LGC
Geotechnical, Inc.

September 12, 2023 Project No. 23155-01

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Subject: ***Supplemental Preliminary Geotechnical Evaluation for Proposed Self Storage Development, 68th Street and I-15, APNs 152-020-010, 152-060-007 and 152-060-009, Jurupa Valley, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a supplemental preliminary geotechnical evaluation for the proposed self-storage development to be located at 68th Street and I-15, APNs 152-020-010, 152-060-007 and 152-060-009, in Jurupa Valley, California. The purpose of our study was to review previous geotechnical reports, evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed development.

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

Respectfully,

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1.0 INTRODUCTION

LGC Geotechnical has performed a supplemental preliminary geotechnical evaluation for the proposed new self-storage building to be located south of 68th Street and east of Interstate I-15 in Jurupa Valley, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical design recommendations relative to the proposed development of the site.

1.1 Project Description and Background

The approximately 10.66-acre, irregular-shaped site is bound in the northerly direction by an existing residential development, in the westerly direction by the Interstate 15 Freeway and easterly and southern direction by undeveloped land. The site is relatively flat with elevations ranging from approximately 596 to 641 feet above mean seal level. The site is currently undeveloped.

LGC Geo-Environmental performed a geotechnical investigation of the site in 2020 (LGC Geo-Environmental, 2020a & b). The field evaluation consisted of the excavation of fifteen hollow-stem auger borings (eight were for infiltration testing) ranging in depth from approximately 5 to 51 feet below existing grade. Groundwater was encountered at a depth of approximately 13 feet below existing grade. Laboratory testing included in-situ dry density and moisture content, maximum dry density, expansion index, corrosion, direct shear, R-value, grain size distribution, and consolidation. Field exploration and laboratory data are summarized in the subsequent sections, and included in Appendix B and Appendix C, respectively.

Based on the preliminary plans, the proposed development will include construction of one at-grade storage building in the westerly portion of the site, RV storage in the middle, and bio-filtration basin in the eastern portion of the site. A new access road is also proposed to be constructed extending north to 68th Street (W.H. Engineering, 2023). One approximately 4-foot retaining wall is proposed in the southern portion of the site. Proposed cuts and fills are anticipated to be on the order of up to approximately 10 feet each. Proposed graded slopes will be at 4:1 (horizontal to vertical) inclinations or flatter. Proposed cut and fill slopes will be less than 10- and 5-feet-high, respectively.

Preliminary building (dead plus live) loads were not provided at the time of this report. However, we have estimated the maximum column and wall structural (dead plus live) loads to be 100 kips and 6 kips per lineal foot, respectively.

With this report, LGC Geotechnical is taking over the responsibility of geotechnical consultant of record for the project. This report and the recommendations and parameters provided herein shall supersede the referenced previous (Appendix A). Responses to the City of Jurupa Valley geotechnical review comments (City, 2023) are provided in Section 5.0 of this report.

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

1.2 *Subsurface Exploration*

Our subsurface evaluation consisted of the pushing five Cone Penetration Test (CPT) soundings. The Cone Penetration Test soundings (CPT-1 though CPT-5) were performed by Kehoe Testing and Engineering under subcontract with LGC Geotechnical. Three Pore Pressure Dissipation (PPD) tests were performed in CPT-1, CPT-3 and CPT-5. The CPT soundings were pushed to depths ranging from approximately 30 to 50 feet below existing grade in general accordance with the current ASTM standards (ASTM D5778 and ASTM D3441). The CPT equipment consists of a cone penetrometer assembly mounted at the end of 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-inch per second while the cone tip resistance and sleeve friction resistance are recorded at approximately every 2 inches and stored in digital form. A specially designed all-wheel drive 25-ton truck provides the required reaction weight for pushing the cone assembly. The CPT soundings were backfilled with grout as the probe was retracted.

The approximate locations of our subsurface explorations are provided on Sheet 1. The CPT logs and PPD plots are provided in Appendix B.

1.3 *Laboratory Testing by Others*

Representative bulk and driven samples were obtained by others for laboratory testing during previous field evaluations. Laboratory testing included in-situ moisture content and dry density, maximum dry density, expansion index, fines content, consolidation, direct shear, R-Value and corrosion (sulfate, chloride, pH and minimum resistivity). A summary of the laboratory test results is presented in Appendix C.

- Dry density of the samples collected ranged from approximately 86 pounds per cubic foot (pcf) to 135 pcf, with an average of 104 pcf. Field moisture contents ranged from approximately 2 percent to 23 percent, with an average of 9 percent.
- Two fines content tests were performed and indicated a fines content (passing No. 200 sieve) of approximately 4 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as “coarse-grained.”
- Consolidation tests were performed. Negligible collapse/swell occurred at water inundation. The deformation versus vertical stress plot is provided in Appendix C.
- One Expansion Index (EI) test indicated an EI value of 21, corresponding to “Low” expansion potential.
- A laboratory compaction curve resulted in maximum dry density value of 110.5 pcf with optimum moisture content of 15.0 percent.
- Direct shear testing resulted in an internal friction angle of 29 and cohesion of 260 psf.
- A R-Value test was performed, results indicated a value of 38.
- Corrosion testing indicated soluble sulfate contents of approximately 0.032 percent, a chloride content of 670 parts per million (ppm), pH of 7.9, and a minimum resistivity of 640 ohm-centimeters.

2.0 GEOTECHNICAL CONDITIONS

2.1 Geologic Conditions

The subject site is located south of the San Gabriel Mountains within the broad alluvial plain of the Santa Ana River Basin, within the Peninsular Ranges Geomorphic Province. Specifically, the site is located within the northern portion of the Perris Block, a geologic zone consisting of granitics overlain by sedimentary deposits that are bounded by active faults including the northwest-trending Whittier-Elsinore Fault Zone at the southwest and the northwest-trending San Jacinto Fault Zone at the northeast. The roughly rectangular Perris Block is transected by the southwest-trending Santa Ana River. The site is located on the north bank of the Santa Ana River.

2.2 Generalized Subsurface Conditions

Based on our review of regional geologic maps for the area of the site (CGS, 2002 & 2004), the project area is mapped as being underlain by Quaternary young wash deposits, which generally include; gravelly sand and sandy alluvium.

As indicated in the field explorations, soils generally consist of predominantly loose to very dense sands with varying amounts of gravel and occasional layers of stiff to very stiff silt with minor amounts clay with varying amounts of sand to the maximum explored depth of approximately 50 feet below existing grade. Soils in the upper 10 feet were predominantly found to be below optimum moisture. Descriptions of the subsurface conditions are presented on the boring and CPT logs located in Appendix B.

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

2.3 Geologic Structure

Geologic structure was not identified in the subject site geotechnical evaluation. The alluvial materials encountered are likely bedded with a subtle westerly inclination.

2.4 Landslides

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

2.5 ***Groundwater***

Groundwater was encountered during our subsurface evaluation at a depth of approximately 8 to 11 feet below existing ground surface based on our recent groundwater measurements. Historic high groundwater is estimated to be approximately 8 feet below existing grade (approximate elevation of 589 feet above MSL) near CPT-1 which approximately coincides with the elevations adjacent to the active Santa Ana River Channel. This high groundwater elevation was used for the entire site. Note that groundwater was found at a depth of approximately 13 to 17 feet below existing grade during the previous evaluations.

Groundwater and/or groundwater seepage conditions may occur in the future due to changes in land use and/or following periods of heavy rain. Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local landscape irrigation or precipitation especially during rainy seasons.

2.6 ***Faulting***

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

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

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site. The closest known active faults are the Elsinore, Whittier, Fontana, San Jacinto, San Andreas, and Chino Faults.

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

2.6.1 __Lurching and Shallow Ground Rupture__

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

2.6.2 __Liquefaction and Dynamic Settlement__

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

Based on our review of the Riverside County Liquefaction Hazard Map (Riverside, 2019), the subject site is located in an area of “very high” liquefaction susceptibility. The data obtained from our field evaluation indicates that the site contains sandy layers susceptible to liquefaction in the upper 50 feet. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008). Liquefaction analysis was based on the applicable seismic criteria (e.g., PGA_M from 2022 CBC) and the estimated historic high groundwater depth of 8 feet below existing grade. It is our opinion that the soil type interpretations of CPT soundings generally show good agreement to adjacent borings and laboratory testing. Estimated total and differential seismic settlement due to liquefaction potential is provided in Table 1. Liquefaction calculations are provided in Appendix D.

TABLE 1

Estimated Settlement Due to Liquefaction Potential

Estimated Total Seismic Settlement	Estimated Differential Seismic Settlement
2 inches	1 inch over 40 feet

2.6.3 Lateral Spreading

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

The sandy soil anticipated to be left in place (below the recommended temporary removal and recompaction depths presented on the Geotechnical Map and in Section 4.1 of this report) generally have a normalized clean sand tip resistance well above 70. A normalized clean sand tip resistance of 70 corresponds to a blow count $(N_1)_{60}$ of at least 15. Soils with a corrected SPT $(N_1)_{60}$ blow count of 15 or greater are generally not considered susceptible to lateral spreading (Youd, Hansen, Bartlett, 2002). Furthermore, isolated sandy layers susceptible to liquefaction were generally found not to be laterally continuous.

Due to the depth of proposed earthwork removals, presence of medium dense sandy soils below the recommended earthwork removals, and limited lateral nature of potentially liquefiable soils, the potential for lateral spreading is considered low.

2.7 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the 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 7/2022 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.** Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 33.9582 degrees north and longitude -117.5477 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. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

TABLE 2

Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**	**Notes/Exceptions**
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.618g	From SEAOC, 2023
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.581g	From SEAOC, 2023
F_a (per Table 1613.2.3(1))	1.0	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.719	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.618g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	0.999g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^{2}/_{3})S_{MS}$]	1.078g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^{2}/_{3})S_{M1}$]	0.666g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.942	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.919	ASCE 7 Chapter 22
*Since site soils are Site Class D and S1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of T ≤ 1.5Ts and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for TL ≥ T > Ts, or Eq. 12.8-4 for T > TL. Refer to ASCE 7-16. Site Class F modified to Site Class D, seismic parameters only applicable for structure period ≤ 0.5 second, refer to discussion above.		

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.74 at a distance of approximately 15.05 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.70 at a distance of approximately 17.52 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 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 0.730g (SEAOC, 2023). The design PGA is equal to 0.487g (2/3 of PGA_M).

2.8 <u>Rippability</u>

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

2.9 <u>Oversized Material</u>

Generation of a surplus of oversized material (material greater than 8 inches in maximum dimension) is generally not anticipated during site grading. However, some oversized material may be encountered, which may result in excavation difficulty for narrow excavations. Recommendations are provided for appropriate handling of oversized materials in Appendix F. If feasible, crushing oversized materials or exporting to an offsite location may be considered.

3.0 FINDINGS AND CONCLUSIONS

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

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

- In general, previous borings and our CPT soundings indicate that the site soils generally consist of predominantly loose to very dense sand with varying amounts of gravel with occasional layers of stiff to very stiff silt with minor amounts clay with varying amounts of sand to the maximum explored depth of approximately 50 feet below existing grade. Soils in the upper 10 feet were predominantly below optimum moisture. The near surface loose, soft and compressible soils 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 8 to 11 feet below existing ground surface based on our recent groundwater measurements. Historic high groundwater is estimated to be approximately 8 feet below existing grade (approximate elevation of 589 feet above MSL) near CPT-1. This high groundwater elevation was used for the entire site.
- The proposed development will likely be subjected to strong seismic ground shaking during its design life from one of the regional faults. The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation.
- The site is located in a seismic hazard zone for liquefaction potential. Subsurface data indicates that isolated sandy layers are susceptible to liquefaction and liquefaction-induced settlement. Our analysis indicates approximately 2 inches of seismically-induced settlement may occur at the site during a significant earthquake. Differential seismic settlement may be taken as 1 inches over a horizontal span of 40 feet.
- It is our opinion that the possible impacts of liquefaction can by reasonably mitigated by the use of a rigid mat slab or conventional foundation designed to accommodate the estimated seismic and static settlements. However, as with many structures in Southern California risk does remain that the proposed structures could suffer some damage if liquefaction occurs. Repair and remedial work may be required after a liquefaction event.
- Due to shallow groundwater, stabilization of removal bottom subgrade may be necessary during remedial grading.
- The potential need for protection of the site from channel migration and/or erosion related to the adjacent Santa Ana River should be addressed by the project Civil Engineer.
- Soils exposed at the proposed foundation level are anticipated to have a “Low” expansion potential (EI less than 50). This shall be confirmed at the completion of site earthwork.
- Excavation for foundations and underground improvements should be achievable with the appropriate equipment.
- From a geotechnical perspective, the existing onsite soils are considered suitable material for use as general fill (with the exception of retaining wall backfill), provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris and significant organic material. Moisture conditioning will be required to obtain the required compaction.

- Site contains clayey soils with high fines content and expansion potential that are not suitable for backfill of site retaining walls. Therefore, select grading and stockpiling and/or import of sandy soils meeting project recommendations will be required.
- Due to site liquefaction potential and shallow depth to groundwater, intentional infiltration of storm water is not considered feasible from a geotechnical standpoint and therefore should not be performed.
- From a geotechnical point of view, provided the geotechnical recommendations and parameters of the project geotechnical report are appropriately incorporated into the design and construction of the project, the proposed site grading and construction is not anticipated to impact the adjacent properties and improvements.
- Due to the relatively shallow site groundwater (about 8 feet below existing ground surface), dewatering or stabilization of subgrade for removal bottoms or deep utility trenches may be locally required, prior to subsequent fill placement.

4.0 RECOMMENDATIONS

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

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 California Building Code (CBC) requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "the level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

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

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of required earthwork removals, foundation construction and utility line construction and backfill. We recommend that earthwork onsite be performed in accordance with the following recommendations, 2022 CBC/ City of Jurupa Valley and the General Earthwork and Grading Specifications included in Appendix F. In case of conflict, the following recommendations shall supersede previous recommendations and those included as part of Appendix F.

4.1.1 Site Preparation

Prior to grading, areas to be developed should undergo the stripping and clearing of vegetation and clearing of surface obstructions from the site. Vegetation, debris and excessive organic material should be removed and properly disposed of offsite. Holes resulting from removals of buried obstructions, which extend below proposed remedial and/or finish grades, should be replaced with suitable compacted fill material.

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, where deemed appropriate by the project geotechnical consultant, 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 building structures, the upper loose/compressible native soils are to be removed and replaced as properly compacted fills. For preliminary planning purposes, the depth of recommended earthwork removals may be estimated as indicated below.

Building Structures: In order to provide a relatively uniform bearing condition for the planned structural improvements, we recommend that removals extend to the minimum depth below existing grade presented on the Sheet 1 Geotechnical Map (5 to 8 feet) or 3 feet below the base of the foundations, whichever is deeper. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

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

Pavement and Hardscape Areas: Removals should extend to a depth of at least 3 feet below the existing grade. Removals in any design cut areas of the pavement may be reduced by the depth of the design cut but should not be less than 1-foot below the finished subgrade (i.e., below planned aggregate base/asphalt concrete). In general, the envelope for removals should extend laterally a minimum lateral distance of 3 feet beyond the edges of the proposed improvements where space is available.

Based on our findings, the recommended removal and recompaction depths may extend to a depth just above the anticipated groundwater table in portions of the site. We recommend that the removal bottom depth does not extend to within 3 feet of the anticipated groundwater. Shallower earthwork removals than what is recommended above may occur as a result but must be confirmed and approved by LGC Geotechnical during grading. Care should be taken in order to avoid creating an unstable removal bottom during grading. Recommendations for subgrade stabilization are included in Section 4.1.4.

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

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field evaluation, site soils upper approximate 8 feet are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person", required by OSHA standards, to evaluate soil conditions. Sandy soils are present and should be considered susceptible to caving. The contractor shall be responsible for providing the "competent person", required by OSHA standards, to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation, or 5 feet whichever is greater. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

4.1.4 Removal Bottoms and Subgrade Preparation

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

Based on the presence of shallow groundwater and the potential to encounter saturated alluvial materials at or near the estimated removal depths and deep utility trenches, some of the removal bottoms are anticipated to be wet and unstable. We recommend all wet/unstable removal bottoms be stabilized by the placement and "working in" of 2 to 4-inch nominal diameter crushed aggregate or an approved alternate stabilization method. Based on our experience with similar projects, we anticipate the thickness of crushed rock (stabilization aggregate) needed to stabilize the removal bottoms will be on the order of 12 to 24 inches thick. The actual thickness of aggregate required to stabilize the excavation bottom shall be determined in the field based on the actual conditions and equipment used. It should be anticipated that the first lift of crushed aggregate will be worked into the pumping subgrade. Subsequent lifts should be properly compacted and will help bridge the pumping conditions. Thickness of required aggregate stabilization may be reduced by placing a layer of biaxial geogrid reinforcement (Tensar InterAX or acceptable equivalent) directly on the subgrade prior to aggregate base placement.

Contractor may have to minimize construction traffic on the removal bottom to reduce disturbance. Soft and yielding subgrade should be evaluated on a case-by-case basis during earthwork operations.

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

4.1.5 <u>Material for Fill</u>

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

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

Any required retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris and any material greater than 3 inches in maximum dimension. In general, the site soils in the upper 8 feet may not be suitable for retaining wall backfill due to their fines content and expansion potential, therefore select grading and stockpiling and/or import will be required by the contractor 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 Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

4.1.6 <u>Fill Placement and Compaction</u>

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

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

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

If gap-graded rock (e.g., ¾-inch crushed rock, etc.) 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

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

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

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

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 10 to 15 percent reduction in volume of near-surface (upper approximate 5 to 8 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence, due to earthwork operations, is expected to be on the order of 0.1-foot. These values are estimates only and exclude losses due to removal of any vegetation or debris. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor and accuracy of the topographic survey.

4.2 Preliminary Foundation Recommendations

Preliminary foundation recommendations for both mat and conventional foundations are provided in the following sections. Proposed building foundations should be designed in consideration of site liquefaction potential and seismic and static settlement as outlined below. Due to liquefaction potential (Site Class “F”) and seismic settlement isolated pad footings should be interconnected with grade beams.

Site soils are anticipated to have “Low” expansion potential (EI of 50 or less per ASTM D4829). Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading. Allowable soil bearing is provided in Section 4.3.

4.2.1 Preliminary Mat Foundation Design Parameters

A stiffened mat foundation may be used for support of the proposed building structures to reduce the effect of differential seismic settlement due to liquefaction potential. The magnitude of total and differential settlements of the mat foundation will be a function of the structural design and stiffness of the mat. Total and differential seismic settlement due to liquefaction potential is provided in Section 2.6.2. Estimated static settlement is provided in Section 4.3. Earthwork removals are required for support of the mat foundation as outlined in Section 4.1 and related subsections.

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

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

Moisture condition slab subgrade soils to optimum moisture content to a minimum depth of 12 inches prior to trenching. The moisture content of the slab subgrade should be verified by the geotechnical consultant within 1 to 2 days prior to concrete placement. In addition, this moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures.

4.2.2 __Preliminary Conventional Foundation Design Parameters__

Due to liquefaction potential (Site Class "F") and dynamic settlement any isolated structural footings should be interconnected with grade beams. The proposed building foundation should be designed in consideration of site liquefaction potential and seismic settlement outlined in Section 2.6.2 as well as the estimated static settlement presented in Section 4.3. The foundation/structural engineer may design a conventional foundation system that is tied together based upon the anticipated dead and live loads (wind, seismic) that will be imparted by the structure. The recommendations provided in the "Soil Bearing and Lateral Resistance" section may be utilized in the design of a conventional slab-on-grade foundation designed in accordance with Section 1808 of the 2022 C.B.C.

Moisture condition slab subgrade soils to optimum moisture content to a minimum depth of 12 inches prior to trenching. The moisture content of the slab subgrade should be verified by the geotechnical consultant within 1 to 2 days prior to concrete placement. In addition, this moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures.

4.2.3 __Shallow Foundation Maintenance__

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

In addition to the factors mentioned above, roots that extend near the vicinity of foundations can cause distress to foundations. Trees/large shrubs should not be planted closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the building foundation.

4.2.4 Slab Underlayment Guidelines

The following recommendations are for informational purposes only, as they are unrelated to the geotechnical performance of the foundation. The following recommendations may be superseded by the foundation engineer and/or owner. Some post-construction moisture migration should be expected below the foundation. In general, interior floor slabs with moisture sensitive floor coverings should be underlain by a minimum 10 mil thick polyolefin material vapor retarder, which has a water vapor transmission rate (permeance) of less than 0.03 perms. The need for sand and/or the sand thickness (above and/or below the vapor retarder) should be specified by the structural engineer, architect or concrete contactor. The selection and thickness of sand is not a geotechnical engineering issue and is therefore outside our purview.

4.3 Soil Bearing and Lateral Resistance

For the proposed industrial warehouse structures, minimum continuous wall and column footing widths are to be 12 inches and 24 inches, respectively, minimum foundation embedment is to extend a minimum of 18 inches below the adjacent exterior grade, and interior column footings should be embedded a minimum of 12 inches beneath the adjacent subgrade. The following allowable bearing pressures for both continuous and column spread footings presented in Table 3 below are recommended for corresponding footing widths and embedments.

TABLE 3

Allowable Soil Bearing Pressures

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

* Refers to minimum depth measured below lowest adjacent grade.

It should be noted that a 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 5 horizontal feet to 1-foot vertical) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads). Due to liquefaction potential (Site Class "F") and dynamic settlement any isolated pad footings should be interconnected with grade beams.

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 settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet). Total and differential seismic settlement due to liquefaction potential is provided in Section 2.6.2.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.35 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 230 psf per foot of depth (or pcf) to a maximum of 2,300 psf may be used for lateral resistance. This passive pressure is applicable for level (ground slope equal to or flatter than 5 horizontal feet to 1-foot vertical) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt concrete. The provided allowable passive pressure is based on a factor of safety of 1.5 and may be increased by one-third for short duration wind or seismic loading. This increase is based on a reduced factor of safety for short duration loading.

4.4 *Lateral Earth Pressures for Retaining Walls*

The following preliminary lateral earth pressures may be used for any site retaining walls 6 feet or less. Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design.

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

TABLE 4

Lateral Earth Pressures – Select Sandy Backfill

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. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Refer to Figure 2). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, basement/retaining walls adjacent to streets should be designed to resist vehicular traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.5 and 0.33 may be used for at-rest and active conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The structural designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If retaining wall greater than 6 feet in height are proposed, 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.

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 <u>Preliminary Asphalt Concrete Pavement Sections</u>

The following preliminary minimum asphalt concrete (AC) pavement sections are provided in Table 5 based on an R-value of 38. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Determination of the Traffic Index (TI) is not the purview of the geotechnical consultant. Final pavement sections should be confirmed by the project civil/transportation engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 5

Asphalt Concrete Pavement Section Options

Assumed Traffic Index	5.0 (or less)	6.0	7.0
R -Value Subgrade	38	38	38
AC Thickness	4.0 inches	4.0 inches	4.0 inches
Aggregate Base Thickness	4.0 inches	5.5 inches	7.5 inches

The provided preliminary Portland Cement concrete pavement section is based on the guidelines of the American Concrete Institute (ACI 330R-08). For the final design section, we recommend a traffic study be performed as LGC Geotechnical does not perform traffic engineering. Traffic study should include the design vehicle (number of axles and load per axle) and estimated number of daily repetitions/trips. Based on an assumed Traffic Category B with an assumed Average Daily Truck Traffic (ADTT) of 25, we recommend a preliminary section of a minimum of 6.0 inches of concrete over 4 inches of compacted aggregate base over compacted subgrade. The concrete should have a minimum compressive strength of 4,000 psi and a minimum flexural strength of 550 psi at the time the pavement is subjected to traffic. Steel reinforcement is not required (ACI, 2013). This pavement section assumes that edge restraints like a curb and gutter will be provided. To reduce the potential (but not eliminate) for cracking, paving should provide control joints at regular intervals not exceeding 8 feet in each direction. Decreasing the spacing of these joints will further reduce, but not eliminate the potential for unsightly cracking. Preliminary pavement section is based on a 20-year design. Truck loading is defined as one 16-kip axle and two 32-kip tandem axles (80 kips). Alternate section(s) may be provided based on anticipated specific traffic loadings and repetitions provided by others. LGC Geotechnical does not perform traffic engineering and determination of traffic loading is not the purview of the geotechnical consultant.

