefaction evaluation was performed on CPTs (GeoLogismiki, 2017) and the boring B-1. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and the applicable seismic criteria (e.g., 2016 CBC). Furthermore, isolated layers may be susceptible to dry sand seismic settlement. Seismically induced dry sand settlements were estimated by the procedures outlined by Pradel (1998). Liquefaction induced settlement and dry sand settlement were estimated using the PGA_M per the 2016 CBC and a moment magnitude of 6.79 (USGS, 2008).

Results indicate total seismic settlement on the order of 1.5 inches or less. Differential seismic settlement can be estimated as half of the total estimated settlement over a horizontal span of about 40 feet. Liquefaction calculations are provided in Appendix E.

2.6.2 Lateral Spreading

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

Due to the depth to groundwater, low potential for shallow liquefaction and lack of nearby "free face" conditions, the potential for lateral spreading is considered low.

2.7 Expansion Potential

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

3.0 CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are implemented.

The following is a summary of the primary geotechnical factors that may affect future development of the site:

- In general, our borings and CPTs indicate primarily very stiff silts with varying amounts of medium dense to dense sand and gravelly sand layers to the maximum explored depth of approximately 50 feet below existing grade. The near-surface soils are generally collapsible and are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was encountered during our subsurface evaluation at a depth of approximately 41.5 feet below existing ground surface. Historic high groundwater is estimated to be about 15 feet below existing grade (CDMG, 1998).
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo). The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Site soils are considered susceptible to liquefaction. The site is located in a State of California Seismic Hazard Zone for liquefaction. Total dynamic settlement is estimated to be on the order of 1.5 inches or less. Differential dynamic settlement can be estimated at half of the total settlement over a horizontal span of 40 feet for design of foundations.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have "Medium" expansion potential. Mitigation measures are required for foundations and site improvements like concrete flatwork to minimize the impacts of expansive site soils. Final design expansion potential must be determined at the completion of grading.
- Pre-soaking of the subgrade for building slabs will be required due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 3 weeks).
- 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.
- The site contains soils that are not suitable for retaining wall backfill due to their fines content and expansion potential, therefore import of sandy soils will be required by the contractor for obtaining suitable backfill soil for planned site retaining walls.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that the on-site earth materials generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.

4.0 PRELIMINARY RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and 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 owner.

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 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 "that 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 improvements 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 as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of demolition of the existing site improvements, required earthwork removals, subgrade preparation, precise grading and construction of the proposed new improvements, including the residential structures, neighborhood amenities, subsurface utilities, interior streets, etc.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2016 CBC/County of Los Angeles grading requirements, and the General Earthwork and Grading Specifications included in Appendix F. In case of conflict, the following recommendations shall supersede those included in Appendix F. The following recommendations should be considered preliminary and may be revised based upon future evaluation and review of the project plans and/or based on the actual conditions encountered during site grading/construction.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of existing building structures, asphalt, surface obstructions, and demolition debris.

Vegetation and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material. Any abandoned sewer or storm drain lines should be completely removed and replaced with properly placed compacted fill. Deeper demolition may be required in order to remove existing foundations. We recommend the trenches associated with demolition which extend below the remedial grading depth be backfilled and properly compacted prior to the demolition contractor leaving the site.

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

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend the site soils be removed and recompacted. We recommend that soils within building pads be removed and recompacted to a minimum depth of 5 feet below existing grade. The envelope for removal and recompaction should extend laterally a minimum distance of 5 feet beyond the edges of the proposed improvements. To promote soil uniformity in areas of design cut, over-excavation shall extend a minimum of 5 feet below existing grade or a minimum of 3 feet below finished grade, whichever is deeper. The Remedial Grading Diagram (Figure 4) provides a simplified example of the building pad remedial recommendations.

For minor site 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. Within pavement and hardscape areas, removals should extend to a depth of at least 2 feet below the existing grade. Pavement area over-excavation (design cut areas) 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 over-excavation should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements mentioned above.

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 and areas to be over-excavated 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 investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. 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. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. <u>Excavation safety is the sole responsibility of the contractor.</u>

Where proposed building structures will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing "ABC" slot cuts. Slot cuts should be no wider than 12 feet and no deeper 6 feet and should be backfilled <u>immediately</u> with properly placed compacted fill to finish grade prior to excavation of adjacent slots. Due to the potential presence of sands which are susceptible to caving, narrower slot cuts may be required. This should be further evaluated during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum 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 bottoms, over-excavation bottoms and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project recommendations.

Removal bottoms, over-excavation 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, provided they are screened of organic materials, construction debris and oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of soils of "Very Low" to "Medium" expansion potential (expansion index 90 or less based on American Society for Testing and Materials [ASTM] D 4829), and free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. <u>Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.</u>

Retaining wall backfill should consist of imported sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per 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, import of soils will be required by the contractor for obtaining suitable retaining wall backfill soil.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the most recent version of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) and/or County of Los Angeles requirements.

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

4.1.6 Placement and Compaction of Fills

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Drying and or mixing of very moist soils will be required prior to reusing the materials in compacted fills. Soils are also present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in 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 performed by the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

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

Aggregate base material should be compacted to at least 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at near-optimum moisture content (generally within optimum and 2 percent 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 material 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 or greater (per California Test Method [CTM] 217) may be used to bed and shade the pipes. Based on our field evaluation, onsite soils will not meet this sand equivalent requirement. 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 the recommended minimum relative compaction (per ASTM D1557).

Retaining wall backfill should consist of sandy soils as outlined in preceding 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 (Figure 3). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

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

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 0 to 5 percent reduction in volume of near-surface (upper approximate 5 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence, due to earthwork operations, is expected to be on the order of 0.1 feet. 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.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability of on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil. We recommend a "balance" area that can accommodate on the order of 5 percent (plus or minus) of the total grading volume be considered.

4.2 Preliminary Foundation Recommendations

Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the residential structures using a conventional or post-tensioned foundation system designed to resist the impacts of expansive soils. Site soils are anticipated to be "Medium" expansion potential (EI of 90 or less per ASTM D4829) and special design considerations from a geotechnical perspective are required. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading.

Preliminary foundation recommendations are provided in the following sections. Recommended soil bearing and estimated settlement due to structural loads are provided in Section 4.3.

4.2.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with the Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2016 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 25
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer
- Minimum Footing Depth: 18 inches below lowest adjacent grade.
- Moisture-condition (presoak) slab subgrade to 120% of optimum moisture content to a minimum depth of 18 inches prior to trenching.

The recommended moisture content should be maintained up to the time of concrete placement.

4.2.2 Provisional Post-Tensioned Foundation Design Parameters

The geotechnical parameters provided herein may be used for post-tensioned slab foundations. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI, 2012) Standard Requirements (PTI DC 10.5), referenced in

Chapter 18 of the 2016 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method to resist expansive soils.

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

TABLE 3

Provisional Geotechnical Parameters for Post-Tensioned Foundation Slab Design

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	Medium[1]	Medium[1]
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 9.0 feet 0.5 inch	 9.0 feet 0.6 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 4.7 feet 1.1 inch	 4.7 feet 1.3 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	150 pci	150 pci
Minimum perimeter footing/thickened edge embedment below finish grade	18 inches	6 inches
Perimeter foundation reinforcement	N/A[2]	N/A[2]
Minimum slab thickness	5 inches[2]	8 inches[2]
Presoak (moisture conditioning)	120% of Opt. 18 inches	120% of Opt. 18 inches

1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Recommendations for sand below slabs have traditionally been included with geotechnical foundation recommendations, although they are not the purview of the geotechnical consultant. The sand layer requirements are the purview of the foundation engineer/structural engineer, and should be provided in accordance with ACI Publication 302 “Guide for Concrete Floor and Slab Construction”.
4. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.

4.2.3 Post-Tensioned Foundation Subgrade Preparation and Maintenance

Moisture conditioning of the subgrade soils is recommended prior to trenching the foundation. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 3 weeks). The recommendations specific to the anticipated site soil conditions, including recommended presoak, are presented in Table 3. The subgrade moisture condition of the building pad soils should be maintained at near-optimum moisture content 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 of the homes.

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

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

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

4.2.4 Slab Underlayment Guidelines

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

4.3 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 2,000 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 400 psf for each additional foot of embedment and 200 psf for each additional foot of foundation width to a maximum value of 3,000 psf. A post-tensioned mat foundation a minimum of 6 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 5H:1V) 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).

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential static settlement may be taken as half of the static settlement (i.e., ½-inch over a horizontal span of 40 feet). Furthermore, seismic settlement is anticipated to be 1.5 inches or less. Differential seismic settlement may be taken as half of the seismic settlement (i.e., ¾-inch over a horizontal span of 40 feet).

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. For slabs constructed over a moisture retarder, the allowable friction coefficient should be provided by the manufacturer. An allowable passive lateral earth pressure of 240 psf per foot of depth (or pcf) to a maximum of 2,400 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 320 pcf (maximum of 3,200 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. 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. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.4 Lateral Earth Pressures for Retaining Walls

The site contains soils that are not suitable for retaining wall backfill due to their fines content and expansion potential, therefore import of sandy soils will be required by the contractor for obtaining suitable backfill soil for planned site retaining walls. Lateral earth pressures for import soils (sandy soils) meeting indicated project recommendations (Section 4.1.5) are provided below. Lateral earth pressures are provided as equivalent fluid unit weights, in 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. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented in Table 4 for approved imported sandy soils. The retaining wall designer should clearly indicate on the retaining wall plans the required select imported sandy soil backfill.

TABLE 4

Lateral Earth Pressures – Imported Sandy Soils

Conditions	Equivalent Fluid Unit Weight (pcf)	Equivalent Fluid Unit Weight (pcf)
	Level Backfill	**2:1 Sloped Backfill**
	Approved Sandy Soils	**Approved Sandy Soils**
Active	35	55
At-Rest	55	70

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. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal: vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining wall. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. The retaining wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 15 pcf for a level backfill condition. This increment should be applied in addition to the provided static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). 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. The provided seismic lateral earth pressure should not be used for retaining walls exceeding 10 feet in height. If a retaining wall greater than 10 feet in height is proposed or a retaining wall with a sloping backfill condition, the retaining wall designer should contact the geotechnical engineer for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near-surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care and information should be provided to homeowners to maintain these drains. Typical retaining wall drainage is illustrated in Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water containing soluble salts migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

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 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 of a near-surface bulk sample indicated a soluble sulfate content of approximately 0.017 percent, a chloride content of 66 parts per million (ppm), pH of 7.0, and a minimum resistivity of 1,000 ohm-centimeters. Based on Caltrans Corrosion Guidelines (Caltrans, 2015), soils are considered corrosive to structural elements 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 a severity categorization of “Not Applicable” and are designated to a class “S0” per ACI 318, Table 19.3.1.1 with respect to sulfates.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

4.6 Control of Surface Water and Drainage Control

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed residences be sloped away from the proposed residence 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 the side yard 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.

4.7 Subsurface Water Infiltration

The design infiltration rate is determined by dividing the measured infiltration rate by a series of reduction factors including; test procedure (RF_t), site variability (RF_v) and long-term siltation plugging and maintenance (RF_s). The reduction factor for long-term siltation plugging and maintenance (RF_s) is the purview of the infiltration system designer per the Los Angeles County testing guidelines (2017). The following site variability reduction factor applied to the measured infiltration rate is provided below in Table 5. The design infiltration rate is the measured percolation rate divided by the total reduction factor (RF_t x RF_v x RF_s).

TABLE 5

Reduction Factors Applied to Measured Infiltration Rate

Consideration	Reduction Factor
Test procedure, boring percolation, RF_t	2
Site variability, number of tests, etc., RF_v	2
Long-term siltation plugging and maintenance, RF_s	Per Infiltration Designer
Total Reduction Factor, RF = RF_t x RF_v x RF_s	**TBD**

Per the requirements of the Los Angeles County testing guidelines (2017), subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. Based on the minimum Reduction Factor of 4 calculated above (not including long-term siltation plugging and maintenance), the infiltration tests do not meet the minimum requirements of the County of Los Angeles testing guidelines; therefore, considering the results of the field infiltration test indicating unacceptable infiltration rates and that the onsite soils primarily consisting of mixed fine-grained and very dense coarse-grained soils considered to have low permeability, we do not recommend that surface water be intentionally infiltrated into subsurface soils.

4.7 Preliminary Asphalt Concrete Pavement Sections

The following provisional minimum asphalt concrete (AC) street sections are provided in Table 6 for Traffic Indices (TI) of 5.0, 6.0 and 6.5. These sections are based on preliminary laboratory testing results indicating an R-value of 12. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final pavement sections should be confirmed by the project civil engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values. It is our understanding that the County of Los Angeles follows the Caltrans

Highway Design Manual which requires a minimum pavement section consisting of 4.2 inches of asphalt concrete over 4.2 inches of aggregate base (AB).

TABLE 6

Preliminary Pavement Section Options

Assumed Traffic Index	5.0	6.0	6.5
R -Value Subgrade	12	12	12
AC Thickness	4.5 inches	5.0 inches	5.0 inches
Aggregate Base Thickness	5.0 inches	8.0 inches	10.0 inches

The pavement section thicknesses provided above are considered 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 throughout 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.

4.8 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, private drives, patio slabs, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 7. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 7

Preliminary Geotechnical Parameters for Nonstructural Concrete Flatwork Placed on Medium Expansion Potential Subgrade

	Homeowner Sidewalks	Private Drives	Patios/Entryways	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (nominal)	5 (full)	5 (full)	City/Agency Standard
Presoaking	Wet down	Presoak to 12 inches	Presoak to 12 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	2	City/Agency Standard

To reduce the potential for driveways to separate from the garage slab, the builder may elect to install dowels to tie these two elements together. Similarly, future homeowners should consider the use of dowels to connect flatwork to the foundation.

4.9 Geotechnical Plan Review

When available, grading, retaining wall and foundation plans should be reviewed by LGC Geotechnical in order to verify our geotechnical recommendations are implemented. Updated recommendations and/or additional field work may be necessary.

4.10 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 retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placing steel reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

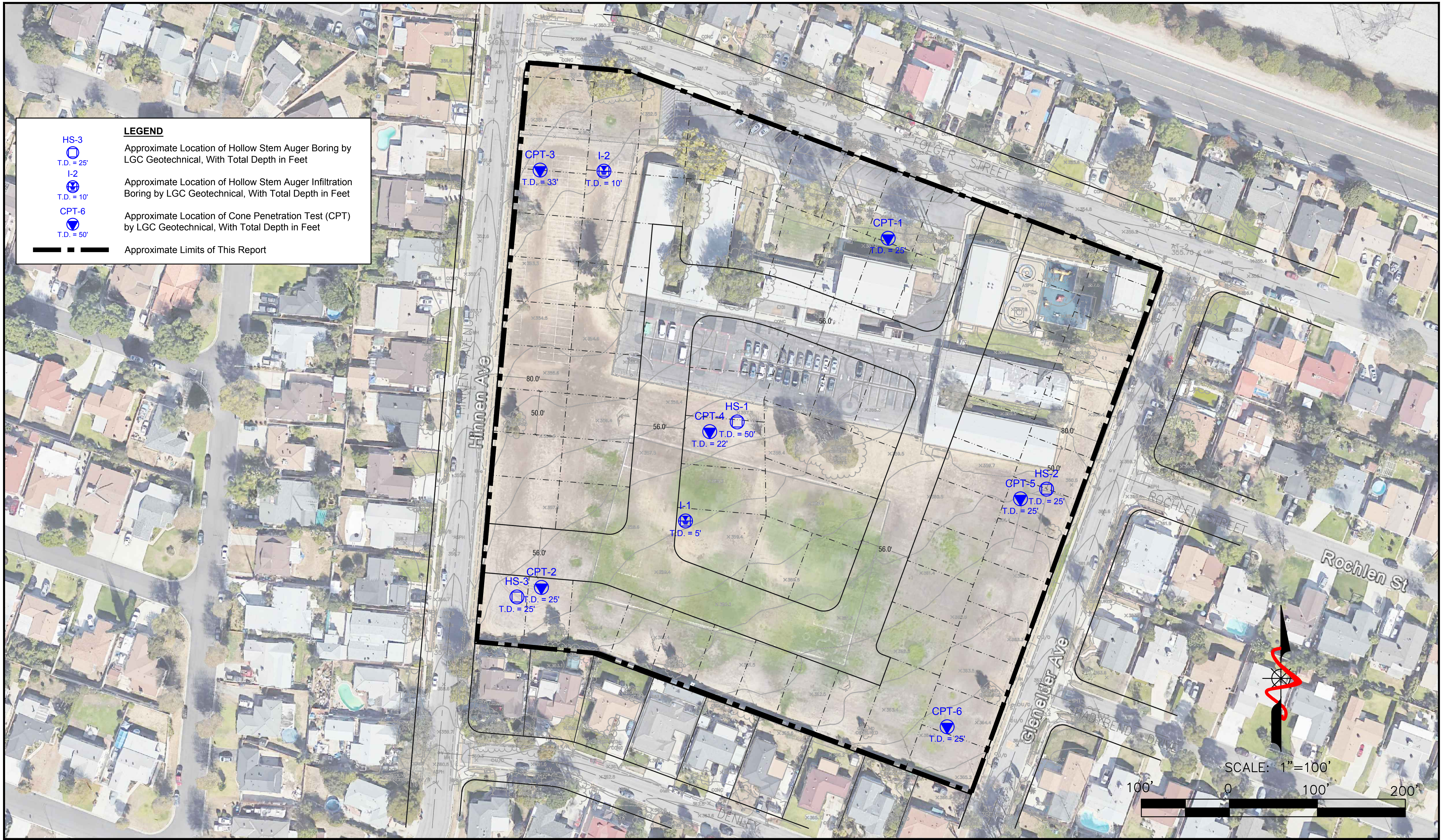
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included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

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

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

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

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



LGC Geotechnical, Inc.

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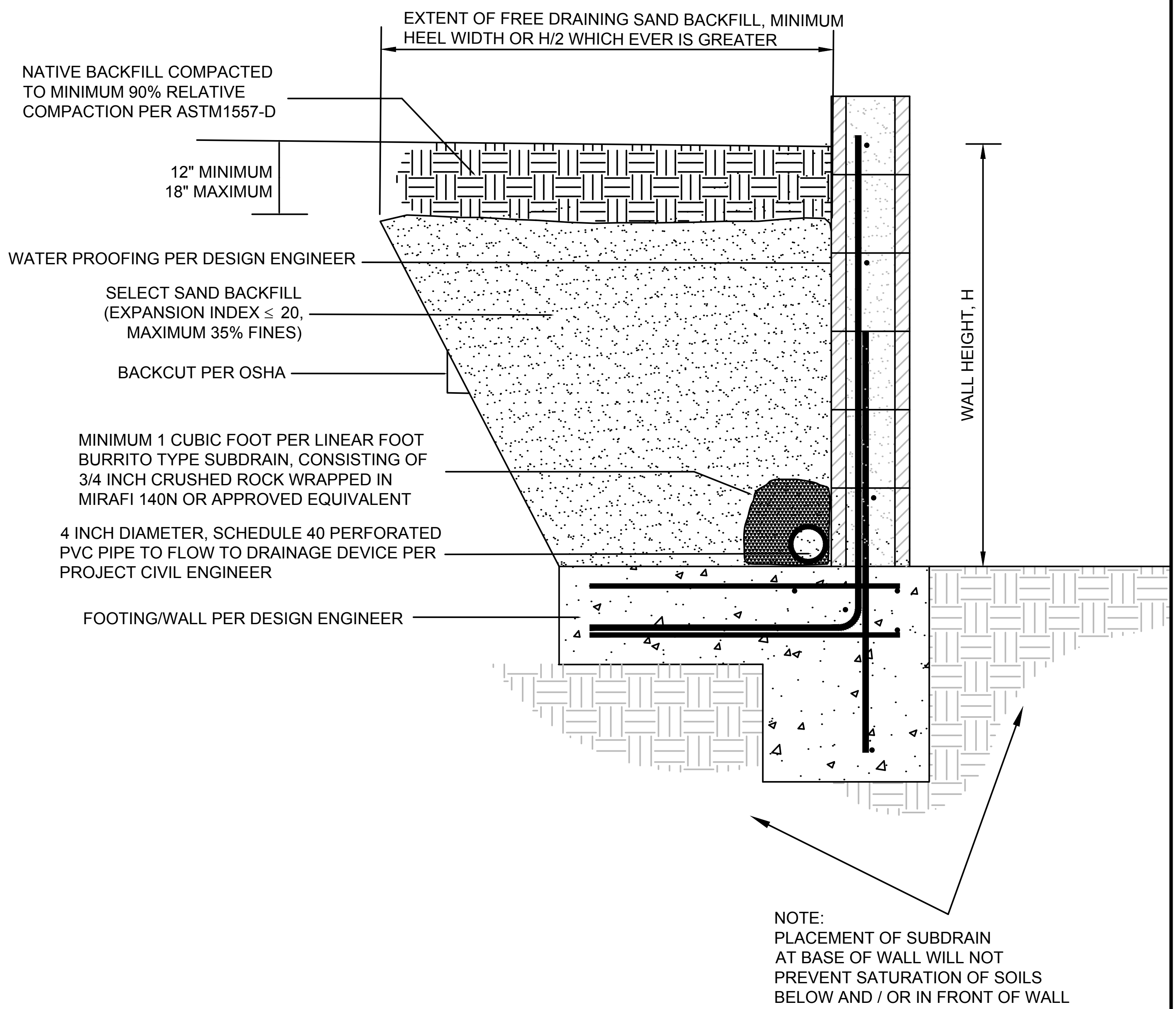
Figure 2
Geotechnical Exploration Location Map

