Appendix D
Preliminary Geotechnical Evaluation

LGC Geotechnical, Inc.

September 14, 2023 Project No. 19035-01

Mr. Sam Juarez
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Subject: ***Preliminary Geotechnical Evaluation and Design Recommendations for Proposed Great Scott Tree Service Commercial Development, APNs 610-301-07, -20, and -21, 20865 Canada Road, Lake Forest, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed Great Scott Tree Service commercial development, APNs 610-301-07, -20, and -21, located at 20865 Canada Road in the City of Lake Forest, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed development.

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

Respectfully Submitted,

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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 commercial development located at 20865 Canada Road in the City of Lake Forest, California. Refer to the Site Location Map (Figure 1).

The purpose of our study was to provide a preliminary geotechnical evaluation relative to the proposed development. As part of our scope of work, we have: 1) reviewed available geotechnical background information including in-house regional geologic maps and published geotechnical literature pertinent to the site (Appendix A); 2) performed a limited subsurface geotechnical evaluation of the site consisting of the excavation of three small-diameter borings ranging in depth from approximately 5 to 50 feet below existing ground surface; 3) performed one field infiltration test; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this geotechnical summary report presenting our preliminary findings, conclusions and recommendations for the development of the proposed project.

1.2 Existing Conditions

The approximately 6.72-acre project site is comprised of three parcels (APNs 610-301-07, -20, and -21) and is currently developed with one single-family residence, along with a second residence that was converted for office use which has since been demolished, and a barn with multiple structures. Open unpaved (dirt) areas within the site are used for parking and storage. Serrano Creek forms the project site's northern boundary. There are several trees with brush and vegetation distributed throughout the site. The Project site is currently accessed from two driveways on Linear Lane and a driveway on Canada Road.

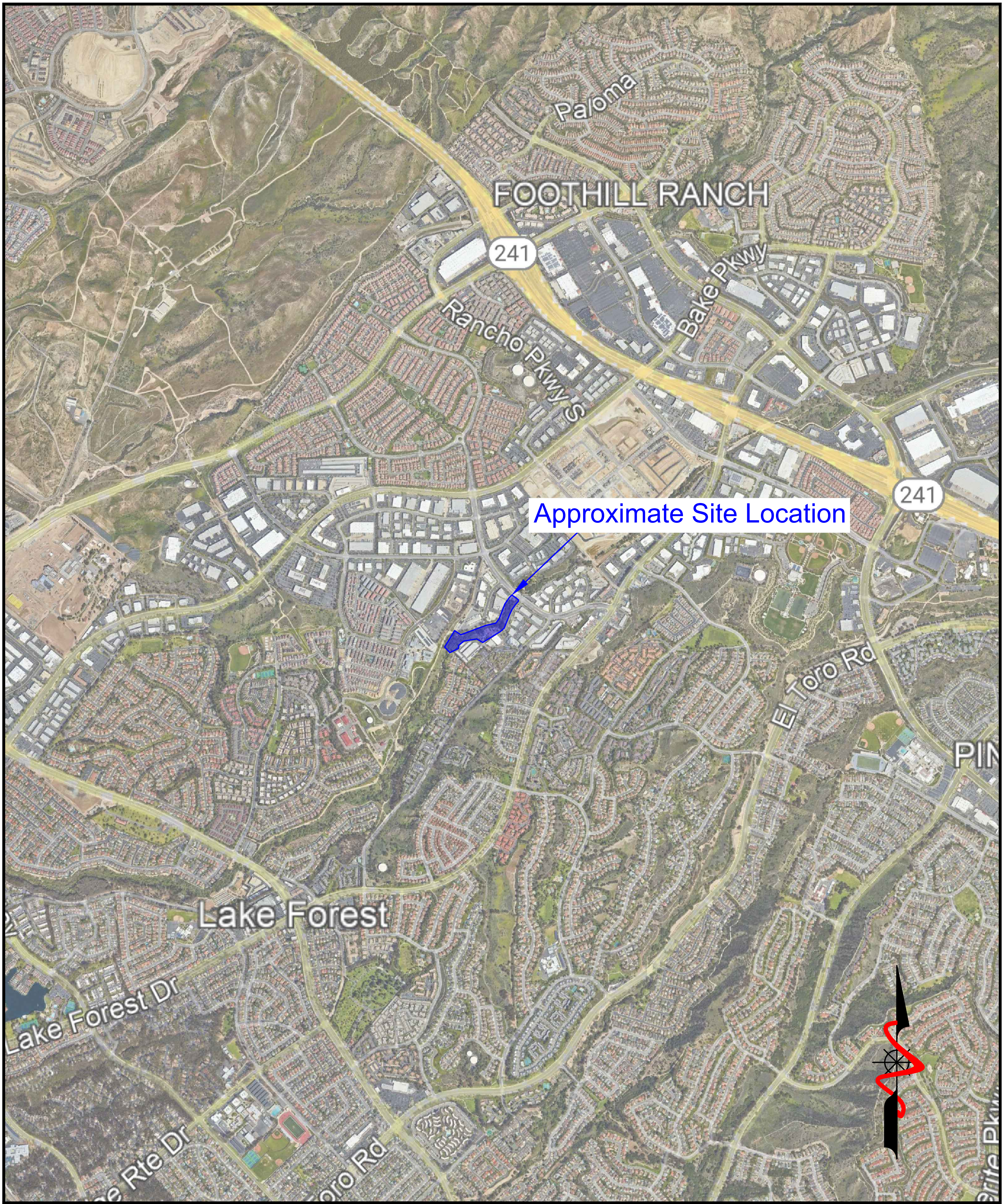
Based on review of historic aerials, it appears the barn structure was constructed some time before the year 1938 and the residential structure was constructed sometime between the years 1946 and 1952.

1.3 Project Description

Based on the preliminary grading plan (Huitt-Zollars, 2023), the proposed improvements include the construction of a gravel road and gravel parking areas, building remodels, a new trash enclosure, retaining walls, and water quality improvements. The proposed development is located on the southern side of the existing Serrano Creek alignment. Design cuts and fills (not including required remedial grading) are anticipated to be on the order of 2 to 4 feet. The proposed building remodels are anticipated to be relatively light with maximum column and wall loads of approximately 20 kips and 2 kips per linear foot, respectively. Please note no structural loads were provided to us at the time of this report.

Some of the existing structures including the barn (storage building) and residence (office) are anticipated to remain in-place and keep the same building footprint that currently exists.

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



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Great Scott - Lake Forest
PROJECT NO.	19035-01
ENG. / GEOL.	RLD / KTM
SCALE	Not to Scale
DATE	September 2023

1.4 Subsurface Geotechnical Evaluation

LGC Geotechnical performed a limited subsurface geotechnical evaluation of the site consisting of the excavation of three hollow-stem auger borings to evaluate onsite geotechnical conditions.

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

Infiltration testing was performed within one of the borings (I-1) to a depth of 5 feet below existing grade. An LGC Geotechnical staff engineer installed standpipe, backfilled the boring with crushed rock and pre-soaked the infiltration hole prior to testing. Infiltration testing was performed per the County of Orange testing guidelines. The location was subsequently backfilled with native soils at the completion of testing.

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

1.5 Laboratory Testing

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

The following is a summary of the laboratory test results:

- Dry density of the samples collected ranged from approximately 91 pounds per cubic foot (pcf) to 117 pcf, with an average of 106 pcf. Field moisture contents ranged from approximately 1 to 26 percent, with an average of 13 percent.
- Four fines content tests were performed and indicated a fines content (passing No. 200 sieve) ranging from approximately 6 to 37 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as “coarse-grained.”
- One Atterberg Limit (liquid limit and plastic limit) test was performed. Results indicated a Plasticity Index (PI) value of 17.
- One consolidation test was performed. The load versus deformation plot is provided in

Appendix C.

- Expansion potential testing indicated an expansion index value of 8, corresponding to "Very Low" expansion potential.
- One R-value test was performed on a bulk sample collected and resulted in an R-Value of 66.
- Corrosion testing indicated soluble sulfate contents of approximately 0.02 percent, a chloride content of 103 parts per million (ppm), pH of 8.2, and a minimum resistivity of 857 ohm-centimeters.

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

2.0 GEOTECHNICAL CONDITIONS

2.1 Geologic Conditions

The subject site is located within the foothills of the Santa Ana Mountains, part of the Peninsular Ranges Geomorphic Province. The region consists of dissected foothills bordering the Los Angeles Basin to the northwest and the granite-core Santa Ana Mountains to the east. The Southern California Batholith forms the core of the Santa Ana Mountains, which is overlain by a thick sequence of sedimentary units, which comprise the foothills including the subject site. Late Miocene to Early Pliocene bedrock materials of the Oso Member of the Capistrano Formation that underlie the subject site at depth are primarily composed of sandstone and silty sandstone (USGS, 2004).

The site is specifically located within the Serrano Creek drainage course and the area just southeast of the active drainage. The southwest-flowing creek has deposited variable alluvial materials as observed during our subsurface investigation.

2.2 Generalized Subsurface Conditions

The subsurface evaluation performed at the subject site indicated that site soils consist of variable alluvium ranging from very moist to wet, moderate to dark brown clayey sand and sand, to an alluvial deposit consisting of light gray, relatively dry, medium to coarse sand with few pebbles. The material is labelled "younger alluvium" on boring logs. Bedrock of the Capistrano Formation, Oso Member was encountered at depth below the alluvium, consisting of light yellowish brown, silty sandstone, moist, very dense, observed to the maximum explored depth of approximately 50 feet below existing grade.

It should be noted that the 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.

2.3 Groundwater

Groundwater was encountered in our boring HS-1 at a depth of approximately 15 feet below existing grade. Historic high groundwater is estimated to be about 10 feet below existing grade (CDMG, 2000).

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

2.4 ***Field Infiltration Testing***

One field percolation test was performed in the area of the proposed infiltration trench and the location is depicted on Figure 2 – Geotechnical Exploration Location Map. Test well installation consisted of placing a 3-inch diameter perforated PVC pipe in the excavated borehole and backfilling the annulus with crushed rock including the placement of approximately 2 inches of crushed rock at the bottom of the borehole. The infiltration test well was presoaked the day of installation and testing took place within 24 hours of presoaking. During the pre-test, the water level was observed to drop less than 6 inches in 25 minutes for two consecutive readings. Therefore, the test procedure for fine-grained soils or "slow test" was followed. Test well installation and the estimation of infiltration rates were accomplished in general accordance with the guidelines set forth by the County of Orange (2013). In general, three-dimensional flow out of the test well (*percolation*), as observed in the field, is mathematically reduced to one-dimensional flow out of the bottom of the test well (*infiltration*). Infiltration tests are performed using relatively clean water, free of particulates, silt, etc. The results of our recent field infiltration testing are presented in Appendix D and summarized below.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Identification	Approx. Depth Below Existing Grade (ft)	Observed Infiltration Rate* (in./hr.)	Measured Infiltration Rate** (in./hr.)
I-1	5	0.7	0.35

*Observed Infiltration Rates Do Not Include Factor of Safety.
**Measured Infiltration Rates Include a Factor of Safety of 2 in Order to Evaluate Feasibility. The actual design Factor of Safety could be higher and should be verified by the civil engineer.

The tested infiltration rates provided in this report are considered a general representation of the infiltration rates at the location of the proposed infiltration trench. Please note, the testing of infiltration rates is highly dependent upon the materials encountered at the point of testing (i.e. location and depth of testing). Varying subsurface conditions may exist outside of the test location which could alter the calculated infiltration rate. Please refer to Section 4.7 for subsurface water infiltration recommendations.

