Appendix H Preliminary Geotechnical Report

LGC Geotechnical, Inc.

June 21, 2022

Project No. 22061-01

Mr. Ryan Aubry
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Culver City, CA 90232

Subject: ***Preliminary Geotechnical Subsurface Evaluation and Recommendations, Proposed Mixed-Use Development, Southeast Corner of Imperial Highway and Norwalk Boulevard, Norwalk, California***

In accordance with your request, LGC Geotechnical, Inc. (LGC Geotechnical) is providing a preliminary geotechnical report for the planned mixed-use development at the southeast corner of Imperial Highway and Norwalk Boulevard in the City of Norwalk, California. This report presents the results of our limited subsurface explorations and geotechnical analysis and provides a summary of our conclusions and preliminary recommendations relative to the proposed site development.

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

Respectfully,

LGC Geotechnical, Inc.

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TABLE OF CONTENTS

LIST OF TABLES, ILLUSTRATIONS, & APPENDICES

Tables

Figures

Appendices

1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This preliminary geotechnical report is for the planned mixed-use development, located at the southeast corner of Imperial Highway and Norwalk Boulevard in the City of Norwalk, California (see Site Location Map, Figure 1). The proposed development will consist of two up to 7-story mixed-use residential/retail structures. The purpose of our work was to evaluate site geotechnical conditions and to provide preliminary geotechnical recommendations with respect to the proposed development.

1.2 Project Description

Based on the provided information, the proposed development will consist of two mixed-use residential/retail structures. The proposed up to 7-story mixed-use residential/retail structures will consist of approximately 350 on-grade multi-family residential units. We anticipate finish grades will only vary slightly (± 2 feet) from current grade in the proposed site and residential/retail structure areas. The proposed development will also include on-grade parking inside the mixed-use buildings and open space areas surrounding the two buildings. Presented below in Table 1 is a summary of the preliminary estimated structural (dead plus live) loads for the proposed two up to 7-story mixed-use residential/retail structures. Since site specific building loads were unavailable, these preliminary column loads were estimated from previous similar projects

TABLE 1

Preliminary Estimated Structural Loads

Planned Structure	Column Loads (kips)	Wall Loads (kip/ft)
7-Story Mixed-Use Structures	600	25

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

An additional two stories and pedestrian bridge may be added to the existing two-story (three level) parking garage in the southeast portion of the site. We did not perform a geotechnical evaluation in this area, so the soil conditions are unknown, but likely similar to the rest of the site. Due to no structural plans being available for the existing parking structure, we are assuming that the parking garage is supported by a shallow foundation system. Due to adding twice the load that the current parking garage is designed to support and being supported by a shallow foundation system (assumed), significant changes to the columns and footings would be required to support the additional load. This potential should be evaluated by the project structural engineer. If plans become available that show that the parking structure is supported by a deep foundation system, then less structural changes may be required. Any further geotechnical comments on the existing parking structure would require old geotechnical reports, foundation plans, and additional subsurface exploration, lab testing, analysis, etc.

LGC Geotechnical, Inc.	**FIGURE 1** **Site Location Map**	PROJECT NAME	Placeworks - Norwalk
		PROJECT NO.	22061-01
		ENG. / GEOL.	DJB
		SCALE	Not to Scale
		DATE	June 2022

1.3 <u>Existing Conditions</u>

The approximately 13.2-acre site is bound to the north by Imperial Highway, to the east by the Avenida Manuel Salinas, to the south by the Los Angeles County Superior Court Norwalk property, and to the west by Norwalk Boulevard. The site currently has an open lawn area (City Hall Lawn), an asphalt surface parking lot, two buildings (Norwalk City Hall and accessory building to the Los Angeles County Courthouse), and a three-level/two-story parking structure.

1.4 <u>Background</u>

Review of historical aerials indicates the site had been used for agricultural farming as far back as 1953. Aerial photos from 1953 through 1963 suggest the site was used as farmland. By 1972, a majority of the site was developed except for the parking garage. Between the years 1995 and 1996, the parking garage was constructed. Since then, only minor improvements have been made on the site.

1.5 <u>Subsurface Exploration</u>

A subsurface exploration of the site was performed by LGC Geotechnical in April of 2022. The exploration program consisted of the excavation of five hollow-stem auger borings and advancing four CPT soundings to characterize subsurface soils and evaluate onsite geotechnical conditions. Additionally, two field infiltration tests were performed in order to evaluate the preliminary infiltration rate of the subsurface soils.

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

Field infiltration testing was performed within borings (I-1 and I-2) at total depths of 5 feet and 10 feet below existing grade, respectively. An LGC Geotechnical staff engineer installed standpipes, backfilled the boring annulus with crushed rock, and pre-soaked the infiltration wells prior to testing. Infiltration testing was performed in accordance with the County of Los Angeles testing guidelines. The infiltration test wells were subsequently backfilled with native soils and capped with cold patch asphalt concrete where necessary at the completion of testing.

Four CPT soundings (CPT-1 through CPT-4) were pushed to depths of approximately 75 feet below existing grade. The CPT soundings were pushed using an electronic cone penetrometer in general accordance with the current ASTM standards (ASTM D5778 and ASTM D3441). The CPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The interior of the cone penetrometer is instrumented with strain gauges that allow the simultaneous measurement of cone tip and friction sleeve resistance during penetration. The cone penetration assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.8 inches per second while the cone tip resistance and sleeve friction resistance are recorded at approximately every 2 inches and stored in digital form. All CPTs were performed by Kehoe Testing and Engineering, Inc. using a 25-ton CPT rig. At the completion of pushing, the CPT soundings were backfilled with bentonite chips from the surface and capped with cold patch asphalt concrete. Some settlement of the backfill soils may occur over time.

The approximate locations of our borings, percolation test wells, and CPT's are provided on Figure 2 – Boring Location Map and the associated boring logs are provided in Appendix B.

1.6 __Laboratory Testing__

Representative driven and bulk samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ unit weight and moisture content, fines content, Atterberg limits, consolidation, expansion index, laboratory compaction and corrosion (sulfate, chloride, pH, and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 86 pounds per cubic foot (pcf) to 124 pcf, with an average of 103 pcf. Field moisture contents ranged from approximately 1 to 32 percent, with an average of approximately 11 percent.
- Four fines content tests indicated a fines content (percent passing No. 200 sieve) ranging from approximately 8 percent to 68 percent. Based on the Unified Soils Classification System (USCS).
- Three Atterberg Limit (liquid limit and plastic limit) tests were performed. Results indicated a Plasticity Index value ranging from 4 to 13 with one sample being non-plastic.
- Three consolidation test were performed. The stress vs. deformation plot is provided in Appendix C.
- Two Expansion Index (EI) tests were performed. Results were EIs of 0 and 25, corresponding to "Very Low and Low" expansion potential.
- Laboratory compaction (maximum dry density and optimum moisture content) test indicated a maximum dry density value of 131.0 pcf with optimum moisture content of 8.5 percent.
- Corrosion testing indicated soluble sulfate content of approximately 0.012 percent, chloride

content of approximately 40 parts per million (ppm), pH value of approximately 8.40, and minimum resistivity of approximately 1,940 ohm-cm.

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

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The site is located within a coastal sedimentary basin called the Downey Plain, within the Peninsular Ranges Geomorphic Province. The site is located on a laterally extensive young alluvial fan deposits interpreted to be approximately Holocene and late Pleistocene age (CGS, 2016). The sediments are primarily derived from the Rio Hondo and San Gabriel River drainages that run south from the San Gabriel Valley through the northwest trending Puente Hills an area called the Whittier Narrows. The subject site is located about five miles south of the Puente Hills and Whittier Narrows, and about two miles east of the San Gabriel River Channel. The Puente Hills were uplifted and deformed along the Whittier Fault, a section of the Elsinore Fault Zone. The region has a complex geologic history influenced by periods of uplift, folding, faulting, and alluvial deposition; however, no faults are known to transect the site.

2.2 Generalized Subsurface Soils

Based on our subsurface evaluation, the site contains up to approximately 7.5 feet of previously placed undocumented artificial fill over Quaternary Alluvial deposits. Older artificial fill soils encountered were silty sands to sandy silts. Alluvial deposits, where encountered, are primarily medium dense to very dense sands with varying amounts of fine-grained soils to medium stiff to very stiff sandy clays and silts, to the maximum explored depth of approximately 50 feet below exiting grade.

It should be noted that geotechnical explorations are only representative of the location where they are performed, and varying subsurface conditions may exist outside of each 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 the boring logs provided in Appendix B.

2.3 Groundwater

Groundwater was not encountered in our borings to the maximum explored depth of approximately 75 feet below existing grade. Historic high groundwater is estimated to be about 10 feet or greater below existing grade (CDMG, 1998).

It should be noted that higher localized and seasonal perched groundwater conditions may accumulate below the surface and should be expected throughout the design life of the proposed improvements. In general, groundwater conditions below any given site may vary over time depending on numerous factors including seasonal rainfall and local irrigation among others.

2.4 Field Infiltration Testing

Two shallow infiltration tests were performed in Borings I-1 and I-2 to approximate depths of 5 and 10 feet below existing grade, respectively. The approximate locations are shown on the Boring Location Map (Figure 2). The borings for the infiltration tests were excavated using a drill rig equipped with an 8-inch diameter hollow-stem auger. Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Los Angeles (2021). A 3-inch diameter perforated PVC pipe was placed in the borehole above a thin layer of gravel and the annulus was backfilled with gravel. The infiltration wells were pre-soaked 1 hour prior to testing. Initially the procedure for 30-minute reading intervals was followed for the borings (I-1 and I-2). For I-1 during the 30-minute period, water remained in the boring after 30 minutes. Therefore, the test procedure utilizing a thirty-minute reading interval was performed. For I-2 during the 30-minute test, water was generally draining to the top of the gravel layer in less than 30 minutes; therefore, readings were taken at a time interval based on the amount of time it took for the water to drain through the infiltration boring. Readings were taken for a minimum of 3 hours or until a “stabilized rate” was established. A “stabilized rate” is when the highest and lowest readings are within 10 percent of each other over three consecutive readings. At the completion of infiltration testing, the pipe was removed, backfilled with cuttings, tamped, and the asphalt was patched in the necessary areas. Some settlement of the backfill should be expected.

Based on the County of Los Angeles testing guidelines, the raw infiltration is calculated by dividing the volume of water discharged by the surface area of the test section (including sidewalls plus the bottom of the boring), in a given amount of time. The measured infiltration rates are provided in Table 2 below. Please note that the values provided in Table 2 do not include reduction factors for the test procedure, site variability and long-term siltation plugging that are required for the design infiltration rate, refer to Table 8 in Section 4.8. Infiltration tests were performed using relatively clean water free of particulates, silt, etc. Refer to the infiltration test data provided in Appendix D.

TABLE 2

Summary of Field Infiltration Testing

Infiltration Test Location	Infiltration Test Depth (ft)	Measured Infiltration Rate* (inch/hr.)
I-1	5	1.8
I-2**	10	6.4

*Does Not Include Required Reduction Factors, refer to Table 8, Section 4.8.

**It is our opinion that the measured infiltration rate is very high given the onsite soil characteristics. Therefore, only the infiltration rate of I-1 should be used in design.

2.5 Seismic Design Parameters

Since the site contains soils that are susceptible to liquefaction (refer to below 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 7/2019 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.

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. **Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16.** Representative site coordinates of latitude 33.916903 degrees north and longitude -118.070160 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 3. 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 3

Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.654g	From SEAOC, 2022
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.592g	From SEAOC, 2022
F_a (per Table 1613.2.3(1))	1.000	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))	1.708	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.654g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	1.011g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.103g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.674g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.907	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.902	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.84 at a distance of approximately 10.89 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 6.71 at a distance of approximately 16.13 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2019 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.780g (SEAOC, 2022).

2.6 Faulting

The subject site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CGS, 2018). A fault is considered "active" if evidence of surface rupture in Holocene time (the last approximately 11,700 years) is present.

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 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 loose, saturated, 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 (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

The site is located within a State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1999). Subsurface field data indicates that the site contains isolated sandy layers susceptible to liquefaction interfingered with fine-grained non-liquefiable soils and dense sands. The recent explored groundwater elevation of more than 75 feet below existing grade and historic high groundwater elevation of 10 feet below existing grade were used in the liquefaction analysis. The liquefaction evaluation was performed using CPT data and the CLiq program (Geologismiki, 2017). Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and the applicable seismic criteria (e.g., 2019 CBC). Liquefaction induced settlement was estimated using the PGA_M per the 2019 CBC and a moment of magnitude of 6.84 (USGS, 2014).

Results indicate total seismic settlement on the order of approximately 1 inch (Appendix E). Differential seismic settlement can be estimated as half of the total estimated settlement over a horizontal span of about 40 feet. Interconnecting isolated structural pad footings with grade beams may be prudent, given dynamic settlement potential depending on the structure's tolerance to settlement.