PROJECT NAME	Lennar - Glenelder
PROJECT NO.	18012-01
ENG. / GEOL.	RLD/KTM
SCALE	1" = 100'
DATE	March 2018

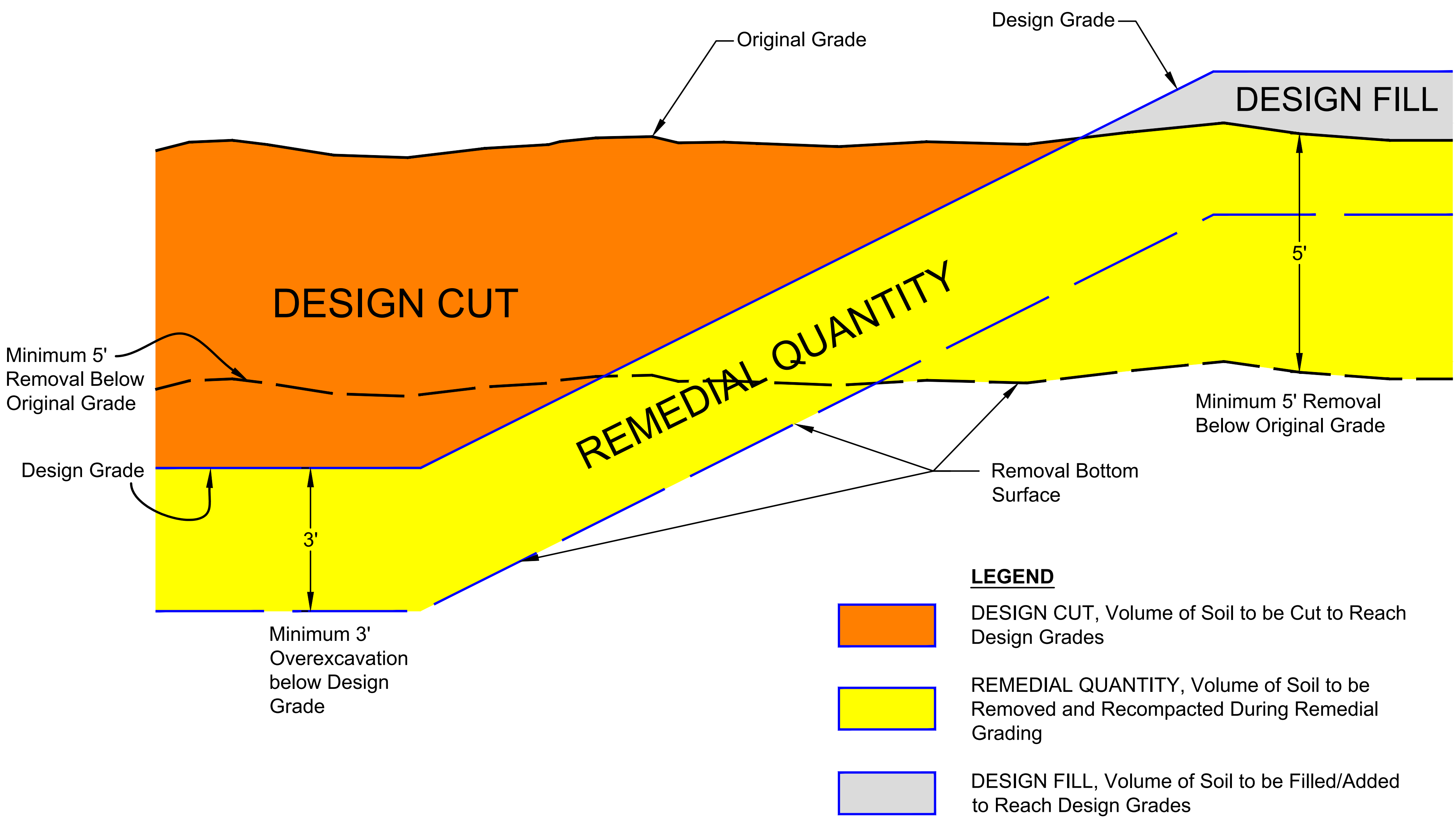


LGC Geotechnical, Inc.

FIGURE 3

Retaining Wall Backfill Detail
Approved Select Backfill
(EI≤20)

PROJECT NAME	*Lennar - Glenelder*
PROJECT NO.	*18012-01*
ENG.	*RLD/KTM*
SCALE	*Not to Scale*
DATE	*March 2018*



LGC Geotechnical, Inc.

FIGURE 4
Remedial Grading Diagram

PROJECT NAME	*Lennar - Glenelder*
PROJECT NO.	*18012-01*
ENG. / GEOL.	*RLD/KTM*
SCALE	*NTS*
DATE	*March 2018*

Appendix A
References

APPENDIX A

References

American Society of Civil Engineers (ASCE), 2013, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10, Third Printing, 2013.

American Society for Testing and Materials (ASTM), Volume 04.08 Soil and Rock (I): D420 – D5876.

Bray, J.D., and Sancio, R. B., 2006, Assessment of Liquefaction Susceptibility of Fine-Grained Soils, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, pp. 1165-1177, dated September 2006.

California Building Standards Commission, 2016, California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2016.

California Department of Transportation (Caltrans), 2015, Corrosion Guidelines, Version 2.1, dated January 2015.

California Division of Mines and Geology (CDMG) 1997, Geologic Map of the Baldwin Park 7.5-Minute Quadrangle, Los Angeles County, California, Open File Report 98-30, Dated 1997.

______, 1998, State of California Seismic Hazard Zone Report for the Baldwin Park 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 98-13, Dated 1998.

______, 1999, State of California Seismic Hazard Zones, Baldwin Park Quadrangle, Official Map, scale: 1:24,000, Release Date: March 25, 1999.

California Geological Survey [CGS], 2008, California Geological Society Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California.

______, 2018, California Geological Survey website, Interactive Fault Map: http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html

County of Los Angeles, 2017, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, Department of Public Works Geotechnical and Materials Engineering Division, GS200.2, dated June 30, 2017.

GeoLogismiki, 2017, CLiq, v.2.1.6.8, Software for Liquefaction Potential Evaluation using Cone Penetration Tests and Standard Penetration Tests.

Historic Aerials, 2018, viewed February 20, 2018, Aerials viewed from: 1948, 1953, 1963, 1972, 1994 and 2012, https://www.historicaerials.com/

Kistner, Wright & Wright (Kistner), 1957, Revised Site and Grading Plan, Gale Avenue School, Puente California, dated June 1957.

Lew, et al, 2010, Seismic Earth Pressures on Deep Basements, Structural Engineers Association of California (SEAOC) Convention Proceedings.

Maurseth and Howe, 1957, Supervised Compaction Report, Cale Avenue School, Hudson School District, File No. 57.918, dated September 25, 1957.

Morton, D.H., 2003, Preliminary Geologic Map of the San Bernardino 30’ by 60’ Quadrangle, Southern California, Version 1.0, Scale 1:100,000, compiled for United States Geological Survey, Open File Report 03-293, dated 2003.

NCEER, 1997, “Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils”, T. L. Youd and I. M. Idriss Editors, Technical Report NCEER-97-0022, NCEER, Buffalo, NY.

Post-Tensioning Institute (PTI), 2012, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12.

Pradel, Daniel, 1998, Procedure to evaluate earthquake-induced settlement in dry sandy soils, *Journal of Geotechnical and Geoenvironmental Engineering*, Volume 124(4), pp. 364-368, dated April and October 1998.

Robertson, P.K., 2010, Evaluation of flow liquefaction and liquefied strength using the cone penetration test, *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, pp. 842-853, dated June 2010.

Southern California Earthquake Center (SCEC), 1999, “Recommended Procedure for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigation Liquefaction Hazards in California”, Edited by Martin, G.R., and Lew, M., dated March 1999.

Tokimatsu, K., and Seed, H. B., 1987, “Evaluation of Settlements in Sands Due to Earthquake Shaking”, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, pp. 861-878.

United States Geological Survey (USGS), 2008, Unified Hazard Tool, Dynamic: Conterminous U.S. 2008 (v3.3.1), Retrieved January 31, 2018, from: https://earthquake.usgs.gov/hazards/interactive/

______, 2018, U.S. Seismic Design Maps, Retrieved January 31, 2018, from: http://geohazards.usgs.gov/designmaps/us/batch.php#csv

WHA, 2018, Conceptual Site Plan, Glenelder School Site, Hacienda Heights, California, dated February 2, 2018.

Appendix B
Field Exploration Logs & Infiltration Data

Geotechnical Boring Log Borehole LGC-HS-1

Date: 1/31/2018	**Drilling Company:** 2R Drilling
Project Name: Glenelder	**Type of Rig:** CME 75
Project Number: 18012-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~358' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By SHH
Sampled By SHH
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						**Quaternary Younger Alluvial Fan Deposits (Qyf):**	EI CR RV
360			R-1	5 9 11	113.5	15.9	ML	@2.5' Sandy SILT: dark brown, moist, stiff	
	5		R-2	5 12 14	116.2	15.0		@5' Sandy SILT: dark brown, moist, very stiff	DS
355			R-3	4 8 17	118.9	13.1	CL-ML	@7.5' Silty CLAY with SAND: dark brown, moist, very stiff	CN AL
	10		R-4	10 18 24	127.6	8.5	SC	@10' Clayey SAND: dark brown, moist, dense	
350									
	15		SPT-1	7 13 16		6.6	SP-SM	@15' SAND with Silt: light brown, moist, dense	-#200
345									
	20		R-5	13 34 50/5"	110.6	4.5		@20' SAND with Silt: orangish brown, slightly moist, very dense	
340									
	25		SPT-2	17 27 47		3.5	SP	@25' SAND with Gravel: light brown, slightly moist, very dense	
335									
	30								

Last Edited: 2/26/2018

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

SAMPLE TYPES:

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

TEST TYPES:

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

Geotechnical Boring Log Borehole LGC-HS-1

Date: 1/31/2018	**Drilling Company:** 2R Drilling
Project Name: Glenelder	**Type of Rig:** CME 75
Project Number: 18012-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~358' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By SHH
Sampled By SHH
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-6	20 24 20	99.8	27.7	SC/ML	@30' Clayey SAND to Sandy SILT: dark brown to yellowish brown, very moist, dense	
330									
	35		SPT-3	8 10 25		19.6	ML	@35' Sandy SILT: brown, very moist, hard	
325									
	40		R-7	16 50/5"	123.7	9.7	SM	@40' Silty SAND with Gravel: brown, wet, very dense	
320									
	45		SPT-4	7 9 6		10.7	SP-SM	@45' SAND with Silt: brown, wet, medium dense	-#200
315									
	50		R-8	7 14 21	106.3	21.1	CL	@50' CLAY: light brown, wet, very stiff	
310								Total Depth = 50' Groundwater Encountered at Approximately 41.5' Backfilled with Cuttings on 1/31/2018	
	55								
305									
	60								

LGC Geotechnical, Inc.

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

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

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

Geotechnical Boring Log Borehole LGC-HS-2

Date: 1/31/2018	**Drilling Company:** 2R Drilling
Project Name: Glenelder	**Type of Rig:** CME 75
Project Number: 18012-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~361' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By SHH
Sampled By SHH
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0								MD
								Quaternary Younger Alluvial Fan Deposits (Qyf):	
			R-1	8 12 18	113.5	12.2	ML	@2.5' Sandy SILT: grayish brown, moist, very stiff	
360	5	B-1	R-2	13 25 21	132.2	7.9		@5' Sandy SILT: brown with rusty brown, slightly moist, hard	
			R-3	5 8 8	114.2	13.4		@7.5' Sandy SILT: brown with rusty brown, moist, stiff	
355	10		R-4	6 8 11	117.2	11.7	SM	@10' Silty SAND: brown with rusty brown, slightly moist, medium dense	
350	15		SPT-1	5 7 12		18.1	ML	@15' SILT: brown, very moist, very stiff	
345	20		R-5	8 14 23	108.1	6.4	SM	@20' Sitly SAND: yellowish brown, slightly moist, medium dense	
340	25		SPT-2	7 23 30		1.7	SP	@25' SAND with Gravel: light brown, dry, very dense	
								Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 1/31/2018	
	30								

LGC Geotechnical, Inc.

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

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

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

Last Edited: 2/26/2018

Geotechnical Boring Log Borehole LGC-HS-3

Date: 1/31/2018 | **Drilling Company:** 2R Drilling
Project Name: Glenelder | **Type of Rig:** CME 75
Project Number: 18012-01 | **Drop:** 30" | **Hole Diameter:** 8"
Elevation of Top of Hole: ~358' MSL | **Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map

Logged By SHH
Sampled By SHH
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						**Quaternary Younger Alluvial Fan Deposits (Qyf):**	
360			R-1	6 16 23	115.7	9.5	ML	@2.5' Sandy SILT: dark brown, moist, very stiff	
	5		R-2	5 27 28	125.3	7.8		@5' Sandy SILT: dark brown, slightly moist, dense	
355			R-3	14 25 33	122.4	9.2	CL-ML	@7.5' Silty CLAY: dark brown, slightly moist, dense	CO
	10		R-4	12 27 45	115.5	17.5	ML	@10' Sandy SILT: dark brown, moist, very dense	
350									
	15		SPT-1	8 12 10		21.1		@15' Sandy SILT: brown, very moist, very stiff	
345									
	20		R-5	8 11 26	112.2	9.9	SM	@20' Silty SAND: yellowish brown, moist, medium dense	
340									
	25		SPT-2	14 28 33		2.1	SP	@25' SAND with Gravel: light brown, slightly moist, very dense	
335								Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 1/31/2018	
	30								

Geotechnical, Inc.

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

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

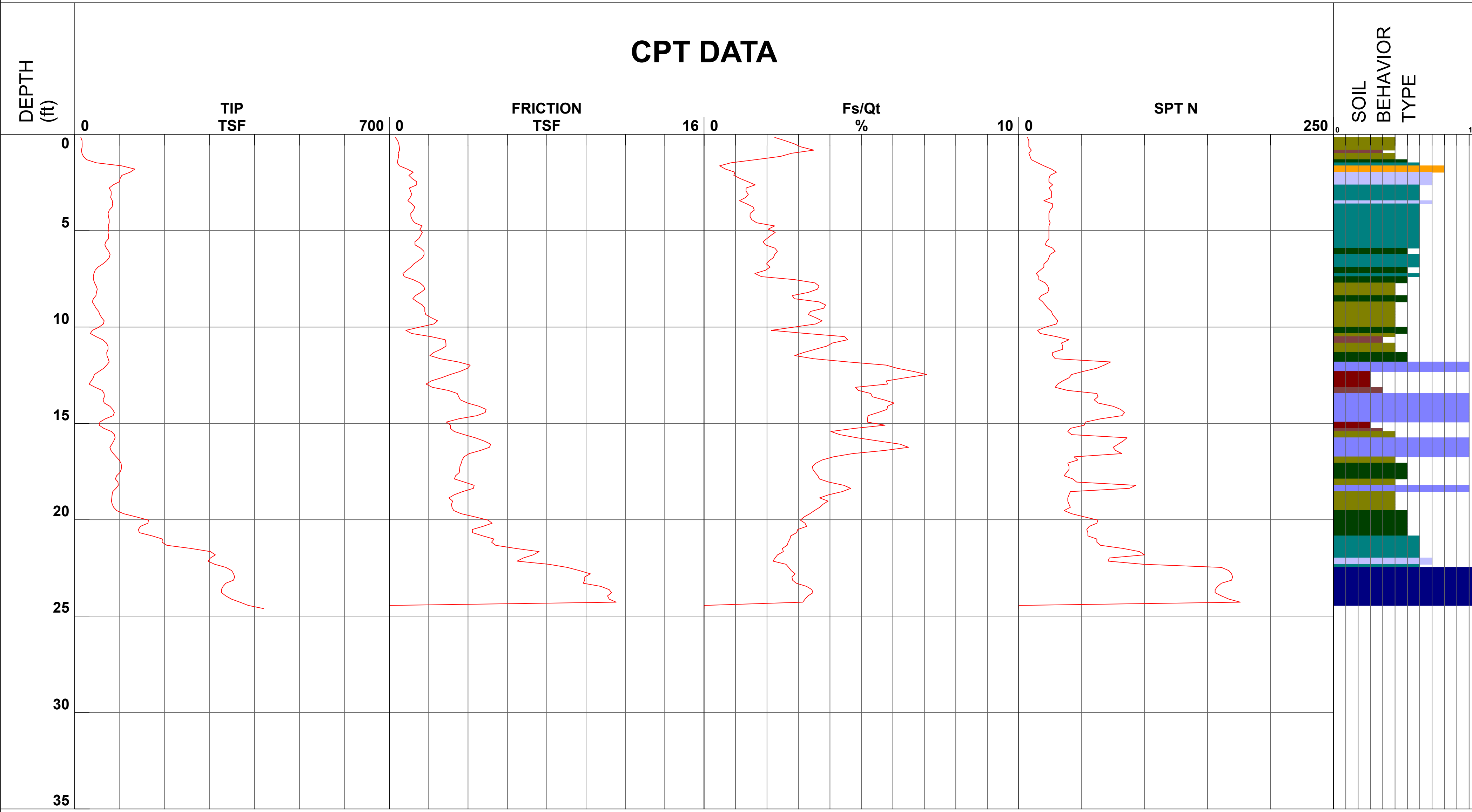
TEST TYPES:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
- S&H SIEVE AND HYDROMETER
- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Last Edited: 2/26/2018

LGC Geotechnical, Inc.

Project	Glenelder	Operator	RC AS	Filename	SDF(502).cpt
Job Number	18012-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	2/2/2018 7:59:29 AM	Maximum Depth	24.61 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

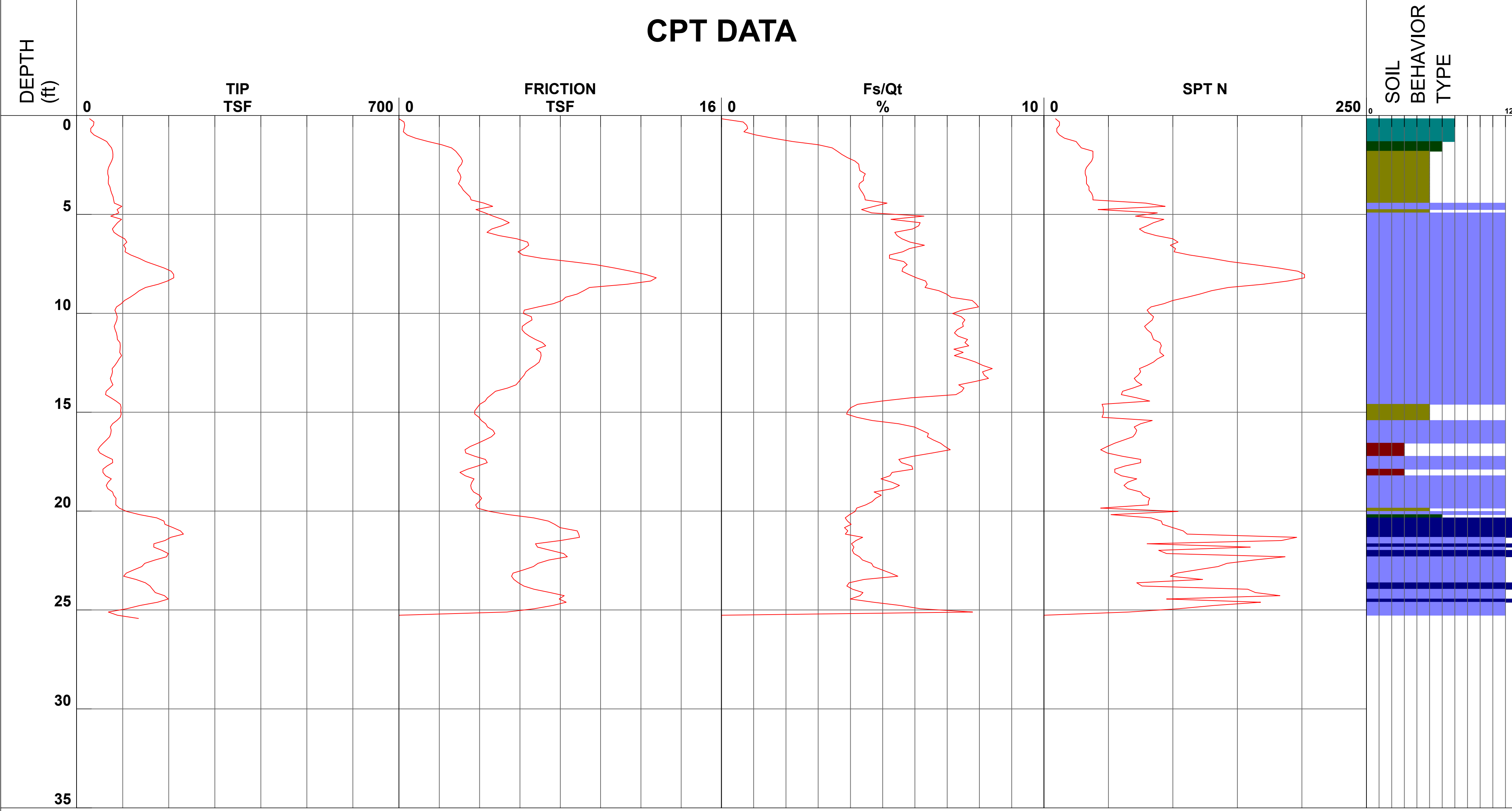
S*Soil behavior type and SPT based on data from UBC-1983

LGC Geotechnical, Inc.