2.5 ***Seismic Design Criteria***

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Since the site contain soils that are susceptible to liquefaction (refer to section below "Liquefaction and Dynamic Settlement"), ASCE 7-16 which has been adopted by the CBC requires that site soils be assigned Site Class "F" and a site-specific response spectrum be performed. However, in accordance with Section 20.3.1 of ASCE 7-16, if the fundamental periods of vibration of the planned structure are equal to or less than 0.5 seconds, a site-specific response spectrum is not required and ASCE 7-16/2022 CBC site class and seismic parameters may be used in lieu of a site-specific response spectrum. **It**

should be noted that the seismic parameters provided herein are not applicable for any structure having a fundamental period of vibration greater than 0.5 seconds. Additionally, 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 33.6606 degrees north and longitude -117.6751 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 6.57 at a distance of approximately 15.98 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 6.57 at a distance of approximately 21.89 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.570 (SEAOC, 2022).

TABLE 2

Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**	**Notes/Exceptions**
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.250g	From SEAOC, 2022
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.446g	From SEAOC, 2022
F_a (per Table 1613.2.3(1))	1.000	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.854	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.250g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	0.827g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.833g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.551g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.940	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.932	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

The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo) and no faults were identified on the site during our site evaluation (CGS, 2018). The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site. The closest known active faults are associated with the San Joaquin Hills Fault, located approximately 3.1 miles from the site; the Elsinore Fault Zone, approximately 12.6 miles northeast of the site; and the Newport Inglewood Fault Zone, approximately 12.7 miles southwest of the site.

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

2.6.1 Liquefaction and Dynamic Settlement

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

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 2001), the site is located within a liquefaction hazard zone. In general, site soils are medium dense to dense and not susceptible to liquefaction. However, isolated loose sand layers are present and considered susceptible to liquefaction. The recent encountered in-situ groundwater depth of 15 feet below existing grade and historic high groundwater depth of 10 feet below existing grade were both used in the liquefaction analysis. The liquefaction evaluation was performed using data from boring HS-1. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and based on the seismic criteria of the 2022 California Building Code (CBC) and historic high groundwater depth. Liquefaction induced settlement was estimated using the PGA_M per the 2022 CBC and a moment magnitude of 6.9 (USGS, 2008).

Results indicate total seismic settlement on the order of 2-inches or less. Differential seismic settlement can be estimated as half of the total estimated settlement over a horizontal span of about 40 feet (i.e., 1-inch over a horizontal span of 40 feet). Seismically induced settlements were estimated by the procedure outlined by Tokimatsu and Seed (1987). Liquefaction calculations are provided in Appendix E.

2.6.2 Lateral Spreading

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

Based on site liquefaction potential, lateral spreading and consequently zones of instability (horizontal displacements) near the banks of the adjacent creek are possible during the design basis earthquake ground motion. A corrected $(N_1)_{60}$ blow count of less than 15 is typically used for screening of potential lateral spreading (Youd, Hansen, Bartlett, 2002). Based on the obtained data, the soils within the lateral zone of the creek generally have corrected $(N_1)_{60}$ values of at least 15. Based on the obtained apparent density (i.e., blow counts) obtained from our field evaluation the potential for lateral spreading is generally considered low.

2.7 <u>Expansion Potential</u>

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

3.0 CONCLUSIONS

Based on the results of our 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:

- Groundwater was encountered during our subsurface evaluation at a depth of approximately 15 feet below existing ground surface. Historic high groundwater is estimated to be about 10 feet below existing grade (CDMG, 2000).
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo). The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Site soils are considered susceptible to liquefaction. The site is located in a State of California Seismic Hazard Zone for liquefaction. Total dynamic settlement is estimated to be on the order of 2-inches or less. Differential dynamic settlement can be estimated at half of the total settlement over a horizontal span of 40 feet for design of foundations.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have "Very Low" expansion potential. Final design expansion potential must be determined at the completion of grading.
- From a geotechnical perspective, the existing onsite soils are suitable material for use as general fill (not retaining wall backfill), provided that they are relatively free from oversized material (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- Some portions of the onsite soils have high fines content and are not suitable for backfill of site retaining walls. Therefore, import and/or select grading and stockpiling of onsite sandy soils meeting project recommendations may be required.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order.
- The proposed grading and construction are not anticipated to pose a significant adverse geotechnical impact on the existing and surrounding improvements and adjoining sites, provided the recommendations presented in this report are implemented during design and construction of 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 2022 CBC requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level.” The “acceptable level” of risk is defined by the California Code of Regulations as “that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

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

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of demolition of the existing site improvements, required earthwork removals, subgrade preparation, precise grading and construction of the proposed new improvements including the remodeled buildings, parking areas, subsurface utilities, water quality facilities, etc.

We recommend that earthwork be performed in accordance with the following recommendations, future grading plan review report(s), the 2022 CBC/City of Lake Forest grading requirements, and the General Earthwork and Grading Specifications included in Appendix F. In case of conflict, the following recommendations shall supersede those included in Appendix F. The following recommendations should be considered preliminary and may be revised based upon future evaluation and review of the project plans and/or based on the actual conditions encountered during site grading/construction.

4.1.1 Site Preparation

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

If cesspools or septic systems are encountered during earthwork, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. 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 and improvements, we recommend the site soils be removed and recompacted.

New Buildings: We recommend that soils within new building pads be removed and recompacted to a minimum depth of 4 feet below existing grade or 3 feet beneath the base of the foundations, whichever is deeper. Where adequate space is available, the base of removal and recompaction bottoms should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or at a minimum distance of 4 feet beyond the edges of the proposed building foundations, whichever is larger.

Building Remodel Areas: We recommend that soils supporting new footings/slabs within the existing building remodel areas be removed and recompacted to a minimum depth of 2 feet below existing grade. Where adequate space is available, the base of removal and recompaction bottoms should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or at a minimum distance of 2 feet beyond the edges of the proposed building foundations, whichever is larger.

Minor Site Structures: For minor site structures such as free-standing, screen walls, trash enclosures, etc., removal and recompaction should extend at least 3 feet beneath the existing grade or 2 feet beneath the base of foundations, whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 3 feet beyond the edges of the proposed improvements mentioned above, where space permits.

Pavement and Hardscape: Within pavement areas, removal and recompaction should extend to a depth of at least 1 foot below the existing grade or 1 foot beneath the finished subgrade (i.e., beneath planned aggregate base/asphalt concrete or gravel). Within hardscape areas, removal and recompaction should extend to a depth of at least 1 foot below the existing grade or 1 foot beneath the finished subgrade (i.e., beneath planned concrete).

Local conditions may be encountered during excavation that could require deep remedial grading 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 and recompaction areas should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Raveling of the sandy soils should be anticipated for temporary slopes. Flatter slope inclinations should be considered if raveling cannot be tolerated. The exposed slope surface may be kept surficially moist (but not saturated) during construction to reduce (not eliminate) potential sloughing. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

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

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

4.1.4 Removal Bottoms and Subgrade Preparation

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

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

4.1.5 Material for Fill

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

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of soils of "Very Low" expansion potential (expansion index 20 or less based on American Society for Testing and Materials [ASTM] D 4829), and free of significant organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. <u>Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.</u>

Retaining wall backfill should consist of imported or onsite free draining, clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve) per ASTM Test Method D1140 (or ASTM D6913/D422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of significant organic materials, construction debris, and any material greater than 3 inches in maximum dimension. The site contains soils that are not suitable for retaining wall backfill due to their fines content; therefore, select grading and stockpiling and/or import 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 most recent version of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) and/or City of Lake Forest requirements.

The placement of inert 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 to 3 inch in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, 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 parking and drive aisle areas (i.e., not within building pad areas).

4.1.6 Placement and Compaction of Fills

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

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

Aggregate base material should be compacted to at least 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at 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 with filter fabric to prevent the migration of fines into the rock backfill.

4.1.7 Trench and Retaining Wall Backfill and Compaction

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

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

Retaining wall backfill should consist of 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.

4.2 Preliminary Foundation Recommendations

The proposed building additions and modular structures may be supported on a conventional slab and spread footings or a mat slab, provided earthwork is performed in accordance with the recommendations presented in this report. All footings should be supported on properly compacted fill. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading.

Preliminary foundation recommendations are provided in the following sections. The foundation design must be performed by the structural engineer based on the following geotechnical parameters and minimum values provided.

4.2.1 Slab Design and Construction

We recommend buildings be founded on a conventional slab with a minimum thickness of 4 inches. Prefabricated modular buildings (office structures, etc.) may also be founded on a mat slab with a minimum thickness of 6 inches. Conventional slabs and mat slabs are to be supported on compacted fill soils properly prepared in accordance with the recommendations provided in this report. Minimum slab reinforcement should be determined by the structural engineer based on the imposed loading, crack control, estimated static settlement and seismic settlement, etc.

It is recommended that subgrade soils below mat slabs be moisture conditioned in order to maintain the recommended moisture content up to the time of concrete placement. The recommended moisture content of the mat slab subgrade soils should be approximately 0 to 2 percent above optimum moisture content to a minimum depth of 12 inches. The moisture content of the mat slab subgrade should be verified by the geotechnical engineer within 1 to 2 days prior to concrete placement. In addition, this moisture content should be maintained around the immediate perimeter of the mat slabs during construction.

4.2.2 ***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 should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.3 ***Soil Bearing and Lateral Resistance***

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 2,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. 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 ½-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ¼-inch over a horizontal span of 40 feet). Seismic settlement recommendations are presented in Section 2.6.1.

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. For slabs constructed over a moisture retarder, the allowable friction coefficient should be provided by the manufacturer. 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 lateral resistance. Allowable passive pressure may be increased to 300 pcf to a maximum of 3,000 psf for short duration seismic or wind loading. These passive pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. For a 2:1 (horizontal to vertical) downward sloping condition, a reduced allowable passive lateral earth pressure of 100 pcf to a maximum of 1,000 psf may be used. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. Frictional resistance and passive pressure may be used in combination without reduction. 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. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

4.4 Lateral Earth Pressures for Retaining Walls

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

The following lateral earth pressures are presented in Table 3 for approved import or onsite free draining, clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). Portions of the onsite soils are not suitable for retaining wall backfill due to their fines content. Therefore, select grading and stockpiling and/or import of soils meeting the criteria outlined above will be required by the contractor for obtaining suitable retaining wall backfill soil. <u>The wall designer should clearly indicate on the retaining wall plans the required select sandy soil backfill criteria.</u> These preliminary findings should be confirmed during grading.

TABLE 3

Lateral Earth Pressures – Approved 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 and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near-surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care should be taken to maintain these drains. Typical conventional retaining wall drainage is shown on Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Waterproofing and outlet systems are not the purview of the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal: vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining wall. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. Uniform lateral surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.45 and 0.3 may be used for at-rest and active conditions, respectively. The retaining wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 10 pcf for a level backfill condition. This increment should be applied in addition to the provided static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). Per Section 1803.5.12 of the 2022 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. The provided seismic lateral earth pressure should not be used for retaining walls exceeding 10 feet in height. If a retaining wall greater than 10 feet in height is proposed or a retaining wall with a sloping backfill condition, the retaining wall designer should contact the geotechnical engineer for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

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

4.5 Soil Corrosivity

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

Corrosion testing of a near-surface bulk sample indicated a soluble sulfate content less than approximately 0.02 percent, a chloride content of 103 parts per million (ppm), pH of 8.2, and a minimum resistivity of 857 ohm-centimeters. Based on Caltrans Corrosion Guidelines

(Caltrans, 2021), 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 1500 ppm (0.2 percent) or greater. Based on the preliminary test results, soils are not considered corrosive using Caltrans criteria.

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

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

4.6 Control of Surface Water and Drainage Control

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

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

4.7 Subsurface Water Infiltration

Recent regulatory changes in some jurisdictions have recommended that low flow runoff be infiltrated rather than discharged via conventional storm drainage systems. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement. Infiltrated water may enter underground utility pipe zones and migrate along the pipe backfill, potentially impacting other improvements located far away from the point of infiltration.

