APPENDIX 9

May 17, 2021 Project No. 21051-01

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Subject: ***Preliminary Geotechnical Evaluation and Recommendations, Proposed Multi-Family Residential Development Located at the Intersection of Philadelphia Street and Comstock Avenue, Whittier, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed multi-family residential development located at the intersection of Philadelphia Street and Comstock Avenue in the City of Whittier, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide geotechnical recommendations, including infiltration testing, relative to the proposed residential development.

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

Respectfully Submitted,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our preliminary geotechnical evaluation for the proposed approximately 0.8-acre residential development located at the intersection of Philadelphia Street and Comstock Avenue in the City of Whittier, California. Refer to the Site Location Map (Figure 1).

The purpose of our study was to provide a geotechnical evaluation relative to the proposed residential development. As part of our scope of work, we have: 1) reviewed available geotechnical information and in-house geologic maps pertinent to the site (Appendix A); 2) performed a subsurface geotechnical evaluation of the site consisting of the excavation and sampling of four small-diameter borings ranging from approximately 15 to 46.5 feet below existing ground surface, 3) performed two falling head infiltration tests within borings; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this preliminary geotechnical summary report presenting our findings and preliminary conclusions and recommendations for the development of the proposed project.

It should be noted that our evaluation and this report only address geotechnical issues associated with the site and do not address any environmental issues.

1.2 Background

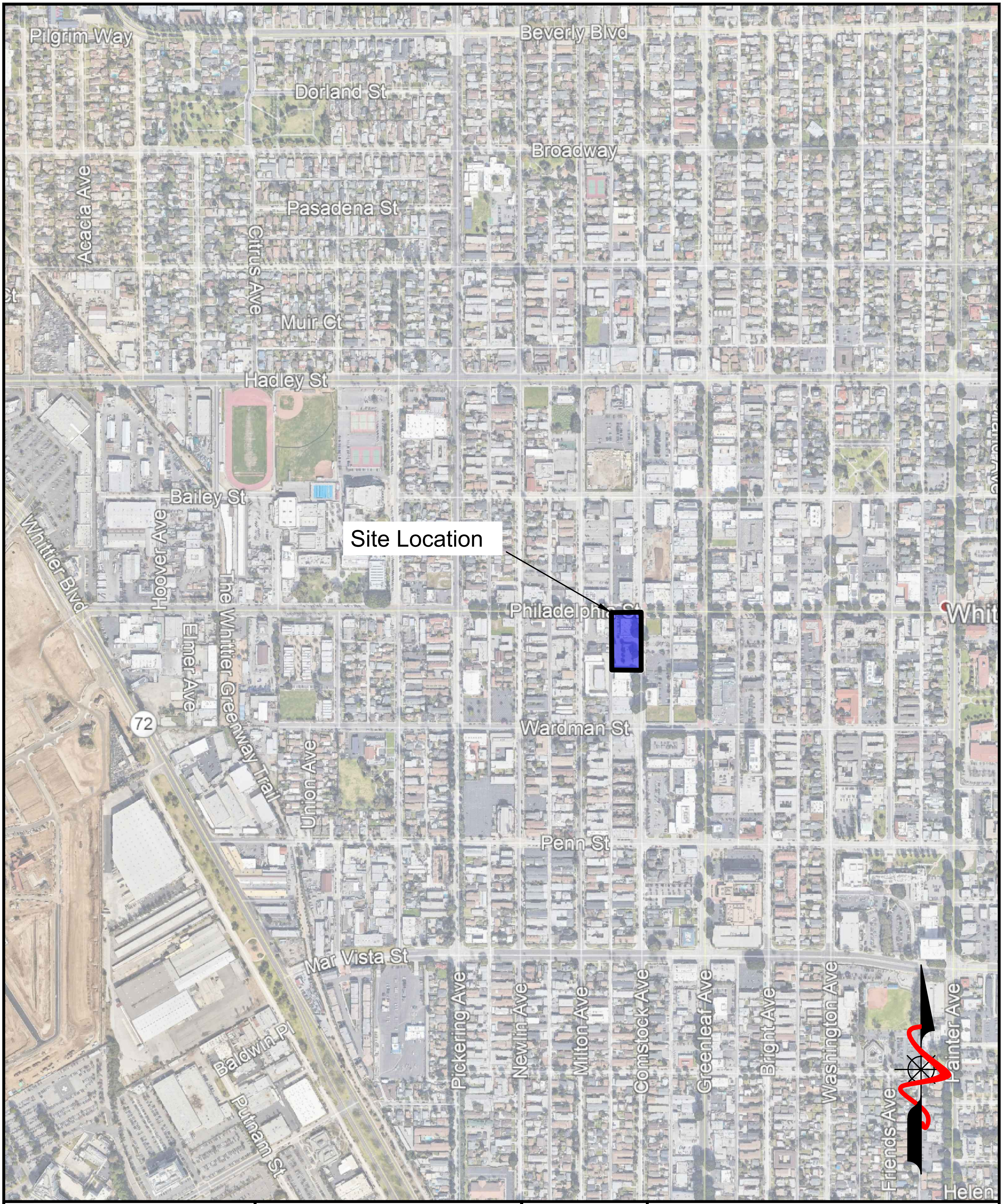
Review of historical aerials indicates the site had been undeveloped in 1954. It appears between the years of 1954 and 1963 the current building and parking lot were constructed (Historic Aerials, 2021).

1.3 Project Description

The approximately 0.8-acre site is bound to the north by Philadelphia Street, to the east by Comstock Avenue, to the south by an existing commercial development, and to the west by an alley. The site is currently occupied by an existing commercial structure and on-grade asphalt concrete parking lot.

Proposed development will consist of one 4-level residential structure with 51 multi-family dwelling units, on-grade parking, amenity deck, and a water quality system. The proposed residential development is anticipated to consist of relatively light building loads (column and wall loads maximum of 40 kips and 2 kips per linear foot, respectively).

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



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	MWIG- Whittier
PROJECT NO.	21051-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	May 2021

1.4 Subsurface Geotechnical Evaluation

A limited subsurface geotechnical evaluation of the site was performed by LGC Geotechnical. Our exploration program consisted of drilling and sampling four small-diameter exploratory hollow-stem borings (HS-1, HS-2, I-1 & I-2) for the purpose of obtaining samples for evaluation and laboratory testing of site soils, with two of the borings (I-1 and I-2) utilized for percolation testing. The borings were drilled by Cal Pac Drilling, Inc., under subcontract to LGC Geotechnical. The depths of the borings ranged from approximately 15 to 46.5 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were performed using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. Bulk samples of the near-surface soils were logged and collected for laboratory testing from select borings. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 and 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 or until refusal. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. The borings were subsequently backfilled with cuttings, tamped and capped with asphalt coldpatch, where necessary.

Infiltration testing was performed within two of the borings, I-1 and I-2, to depths of approximately 15 feet below existing grade. An LGC Geotechnical geologist installed standpipes, backfilled the borings with crushed rock and pre-soaked the infiltration holes prior to testing. Infiltration testing was performed per the County of Los Angeles testing guidelines. Standpipes were removed and the locations were subsequently backfilled with native soils at the completion of testing. Some settlement of the backfill soils may occur over time.

The approximate locations of our subsurface explorations are provided on the Boring Map, Figure 2. The boring and infiltration testing logs are provided in Appendix B and Appendix D, respectively.

1.5 Field Infiltration Testing

Two shallow infiltration test wells were installed in Borings I-1 and I-2 to approximate depths of 15 feet below existing grade. The approximate infiltration test boring locations are shown on the Boring Map (Figure 2).

Estimation of infiltration rates was performed in general accordance with the "Boring Percolation Test Procedure" guidelines set forth by the County of Los Angeles (2017). The borings for the infiltration tests were excavated using a drill rig equipped with 8-inch diameter hollow-stem augers. 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. Infiltration tests were performed using relatively clean water free of particulates, silt, etc. The infiltration wells were pre-soaked approximately 1 hour prior to testing. During the pre-test, water was added to the boring and was observed after 10 minutes and 30 minutes to determine test methodology. In infiltration test holes I-1 and I-2 water remained in the borings after 30 minutes. Therefore, the test procedure utilizing a thirty-minute reading interval was performed on both infiltration test holes (I-1 & I-2). Readings were taken a minimum of 8 times 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 and backfilled with cuttings and tamped. Some settlement of the backfill should be expected.

Based on the County of Los Angeles testing guidelines (2017), the infiltration rate is calculated by dividing the volume of water discharged by the surface area of the test section (including the sidewalls and bottom of the boring) over a specific time period. The measured infiltration rate is taken as the average of the last three readings during which a "stabilized rate" is achieved. The measured infiltration rates are provided in Table 1 below.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Approximate Infiltration Test Depth (ft)	Measured Infiltration Rate* (inch/hr.)
I-1	15	0.2
I-2	15	0.1

*Does Not Include Required Reduction Factors for Design.

Please note that the values provided in Table 1 do not include reduction factors associated with the test procedure, site variability, and long-term siltation plugging that are used to calculate the design infiltration rate. Infiltration test data is presented in Appendix D. Refer to Section 4.6 for recommendations regarding infiltration of stormwater.

1.6 Laboratory Testing

Representative bulk, grab, and driven (relatively undisturbed) samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, sieve analysis, Atterberg limits, consolidation, direct shear, 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 104.9 pounds per cubic foot (pcf) to 119.6 pcf, with an average of 114.1 pcf. Field moisture contents ranged from approximately 1.4 to 17.0 percent, with an average of approximately 12.4 percent.
- Two sieve particle size analysis tests were performed and indicated a fines content (passing No. 200 sieve) of approximately 60 to 82 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as "fine-grained."
- One Atterberg Limit (liquid limit and plastic limit) test was performed. Results indicated a Plasticity Index (PI) value of 19.

- One consolidation test was performed. The load versus deformation plot is provided in Appendix C.
- One direct shear test was performed. The plot is provided in Appendix C.
- Expansion potential testing indicated an expansion index of 47, corresponding to "Low" expansion potential.
- Laboratory compaction of a near-surface bulk sample resulted in a maximum dry density of 120.5 pcf at an optimum moisture content of 10.2 percent.
- Corrosion testing indicated soluble sulfate content of less than 0.02 percent, a chloride content of 60 parts per million (ppm), pH of 6.50 and a minimum resistivity of 1,860 ohm-centimeters.

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

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is generally located within the Peninsular Ranges Geomorphic Province of California, more specifically just north of the Downey Plains region and south of the Puente Hills. The site is located on a younger alluvial fan deposit generated from the nearby canyons in the Puente Hills. Regional topography is mostly flat lying to the south of the site, with the hills to the north of the site defined by the steeper and overturned stratigraphy of the Whittier fault zone (approximately one mile northeast of the site). The trace of the Workman Hill fault is also located further northwest on the northern edge of the Puente Hills. The San Gabriel River is located about 2.4 miles west of the site where it flows in a southwestern direction (CDMG, 1998 and Dibblee, 2001).

2.2 Site-Specific Geology

Based on review of available geologic maps (Dibblee, 2001), the primary geologic unit underlying the site is Holocene age, young alluvial fan deposits. The site is specifically on the Northeastern extent of young alluvial fan deposits emanating from Puente Hills. The fan is largely described as alluvial gravel, sand and silt (Dibblee, 2001). As encountered at the subject site, the alluvial fan deposits generally consist of brown to reddish brown silt, clay and sand with variable amounts of gravel.

2.3 Generalized Subsurface Conditions

The field explorations (borings) indicate minor amounts of undocumented artificial fill soils overlying native alluvial soils. The undocumented artificial fill soils consisted of variable amounts of sand, silt, clay, and gravel, that is brown to grayish brown, slightly moist to moist, and loose to very stiff up to approximately 5 feet below existing grade. The native alluvial soils consisted of primarily silt with varying amounts of sand and clay, that is brown to dark brown, dry to moist, and very stiff to hard for fine-grained soils and medium dense for coarse-grained soils (see Appendix B for Boring Logs).

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

2.4 Groundwater

Groundwater was not encountered to the maximum depth of approximately 46.5 feet below existing ground surface during our subsurface evaluation. Historic high groundwater is approximately 100 feet below current grade per the Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle (CDMG, 1998).

Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present due to local seepage caused by irrigation and/or recent precipitation. Local perched groundwater conditions or surface seepage may develop once site development is completed.