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

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

4.6 Soil Corrosivity

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

Corrosion testing by others indicated soluble sulfate contents of approximately 0.032 percent, a chloride content of 670 ppm, pH of 7.9, and a minimum resistivity of 640 ohm-centimeters. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 1,500 ppm (0.15 percent) or greater. Based on the test results and the Caltrans Corrosion Guidelines, soils are considered corrosive based on the chloride content. Note that based on minimum resistivity the soils are considered severely corrosive to metallic improvements. If improvements that may be susceptible to corrosion are proposed, it is recommended that further evaluation by a corrosion engineer be performed.

Based on laboratory sulfate test results, the near-surface soils have an exposure class of "S0" per

ACI 318-19, Table 19.3.1.1 with respect to sulfates. This must be verified based on as-graded conditions.

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.

4.7 Nonstructural Concrete Flatwork

Nonstructural concrete (such as flatwork, sidewalks, 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 should be designed in accordance with the minimum guidelines outlined below. 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.

Nonstructural and non-vehicular concrete flatwork placed on compacted subgrade may be a minimum 4-inches in thickness with crack control joints spaced 6 feet apart for flatwork slabs and 6 feet apart for flatwork sidewalks. Crack control joints should be sawcut or deep open tool joint to a minimum of 1/3 the concrete thickness. The compacted subgrade below the nonstructural and non-vehicular concrete flatwork should be wet down prior to placing concrete.

To reduce the potential for nonstructural concrete flatwork to separate from entryways and doorways, the owner may elect to install dowels to tie these two elements together.

4.8 Surface Drainage and Landscaping

4.8.1 Precise Grading

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

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

4.8.2 ***Landscaping***

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

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

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

4.9 ***Subsurface Water Infiltration***

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

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement.

It is our understanding that the county guidelines state that there must be a minimum of 10 feet between the bottom of the infiltration facility and the historical high groundwater mark (Riverside County, 2011). Based on the site liquefaction potential and very shallow depth to groundwater (about 8 feet below existing grade), we strongly recommend against the intentional infiltration of stormwater into the subsurface soils.

4.10 ***Geotechnical Plan Review***

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

4.11 ***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 are required per Section 1705 of the 2022 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;
- During precise grading;
- Preparation of building pads and other concrete-flatwork/pavement subgrades, and prior to placement of aggregate base, asphalt concrete, or concrete;
- After building and wall footing excavation and prior to placement of steel reinforcement and/or concrete;
- Preparation of pavement subgrade and placement of aggregate base; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 RESPONSE TO GEOTECHNICAL REVIEW COMMENTS

Geotechnical review comments were provided by the City of Jurupa Valley (City, 2023) for the previous geotechnical reports for the project (LGC Geo-Environmental, 2020a & b). For your convenience, the pertinent review comments have been repeated below along with our geotechnical responses. A copy of the review sheet is provided in Appendix E.

Geotechnical Review Comments dated May 30, 2023

Comment No. 35

"Provide an updated summary of the proposed grading and construction including maximum depths of cut and fill and maximum slope heights, and provide updates to the conclusions and recommendations as appropriate."

Response to Comment No. 35

Based on the preliminary plans, the proposed development will include construction of one at-grade storage building in the westerly portion of the site, RV storage in the middle, and a bio-filtration basin in the eastern portion of the site. A new access road is also proposed to be constructed extending north to 68th Street (W.H. Engineering, 2023). One approximately 4-foot retaining wall is proposed in the southern portion of the site. Proposed cuts and fills are anticipated to be on the order of up to approximately 10 feet each. Proposed graded slopes will be at 4:1 (horizontal to vertical) inclinations or flatter. Proposed cut and fill slopes will be less than 10- and 5-feet-high, respectively.

Our conclusions and recommendations, based on the subject evaluation, are provided herein.

Comment No. 36

"The consultant should review and update the previous liquefaction analyses performed by LGC for the subject site. The analyses results are presented in Appendix D of the LGC report. Borings B-1 and B-3 were used for the analyses. Although boring B-3 was only 21.5 feet deep, liquefaction was evaluated at this location up to a depth of 50 feet. The consultant should explain (A) how they were able to extend this boring to a depth of 50 feet for the sake of liquefaction analyses, and (B) how they chose the sampler blow-counts and fines content of the soil layers below a depth of 21.5 feet at B-3. (C) The consultant should also justify the fines content they chose for all soil layers at borings B-1 and B-3. (D) The ring sampler blow-counts considered in the calculation tables in Appendix D do not match with that shown on the boring logs. The consultant should address this discrepancy. The consultant should also explain (E) why a maximum moment magnitude of 7.0 was used in liquefaction evaluation; (F) what was meant by "design ground motion"; and (G) why was a value of 0.8 used for "design ground motion"? (H) Finally, the consultant should also explain why a design groundwater depth of 15 feet was used in liquefaction analyses, particularly for boring B-3, when groundwater was encountered at a depth of 13.4 feet in boring B-3 during drilling."

Response to Comment No. 36

As part of this report we have performed a supplemental geotechnical evaluation including pushing CPT soundings. This data has been utilized to perform a liquefaction analysis for the site. The analysis is provided in Appendix D of this report and a discussion provided in Section 2.6.2 of this report. Upgraded groundwater findings are presented in Section 2.5.

Comment No. 37

"The consultant references a Preliminary Infiltration Testing Investigation report by LGC dated October 15, 2020 in their "Update" letter. The preliminary infiltration report by LGC was not submitted, but LGC's preliminary geotechnical investigation was submitted. The preliminary infiltration report by LGC should be submitted to the City for review."

Response to Comment No. 37

The requested infiltration boring logs from the previous report are included in Appendix B of this report. Note that infiltration is no longer recommended or proposed for the site. Refer to Section 4.9 for further discussion.

Comment No. 38

"The consultant should present the boring and infiltration test locations on a plan that uses the current grading plan as a base map."

Response to Comment No. 38

The location of our recently performed CPT soundings and the boring and infiltration tests performed by others are included on the Preliminary Geotechnical Map (Sheet 1) presented herein.

Comment No. 39

"The consultant refers to and summarized previous infiltration tests performed within the site and states that three of the eight previous tests were located in the vicinity of the proposed infiltration basin. The tests were performed at a depth of approximately 5 feet. Based on the rough grading plan, excavation for the proposed infiltration basin will be approximately 20 feet below the existing ground surface. Based on the proposed grading, the soils that were tested for infiltration characteristics will be removed during excavation of the basin. The consultant should perform a sufficient number of percolation tests at the planned bottom elevation of the proposed basin, including evaluating the soils below the bottom of the basin through which the water will infiltrate, in accordance with Riverside County guidelines. However, the consultant states that groundwater was previously encountered at depths between 13.4 and 17.0 feet below the ground surface. This depth to groundwater suggests that groundwater has been present at the site at a depth that would be above the bottom of the proposed basin. The consultant should re- evaluate if infiltration in an approximately 20-foot-deep infiltration basin is feasible at the site considering that a 10-foot separation from the historic high groundwater (measured vertically from the bottom of the basin) is required per the Riverside County guidelines. If a

20-foot-deep basin does not meet the Riverside County guidelines, the consultant should coordinate with the project civil engineer to develop a suitable system and provide the appropriate infiltration test results and recommendations.”

Response to Comment No. 39

Infiltration is no longer recommended or proposed for the site. Refer to our response to Comment No. 37.

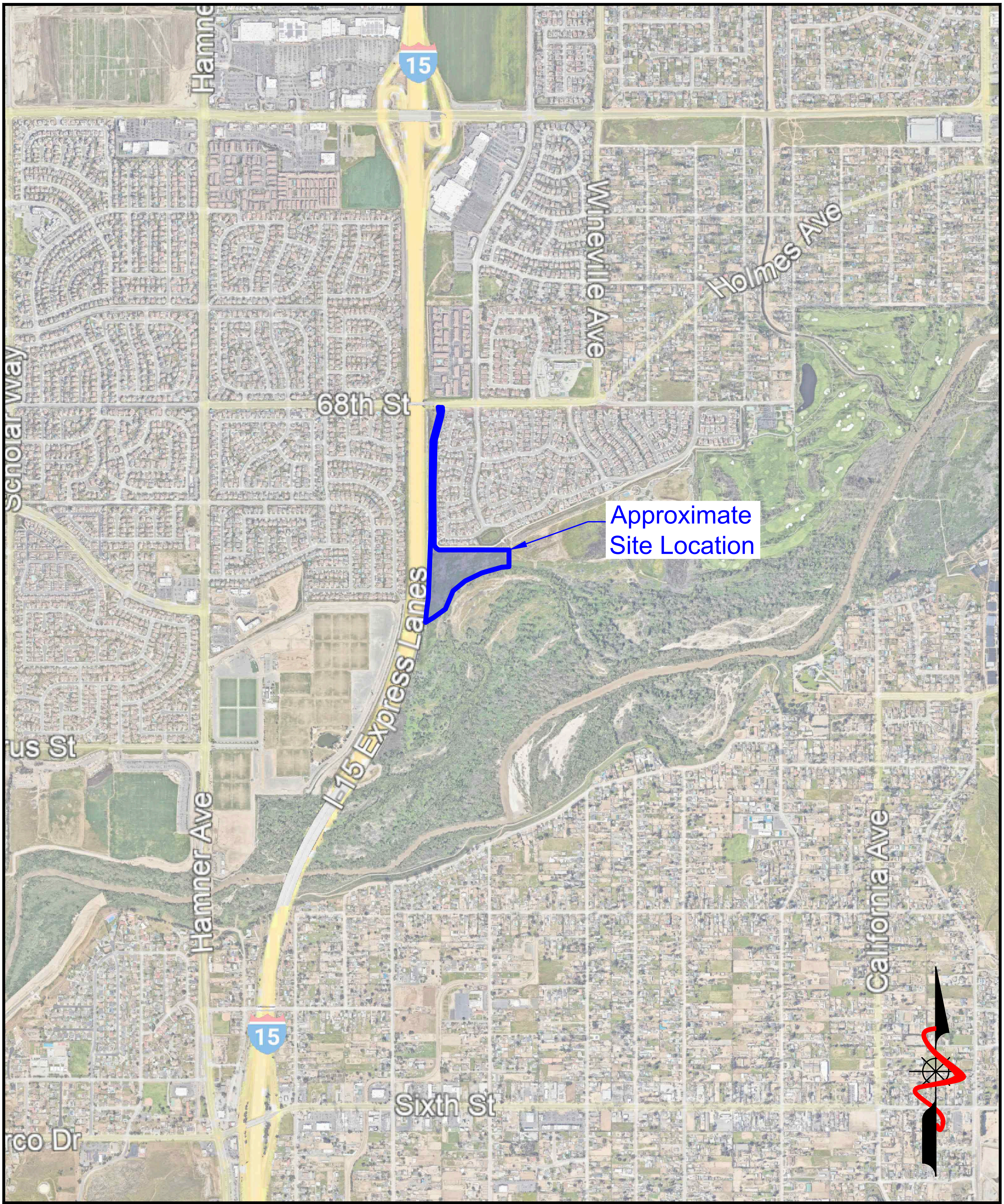
6.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made, and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions revealed by excavation may be different than our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

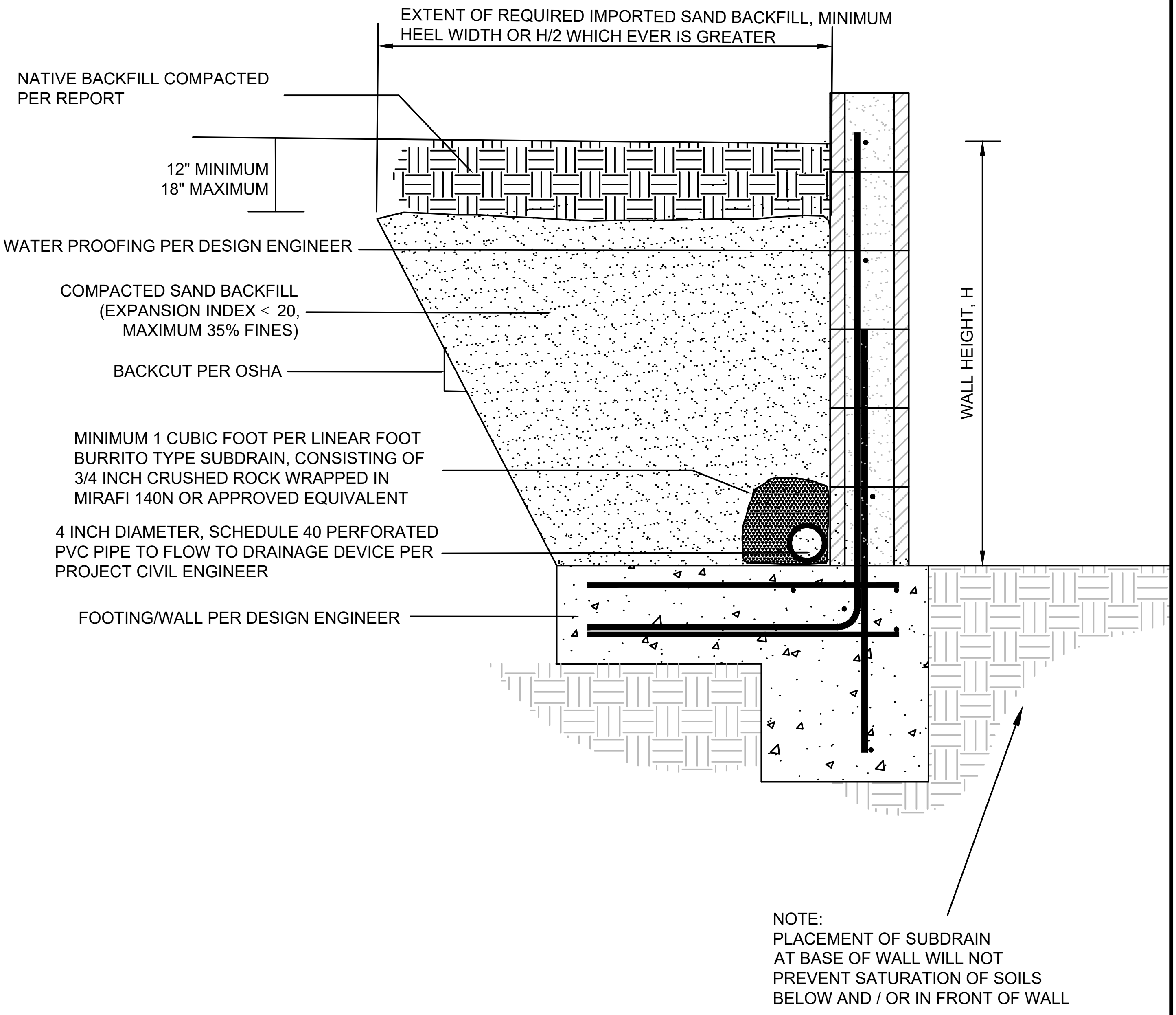
In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification, and should not be relied upon after a period of 3 years.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Jurupa Valley 18 - Self Storage
PROJECT NO.	23155-01
ENG. / GEOL.	RLD / KBC
SCALE	Not to Scale
DATE	September 2023



LGC
Geotechnical, Inc.

FIGURE 3

Retaining Wall Backfill Detail

PROJECT NAME	Jurupa Valley 18 - Self Storage
PROJECT NO.	23155-01
ENG. / GEOL.	RLD / KBC
SCALE	Not to Scale
DATE	September 2023

Appendix A
References

APPENDIX A

References

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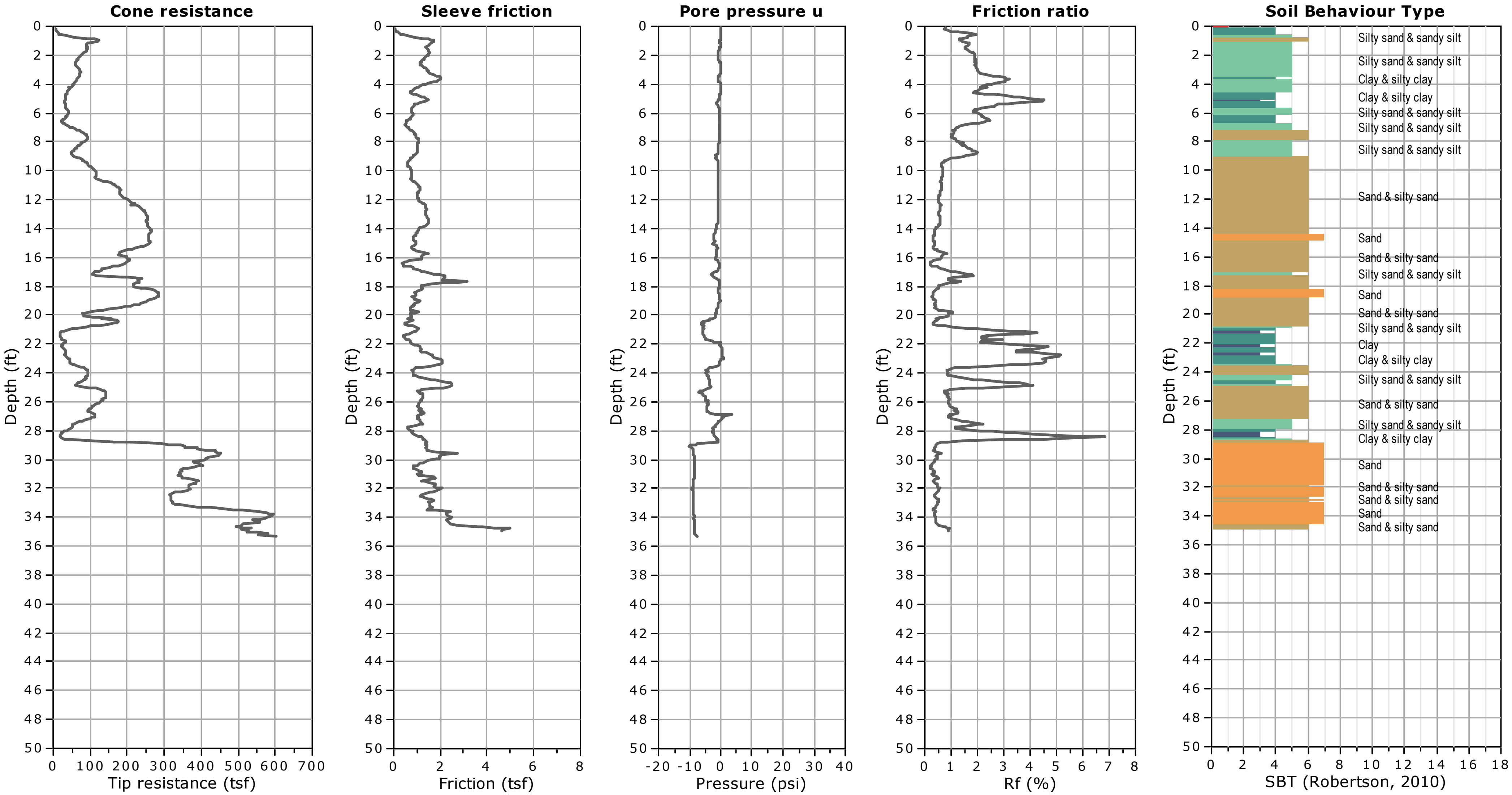
Appendix B
Boring & CPT Logs by Others

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

Project: **LGC Geotechnical / Jurupa Valley 18 - Self Storage**
Location: **68th St & I-15, Jurupa Valley, CA**