Geotechnical stability and integrity of the project site is reliant upon appropriate handling of surface water. Due to the low infiltration rate, shallow groundwater and site liquefaction potential, we strongly recommend against the intentional infiltration of storm water.

4.8 *Preliminary Asphalt Concrete Pavement Sections*

The following provisional minimum asphalt concrete (AC) street sections are provided in Table4 for Traffic Indices (TI) of 5.5, 6.0 and 6.5 to be utilized in the design of the auto and truck parking/circulation areas. These sections are based on an assumed R-value of 50. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final pavement sections should be confirmed by the project civil engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 4

Preliminary Asphalt Concrete Pavement Section Options

Assumed Traffic Index	5.0 (or less)	6.5
R -Value Subgrade	50	50
AC Thickness	4.0 inches	4.0 inches
Aggregate Base Thickness	4.0 inches	4.5 inches

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

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

4.9 *Preliminary Gravel Parking Area Recommendations*

It is our understanding that the equipment parking areas will consist of compacted gravel (1-inch to 1 ½-inch in maximum dimension) over compacted subgrade and asphalt concrete paving is not desired. A minimum gravel pavement section of 4 inches of compacted gravel, over 6 inches of compacted aggregate base, over compacted subgrade is recommended. This parking surface is suitable for a traffic index of 5.0. It should be noted that gravel parking surfaces will require ongoing maintenance during the life of the pavement. Occasional re-grading should also be expected as a result of traffic loading and shifting of the unconfined gravel material.

The thickness shown is a minimum thickness. Increasing the thickness of the above will reduce the likelihood of the equipment parking area 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 equipment parking areas 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 equipment parking areas.

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

4.10 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, 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 5. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 5

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

	Sidewalks	Private Drives	Patios/ Entryways	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (nominal)	4 (full)	4 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	Wet down prior to placing	Wet down prior to placing	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of $^1/_3$ the concrete thickness	Saw cut or deep open tool joint to a minimum of $^1/_3$ the concrete thickness	Saw cut or deep open tool joint to a minimum of $^1/_3$ the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	—	City/Agency Standard

4.11 Geotechnical Plan Review

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

4.12 Geotechnical Observation and Testing During Construction

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2022 California Building Code (CBC).

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

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

5.0 LIMITATIONS

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

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

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

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

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

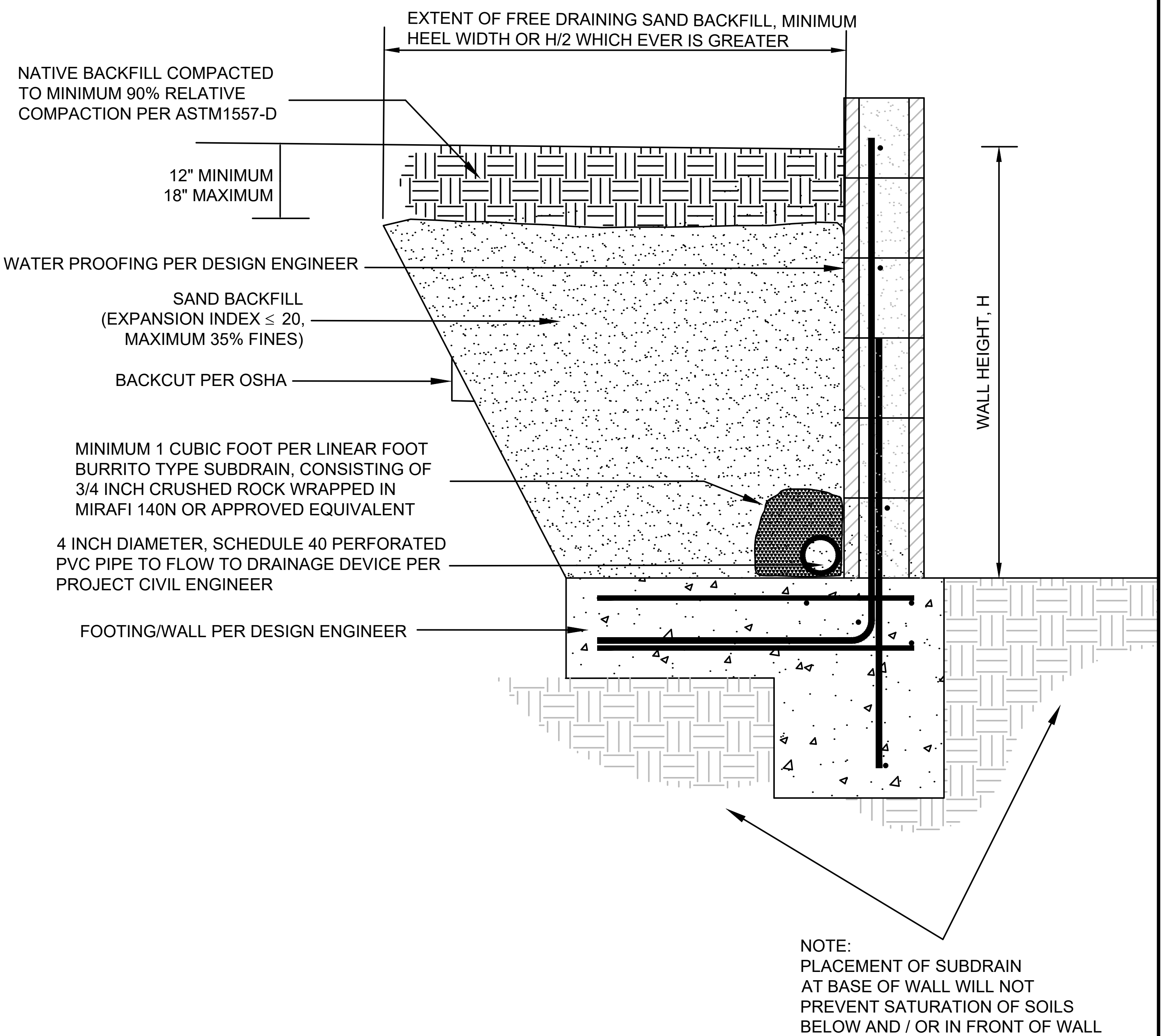


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FIGURE 2
Retaining Wall Detail

PROJECT NAME	Great Scott - Lake Forest
PROJECT NO.	19035-01
ENG. / GEOL.	RLD / KTM
SCALE	Not to Scale
DATE	September 2023

Appendix A
References

APPENDIX A

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Appendix B
Field Exploration Logs

Geotechnical Boring Log Borehole HS-1

Date: 4/8/2019	**Drilling Company:** Cal Pac
Project Name: Great Scott - Lake Forest	**Type of Rig:** Track Rig
Project Number: 19035-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~631' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By BPP
Sampled By BPP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
630	0	B-1					SP-SC	**@0' to 30' - <u>Quaternary Alluvium (Qal)</u>** @0' - Silty SAND: dark brown, slightly moist	EI,CR
			R-1	5 6 6	111.8	12.1	SM	@2.5' - Silty SAND: gray, moist, loose	
625	5		R-2	2 3 3	98.2	14.2		@5' - Silty SAND: brown, very moist, loose	
			R-3	2 4 7	112.2	14.6	SC	@7.5' - Clayey SAND: dark olive brown, very moist, loose	CN
620	10		R-4	2 2 3	101.8	21.8		@10' - Clayey SAND: brown, very moist, loose	-#200
615	15	▽	SPT-1	2 2 3		26.2	CL	@15' - Sandy CLAY: brown, wet, medium stiff; groundwater	AL
610	20		R-5	11 12 11	112.7	15.1	SP-SM	@20' - SAND with Silt: light brown, wet, medium dense	-#200
605	25		SPT-2	5 11 12		18.4	SM	@25' - Silty SAND: brown, wet, medium dense	-#200
	30							**@30' to T.D. - <u>Tertiary Capistrano Formation, Oso Member (Tco)</u>**	

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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: 4/25/2019

Geotechnical Boring Log Borehole HS-1

Date: 4/8/2019	**Drilling Company:** Cal Pac
Project Name: Great Scott - Lake Forest	**Type of Rig:** Track Rig
Project Number: 19035-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~631' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By BPP
Sampled By BPP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
600	30		R-6	9 11 22	117.0	14.5	SC	@30' - Clayey SAND with some gravel: gray-brown, wet, medium dense; extremely weathered bedrock	-#200
595	35		SPT-3	3 11 27		17.2	SM	@35' - Silty SAND: yellowish brown, wet, dense	
590	40		R-7	50/2"	105.4	24.8	SC	@40' - Clayey SAND: gray-brown with iron oxide staining, wet, very dense; slightly weathered bedrock	
585	45		SPT-4	50/6"		17.5		@45' - Clayey SAND: gray with iron oxide staining, wet, very dense @47' - Auger Refusal	
580	50							Total Depth = 50' Groundwater Encountered at Approximately 15' Backfilled with Cuttings on 4/8/2019	
575	55								
	60								

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

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

GROUNDWATER TABLE

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

Geotechnical Boring Log Borehole HS-2

Date: 4/8/2019	**Drilling Company:** Cal Pac
Project Name: Great Scott - Lake Forest	**Type of Rig:** Track Rig
Project Number: 19035-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~646' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By BPP
Sampled By BPP
Checked By N/A

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
645	0	B-1					SM	**@0' to T.D. - Quaternary Young Alluvium (Qya)** @0' - Silty SAND with Gravel: brown and dry	RV
			R-1	8 8 12	105.6	0.8	SP	@2.5' - SAND: pinkish-brown, dry, medium dense	
640	5		R-2	7 7 9	100.0	2.4		@5' - SAND: light brown, dry, medium dense	
			R-3	7 9 12	103.0	1.1		@7.5' - SAND: yellowish brown, dry, medium dense	
635	10		R-4	7 13 14	91.4	0.7		@10' - SAND: pinkish gray, dry, medium dense	
630	15							Total Depth = 10' Groundwater Not Encountered Backfilled with Cuttings on 4/8/2019	
625	20								
620	25								
	30								

LGC Geotechnical, Inc.

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

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

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

Last Edited: 4/25/2019

Geotechnical Boring Log Borehole I-1

Date: 4/8/2019	**Drilling Company:** Cal Pac
Project Name: Great Scott - Lake Forest	**Type of Rig:** Track Rig
Project Number: 19035-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~630' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By BPP
Sampled By BPP
Checked By N/A

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0						SM	**@0' to T.D. - Quaternay Alluvium (Qal)** @0' - Silty SAND: brown and moist	
			R-1	2 2 3	110.6	10.2		@2.5' - Silty SAND: gray-brown, wet, very loose	
625	5							Total Depth = 5' Groundwater Not Encountered Backfilled with Cuttings on 4/8/2019	
620	10								
615	15								
610	20								
605	25								
	30								

LGC Geotechnical, Inc.