Project	Glenelder	Operator	RC AS	Filename	SDF(503).cpt
Job Number	18012-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	2/2/2018 8:52:54 AM	Maximum Depth	25.43 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

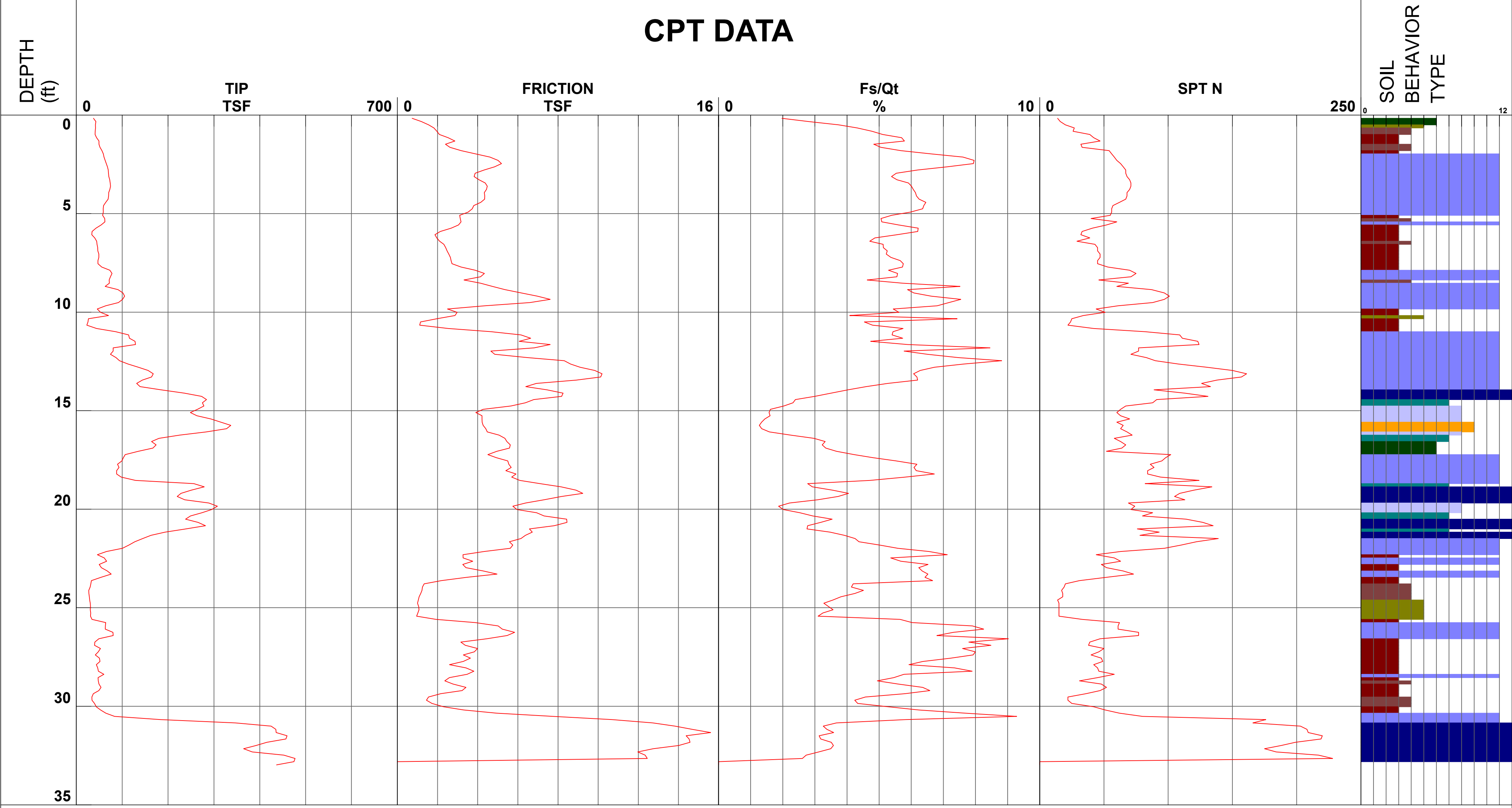
S*Soil behavior type and SPT based on data from UBC-1983

LGC Geotechnical, Inc.

Project	Glenelder	Operator	RC AS	Filename	SDF(504).cpt
Job Number	18012-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	2/2/2018 9:14:46 AM	Maximum Depth	32.97 ft
EST GW Depth During Test			20.00 ft		

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

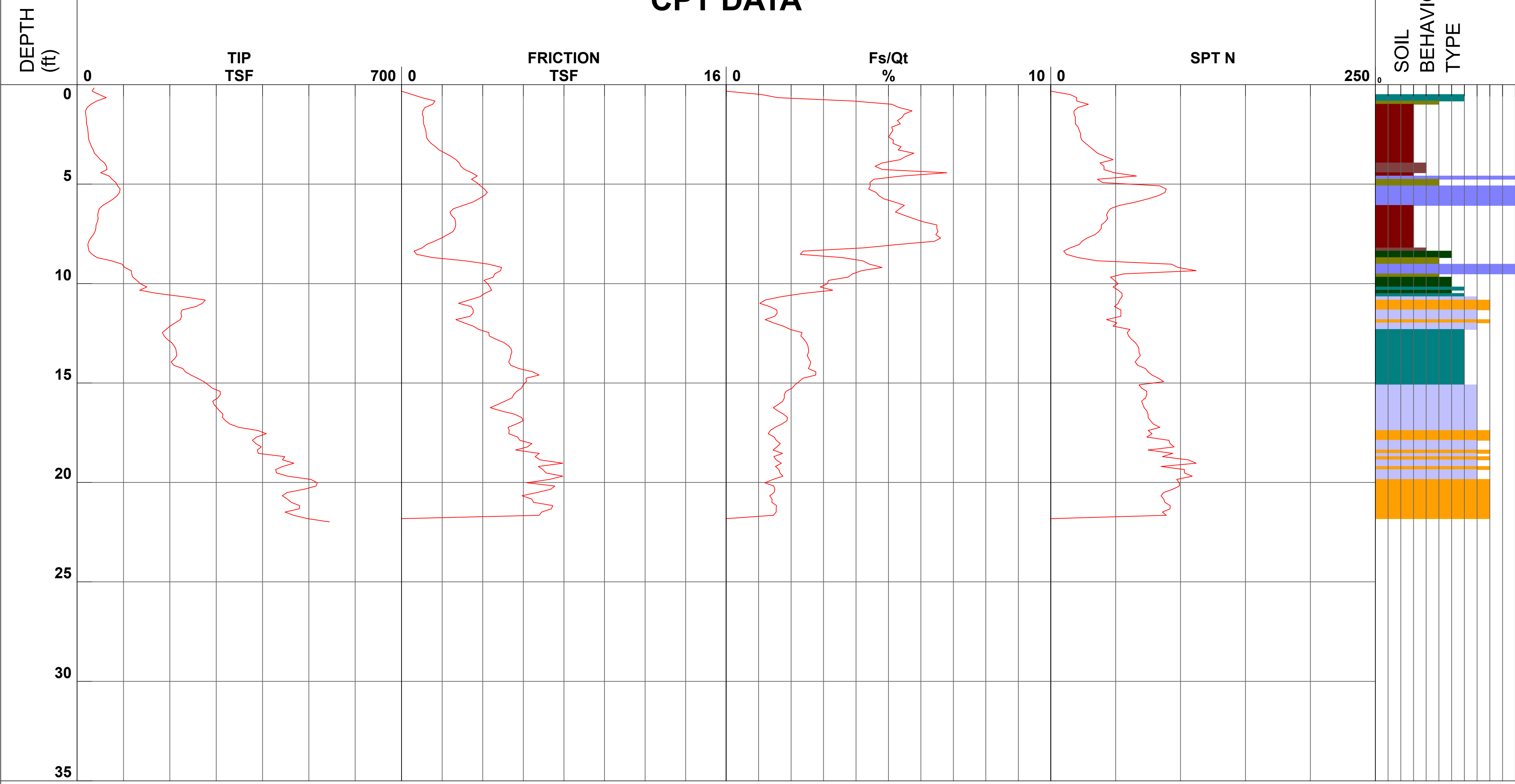
LGC Geotechnical, Inc.

Middle Earth
Geo Testing Inc.

Project	Glenelder	Operator	RC AS	Filename	SDF(505).cpt
Job Number	18012-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	2/2/2018 10:05:32 AM	Maximum Depth	21.98 ft
EST GW Depth During Test			20.00 ft		

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

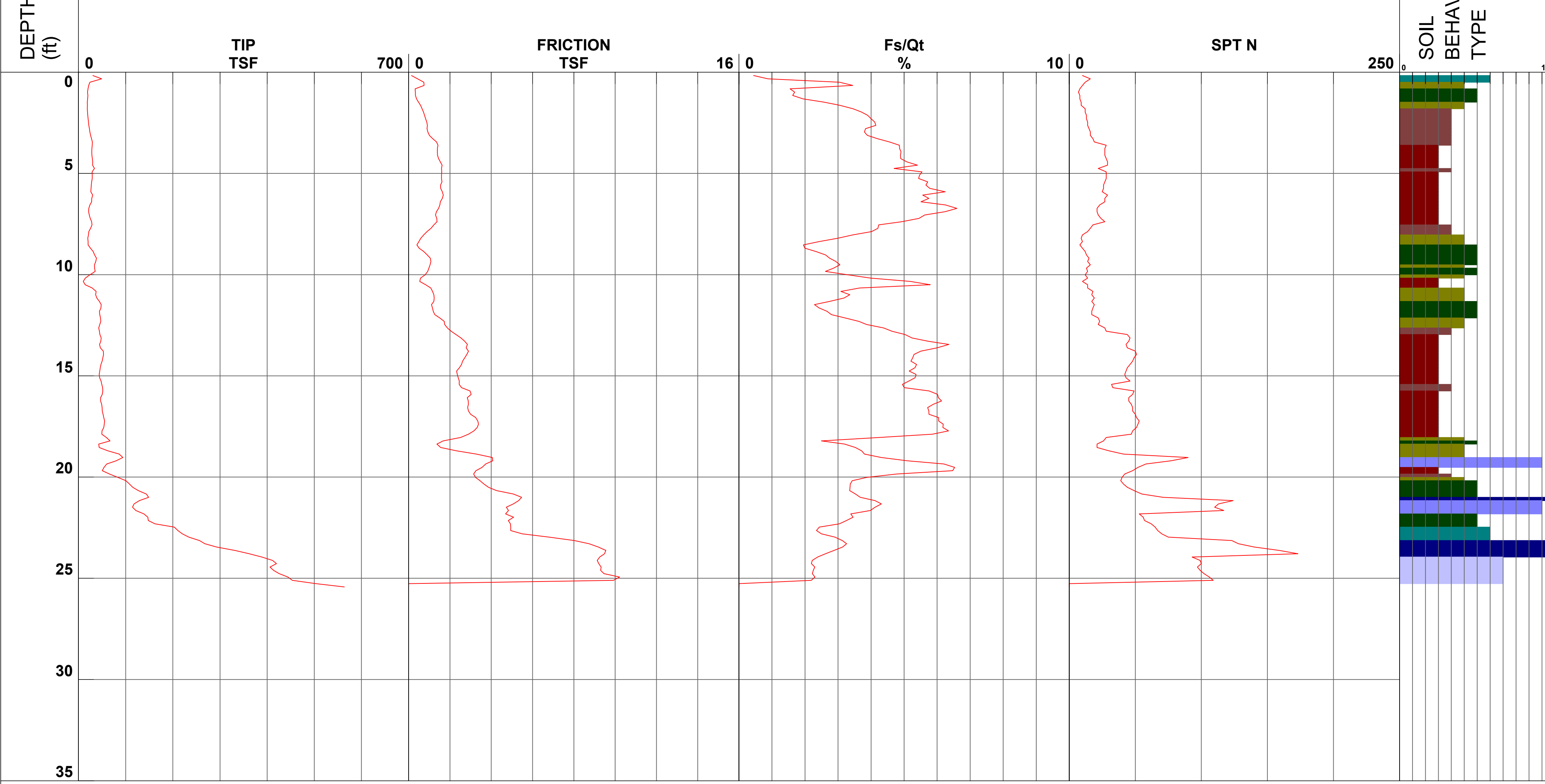
S*Soil behavior type and SPT based on data from UBC-1983

LGC Geotechnical, Inc.

Project	Glenelder	Operator	RC AS	Filename	SDF(507).cpt
Job Number	18012-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-05	Date and Time	2/2/2018 11:00:08 AM	Maximum Depth	25.43 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

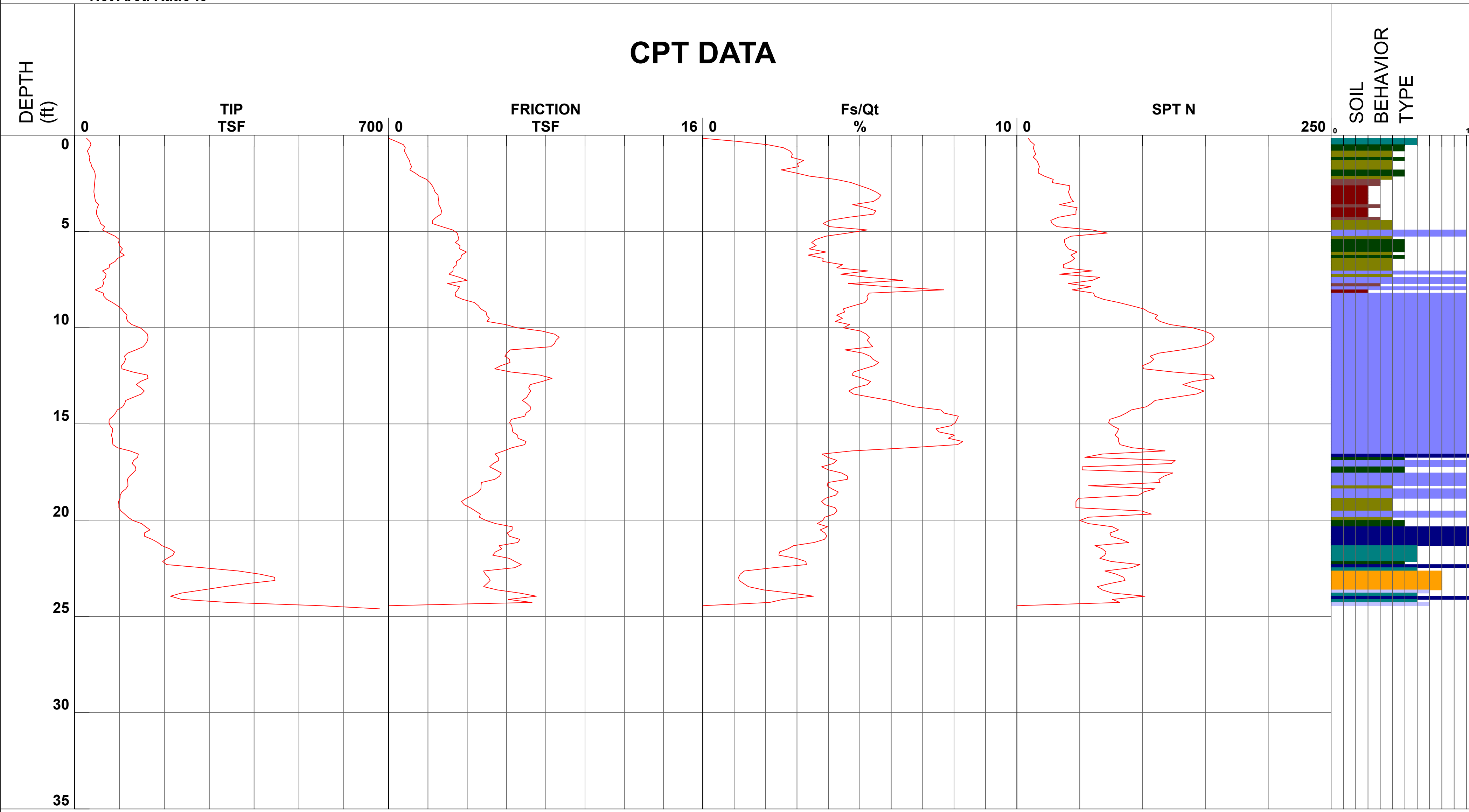
S*Soil behavior type and SPT based on data from UBC-1983

LGC Geotechnical, Inc.

Project	Glenelder	Operator	RC AS	Filename	SDF(508).cpt
Job Number	18012-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-06	Date and Time	2/2/2018 11:26:54 AM	Maximum Depth	24.61 ft
EST GW Depth During Test		20.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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. Where applicable, only moisture content was determined from SPT or disturbed samples.

Grain Size Distribution/Fines Content: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve) or ASTM D422 (sieve and hydrometer).

Sample Location	**Description**	**% Passing # 200 Sieve**
HS-1 @ 15 ft	Sand with Silt	8
HS-1 @ 45 ft	Sand with Silt	6

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 plot is provided in this Appendix.

Sample Location	**Liquid Limit (%)**	**Plastic Limit (%)**	**Plasticity Index (%)**	**USCS Soil Classification**
HS-3 @ 7.5 ft	20	15	5	CL-ML

Direct Shear: One direct shear test was performed on a relatively undisturbed driven sample. The ring samples were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plot is provided in this Appendix.

Laboratory Test Results

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

Collapse/Swell Potential: One collapse test was performed per ASTM D4546. A sample (2.4 inches in diameter and 1-inch in height) was placed in a consolidometer and loaded to its approximate in-situ effective stress. The curve is presented in this Appendix.

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2 @ 0-5 ft	Sandy Silt	121.0	13.0

Expansion Index: The expansion potential of a selected representative sample was evaluated by the Expansion Index Test per ASTM D4829.

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 0-5 ft	60	Medium

* Per ASTM D4829

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

Sample Location	R-value
HS-1 @ 0-5 ft	12

Laboratory Test Results

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

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

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

Sample Location	Chloride Content (ppm)
HS-1 @ 0-5 ft	66

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-1 @ 0-5 ft	7.0	1,000

LGC Geotechnical, Inc.