CPT-1

Total depth: 35.30 ft, Date: 8/9/2023

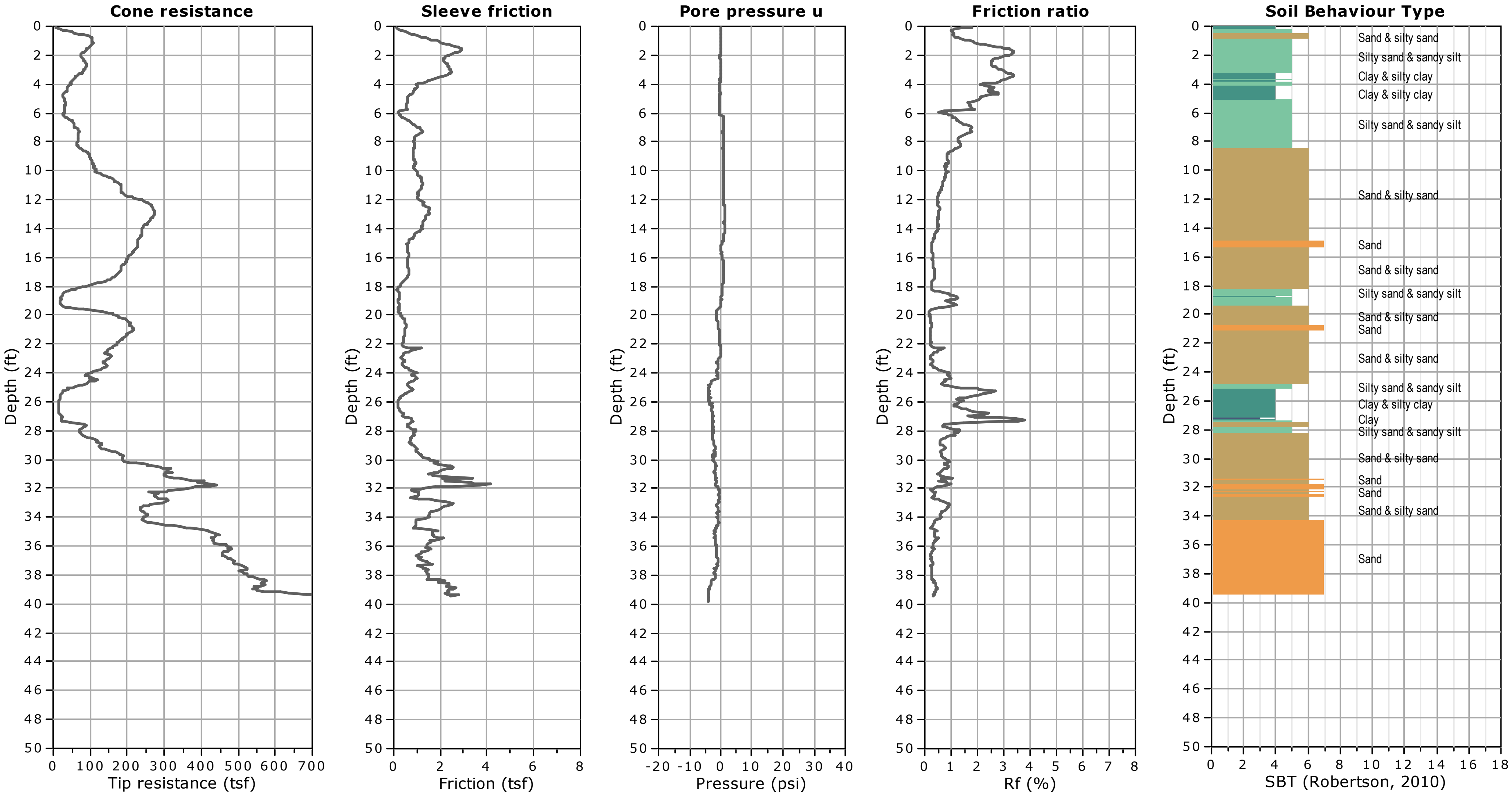


Project: **LGC Geotechnical / Jurupa Valley 18 - Self Storage**

Location: **68th St & I-15, Jurupa Valley, CA**

CPT-2

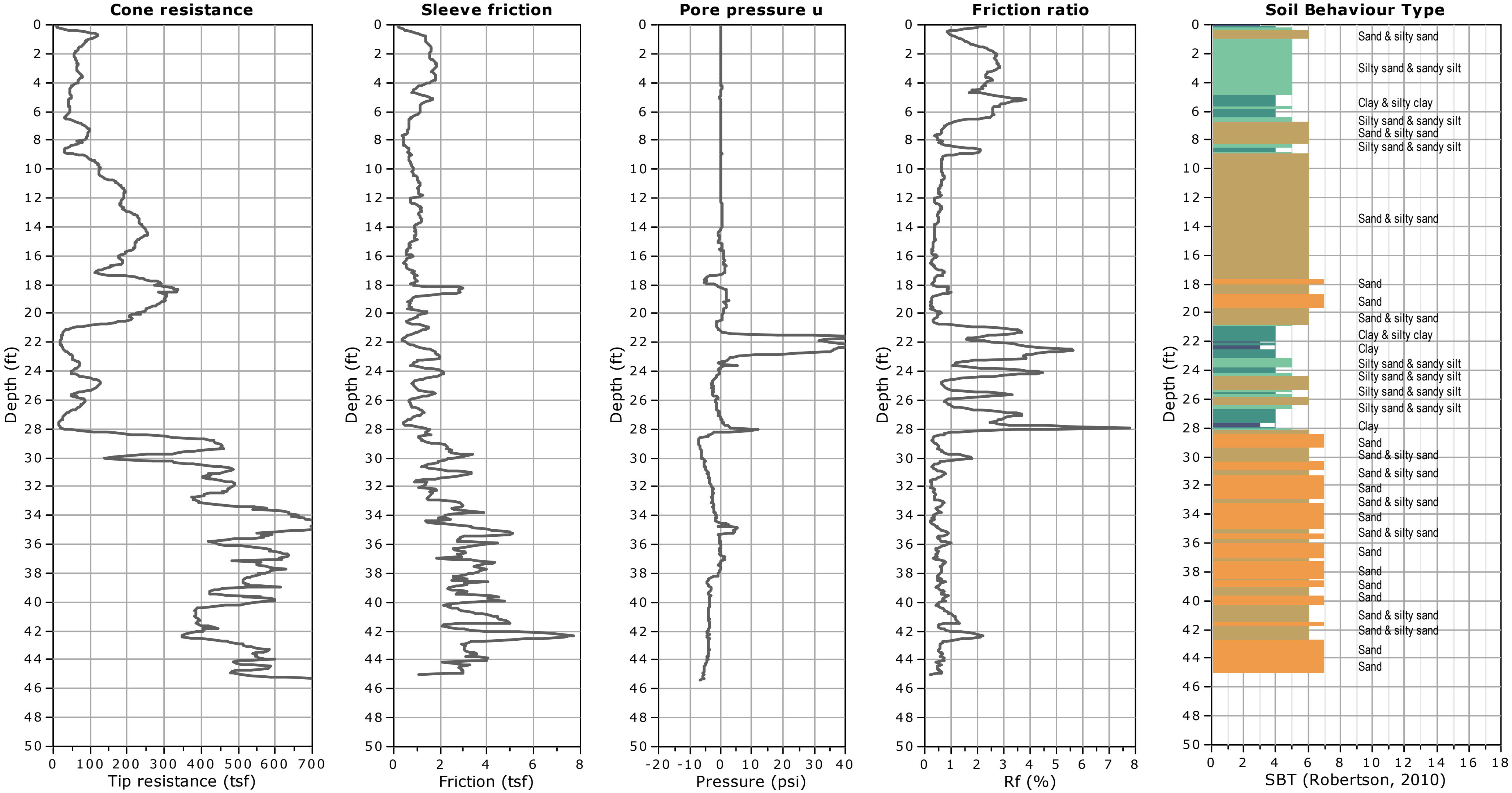
Total depth: 39.85 ft, Date: 8/9/2023



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

Project: LGC Geotechnical / Jurupa Valley 18 - Self Storage
Location: 68th St & I-15, Jurupa Valley, CA

CPT-3
Total depth: 45.41 ft, Date: 8/9/2023



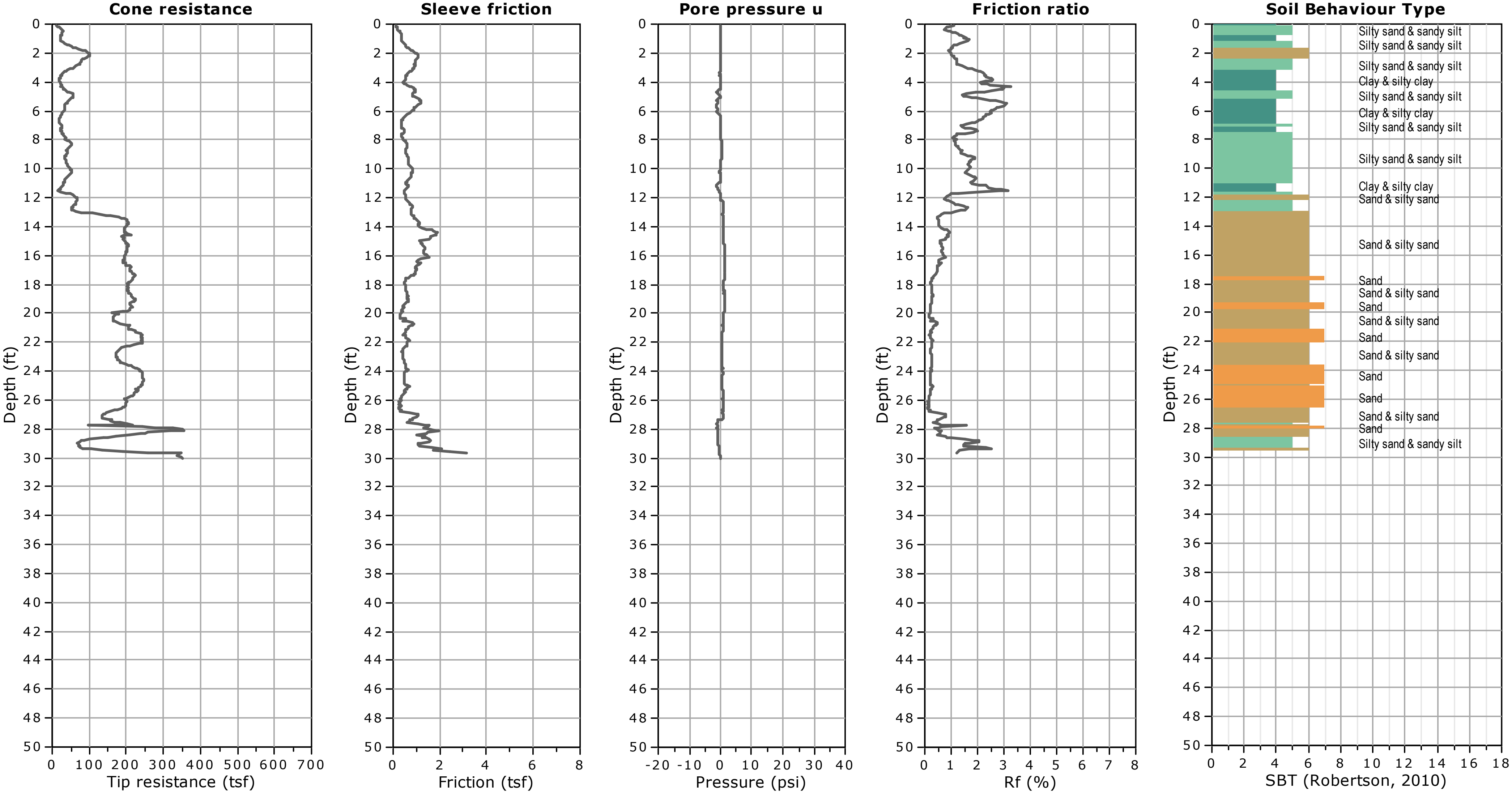
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **LGC Geotechnical / Jurupa Valley 18 - Self Storage**

Location: **68th St & I-15, Jurupa Valley, CA**

CPT-4

Total depth: 30.01 ft, Date: 8/9/2023



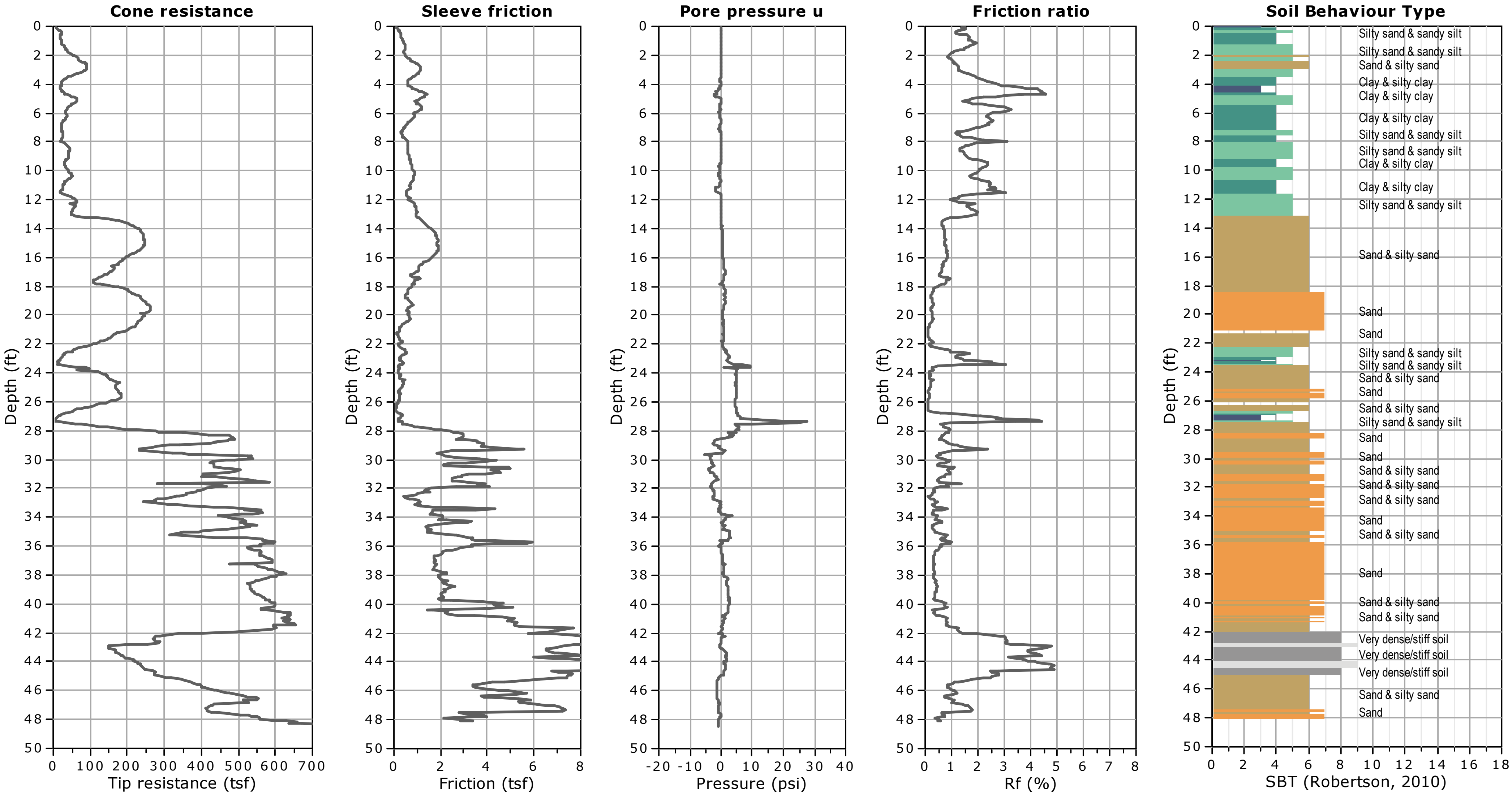
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **LGC Geotechnical / Jurupa Valley 18 - Self Storage**

CPT-5

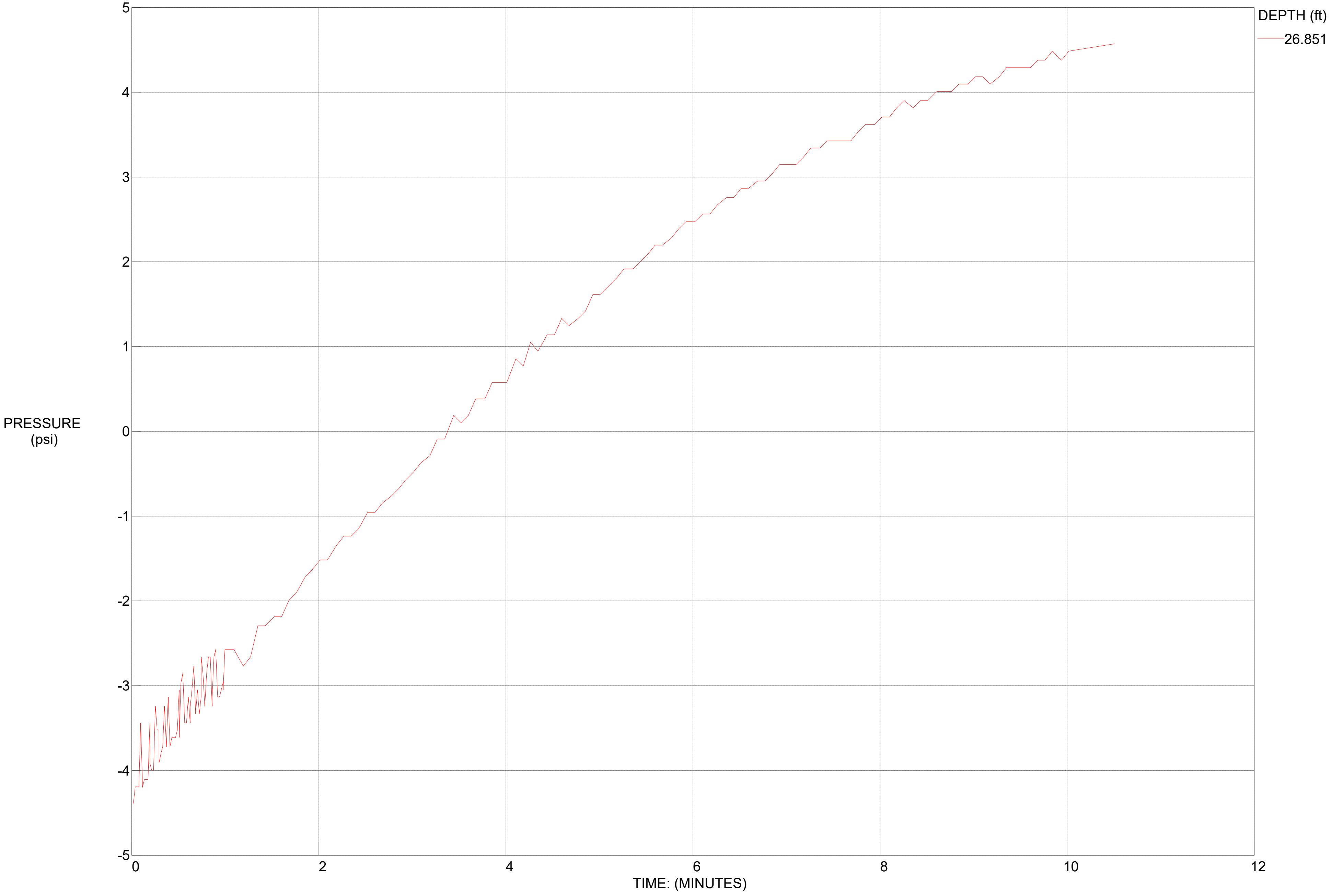
Location: **68th St & I-15, Jurupa Valley, CA**

Total depth: 48.50 ft, Date: 8/9/2023



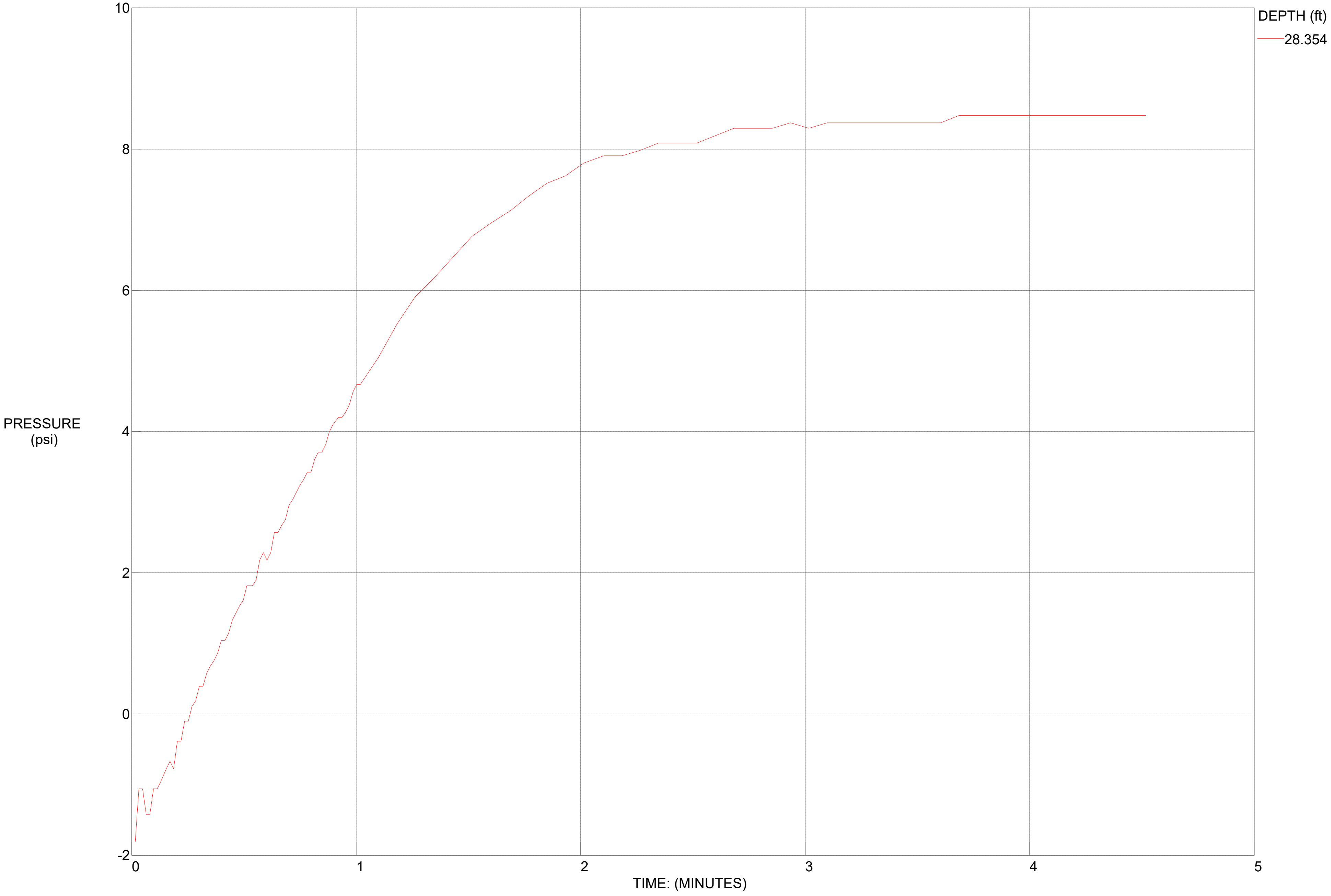
Project file: C:\Users\stevek\OneDrive - Kehoe Testing and Engineering Inc\Documents\CPT Current Data\LGC-JurupaValley8-23\CPT Report\CPeT.cpt

TEST ID: CPT-1

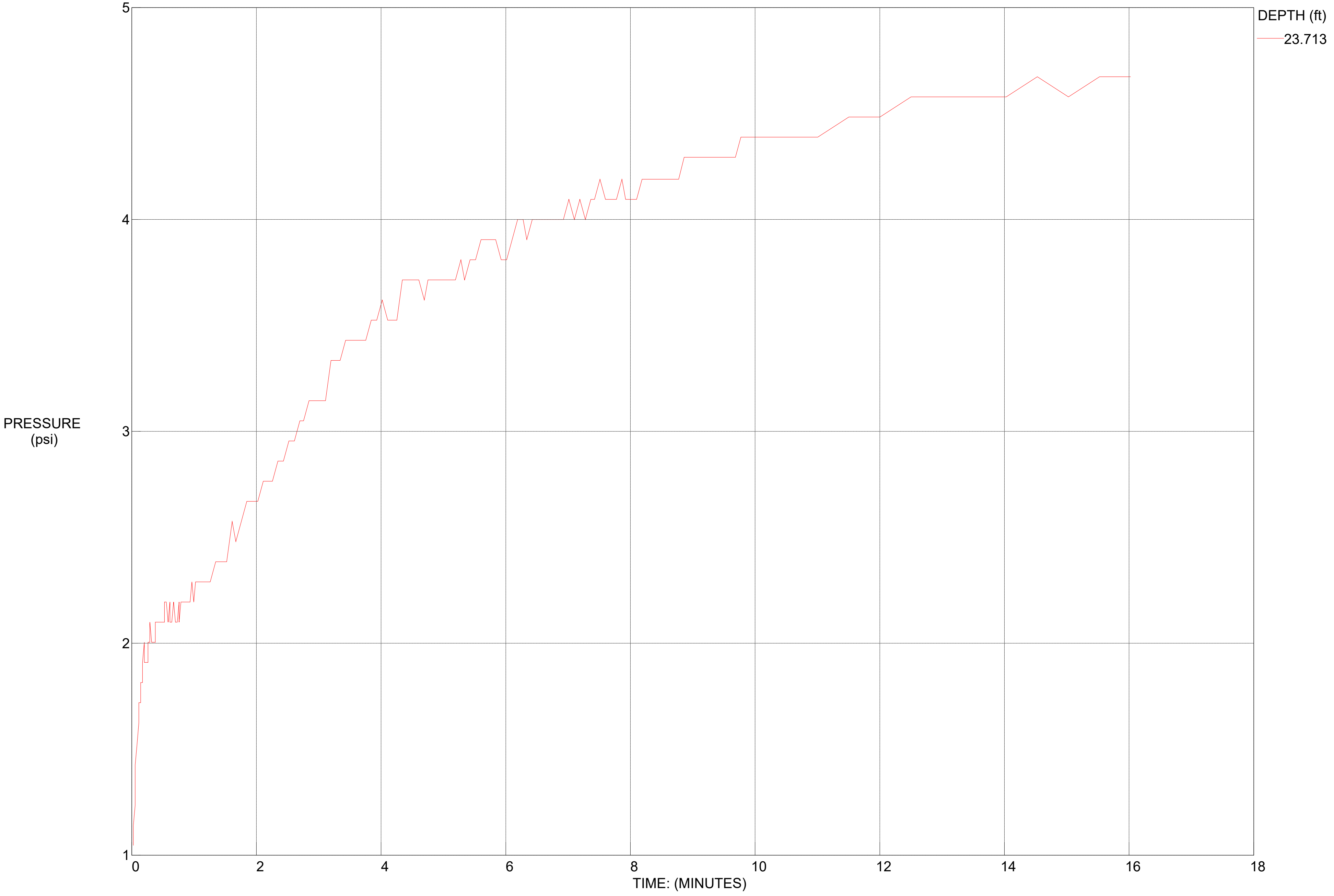


PRESSURE
(psi)
5
4
3
2
1
0
-1
-2
-3
-4
-5
0
2
4
6
8
10
12
TIME: (MINUTES)
DEPTH (ft)
26.851

TEST ID: CPT-3



TEST ID: CPT-5

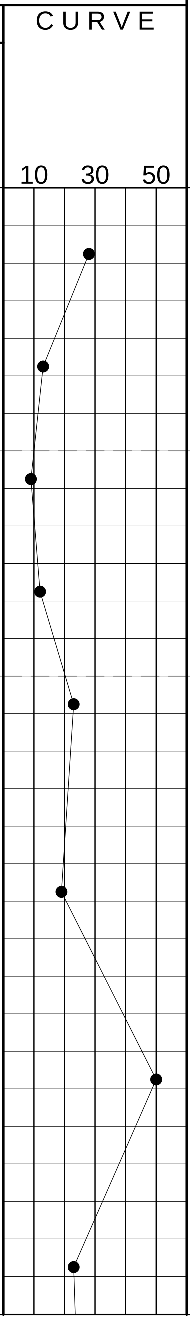


DEPTH (ft)
23.713
PRESSURE
(psi)
5
4
3
2
1
0
2
4
6
8
10
12
14
16
18
TIME: (MINUTES)

Geotechnical Boring Log B-1

Date: 9/21/20
Project Name: 68TH ST SELF-STORAGE
Page 1 of 2
Project Number: G20-1874-10
Logged By: JL
Drilling Company: 2R
Type of Rig: CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140
Drop (in.): 30
Hole Dia. (in.): 8
Top of Hole Elevation (ft): 596
Hole Location: SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	Type of Test
0 595	15 18 24	R1		Qyw ML	**YOUNG WASH DEPOSITS** SILT; gray, olive, light brown, dry, very stiff, very fine to fine grained, mottling, roots, roothairs			1.0-2.5	28	Bulk @ 1'-6'
5	7 9 11	R2			@4.0'; stiff	4.6	96.2	4.0-5.5	13	
590										
	4 6 7	R3		SM-ML	Silty SAND/SILT; gray, orange, brown, moist, loose to medium dense/stiff, very fine to fine grained with occasional medium grains, oxidation staining	23.0	86.2	7.0-8.5	9	
10 585	5 8 10	R4			@10.0'; NO RECOVERY			10.0-11.5	12	
	10 15 20	R5		SW	Well-graded SAND; gray, white, orange, light brown, damp to moist, medium dense, very fine to coarse grained	4.2	103.2	13.0-14.5	23	
15 580						▽				
	8 12 16	R6			@18.0'; wet, occasional gravels			18.0-19.5	19	
20 575										
	11 39 36	R7			@23.0'; dense to very dense	13.2	115.2	23.0-24.5	50	
25 570										
	9 15 19	R8			@28.0'; medium dense			28.0-29.5	23	
30										



Sample Legend
◪ SPT
⧓ Ring Sample (CA modified)

LGC Geo-Environmental, Inc.