2.6.2 Lateral Spreading

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

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

2.7 Expansion Potential

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

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and understanding of the proposed development, it is our opinion that the proposed development is feasible from a geotechnical standpoint. A summary of our conclusions are as follows:

- Based on our subsurface evaluation (hollow-stem auger borings and CPT's) the site is estimated to contain a thin veneer up to approximately 7.5 feet of previously placed undocumented artificial fill over native alluvial fan deposits. Older artificial fill soils encountered were primarily silty sand and sandy silt. Alluvial deposits, where encountered, are primarily medium dense to very dense sands with varying amounts of fine-grained soils to medium stiff to very stiff sandy clays and silts, to the maximum explored depth of approximately 50 feet below exiting grade.
- The near-surface soils are generally loose and compressible and are not suitable for the planned improvements in their present condition (refer to Section 4.1); temporary removal and recompaction will be required.
- Groundwater was not encountered during our recent subsurface evaluation to the maximum explored depth of approximately 75 feet below existing ground surface. Historic high groundwater for the site is about 10 feet or greater below existing ground surface (CDMG, 1998).
- The site is located within a State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1999). Subsurface data indicates that isolated layers are susceptible to liquefaction and dynamic settlement. We estimate total seismic settlement due to liquefaction potential to be approximately 1 inch. Differential seismic settlement may be estimated as half of the total estimated settlement over a horizontal span of about 40 feet.
- The proposed development will likely be subjected to strong seismic ground shaking during its design life. The site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CGS, 2018).
- Provided our earthwork removal and recompction recommendations are implemented (refer to Section 4.1), the proposed two up to 7-story mixed-use buildings may be supported on a shallow foundation system. Preliminary long-term static settlement estimates based on the estimated building loads are presented in Section 4.3.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have "Very Low" to "Low" expansion potential. Final design expansion potential must be determined at the completion of grading. Recommendations are required for foundations and site improvements like concrete flatwork to minimize the impacts of expansive site soils. Final design expansion potential must be determined at the completion of grading.
- Based on the corrosion test results, soils are not considered corrosive per the Caltrans criteria (Caltrans, 2021).
- From a geotechnical perspective, the existing onsite soils are suitable material for use as general fill (not retaining wall backfill), provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- The site contains some soils that are not suitable for retaining wall backfill due to their fines content and expansion potential, therefore import of sandy soils may be required by the contractor for obtaining suitable backfill soil for planned site retaining walls.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

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

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

4.1 Site Earthwork

We anticipate that earthwork will generally consist of demolition of existing improvements, required temporary removal and recompaction of near surface soils, subgrade preparation, and construction of the proposed new improvements including the mixed-use structures, site amenities, courtyards, subsurface utilities, driveways, etc.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future geotechnical reports, the 2019 CBC/City of Norwalk requirements and the General Earthwork and Grading Specifications included in Appendix F. In case of conflict, the following recommendations shall supersede those included in Appendix F. The following recommendations should be considered preliminary and may be revised based upon future evaluation and our review of updated project plans and/or the field conditions exposed during site grading/construction.

4.1.1 Clearing and Grubbing

Prior to earthwork of areas to receive structural fill, engineered structures or improvements, the areas should be cleared of existing vegetation, building structures, pavement, utilities, surface obstructions, existing debris and potentially compressible or otherwise unsuitable material. Vegetation (including grass and topsoil) and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed removal bottoms, should be replaced with properly compacted fill material. Any abandoned sewer, storm drain, or utility lines should be completely removed and replaced with properly placed compacted fill. Deeper demolition may be required in order to remove existing foundations or utilities. We recommend the trenches associated with demolition which extend below the remedial grading depth be backfilled and properly compacted prior to the demolition contractor leaving the site.

If cesspools or septic systems are encountered during earthwork, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further earthwork.

4.1.2 Removal and Recompaction Depths and Limits

In order to provide a relatively uniform bearing condition for the planned building structure and improvements, we recommend the site soils be removed and recompacted. Compressible near surface soils shall be removed to suitable, competent native materials prior to re-placement as compacted fill to design grades. Subsurface site soils should be removed and recompacted according to the criteria outlined below. Updated recommendations may be required based on additional field evaluations, changes to building layouts and actual structural loads.

7-Story Mixed-Use Structure: We recommend that soils within the proposed mixed-use structure footprint areas be removed and recompacted to a minimum depth of 8 to 10 feet below existing grade or 5 feet beneath the base of the foundations, whichever is deeper. Localized deeper removal and recompaction may be required.

The base of removal bottoms should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade. Specifically, soils located within a 1:1 (horizontal to vertical) projection of the bottom of footings must be engineered compacted fill or competent natural ground. Building lines may be defined as the perimeter of the building proper, plus attached or adjacent foundation supported features, including canopies, elevators, walls, etc.

For minor site structures, such as free-standing, minor retaining walls, etc., removal and recompaction should extend at least 5 feet beneath existing grade or 2 feet beneath the base of foundations, whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 5 feet beyond the edges of the proposed improvements mentioned above.

Within non-structural areas (i.e., areas designed to receive concrete/asphalt paving, pavers, flatwork, etc.), the soils should be removed and replaced as properly compacted fill to a minimum depth of 3 feet below existing grade or 1-foot below the proposed finished subgrade, whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements mentioned above.

Local conditions may be encountered which could require additional removal and recompaction beyond the above-noted minimum to obtain an acceptable subgrade. The actual depths and lateral extents of removal and recompaction should be determined by the geotechnical consultant based on the subsurface conditions encountered during grading. Removal and recompaction areas and 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 investigation, the majority of site soils are anticipated to be OSHA Type "C" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Surcharge loads (vehicular traffic, soil stockpiles, construction equipment, etc.) should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 (horizontal to vertical) projection from the bottom of the excavation or 5 feet, whichever is greater, unless the cut is properly shored and designed for the applicable surcharge load. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

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

4.1.4 Removal Bottoms and Subgrade Preparation

In general, removal bottoms and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project recommendations.

Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement. Soil subgrade for planned footings and improvements (e.g., slabs, etc.) should be firm and competent.

4.1.5 Material for Fill

From a geotechnical perspective, the onsite soils are generally suitable for use as general compacted fill, provided they are screened of oversized material (8 inches in greatest dimension), construction debris and significant organic materials.

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of soils of granular soils of “Low” expansion potential (expansion index 50 or less based on American Society for Testing and Materials [ASTM] D 4829), and free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.

Retaining wall backfill should consist of onsite clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) D1140 (or ASTM D6913/ASTM 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. Some of the onsite soils should be suitable for retaining wall. Therefore, import or select grading, screening and stockpiling of onsite soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. These preliminary findings should be confirmed during grading.

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 (“Green Book”) for untreated base materials (except processed miscellaneous base), Caltrans Class 2 aggregate base or the City of Norwalk requirements.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 2-3-inch in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, brick, etc.) and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned building structure footprint (i.e., not within building pad areas).

4.1.6 Placement and Compaction of Fills

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning (adding water) of site soils will be required in order to achieve adequate compaction prior to reusing the materials in compacted fills. Additionally, drying and/or mixing very moist soils may also be required prior to reusing the materials a compacted fill.

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 the local grading ordinances with observation and testing performed by the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

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

Aggregate base material should be compacted to 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 near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content).

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

4.1.7 Trench and Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks, construction debris, other material greater than 6 inches in diameter and significant organic matter. If trenches are shallow or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 or greater (per Caltrans Test Method [CTM] 217) may be used to bed and shade the pipes within the bedding zone. Based on our field evaluation, the majority of the onsite soils may not meet this sand equivalent requirement. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform lifts (as outlined above in section "Material for Fill") by mechanical means to at least 90 percent relative compaction (per ASTM D1557).

Utility trenches running parallel to footings should not be excavated within a 1:1 (horizontal to vertical) downward projection from adjacent footings ("footing influence zone") to avoid potential undermining. Depending on the utility line and structural loading of the footing, utility trenches running perpendicular to footings may require special provisions such as sand-cement slurry backfill of the utility trench in this zone or flexible sleeves through the footings. These conditions should be evaluated on a case-by-case basis.

Retaining wall backfill should consist of predominately granular, sandy soils as outlined in above Section 4.1.5. For conventional retaining walls, the limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater, refer to Figure 3. Retaining wall backfill soils should be compacted in relatively uniform thin lifts to a minimum of 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

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

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

4.1.8 Shrinkage and Subsidence

Allowance in the earthwork volumes budget should be made for an estimated 0 to 15 percent reduction in volume of the upper approximate 8 to 10 feet of site 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 equipment is expected to be on the order of 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. In addition, additional shrinkage may occur due to export of site oversize material. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor and accuracy of the topographic survey.

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

4.2 Preliminary Foundation Design Parameters

Provided our earthwork removal and recompaction recommendations are implemented, the proposed up to 7-story mixed-use building may be supported on a shallow foundation system. Preliminary foundation recommendations are provided below. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork.

4.2.1 Building Slabs

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

- Minimum Footing Depth: 12 inches below lowest adjacent grade.
- Minimum Slab Thickness: 5 inches (Structural conditions may govern)
- Minimum Slab Reinforcement: No. 3 bars at maximum 18-inches on-center each way (Structural conditions may govern)
- Moisture-condition (maintain) slab subgrade to near optimum moisture content to a minimum depth of 12 inches prior to pouring slab.

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

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

4.3 Allowable Bearing Pressures and Passive Resistance

The following minimum footing widths and embedments recommended for the corresponding allowable bearing pressures for both continuous wall and column spread footings are presented in Table 4 on the following page.

TABLE 4

Allowable Soil Bearing Pressures

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

* Refers to minimum depth measured below lowest adjacent grade.

These net bearing pressures (exclusive of the weight of the footings) are for dead plus live loads and may be increased one-third for short-term, transient, wind and seismic loading. The allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values. For any bearing pressures, less than 2,000 psf, a minimum footing width of 18 inches and depth of 12 inches below lowest adjacent grade should be used.

Soil settlement is a function of footing dimensions and applied soil bearing pressure. In utilizing the above-mentioned allowable bearing capacity and recommended earthwork removals, total and differential foundation settlement due to preliminary estimated structural loads for the up to 7-story mixed-use structure are presented in Table 5 below. Differential settlement should be anticipated between nearby columns or walls where a large differential loading condition exists. Static settlement is anticipated to occur relatively quickly after construction. Final settlement estimates should be evaluated by LGC Geotechnical when foundation plans and finalized building loads are made available.

TABLE 5

Preliminary Estimated Static Settlement due to Structural Loads

Planned Structure	**Total Settlement**	**Differential Settlement**
Up to 7-Story Mixed-Use Structure	<1 ½ inch	<¾ inch over 40 feet

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.25 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 200 pcf to a maximum of 2,000 pcf may be used for lateral resistance for properly compacted fill and suitable dense native soils. This allowable passive pressure may be increased to 270 pcf to a maximum of 2,700 pcf for short-duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. The provided allowable passive pressure includes a static and seismic factor of safety of 1.5 and 1.1, respectively.

4.4 Lateral Earth Pressures for Retaining Wall Design

The following may be used for design of site retaining walls. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented in Table 6 below, for approved import or onsite free draining, clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). The site soils are not suitable for retaining wall backfill due to their fines content and expansion index; therefore, import of soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. The wall designer should clearly indicate on the retaining wall plans the required select sandy soil backfill criteria. These preliminary findings should be confirmed during grading.

TABLE 6

Lateral Earth Pressures – On-Site or Approved Imported Sandy Soils

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

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near-surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care should be taken to maintain these drains. Typical conventional retaining wall drainage is shown on Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Waterproofing and outlet systems are not the purview of 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 wall. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. Uniform lateral surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.45 and 0.3 may be used for at-rest and active conditions, respectively. The retaining wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

If retaining walls 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 Soil Corrosivity

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

Corrosion testing indicated a soluble sulfate content of approximately 0.012 percent, chloride content of approximately 40 parts per million (ppm), pH values of approximately 8.40, and minimum resistivity of 1,940 ohm-cm. 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, soils are not considered corrosive using Caltrans criteria.

Based on laboratory sulfate test results, the near surface soils are designated to a class “S0” per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the “S0” sulfate classification.

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

4.6 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as sidewalks, patios/entryways 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 in Table 7 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 7

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

	Sidewalks	**Flatwork**	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 (nominal)	4 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	Wet down prior to placing	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	City/Agency Standard

4.7 Control of Surface Water and Drainage Control

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed structures be sloped away from the proposed structure and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building, geometry necessitates that the 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 Subsurface Water Infiltration

It should be noted that intentionally infiltrating storm water conflicts with the geotechnical engineering objective of directing surface water away from structures and 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. Distress in the form of movement of foundations and other improvements could occur as a result of soil saturation and loss of soil support of foundations and pavements, settlement, collapse, internal soil erosion, and/or expansion. Additionally, off-site properties and improvements may be subjected to seepage, springs, instability, movements of foundations or other impacts as a result of water infiltration and migration. Infiltrated water may enter underground utility pipe zones or other highly permeable layers and migrate laterally along these layers, potentially impacting other improvements located far away from the point of infiltration. Any proposed infiltration system should not be located near slopes or settlement sensitive existing/proposed improvements in order to reduce the potential for slope failures and geotechnical distress issues related to infiltration.

If water must be infiltrated due to regulatory requirements, we recommend the absolute minimum amount of water be infiltrated and that the infiltration areas not be located near settlement-sensitive existing/proposed improvements, basement/retaining walls, or any slopes. As with all systems that are designed to concentrate surface flow and direct the water into the subsurface soils, some minor settlement, nuisance type localized saturation and/or other water related issues should be expected. Due to variability in geologic and hydraulic conductivity characteristics, these effects may be experienced at the onsite location and/or potentially at other locations beyond the physical limits of the subject site. Infiltrated water may enter underground utility pipe zones or flow along heterogeneous soil layers or geologic structure and migrate laterally impacting other improvements which may be located far away or at an elevation much lower than the infiltration source. Recommendations for subsurface water infiltration are provided below.