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

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

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

Last Edited: 4/25/2019

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Test Results

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

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

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

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 10 ft	Clayey Sand	37
HS-1 @ 20 ft	Sand with Silt	6
HS-1 @ 25 ft	Silty Sand	16
HS-1 @ 30 ft	Clayey Sand with some Gravel	37

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 @ 15 ft	35	18	17	CL

Laboratory Test Results

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

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

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 1-5 ft	8	Very Low

* Per ASTM D4829

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

Sample Location	R-value
HS-2 @ 1-5 ft	66

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

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

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

Sample Location	Chloride Content (ppm)
HS-1 @ 1-5 ft	103

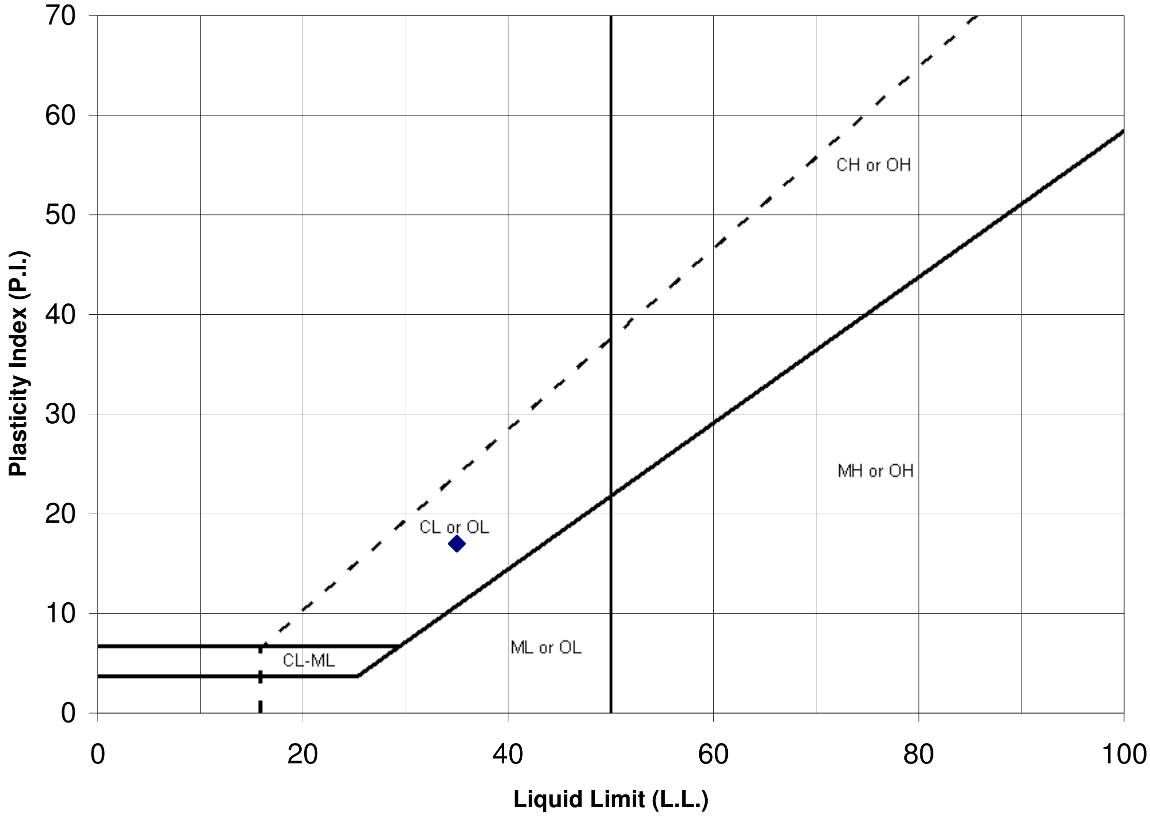
APPENDIX C (Cont'd)

Laboratory Test Results

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-1 @ 1-5 ft	8.2	857

PLASTICITY CHART - CLASSIFICATION OF FINE-GRAINED SOILS



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	HS-1	SPT-1	15	-	35	18	17	CL

LGC Geotechnical, Inc.

ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 19035-01
Date: Apr-19

Great Scott - Lake Forest

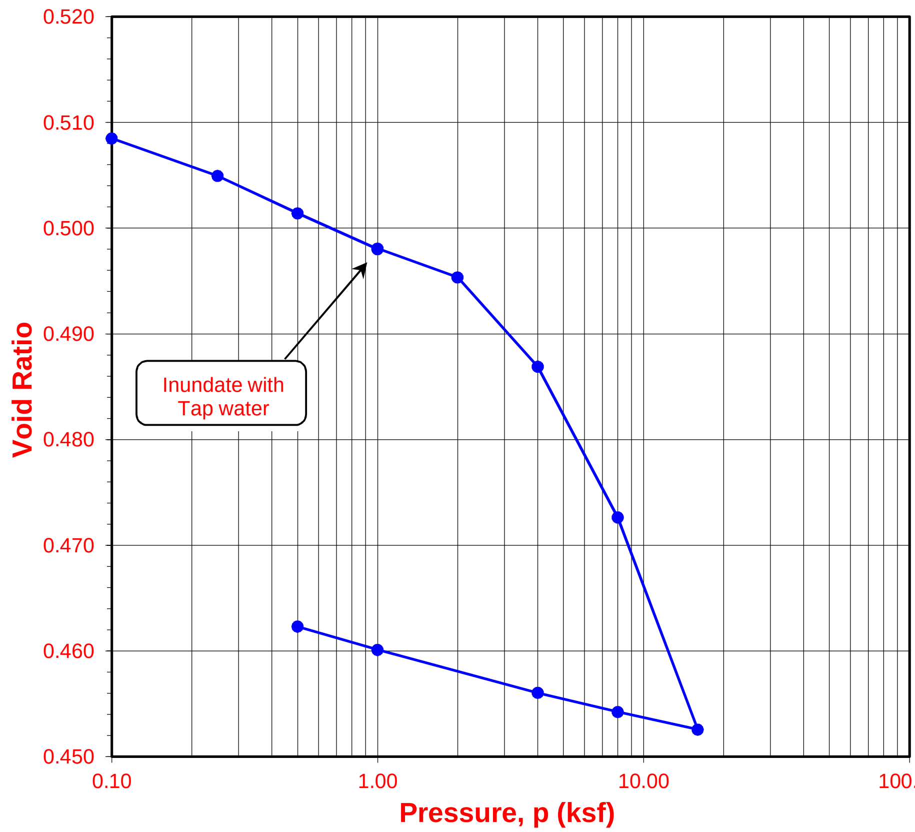
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: Lake Forest
Project No.: 19035-01
Boring No.: HS-1
Sample No.: R-3
Soil Identification: Dark olive brown clayey sand (SC)

Tested By: G. Bathala Date: 04/10/19
Checked By: J. Ward Date: 04/24/19
Depth (ft.): 7.5
Sample Type: Ring

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	199.51
Weight of Ring (g)	45.62
Height after consol. (in.)	0.9688
Before Test	
Wt.Wet Sample+Cont. (g)	208.34
Wt.of Dry Sample+Cont. (g)	190.23
Weight of Container (g)	66.23
Initial Moisture Content (%)	14.6
Initial Dry Density (pcf)	111.7
Initial Saturation (%)	77
Initial Vertical Reading (in.)	0.3164
After Test	
Wt.of Wet Sample+Cont. (g)	269.38
Wt. of Dry Sample+Cont. (g)	250.41
Weight of Container (g)	71.40
Final Moisture Content (%)	14.22
Final Dry Density (pcf)	114.5
Final Saturation (%)	81
Final Vertical Reading (in.)	0.2809
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

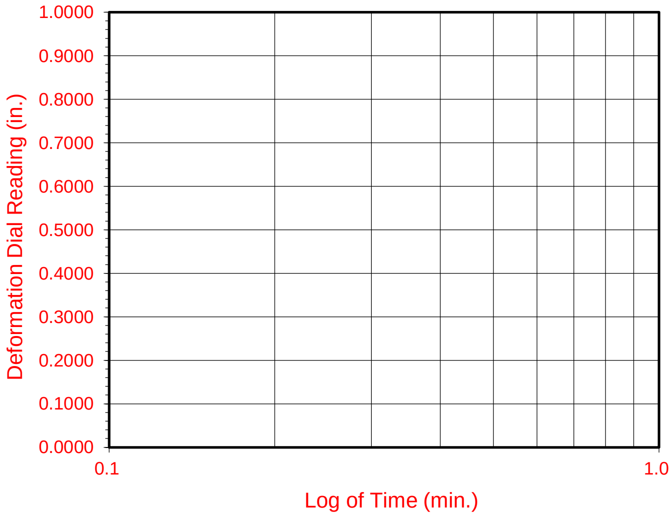


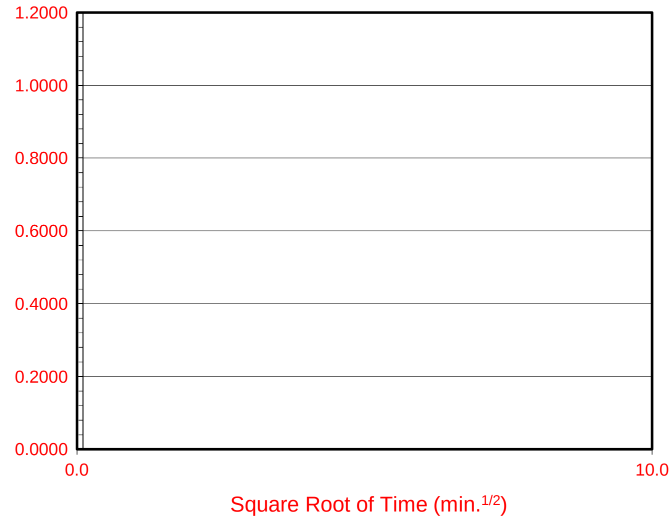
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3158	0.9994	0.00	0.06	0.508	0.06
0.25	0.3128	0.9964	0.07	0.37	0.505	0.30
0.50	0.3098	0.9934	0.13	0.66	0.501	0.53
1.00	0.3068	0.9904	0.21	0.96	0.498	0.75
1.00	0.3068	0.9904	0.21	0.96	0.498	0.75
2.00	0.3038	0.9874	0.33	1.26	0.495	0.93
4.00	0.2969	0.9805	0.46	1.95	0.487	1.49
8.00	0.2857	0.9693	0.64	3.08	0.473	2.44
16.00	0.2702	0.9538	0.86	4.63	0.453	3.77
8.00	0.2721	0.9557	0.78	4.44	0.454	3.66
4.00	0.2743	0.9579	0.68	4.22	0.456	3.54
1.00	0.2788	0.9624	0.50	3.77	0.460	3.27
0.50	0.2809	0.9645	0.43	3.55	0.462	3.12

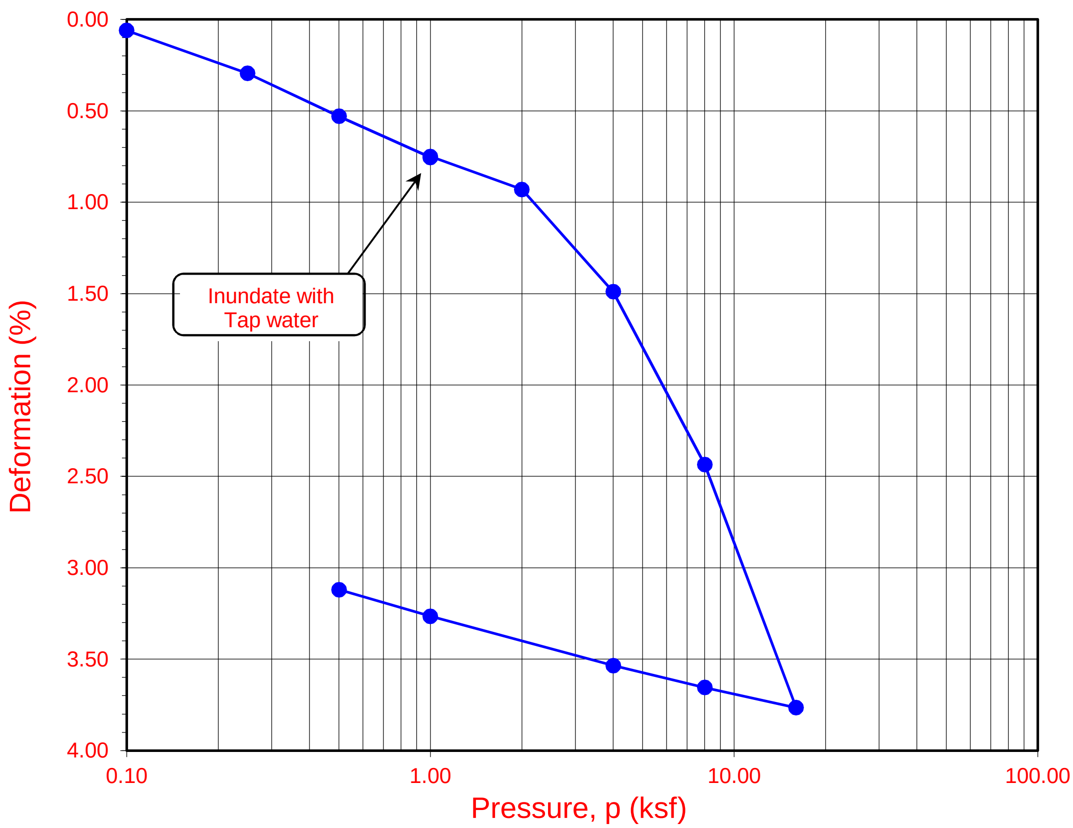
Time Readings

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**	**14.6**	**14.2**	**111.7**	**114.5**	**0.509**	**0.462**	**77**	**81**