Geotechnical Boring Log B-1

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE Page 2 of 2
Project Number: G20-1874-10	**Logged By:** JL
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140	**Drop (in.):** 30 **Hole Dia. (in.):** 8
Top of Hole Elevation (ft): 596	**Hole Location:** SEE GEOTECHNICAL MAP

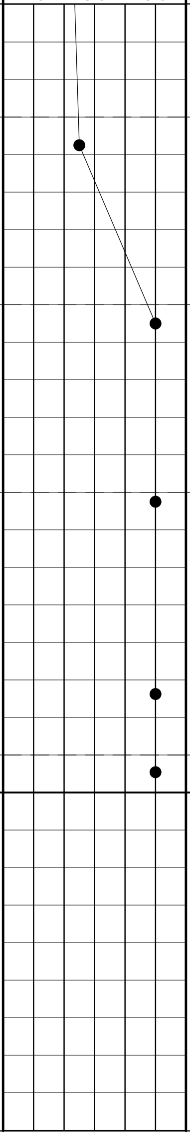
Elevation (MSL) and Depth (ft.)						
Depth (ft.)	30	35	40	45	50	55
Elevation (MSL)	565	560	555	550	545	540

Depth (ft.): 60

Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	Type of Test
7 12 25	R9		SP	Poorly-graded SAND; orange, brown, moist to wet, medium dense, fine to medium grained with occasional coarse grains, mottline, oxidation staining	18.5	116.2	33.0-34.5	25	
28 50/6"	R10		GW-SW	Well-graded GRAVEL/SAND; gray, orange, brown, wet, very dense, fine to very coarse grained	5.7	134.6	38.0-39.0	50+	
50/6"	R11		SW	Well-graded SAND; gray, light brown, wet, very dense, very fine to coarse grained			43.0-43.5	50+	
40 50/3"	R12			@48.0'; white	16.2	113.0	48.0-48.8	50+	
16 50/5"	R13		GW-SW	Well-graded GRAVEL/SAND; gray, white, light brown, wet, very dense, fine to very coarse grained			50.0-50.9	50+	

Total Depth: 51'
Groundwater at 16.5'

Standard Penetration Test C U R V E



Sample Legend

◩ SPT

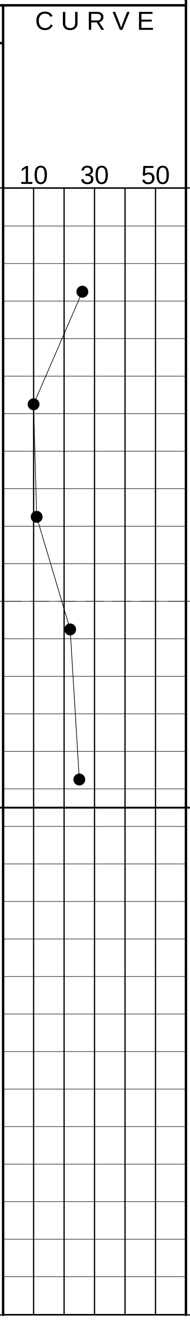
⧓ Ring Sample (CA modified)

LGC Geo-Environmental, Inc.

Geotechnical Boring Log B-2

Date: 9/21/20
Project Name: 68TH ST SELF-STORAGE
Page 1 of 1
Project Number: G20-1874-10
Logged By: JL
Drilling Company: 2R
Type of Rig: CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140
Drop (in.): 30 **Hole Dia. (in.):** 8
Top of Hole Elevation (ft): 596
Hole Location: SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	Type of Test
0 595				Qyw ML	**YOUNG WASH DEPOSITS** SILT; gray, olive, light brown, dry, very stiff, very fine to fine grained with occasional medium grains and gravel, mottling, roots, roothairs					
	9 15 24	R1				2.6	93.9	2.0-3.5	26	
5 590	6 7 8	R2			@5.0'; stiff			5.0-6.5	10	
	5 8 8	R3			@8.0'; damp	18.5	87.7	8.0-9.5	11	
10 585	9 15 18	R4		SW	Well-graded SAND; gray, white, light brown, damp, medium dense, very fine to coarse grained			11.0-12.5	22	
15 580	8 12 13	S1			@15.0'; oxidation staining	2.7		15.0-16.5	25	
20 575 25 570 30					Total Depth: 16.5' NO GROUNDWATER					



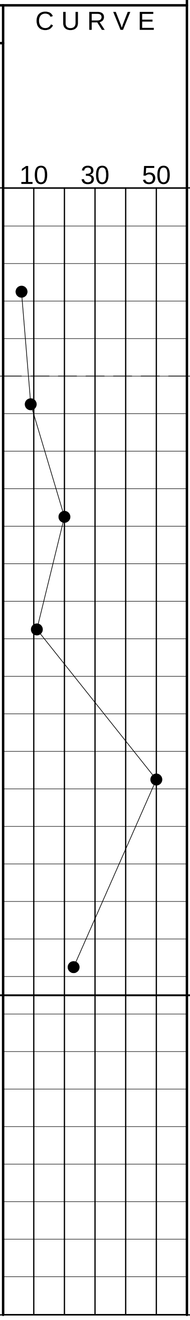
Sample Legend
◪ SPT
⧗ Ring Sample (CA modified)

LGC Geo-Environmental, Inc.

Geotechnical Boring Log B-3

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE Page 1 of 1
Project Number: G20-1874-10	**Logged By:** JL
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140	**Drop (in.):** 30 **Hole Dia. (in.):** 8
Top of Hole Elevation (ft): 593	**Hole Location:** SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	Type of Test
0				Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, orange, light brown, dry, loose/medium, very fine to medium grained, oxidation staining, mottling, roots, roothairs					
590	5 4 5	R1						2.0-3.5	6	
5	4 5 9	R2		SW	Well-graded SAND; orange, light brown, damp, loose to medium dense, very fine to coarse grained with occasional gravel, oxidation staining, roothairs			5.0-6.5	9	
585	7 11 19	R3			@8.0'; medium dense, more gravel			8.0-9.5	20	
10										
	8 8 8	R4			@11.0'; loose to medium dense			11.0-12.5	11	
580						▽				
15	17 34 34	S1			@15.0'; wet, very dense			15.0-16.5	50+	
575										
20	10 14 20	R5			@20.0'; medium dense			20.0-21.5	23	
					Total Depth: 21.5' Groundwater at 13.4'					
570										
25										
565										
30										



Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

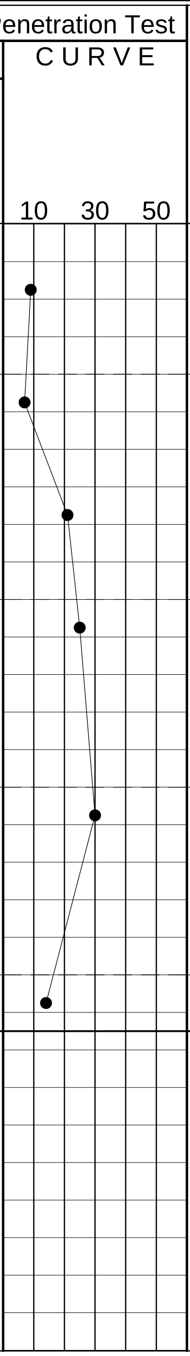
LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log B-4

Date: 9/21/20
Project Name: 68TH ST SELF-STORAGE
Page 1 of 1
Project Number: G20-1874-10
Logged By: JL
Drilling Company: 2R
Type of Rig: CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140
Drop (in.): 30
Hole Dia. (in.): 8
Top of Hole Elevation (ft): 590
Hole Location: SEE GEOTECHNICAL MAP

Elevation (MSL)	Depth (ft.)	Blow Count / 6"	Sample No.	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	SPT Depth	SPT N	Type of Test
590	0			Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, orange, brown, dry, loose to medium dense/stiff, very fine to medium grained, oxidation staining, mottling, roots, roothairs					
		4 7 7	R1			7.7	93.2	1.0-2.5	9	
585	5	4 5 6	R2	SW	Well-graded SAND; white, orange, light brown, damp, loose, very fine to coarse grained, oxidations staining			4.0-5.5	7	
		8 13 19	R3		@7.0'; medium dense	2.1	116.0	7.0-8.5	21	
580	10	10 17 21	R4	GW-SW	Well-graded GRAVEL/SAND; gray, orange, light brown, moist, medium dense, fine to very coarse grained			10.0-11.5	25	
575	15	9 16 29	R5	SW	Well-graded SAND; gray, orange, light brown, moist, medium dense to dense, very fine to coarse grained	14.8 ▽	110.0	15.0-16.5	30	
570	20	6 7 7	S1	SP-SM	Poorly-graded SAND/Silty SAND; orange, brown, wet, fine to medium grained with occasional coarse grains			20.0-21.5	14	
					Total Depth: 21.5' Groundwater at 17.0'					
565	25									
560	30									

Standard Penetration Test



Sample Legend
◩ SPT
⧗ Ring Sample (CA modified)

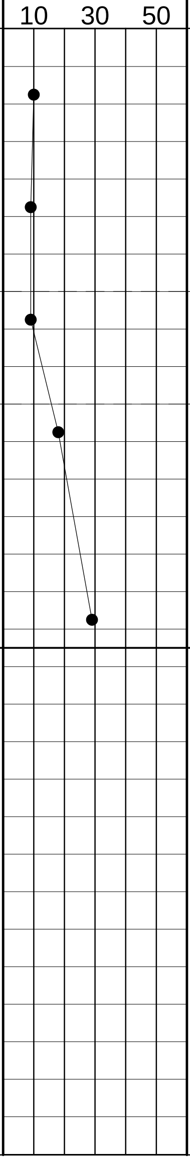
LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log B-5

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE — Page 1 of 1
Project Number: G20-1874-10	**Logged By:** JL
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140	**Drop (in.):** 30 **Hole Dia. (in.):** 8
Top of Hole Elevation (ft): 596	**Hole Location:** SEE GEOTECHNICAL MAP

Elevation (MSL)	Depth (ft.)		Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	Type of Test
	0					Qyw ML	**YOUNG WASH DEPOSITS**					
595		⧗	4 6 9	R1			SILT; gray, light brown, dry, loose to medium dense/stiff, very fine to fine grained with occasional medium grains, oxidation staining, mottling, roots, roothairs			1.0-2.5	10	
	5	⧗	6 6 8	R2			@4'; damp	2.6	97.2	4.0-5.5	9	
590												
		⧗	4 6 8	R3		SP	Poorly-graded SAND; gray, orange, light brown, damp, loose to medium dense, fine to medium grained with occasional coarse grains, oxidation staining, mottling, porous, roothairs			7.0-8.5	9	
585	10	⧗	8 11 16	R4		SW	Well-graded SAND; gray, orange, light brown, damp to moist, medium dense, very fine to coarse grained, oxidation staining, mottling, roothairs	1.6	103.2	10.0-11.5	18	
580	15	⧗	11 18 25	R5			@29'; NO RECOVERY			15.0-16.5	29	
							Total Depth: 16.5' NO GROUNDWATER					
575	20											
570	25											
	30											

C U R V E



Sample Legend

◪ SPT

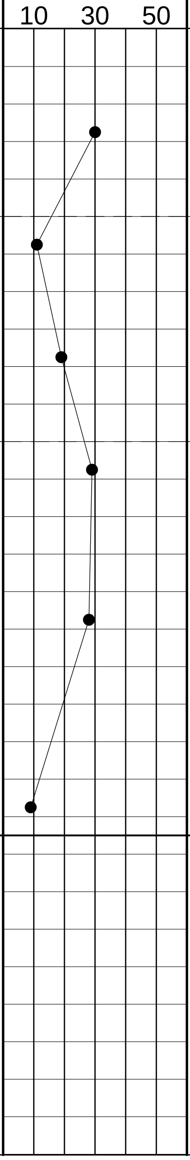
⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log B-6

Date: 9/21/20
Project Name: 68TH ST SELF-STORAGE
Page 1 of 1
Project Number: G20-1874-10
Logged By: JL
Drilling Company: 2R
Type of Rig: CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140
Drop (in.): 30
Hole Dia. (in.): 8
Top of Hole Elevation (ft): 598
Hole Location: SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist. (%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE	Type of Test
0				Qyw ML	**YOUNG WASH DEPOSITS** SILT; gray, brown, dry, very stiff, very fine to fine grained with occasional medium grains, oxidation staining, mottling, roots, roothairs						
595	12 19 26	R1				6.1	99.6	2.0-3.5	30		
5	6 7 9	R2		SM-ML	Silty SAND/SILT; gray, brown, damp, loose to medium dense/stiff, very fine to medium grained, oxidation staining, mottling, porous			5.0-6.5	11		
590	6 12 17	R3			@8.0'; orange, medium dense	2.2	110.9	8.0-9.5	19		
10	12 19 24	R4		SW	Well-graded SAND; gray, white, orange, light brown, damp, medium dense to dense, very fine to coarse grained			11.0-12.5	29		
585 15	11 18 23	R5			@15'; moist to wet	▽		15.0-16.5	28		
580 20	4 6 8	R6			@20'; NO RECOVERY, loose to medium dense			20.0-21.5	9		
575 25 570 30					Total Depth: 21.5' Groundwater at 15.8'						



Sample Legend
◩ SPT
⧗ Ring Sample (CA modified)

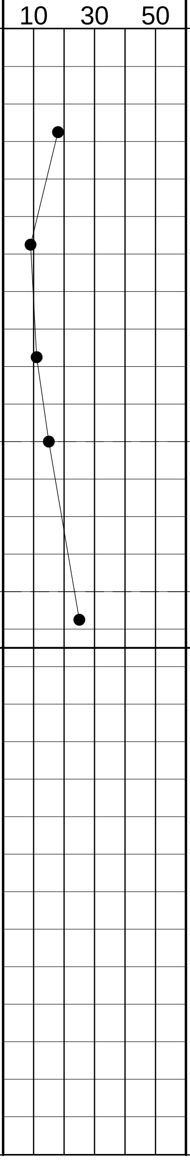
LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log B-7

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE Page 1 of 1
Project Number: G20-1874-10	**Logged By:** JL
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.): 140	**Drop (in.):** 30 **Hole Dia. (in.):** 8
Top of Hole Elevation (ft): 596	**Hole Location:** SEE GEOTECHNICAL MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist. (%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	Type of Test
0 (596)				Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry, medium dense/very stiff, very fine to fine grained with occasional medium grains, oxidation staining, mottling, roots, roothairs					
	6 9 18	R1						2.0-3.5	18	
5	6 6 7	R2			@5.0'; damp, loose to medium dense/stiff	7.2	95.8	5.0-6.5	9	
	5 7 9	R3			@8.0'; more oxidation staining	8.5	90.6	8.0-9.5	11	
11 (585)	5 7 16	R4		SP-SM	Poorly-graded SAND/Silty SAND; gray, orange, light brown, damp, medium dense, fine to medium grained with occasional coarse grains and gravel, oxidation staining, mottling			11.0-11.0	15	
15	12 16 21	R5		SW	Well-graded SAND; gray, white, orange, light brown, moist, medium dense, very fine to coarse grained, oxidation staining	▽ 11.9	109.3	15.0-16.5	25	

Total Depth: 16.5'
Groundwater at 15.1'



Elevation: 595, 590, 585, 580, 575, 570
Depth: 0, 5, 10, 15, 20, 25, 30

Sample Legend
◩ SPT
⧗ Ring Sample (CA modified)

LGC Geo-Environmental, Inc.

Geotechnical Boring Log IB-1

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE	Page 1 of 1
Project Number: G20-1874-20	**Logged By:** JL	
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER	
Drive Weight (lbs.):	**Drop (in.):**	**Hole Dia. (in.):** 8
Top of Hole Elevation (ft):	**Hole Location:** SEE INFILTRATION TEST LOCATION MAP	

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0			▨	Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry, very fine to medium grained, oxidation staining, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC Geo-Environmental, Inc.

Geotechnical Boring Log IB-2

Date: 9/21/20

Project Name: 68TH ST SELF-STORAGE

Page 1 of 1

Project Number: G20-1874-20

Logged By: JL

Drilling Company: 2R

Type of Rig: CME-75 HOLLOW STEM AUGER

Drive Weight (lbs.):

Drop (in.):

Hole Dia. (in.): 8

Top of Hole Elevation (ft):

Hole Location: SEE INFILTRATION TEST LOCATION MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test			Type of Test
								SPT		CURVE	
								Depth	N	10 30 50	
0			▨	Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained, oxidation staining, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log IB-3

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE	Page 1 of 1
Project Number: G20-1874-20	**Logged By:** JL	
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER	
Drive Weight (lbs.):	**Drop (in.):**	**Hole Dia. (in.):** 8
Top of Hole Elevation (ft):	**Hole Location:** SEE INFILTRATION TEST LOCATION MAP	

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test			Type of Test
								SPT		C U R V E	
								Depth	N	10 30 50	
0			▨	Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained, oxidation staining, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log IB-4

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE Page 1 of 1
Project Number: G20-1874-20	**Logged By:** JL
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.):	**Drop (in.):** **Hole Dia. (in.):** 8
Top of Hole Elevation (ft):	**Hole Location:** SEE INFILTRATION TEST LOCATION MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0			▒	Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained with occasional coarse grains, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC Geo-Environmental, Inc.

Geotechnical Boring Log IB-5

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE Page 1 of 1
Project Number: G20-1874-20	**Logged By:** JL
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.):	**Drop (in.):** **Hole Dia. (in.):** 8
Top of Hole Elevation (ft):	**Hole Location:** SEE INFILTRATION TEST LOCATION MAP

Elevation (MSL)	and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
	0				Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained with occasional coarse grains, roots, roothairs5						
	5					Total Depth: 5' NO GROUNDWATER						
	10											
	15											
	20											
	25											
	30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log IB-6

Date: 9/21/20
Project Name: 68TH ST SELF-STORAGE
Page 1 of 1
Project Number: G20-1874-20
Logged By: JL
Drilling Company: 2R
Type of Rig: CME-75 HOLLOW STEM AUGER
Drive Weight (lbs.):
Drop (in.):
Hole Dia. (in.): 8
Top of Hole Elevation (ft):
Hole Location: SEE INFILTRATION TEST LOCATION MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0				Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log IB-7

Date: 9/21/20

Project Name: 68TH ST SELF-STORAGE

Page 1 of 1

Project Number: G20-1874-20

Logged By: JL

Drilling Company: 2R

Type of Rig: CME-75 HOLLOW STEM AUGER

Drive Weight (lbs.):

Drop (in.):

Hole Dia. (in.): 8

Top of Hole Elevation (ft):

Hole Location: SEE INFILTRATION TEST LOCATION MAP

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0				Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained with occasional coarse grains, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Geotechnical Boring Log IB-8

Date: 9/21/20	**Project Name:** 68TH ST SELF-STORAGE	Page 1 of 1
Project Number: G20-1874-20	**Logged By:** JL	
Drilling Company: 2R	**Type of Rig:** CME-75 HOLLOW STEM AUGER	
Drive Weight (lbs.):	**Drop (in.):**	**Hole Dia. (in.):** 8
Top of Hole Elevation (ft):	**Hole Location:** SEE INFILTRATION TEST LOCATION MAP	

Elevation (MSL) and Depth (ft.)	Blow Count / 6"	Sample No.	Soil Graphic	Geologic / Group Symbol	DESCRIPTION	In-Situ Moist.(%)	Dry Density (pcf)	Standard Penetration Test SPT Depth	N	CURVE 10 30 50	Type of Test
0			▨	Qyw SM-ML	**YOUNG WASH DEPOSITS** Silty SAND/SILT; gray, light brown, dry to damp, very fine to medium grained with occasional coarse grains, roots, roothairs						
5					Total Depth: 5' NO GROUNDWATER						
10											
15											
20											
25											
30											

Sample Legend

◩ SPT

⧗ Ring Sample (CA modified)

LGC GEO-ENVIRONMENTAL, INC.

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soil. 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.

***Soil Classification*:** Soil were classified according the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. The soil classifications (or group symbol) are shown on the laboratory test data, and boring logs.