2.5 <u>Seismic Design Criteria</u>

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 33.9786 degrees north and longitude -118.0389 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 2 on the following page. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.85 at a distance of approximately 8.9 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 6.74 at a distance of approximately 13.9 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 0.884g (SEAOC, 2021).

TABLE 2

Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**	**Notes/Exceptions**
Distance to applicable faults classifies the site as a “Near-Fault” site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.852g	From SEAOC, 2021
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.66g	From SEAOC, 2021
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))	1.700	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.852g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	1.122g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.235g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.748g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.897	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.899	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.		

2.6 Faulting

Prompted by damaging earthquakes in California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the State requires detailed fault evaluations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from zones of previous ground rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependant on the distance between the site and causative fault and the onsite geology. Some of the major active nearby faults that could produce these secondary effects include the Whittier, Puente Hills, Compton, Elysian Park, and Anaheim Fault Zones, among others (CGS, 2018). A discussion of these secondary effects is provided in the following sections.

2.6.1 <u>Liquefaction and Dynamic Settlement</u>

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

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1999), the site is <u>not</u> located within a liquefaction hazard zone. Due to the absence of groundwater and the presence of very stiff fine-grained soils in the upper 50 feet, the potential for liquefaction is considered very low to remote.

2.6.2 <u>Lateral Spreading</u>

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move 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 depth to groundwater, very low potential for liquefaction and lack of nearby "free face" conditions, the potential for lateral spreading is also considered very low to remote.

2.7 ***Oversized Material***

Oversized material (material larger than 8 inches in maximum dimension) is not anticipated during site grading. However, if encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E. If feasible, crushing oversized materials onsite or exporting oversized materials may be considered. Special handling recommendations should be provided on a case-by case basis, if encountered.

2.8 ***Expansion Potential***

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

3.0 CONCLUSIONS

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

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

- In general, field explorations (borings) indicate minor amounts of undocumented artificial fill soils overlying native alluvial soils. The undocumented artificial fill soils consisted of variable amounts of sand, silt, clay, and gravel, that is brown to grayish brown, slightly moist to moist, and loose to very stiff up to approximately 5 feet below existing grade. The native alluvial soils consisted of primarily silt with varying amounts of sand and clay, that is brown to dark brown, dry to moist, and very stiff too hard for fine-grained soils and medium dense for coarse-grained soils (see Appendix B for Boring Logs). The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was not encountered during our subsurface evaluation to the maximum explored depth of approximately 46.5 feet below current grade. Historic high groundwater is estimated to be approximately 100 feet below current grade (CDMG, 1998).
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo). The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The site is not located within a State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1999). The potential for liquefaction is considered very low to remote due to the presence of very dense fine-grained soils and the lack of a groundwater in the upper 50 feet.
- Based on the results of preliminary laboratory testing and our experience with similar soils in the area, site soils are anticipated to have “Low” to “Medium” expansion potential. Final design expansion potential must be determined at the completion of grading.
- Pre-soaking of the subgrade for building slabs will be required due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 2 weeks).
- Based on the corrosion test results, soils are not considered corrosive per the Caltrans criteria (Caltrans, 2018).
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. From a geotechnical perspective, the existing onsite soils are suitable material for use as fill, provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- Oversize particles (larger than 8 inches in maximum dimension) are not anticipated; however, if encountered, it will require reduction in size or export from the site.

4.0 PRELIMINARY RECOMMENDATIONS

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

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

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

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of the removal of existing improvements associated with the former land use followed by the required earthwork removals, precise grading and construction of the proposed new improvements, including the residential structures, subsurface utilities, interior streets, etc.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2019 CBC/City of Whittier grading requirements, and the General Earthwork and Grading Specifications included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations should be considered preliminary and may be revised within the future grading plan review report or based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

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

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

4.1.2 Removal and Recompaction Depths and Limits

In order to provide a relatively uniform bearing condition for the planned building structures, upper loose/compressible soils are to be temporarily removed and recompacted as properly compacted fills. Existing undocumented artificial fill within the influence of the proposed structural improvements should be removed to suitable, competent native materials prior to placement of artificial fill to design grades. For preliminary planning purposes, the depth of required removals and recompaction may be estimated as indicated below. It should be noted that updated recommendations may be required based on changes to building layouts and/or grading plan.

Building Structures: We recommend that soils within building pads be removed and recompacted to a minimum depth of 5 feet below existing grade or 3 feet below the base of the foundations, whichever is deeper. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

Minor Site Structures: For minor site structures such as free-standing walls, retaining walls, etc., removal and recompaction should extend a minimum of 3 feet below existing grade or 2 feet below proposed footings, whichever is greater. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance of 3 feet beyond the edges of the proposed minor site structure improvements.

Pavement and Hardscape Areas: Within pavement and hardscape areas, removal and recompaction should extend to a depth of at least 2 feet below existing grade. Removal and recompaction in any design cut areas of the pavement may be reduced by the depth of the design cut but should not be less than 1-foot below the finished subgrade (i.e., below planned aggregate base/asphalt concrete). In general, the envelope for removal and recompaction should extend laterally a minimum lateral distance of 2 feet beyond the edges of the proposed pavement or hardscape improvements.

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

4.1.3 Temporary Excavations

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

Based on our field evaluation, the majority of the site soils within the upper 5 to 10 feet are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person", required by OSHA standards, to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Where proposed improvements will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing "ABC" slot cuts while performing earthwork removal and recompaction. "ABC" slot cuts are defined as excavations perpendicular to sensitive property boundaries that are divided into multiple "slots" of equal width. If slots are labeled A, B, C, A, B, C, etc., then all "A" slots can be excavated at the same time but must be backfilled before all "B" slots can be excavated, etc. Any given slot should be backfilled immediately with properly compacted fill to finish grade prior to excavation of the adjacent two slots. Please note sands susceptible to caving are present at the site. Recommendations for slot cut dimensions should be evaluated during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation. 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 bottom areas and any areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project recommendations.

Removal bottoms, over-excavation bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement. 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 considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of “Low” expansion potential (expansion index 50 or less based on ASTM D 4829), and generally free of organic materials, construction debris and material greater than 3 inches in maximum dimension. Import for 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 planned importation.

Retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a “Very Low” expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. Onsite 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. 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 (“Greenbook”) for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 1-inch in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (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 streets, alleys or non-structural areas (i.e., not within building pad areas).

4.1.6 Placement and Compaction of Fills

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

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing performed by the geotechnical consultant. Oversized material as previously defined should be removed from site fills. During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to at least 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at or slightly above optimum moisture content (generally within optimum and 2 percent above optimum moisture content).

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric (Mirafi 140N or approved alternative) or at the very minimum to be vertically separated from the trench backfill to prevent the migration of fines into the rock backfill.

4.1.7 Trench and Retaining Wall Backfill and Compaction

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

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

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

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

4.1.8 Shrinkage and Subsidence

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

4.2 Preliminary Foundation Recommendations

The site may be considered suitable for the support of the proposed structures using a rigid slab-on-grade conventionally reinforced or post-tensioned slab foundation designed in accordance with Chapter 18 of the 2019 CBC. It should be noted that, as with many structures in Southern California, risk does remain that the proposed structures could suffer some damage as a result of an earthquake. Repair and remedial work may be required after a seismic event.

The following sections summarize our preliminary recommendations. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading. The proposed foundations should be designed by the foundation engineer in accordance with the following recommendations. The following recommendations may be superseded by the requirements of the foundation engineer, structural engineer and/or local jurisdictions. Proposed foundations should be designed to accommodate estimated site static settlements.

4.2.1 Provisional Conventional Foundation Design Parameters

Given that the expansion index exceeds 20, the foundation systems shall be designed for effects of expansive soil. Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2019 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 25
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer.
- Moisture condition slab subgrade soils to 120 % of optimum moisture content to a depth of 18 inches prior to trenching for footings.

Other types of stiff slabs may be used in place of the WRI design procedure provided that, in the opinion of the foundation structural designer, the alternative of slab is at least as stiff and strong as that designed by the WRI to resist expansive soils.

4.2.2 Post-Tensioned Foundation Design Parameters

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

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

TABLE 3

Provisional Geotechnical Parameters for Post-Tensioned Foundation Slab Design with "Medium" Expansion Potential Subgrade Soils

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	Medium	Medium
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 9.0 feet 0.5 inch	 9.0 feet 0.6 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 4.7 feet 1.1 inch	 4.7 feet 1.3 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	150 pci	150 pci
Minimum perimeter footing/thickened edge embedment below finish grade	18 inches	6 inches
1. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations. 2. The sand layer requirements are the purview of the foundation engineer/structural engineer and should be provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction". 3. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements. 4. Moisture condition to 120% of optimum moisture content to a depth of 18 inches prior to trenching.		

4.2.3 Foundation Subgrade Preparation and Maintenance

Moisture conditioning of the subgrade for building slabs will be required due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 2 weeks). The subgrade moisture condition of the building pad soils should be maintained at the recommended moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures. As an alternative to presoaking, the upper 18 inches of subgrade soils may be placed at a higher moisture content and maintained up until the time of concrete placement. The upper 18 inches of subgrade would have to be placed at 120% of optimum moisture content, compacted to

90 percent relative compaction per the project specifications and maintained up until the time of concrete placement.

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

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

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

4.2.4 Slab Underlayment Guidelines

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

4.3 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment of 150 psf for each additional foot of foundation width to a maximum value of 2,500 psf. A mat foundation a minimum of 6 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

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

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.3 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 225 psf per foot of depth (or pcf) to a maximum of 2,250 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 300 pcf (maximum of 3,000 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.4 Lateral Earth Pressures for Retaining Walls

Lateral earth pressures for approved native sandy or import soils meeting indicated project requirements are provided below. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented in Table 4 on the following page for approved granular 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 imported select sandy soil backfill criteria.

TABLE 4

Lateral Earth Pressures – Approved Imported Sandy Soils

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

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

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near-surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care and information should be provided to homeowners to maintain these drains. Typical retaining wall drainage is illustrated in Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water containing soluble salts migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential. Please note that 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 80 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 engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use seismic lateral earth pressure increment of 10 or 20 pcf for level backfill or 2:1 sloped backfill conditions, respectively. These increments should be applied (in addition to the provided static lateral earth pressure) using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). Per Section 1803.5.12 of the 2019 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.5 Control of Surface Water and Drainage Control

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

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

4.6 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 (2017), 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 5 below. The design infiltration rate is the measured infiltration rate divided by the total reduction factor (RF_t x RF_v x RF_s).

<u>TABLE 5</u>

<u>Shallow Surface Infiltration - Reduction Factors Applied to Measured Infiltration Rate</u>

Consideration	Reduction Factor
Test procedure, boring percolation, RF_t	2
Site variability, number of tests, etc., RF_v	1.5
Long-term siltation plugging and maintenance, RF_s	Per Infiltration Designer
Total Reduction Factor, RF = RF_t x RF_v x RF_s	**TBD**

Per the requirements of the Los Angeles County testing guidelines (2017), subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. The test procedure and site variability considerations (RF_t and RF_v) result in a minimum reduction factor of 3 (not including long-term siltation plugging and maintenance). When total reduction factor is applied to the measured infiltration rates presented in Table 1, the resulting design infiltration rate is anticipated to be less than the minimum required by the County of Los Angeles for infiltration.

Based on the results of field infiltration testing indication of extremely low infiltration rates (less than the minimum County design infiltration rate) and the presence of low permeability silts and clays to maximum explored depth of approximately 46.5 feet below existing grade, we strongly recommend against the intentional infiltration of stormwater into the subsurface soils.

4.7 ***Preliminary Asphalt Pavement Sections***

For the purpose of these preliminary recommendations, we have selected a preliminary design R-value of 25 (assumed) and calculated pavement sections for Traffic Index (TI) of 5.0 (or less) and 5.5. The California Department of Transportation Highway Design Manual (Caltrans, 2017) allows for a maximum R-Value of 50 to be used in pavement design. These recommendations must be confirmed with R-Value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final street sections should be confirmed by the project civil engineer based upon the final design Traffic Index. Determination of the TI is not the purview of the geotechnical consultant. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 6

Preliminary Pavement Sections

Assumed Traffic Index	5.0 or less	5.5
R -Value Subgrade	25	25
AC Thickness	4.0 inches	4.0 inches
Base Thickness	5.0 inches	6.0 inches

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

Earthwork recommendations regarding aggregate base and subgrade are provided in Section 4.1 "Site Earthwork" and the related sub-sections of this report.