Soil Identification: Dark olive brown clayey sand (SC)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 19035-01

Lake Forest

04-19

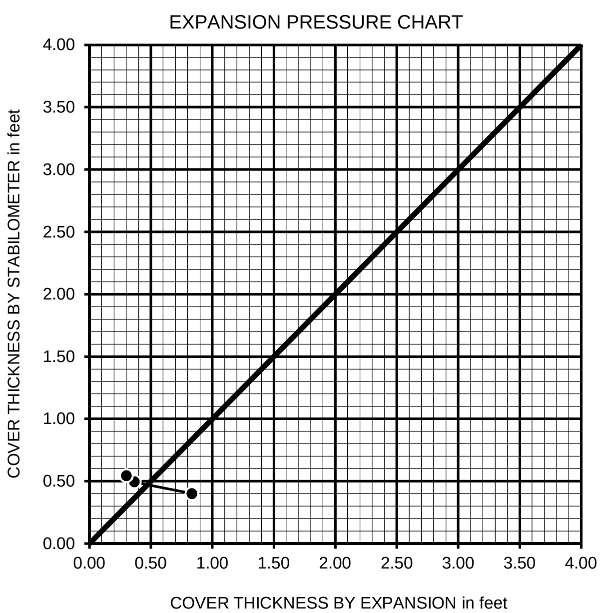
R-VALUE TEST RESULTS

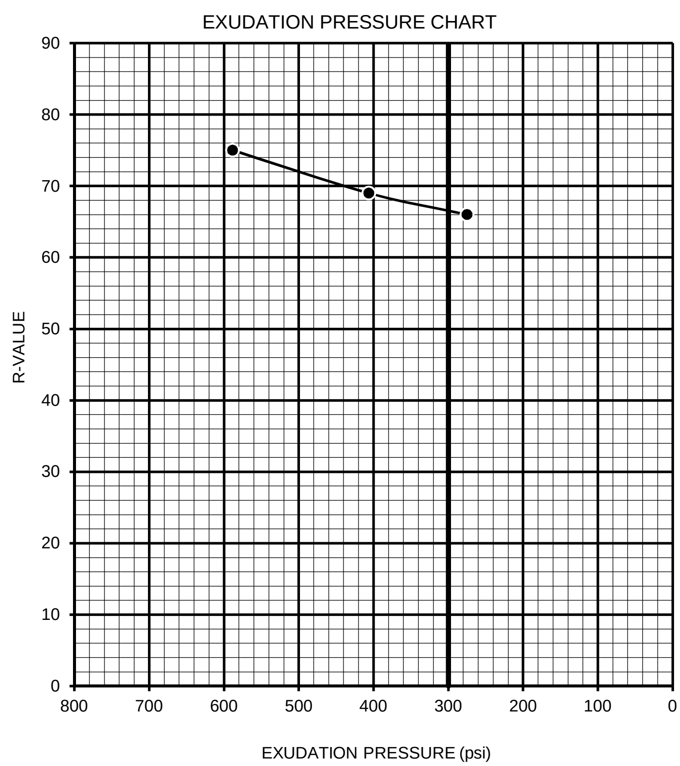
DOT CA Test 301

PROJECT NAME:	Lake Forest	PROJECT NUMBER:	19035-01
BORING NUMBER:	HS-2	DEPTH (FT.):	1-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	S. Felter
SAMPLE DESCRIPTION:	Brown silty sand with Gravel (SM)	DATE COMPLETED:	4/16/2019

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.0	11.4	11.8
HEIGHT OF SAMPLE, Inches	2.49	2.50	2.59
DRY DENSITY, pcf	118.4	120.3	116.5
COMPACTOR PRESSURE, psi	350	300	250
EXUDATION PRESSURE, psi	588	406	275
EXPANSION, Inches x 10exp-4	25	11	9
STABILITY Ph 2,000 lbs (160 psi)	26	33	38
TURNS DISPLACEMENT	4.26	4.32	4.42
R-VALUE UNCORRECTED	75	69	64
R-VALUE CORRECTED	75	69	66

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.40	0.50	0.54
EXPANSION PRESSURE THICKNESS, ft.	0.83	0.37	0.30





R-VALUE BY EXPANSION:	70
R-VALUE BY EXUDATION:	66
EQUILIBRIUM R-VALUE:	66

Appendix D
Infiltration Test Data

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Great Scott - Lake Forest
Project Number: 19035-01
Date: 4/4/2019
Boring Number: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	5
Boring Diameter (inches):	8
Pipe Diameter (inches):	2

*measured at time of test

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

Minimum test Head (D_o):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 3.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

*Pre-Test (Sandy Soil Criteria)**

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:37	9:02	25.0	2.54	2.92	0.38	No
2	9:02	9:27	25.0	2.60	2.93	0.33	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	9:28	9:58	30.0	2.50	2.88	0.38	0.6
2	9:59	10:29	30.0	2.47	2.88	0.41	0.7
3	10:30	11:00	30.0	2.53	2.91	0.38	0.6
4	11:01	11:31	30.0	2.53	2.93	0.4	0.7
5	11:32	12:02	30.0	2.54	2.95	0.41	0.7
6	12:03	12:33	30.0	2.49	2.91	0.42	0.7
7	12:34	13:04	30.0	2.58	3.00	0.42	0.7
8	13:05	13:35	30.0	2.52	2.93	0.41	0.7
9	13:36	14:06	30.0	2.53	2.95	0.42	0.7
10	14:07	14:37	30.0	2.48	2.89	0.41	0.7
11	14:38	15:08	30.0	2.45	2.90	0.45	0.7
12	15:09	15:39	30.0	2.53	2.98	0.45	0.7

Calculated Infiltration Rate (No factors of safety)	**0.7**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**0.35**

Sketch:

Notes:

Based on Guidelines from: Orange County 05/19/2011

Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Appendix E
Liquefaction Analysis

LIQUEFACTION EVALUATION

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

Seismic Event		**Profile Constants**		**Depth to GWT**		**Project Name**	Great Scott M.V.
Moment Magnitude	6.57	Total Unit Weight (lb/ft^3)	120	During Investigation (ft)	15	**Project Number**	19035-01
Peak Ground Acceleration	0.57 g (2022 CBC)	Unit Weight of Water (lbs/ft^3	62.4	During Design Event (ft)	10	**Boring**	HS- 1

Determination of Cyclic Resitance Ratio

Sampling Data					During Investigation			Sampling Correction Factors											
		Blow Count		Thickness	Total Stress	Pore Pressure	Effective	Sampler	SPT	Overburden	Energy	Borehole	Rod Length	Sampler Type		Fines			
Depth (ft)	Depth (m)	SPT	Rings	(ft)	Stress (psf)	Pressure (psf)	Stress (psf)	Diameter	N_m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Content	**$(N_1)_{60cs}$**	K_σ	$CRR_{7.5}$
2.5	0.8		12	3.75	420	0	420	0.62	7.44	1.70	1.25	1.00	0.75	1.00	11.86	5	**11.86**	1.000	0.128
5	1.5		6	2.5	720	0	720	0.62	3.72	1.70	1.25	1.00	0.75	1.00	5.94	25	**10.91**	1.000	0.118
7.5	2.3		11	3.75	1020	0	1020	0.62	6.82	1.43	1.25	1.00	0.75	1.00	9.15	25	**14.49**	1.000	0.157
10	3.0		5	2.5	1320	0	1320	0.62	3.10	1.26	1.25	1.00	0.75	1.00	3.66	**37**	**9.39**	1.000	0.102
15	4.6	5		5	1920	0	1920	1.00	5.00	1.04	1.25	1.00	0.85	1.10	6.09	50	**12.31**	1.000	0.133
20	6.1		23	5	2520	312	2208	0.62	14.26	0.97	1.25	1.00	0.95	1.00	16.47	**6**	**16.57**	0.986	0.176
25	7.6	23		5	3120	624	2496	1.00	23.00	0.91	1.25	1.00	0.95	1.10	27.48	**16**	**31.73**	0.965	SPT >30 NF
30	9.1		33	5	3720	936	2784	0.62	20.46	0.87	1.25	1.00	0.95	1.00	21.04	**37**	**30.25**	0.946	SPT >30 NF
35	10.7	38		5	4320	1248	3072	1.00	38.00	0.82	1.25	1.00	1.00	1.10	43.08	15	**47.65**	0.927	SPT >30 NF
40	12.2		100	5	4920	1560	3360	0.62	62.00	0.79	1.25	1.00	1.00	1.00	61.09	15	**66.53**	0.910	SPT >30 NF
45	13.7	100		2.5	5520	1872	3648	1.00	100.00	0.76	1.25	1.00	1.00	1.10	104.03	15	**111.53**	0.893	SPT >30 NF
				45															

Determination of Cyclic Stress Ratio

Sampling Data					During Design Event						
		Blow Count			Total Stress	Pore Pressure	Effective				
Depth (ft)	Depth (m)	SPT	Rings	Thickness	Stress (psf)	Pressure (psf)	Stress (psf)	r_d	CSR	MSF	FS
2.5	0.76		12	3.75	300	0	300	0.99615	0.369074	1.403	Above GWT
5	1.52		6	2.5	600	0	600	0.99024	0.366883	1.403	Above GWT
7.5	2.29		11	3.75	900	0	900	0.98456	0.36478	1.403	Above GWT
10	3.05		5	2.5	1200	0	1200	0.97914	0.362772	1.403	0.40
15	4.57	5		5	1800	312	1488	0.96856	0.434094	1.403	**Bray-Clay**
20	6.10		23	5	2400	624	1776	0.9569	0.479095	1.403	0.52
25	7.62	23		5	3000	936	2064	0.94183	0.507194	1.403	Corr. SPT>30
30	9.14		33	5	3600	1248	2352	0.92058	0.522052	1.403	Corr. SPT>30
35	10.67	38		5	4200	1560	2640	0.89062	0.524959	1.403	Corr. SPT>30
40	12.19		100	5	4800	1872	2928	0.85103	0.516899	1.403	Corr. SPT>30
45	13.72	100		2.5	5400	2184	3216	0.80363	0.499945	1.403	Corr. SPT>30

Appendix F
General Earthwork and 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.