The design infiltration rate is determined by dividing the measured infiltration rate by a series of reduction factors including; test procedure (RF_t), site variability (RF_v) and long-term siltation plugging and maintenance (RF_s). Based on the Los Angeles County testing guidelines (2021), the reduction factor for long-term siltation plugging and maintenance (RF_s) is the purview of the infiltration system designer. The test procedure reduction factor and recommended site variability reduction factor applied to the measured infiltration rate is provided in Table 8 on the following page. The design infiltration rate is the measured infiltration rate divided by the total reduction factor ($RF_t + RF_v + RF_s$).

TABLE 8

Reduction Factors Applied to Measured Infiltration Rate

Consideration	Reduction Factor
Test procedure, boring percolation, RF_t	1.0
Site variability, number of tests, etc., RF_v	2.0
Long-term siltation plugging and maintenance, RF_s	Per Infiltration Designer
Total Reduction Factor, $RF = RF_t + RF_v + RF_s$	**TBD**

Per the requirements of the Los Angeles County testing guidelines (2021), subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. The test procedure and site variability considerations (RF_t and RF_v) result in a minimum reduction factor of 3.0 (not including long-term siltation plugging and maintenance). When the Total Reduction Factor (to be determined) is applied to the measured infiltration rate of infiltration test I-1, the resulting design infiltration rate should be equal to or greater than the minimum infiltration rate required by the County of Los Angeles for infiltration. Therefore, considering the results of the infiltration testing, if required, stormwater may be infiltrated into the subsurface soils at a depth greater than 10 feet below existing grade (below the fill), using the values presented in Table 1 and Reduction Factors presented above in Table 8. Results of field infiltration testing are provided in Appendix D.

The following should be considered for design of any required infiltration system:

- Water discharge from any infiltration systems should not occur within the zone of influence of foundation footings (column and load bearing wall locations). For preliminary purposes we recommend a minimum setback of 20 feet from the structural improvements.
- Given the presence of fine-grained soils in the upper 30 feet, we recommend 35-foot-deep boreholes (8” diameter) be excavated periodically along the length of the infiltration basin and backfilled with pea-gravel or coarse-grained sand to bottom of the planned infiltration basin. This will facilitate infiltration into deeper sandy layers.
- An adequate setback distance between any infiltration facility and adjacent private property should be maintained.
- The water quality infiltration system should be designed with an overflow system directly connected to the storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration and/or very high flow volumes.
- The infiltration values provided are based on clean water and this requires the removal of trash, debris, soil particles, etc., and on-going maintenance. Over time, siltation and plugging may reduce the infiltration rate and subsequent effectiveness of the infiltration system. It should be noted that methods to prevent this shall be the responsibility of the infiltration designer and are not the purview of the geotechnical consultant. If adequate

measures cannot be incorporated into the design and maintenance of the system, then the infiltration rates may need to be further reduced. These and other factors should be considered in selecting a design infiltration rate.

- Any designed infiltration system will require routine periodic maintenance.
- As with any systems that are designed to concentrate the surface flow and direct the water into the subsurface soils, some type of nuisance water and/or other water-related issues should be expected.
- Contamination and environmental suitability of the site for infiltration was not evaluated by us and should be evaluated by others (environmental consultant). We only addressed the geotechnical issues associated with stormwater infiltration.

LGC Geotechnical should be provided with details for any planned required infiltration system early in the design process for geotechnical input.

4.9 ***Geotechnical Plan Review***

Project plans (grading, foundation, retaining wall, etc.) should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been incorporated. Additional field work and/or modified geotechnical recommendations may be necessary.

4.10 ***Geotechnical Observation and Testing During Construction***

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2019 CBC.

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

- During grading (removal bottoms, fill placement, etc.);
- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- After footing excavation and prior to placing concrete and/or reinforcement;
- 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 LIMITATIONS

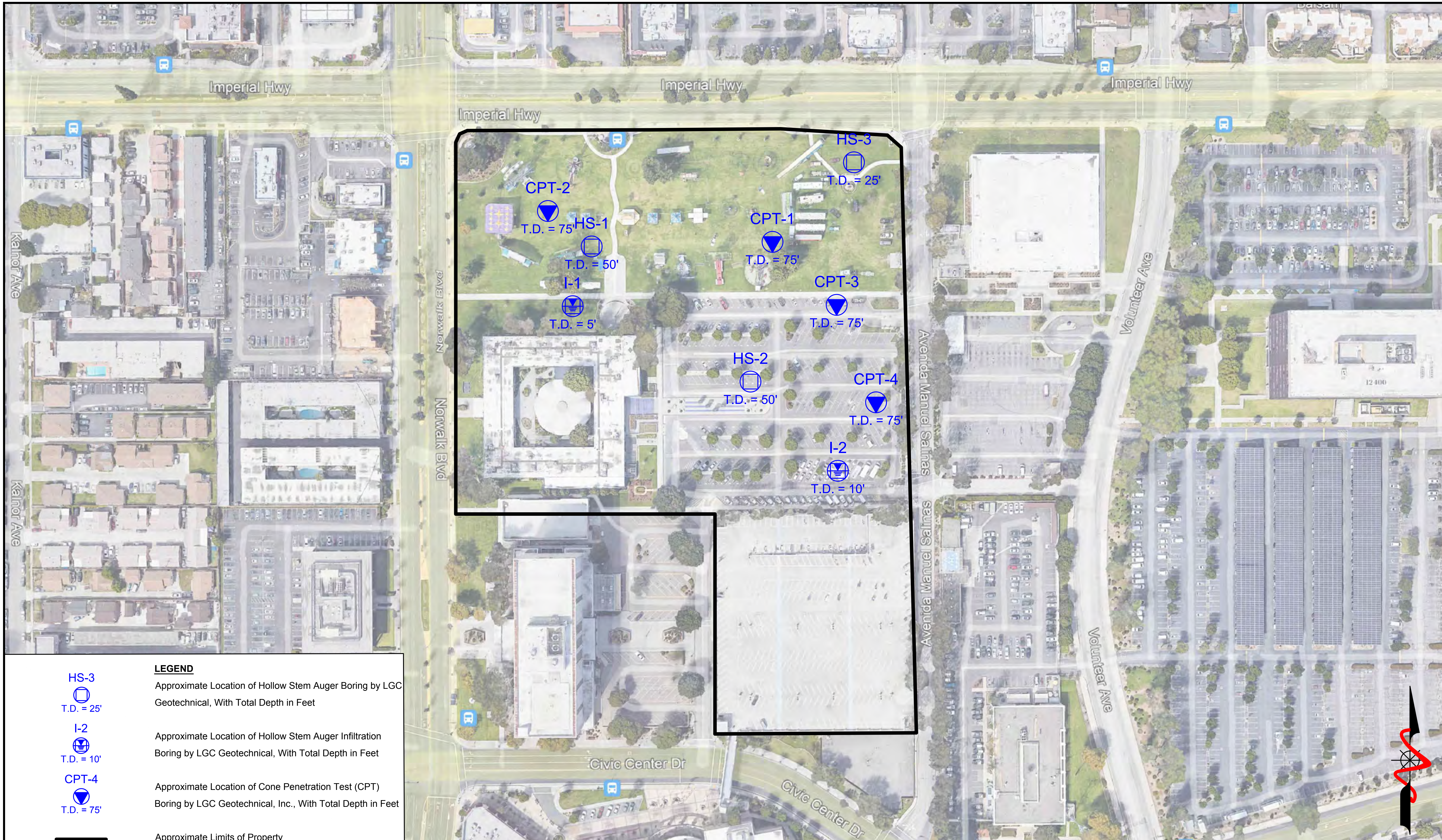
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included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

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

This report is issued with the understanding that it is the responsibility of the owner, or of their representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants and incorporated into the plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

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

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



LEGEND

HS-3 T.D. = 25' — Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet

I-2 T.D. = 10' — Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet

CPT-4 T.D. = 75' — Approximate Location of Cone Penetration Test (CPT) Boring by LGC Geotechnical, Inc., With Total Depth in Feet

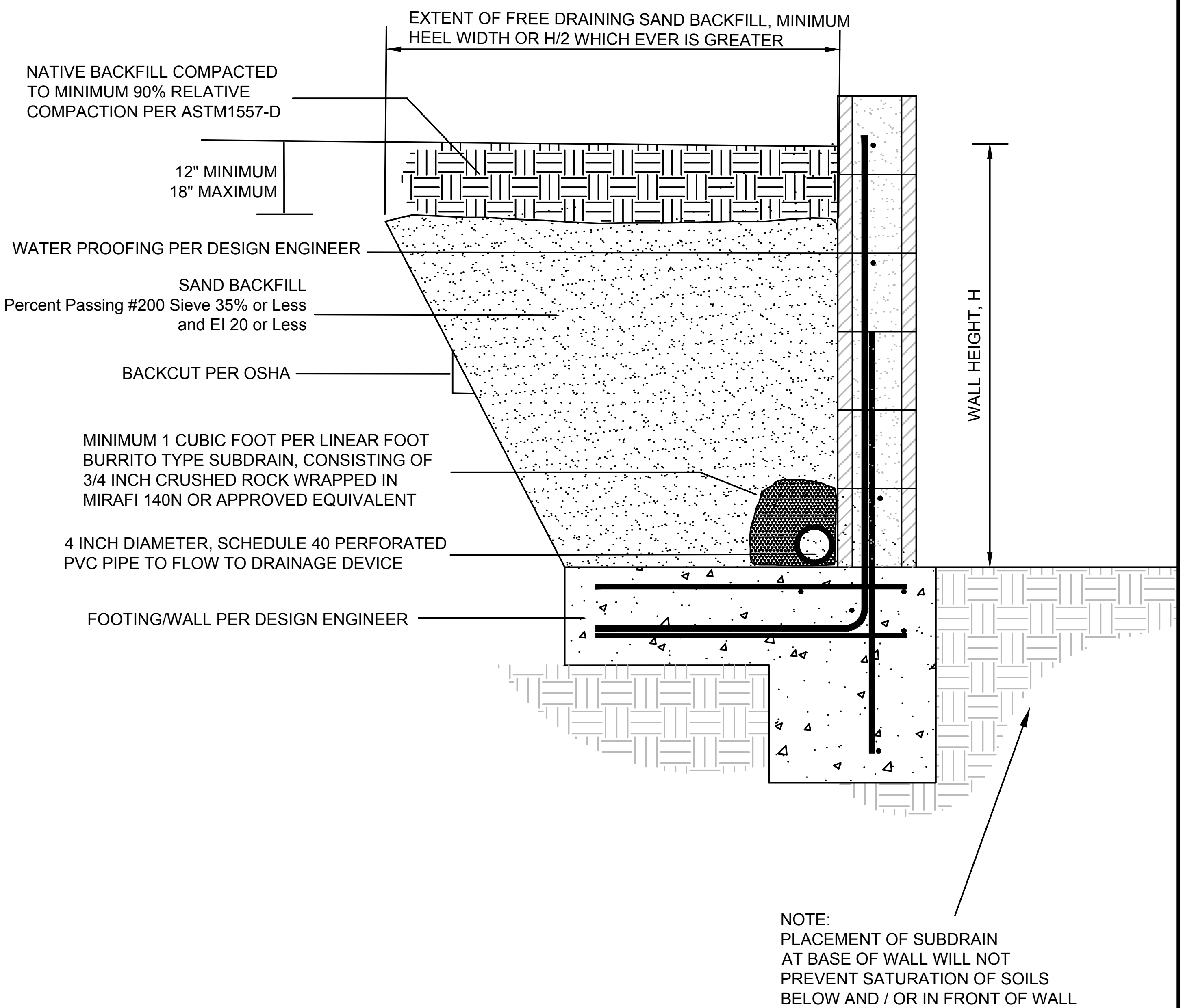
Approximate Limits of Property

LGC Geotechnical, Inc.

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

FIGURE 2
Geotechnical Exploration Location Map

PROJECT NAME	Placeworks - Norwalk
PROJECT NO.	22061-01
ENG. / GEOL.	DJB
SCALE	Not to Scale
DATE	June 2022



LGC Geotechnical, Inc.

FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	Placeworks - Norwalk
PROJECT NO.	22061-01
ENG. / GEOL.	DJB
SCALE	Not to Scale
DATE	June 2022

Appendix A
References

APPENDIX A

References

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16, 2017.

________, 2018, Standard 7-16, Minimum Design Loads for Buildings and Associated Criteria for Buildings and Other Structures, Supplement 1, effective: December 12, 2018

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

Bray, J.D., & Sancio, R. B., 2006, Assessment of liquefaction susceptibility of fine-grained soils, *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, pp. 1165-1177, dated September 2006.

California Building Standards Commission, 2019, California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2019.

California Department of Conservation, Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangles, Los Angeles and Orange Counties, California, Seismic Hazard Zone Report 98-28.

________, 1999, Seismic Hazard Zones, Whittier Quadrangle, Official Map Released: March 25, 1999.

California Department of Transportation (Caltrans), 2021, Corrosion Guidelines, Version 3.2, dated May 2021.

California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, Revision dated September 11, 2008.