***Maximum Dry Density Tests*:** The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM test method D1557. The test results are presented in the table below:

SAMPLE LOCATION	**SAMPLE DESCRIPTION (USCS)**	**MAXIMUM DRY DENSITY (% by weight)**	**OPTIMUM MOISTURE CONTENT (%)**
B-1 @ 1.0′-6.0′	SILT	110.5	15.0

***Expansion Index*:** The expansion potential of a selected sample was evaluated by the Expansion Index Test, U.B.C. Standard No. 18-2 and/or ASTM test method D4829. Specimens are molded under a given compactive energy at or near the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*EXPANSION INDEX*	*EXPANSION POTENTIAL**
B-1 @ 1.0′-6.0′	SILT	21	Low

*Per ASTM D4829

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

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*SULFATE CONTENT (ppm)*	*SULFATE EXPOSURE**
B-1 @ 1.0′-6.0′	SILT	321	Negligible

*Per ACI 318-19

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

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*CHLORIDE CONTENT (ppm)*
B-1 @ 1.0′-6.0′	SILT	670

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

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*pH*	*MINIMUM RESISTIVITY (ohm-cm)*
B-1 @ 1.0'-6.0'	SILT	7.9	640

Direct Shear: Direct shear tests were performed on selected remolded samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inch per minute (depending upon the soil type). The graphical test results are presented in the table below:

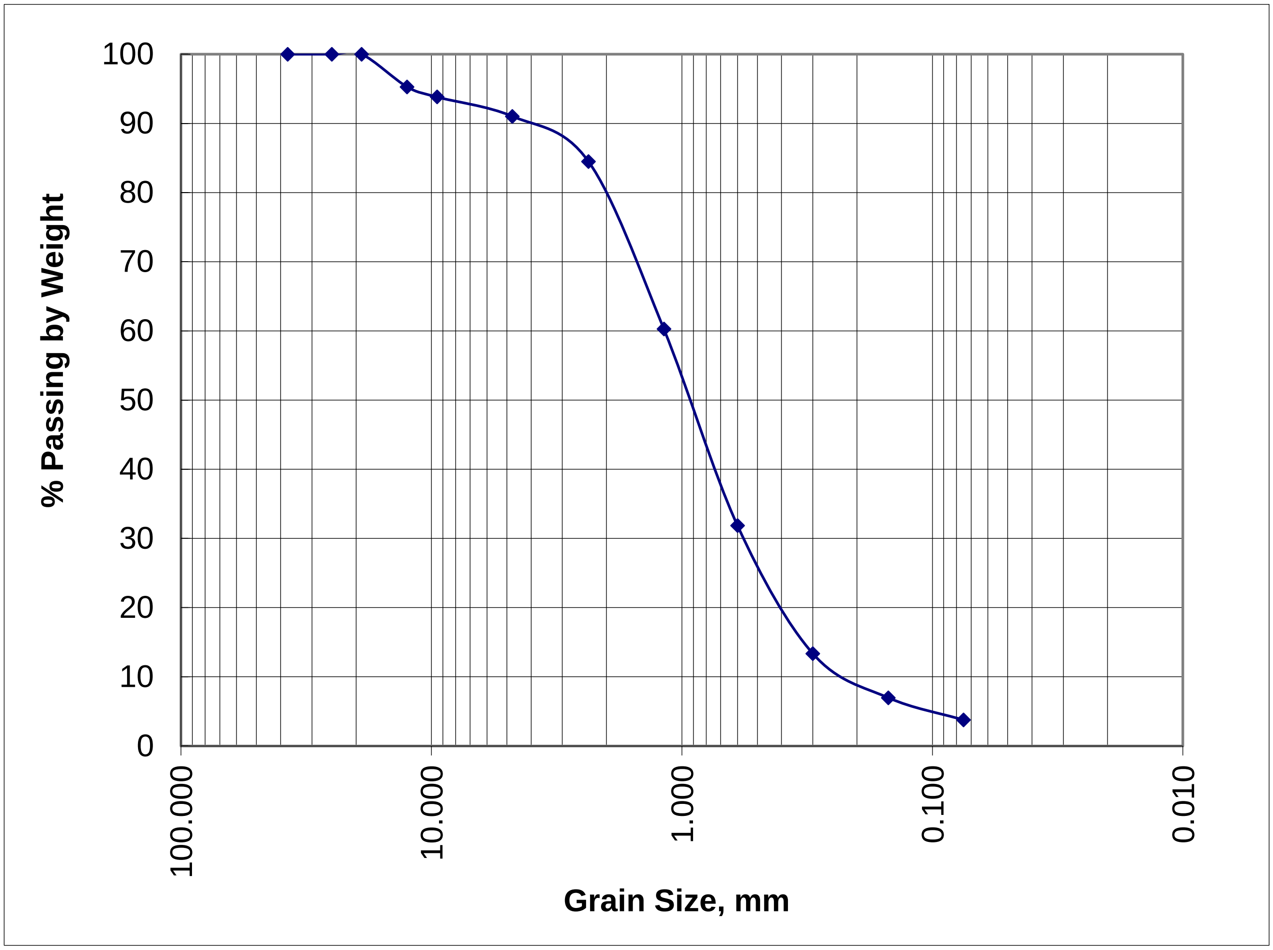
SAMPLE LOCATION	*SAMPLE DESCRIPTION*	*ANGLE OF INTERNAL FRICTION (degrees)*	*COHESION (psf)*
B-1 @ 1.0'-6.0'	SILT	29	260

R-Value: The resistance R-value was determined by the ASTM test method D2844 for base, sub-base, and basement soil. The sample was prepared and exudation pressure and R-value were determined. These results were used for pavement design:

SAMPLE LOCATION	*SAMPLE DESCRIPTION* **(USCS)**	*R-VALUE*
B-7 @ 1.0'-6.0'	Silty SAND/SILT	38

Grain Size Distribution: Representative samples were dried, weighted, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve. 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 D422 (CTM 202). The graphical test results are presented on the following pages.

Consolidation: A consolidation test was performed on an undisturbed sample. (Modified ASTM Test method D2435. The samples (2.42 inches in diameter and 1-inch in height) were 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 stamp was recorded as the ration of the amount of vertical compression to the original sample height. The in progress graphical test results are presented on the following pages.



U.S. Standard Sieve Sizes

3" 1.5" 3/4" 3/8" #4 #8 #30 #50 #100 #200

	Gravel		Sand				
Cobbles	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

GRAIN SIZE DISTRIBUTION

Location	Depth, ft.	Sample ID
B-1	18.0	R6

Classification	Classification
SM	Silty Sand

Project No.	G201874-10
Project Name	68th St. Self Storage
Client	Jurpa Valley 18 LLC

Sieve Size	Grain Size, mm	% Passing
1 1/2 in.	37.500	**100.0**
1 in.	25.000	**100.0**
3/4 in.	19.000	**100.0**
1/2 in.	12.500	**95.3**
3/8 in.	9.500	**93.8**
#4	4.750	**91.0**
#8	2.360	**84.5**
#16	1.180	**60.3**
#30	0.600	**31.9**
#50	0.300	**13.4**
#100	0.150	**7.0**
#200	0.075	**3.8**

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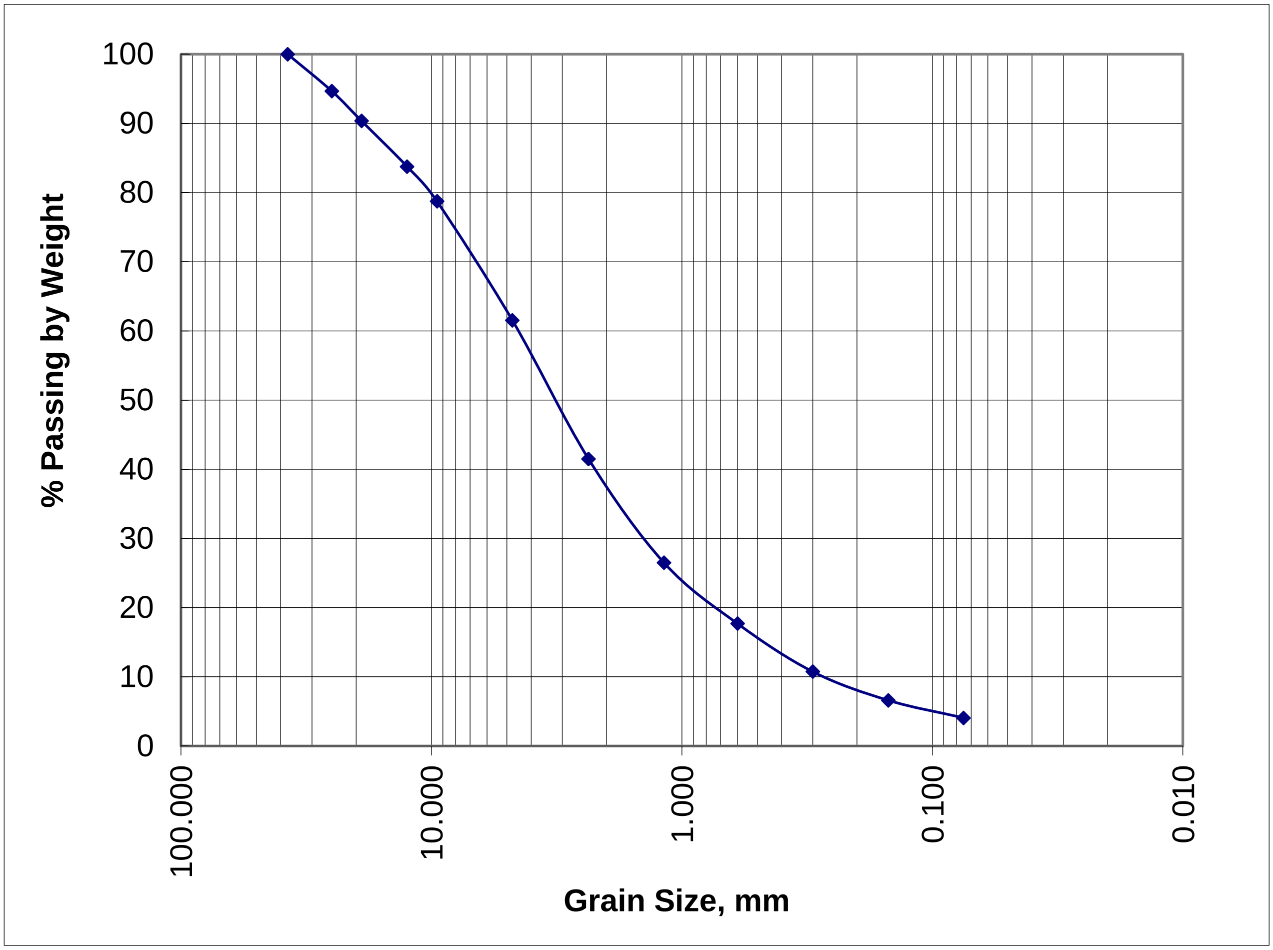
Temecula, CA 92590



U.S. Standard Sieve Sizes

3" 1.5" 3/4" 3/8" #4 #8 #30 #50 #100 #200

Cobbles	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

GRAIN SIZE DISTRIBUTION

Location	Depth, ft.	Sample ID
B-1	38.0	R10

Classification	Classification
SM	Silty Sand w/ Gravel

Project No.	G201874-10
Project Name	68th St. Self Storage
Client	Jurpa Valley 18 LLC

Sieve Size	Grain Size, mm	% Passing
1 1/2 in.	37.500	**100.0**
1 in.	25.000	**94.7**
3/4 in.	19.000	**90.3**
1/2 in.	12.500	**83.7**
3/8 in.	9.500	**78.7**
#4	4.750	**61.5**
#8	2.360	**41.5**
#16	1.180	**26.5**
#30	0.600	**17.7**
#50	0.300	**10.7**
#100	0.150	**6.6**
#200	0.075	**4.1**

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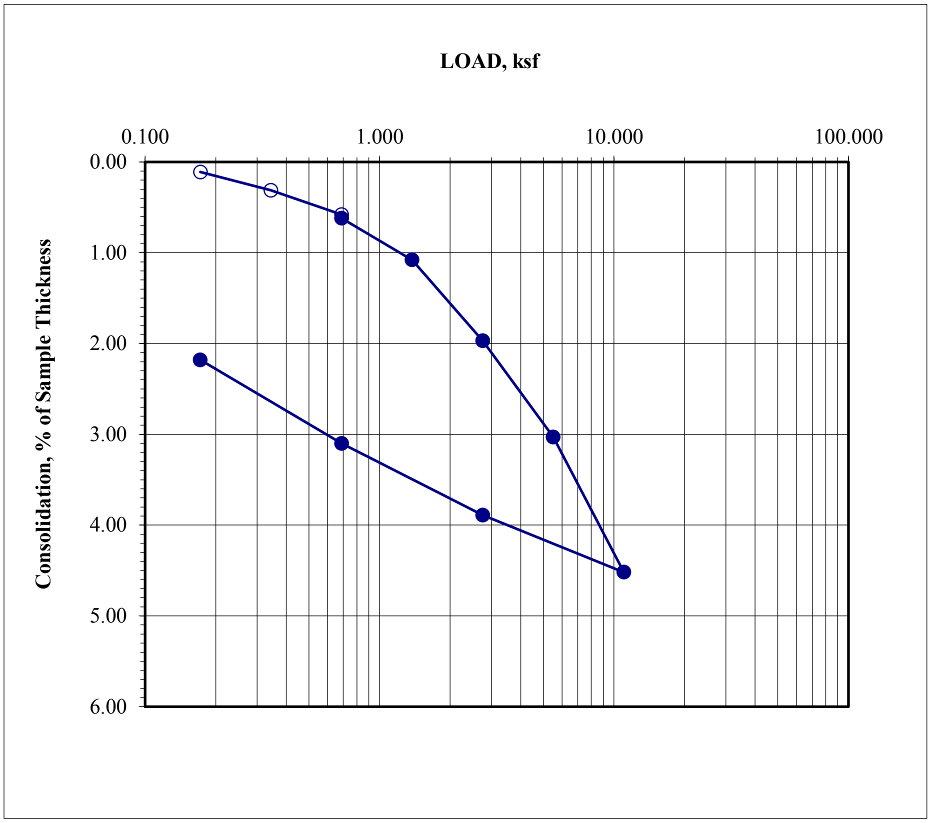
Temecula, CA 92590

CONSOLIDATION TEST RESULTS



Note: Filled circle denotes readings after sample was submerged in water

	In-place	Post Consol
Dry Density, (pcf):	85.9	86.7
Moisture (%):	23.0	31.8

WATER ADDED @ ksf: 0.689
MAXIMUM LOAD, ksf: 11.020
SOIL DESCRIPTION: Silty Sand to Sandy Silt
U.S.C.S. SM/ML
% Collapse/Swell (-): 0.04

P.N. G201874-10 **LOCATION:** B-1 @ 7'
PROJECT: Jurupa Valley 18 LLC

LGC Geo-Environmental, Inc.
27570 Commerce Center Dr, # 128
Temecula, CA 92590

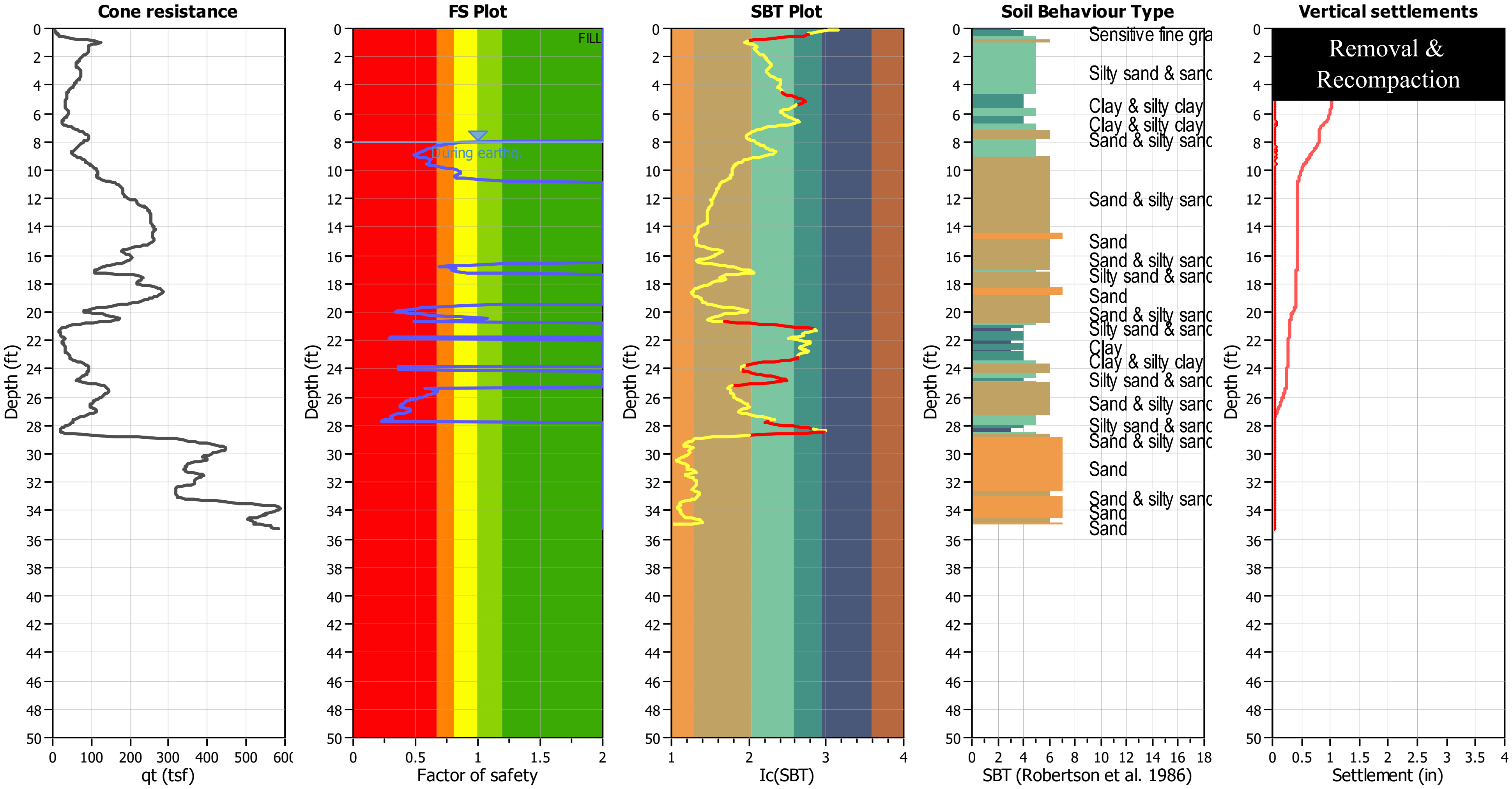
Plate: C-1

Appendix D
Liquefaction Analyses

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

Project: **Jurupa Valley 18**
Location: **23155-01**

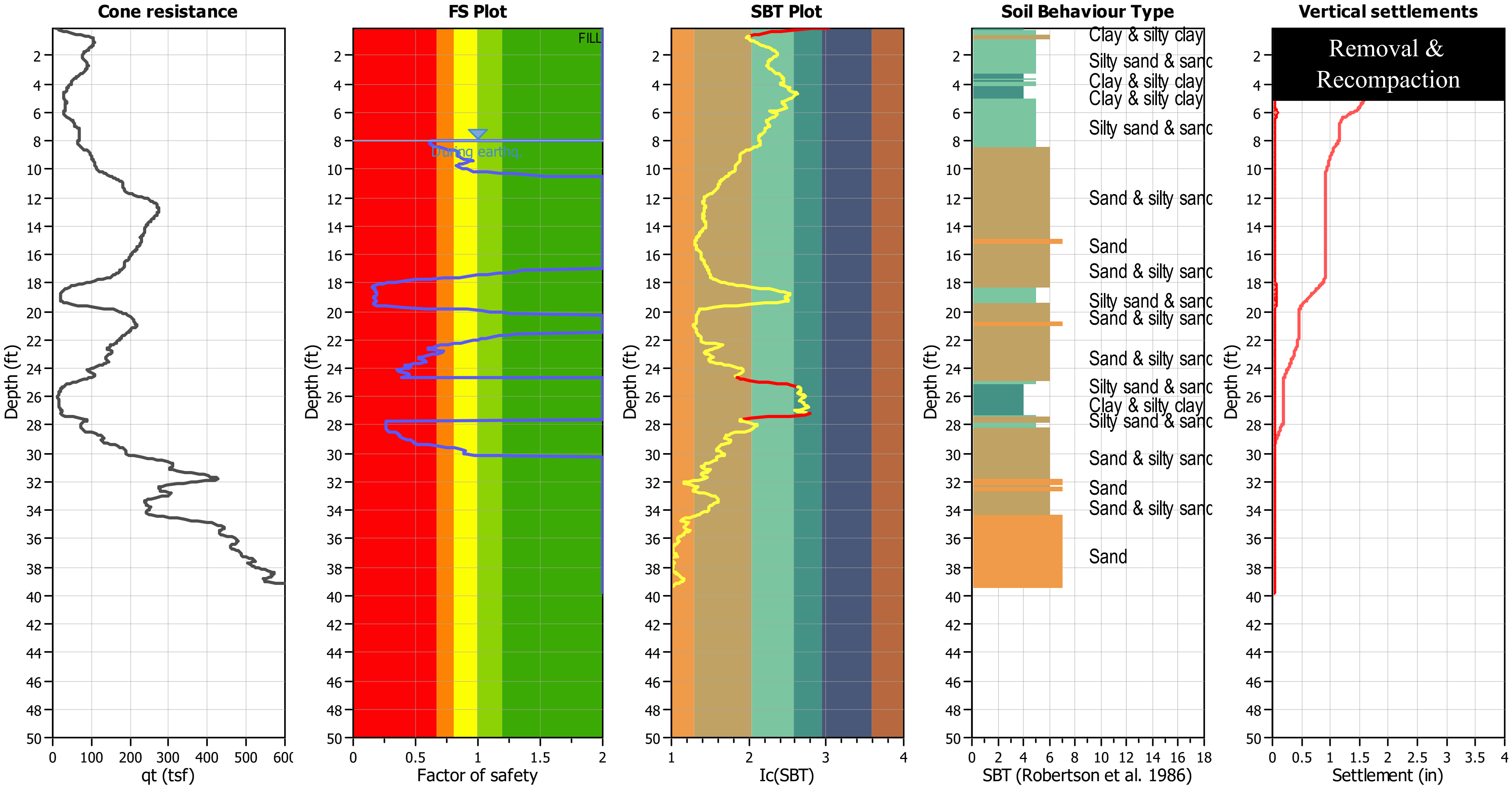
CPT: CPT-1
Total depth: 35.30 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	16.00 ft	Fill height:	8.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **Jurupa Valley 18**
Location: **23155-01**