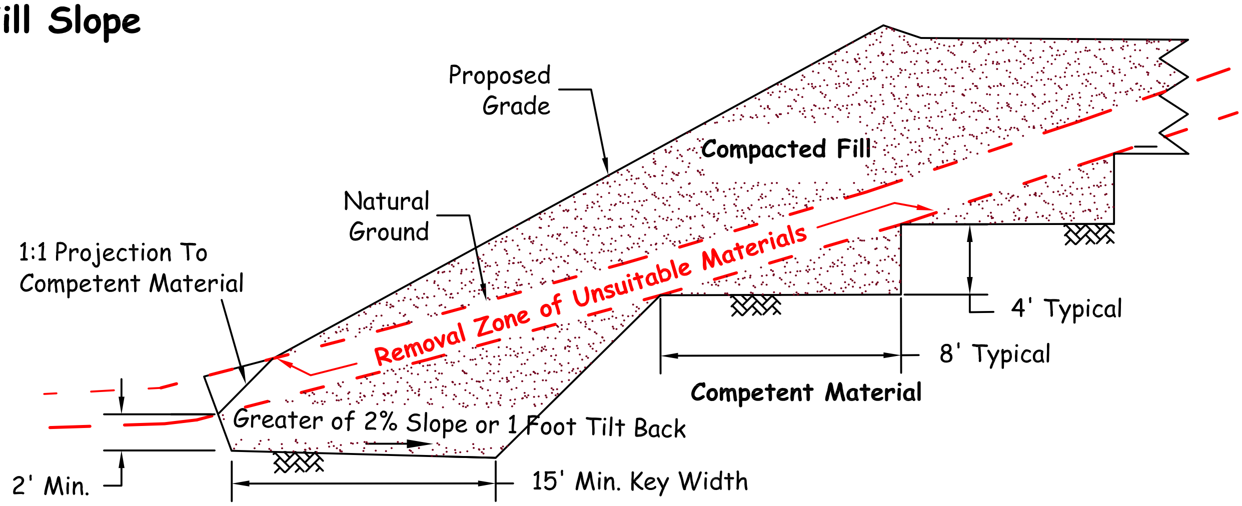
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

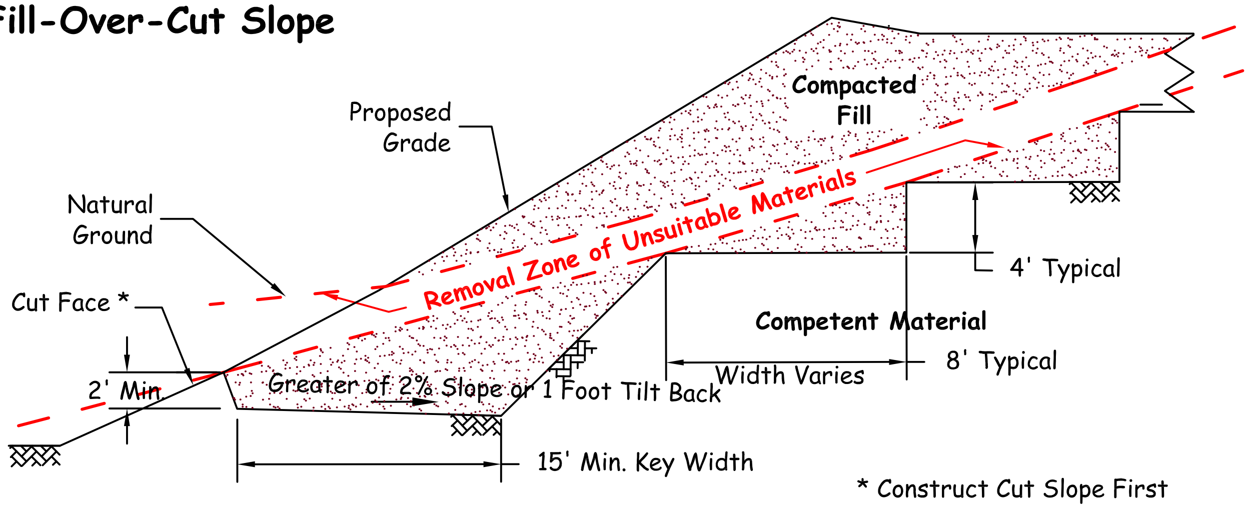
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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.

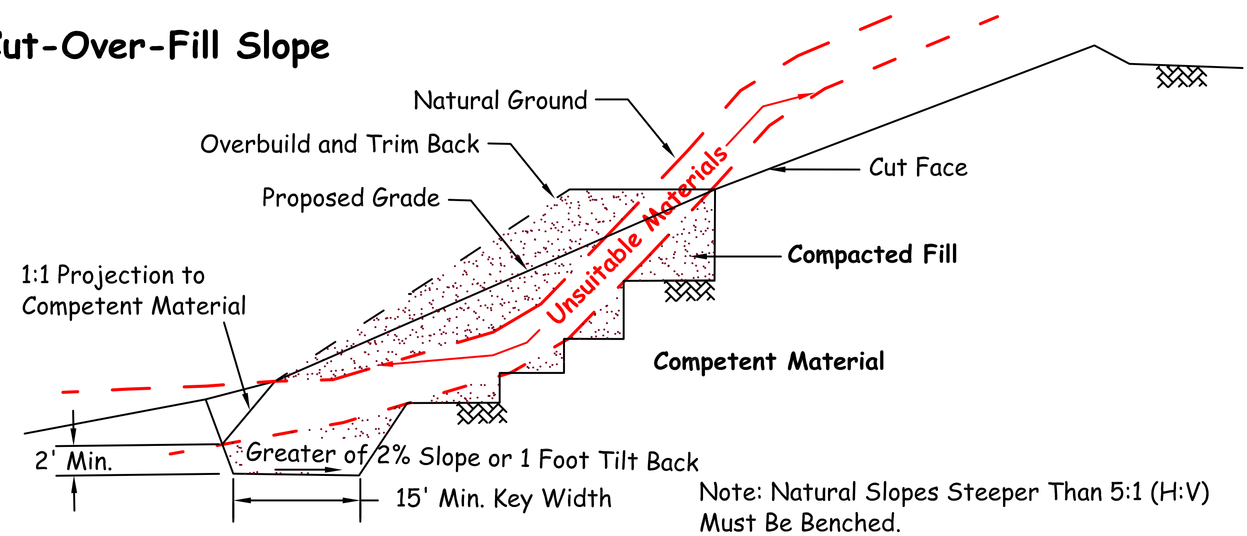
Fill Slope



Fill-Over-Cut Slope



Cut-Over-Fill Slope



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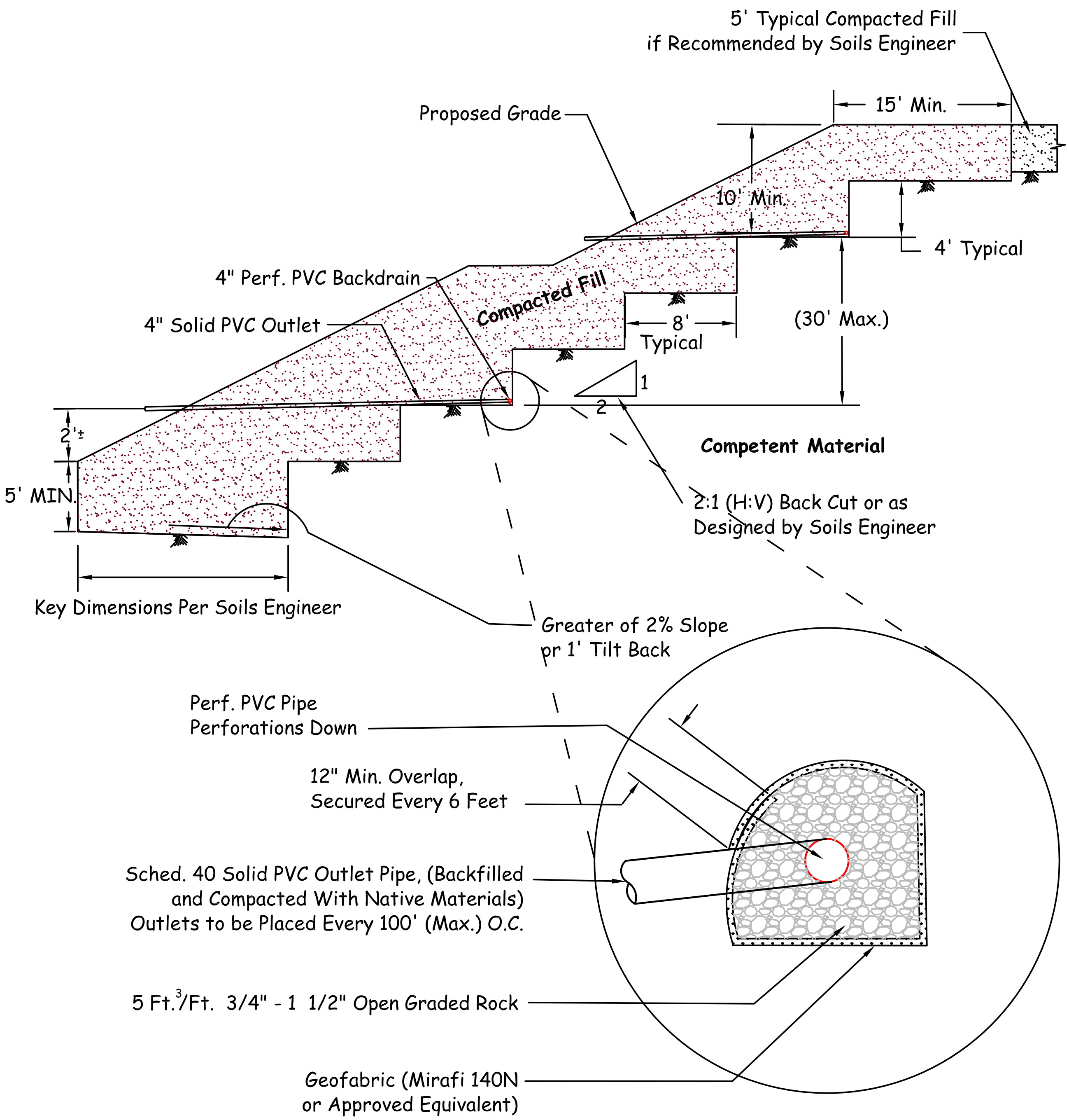
KEYING AND BENCHING