_________, 2016, Geologic Map of the Long Beach 30' X 60' Quadrangle, California, Version 2.0, Compiled by George J. Saucedo.

County of Los Angeles, 2021, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, Los Angeles County Public Works Geotechnical and Materials Engineering Division, GS200.2, dated June 30, 2021.

Historic Aerials, 2022, viewed May 26, 2022, https://www.historicaerials.com/

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

Placeworks, 2022, Norwalk Entertainment District – Civic Center Specific Plan Project.

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

References

Structural Engineers Association of California (SEAOC), 2022, Seismic Design Maps, Retrieved April 28, 2022, from https://seismicmaps.org/

United States Geological Survey (USGS), 2014, Unified Hazard Tool, Dynamic: Conterminous U.S. 2014 (update) (v4.2.0), Retrieved April 28, 2022, from: https://earthquake.usgs.gov/hazards/interactive/

Appendix B
Boring Logs

Geotechnical Boring Log Borehole HS-1

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~102' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						**@0' to 7.5' - Undocumented Artificial Fill (afu):**	
100			R-1	3 5 7	96.3	16.8	ML	@2.5'- Sandy SILT: brown, very moist, stiff	MD
	5		SPT-1	2 3 4		7.6	SM	@5'- Silty SAND: brown, moist, loose	-#200
95								**@7.5' to T.D. - Quaternary Alluvial Deposits (Qal):**	
			R-2	7 10 15	99.5	2.7	SP	@7.5'- SAND: grayish brown, dry, medium dense	
	10		SPT-2	4 7 10		4.0	SM	@10'- Silty SAND: yellowish brown, dry, medium dense	-#200
90									
	15		SPT-3	2 2 2		27.1	CL	@15'- Sandy CLAY: brown, very moist, medium stiff	-#200
85									
	20		R-3	2 4 8	122.7	14.7		@20'- Sandy CLAY: brown, moist, stiff	AL CN
80									
	25		SPT-4	2 3 4		17.4	SM/ML	@25'- Sandy SILT: reddish brown, very moist, stiff	-#200
75									
	30								

LGC Geotechnical, Inc.

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

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

GROUNDWATER TABLE

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

Geotechnical Boring Log Borehole HS-1

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~102' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-4	20 31 37	111.0	3.1	SP	@30'- SAND: light brown, dry, very dense	
70									
	35		SPT-5	34 50/5"		2.7		@35'- SAND with Gravel: yellowish brown, dry, very dense	
65									
	40		R-5	19 32 50/3.5"	108.8	13.3	SM	@40'- Silty SAND: yellowish brown, very moist, very dense	
60									
	45		SPT-6	17 27 21		3.7	SP	@45'- SAND with Gravel: yellowish brown, dry, very dense	
55									
	50		R-6	20 30 25	98.0	3.8		@50'- SAND: pale brown, dry, dense	
50								Total Depth = 50' Groundwater Not Encountered Backfilled with Cuttings on 4/27/2022	
	55								
45									
	60								

LGC Geotechnical, Inc.

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- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~101' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
100	0	B-1						**@0' to 5' - Undocumented Artificial Fill (afu):** @0'- 5" of Asphalt over 4" of Base	EI CR
			R-1	4 5 7	85.8	4.0	ML	@2.5'- Sandy SILT: brown, dry, stiff	
95	5		SPT-1	1 2 3		23.8	ML	**@5' to T.D. - Quaternary Alluvial Deposits (Qal):** @5'- Sandy SILT: pale brown, very moist, medium stiff	
			R-2	4 8 13	96.1	6.0	SM	@7.5'- Silty SAND: pale brown, slightly moist, medium dense	
90	10		SPT-2	6 7 6		2.4	SP	@10'- SAND: light grayish brown, dry, medium dense	
85	15		R-3	3 3 6	94.4	31.7	CL	@15'- Silty CLAY: brown, very moist, medium stiff	AL CN
80	20		SPT-3	3 5 7		13.6	ML	@20'- Sandy SILT: brown, moist, very stiff	
75	25		R-4	4 8 15	102.2	21.1		@25'- SILT: light brown, very moist	
	30								

LGC Geotechnical, Inc.

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

Last Edited: 5/19/2022

Geotechnical Boring Log Borehole HS-2

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~101' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
70	30		SPT-4	8 15 23		1.3	SP	@30'- Gravelly SAND: light yellowish brown, dry, dense	
65	35		R-5	23 37 50/6"	124.4	2.2		@35'- SAND with Gravel: pale yellowish brown, dry, very dense	
60	40		SPT-5	18 30 43		1.7		@40'- SAND with Gravel: light yellowish brown, dry, very dense	
55	45		R-6	17 32 50/6"	108.9	2.0		@45'- SAND: light yellowish brown, dry, very dense	
50	50		SPT-6	10 12 13		14.6	SC	@50'- Clayey SAND: light brown, moist, dense	
45	55							Total Depth = 50' Groundwater Not Encountered Backfilled with Cuttings on 4/27/2022	
	60								

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- EI EXPANSION INDEX
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- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~101' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
100	0	B-1						**@0' to 2.5' - Undocumented Artificial Fill (afu):**	
								@2.5' to T.D. - Quaternary Alluvial Deposits (Qal):	
			R-1	7 17 17	118.1	10.2	SM	@2.5'- Silty SAND: brown, moist, medium dense	EI
95	5		SPT-1	3 4 3		13.1	SC	@5'- Clayey SAND: brown, moist, loose	
			R-2	2 3 5	105.5	16.8	SM	@7.5'- Silty SAND: reddish brown, very moist, loose	AL CN
90	10		SPT-2	1 1 2		27.2		@10'- Silty SAND: dark olive brown, very moist, loose	
85	15		SPT-3	3 6 7		13.4	CL	@15'- Sandy CLAY: reddish brown, moist, very stiff	
80	20		R-3	5 14 19	111.8	16.0	SM	@20'- Silty SAND: brown, very moist, medium dense	
75	25		SPT-4	4 5 7		16.6	ML	@25'- SILT: brown, very moist, very stiff	
	30							Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 4/27/2022	

Geotechnical, Inc.

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- GROUNDWATER TABLE

TEST TYPES:
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- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-1

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~103' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to 3.5' - Undocumented Artificial Fill (afu):**	
100								**@3.5' to T.D. - Quaternary Alluvial Deposits (Qal):**	
			R-1	3 5 6	98.3	7.4	ML	@3.5'- SILT: light brown, slightly moist, stiff	
	5							Total Depth = 5' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed and Surrounded by Gravel. Presoaked on 4/27/22 Pipe Removed and Boring Backfilled with Cuttings on 4/28/22.	
95	10								
90	15								
85	20								
80	25								
75	30								

LGC Geotechnical, Inc.

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

Last Edited: 5/19/2022

Geotechnical Boring Log Borehole I-2

Date: 4/27/2022	**Drilling Company:** Martini Drilling
Project Name: Placeworks - Norwalk	**Type of Rig:** Truck Mounted
Project Number: 22061-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~100' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By DJB

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to 2.5' - Undocumented Artificial Fill (afu):**	
								@0'- 5" of Asphalt over 4" of Base	
								@2.5' to T.D. - Quaternary Alluvial Deposits (Qal):	
			R-1	3 5 8	95.5	6.2	ML	@2.5'- SILT: light brown, slightly moist, medium stiff	
95	5		R-2	6 9 13	101.4	7.0		@5'- Sandy SILT: light brown, slightly moist, very stiff	
			R-3	5 10 15	86.4	1.6	SP	@8.5'- SAND: light brown, dry, medium dense	
90	10							Total Depth = 10' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed and Surrounded by Gravel. Presoaked on 4/27/2022 Pipe Removed and Boring Backfilled with Cuttings on 4/28/2022	
85	15								
80	20								
75	25								
	30								

Geotechnical, Inc.

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

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

Last Edited: 5/19/2022

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

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

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

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

Sample Location	Expansion Index	Expansion Potential*
HS-2 @ 1-5 feet	0	Very Low
HS-3 @ 1-5 feet	25	Low

* ASTM D4829

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

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 5 feet	Silty Sand	24
HS-1 @ 10 feet	Silty Sand	8
HS-1 @ 15 feet	Sandy Clay	68
HS-1 @ 25 feet	Sandy Silt	61

Laboratory Testing Procedures and Test Results

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

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-1 @ 20 feet	20	16	4	CL-ML
HS-2 @ 15 feet	37	24	13	CL
HS-3 @ 7.5 feet	NP	NP	NP	SM

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

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-1 @ 1-5 feet	Brown Silty Sand	131.0	8.5

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

Sample Location	Chloride Content, ppm
HS-2 @ 1-5 feet	40

APPENDIX C (Cont'd)

<u>Laboratory Testing Procedures and Test Results</u>

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

Sample Location	Sulfate Content (ppm)	Sulfate Exposure Class *
HS-2 @ 1-5 feet	123	S0

*Based on ACI 318R-14, Table 19.3.1.1

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-2 @ 1-5 feet	8.40	1,940

Appendix D
Infiltration Test Data

Infiltration Test Data Sheet

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

Project Name: Placeworks - Norwalk
Project Number: 22061-01
Date: 4/27/2022
Location: I-1

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

*measured at time of test

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

Pre-Soak /Pre-Test

No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Comments
PS-1	12:23	13:23	60.0	2.56	4.89	2.33	
Pre-Test	13:25	13:55	30.0	3.08	4.13	1.05	

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	13:57	14:27	30.0	3.03	3.96	0.93	3.50	2.2
2	14:29	14:59	30.0	2.32	3.36	1.04	4.87	1.8
3	15:01	15:31	30.0	2.48	3.43	0.95	4.63	1.7
4	15:34	16:04	30.0	2.19	3.21	1.02	5.17	1.7
5	16:06	16:36	30.0	2.42	3.40	0.98	4.73	1.7
6	16:38	17:08	30.0	2.48	3.46	0.98	4.60	1.8
7								
8								
9								
10								
11								
12								

Measured Infiltration Rate	**1.8**
Feasibility Factor of Safety	See Report
Feasibility Infiltration Rate	**See Report**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2021

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

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

Project Name: Placeworks - Norwalk
Project Number: 22061-01
Date: 4/27/2022
Location: I-2

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

*measured at time of test

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

Pre-Soak /Pre-Test

No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Comments
PS-1	12:28	13:28	60.0	5.16	9.75	4.59	
Pre-Test	13:31	13:51	20.0	6.25	10	3.75	

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	13:54	14:14	20.0	7.44	9.53	2.09	3.52	7.5
2	14:16	14:36	20.0	7.26	9.38	2.12	3.87	6.9
3	14:38	14:58	20.0	7.41	9.48	2.07	3.61	7.2
4	15:00	15:20	20.0	7.27	9.36	2.09	3.88	6.8
5	15:23	15:43	20.0	7.05	9.33	2.28	4.14	6.9
6	15:45	16:05	20.0	7.04	9.29	2.25	4.19	6.7
7	16:07	16:27	20.0	6.95	9.06	2.11	4.53	5.9
8	16:30	16:50	20.0	7.14	9.28	2.14	4.10	6.6
9	16:52	17:12	20.0	7.21	9.32	2.11	3.98	6.7
10								
11								
12								

Measured Infiltration Rate	**6.4**
Feasibility Factor of Safety	See Report
Feasibility Infiltration Rate	**See Report**

Sketch:

Notes:

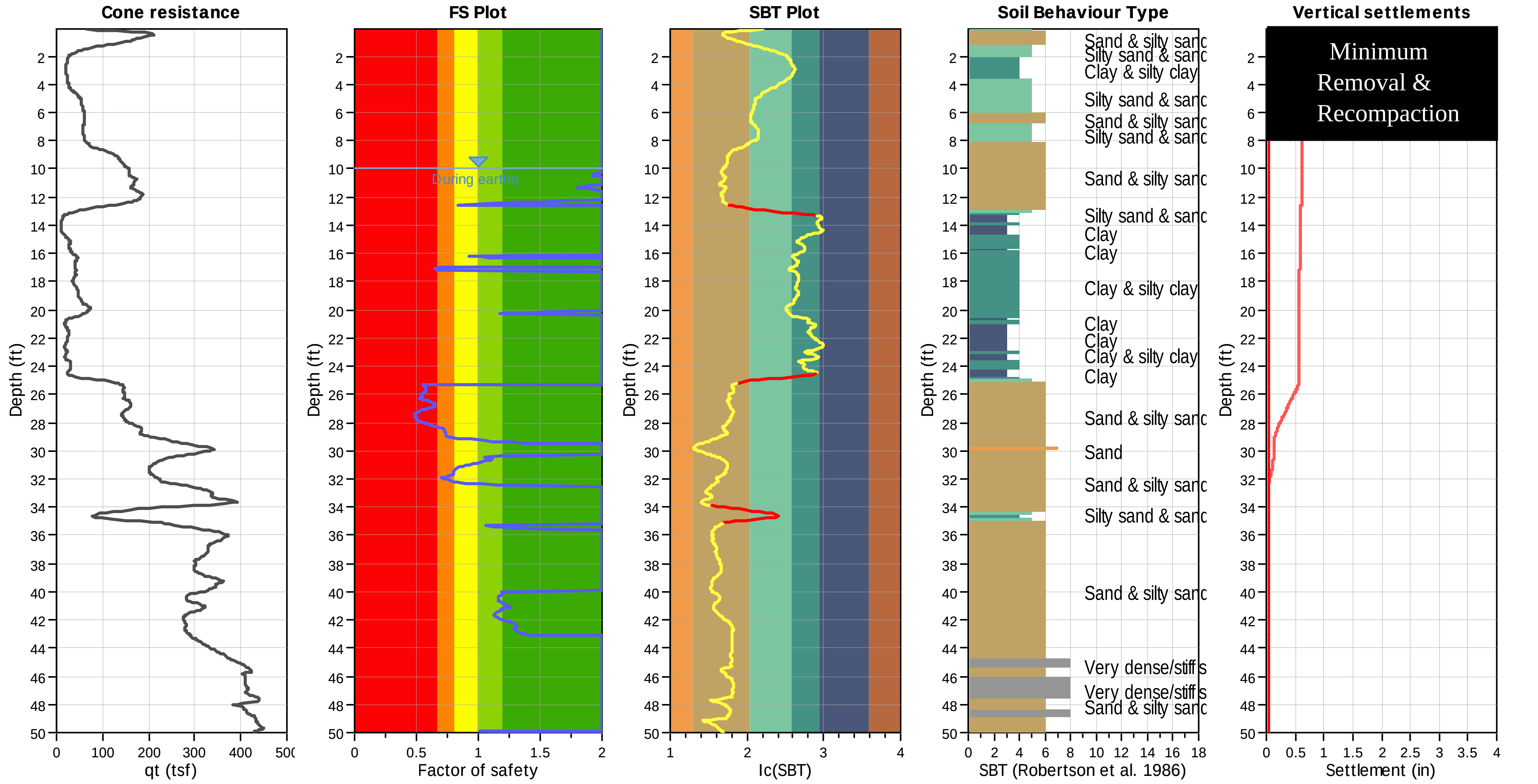
Based on Guidelines from: LA County dated 06/2021