Percent Passing No. 200 Sieve

Project Name : Glenelder **Tested By :** CB

Project Number : 18012-01 **Date:** 1/31/2018

Boring/ Trench No.	Sample No.	Depth (ft)	Total Dry Weight (grams)	Weight *Retained* No. 200 Sieve (grams)	Weight *Passing* No. 200 Sieve (grams)	% Passing No. 200 Sieve (Fines Content) (%)
			A	B	C = A-B	D= (C / A) * 100
HS-1	S-1	15	51.2	46.9	4.25	8%
HS-1	S-4	45	56.9	53.6	3.27	6%

ATTERBERG LIMITS

ASTM D 4318

Project Name:	Glenelder	Tested By:	R. Manning	Date:	02/13/18
Project No. :	18012-01	Input By:	J. Ward	Date:	02/21/18
Boring No.:	HS-1	Checked By:	J. Ward		
Sample No.:	R-3	Depth (ft.)	7.5		
Soil Identification:	Dark brown silty clay with sand (CL-ML)				

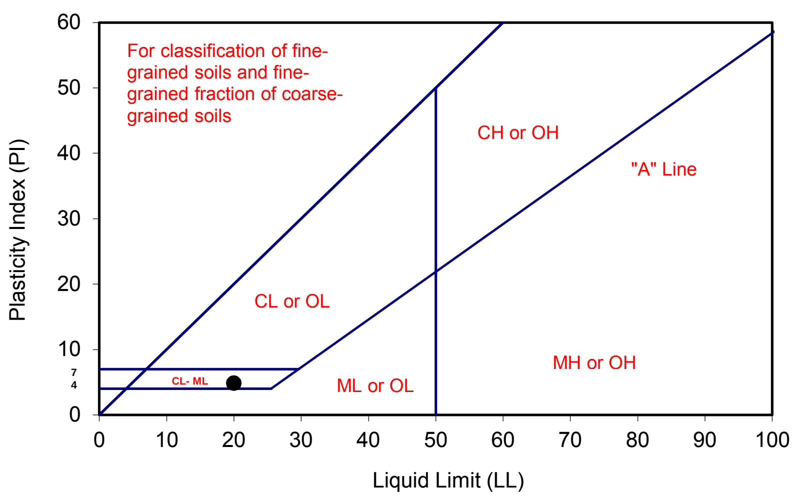
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			29	22	16	
Wet Wt. of Soil + Cont. (g)	20.63	21.47	28.57	28.25	26.41	
Dry Wt. of Soil + Cont. (g)	19.69	20.44	26.10	25.81	24.19	
Wt. of Container (g)	13.49	13.61	13.61	13.76	13.56	
Moisture Content (%) [Wn]	15.16	15.08	19.78	20.25	20.88	

Liquid Limit	**20**
Plastic Limit	**15**
Plasticity Index	**5**
Classification	**CL-ML**

PI at "A" - Line = 0.73(LL-20) | 0

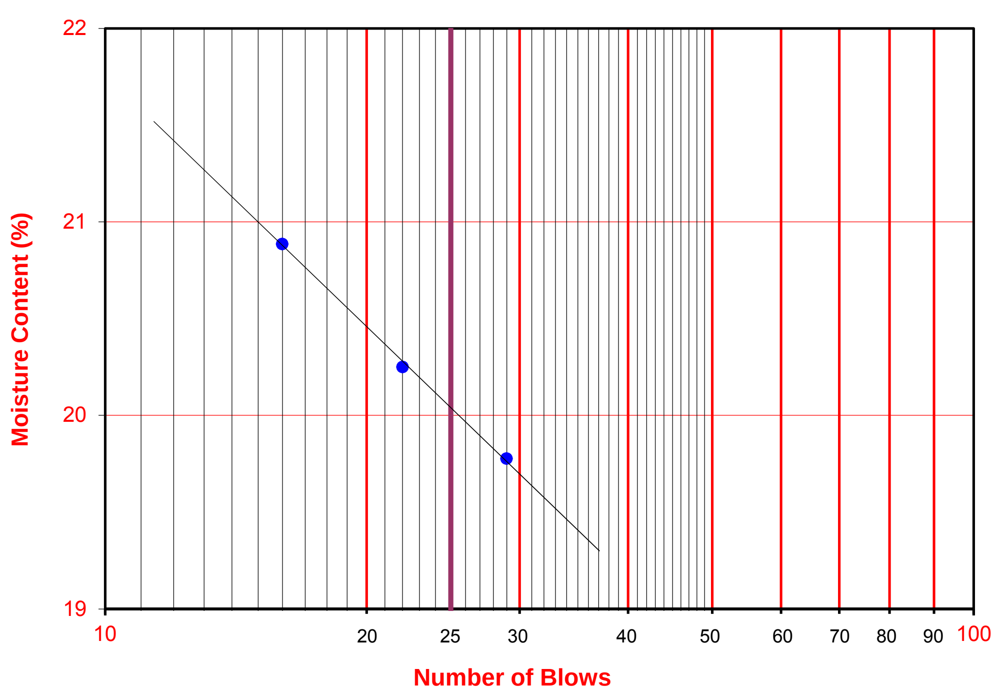
One - Point Liquid Limit Calculation

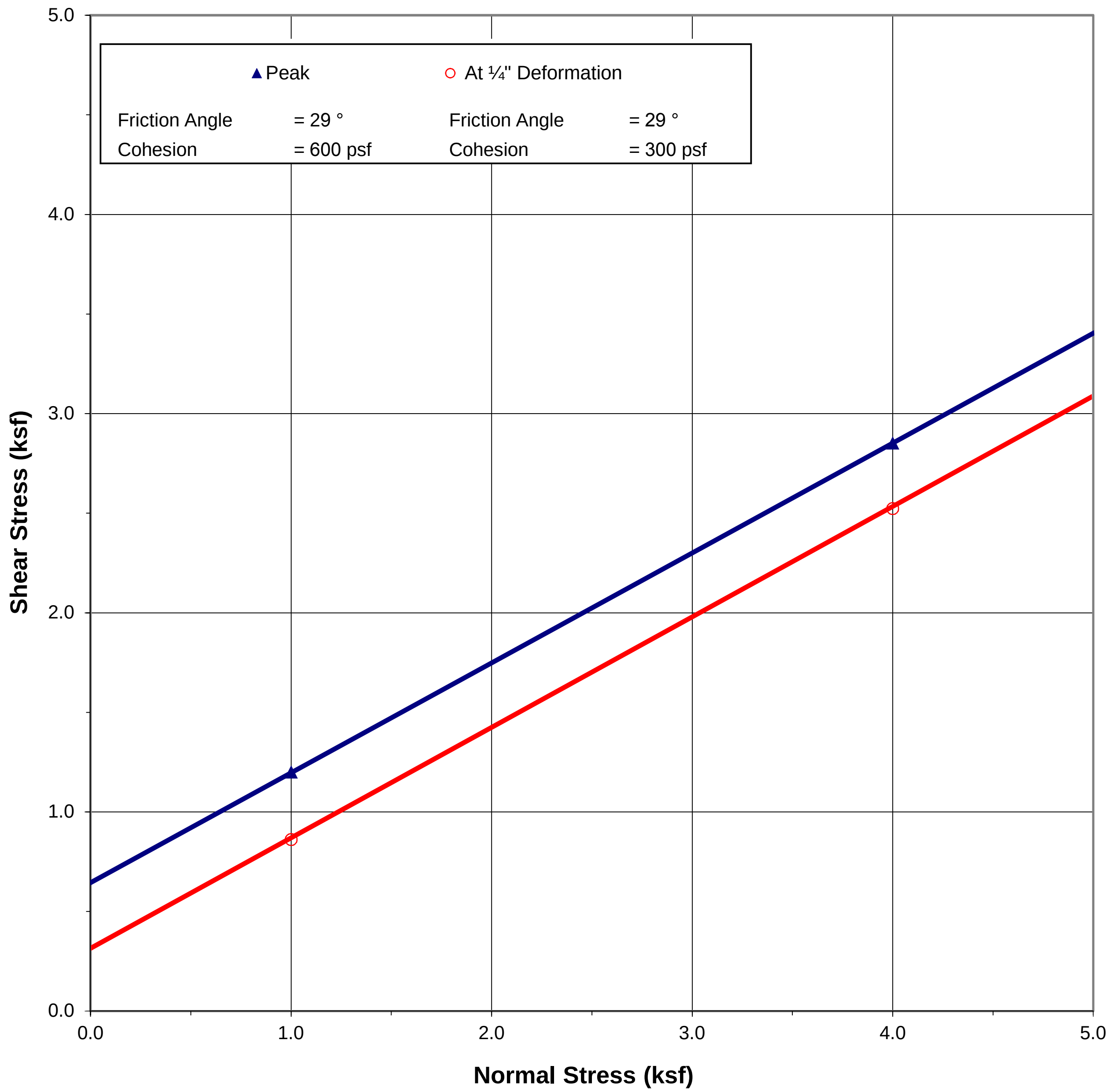
$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

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





Location:	Sample No.:	Depth (ft)	Sample Type	Shear Rate (inch/min)	Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)
HS-1	**R-2**	**5'**	**Ring**	**0.0005**	**116.2**	**15.0**	**19.0**

Sample Description: Dark brown sandy silt (ML)

LGC Geotechnical, Inc.

DIRECT SHEAR PLOT

Project Number: 18012-01
Date: Jan-18

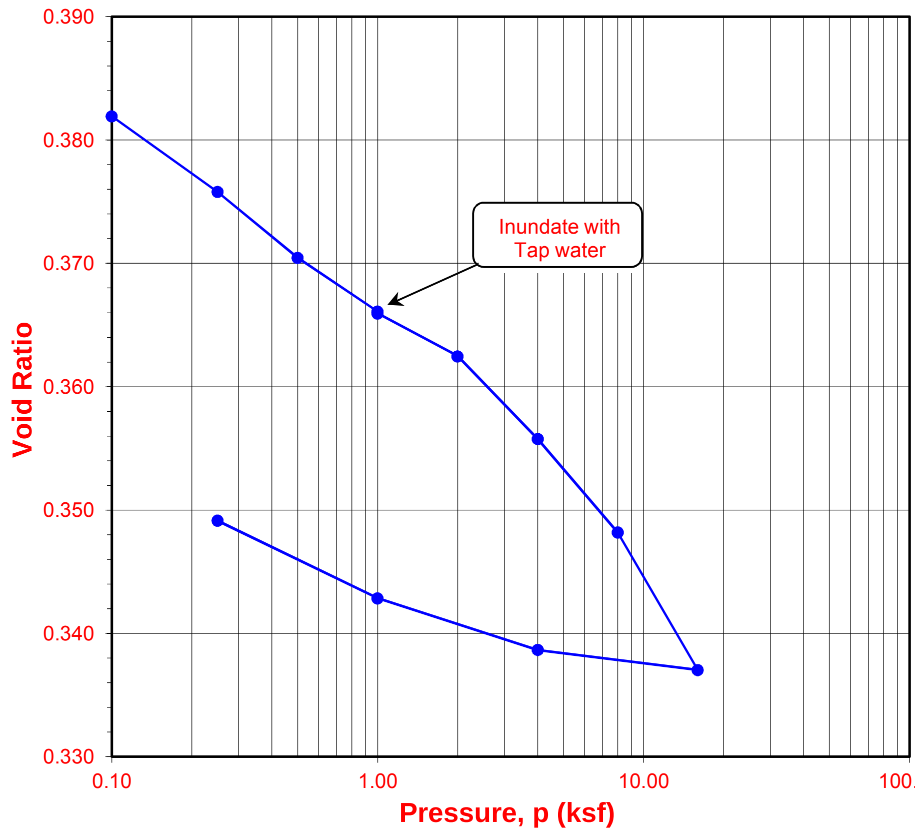
Glenelder

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Glenelder	Tested By: GB/ACS	Date: 02/06/18
Project No.:	18012-01	Checked By: J. Ward	Date: 02/21/18
Boring No.:	HS-1	Depth (ft.): 7.5	
Sample No.:	R-3	Sample Type: Ring	
Soil Identification:	Dark brown silty clay with sand (CL-ML)		

Sample Diameter (in.)	2.415
Sample Thickness (in.)	0.8580
Wt. of Sample + Ring (g)	179.71
Weight of Ring (g)	37.71
Height after consol. (in.)	0.8313
Before Test	
Wt.Wet Sample+Cont. (g)	328.44
Wt.of Dry Sample+Cont. (g)	294.89
Weight of Container (g)	39.30
Initial Moisture Content (%)	13.1
Initial Dry Density (pcf)	121.7
Initial Saturation (%)	92
Initial Vertical Reading (in.)	0.3023
After Test	
Wt.of Wet Sample+Cont. (g)	246.01
Wt. of Dry Sample+Cont. (g)	230.16
Weight of Container (g)	67.23
Final Moisture Content (%)	12.66
Final Dry Density (pcf)	125.3
Final Saturation (%)	99
Final Vertical Reading (in.)	0.2765
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

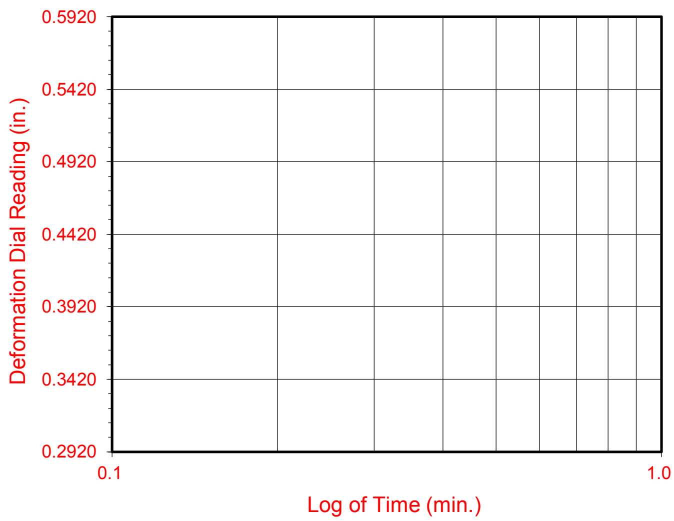


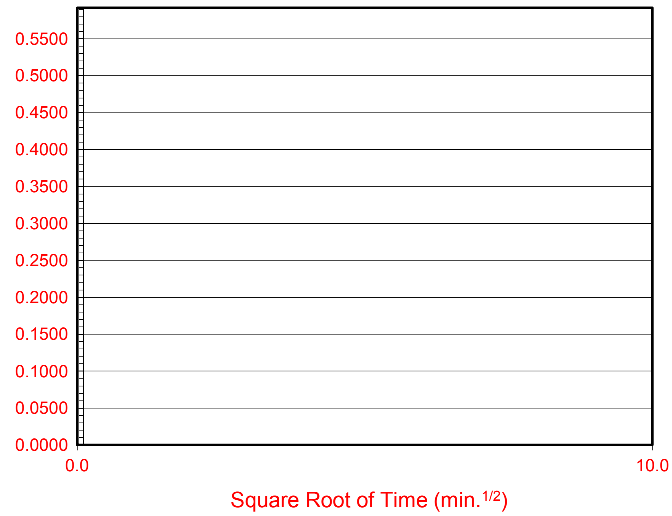
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3001	0.8559	0.00	0.25	0.382	0.25
0.25	0.2960	0.8518	0.03	0.73	0.376	0.70
0.50	0.2923	0.8481	0.07	1.16	0.370	1.09
1.00	0.2886	0.8444	0.17	1.59	0.366	1.42
1.00	0.2885	0.8443	0.17	1.60	0.366	1.43
2.00	0.2851	0.8408	0.30	2.00	0.362	1.70
4.00	0.2794	0.8352	0.45	2.66	0.356	2.21
8.00	0.2726	0.8284	0.66	3.46	0.348	2.80
16.00	0.2631	0.8189	0.92	4.56	0.337	3.64
4.00	0.2670	0.8228	0.63	4.11	0.339	3.48
1.00	0.2716	0.8274	0.43	3.57	0.343	3.14
0.25	0.2765	0.8323	0.33	3.00	0.349	2.67

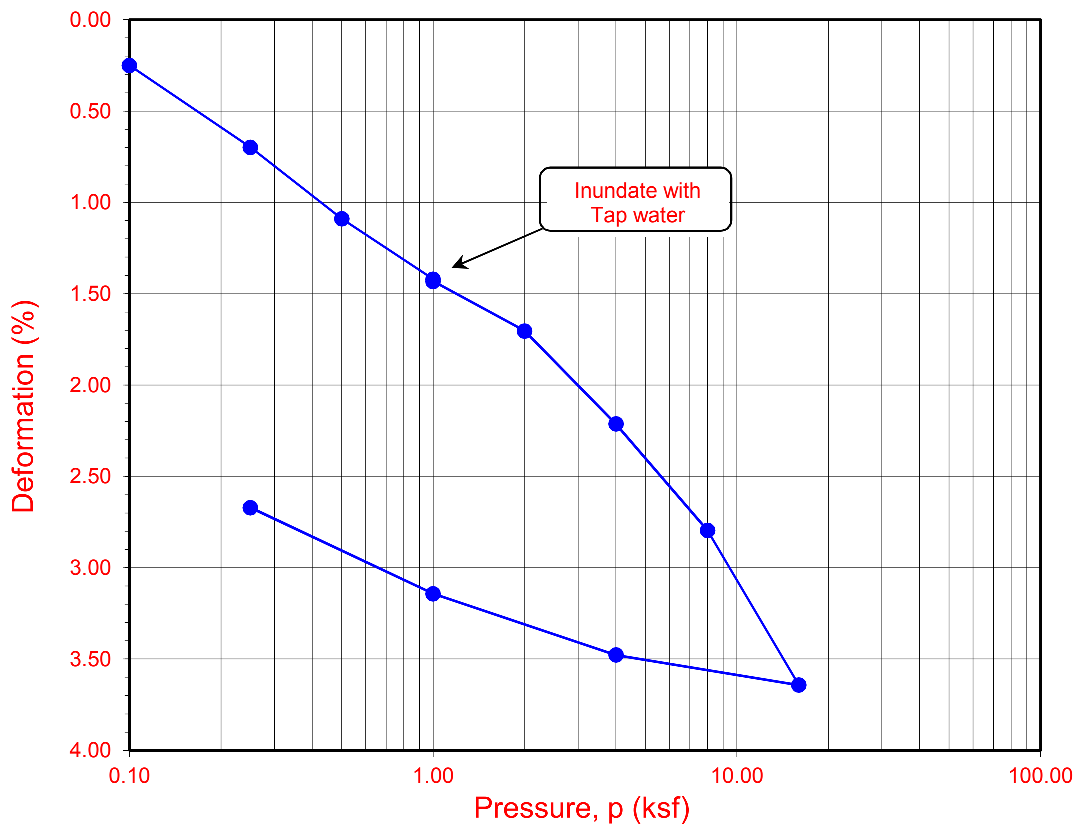
Time Readings

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

Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-1	**R-3**	**7.5**	**13.1**	**12.7**	**121.7**	**125.3**	**0.385**	**0.349**	**92**	**99**

Soil Identification: Dark brown silty clay with sand (CL-ML)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 18012-01

Glenelder

02-18

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

Project Name:	Glenelder	Tested By:	G. Bathala	Date:	02/06/18
Project No.:	18012-01	Checked By:	J. Ward	Date:	02/21/18
Boring No.:	HS-3	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.)	7.5		

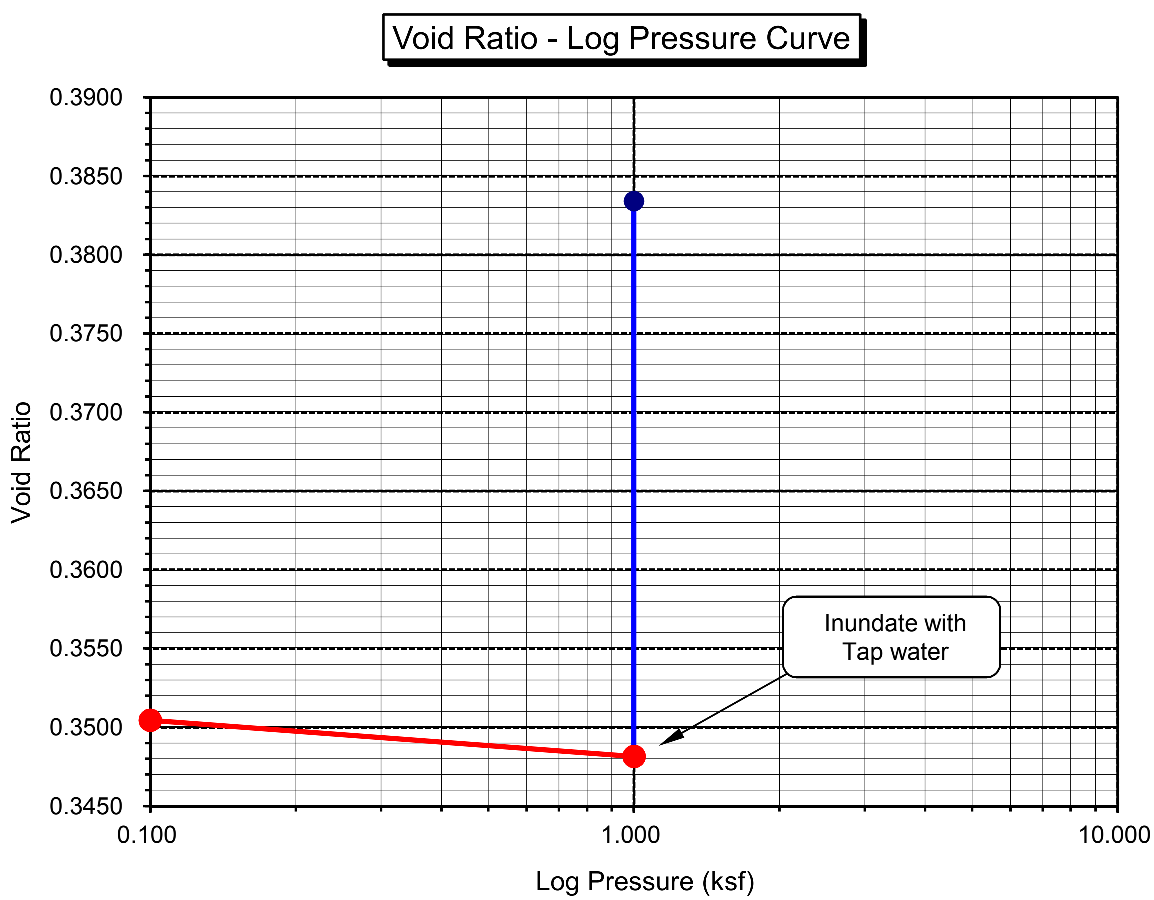
Sample Description: Dark brown silty clay (CL-ML)