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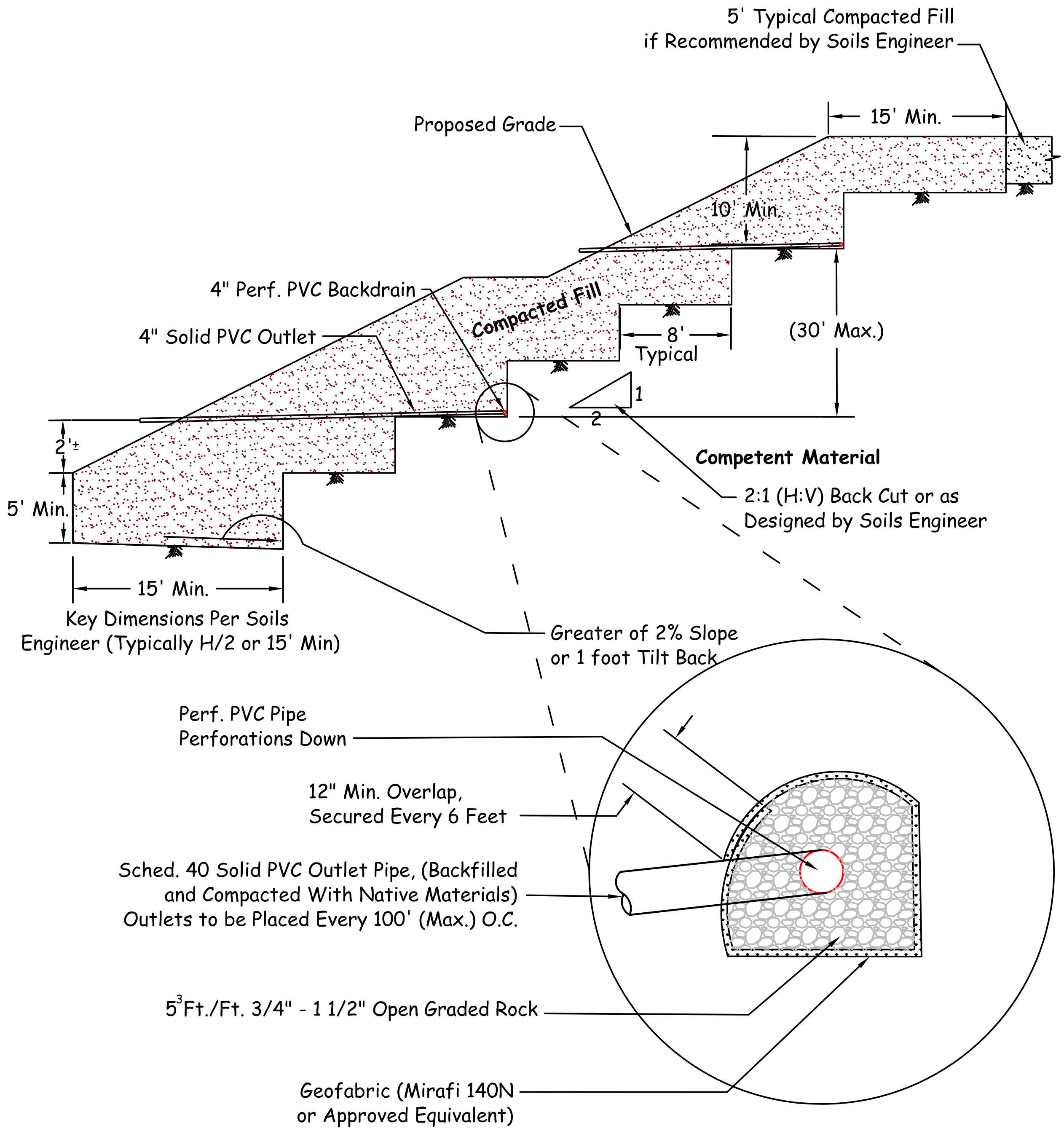
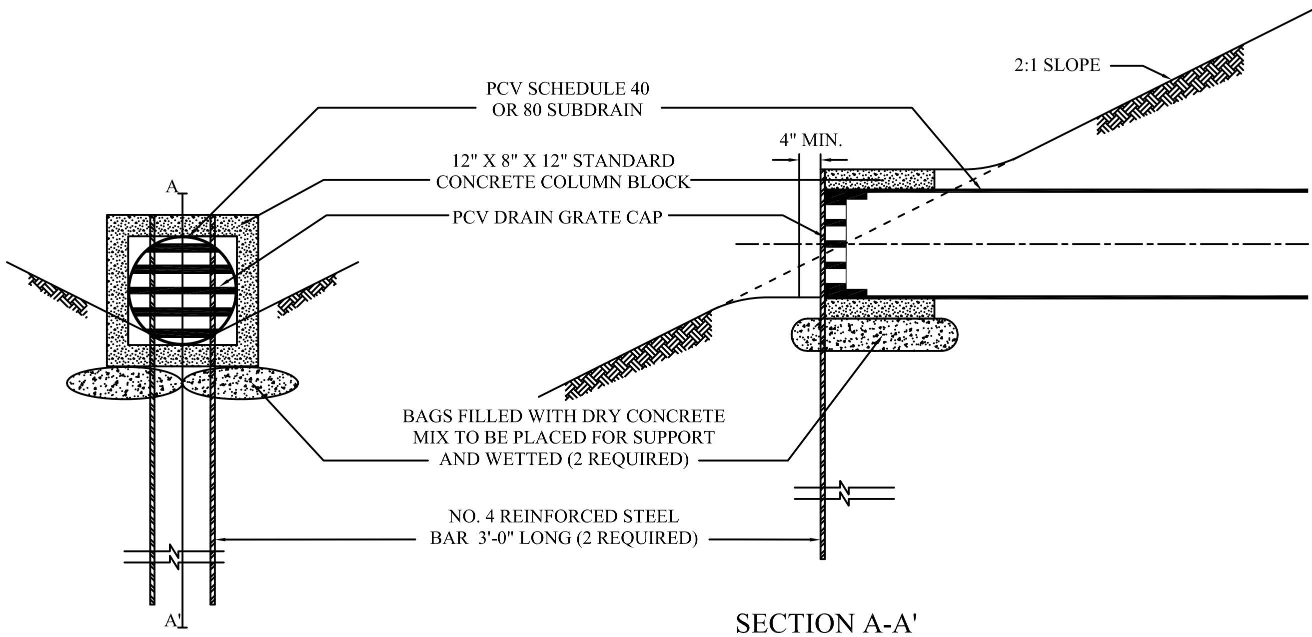
TYPICAL BUTTRESS DETAIL



LGC Geotechnical, Inc.

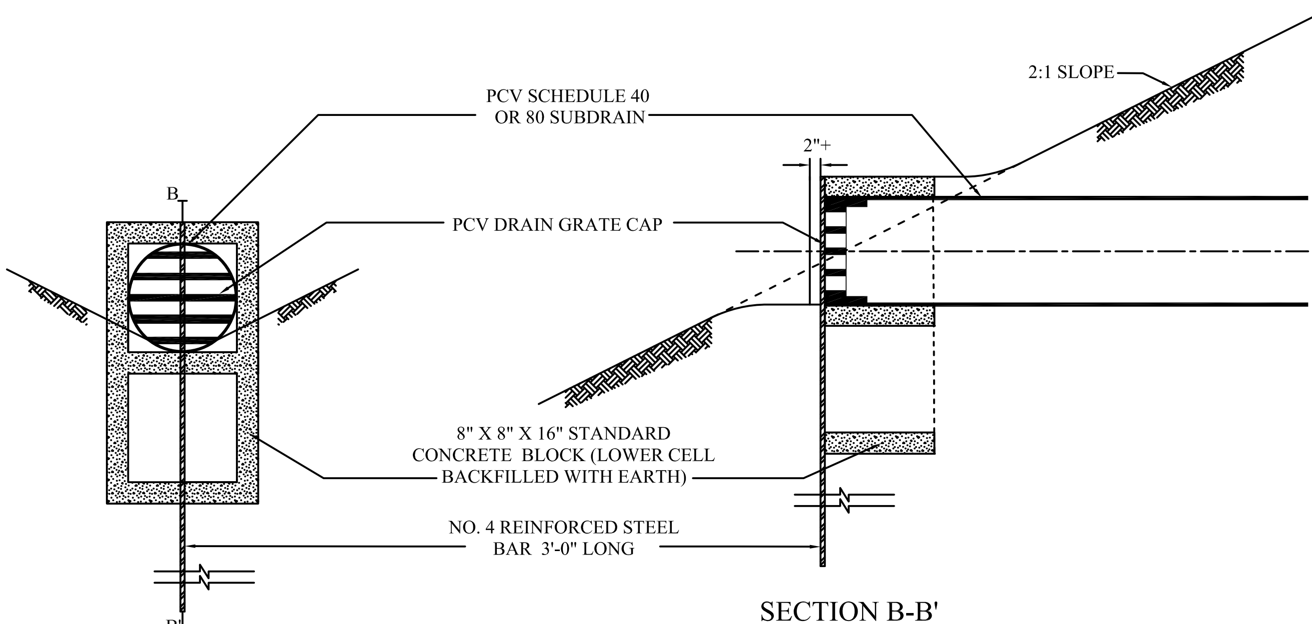
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SECTION A-A'

SUBDRAIN OUTLET MARKER -4" PIPE



SECTION B-B'

NOT TO SCALE

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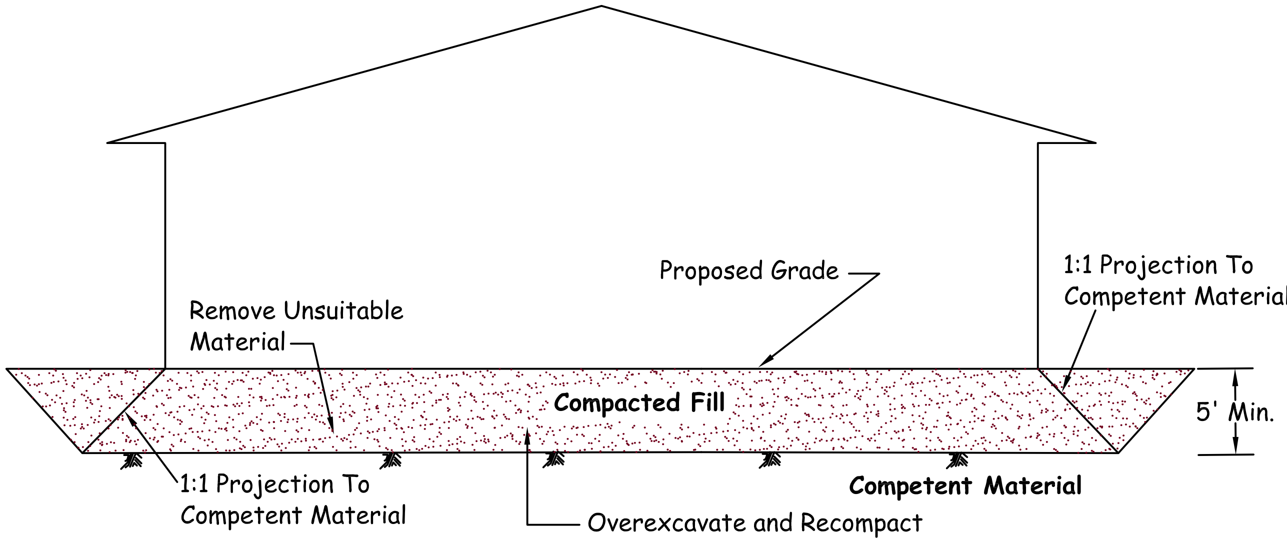
SUBDRAIN OUTLET
MARKER DETAIL

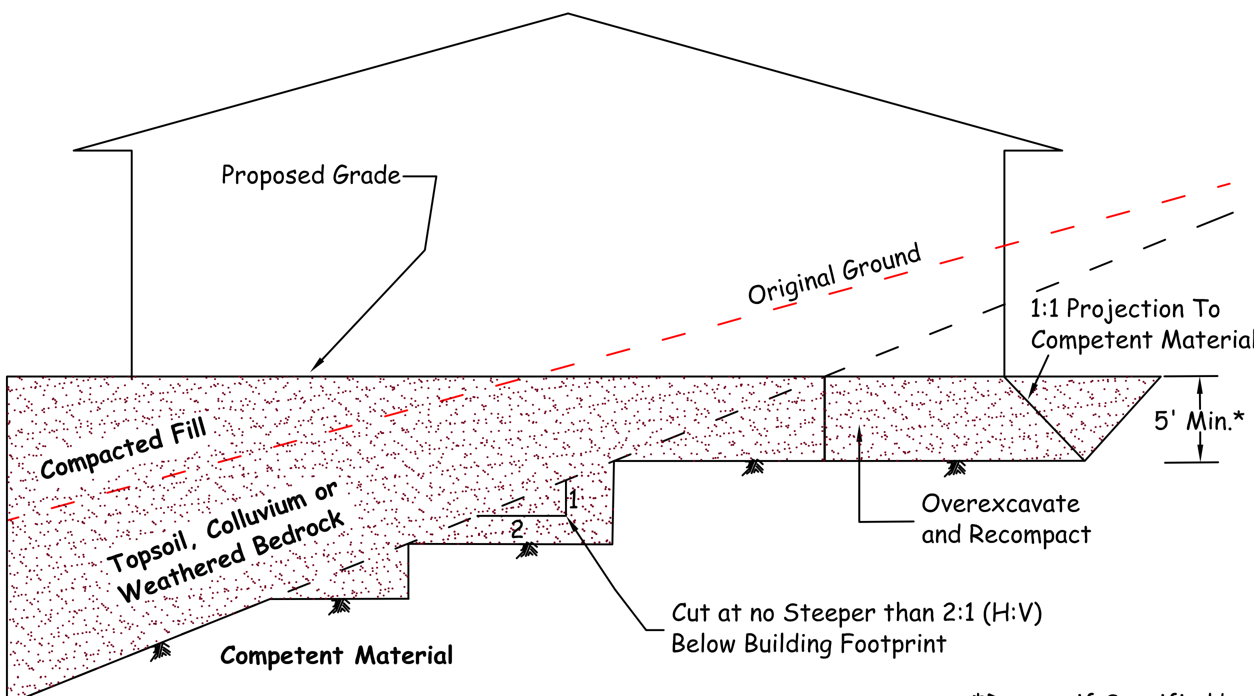
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