Appendix E
Liquefaction Analysis

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

Project:
Location:

CPT: CPT-1
Total depth: 75.35 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.84	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

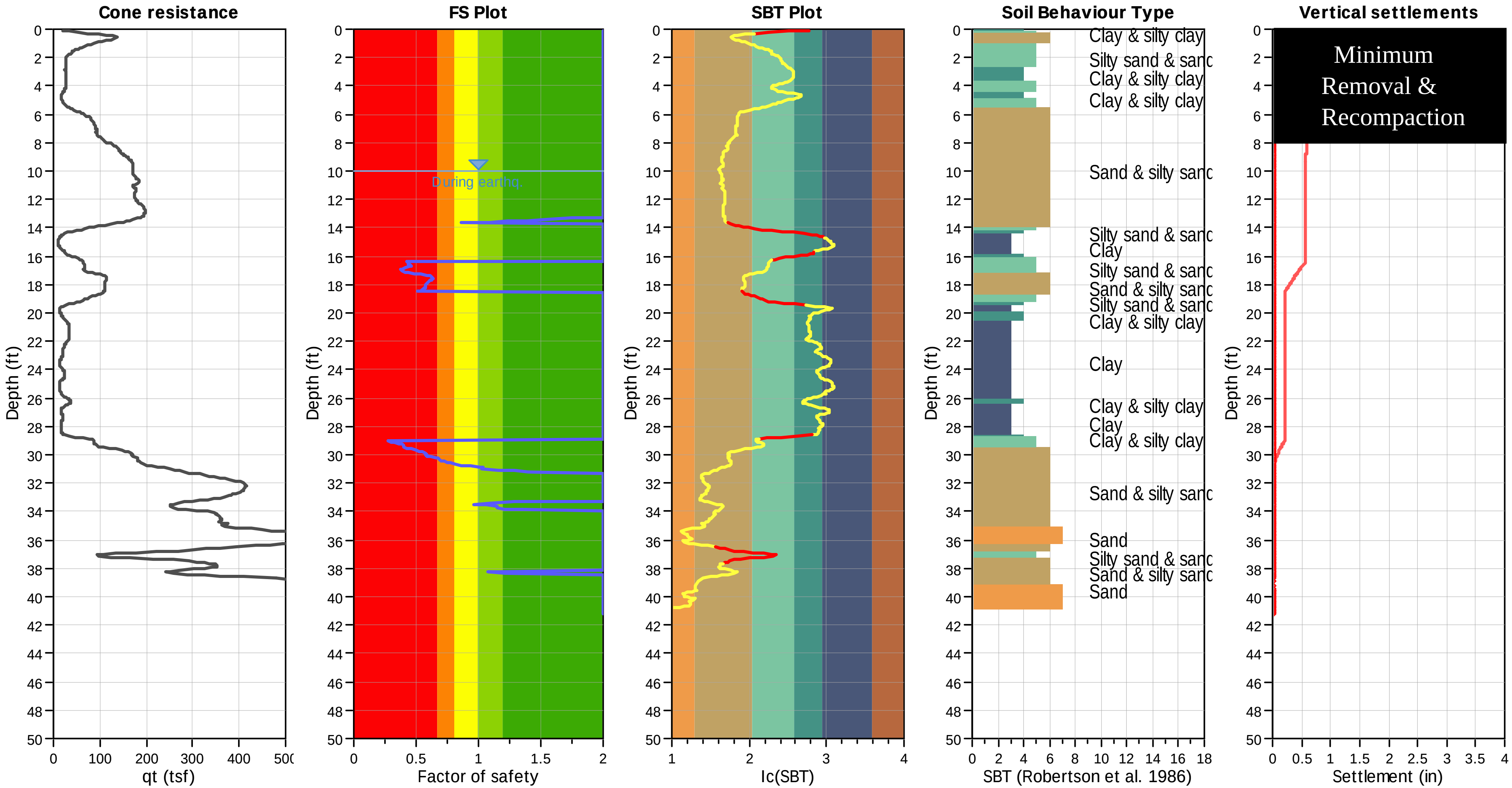
GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

Project:

Location:

CPT: CPT-2

Total depth: 41.21 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.84	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

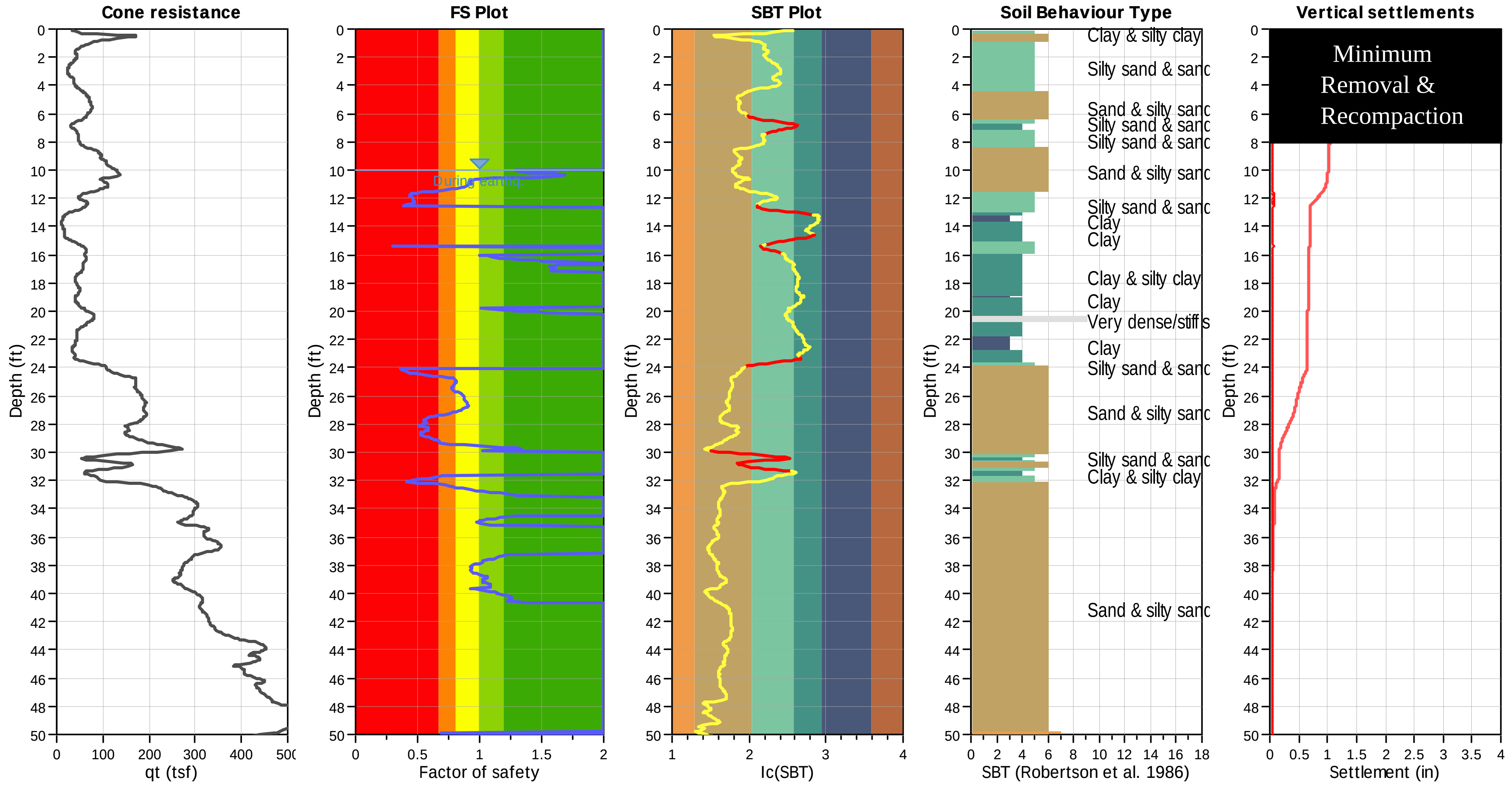
GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

Project:

Location:

CPT: CPT-3

Total depth: 75.13 ft



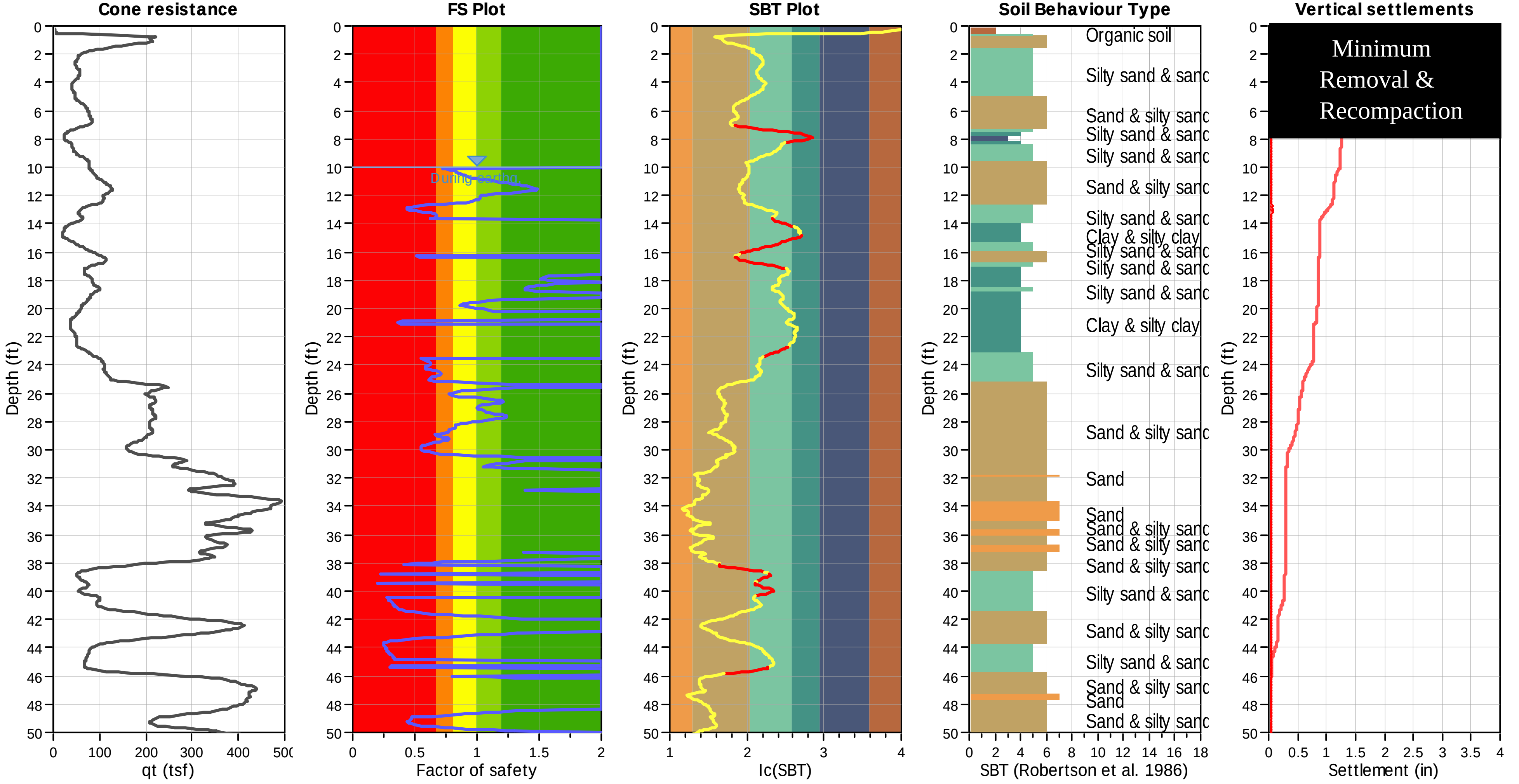
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.84	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project:

Location:

CPT: CPT-4

Total depth: 75.27 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.84	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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.