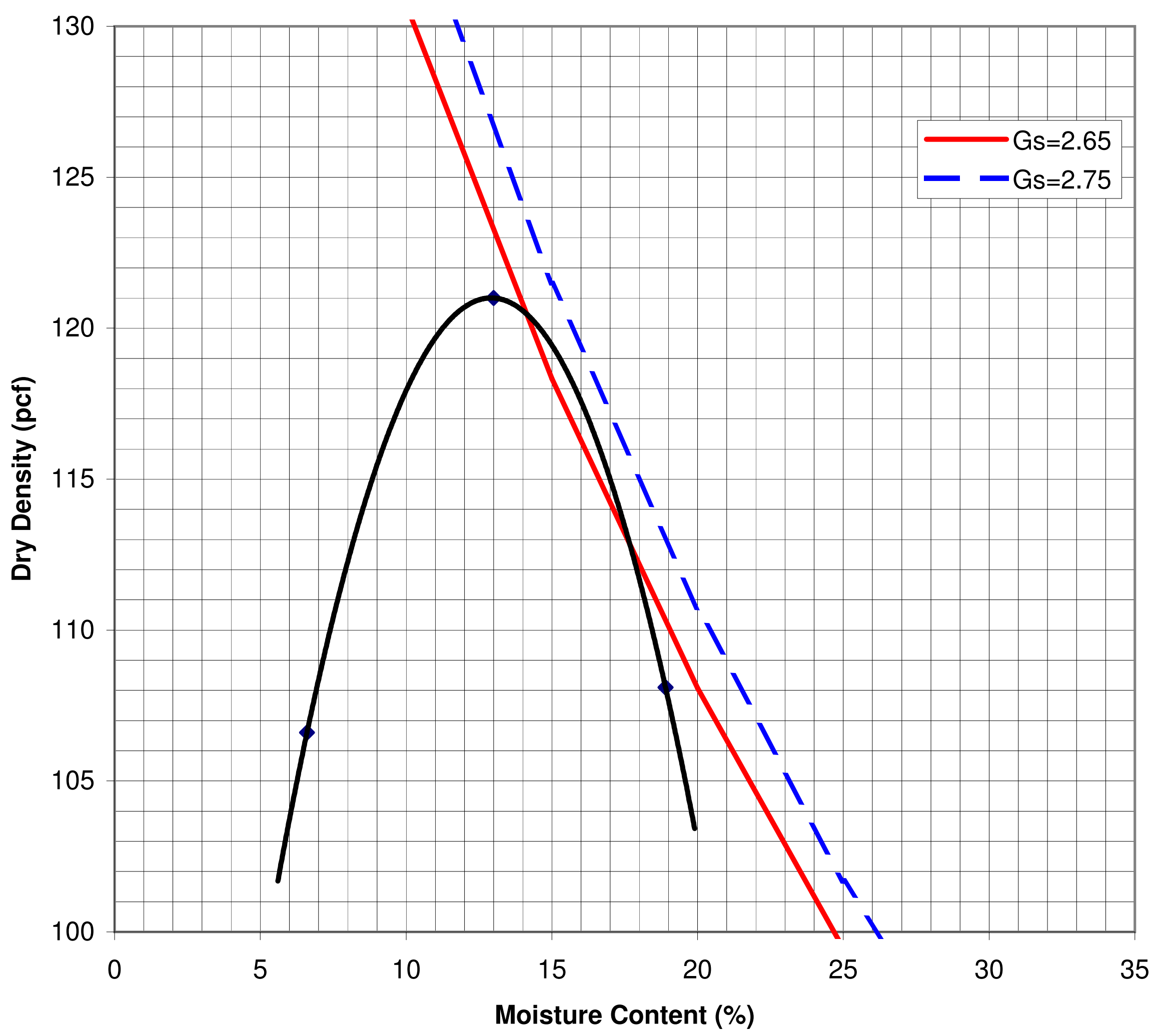
Initial Dry Density (pcf):	124.8
Initial Moisture (%):	9.21
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2247
Diameter(in):	2.415

Final Dry Density (pcf):	121.8
Final Moisture (%) :	13.9
Initial Void Ratio:	0.3508
Specific Gravity(assumed):	2.70
Initial Saturation (%)	70.9

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2244	0.9997	0.00	-0.03	0.3504	-0.03
1.000	0.2206	0.9959	0.21	-0.41	0.3481	-0.20
H2O	0.2467	1.0220	0.21	2.20	0.3834	2.41

Percent Swell (+) / Settlement (-) After Inundation = 2.62





Location:	Sample No.:	Depth (ft)	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2	**HS-2 B-1**	**0-5'**	**Grayish brown sandy silt**	**121.0**	**13.0**

LGC Geotechnical, Inc.

LABORATORY COMPACTION
(ASTM D 1557)

Project Number: 18012-01
Date: Jan-18

Glenelder

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Glenelder	Tested By:	S. Felter	Date:	02/06/18
Project No.:	18012-01	Checked By:	J. Ward	Date:	02/21/18
Boring No.:	HS-1	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark brown sandy silt (ML)				

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	1.0590
Wt. Comp. Soil + Mold	(g)	577.60	428.57
Wt. of Mold	(g)	190.30	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	778.10	618.87
Dry Wt. of Soil + Cont.	(g)	701.00	539.22
Wt. of Container	(g)	0.00	190.30
Moisture Content	(%)	11.00	22.83
Wet Density	(pcf)	116.8	122.1
Dry Density	(pcf)	105.3	99.4
Void Ratio		0.602	0.696
Total Porosity		0.376	0.410
Pore Volume	(cc)	77.8	90.0
Degree of Saturation (%) [S_{meas}]		**49.4**	88.5

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.)
02/06/18	10:57	1.0	0	0.1470
02/06/18	11:07	1.0	10	0.1465
Add Distilled Water to the Specimen				
02/06/18	11:40	1.0	33	0.1930
02/07/18	6:45	1.0	1178	0.2060
02/07/18	8:07	1.0	1260	0.2060

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**60**

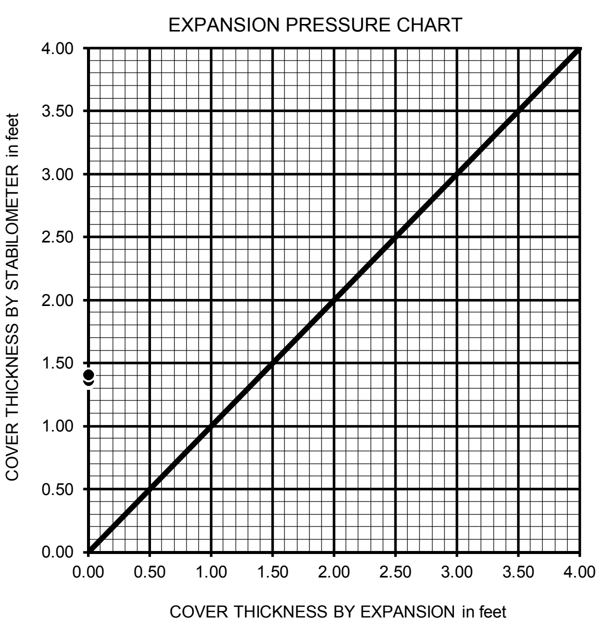
R-VALUE TEST RESULTS

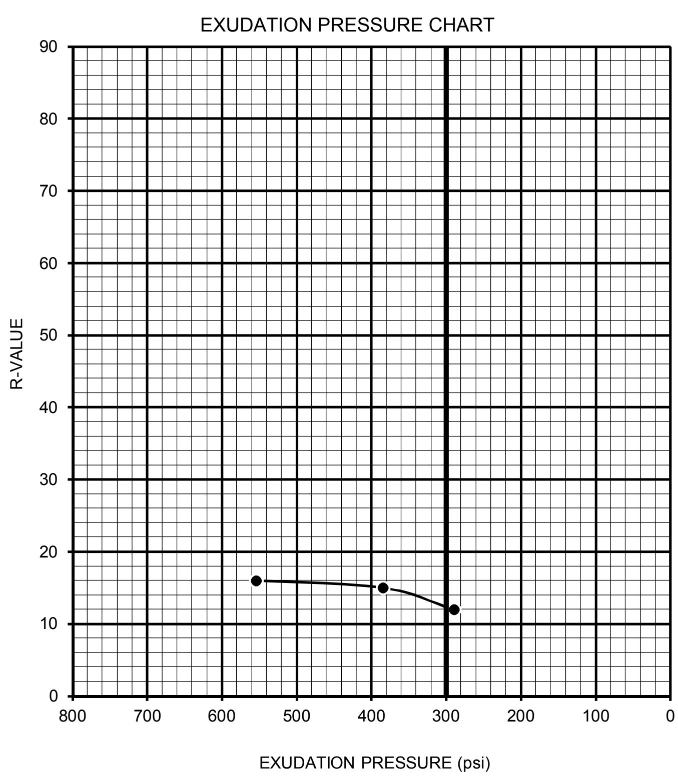
DOT CA Test 301

PROJECT NAME:	Glenelder	PROJECT NUMBER:	18012-01
BORING NUMBER:	HS-1	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	S. Felter
SAMPLE DESCRIPTION:	Dark brown sandy silt (ML)	DATE COMPLETED:	2/12/2018

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	16.9	17.4	17.8
HEIGHT OF SAMPLE, Inches	2.53	2.50	2.60
DRY DENSITY, pcf	114.1	113.9	108.7
COMPACTOR PRESSURE, psi	100	50	50
EXUDATION PRESSURE, psi	555	384	290
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	123	124	132
TURNS DISPLACEMENT	3.86	4.08	4.15
R-VALUE UNCORRECTED	16	15	11
R-VALUE CORRECTED	16	15	12

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.34	1.36	1.41
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00





R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	12
EQUILIBRIUM R-VALUE:	12

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Glenelder　　Tested By : G. Berdy　　Date: 02/06/18

Project No. : 18012-01　　Data Input By: J. Ward　　Date: 02/21/18

Boring No.	HS-1			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Dark brown (ML)			
Wet Weight of Soil + Container (g)	221.05			
Dry Weight of Soil + Container (g)	208.03			
Weight of Container (g)	69.81			
Moisture Content (%)	9.42			
Weight of Soaked Soil (g)	100.05			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	200A			
Crucible No.	12			
Furnace Temperature (°C)	860			
Time In / Time Out	9:00/9:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.6927			
Wt. of Crucible (g)	22.6889			
Wt. of Residue (g) (A)	0.0038			
PPM of Sulfate (A) x 41150	156.37			
PPM of Sulfate, Dry Weight Basis	**173**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.5			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**66**			

pH TEST, DOT California Test 643

pH Value	**6.97**			
Temperature °C	21.3			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Glenelder	Tested By :	G. Berdy	Date:	02/06/18
Project No. :	18012-01	Data Input By:	J. Ward	Date:	02/21/18
Boring No.:	HS-1	Depth (ft.) :	0-5		
Sample No. :	B-1				

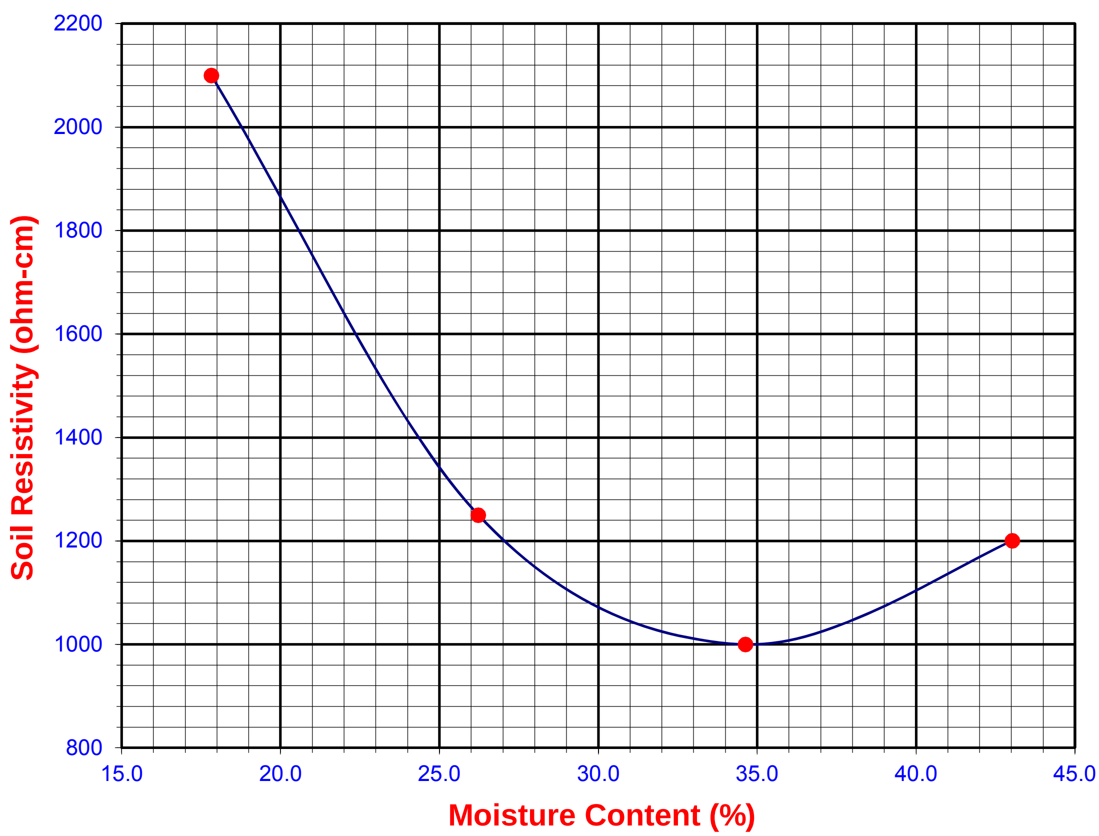
Soil Identification:* Dark brown (ML)

*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	17.82	2100	2100
2	20	26.23	1250	1250
3	30	34.63	1000	1000
4	40	43.03	1200	1200
5				

Moisture Content (%) (MCi)	9.42
Wet Wt. of Soil + Cont. (g)	221.05
Dry Wt. of Soil + Cont. (g)	208.03
Wt. of Container (g)	69.81
Container No.	
Initial Soil Wt. (g) (Wt)	130.21
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1000	**34.8**	**173**	**66**	**6.97**	**21.3**



Appendix D
Infiltration Test Data

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lennar - Glenelder
Project Number: 18012-01
Date: 2/2/2018
Location: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

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

Minimum Pre-Soak Head (D_o):

(What the sounder tape should read) Boring Depth - 1 ft = **4**

The value on the sounder tape should be slightly **Greater** than this value during testing

Pre-Soak /Pre-Test

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)	Comments
PS-1	7:53	8:53	60.0	3.32	3.32		
Pre-Test	8:53	9:03	10.0	3.32	3.32	0	
Pre-Test	8:53	9:23	30.0	3.32	3.33	0.01	

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)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	9:25	9:55	30.0	3.33	3.33	0.00	3.85	0.0
2	9:55	10:25	30.0	3.33	3.34	0.01	3.84	0.0
3	10:25	10:55	30.0	3.34	3.35	0.01	3.82	0.0
4	10:55	11:25	30.0	3.35	3.37	0.02	3.78	0.0
5	11:25	11:55	30.0	3.37	3.38	0.01	3.75	0.0
6	11:55	12:25	30.0	3.38	3.39	0.01	3.73	0.0
7	12:25	12:55	30.0	3.39	3.40	0.01	3.71	0.0
8	12:55	13:25	30.0	3.40	3.41	0.01	3.69	0.0
9								
10								
11								
12								

Measured Infiltration Rate	**0.0**
Minimum Reduction Factor	4
Max. Design Infiltration Rate	**0.0**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017

Spreadsheet Revised on: 11/29/2017

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lennar - Glenelder
Project Number: 18012-01
Date: 2/2/2018
Location: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	10
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

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

Minimum Pre-Soak Head (D_o):

(What the sounder tape should read) Boring Depth - 1 ft = **9**

The value on the sounder tape should be slightly **Greater** than this value during testing

Pre-Soak /Pre-Test

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)	Comments
PS-1	7:49	8:49	60.0	8.50	9.00		
Pre-Test	8:50	9:00	10.0	8.8	8.88	0.08	
Pre-Test	8:50	9:20	30.0	8.8	9	0.2	

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)	Surface Area of Test Section (feet ^2)	Raw Infiltration Rate (in/hr)
1	9:21	9:51	30.0	8.67	8.93	0.26	2.86	0.8
2	9:51	10:21	30.0	8.63	8.80	0.17	3.04	0.5
3	10:21	10:51	30.0	8.56	8.79	0.23	3.12	0.6
4	10:51	11:21	30.0	8.61	8.83	0.22	3.03	0.6
5	11:21	11:51	30.0	8.55	8.77	0.22	3.16	0.6
6	11:51	12:21	30.0	8.60	8.84	0.24	3.03	0.7
7	12:21	12:51	30.0	8.64	8.86	0.22	2.97	0.6
8	12:51	13:21	30.0	8.66	8.88	0.22	2.93	0.6
9								
10								
11								
12								

Measured Infiltration Rate	**0.6**
Minimum Reduction Factor	4
Max. Design Infiltration Rate	**0.2**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017

Spreadsheet Revised on: 11/29/2017

LGC Geotechnical, Inc.

Appendix E
Liquefaction Analysis

LIQUEFACTION EVALUATION

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

Seismic Event
Moment Magnitude 6.8
Peak Ground Acceleration 0.82 g

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

Depth to GWT
During Investigation (ft) 41.5
During Design Event (ft) 15

Project Name Glenelder, Hacienda Heights
Project Number 18012-01
Boring HS-1

Determination of Cyclic Resitance Ratio

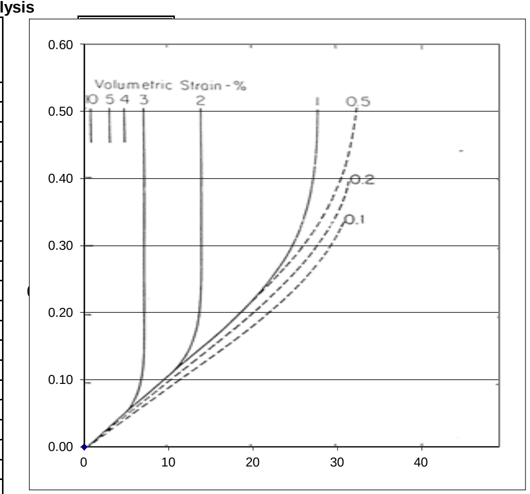
Sampling Data		Blow Count			During Investigation			Sampling Correction Factors												
Depth (ft)	Depth (m)	SPT	Rings	Thickness (ft)	Total Stress (psf)	Pore Pressure (psf)	Effective Stress (psf)	Sampler Diameter	SPT N_m	Overburden C_N	Energy C_E	Borehole C_B	Rod Length C_R	Sampler Type C_S	$(N_1)_{60}$	Fines Content	$(N_1)_{60cs}$	K_σ	$CRR_{7.5}$	Depth
2.5	0.8		20	2.5	420	0	420	0.62	12.40	1.70	1.30	1.00	0.75	1.00	20.55	75	**29.66**	1.000	0.413	2.5
5	1.5		26	2.5	720	0	720	0.62	16.12	1.70	1.30	1.00	0.75	1.00	26.77	75	**37.12**	1.000	SPT >30 NF	5
7.5	2.3		25	2.5	1020	0	1020	0.62	15.50	1.43	1.30	1.00	0.75	1.00	21.62	75	**30.95**	1.000	SPT >30 NF	7.5
10	3.0		42	2.5	1320	0	1320	0.62	26.04	1.26	1.30	1.00	0.75	1.00	31.93	35	**43.20**	1.000	SPT >30 NF	10
15	4.6	29		5	1920	0	1920	1.00	29.00	1.04	1.30	1.00	0.85	1.10	36.76	8	**37.52**	1.000	SPT >30 NF	15
20	6.1		84	5	2520	0	2520	0.62	52.08	0.91	1.30	1.00	0.95	1.00	58.55	8	**59.58**	0.964	SPT >30 NF	20
25	7.6	74		5	3120	0	3120	1.00	74.00	0.82	1.30	1.00	0.95	1.10	82.24	8	**83.58**	0.924	SPT >30 NF	25
30	9.1		44	5	3720	0	3720	0.62	27.28	0.75	1.30	1.00	0.95	1.00	25.24	50	**35.29**	0.889	SPT >30 NF	30
35	10.7	35		5	4320	0	4320	1.00	35.00	0.70	1.30	1.00	1.00	1.10	34.80	65	**46.76**	0.858	SPT >30 NF	35
40	12.2		100	5	4920	0	4920	0.62	62.00	0.65	1.30	1.00	1.00	1.00	52.51	6	**52.78**	0.830	SPT >30 NF	40
45	13.7	15		5	5520	218.4	5301.6	1.00	15.00	0.63	1.30	1.00	1.00	1.10	13.46	6	**13.55**	0.813	0.119	45
50	15.2		35	5	6120	530.4	5589.6	0.62	21.70	0.61	1.25	1.00	1.00	1.00	16.58	75	**24.89**	0.802	0.225	50
				50																

Determination of Cyclic Stress Ratio

Sampling Data		Blow Count			During Design Event							
Depth (ft)	Depth (m)	SPT	Rings	Thickness	Total Stress (psf)	Pore Pressure (psf)	Effective Stress (psf)	r_d	CSR	MSF	FS	Depth
2.5	0.76		20	2.5	300	0	300	0.99615	0.530948	1.285	Above GWT	2.5
5	1.52		26	2.5	600	0	600	0.99024	0.527796	1.285	Above GWT	5
7.5	2.29		25	2.5	900	0	900	0.98456	0.524771	1.285	Above GWT	7.5
10	3.05		42	2.5	1200	0	1200	0.97914	0.521883	1.285	Above GWT	10
15	4.57	29		5	1800	0	1800	0.96856	0.516241	1.285	Corr. SPT>30	15
20	6.10		84	5	2400	312	2088	0.9569	0.586236	1.285	Corr. SPT>30	20
25	7.62	74		5	3000	624	2376	0.94183	0.633835	1.285	Corr. SPT>30	25
30	9.14		44	5	3600	936	2664	0.92058	0.663065	1.285	Corr. SPT>30	30
35	10.67	35		5	4200	1248	2952	0.89062	0.675385	1.285	Corr. SPT>30	35
40	12.19		100	5	4800	1560	3240	0.85103	0.672002	1.285	Corr. SPT>30	40
45	13.72	15		5	5400	1872	3528	0.80363	0.655614	1.285	0.23	45
50	15.24		35	5	6000	2184	3816	0.75271	0.630813	1.285	Bray-Clay	50
												0
												0
												0
												0
												0
												0
												0
												0