4.8 ***Soil Corrosivity***

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

Corrosion testing of near-surface bulk samples indicated soluble sulfate contents less than 0.02 percent, a chloride content of 60 parts per million (ppm), pH of 6.50 and minimum resistivity of 1,860 ohm-centimeters. Based on Caltrans Corrosion Guidelines (Caltrans, 2018), soils are considered corrosive to structural elements if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 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.

4.9 Nonstructural Concrete Flatwork

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

TABLE 7

Nonstructural Concrete Flatwork for Medium Expansion Potential

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

Aggregate Base Thickness (in.)	—	—	City/Agency Standard

4.10 ***Geotechnical Plan Review***

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

4.11 ***Geotechnical Observation and Testing During Construction***

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing 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;
- Preparation of pavement subgrade and placement of aggregate base and asphalt concrete pavement;
- After building and wall footing excavation and prior to placing reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

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

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

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

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

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

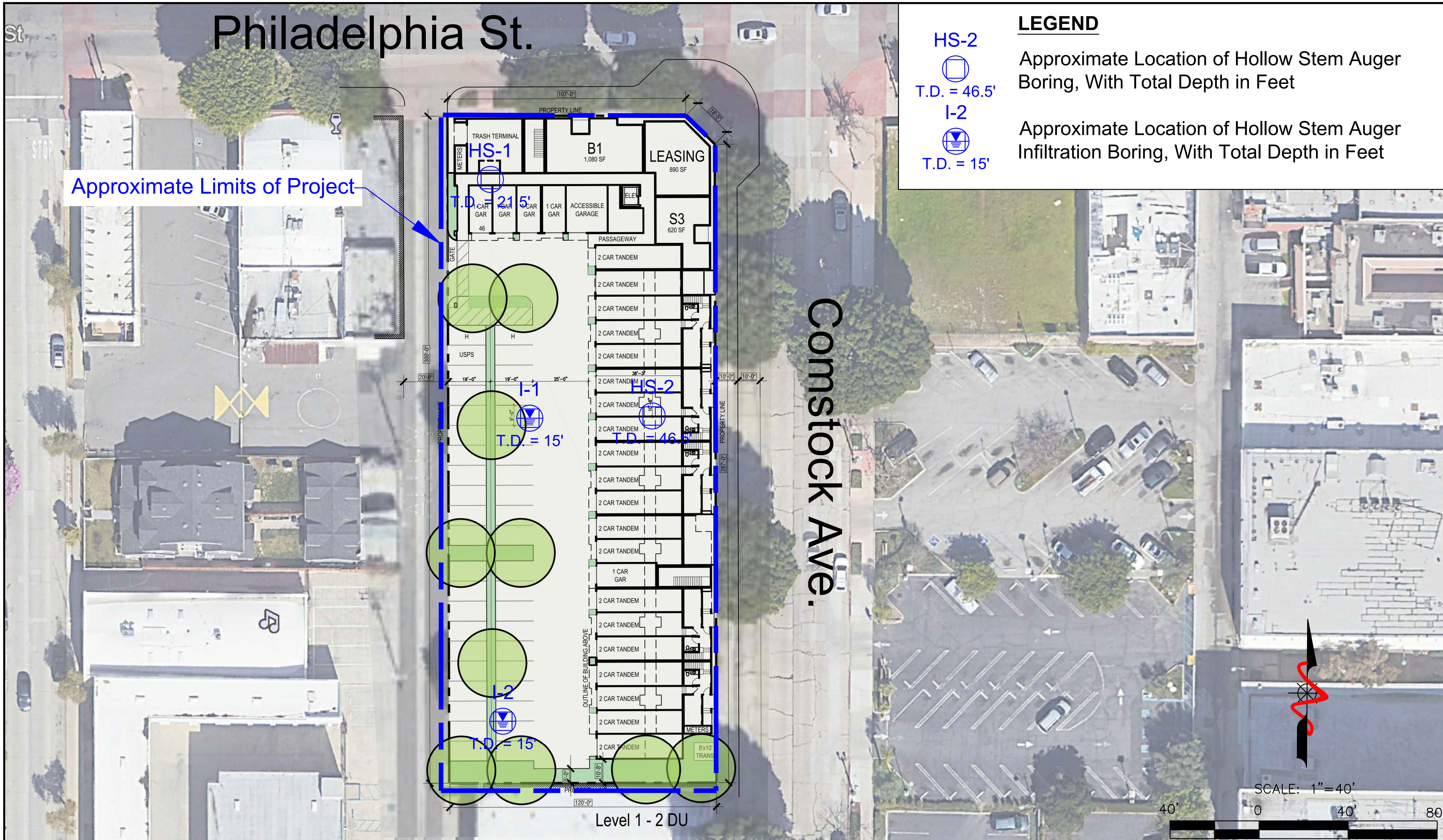


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Figure 2
Boring Location Map

PROJECT NAME	MWIG - Whittier
PROJECT NO.	21051-01
ENG. / GEOL.	RLD
SCALE	1:40
DATE	May 2021

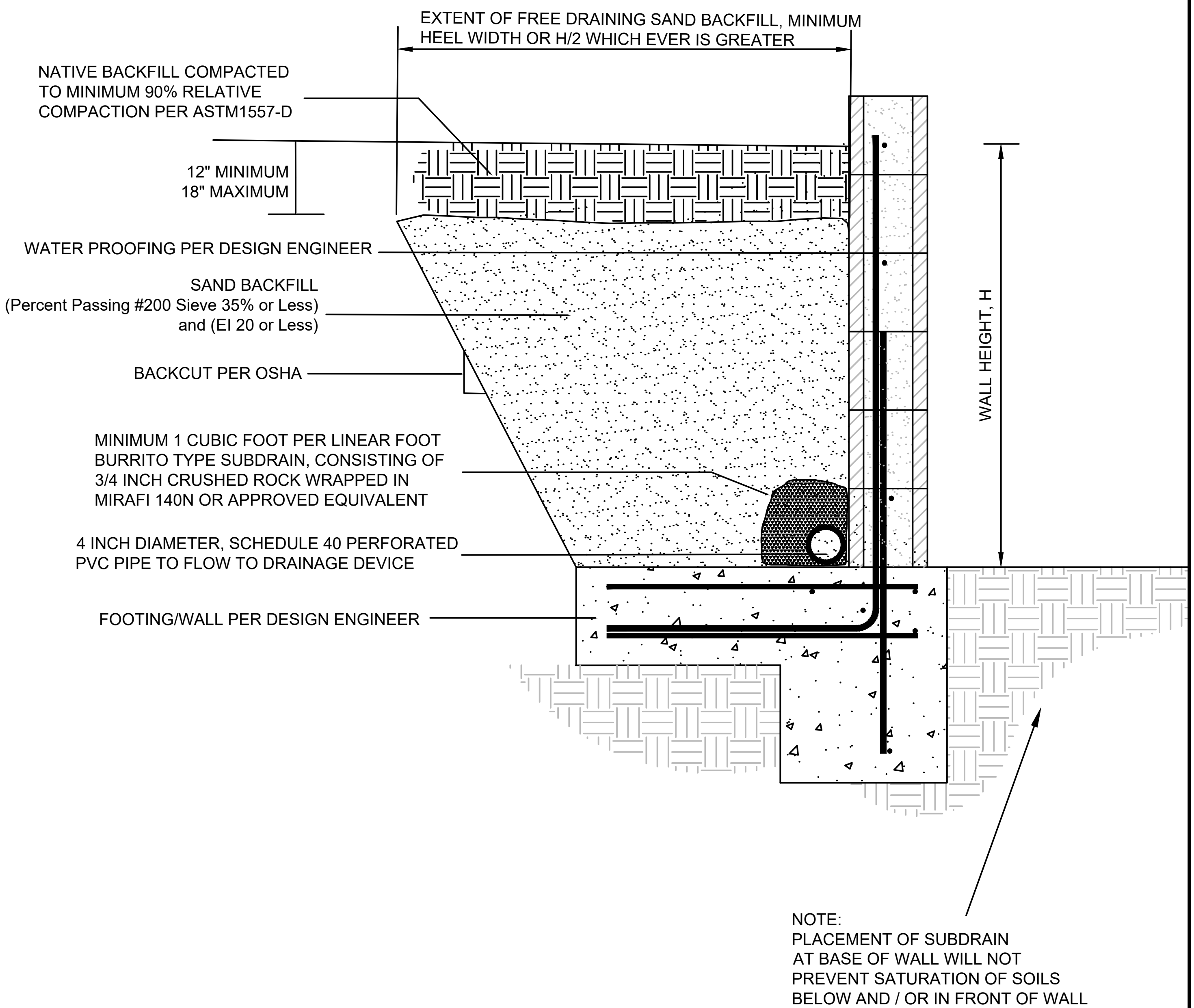

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FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	MWIG - Whittier
PROJECT NO.	21051-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	May 2021

Appendix A
References

APPENDIX A

References

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Appendix B
Field Exploration Logs

Geotechnical Boring Log Borehole HS-1

Date: 4/6/2021	**Drilling Company:** Cal Pac Drilling
Project Name: MWIG - Whittier	**Type of Rig:** CME 75
Project Number: 21051-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~319' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By MJG
Sampled By MJG
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0		B-1					**@0' to 5'- Undocumented Artificial Fill (afu):**	MD EI CR DS
								@0'- 3 inches of Asphalt over 9 inches of Base	
			R-1	6 10 12	111.4	11.0	ML	@2.5'- Sandy SILT: brown, moist, very stiff	
315	5		R-2	8 17 31	116.6	15.5	CL	**@5' to T.D.- Quaternary Alluvium (Qa):** @5'- CLAY with Sand: brown, moist, hard	
			R-3	7 14 11	111.2	12.9		@7.5'- CLAY: dark yellowish brown, moist, very stiff	CO AL
310	10		SPT-1	3 3 5		3.5	SM	@10'- Silty SAND: dusky brown, dry, medium dense	
305	15		R-1	17 40 50/3"	119.6	14.7	CL	@15'- CLAY with Gravel: brown, moist, hard	
300	20		SPT-2	8 10 10		8.5	ML	@20'- Sandy SILT: dusky brown, slightly moist, very stiff	
295	25							Total Depth = 21.5' Groundwater Not Encountered Backfilled with Cuttings on 4/6/2021	
290	30								

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

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

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

Last Edited: 5/3/2021

Geotechnical Boring Log Borehole HS-2

Date: 4/6/2021	**Drilling Company:** Cal Pac Drilling
Project Name: MWIG - Whittier	**Type of Rig:** CME 75
Project Number: 21051-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~319' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By MJG
Sampled By MJG
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						**@0' to 5'- Undocumented Artificial Fill (afu):** @0'- 2 inches of Asphalt over 3 inches of Base	
			R-1	6 8 4	104.9	5.8	SP	@2.5'- SAND with Gravel: gray/brown, slightly moist, loose	
315	5		R-2	3 7 11	110.8	1.4	SP	**@5' to T.D.- Quaternary Alluvium (Qa):** @5'- SAND with Gravel: dusky gray/brown, dry, medium dense	
			R-3	11 16 23	117.4	14.0	ML	@7.5'- Sandy SILT: brown, moist, hard	
310	10		R-4	10 20 26	118.5	14.4		@10'- Sandy SILT: brown, moist, hard	
305	15		SPT-1	5 7 12		17.0		@15'- Sandy SILT: brown, very moist, very stiff	-#200
300	20		R-5	26 50/5"	115.8	14.9		@20'- Sandy SILT: brown, moist, hard	
295	25		SPT-2	8 13 16		14.4		@25'- Sandy SILT: dusky brown, moist, hard	
290	30								

LGC Geotechnical, Inc.

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

Geotechnical Boring Log Borehole HS-2

Date: 4/6/2021	**Drilling Company:** Cal Pac Drilling
Project Name: MWIG - Whittier	**Type of Rig:** CME 75
Project Number: 21051-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~319' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	

Logged By MJG
Sampled By MJG
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-6	16 39 50/4"	118.0	15.0	CL	@30'- CLAY with Sand: brown, moist, hard	
285	35		SPT-3	8 12 10		11.8	ML	@35'- Sandy SILT: brown, moist, very stiff	-#200
280	40		R-7	9 23 50/6"	111.1	9.8		@40'- Sandy SILT: pinkish brown, slightly moist, hard	
275	45		SPT-4	10 21 41		15.1	CL	@45'- CLAY: reddish brown, moist, hard @46.5'- Refusal	
270	50							Total Depth = 46.5' Groundwater Not Encountered Backfilled with Cuttings on 4/6/2021	
265	55								
260	60								

Geotechnical, Inc.