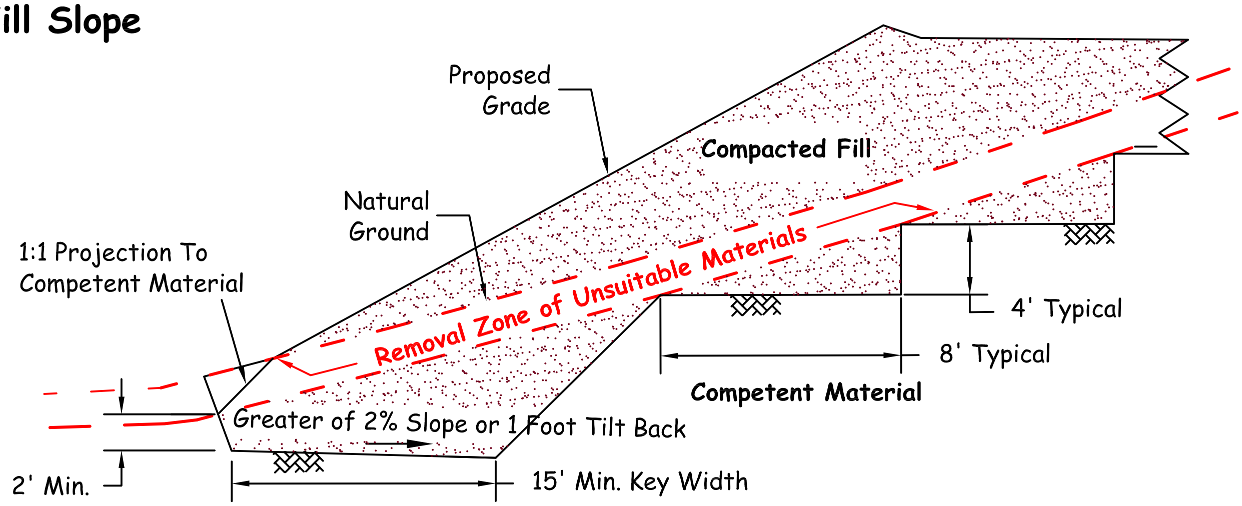
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

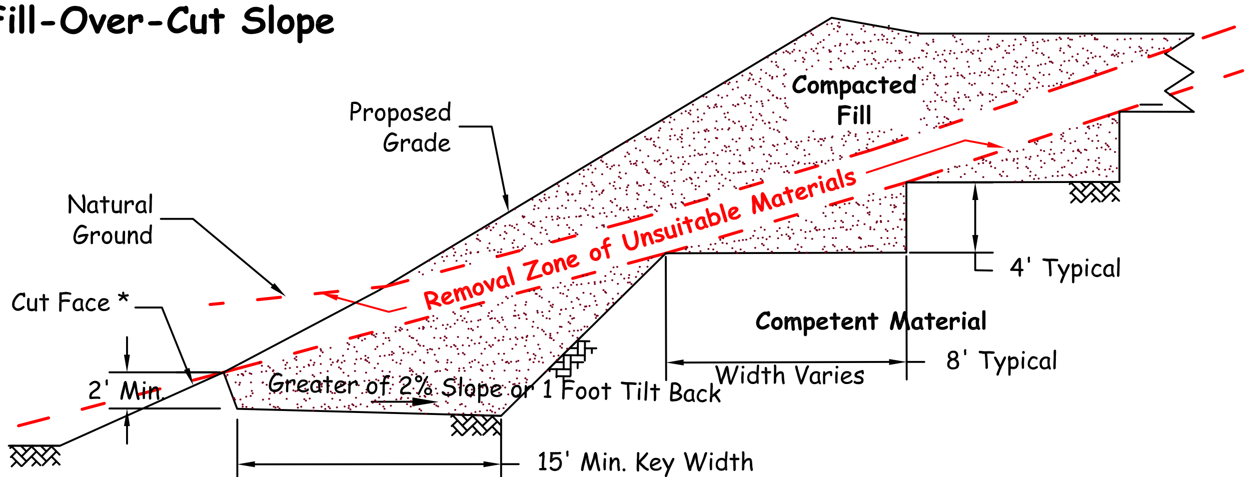
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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

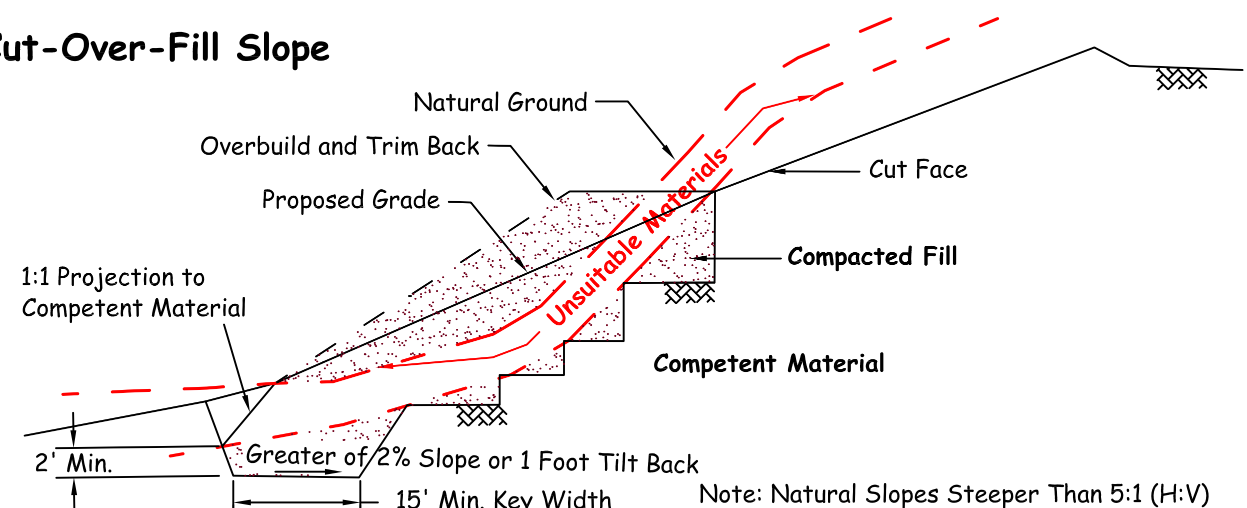


Fill-Over-Cut Slope



* Construct Cut Slope First

Cut-Over-Fill Slope



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

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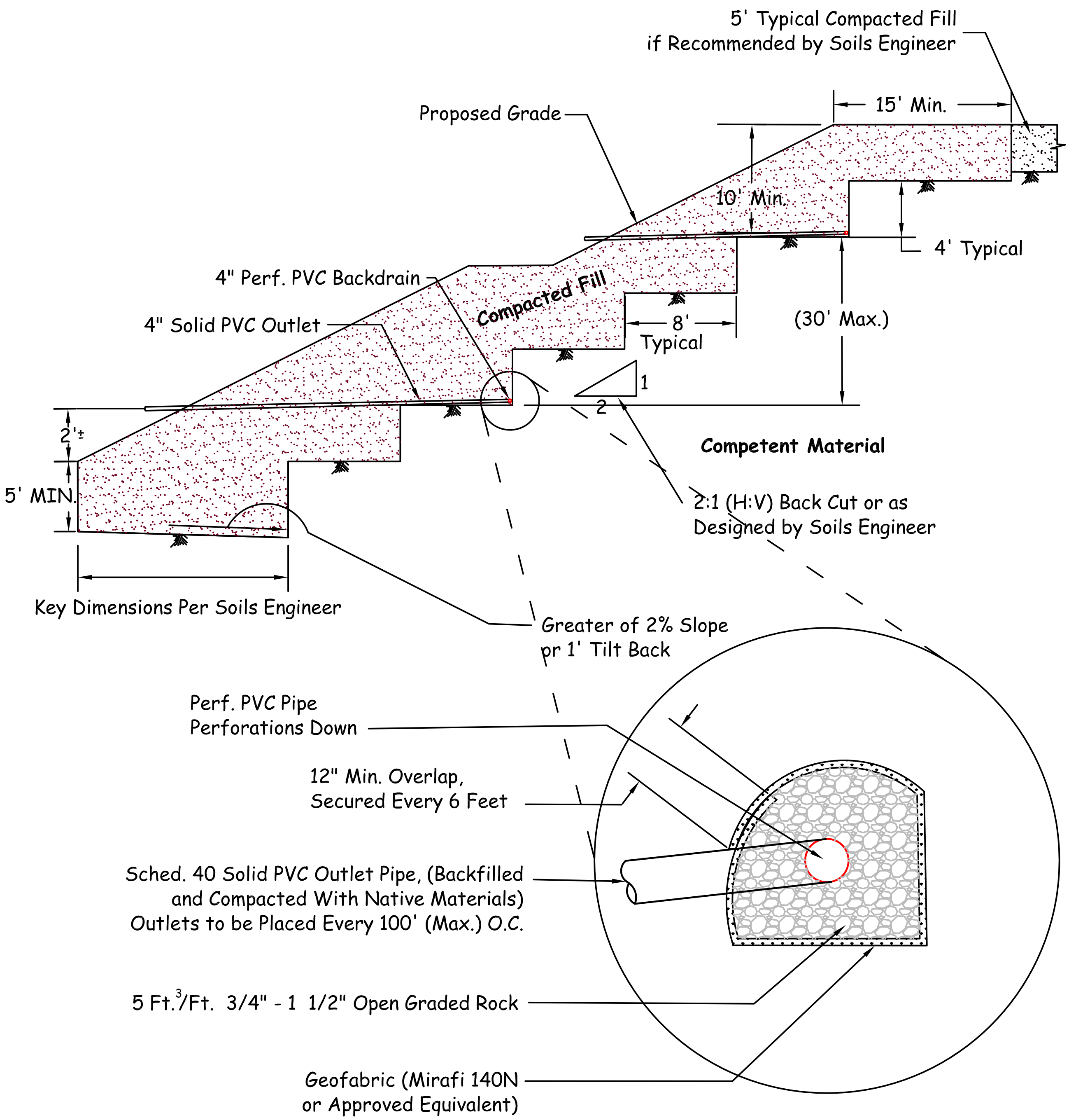
KEYING AND BENCHING



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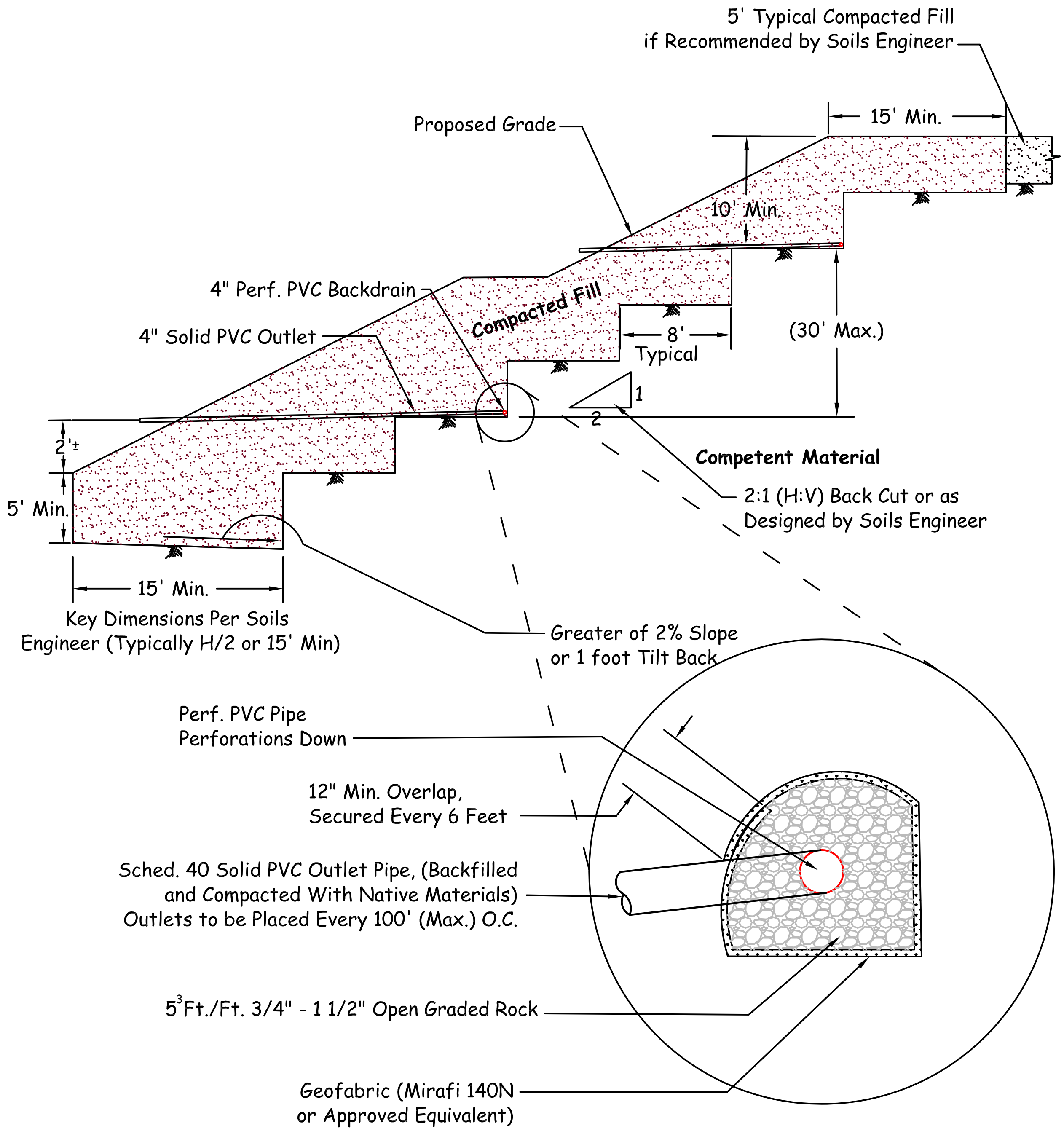
TYPICAL BUTTRESS DETAIL