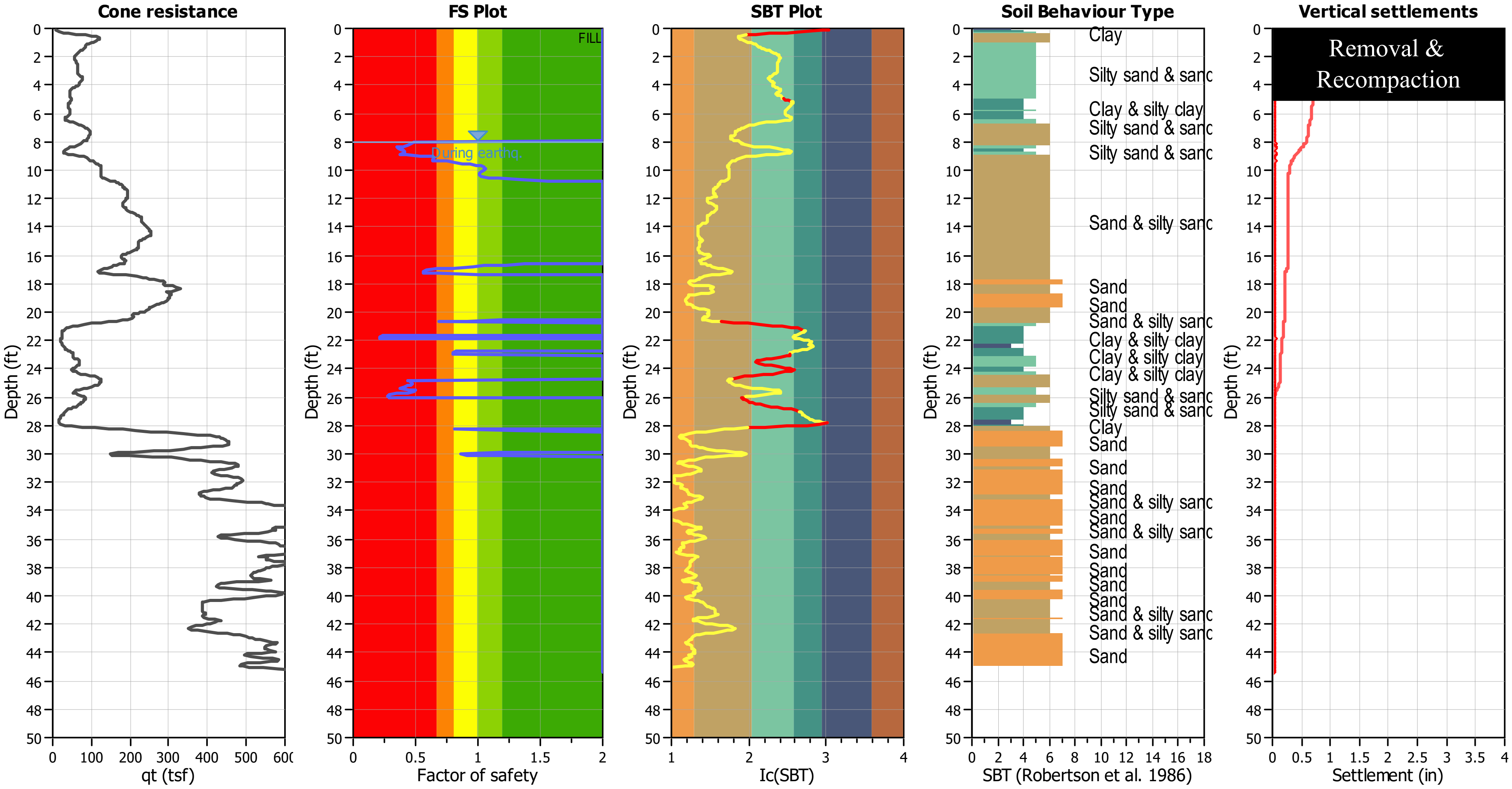
CPT: CPT-2
Total depth: 39.85 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	16.00 ft	Fill height:	8.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **Jurupa Valley 18** **CPT: CPT-3**

Location: **23155-01** Total depth: 45.41 ft



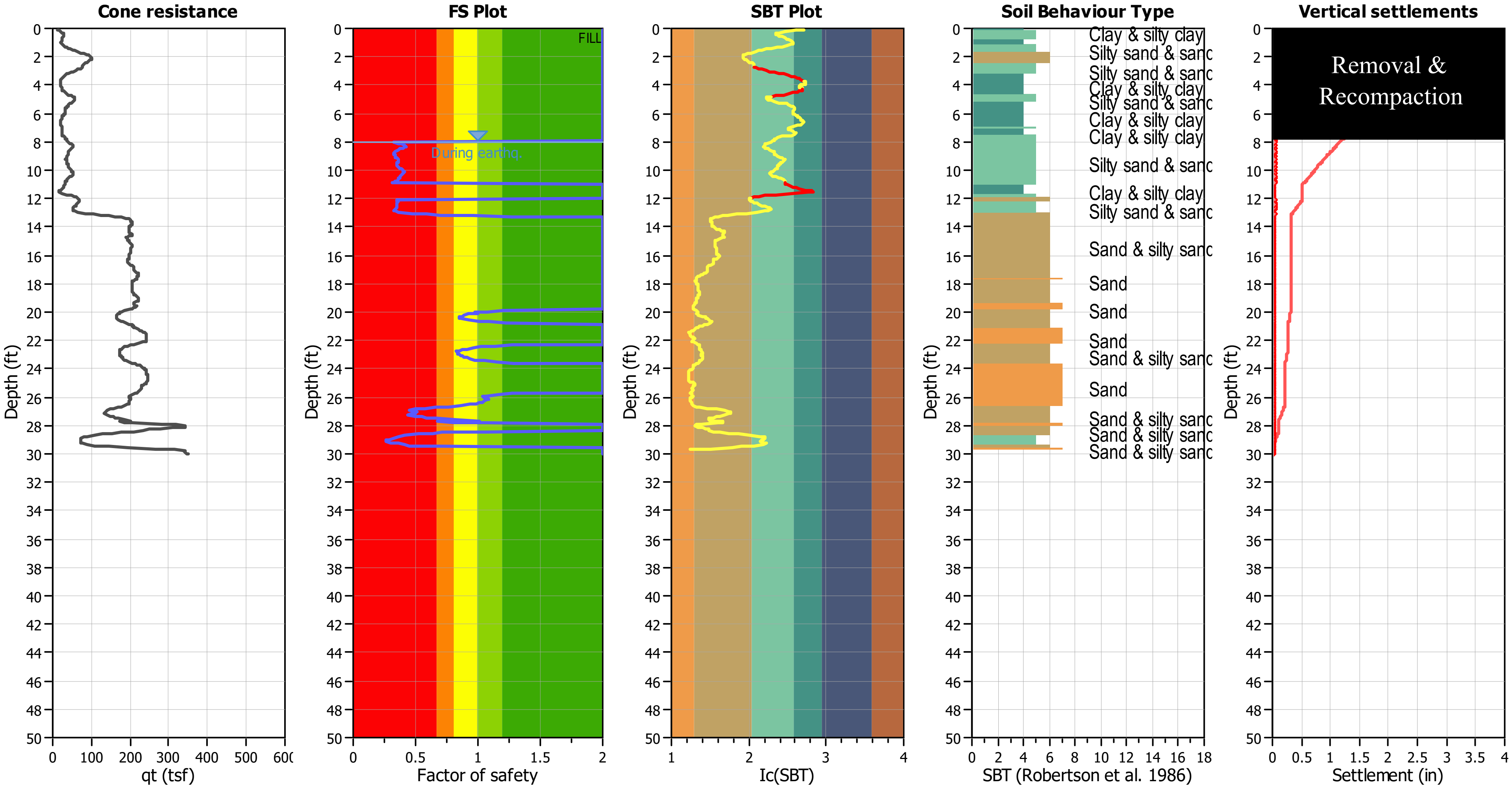
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	16.00 ft	Fill height:	8.00 ft		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Project: **Jurupa Valley 18**

Location: **23155-01**

CPT: CPT-4

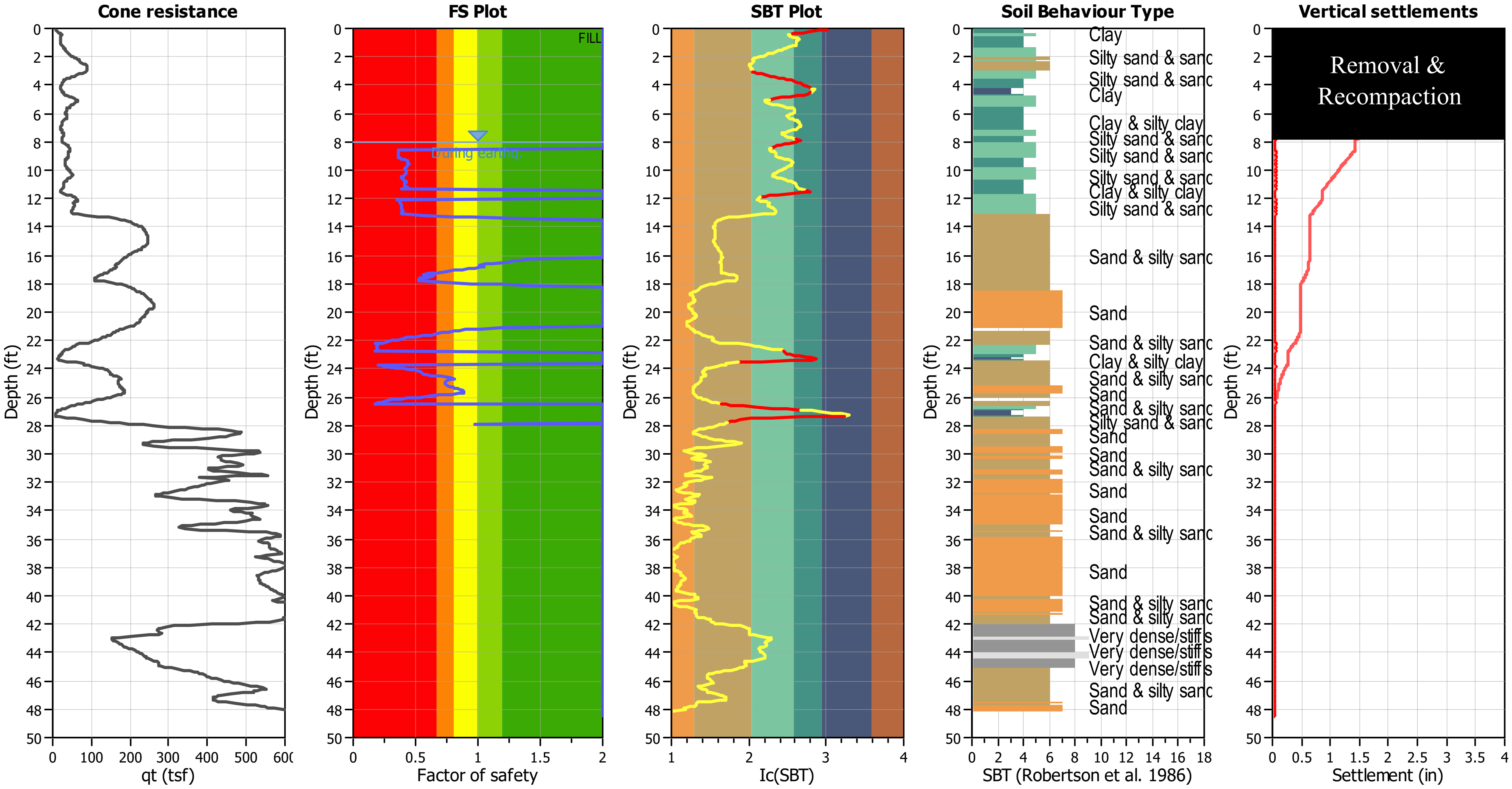
Total depth: 30.01 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	11.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	13.00 ft	Fill height:	5.00 ft		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Project: **Jurupa Valley 18**
Location: **23155-01**

CPT: CPT-5
Total depth: 48.50 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	11.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	13.00 ft	Fill height:	5.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	6.74	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Appendix E
City of Jurupa Valley (2023)

INTEROFFICE MEMORANDUM

TO: MIGUEL DEL RIO, ASSOCIATE PLANNER

FROM: LILLYANNA DIAZ, ASSISTANT ENGINEER

SUBJECT: MA20269 – 3RD REVIEW

DATE: MAY 30, 2023

REVIEWED: PROJECT DISCRIPTION & NARRATIVE

RV SELF STORAGE SITE PLAN; PREPARED BY J. CRAIG MANN; DATED FEBRUARY 18, 2023

ROUGH GRADING PLANS; PREPARED BY WH ENGINEERING GROUP; DATED APRIL 27, 2023

APPLICANT'S RESPONSE TO COMMENTS; PREPARED BY WH ENGINEERING; DATED APRIL 27, 2023

PRELIMINARY WATER QUALITY MANAGEMENT PLAN; PREPARED BY WH ENGINEERING; DATED MARCH 28, 2023

PRELIMINARY HYDROLOGY STUDY; PREPARED BY WH ENGINEERING; DATED MARCH 29, 2023

CC: ENGINEERING DEPARTMENT

Engineering has reviewed the documents submitted to the Engineering Department, for the new RV and Self-Storage facility located south of 68th Street and east of I-15 and has the following comments:

General:

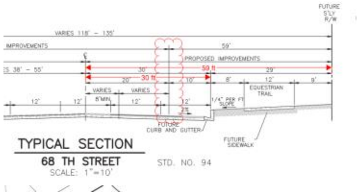
1. *NOTE: Engineering will review next submittal package to determine if all the required plans and/or studies were included. Incomplete submittals will not be reviewed.*
   3rd Review: No action is required.
2. *NOTE: Due to the significant change in the project layout some comments may be repeated from previous review to notify applicant that certain conditions still apply.*
   3rd Review: No action is required.
3. A site plan from prepared by Becklund Civil was provided and included some proposed spot elevations, contours, and slopes. However, a conceptual grading plan should provide additional information such as cross sections at property lines, existing and/or proposed contours, flow lines throughout the site and the existing contours around the site to show the transition from proposed project to existing grades outside of the project site. Resubmit conceptual grading plan.
   3rd Review: Provide cross sections at property lines and basin on the conceptual grading plan. Cross section of the basin shall show the elevation of the 100-yr storm event and the FEMA flood level. Add labels to proposed contour lines. For entitlement purposes please revise plan title to "Conceptual Grading Plan".
4. The project proposed to treat all discharge onsite and will not discharge into the Santa Ana River nor connect to any Riverside County Flood Control and Water Conservation District

(RCFC&WCD) facilities and infrastructure. Based on the proposed design, no additional action is required.
3rd Review: No action is required.

5. Hyperlinks provided in the preliminary title report submitted in 1st submittal do not work. Resubmit a preliminary title report with working hyperlinks. Applicant will be responsible for obtaining approval and consent for work within all easements within the property from the appropriate easement holder, as applicable.
3rd Review: Comment is addressed.
6. Based on architectural plans, it seems like this project will have a subdivision. Applicant shall identify type of subdivision(s) proposed such as a lot merger, tentative parcel map, etc.
3rd Review: A lot line adjustment will be conditioned. No immediate action is required.
7. Verify with Building and Safety Department if an ADA path of travel is required from public right-of-way. Provide comment letter.
3rd Review: Please provide comment letter.

Street Improvements:

8. Revision to previous comments: 68th Street is designated as a Major Highway and will be improved to match improvements to the development on the east side. Revise street section to revise the dimensions shown and identify the existing southerly r/w line. See figure below. The following shall be conditioned.
    a. Provide a 59-ft half-width right-of-way along project frontage
    b. Provide 30-ft paved section shall be provided from centerline to curb face
    c. Provide a 29-ft parkway as shown in the street section provided.



3rd Review: Comment is addressed.

Conceptual Grading Plans:

9. See comment no. 3 and provide the following on the conceptual grading plan:
    a. Provide distance and bearings for property lines. Provide cross sections to show how the proposed development and how improvements will transition to adjacent existing properties and right-of-way. Provide various N-S sections through the property lines including information on existing road improvements versus proposed and E-W sections through property lines. Include offsets and call outs for improvements.
    3rd Review: Provide cross section at property lines and basin. Add labels to contour lines for proposed grading.
    b. Show utility connections to JCSD sewer and water facilities.
    3rd Review: Is there a proposed connection to a waterline for potable water?
    c. Show offer of dedication along 68th Street to provide the 59-ft half-width right-of-way. Include the corner cut cutbacks. It is understood that complete corner cutbacks cannot be provided due to adjacent property owners.
    3rd Review: Comment is addressed.
    d. Identify existing and proposed r/w line.
    3rd Review: Comment is addressed.
    e. A list of easements was shown on plan was shown. Identify if the easements are plottable or non-plottable. If plottable, call them out on conceptual grading

and site plans.
<u>3rd Review</u>: Proposed easements are shown; however, also show existing easements. Indicate if they are plottable and if so, show on plans.

f. Site plans shows a 20-ft equestrian trail; however, it is not clear how that will connect with existing trails or future trails. Engineering Department will not require an equestrian trail. Due to the nature of the trails in the river, it is recommended that parcels located in the "preserved habitat" be used for open space purposes.
<u>3rd Review</u>: A) Based on project information and responses provided the 20-ft "trail" easement is an access easement to access basin and habitat portion. This will not be for the public. Please confirm or provide additional information, if needed. B) Site plan shows a 20-ft equestrian trail with a proposed trailhead, hitching post, and DG area. Identify entity that will maintain these improvements. It is still not clear where the trail will end.

g. Show any inlets and storm drains that will be used to collect and direct flow to the basin.
<u>3rd Review</u>: Comment is addressed.

h. Provide a legend for line types, symbols, and abbreviations.
<u>3rd Review</u>: Comment is addressed.

i. Show how maintenance ramp will meet the basin's bottom.

j. **<u>3rd Review</u>: Comment is addressed. Note that the proposed 15% slope is acceptable provided that it is constructed with asphalt or concrete paving.**

k. Show all proposed and existing fences and walls.
<u>3rd Review</u>: Identify gate to access the septic system for maintenance purposes.

l. Interior side slopes of the basin are designed with 3:1 slopes. The surrounding (i.e., embankment?) slopes are unknown. Revise conceptual grading plans to provide details and if needed, show fence as required by the LID BMP Design Handbook.
3rd Review: Portions of the external embankment slope is steeper than 4:1. Max. slope is 4:1.

<u>Site Plan:</u>

10. Plans shall clearly identify the location of existing and proposed right-of-way and easements.
<u>3rd Review</u>: Item has not been addressed.
11. Proposed access road shall meet minimum Fire Department requirements and be a paved road with safety lighting. Show proposed lighting.
<u>3rd Review</u>: Item will be conditioned.
12. Consider relocating gated entrance closer to the project driveway on 68th Street.
<u>3rd Review</u>: Comment is addressed. Secondary gates are proposed.
13. A turnaround area should be provided at the project's gated entrance.
<u>3rd Review</u>: No additional action is required.
14. It is recommended that guest parking be provided at the project's gated entrance.
<u>3rd Review</u>: How will new customers access the main office?

<u>Preliminary Drainage Study:</u>

15. Preliminary Hydrology Report is deemed adequate for preliminary Planning purposes. A final report shall be submitted, upon entitlement, to the Engineering department for review and approval of the City Engineer.

3rd Review: An updated preliminary drainage report report was submitted. The 10-year and 100-year 24-hour needs to be analyzed. Report should include discussion of the floodplain and impacts to the basin located within the floodplain and below the BFE.

Preliminary Water Quality Management Program:

16. On page 9, the report identifies a 96% of existing pervious area (17.5 areas) and 62% of pervious area post project completion (11.4 areas). The project description identifies 28.9 areas will be improved and 71 areas will be dedicated to a conservation agency (remain impervious). Clarify discrepancy in area of impervious versus pervious area.
    3rd Review: Comment is no longer applicable. See comment #40-43.
17. Page 9 identifies infiltration rate as low as 1 inch per hour for the basin; however, on page 13, applicant checked that in-situ infiltration rates are greater than 1.6 inches/hour. Page 13 checks that in-situ infiltration rates are greater than 1.6 inches/hour. The infiltration testing investigation includes infiltration rates as low as 0.7 inches/hour (Page 38 of PWQMP). Please provide clarification.
    3rd Review: Comment is no longer applicable. See comment #40-43.
18. Check one of the options under Section D.2.
    3rd Review: Comment is no longer applicable. See comment #40-43.
19. Clarify if second DMA listed in Table C.1 is "DMA 1 – Landscaping".
    3rd Review: Comment is no longer applicable.
20. It is estimated that this project will have over 150,000 sf for parking. Under Table E.1, check parking lots.
    3rd Review: Comment is no longer applicable. See comment #40-43.
21. Due to its proximity to the Santa Ana River and the potential spill of pollutants the proposed permeable pavers for the RV parking area will not be permitted. Provide alternative
    3rd Review: Comment is no longer applicable. See comment #40-43.
22. Include conceptual grading plans in Appendix 2.
    3rd Review: Comment is no longer applicable. See comment #40-43.
23. Although applicant has noted that an outlet structure was not needed due to basin sized, provide volume of the 100-year storm, and demonstrate that basin can accommodate storm event including. Provide grades/slopes within and along the perimeter of the basin and identify the emergency overflow pathway. Proposed design is not consistent with LID BMP designs and will need the City Engineer's approval.
    3rd Review: Show proposed outlet and emergency spillway structures. Provide cross section of the basin per other comments.

FEMA Floodplain:

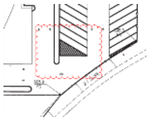
24. Per National Flood Insurance Program Flood Insurance Rate Map (FIRM) 060256, site is within or partially within a special flood hazard area. Clearly identify floodplain limits on conceptual grading plan. Resubmit conceptual grading plans.
    3rd Review: Show the current boundary of Zone AE boundary on CGP.
25. A development permit shall be submitted (application available can be obtained from the City).
    3rd Review: Item will be conditioned.
26. Show and identify the Special Flood Hazard Areas (SFHA) and base flood elevations (BFE). Grading plan was not resubmitted.
    a. Non-residential structures must be additionally dry flood proofed and for qualified non-habitable structures, the lowest floor must be wet flood proofed to one-foot minimum above BFE.

    3rd Review: BFE is shown. Comment is addressed.

27. Identify the elevations of the lowest floors of all proposed structures and pads. This is shown on the site plan and should be shown on the conceptual grading plan when submitted.
    3rd Review: Comment is addressed.

Traffic, Transportation, and Circulation:

28. No additional traffic or VMT study is required. Project is exempt from LOS and VMT analysis.
    3rd Review: VMT and traffic-related documents are being reviewed separately.
29. No on-street parking will be allowed on access road.
    3rd Review: Item will be conditioned.
30. Show line of sight on plans at project entrance. Show line of sight for both the plan and profile view. Use a speed of 50 mph. If there are any obstructions to the driver's line of sight, left-out movements may be prohibited, and applicant shall recommend a design that physically prohibits said movement.
    3rd Review: VMT and traffic-related documents are being reviewed separately.
31. Provide an exhibit of proposed improvements on 68th Street, specifically the at the project entrance. Some improvements may not be feasible due to Caltrans r/w, bridge width, etc. Include dimensions of lane widths and transition lengths. Note that project, north of 68th Street will widen; however, the existing lane configuration will remain. Plans can be provided upon applicant's request.
    3rd Review: Comment is addressed. Transition improvements along 68th Street, west of the project will be conditioned.
32. Provide turning template showing RVs can access clouded parking stalls.

    3rd Review: Comment is addressed.

Preliminary Geotechnical Report:

33. The geotechnical report was not submitted and should address comments from previous comment letter.
    3rd Review: See comments 35 through 39.
34. Refer to PWQMP comments. Infiltration testing should take borings at areas proposed to infiltrate.
    3rd Review: See comments 35 through 39.
35. **3rd Review: Provide an updated summary of the proposed grading and construction including maximum depths of cut and fill and maximum slope heights, and provide updates to the conclusions and recommendations, as appropriate.**
36. **3rd Review: The consultant should review and update the previous liquefaction analyses performed by LGC for the subject site. The analyses results are presented in Appendix D of the LGC report. Borings B-1 and B-3 were used for the analyses. Although, boring B-3 was only 21.5 feet deep, liquefaction was evaluated at this location up to a depth of 50 feet. The consultant should explain (A) how they were able to extend this boring to a depth of 50 feet for the sake of liquefaction analyses, and (B) how they chose the sampler blow-counts and fines content of the soil layers below a depth of 21.5 feet at B-3. (C) The consultant should also justify the fines content they chose for all soil layers at borings B-1 and B-3. (D) The ring sampler blow-counts considered in the calculation tables in Appendix D do not match with that shown on the boring logs. The consultant should**

also explain (E) why a maximum moment magnitude of 7.0 was used in liquefaction evaluation; (F) what was meant by “design ground motion”; and (G) why was a value of 0.8 used for “design ground motion”? (H) Finally, the consultant should also explain why a design groundwater depth of 15 feet was used in liquefaction analyses, particularly for boring B-3, when groundwater was encountered at a depth of 13.4 feet in boring B-3 during drilling.