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SAMPLE TYPES:
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- G GRAB SAMPLE
- SPT STANDARD PENETRATION TEST SAMPLE
- GROUNDWATER TABLE

TEST TYPES:
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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 I-1

Date: 4/6/2021	**Drilling Company:** Cal Pac Drilling
Project Name: MWIG - Whittier	**Type of Rig:** CME 75
Project Number: 21051-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~316' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By MJG
Sampled By MJG
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
315	0							**@0' to 5'- <u>Undocumented Artificial Fill (afu):</u>** @0'- 4 inches of Asphalt over 5 inches of Base	
310	5		SPT-1	7 11 14		14.0	CL	**@5' to T.D.- <u>Quaternary Alluvium (Qa):</u>** @5'- CLAY with Sand: dark brown, moist, hard	
305	10		R-1	6 22 34	112.3	12.4	ML	@10'- Sandy SILT: dusky brown, moist, hard	
			SPT-2	13 19 19		10.7	CL	@13'- Sandy CLAY: brown, slightly moist, hard	
300	15							Total Depth = 15' Groundwater Not Encountered Pipe Pulled and Backfilled with Cuttings on 4/7/2021	
295	20								
290	25								
	30								

LGC Geotechnical, Inc.

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SAMPLE TYPES:

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

TEST TYPES:

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

Last Edited: 5/3/2021

Geotechnical Boring Log Borehole I-2

Date: 4/6/2021	**Drilling Company:** Cal Pac Drilling
Project Name: MWIG - Whittier	**Type of Rig:** CME 75
Project Number: 21051-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~313' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By MJG
Sampled By MJG
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**@0' to 5'- Undocumented Artificial Fill (afu):** @0'- 6 inches of Asphalt over 5 inches of base	
310									
	5		R-1	9 11 18	111.2	16.5	CL	**@5' to T.D.- Quaternary Alluvium (Qa):** @5'- Sandy CLAY: brown, moist, very stiff	
305									
	10		SPT-1	5 8 8		12.0	ML	@10'- Sandy SILT: dusky brown, moist, very stiff	
300			R-2	13 23 30	119.0	10.1		@13'- Sandy SILT: dusky brown, moist, hard	
	15							Total Depth = 15' Groundwater Not Encountered Pipe Pulled and Backfilled with Cuttings on 4/7/2021	
295									
	20								
290									
	25								
285									
	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
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EI EXPANSION INDEX
CN CONSOLIDATION
CR CORROSION
AL ATTERBERG LIMITS
CO COLLAPSE/SWELL
RV R-VALUE
-#200 % PASSING # 200 SIEVE

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 a selected sample 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-1 @ 1-5 feet	47	Low

* ASTM D4829

Grain Size Distibution/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-2 @ 15 feet	Sandy Silt	82
HS-2 @ 35 feet	Sandy Silt	60

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.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-1 @ 7.5 ft	35	16	19	CL

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

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

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-1 @ 1-5 feet	Dark Yellowish Brown Clay with Sand	120.5	10.2

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-1 @ 1-5 feet	60

Laboratory Testing Procedures and Test Results

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

Sample Location	Sulfate Content (ppm)	Sulfate Exposure Class *
HS-1 @ 1-5 feet	140	S0

*Based on ACI 318R-14, Table 19.3.1.1

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

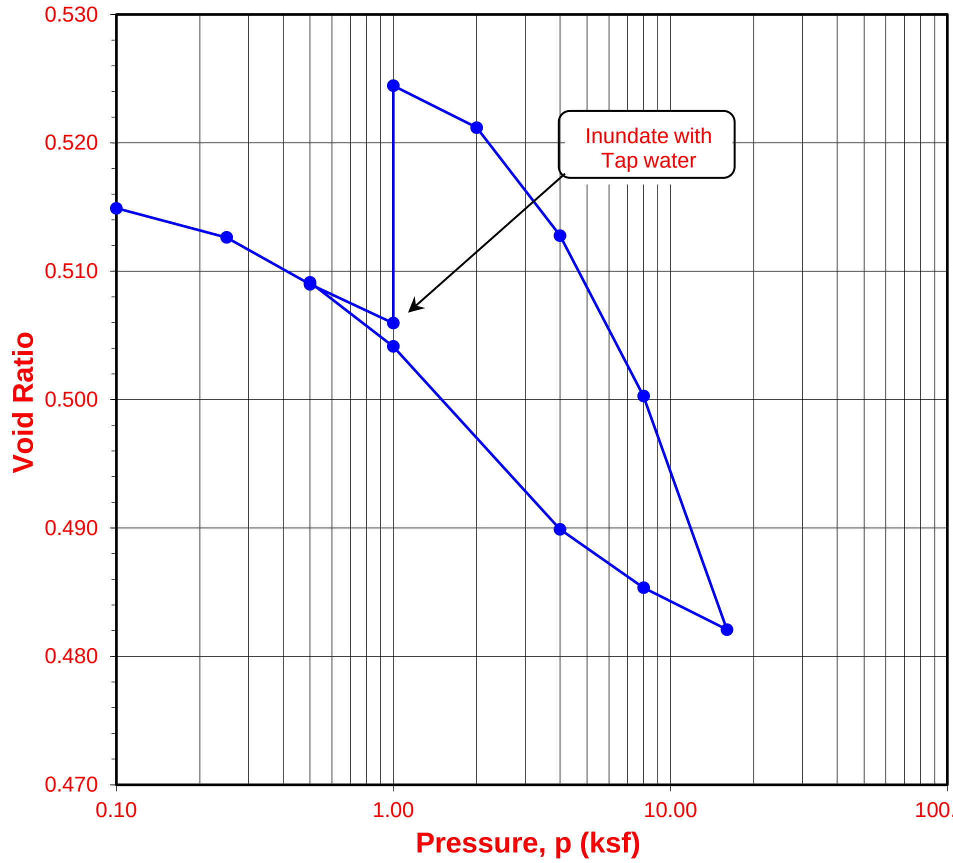
Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-1 @ 1-5 feet	6.50	1860

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Whittier	Tested By:	GB/YN	Date:	04/13/21
Project No.:	21051-01	Checked By:	J. Ward	Date:	05/04/21
Boring No.:	HS-1	Depth (ft.):	7.5		
Sample No.:	R-3	Sample Type:	Ring		
Soil Identification:	Dark yellowish brown lean clay (CL)				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	194.04
Weight of Ring (g)	43.10
Height after consol. (in.)	0.9958
Before Test	
Wt.Wet Sample+Cont. (g)	297.31
Wt.of Dry Sample+Cont. (g)	272.29
Weight of Container (g)	77.78
Initial Moisture Content (%)	12.9
Initial Dry Density (pcf)	111.2
Initial Saturation (%)	67
Initial Vertical Reading (in.)	0.3014
After Test	
Wt.of Wet Sample+Cont. (g)	264.23
Wt. of Dry Sample+Cont. (g)	241.02
Weight of Container (g)	64.37
Final Moisture Content (%)	17.38
Final Dry Density (pcf)	111.5
Final Saturation (%)	92
Final Vertical Reading (in.)	0.2937
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

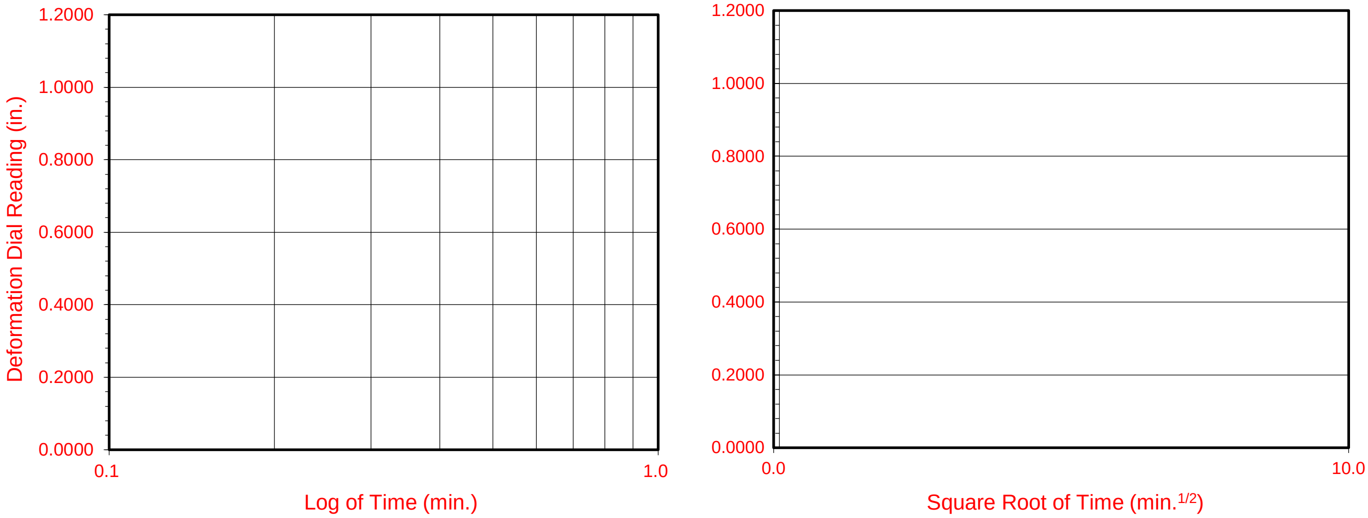


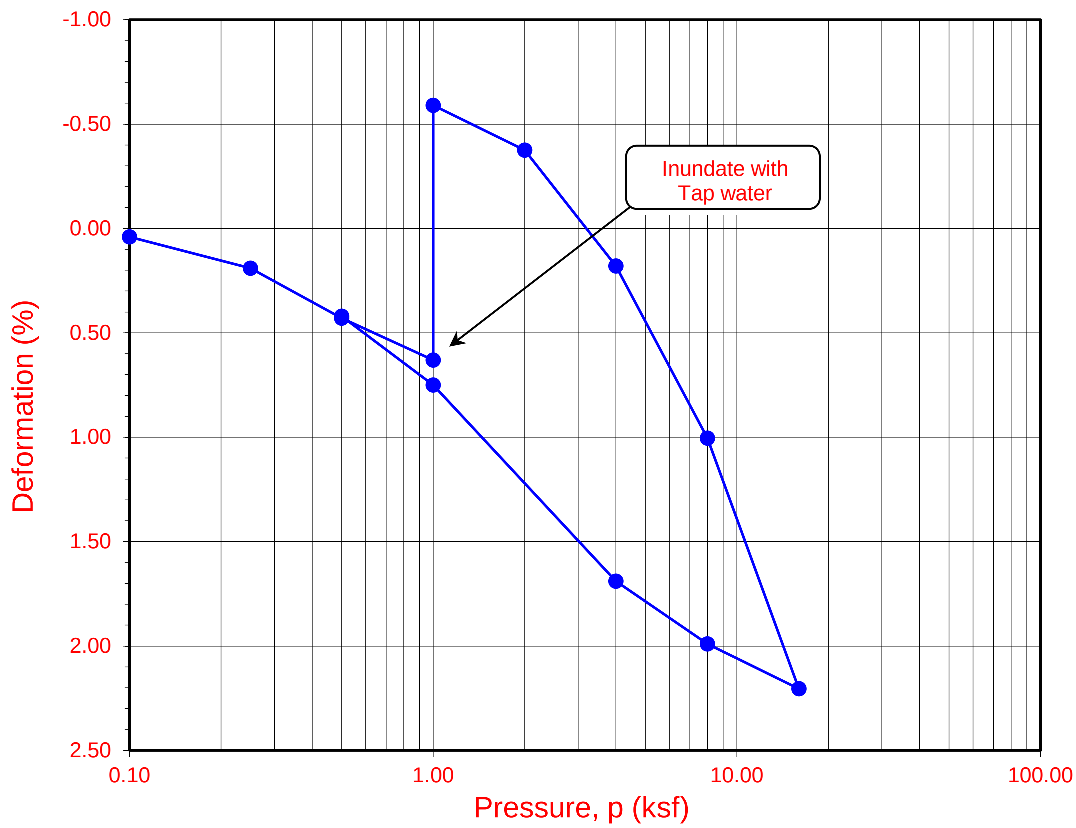
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3010	0.9996	0.00	0.04	0.515	0.04
0.25	0.2992	0.9978	0.03	0.22	0.513	0.19
0.50	0.2965	0.9951	0.06	0.49	0.509	0.43
1.00	0.2940	0.9926	0.11	0.74	0.506	0.63
1.00	0.3062	1.0048	0.11	-0.48	0.524	-0.59
2.00	0.3032	1.0018	0.20	-0.17	0.521	-0.37
4.00	0.2963	0.9949	0.33	0.51	0.513	0.18
8.00	0.2866	0.9852	0.48	1.49	0.500	1.01
16.00	0.2727	0.9713	0.67	2.88	0.482	2.21
8.00	0.2757	0.9743	0.58	2.57	0.485	1.99
4.00	0.2795	0.9781	0.50	2.19	0.490	1.69
1.00	0.2900	0.9886	0.39	1.14	0.504	0.75
0.50	0.2937	0.9923	0.35	0.77	0.509	0.42