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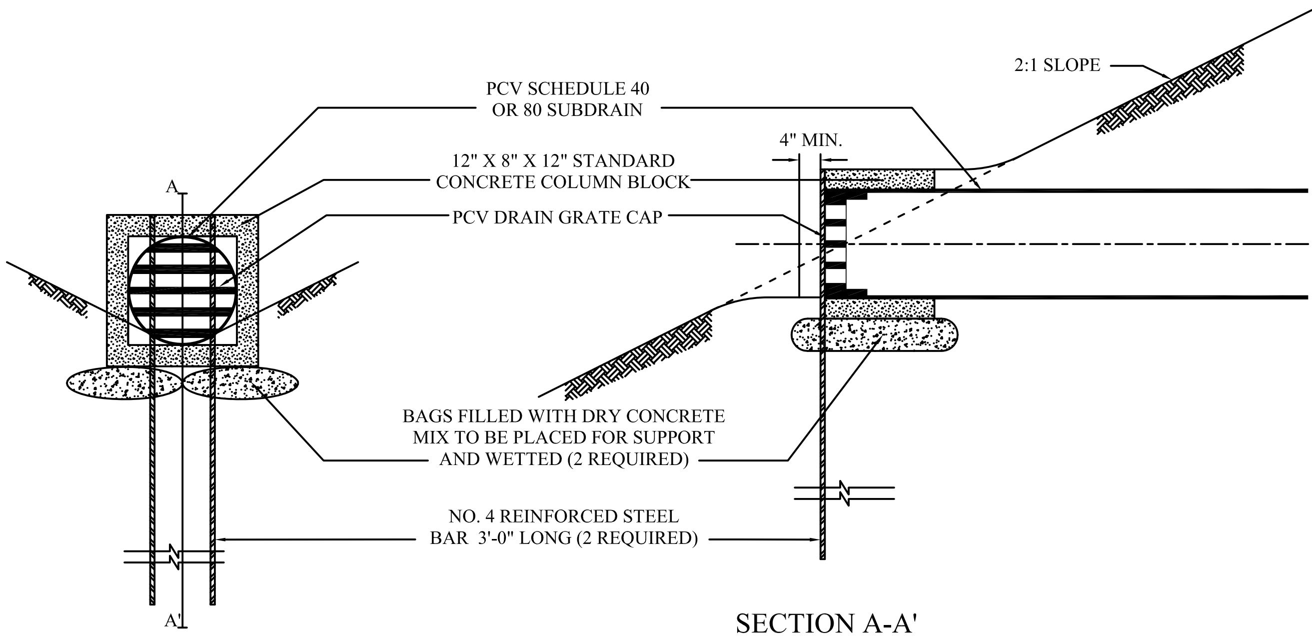
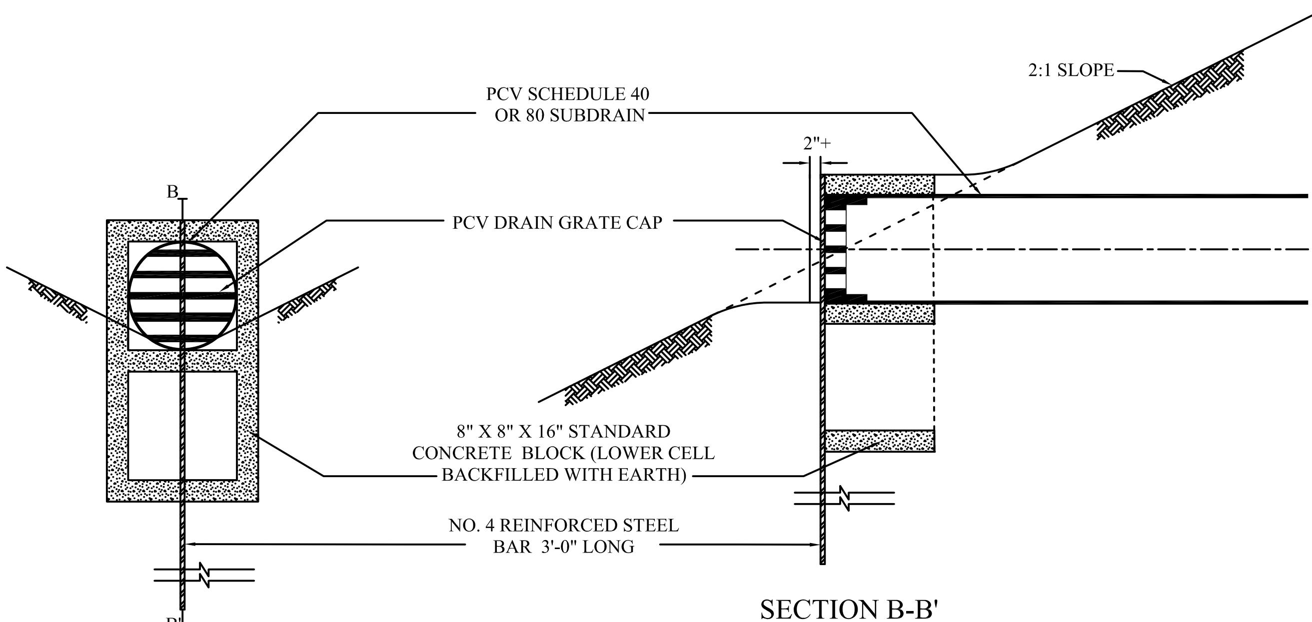
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SUBDRAIN OUTLET MARKER -4" PIPE



NOT TO SCALE

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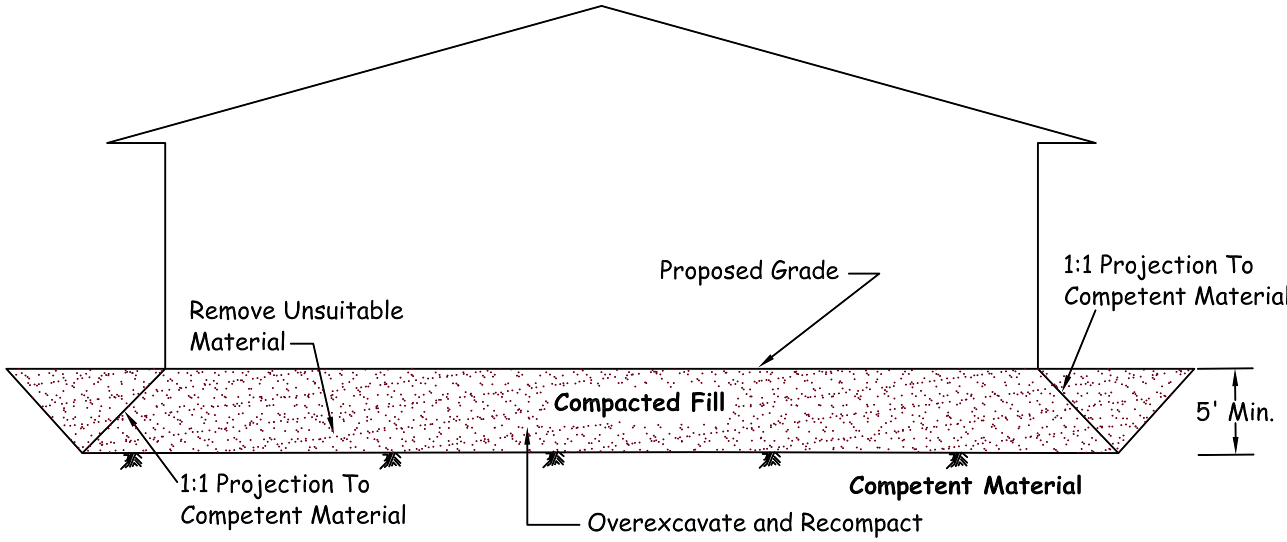
SUBDRAIN OUTLET MARKER DETAIL

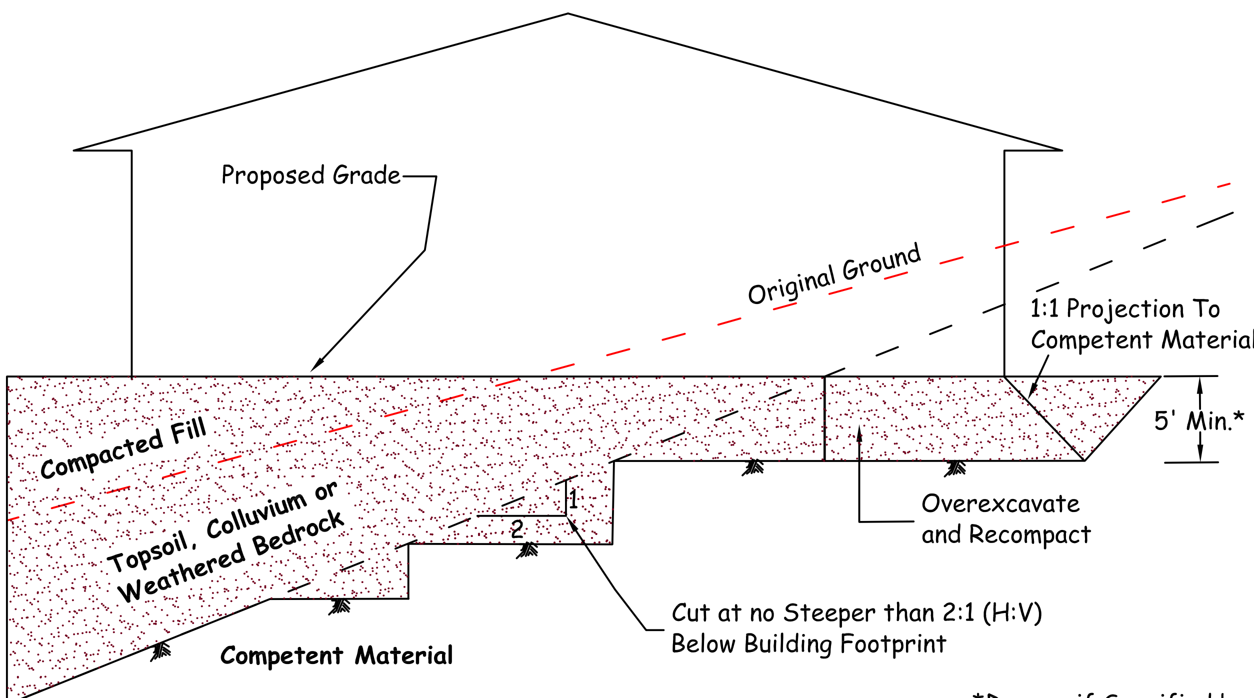
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