Liquefaction-Induced Settlement Analysis

Depth	Vol. Strain (%) SP117 Fig7.11	Settlement (in.)
2.5		
5.0		
7.5		
10.0		
15.0		
20.0		
25.0		
30.0		
35.0		
40.0		
45.0	2.00	1.2
50.0		
	Total =	**1.2**



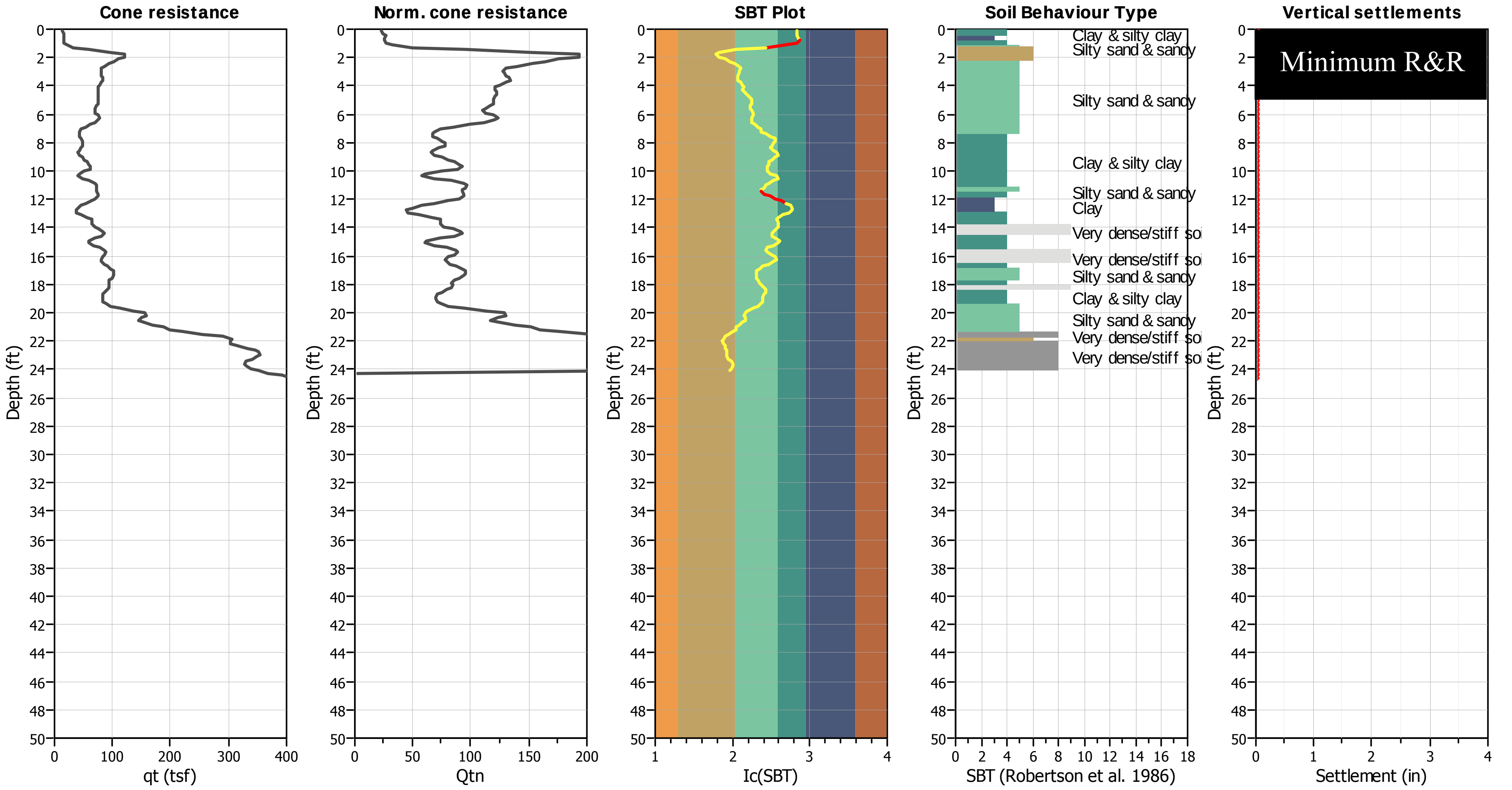
Assumptions:
8 inch diameter Boring
Standard California Modified Rings
Anvil 3 feet above top of boring

2/23/2018

LGC Geotechnical Inc.
131 Calle Iglesia Ste. 200, San Clemente, CA 92672
949-369-6141
www.lgcgeotechnical.com

Project: Lennar - Glenelder (18012-01)
Location: Hacienda Heights

CPT: CPT-01
Total depth: 24.61 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	41.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.79	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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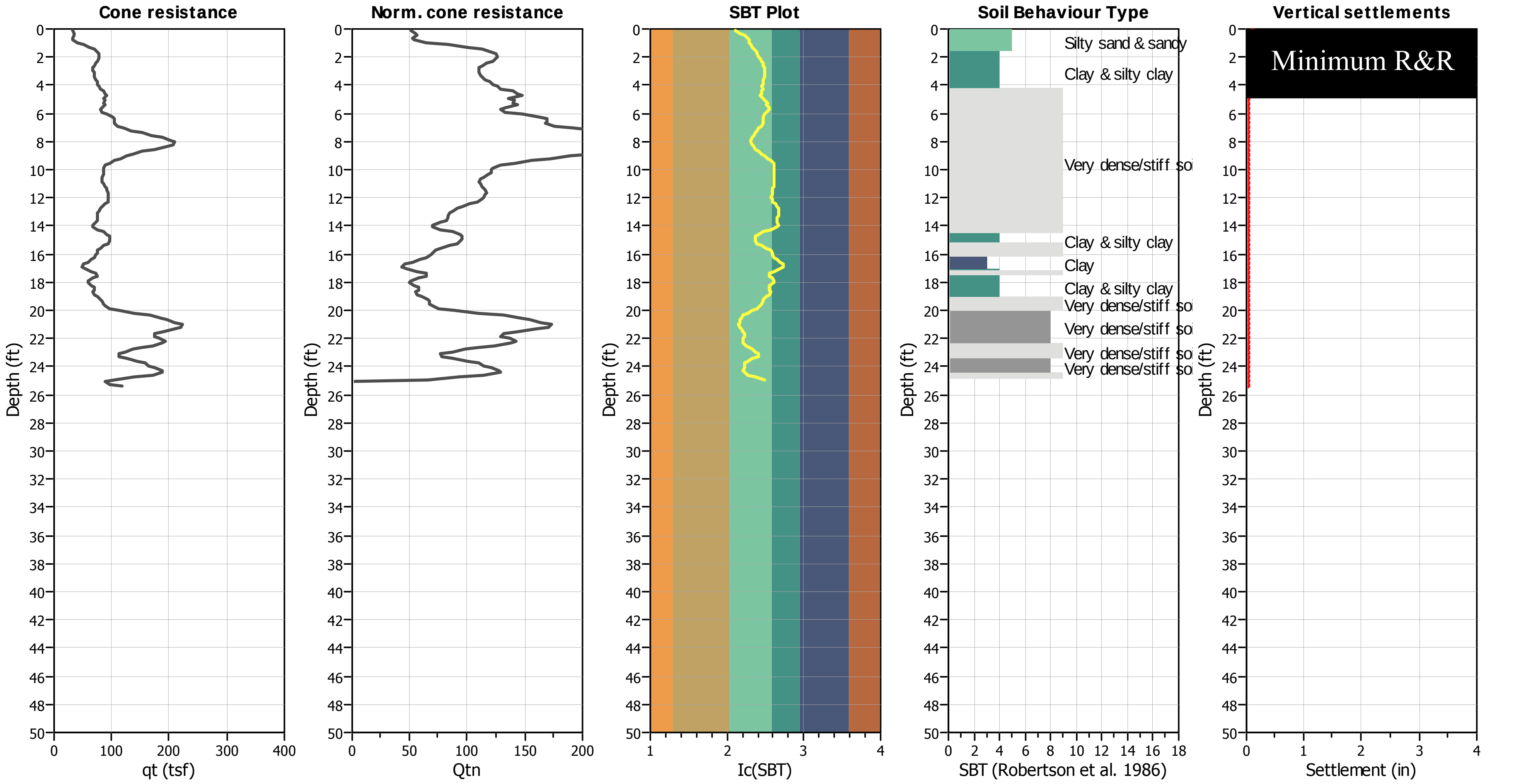
Project: **Lennar - Glenelder (18012-01)**

Location: **Hacienda Heights**

CPT: CPT-02

Total depth: 25.43 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	41.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.79	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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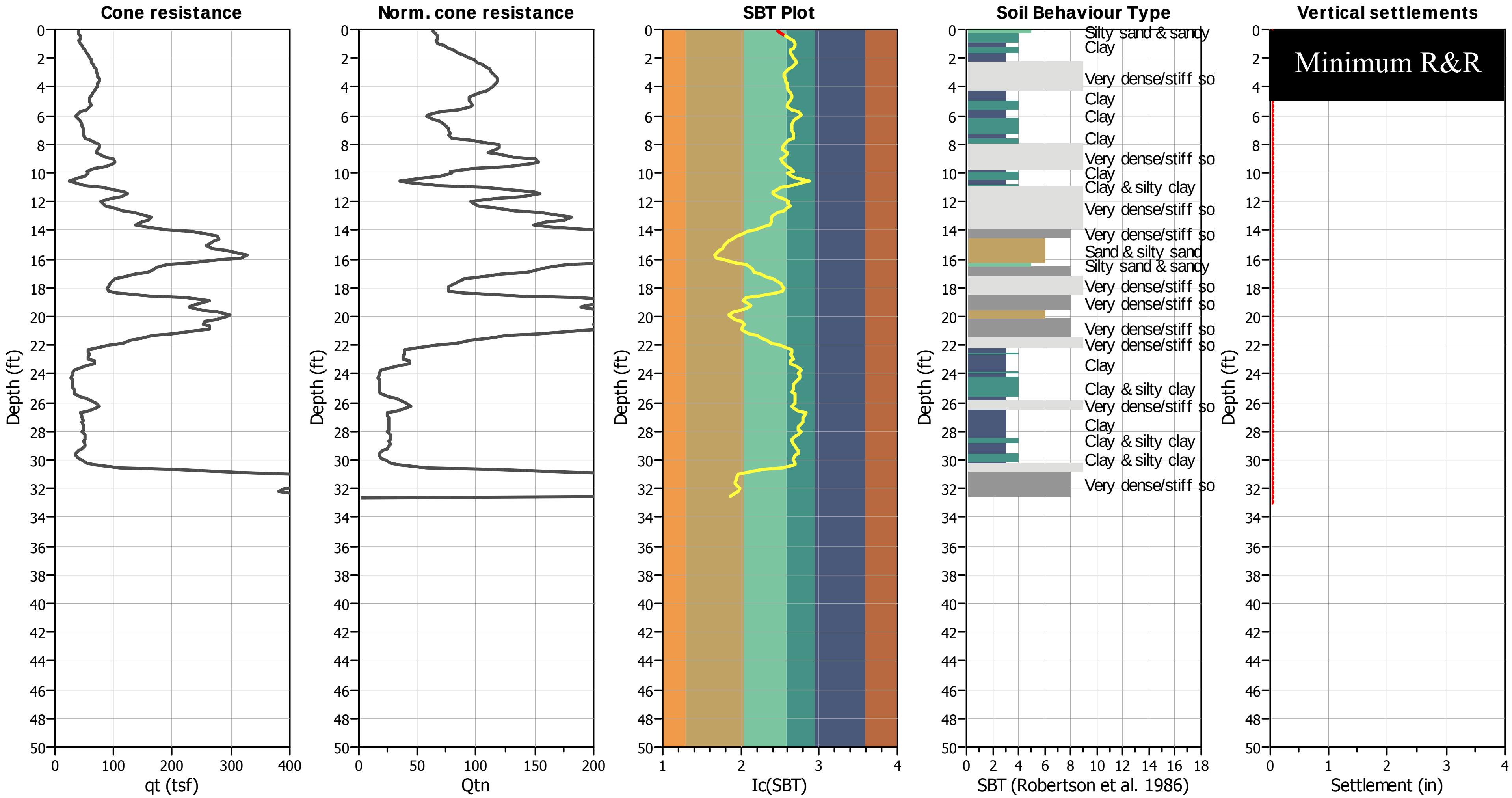
Project: **Lennar - Glenelder (18012-01)**
Location: **Hacienda Heights**

CPT: CPT-03
Total depth: 32.97 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	41.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.79	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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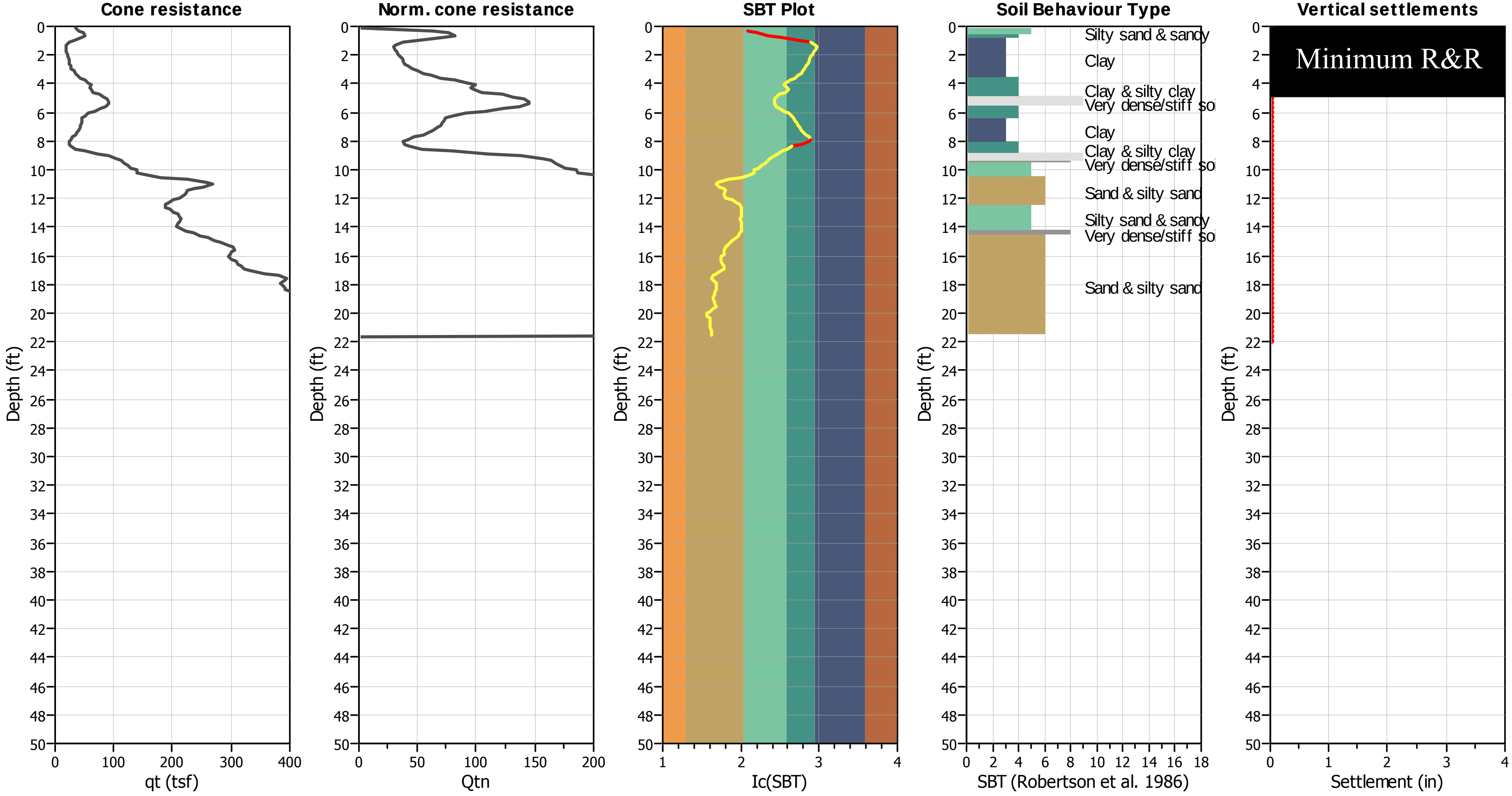
Project: Lennar - Glenelder (18012-01)

Location: Hacienda Heights

CPT: CPT-04

Total depth: 21.98 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	41.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.79	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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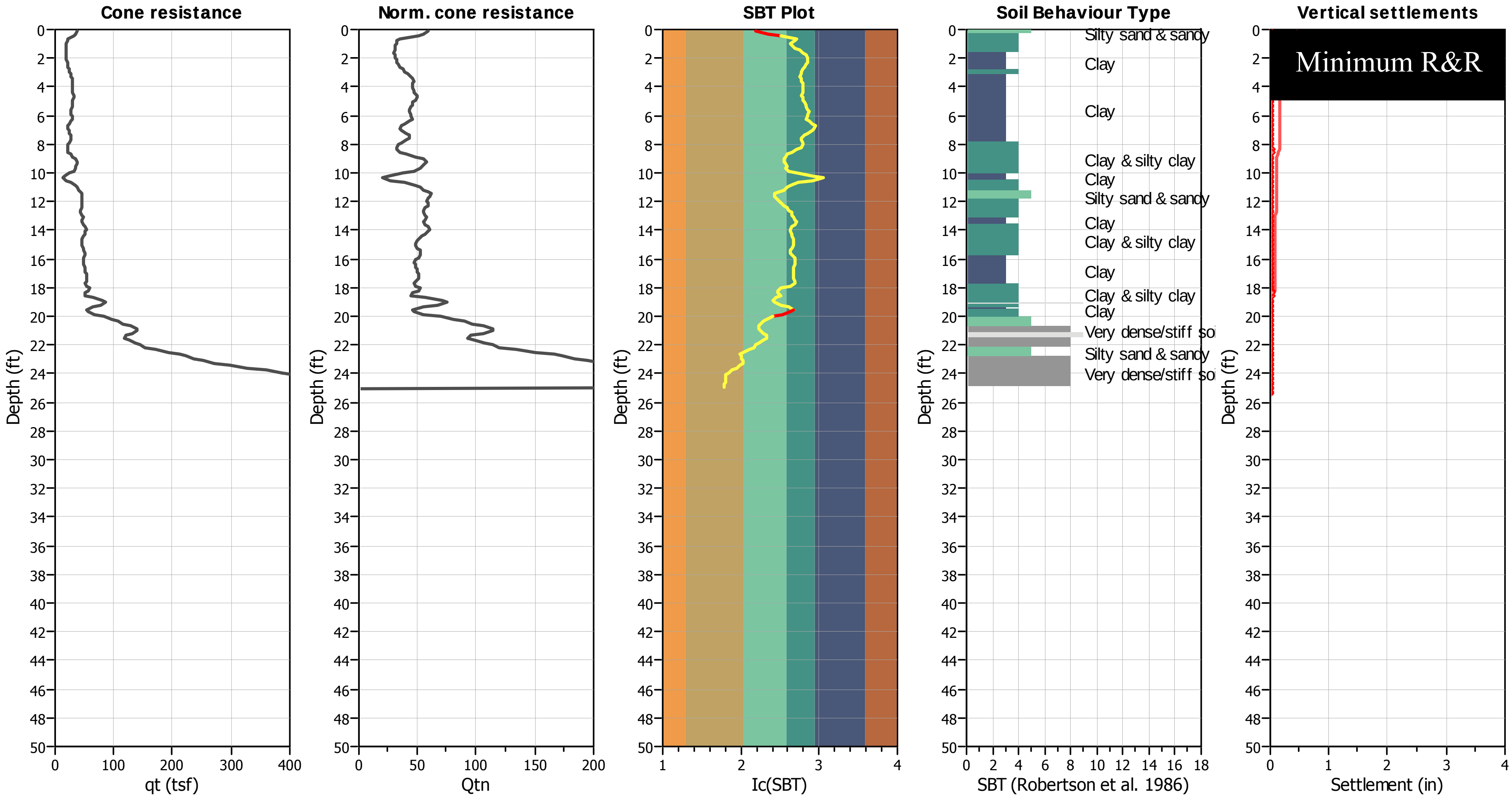
Project: Lennar - Glenelder (18012-01)
Location: Hacienda Heights