Time Readings

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

Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-1	**R-3**	**7.5**	**12.9**	**17.4**	**111.2**	**111.5**	**0.516**	**0.509**	**67**	**92**

Soil Identification: Dark yellowish brown lean clay (CL)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 21051-01

Whittier

05-21

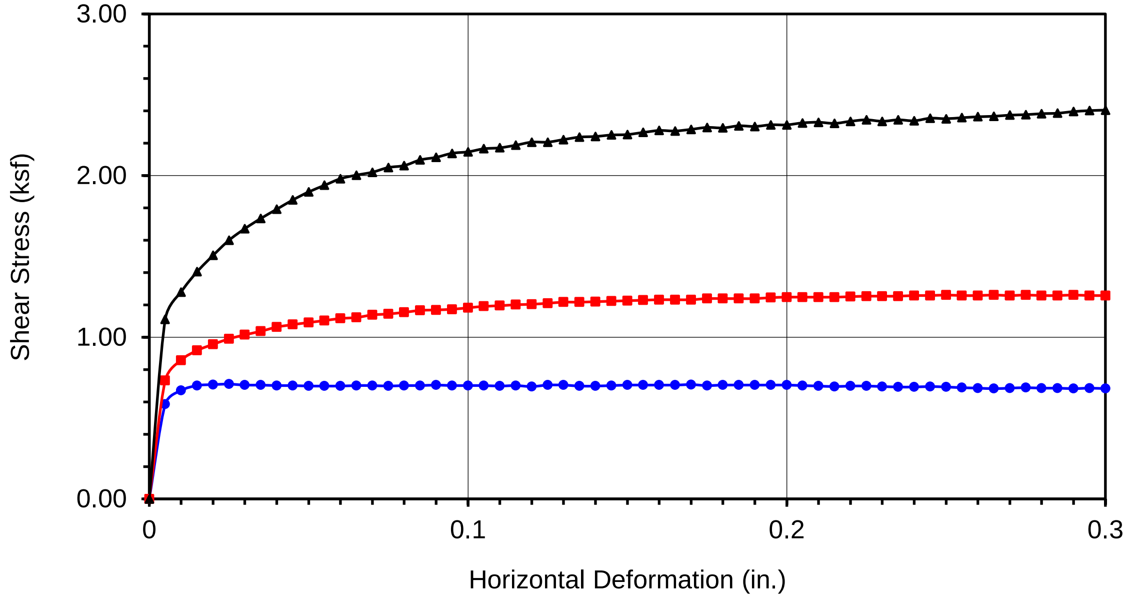
DIRECT SHEAR TEST

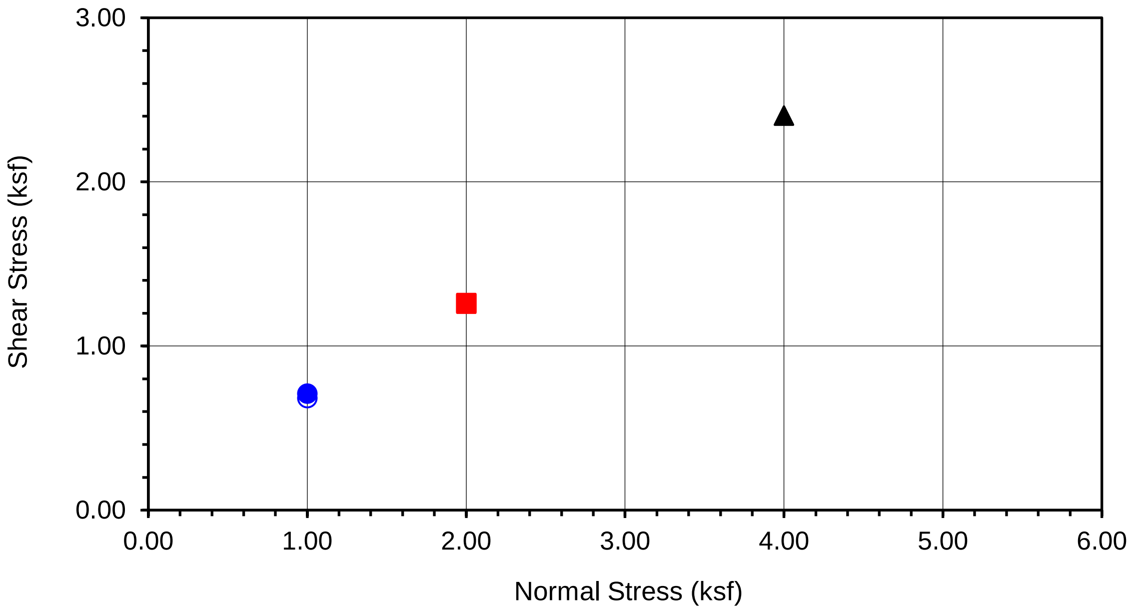
Consolidated Drained - ASTM D 3080

Project Name:	Whittier	Tested By:	G. Bathala	Date:	04/19/21
Project No.:	21051-01	Checked By:	J. Ward	Date:	05/04/21
Boring No.:	HS-1	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	1-5		

Soil Identification: Dark yellowish brown lean clay with sand (CL)s

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	186.81	187.77	188.30
Weight of Ring(gm):	44.44	45.35	45.45
Before Shearing			
Weight of Wet Sample+Cont.(gm):	157.30	157.30	157.30
Weight of Dry Sample+Cont.(gm):	148.05	148.05	148.05
Weight of Container(gm):	61.21	61.21	61.21
Vertical Rdg.(in): Initial	0.0000	0.2442	0.2661
Vertical Rdg.(in): Final	0.0153	0.2558	0.2892
After Shearing			
Weight of Wet Sample+Cont.(gm):	226.79	218.56	214.49
Weight of Dry Sample+Cont.(gm):	200.66	193.84	191.46
Weight of Container(gm):	72.43	65.76	63.41
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**HS-1**
Sample No.	**B-1**
Depth (ft)	**1-5**

Sample Type:

90% Remold

Soil Identification:
Dark yellowish brown lean clay with sand (CL)s

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.710	■ 1.261	▲ 2.405
Shear Stress @ End of Test (ksf)	○ 0.682	□ 1.258	△ 2.405
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.65	10.65	10.65
Dry Density (pcf)	107.0	107.0	107.4
Saturation (%)	50.0	50.0	50.5
Soil Height Before Shearing (in.)	1.0153	0.9884	0.9769
Final Moisture Content (%)	20.4	19.3	18.0

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 21051-01

Whittier

04-21

Appendix D
Infiltration Test Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: MWIG - Whittier
Project Number: 21051-01
Date: 4/8/2021
Location: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	15
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	8:23	8:33	10.0	12.93	13.00	0.07	
PS-2	8:33	8:43	10.0	13	13.05	0.05	
Pre-Test	8:43	8:53	10.0	13.05	13.09	0.04	

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	8:56	9:26	30.0	12.90	13.03	0.13	4.75	0.2
2	9:26	9:56	30.0	13.03	13.10	0.07	4.48	0.1
3	9:56	10:26	30.0	13.10	13.19	0.09	4.33	0.2
4	10:26	10:56	30.0	13.06	13.14	0.08	4.41	0.2
5	10:56	11:26	30.0	12.93	13.03	0.10	4.68	0.2
6	11:26	11:56	30.0	13.03	13.12	0.09	4.48	0.2
7	11:56	12:26	30.0	12.93	13.02	0.09	4.68	0.2
8	12:26	12:56	30.0	13.02	13.10	0.08	4.50	0.1
9								
10								
11								
12								

Measured Infiltration Rate	**0.2**
Feasibility Reduction Factor	See Report
Feasibility Infiltration Rate	**See Report**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017
Spreadsheet Revised on: 12/23/2019

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: MWIG - Whittier
Project Number: 21051-01
Date: 4/8/2021
Location: I-2

Test hole dimensions (if circular)

Boring Depth (feet)*:	15
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	8:29	8:39	10.0	11.91	11.95	0.04	
PS-2	8:39	8:49	10.0	11.95	11.98	0.03	
Pre-Test	8:49	8:59	10.0	11.98	12.01	0.03	

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	9:00	9:30	30.0	12.01	12.11	0.10	6.61	0.1
2	9:30	10:00	30.0	12.11	12.20	0.09	6.40	0.1
3	10:00	10:30	30.0	12.20	12.27	0.07	6.21	0.1
4	10:30	11:00	30.0	12.10	12.19	0.09	6.42	0.1
5	11:00	11:30	30.0	11.96	12.05	0.09	6.72	0.1
6	11:30	12:00	30.0	12.05	12.13	0.08	6.53	0.1
7	12:00	12:30	30.0	12.13	12.22	0.09	6.36	0.1
8	12:30	13:00	30.0	12.09	12.17	0.08	6.44	0.1
9								
10								
11								
12								

Measured Infiltration Rate	**0.1**
Feasibility Reduction Factor	See Report
Feasibility Infiltration Rate	**See Report**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2017
Spreadsheet Revised on: 12/23/2019

LGC Geotechnical, Inc.

Appendix E
General Earthwork & Grading Specifications for Rough Grading

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

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

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

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

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

2.2 Processing

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

2.3 Over-excavation

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

2.4 Benching

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

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 *Import*

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

4.0 *Fill Placement and Compaction*

4.1 *Fill Layers*

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

4.2 *Fill Moisture Conditioning*

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

4.3 *Compaction of Fill*

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

4.4 *Compaction of Fill Slopes*

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

4.5 *Compaction Testing*

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

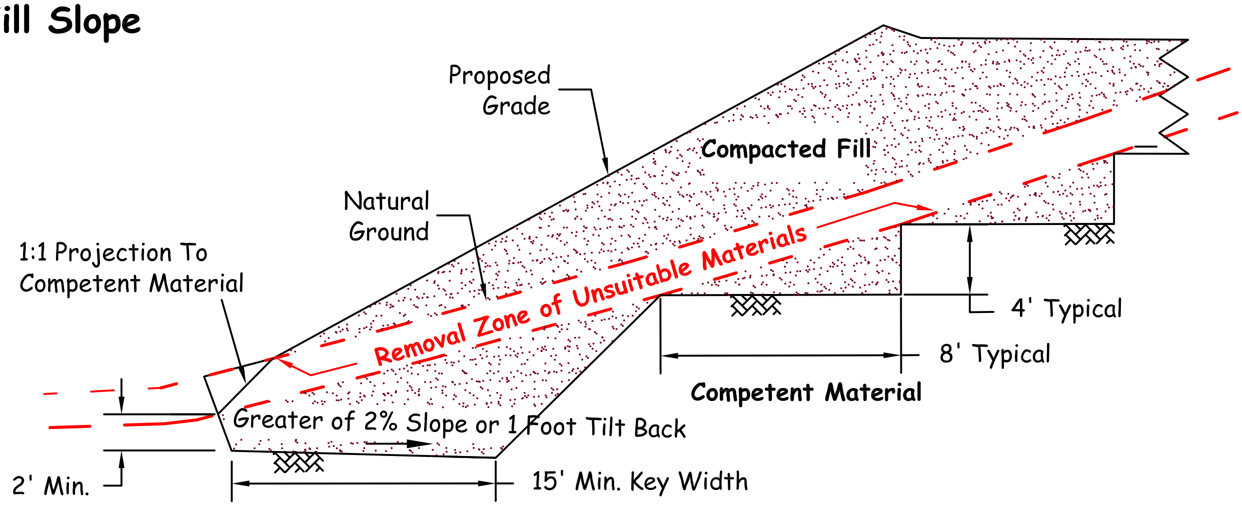
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