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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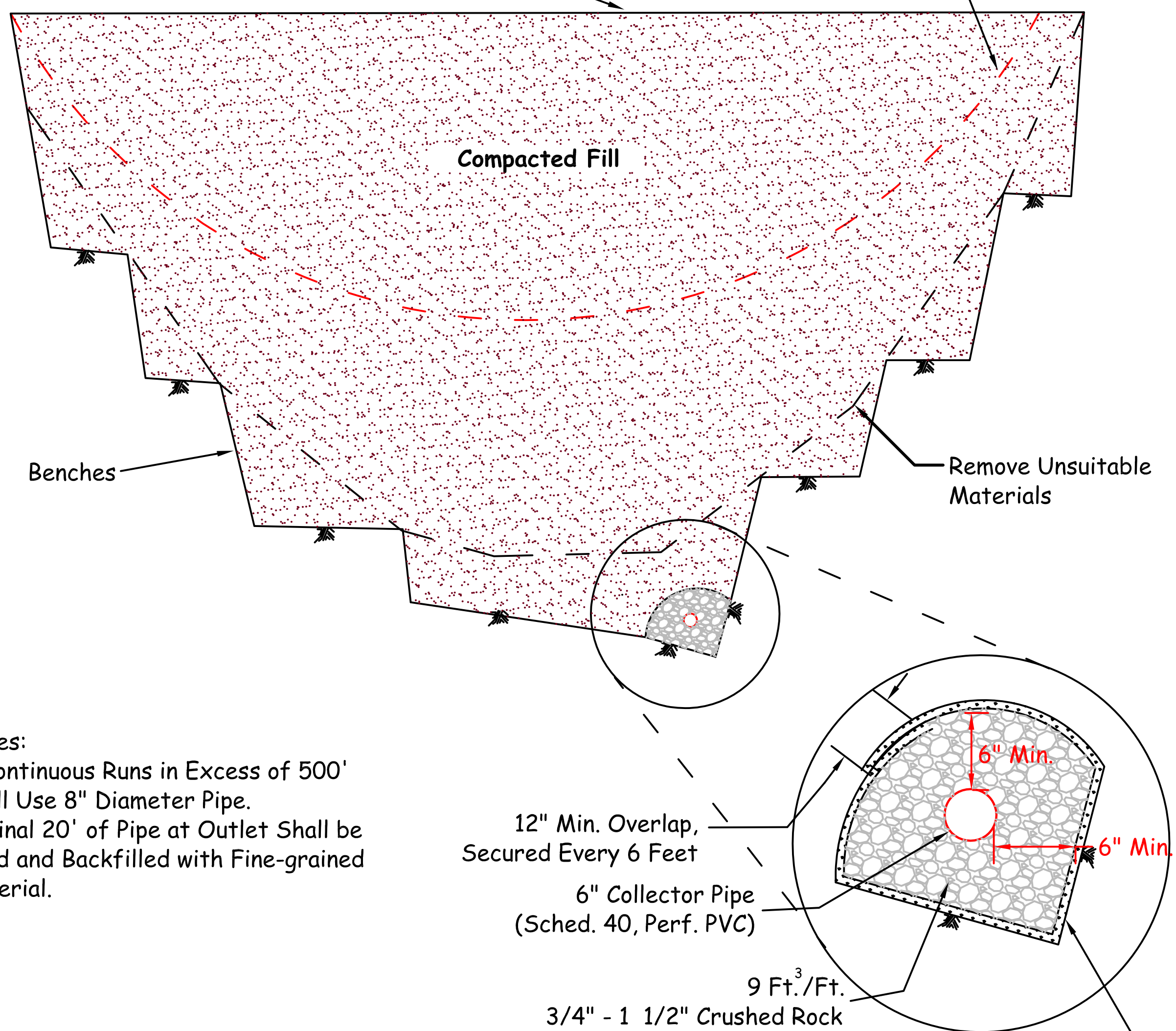
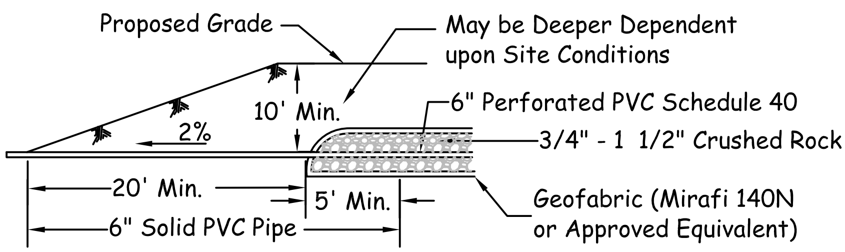
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



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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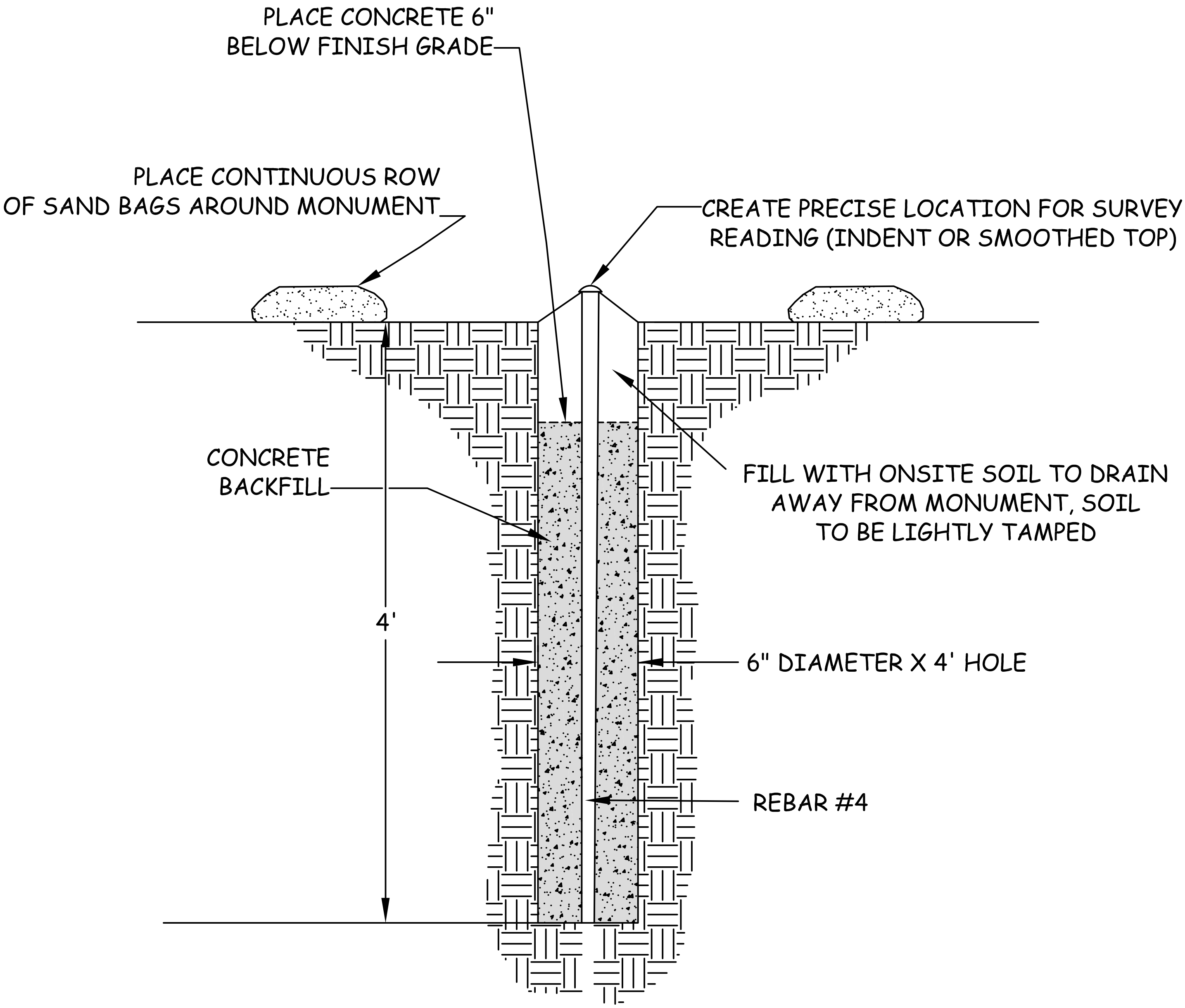
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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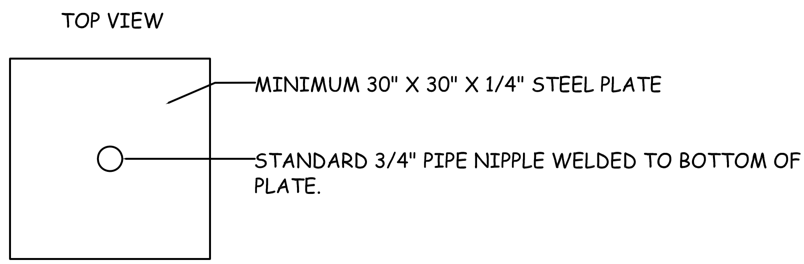
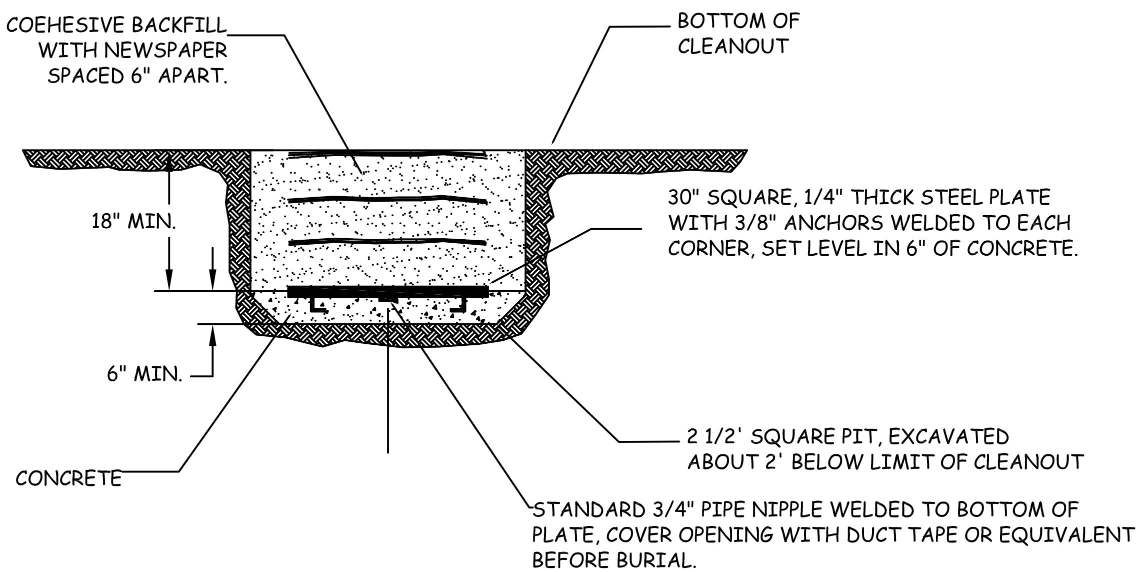
TYPICAL SURFACE SETTLEMENT MONUMENT





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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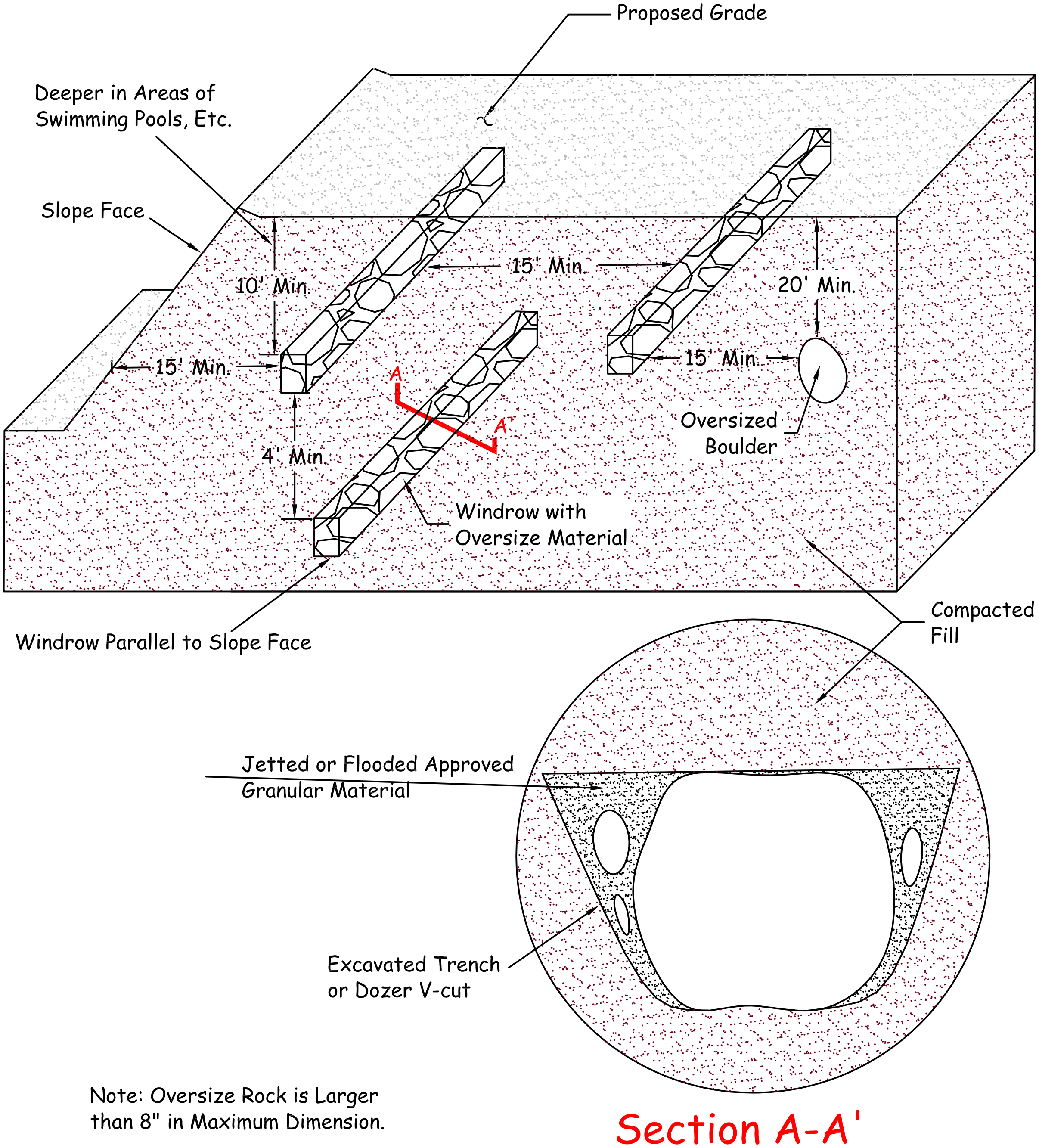
TYPICAL SETTLEMENT PLATE AND RISER



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

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OVERSIZE ROCK DISPOSAL DETAIL

Appendices

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