37. **3rd Review: The consultant references a Preliminary Infiltration Testing Investigation report by LGC dated October 15, 2020 in their “Update” letter. The preliminary infiltration report by LGC was not submitted, but LGC’s preliminary geotechnical investigation was submitted. The preliminary infiltration report by LGC should be submitted to the City for review.**
38. **3rd Review: The consultant should present the boring and infiltration test locations on a plan that uses the current grading plan as a base map.**
39. **3rd Review: The consultant refers to and summarized previous infiltration tests performed within the site and states that three of the eight previous tests were located in the vicinity of the proposed infiltration basin. The tests were performed at a depth of approximately 5 feet. Based on the rough grading plan, excavation for the proposed infiltration basin will be approximately 20 feet below the existing ground surface. Based on the proposed grading, the soils that were tested for infiltration characteristics will be removed during excavation of the basin. The consultant should perform a sufficient number of percolation tests at the planned bottom elevation of the proposed basin, including evaluating the soils below the bottom of the basin through which the water will infiltrate, in accordance with Riverside County guidelines. However, the consultant states that groundwater was previously encountered at depths between 13.4 and 17.0 feet below the ground surface. This depth to groundwater suggests that groundwater has been present at the site at a depth that would be above the bottom of the proposed basin. The consultant should re- evaluate if infiltration in an approximately 20-foot-deep infiltration basin is feasible at the site considering that a 10-foot separation from the historic high groundwater (measured vertically from the bottom of the basin) is required per the Riverside County guidelines. If a 20-foot-deep basin does not meet the Riverside County guidelines, the consultant should coordinate with the project civil engineer to develop a suitable system and provide the appropriate infiltration test results and recommendations.**

Preliminary WQMP (continued):

40. **3rd Review: Section A - Area of impervious project footprint is the same as total area of proposed impervious surface. "Area of Impervious Project Footprint" is the overall project footprint and should be larger than the proposed impervious surface area. Based on Section C, area of project footprint is 459,543 SF.**
41. **3rd Review: Section A.3 – A permit will be required from the agencies listed below. Check all applicable boxes in Table A.2.**
    a. **State Department of Fish and Game, 1602 Streambed Alteration Agreement**
    b. **State Water Resources Control Board, Clean Water Act (CWA) Section 401 Water Quality Cert**
    c. **US Army Corps of Engineers, CWA Section 404 Permit**
42. **3rd Review: Section C – Add DMA-4 to Table C.1.**
43. **3rd Review: Table C.2 – Revise information provided in the area and stabilization type.**

Thank you.

Appendix F
General Earthwork and Grading Specifications

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 *Preparation of Areas to be Filled*

2.1 *Clearing and Grubbing*

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

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

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

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

2.2 *Processing*

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

2.3 Over-excavation

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

2.4 Benching

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

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 Import

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

4.0 Fill Placement and Compaction

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 *Frequency of Compaction Testing*

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

4.7 *Compaction Test Locations*

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

5.0 *Subdrain Installation*

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

6.0 *Excavation*

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

7.0 *Trench Backfills*

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

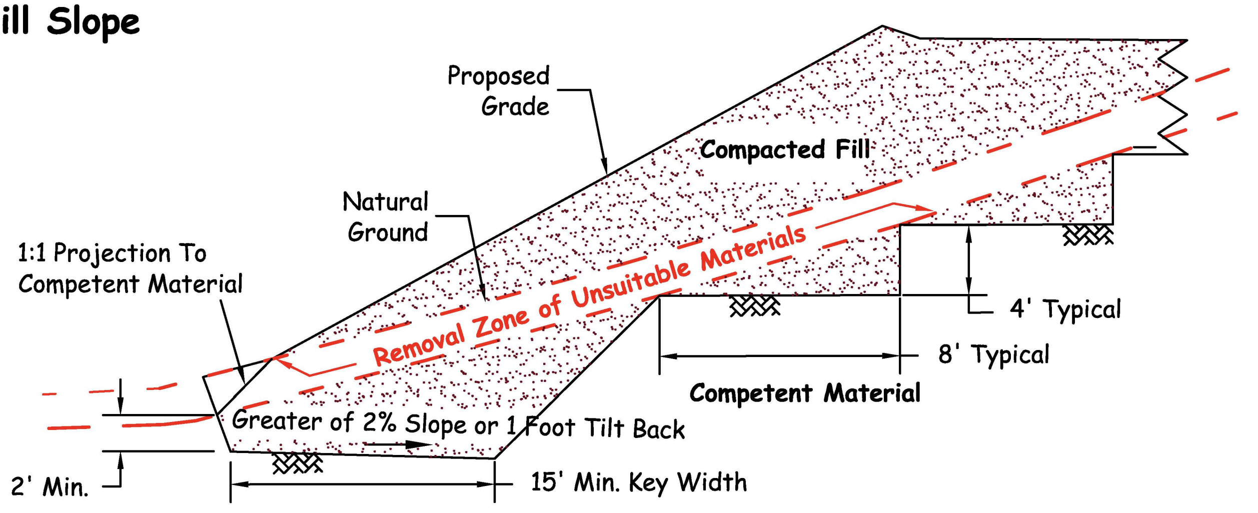
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

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

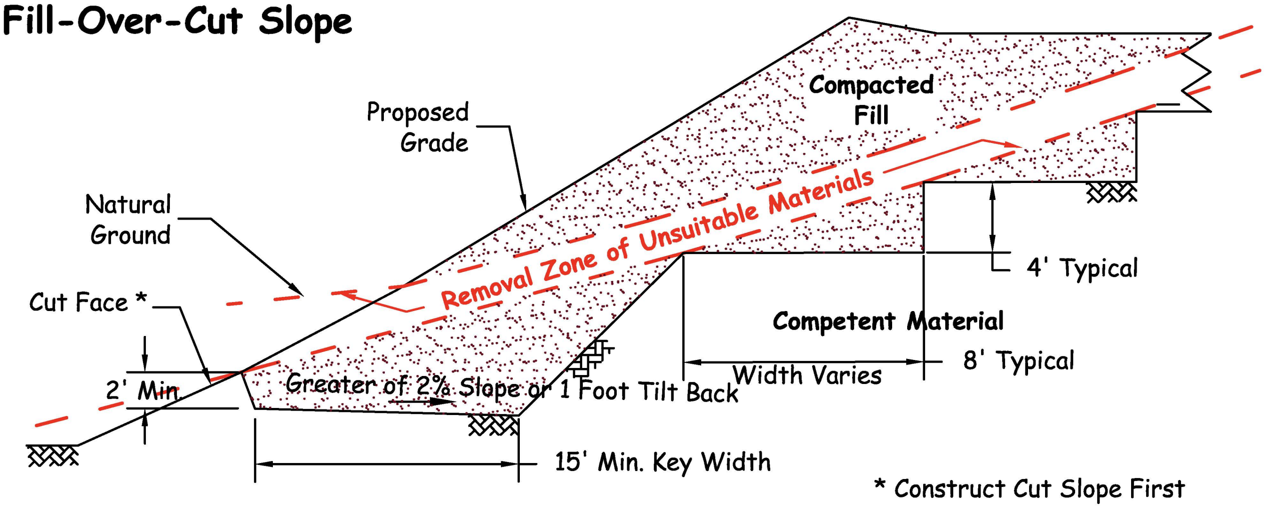
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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

Fill Slope

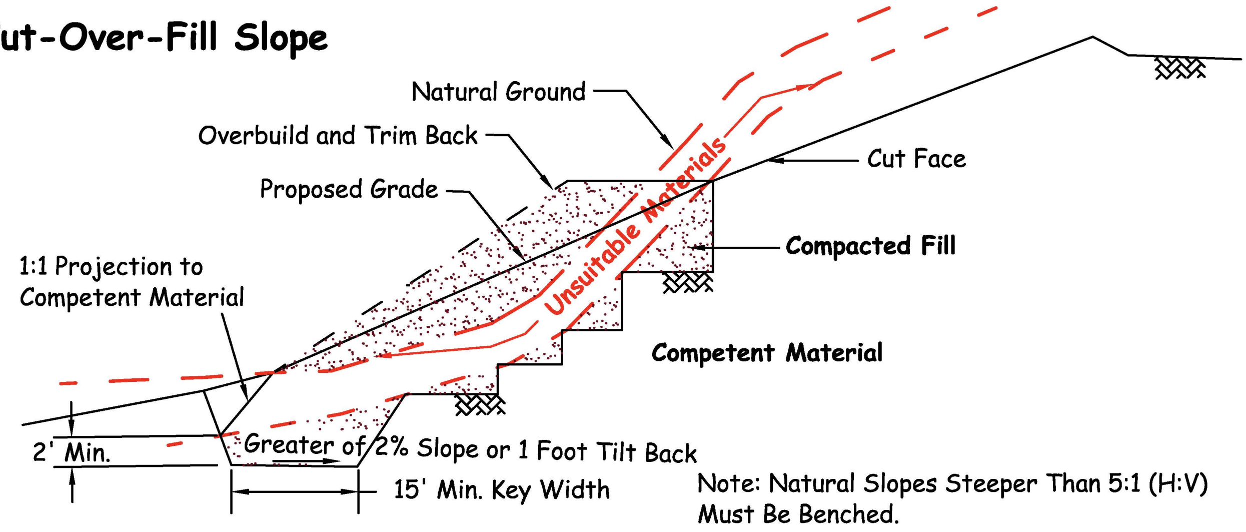


Fill-Over-Cut Slope



* Construct Cut Slope First

Cut-Over-Fill Slope



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

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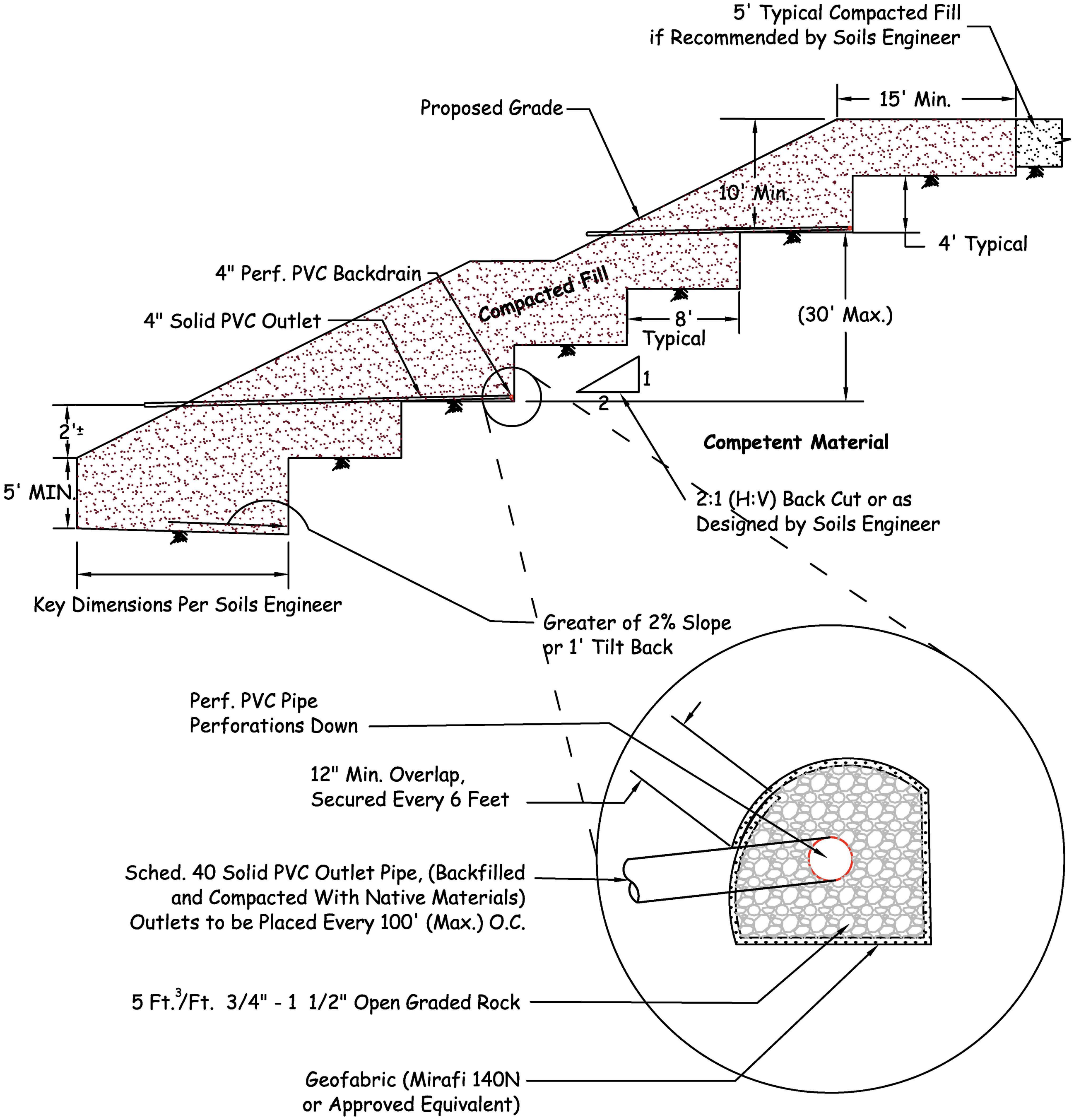
KEYING AND BENCHING



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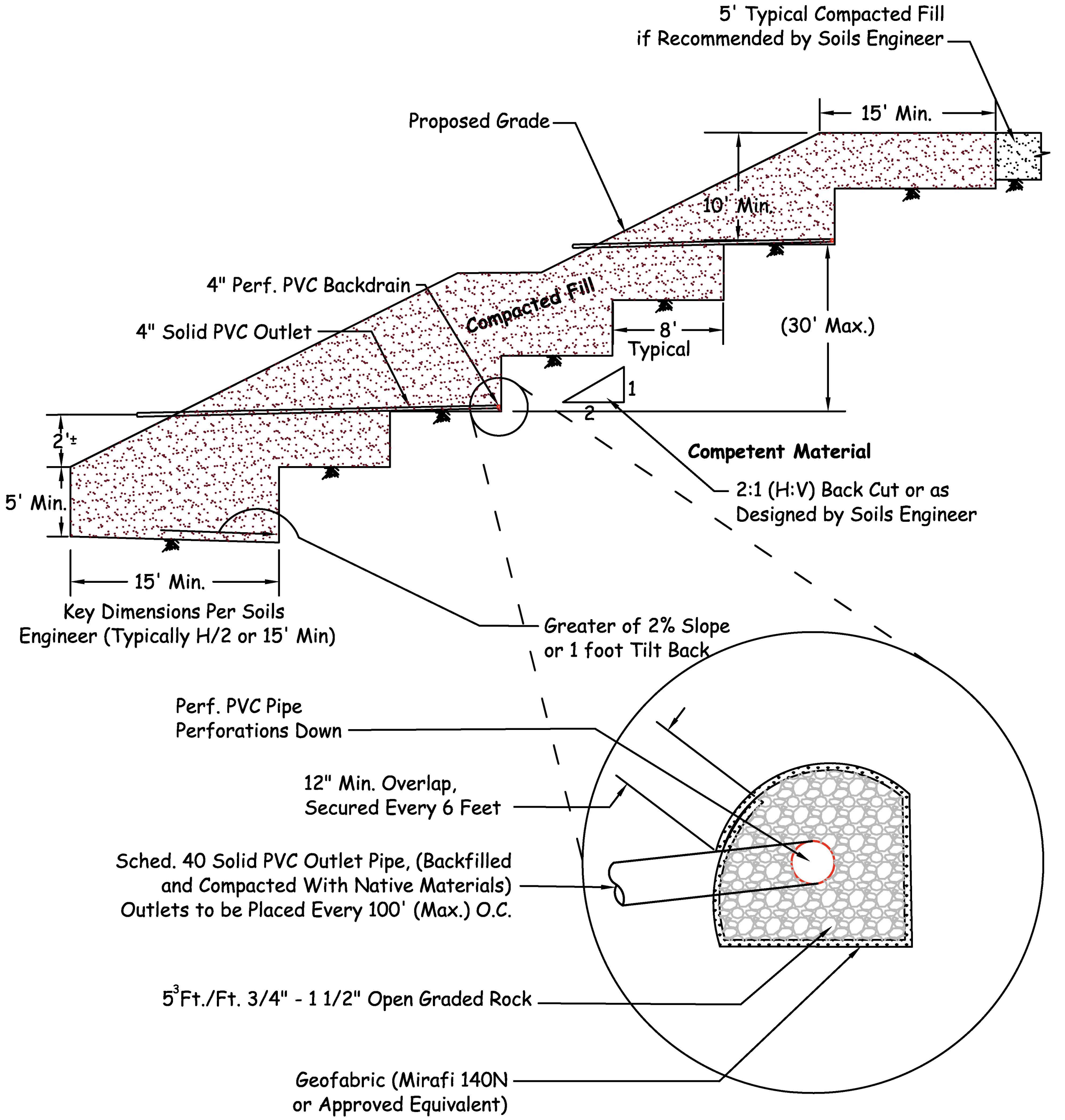
TYPICAL BUTTRESS DETAIL



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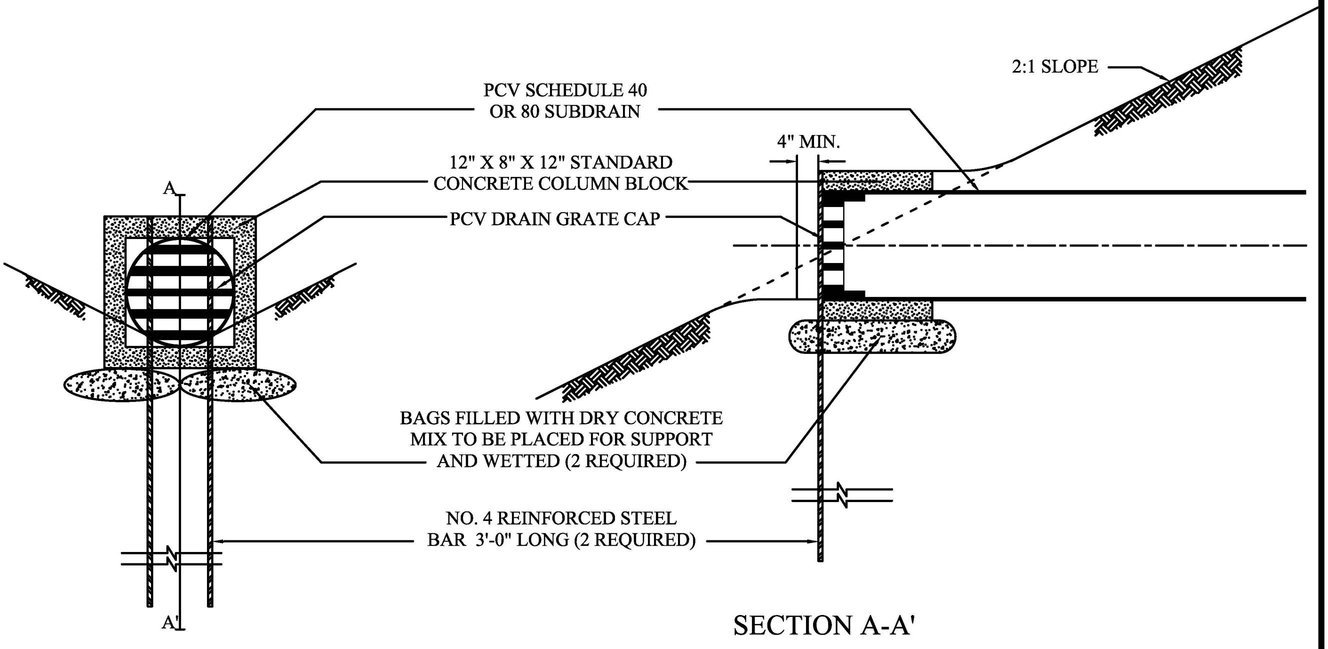
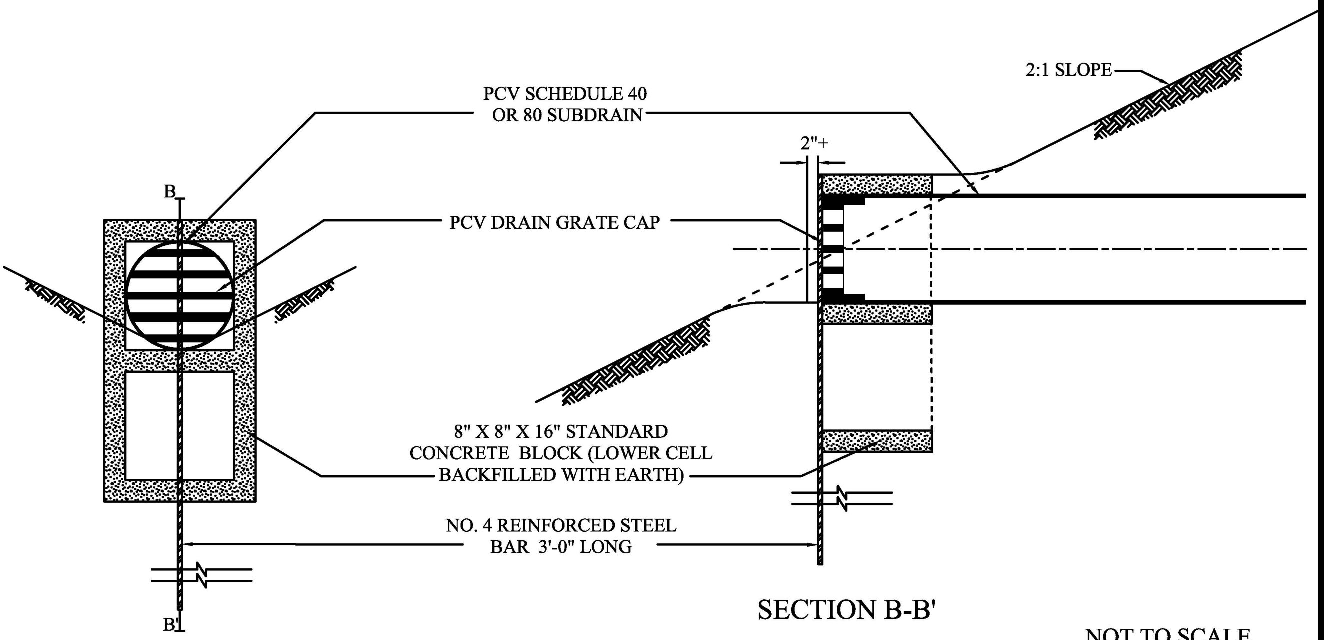
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SUBDRAIN OUTLET MARKER -4" PIPE



NOT TO SCALE

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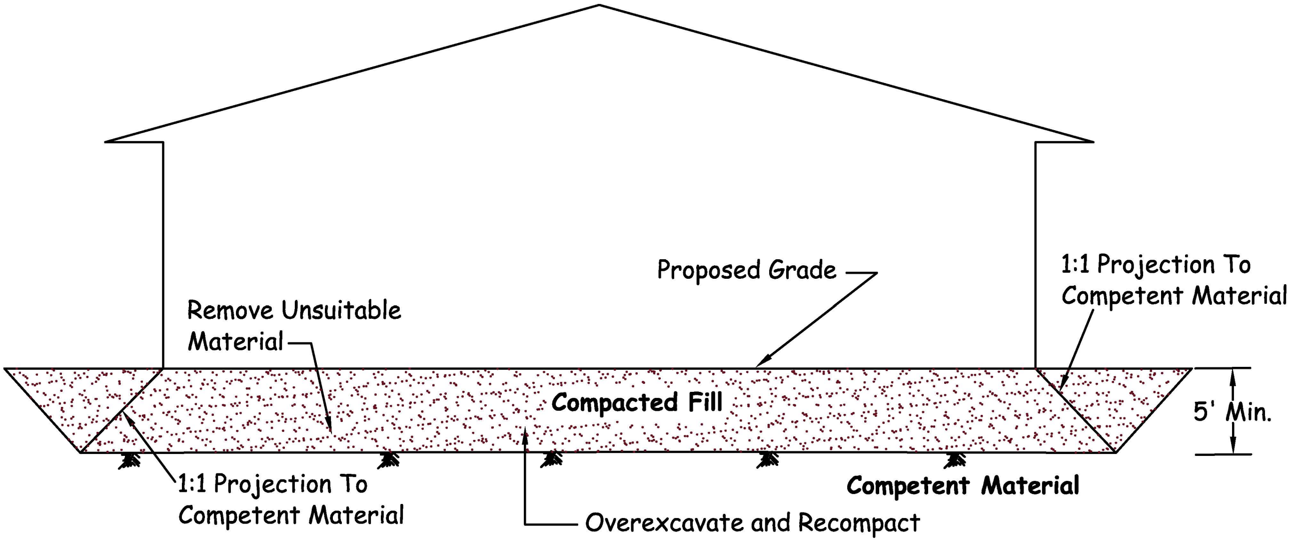
**SUBDRAIN OUTLET
MARKER DETAIL**