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

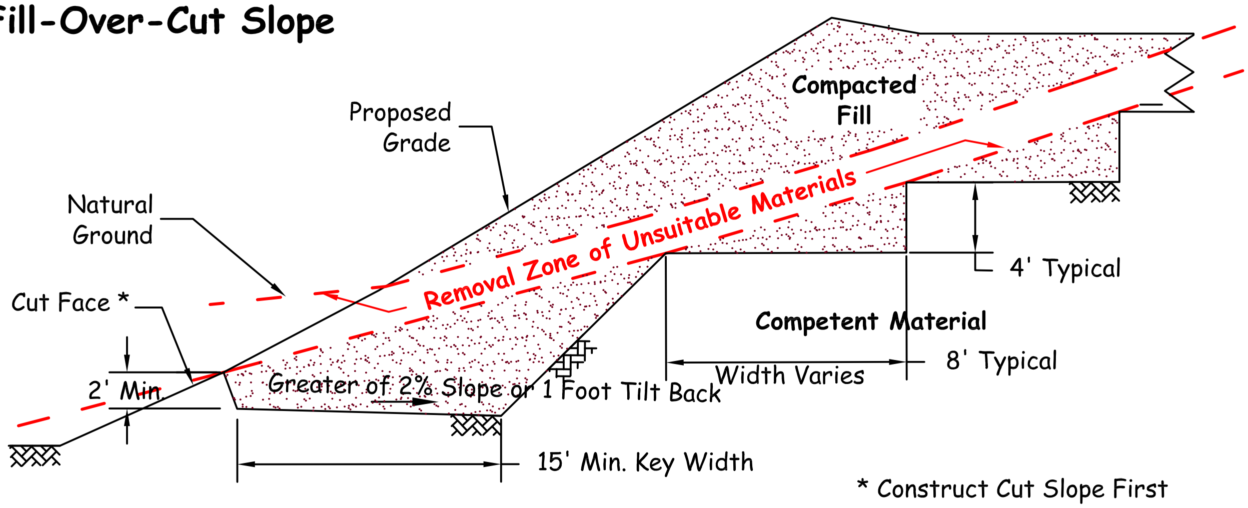
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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

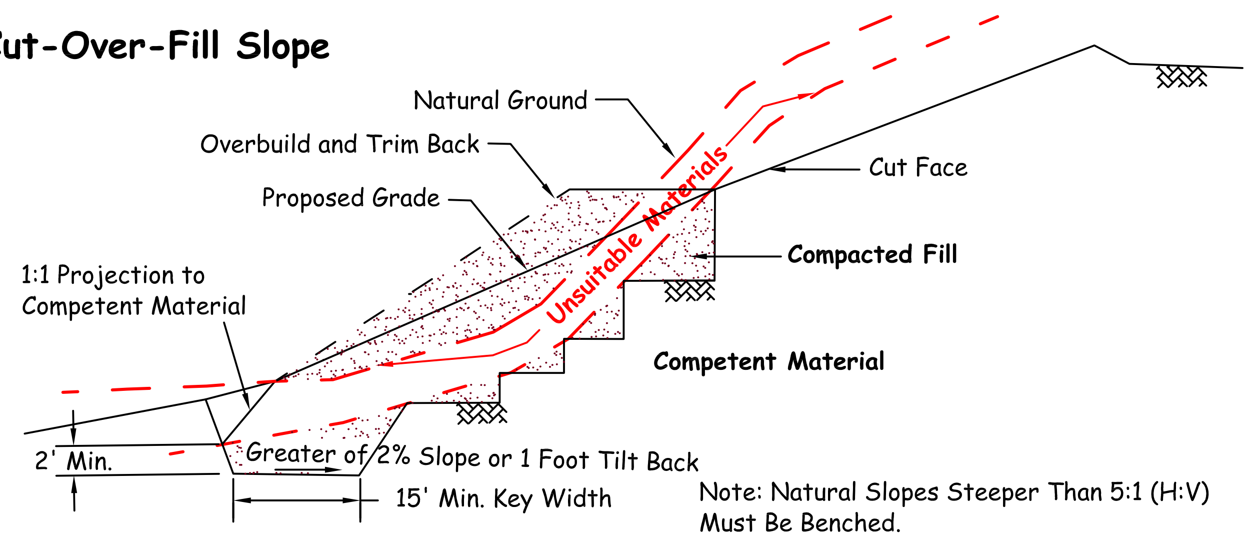
Fill Slope



Fill-Over-Cut Slope



Cut-Over-Fill Slope



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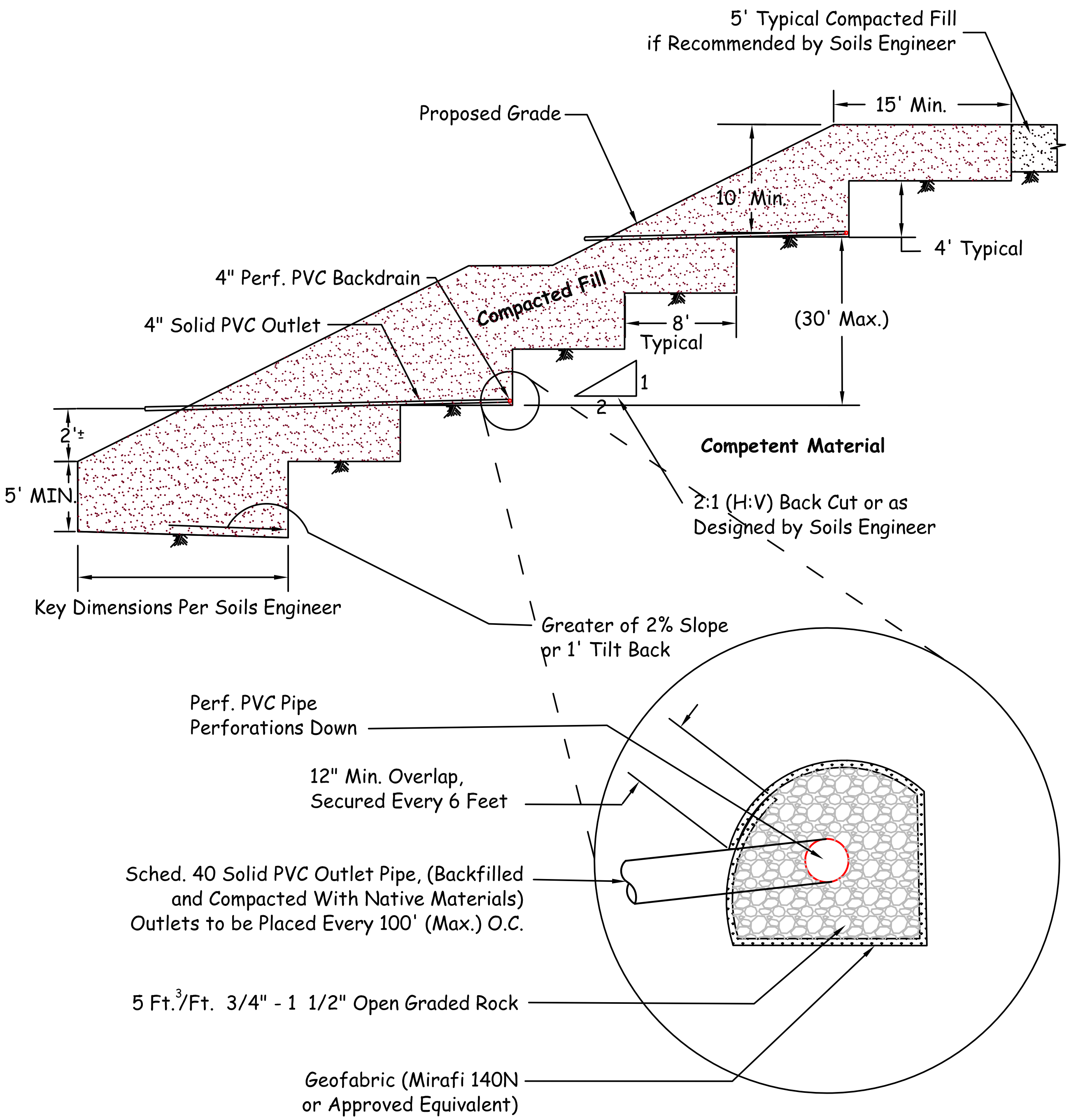
KEYING AND BENCHING



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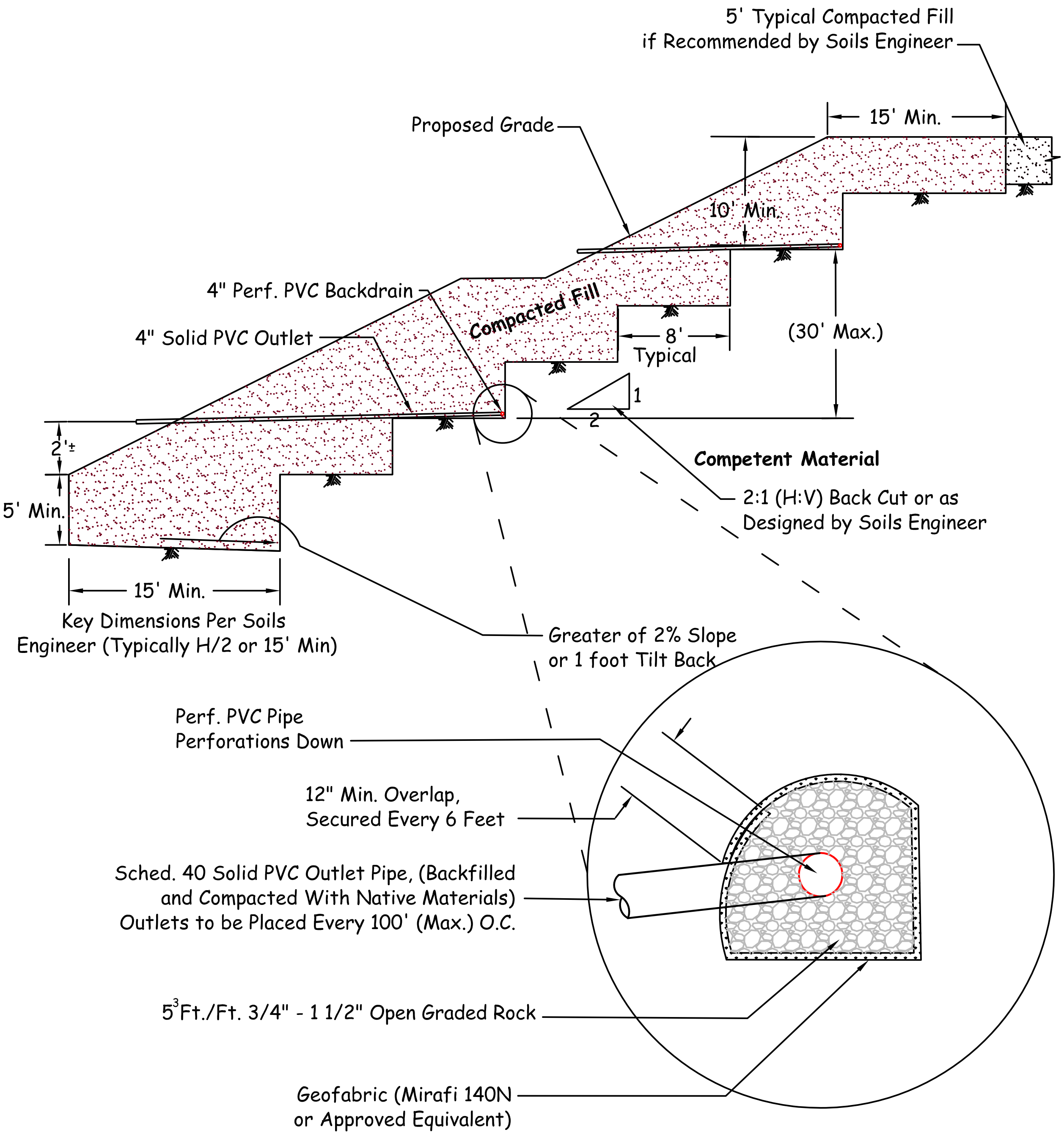
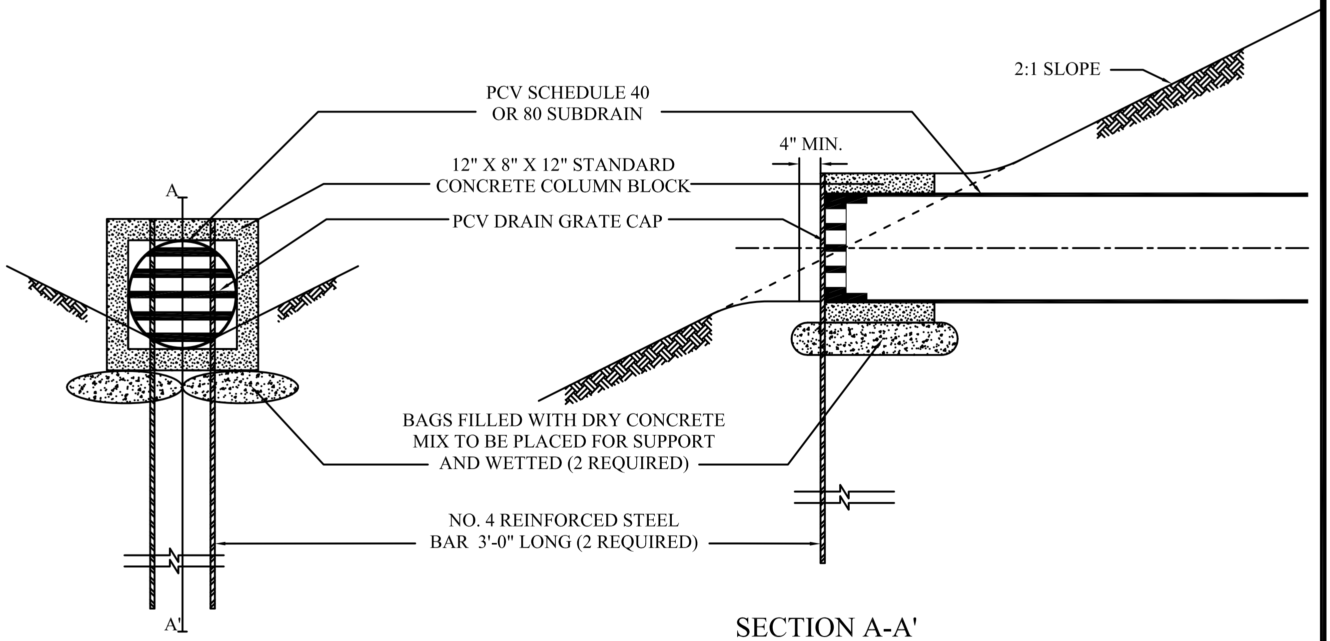
TYPICAL BUTTRESS DETAIL