CPT: CPT-05
Total depth: 25.43 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	41.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.79	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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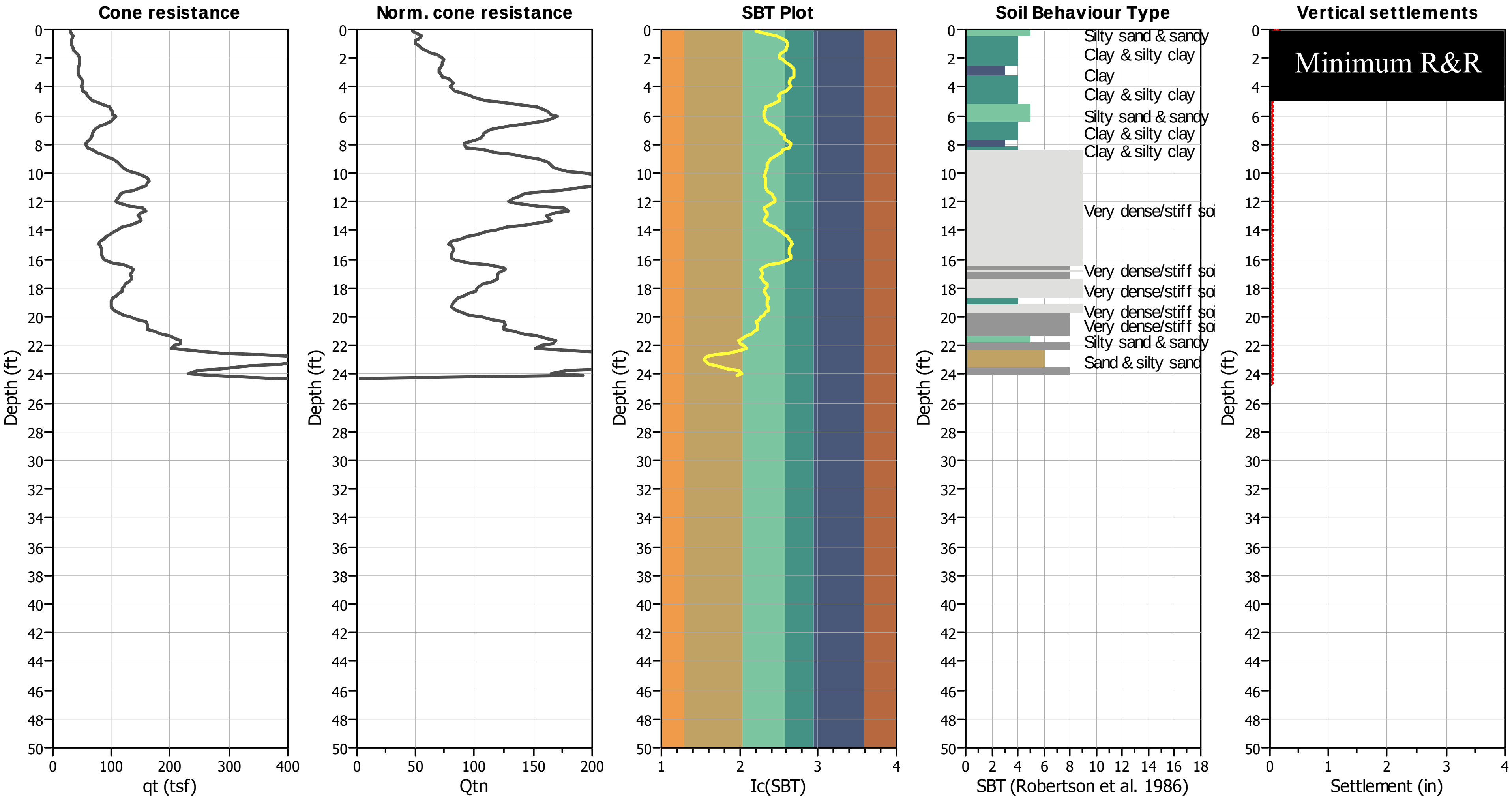
Project: **Lennar - Glenelder (18012-01)**
Location: **Hacienda Heights**

CPT: CPT-06
Total depth: 24.61 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	41.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.79	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Appendix F
General Earthwork & Grading Specifications for Rough Grading

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

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

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork

contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the
Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 Import

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

4.0 Fill Placement and Compaction

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

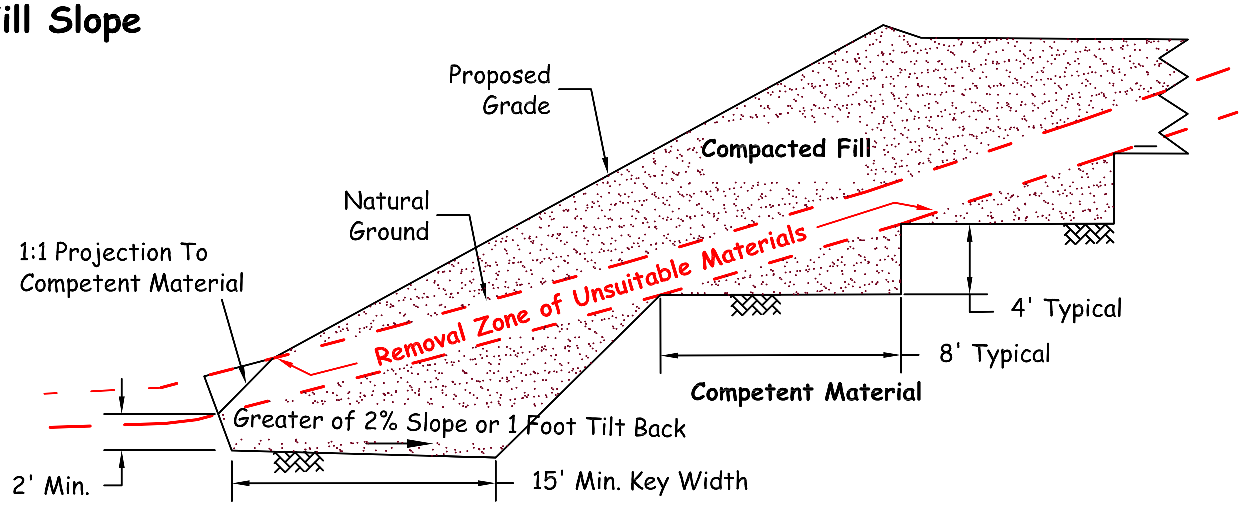
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

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

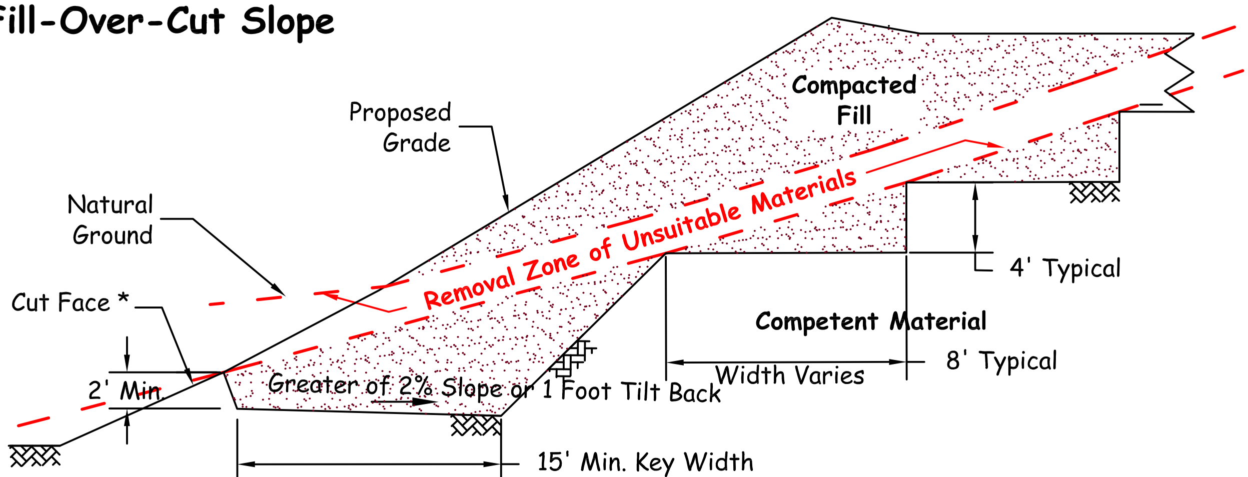
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

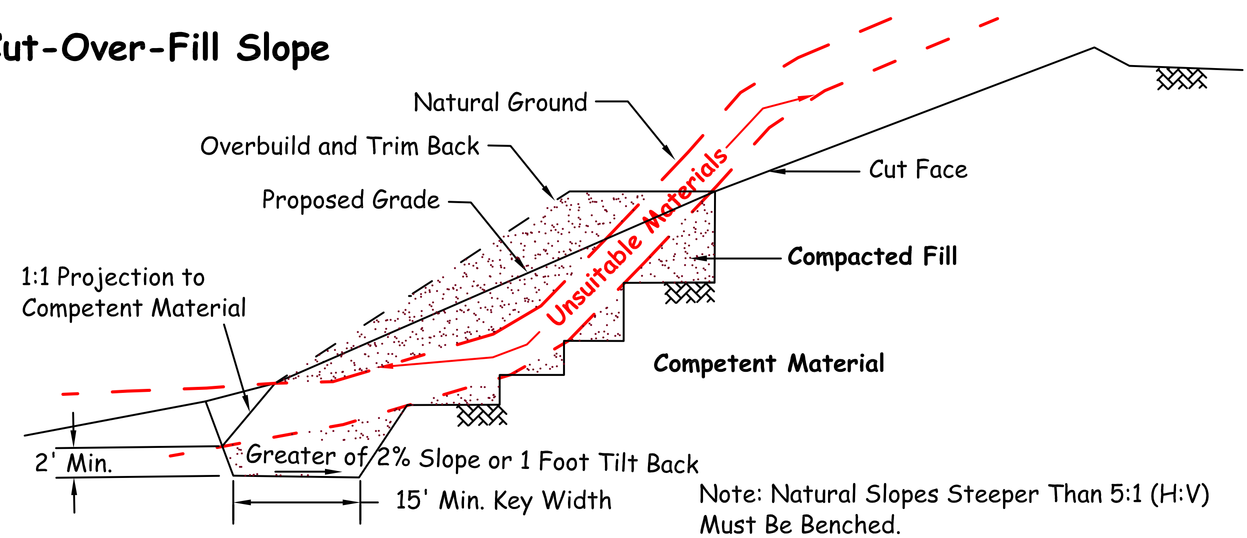
Fill Slope



Fill-Over-Cut Slope



Cut-Over-Fill Slope



Note: Natural Slopes Steeper Than 5:1 (H:V) Must Be Benched.

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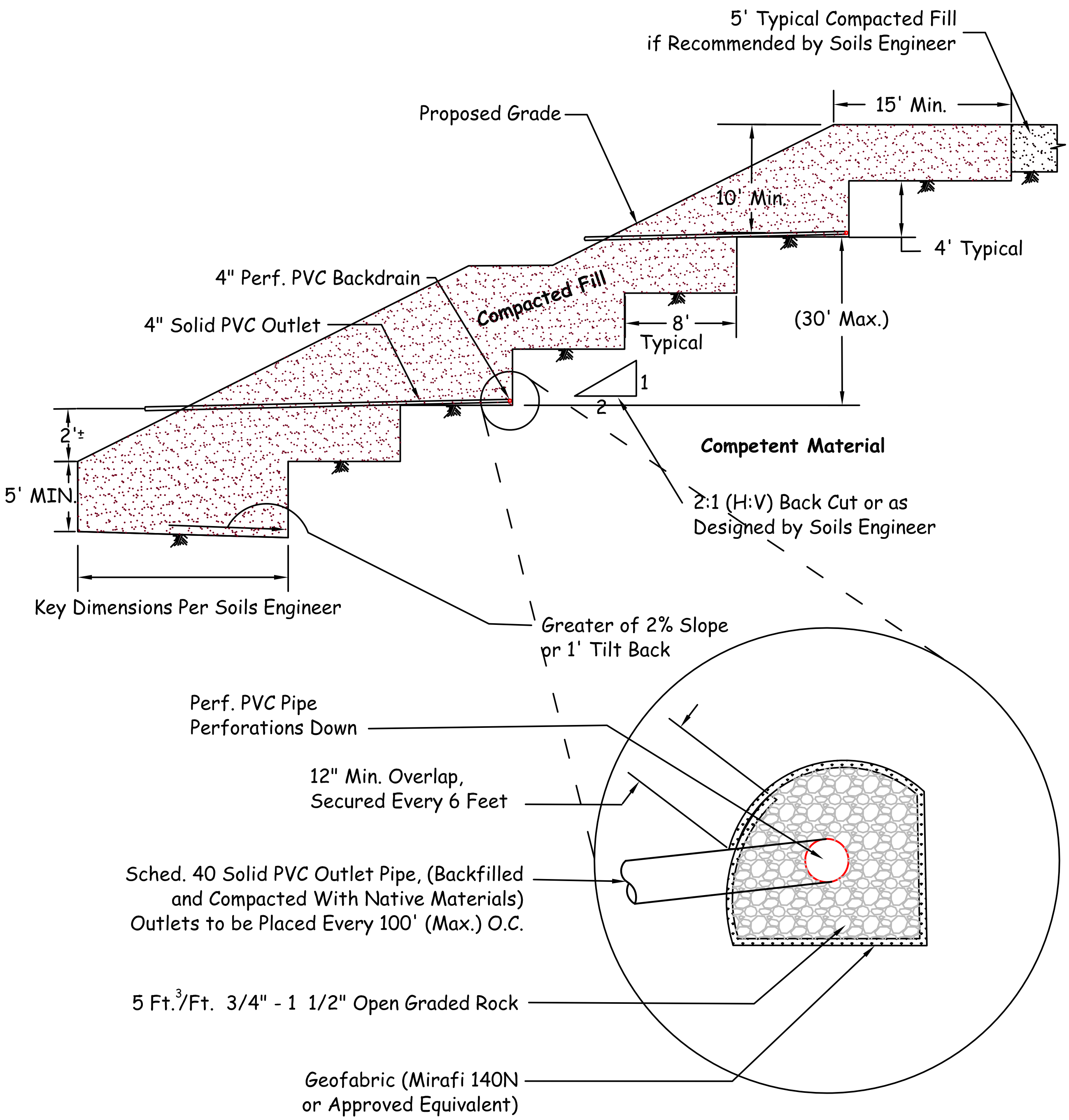
KEYING AND BENCHING



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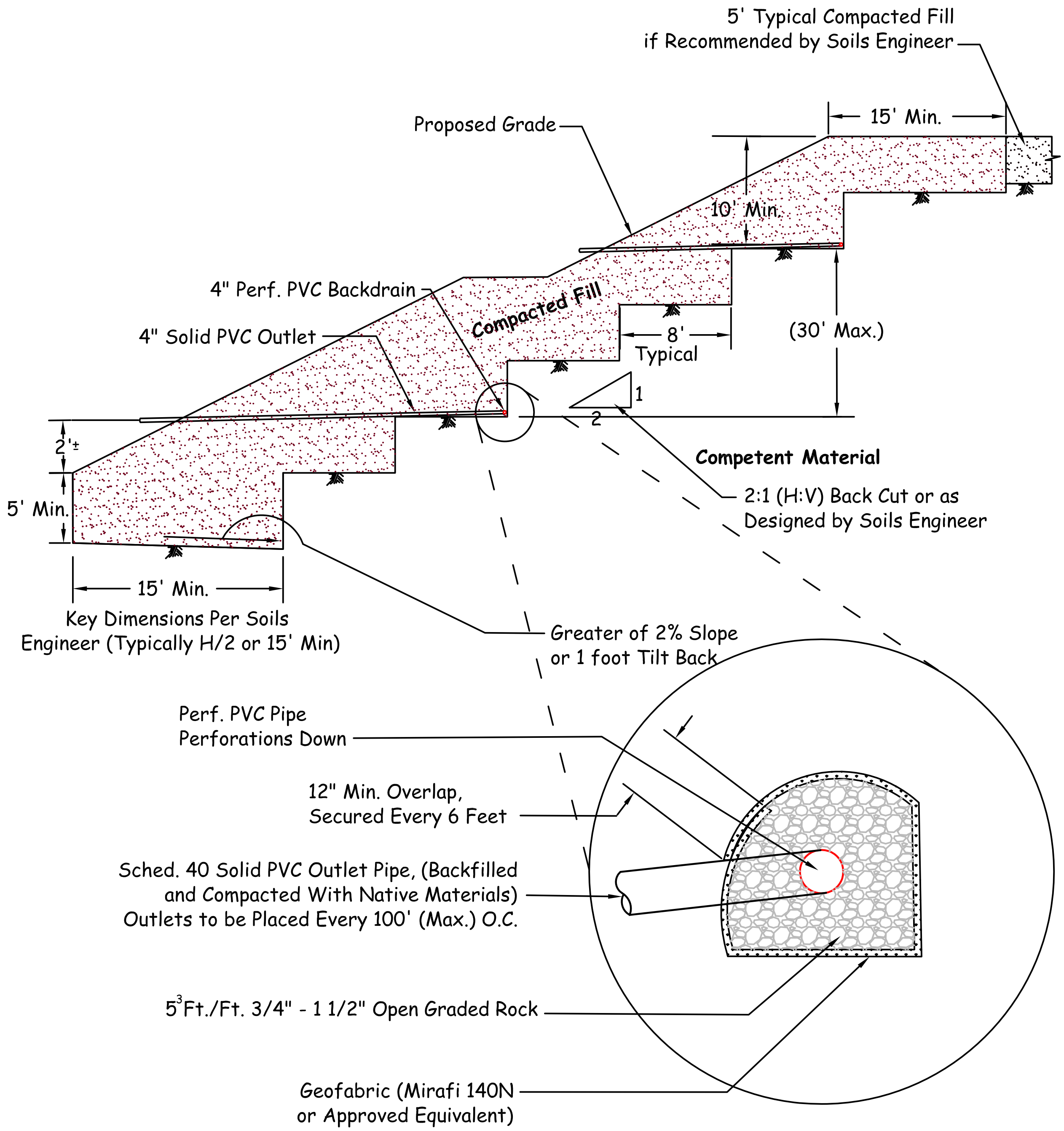
TYPICAL BUTTRESS DETAIL



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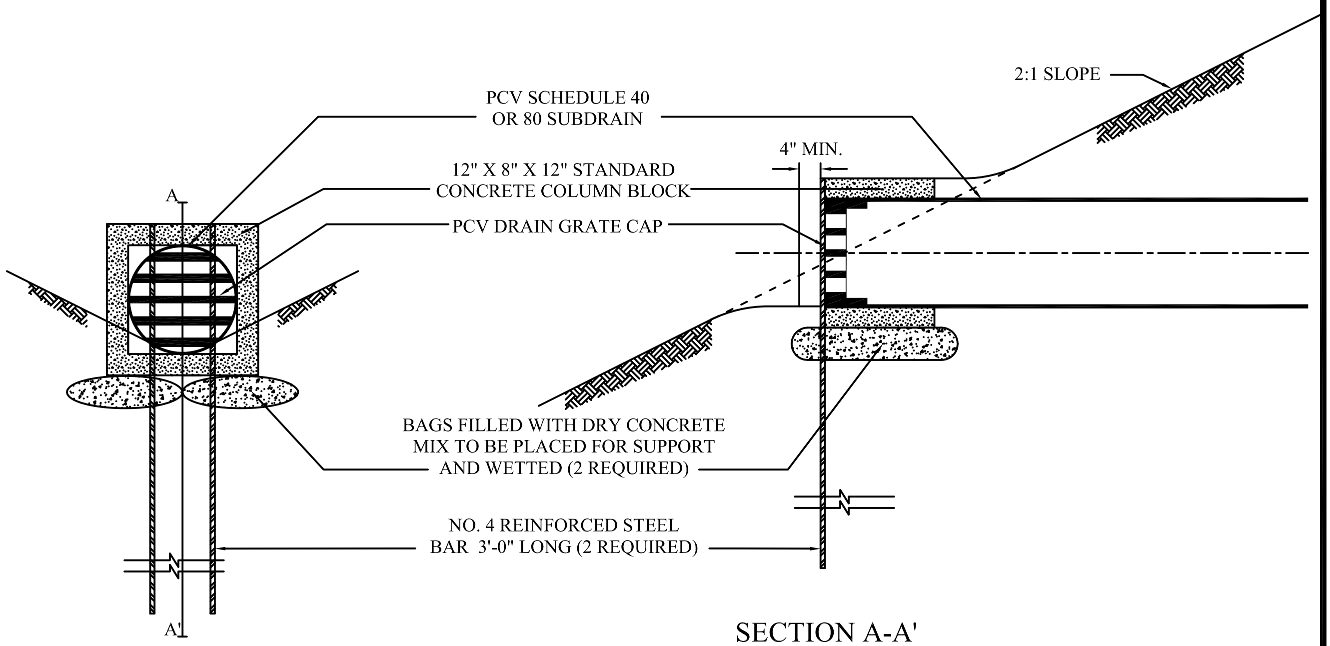
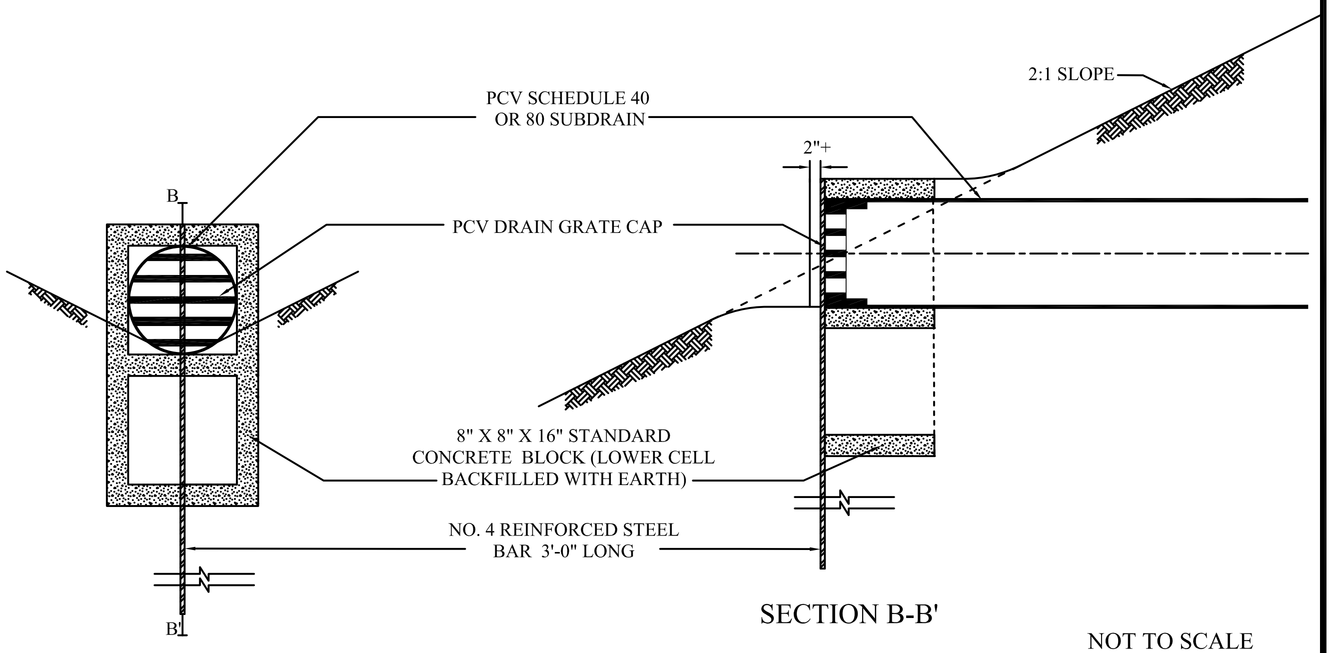
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SECTION A-A'