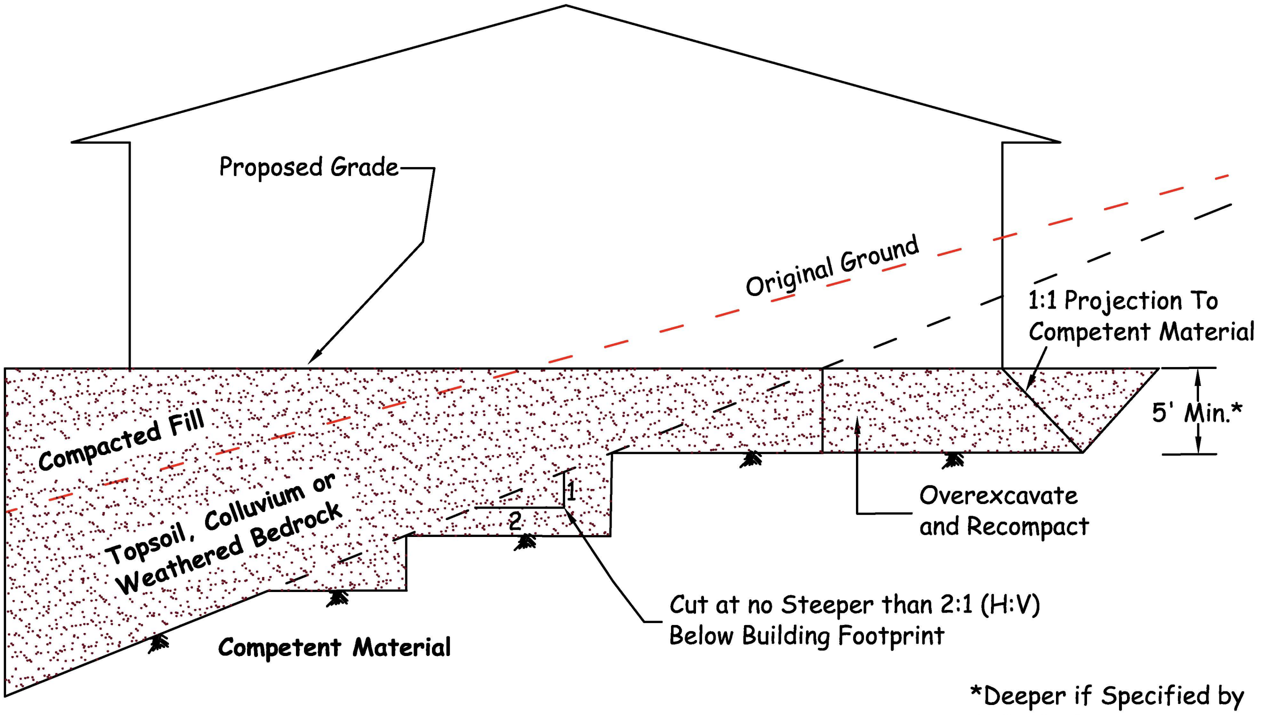
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



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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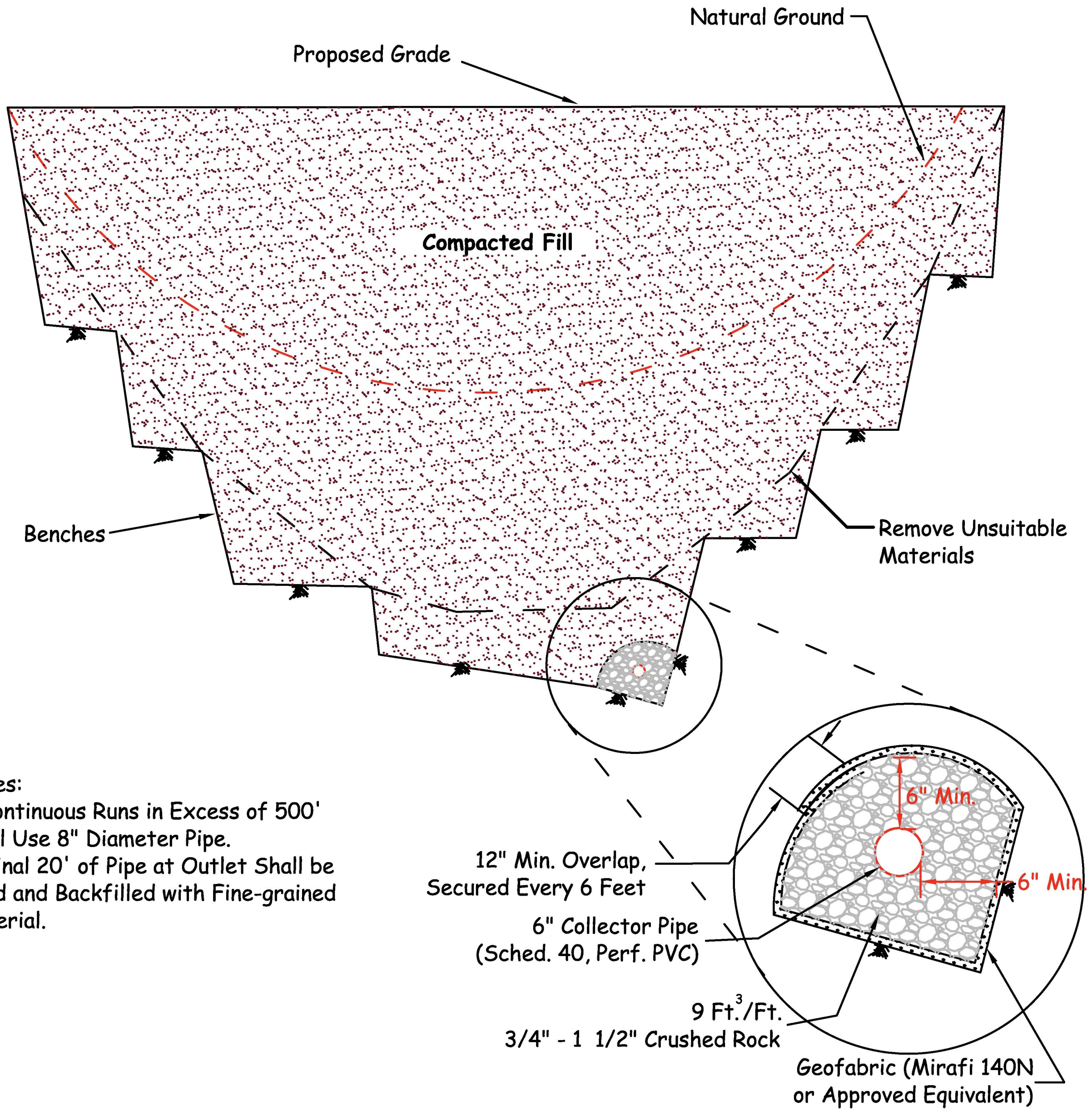
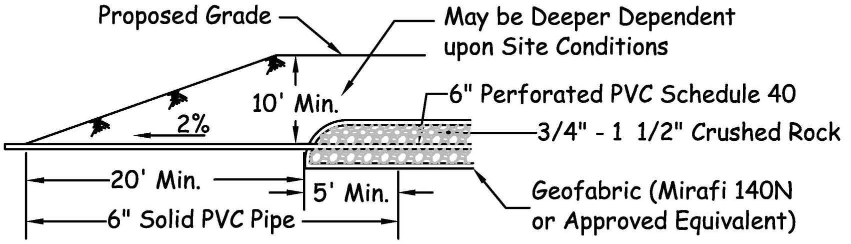
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



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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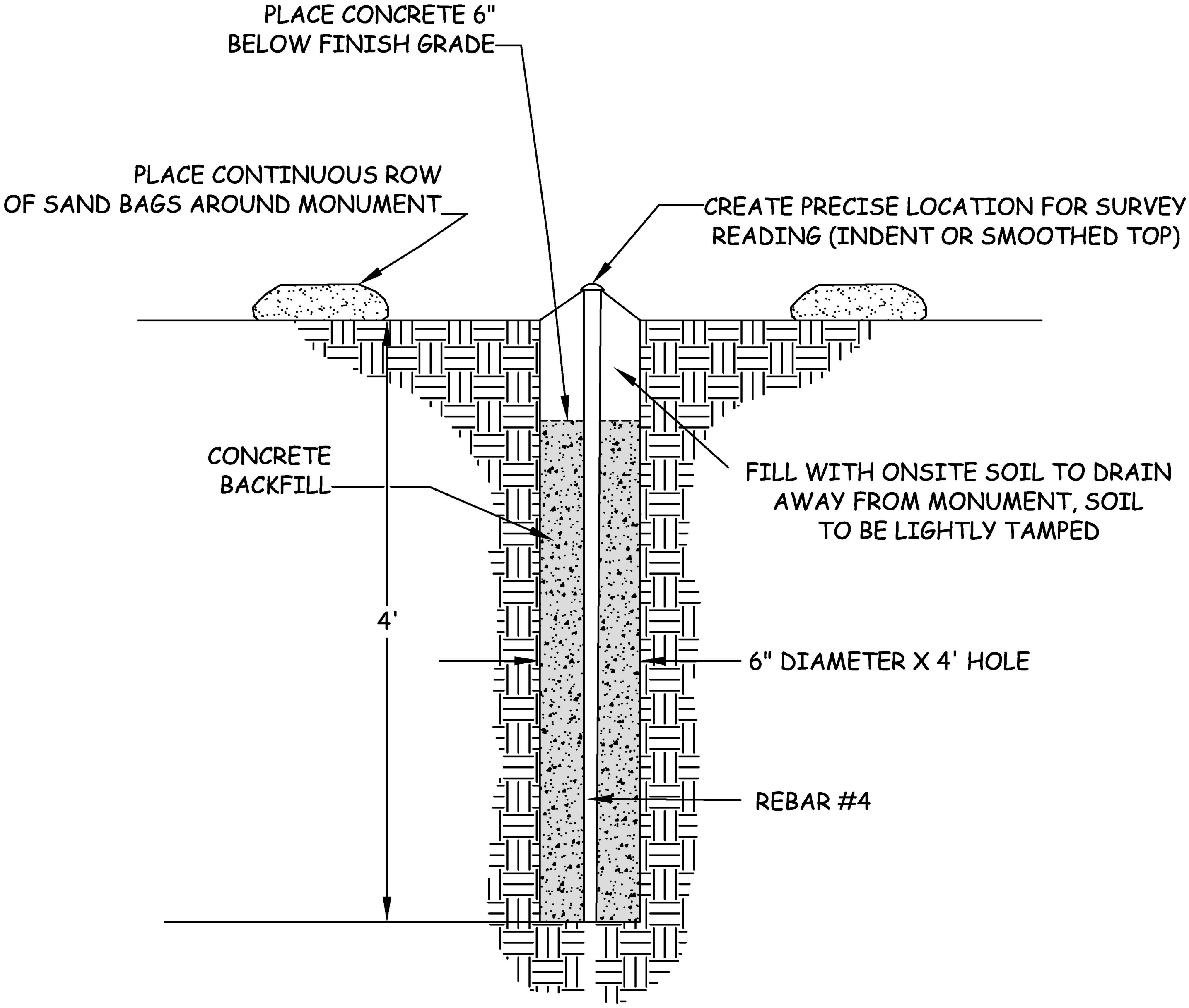
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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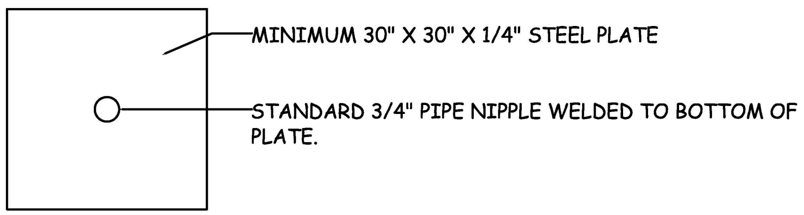
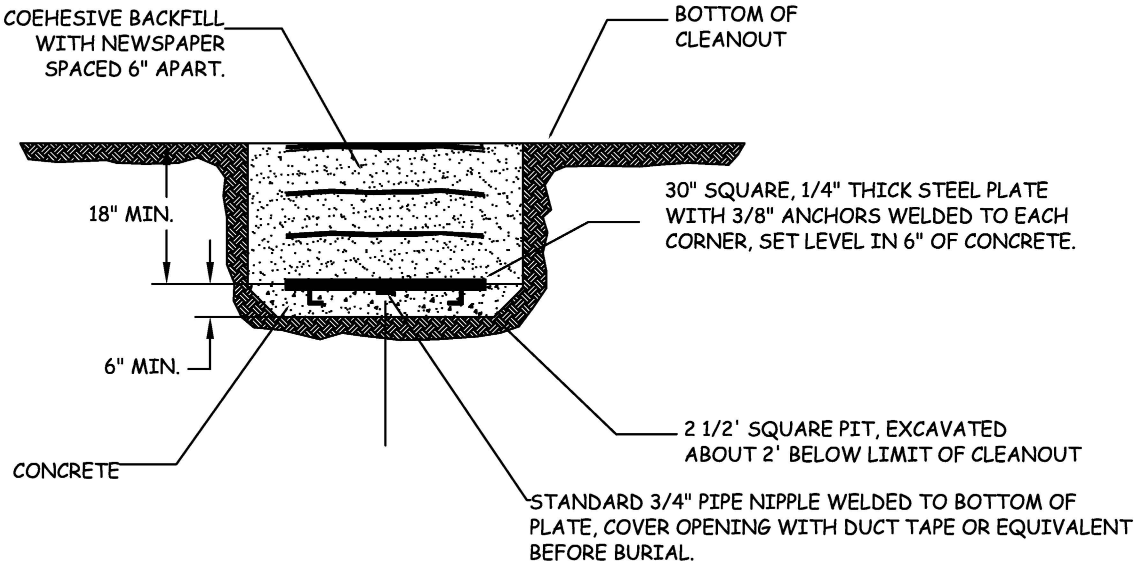
TYPICAL SURFACE SETTLEMENT MONUMENT

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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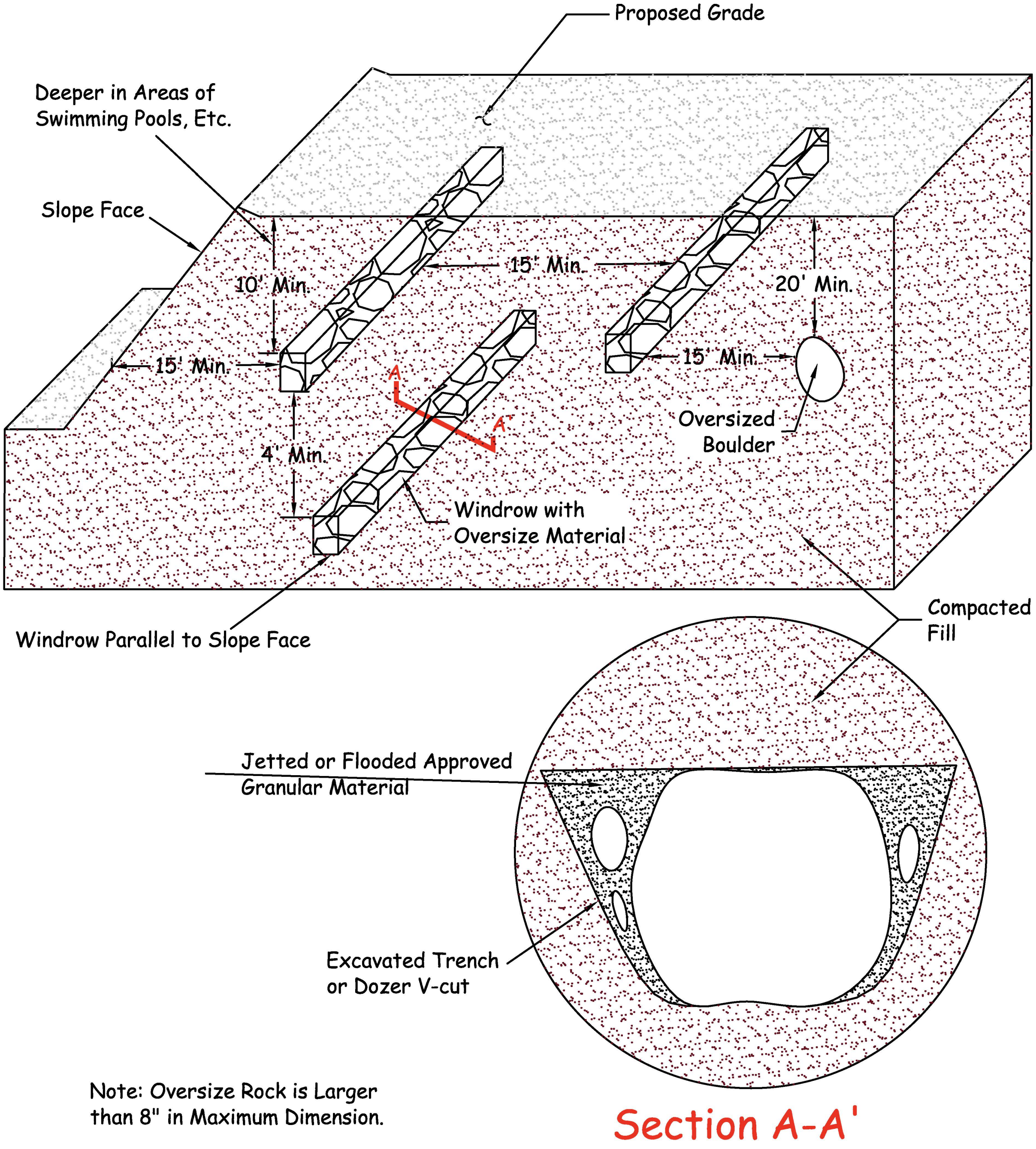
TYPICAL SETTLEMENT PLATE AND RISER



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

Section A-A'

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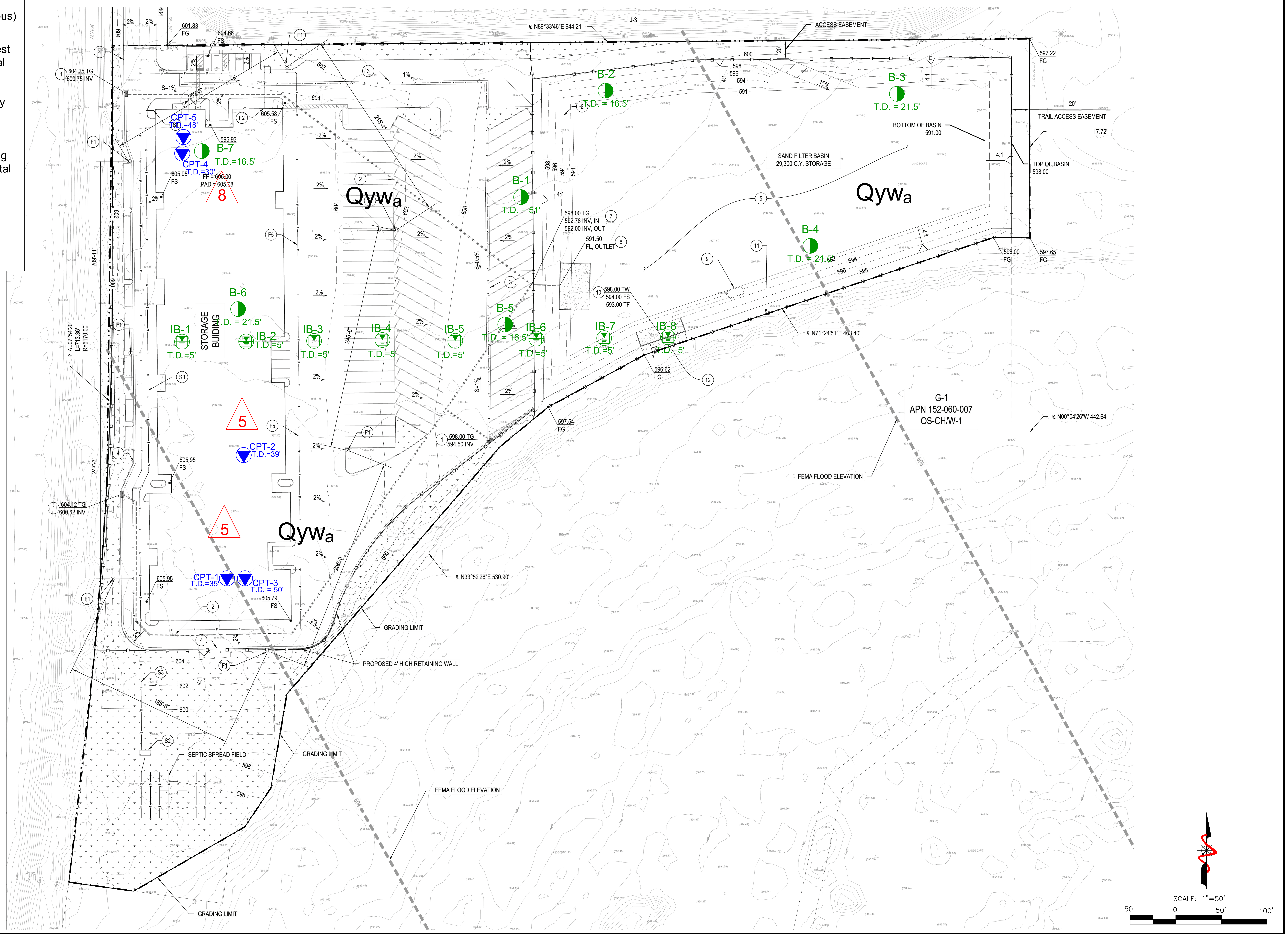
OVERSIZE ROCK DISPOSAL DETAIL

LEGEND

Qyw$_a$	Quarternary Young Wash Deposits (arenaceous)
CPT-5 ▼ T.D. = 48'	Approximate Location of Cone Penetration Test (CPT) Boring by LGC Geotechnical, With Total Depth in Feet
B-7 ◐ T.D. = 16.5'	Approximate Location of Exploratory Boring by Logical Goetechnical Consultants, With Total Depth in Feet, 2020a
IB-8 ⊕ T.D.=5'	Approximate Location of Infiltration Test Boring by Logical Geotechnical Consultants, With Total Depth in Feet, 2020b
—·—·—·—	Approximate Limits of Report
△ 8	Approximate Removal Depth Below Existing Grade, Shown in Feet



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TEL (949) 369-6141 FAX (949) 369-6142

Preliminary Geotechnical Map

PROJECT NAME	Jurupa Valley 18 - Self Storage
PROJECT NO.	23155-01
ENG. / GEOL.	RLD / KBC
SCALE	1" = 50'
DATE	September 2023

SHEET 1 of 1