LGC Geotechnical, Inc.

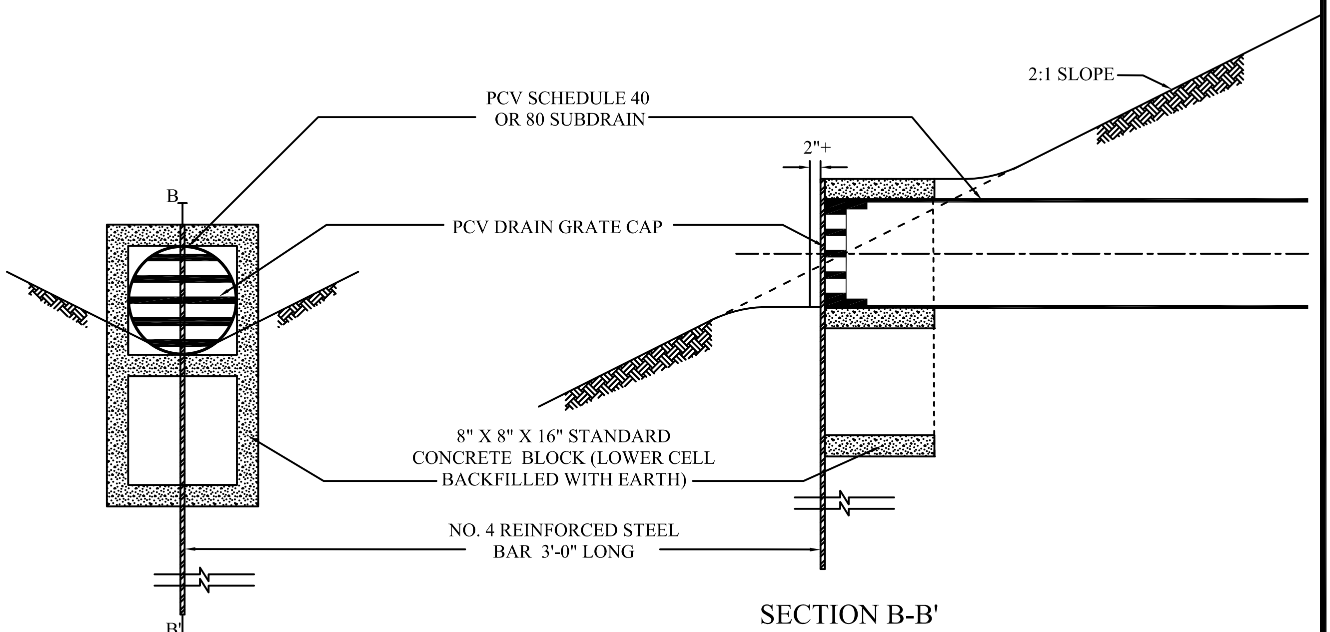
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SECTION A-A'

SUBDRAIN OUTLET MARKER -4" PIPE



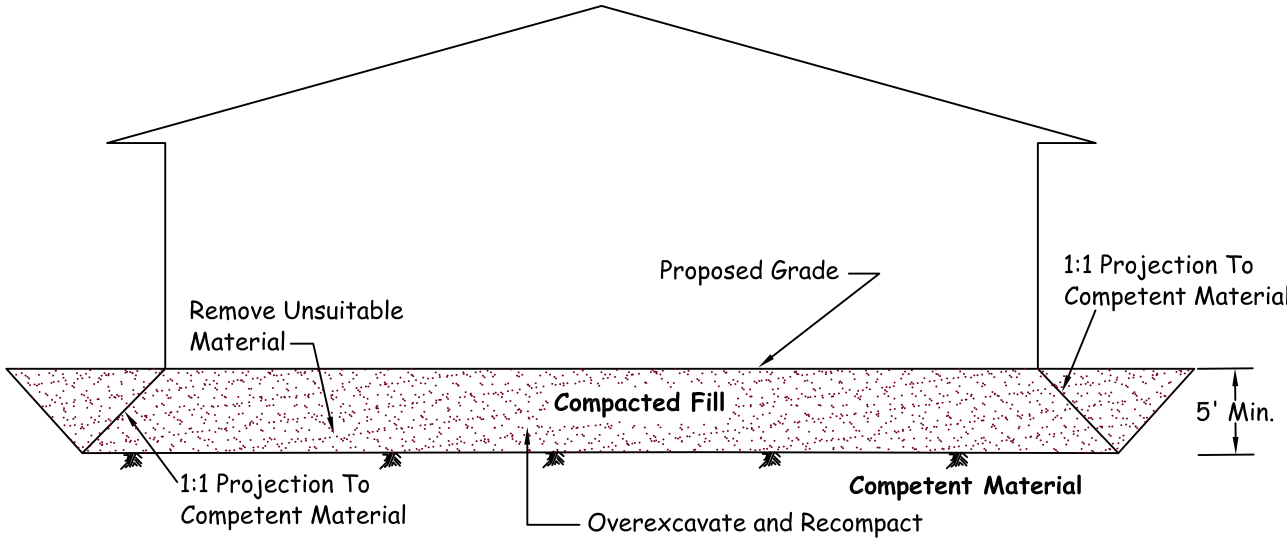
SECTION B-B'

NOT TO SCALE

LGC Geotechnical, Inc.

SUBDRAIN OUTLET MARKER DETAIL

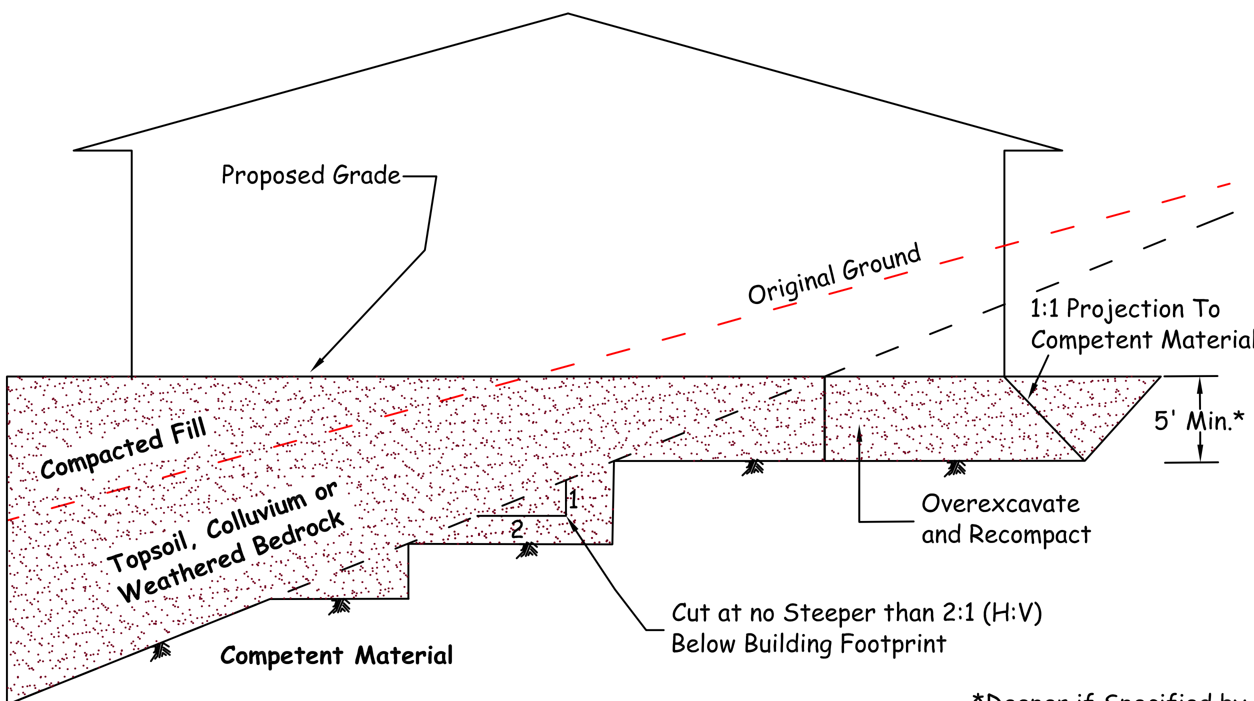
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



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.