SUBDRAIN OUTLET MARKER -4" PIPE



SECTION B-B'

NOT TO SCALE

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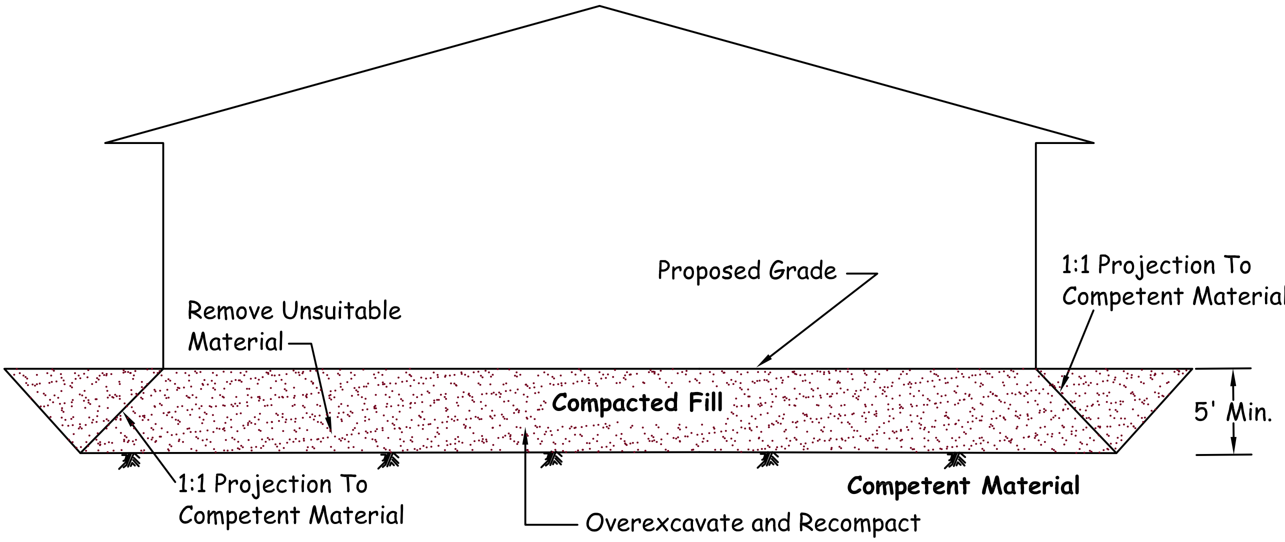
SUBDRAIN OUTLET MARKER DETAIL

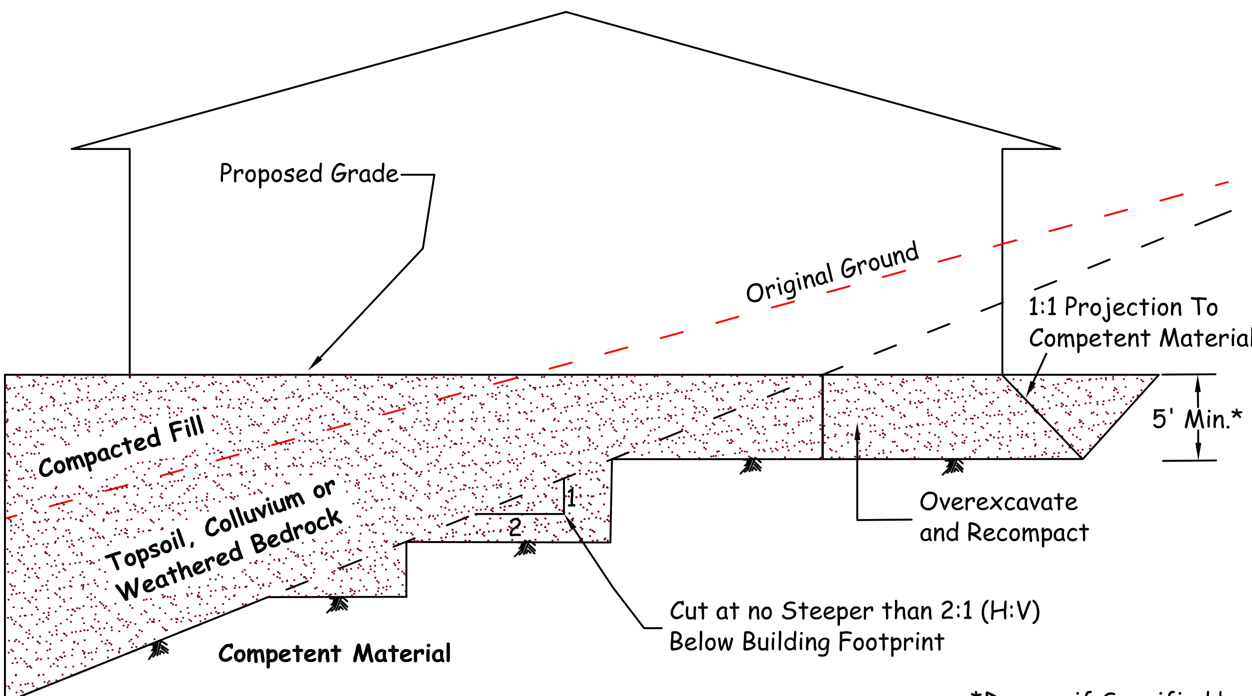
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

Cut/Fill Transition Lot



*Deeper if Specified by Soils Engineer

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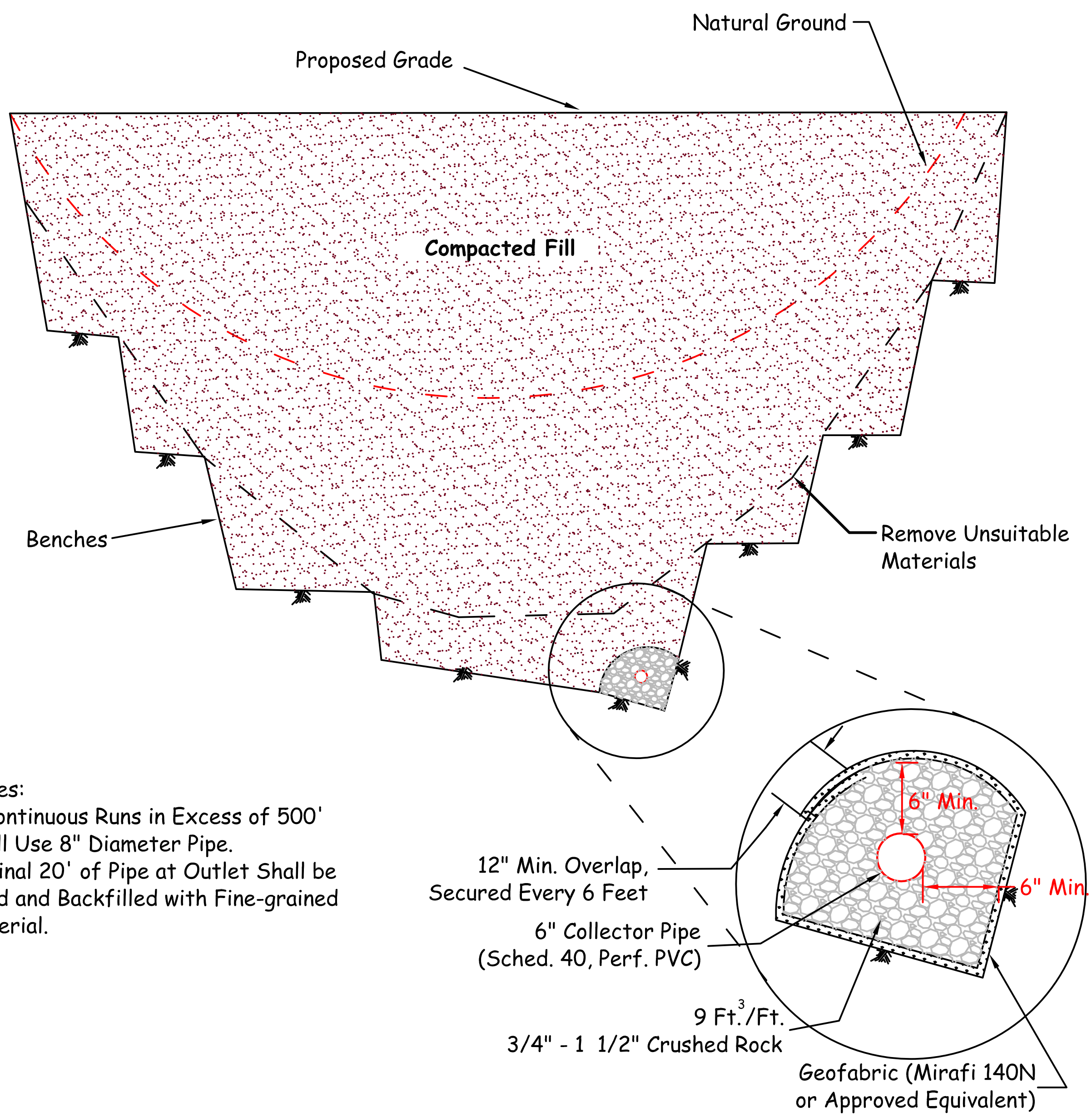
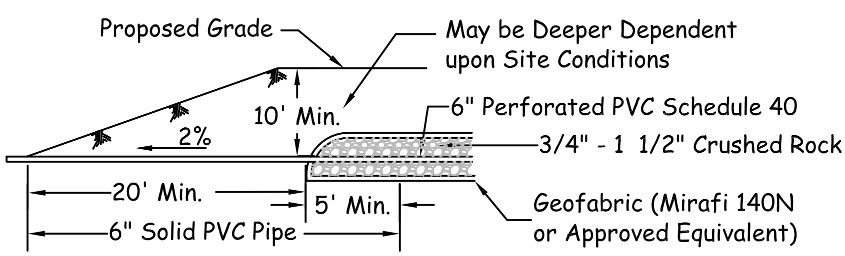
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

Proposed Outlet Detail



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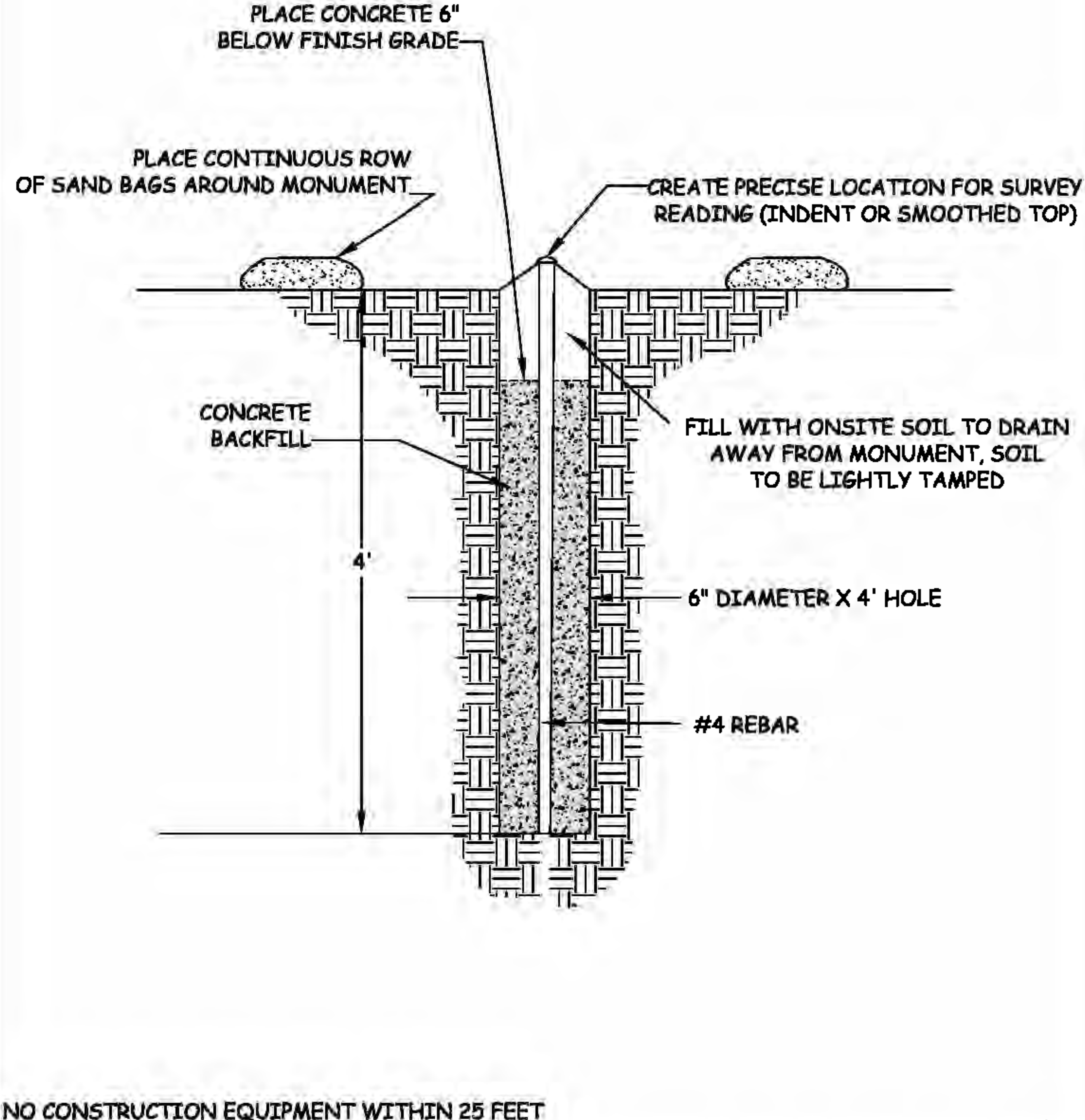
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

Revised 11/15

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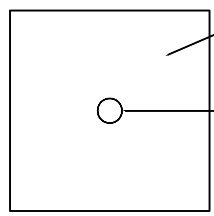
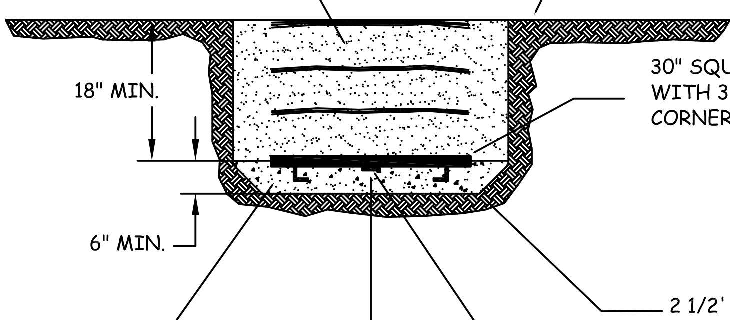
TYPICAL SURFACE SETTLEMENT MONUMENT

TOP VIEW





1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITELY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.

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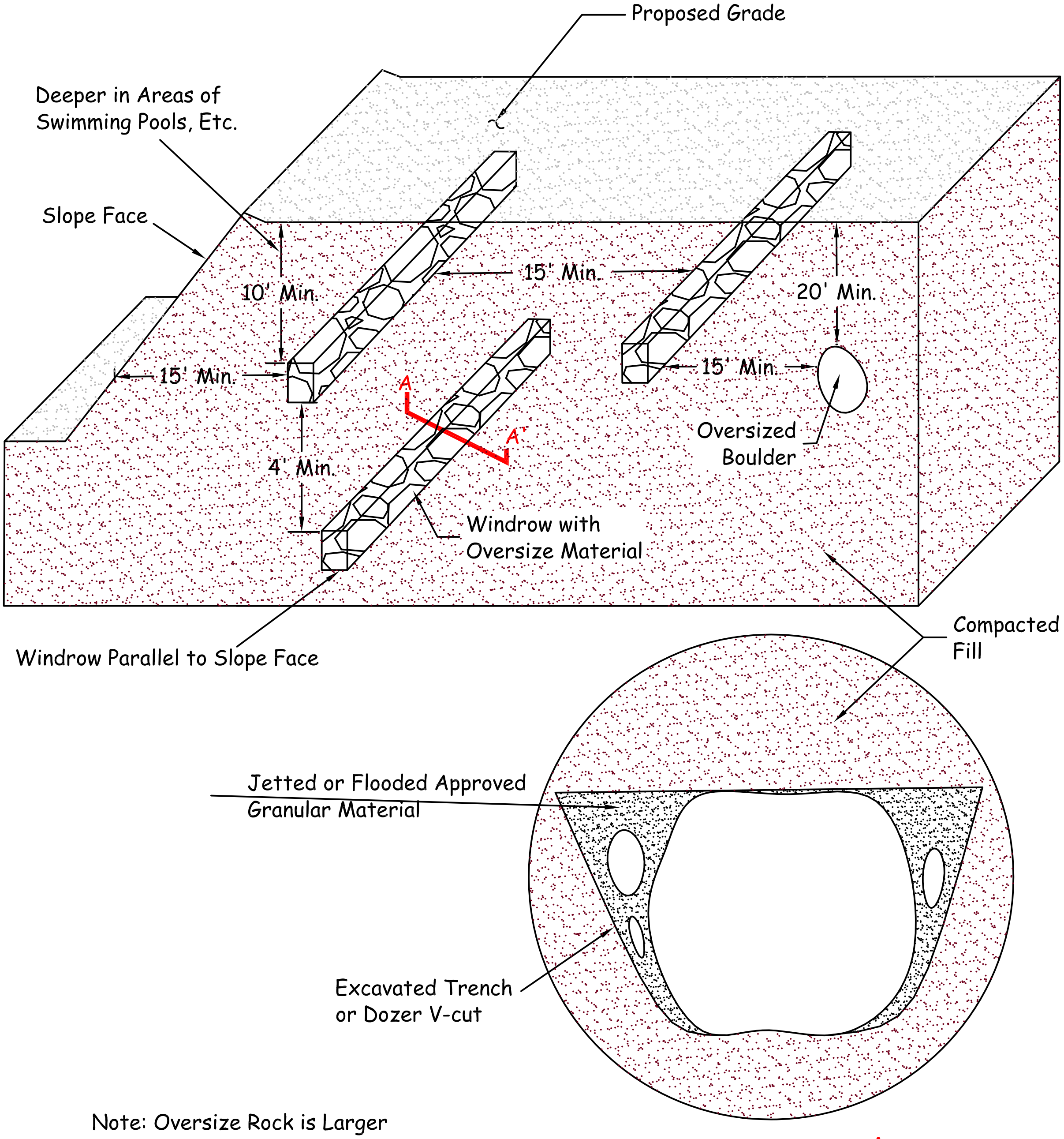
TYPICAL SETTLEMENT PLATE AND RISER



Note: Oversize Rock is Larger than 8" in Maximum Dimension.

Section A-A'

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OVERSIZE ROCK DISPOSAL DETAIL