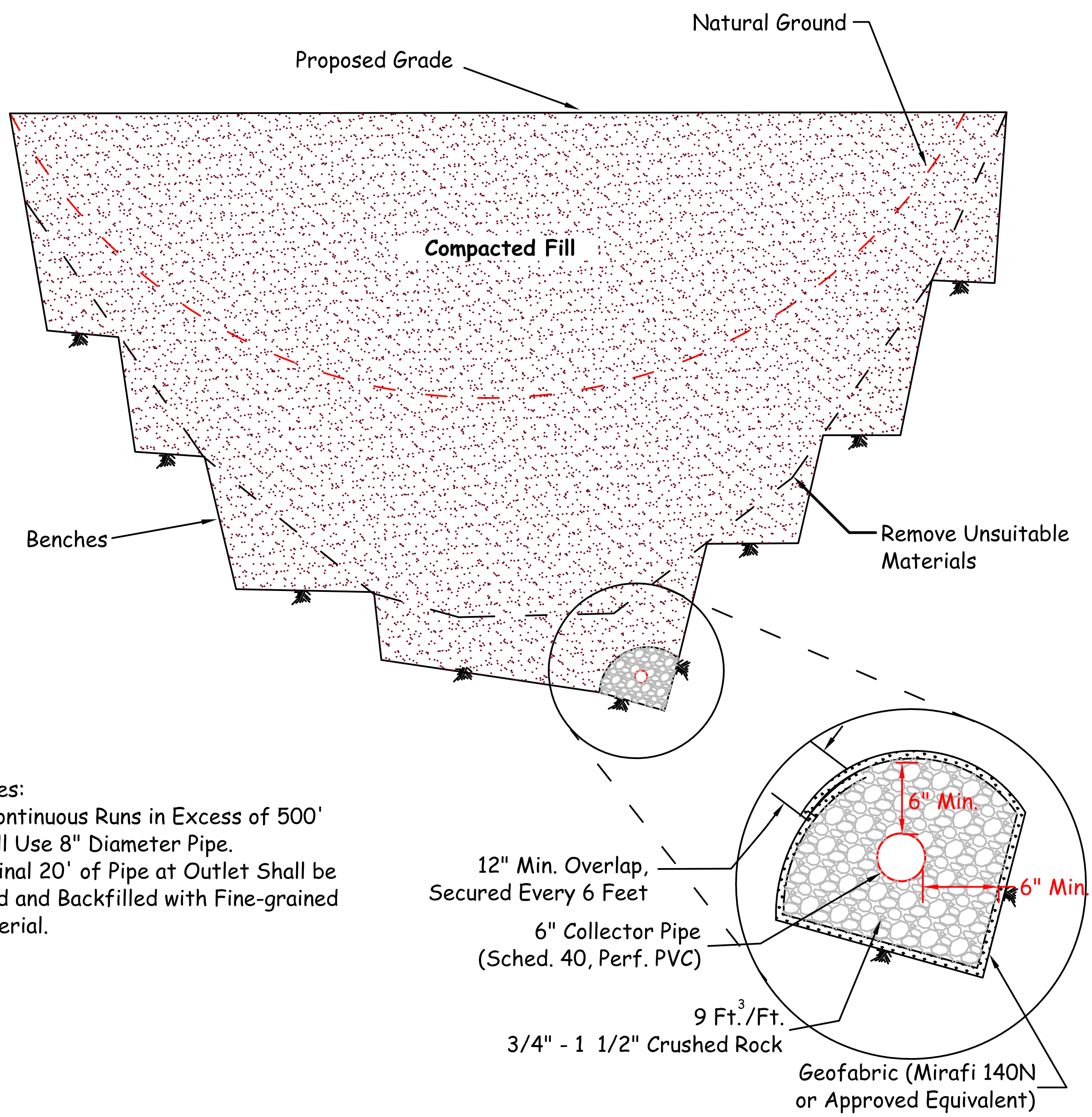
Cut/Fill Transition Lot



*Deeper if Specified by Soils Engineer

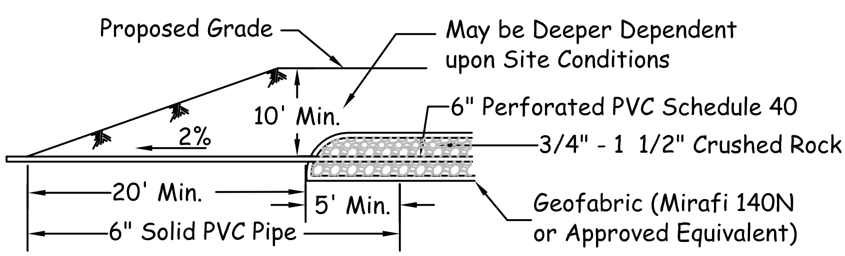
Geotechnical, Inc.

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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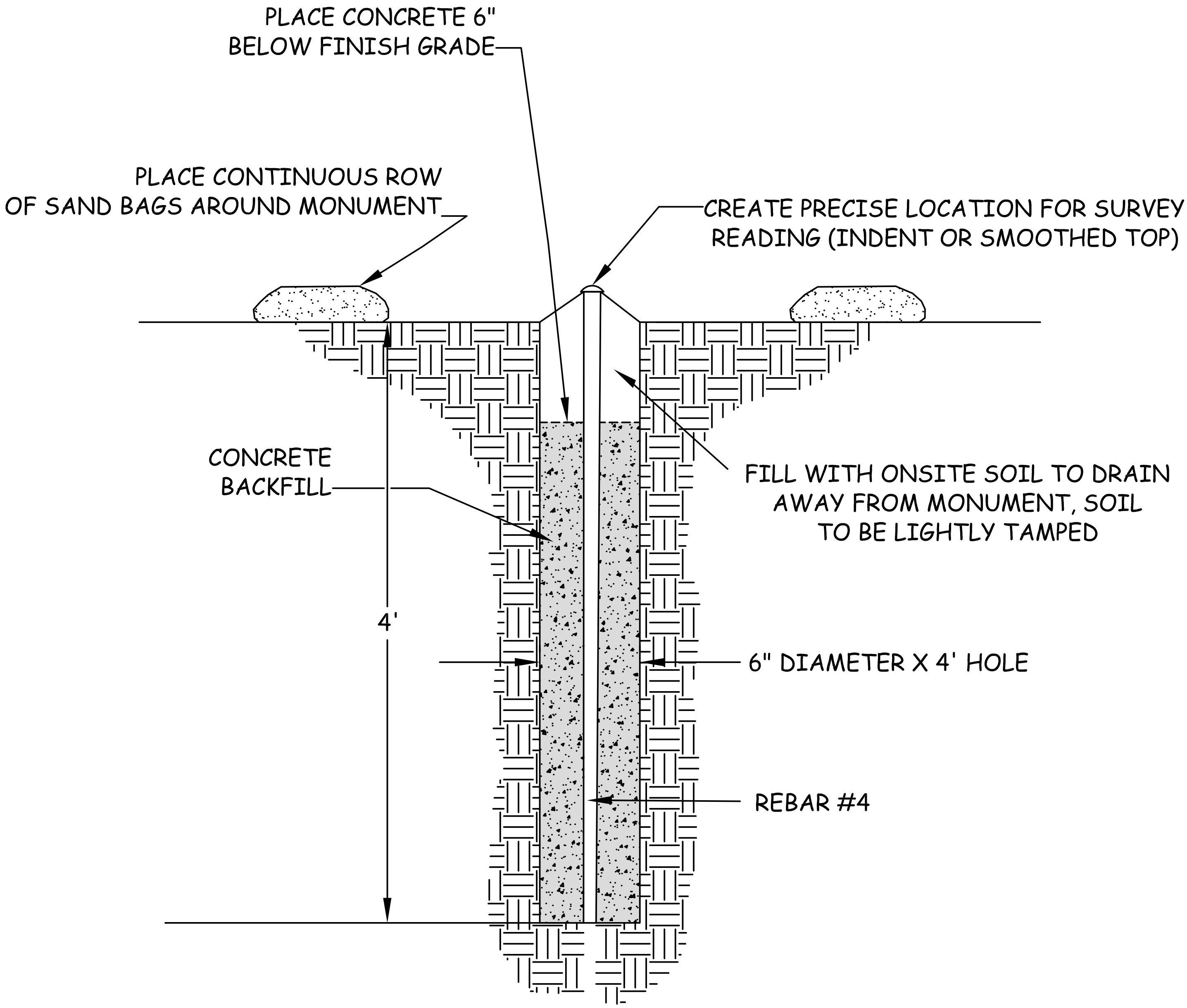
CANYON SUBDRAINS

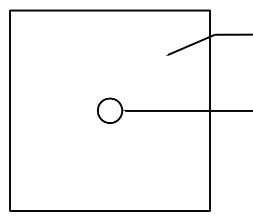


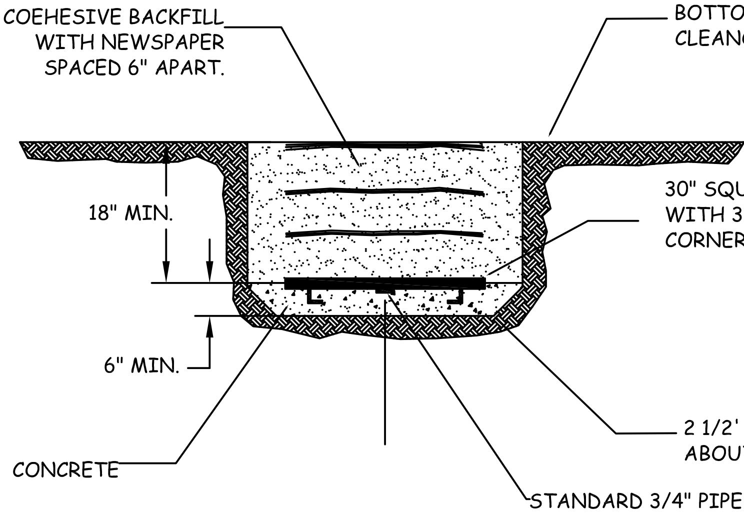
NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

LGC Geotechnical, Inc.

TYPICAL SURFACE SETTLEMENT MONUMENT

TOP VIEW





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

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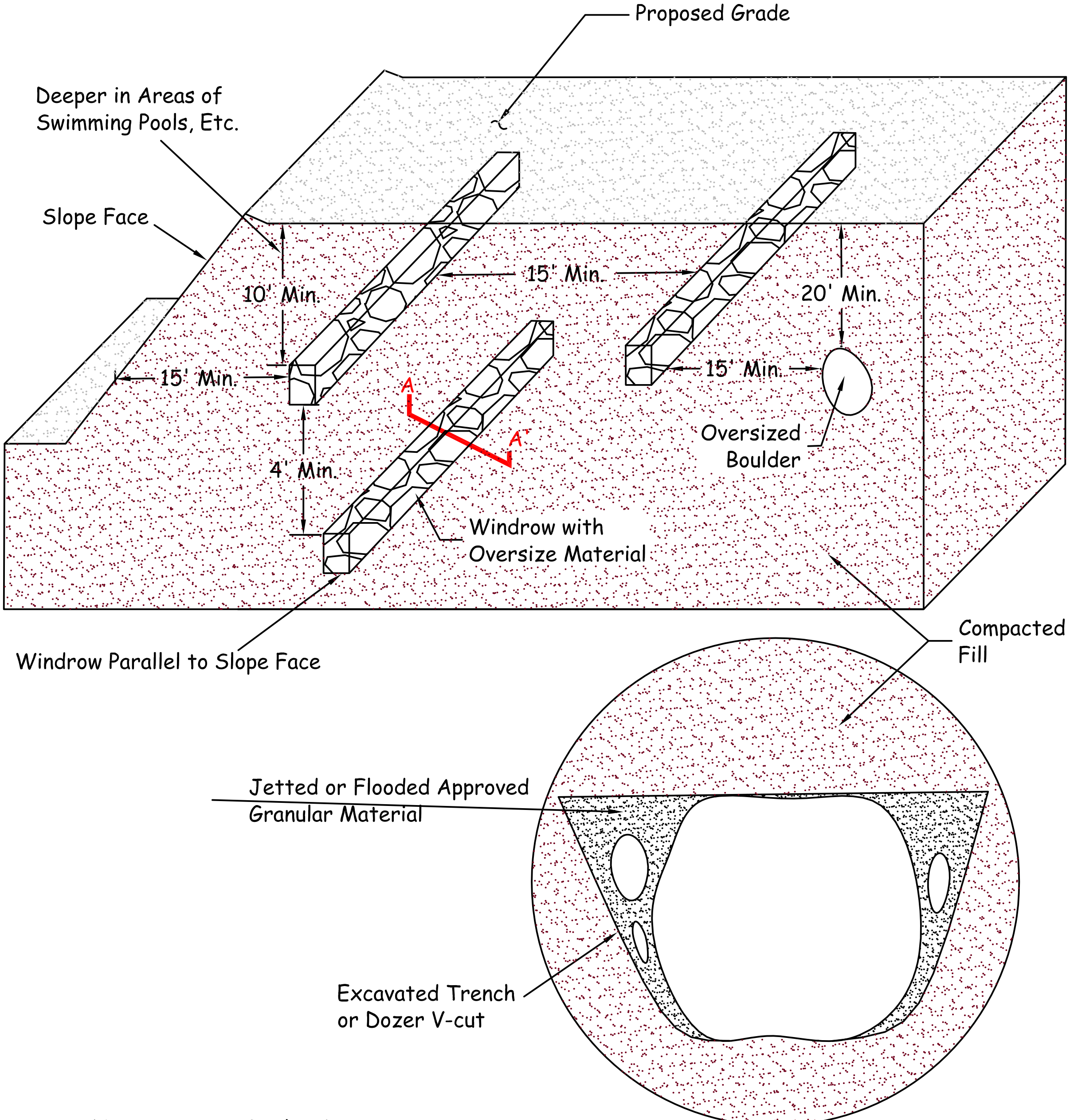
TYPICAL SETTLEMENT PLATE AND RISER



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

Section A-A'

LGC Geotechnical, Inc.

OVERSIZE ROCK DISPOSAL DETAIL