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

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

Cut/Fill Transition Lot



*Deeper if Specified by Soils Engineer

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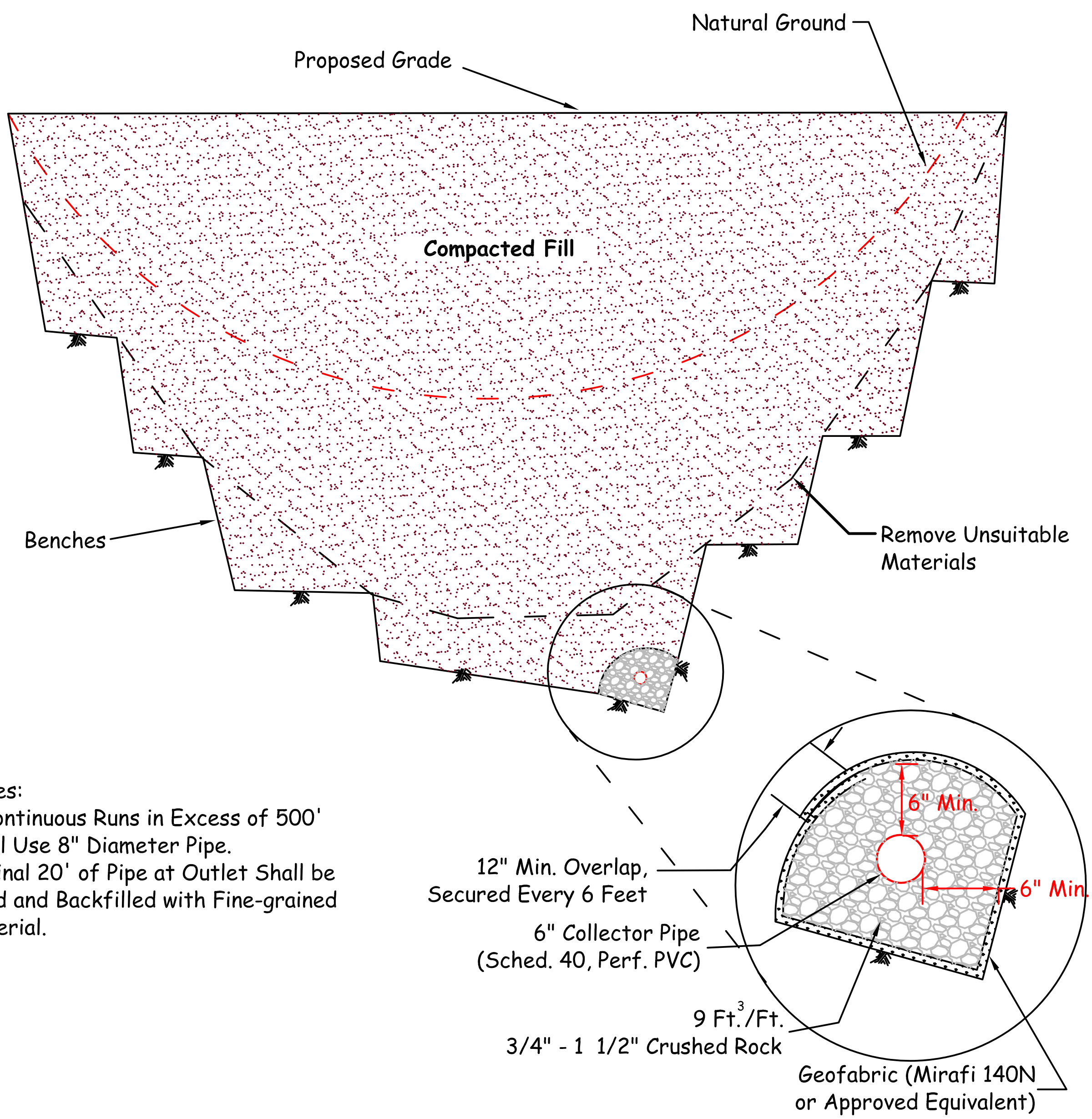
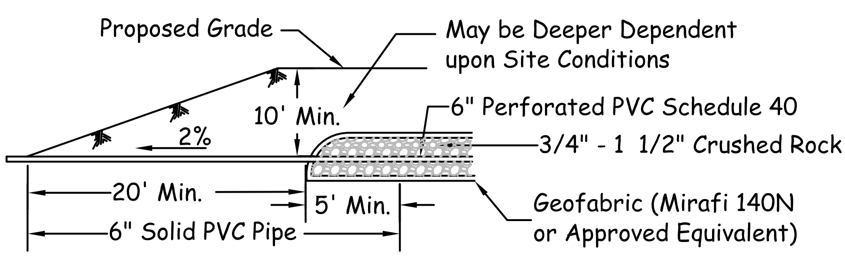
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



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

Proposed Outlet Detail



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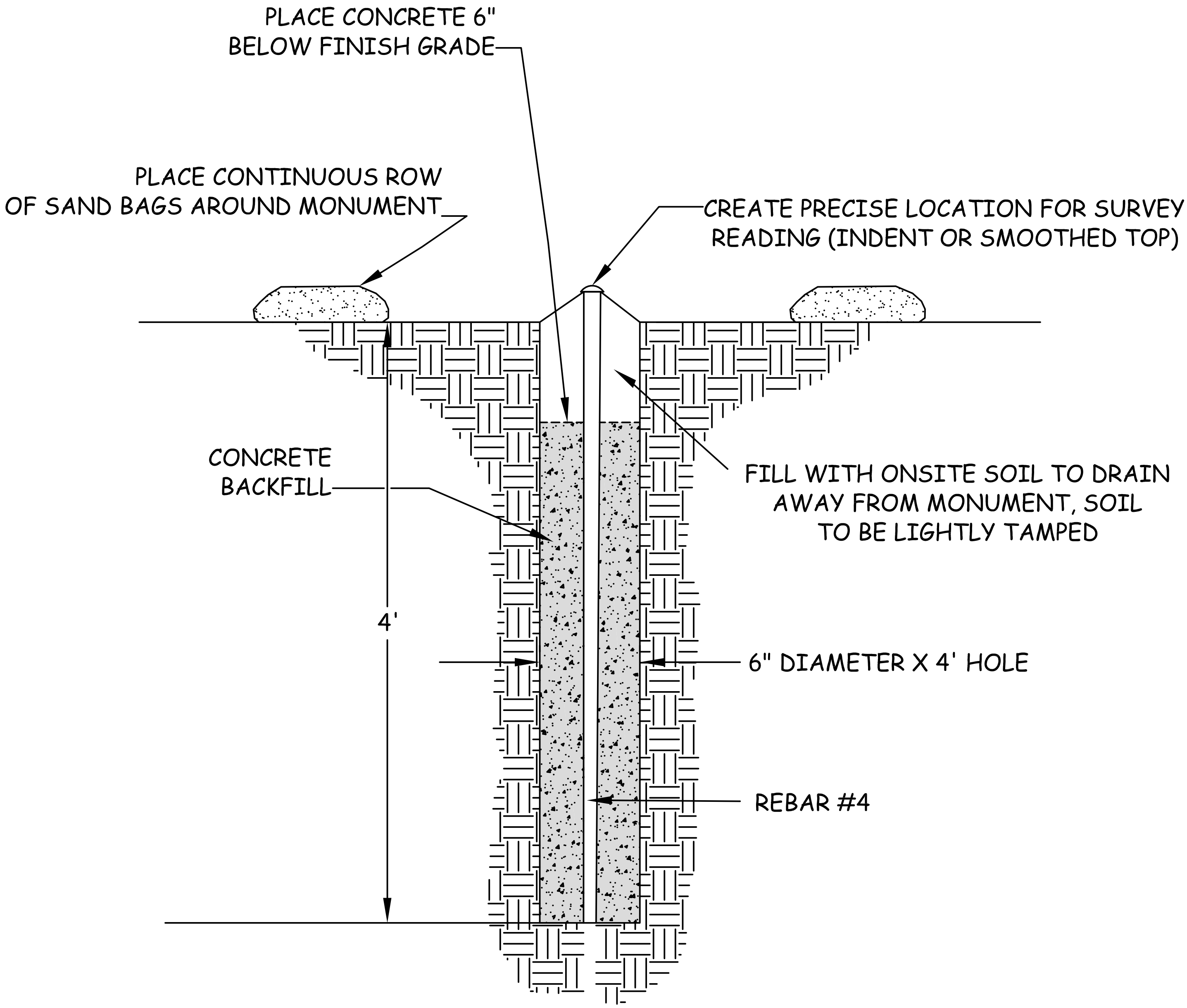
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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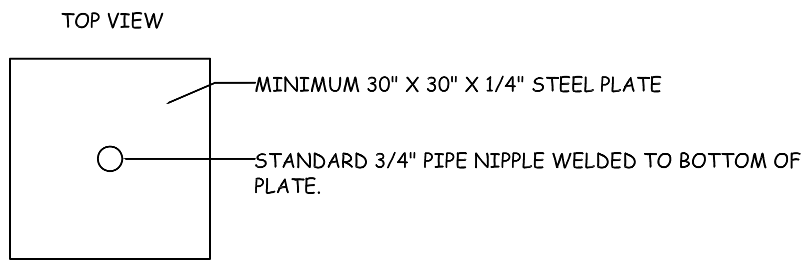
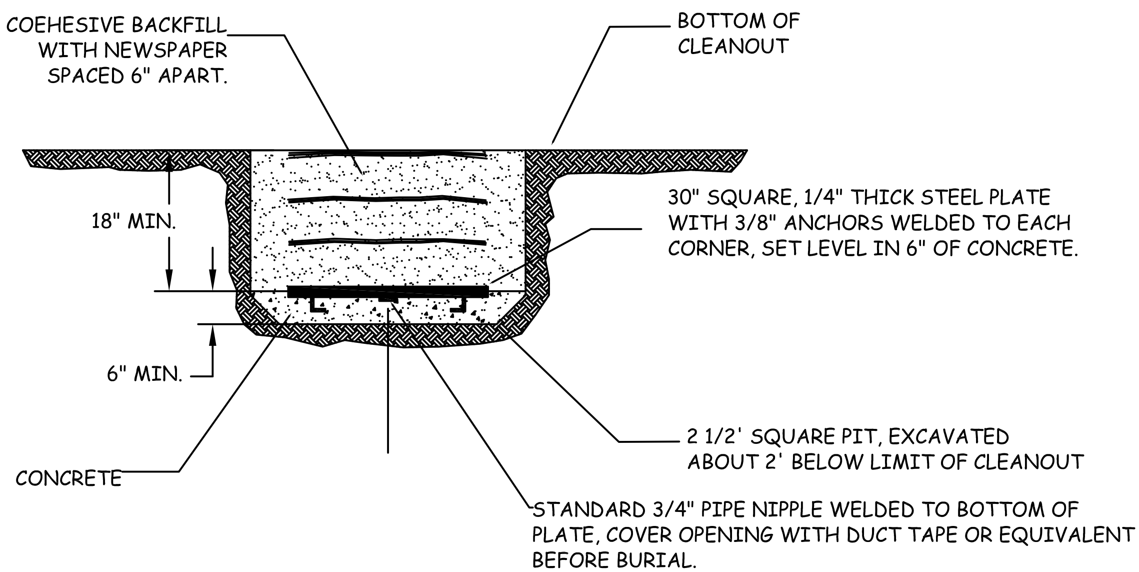
TYPICAL SURFACE SETTLEMENT MONUMENT





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

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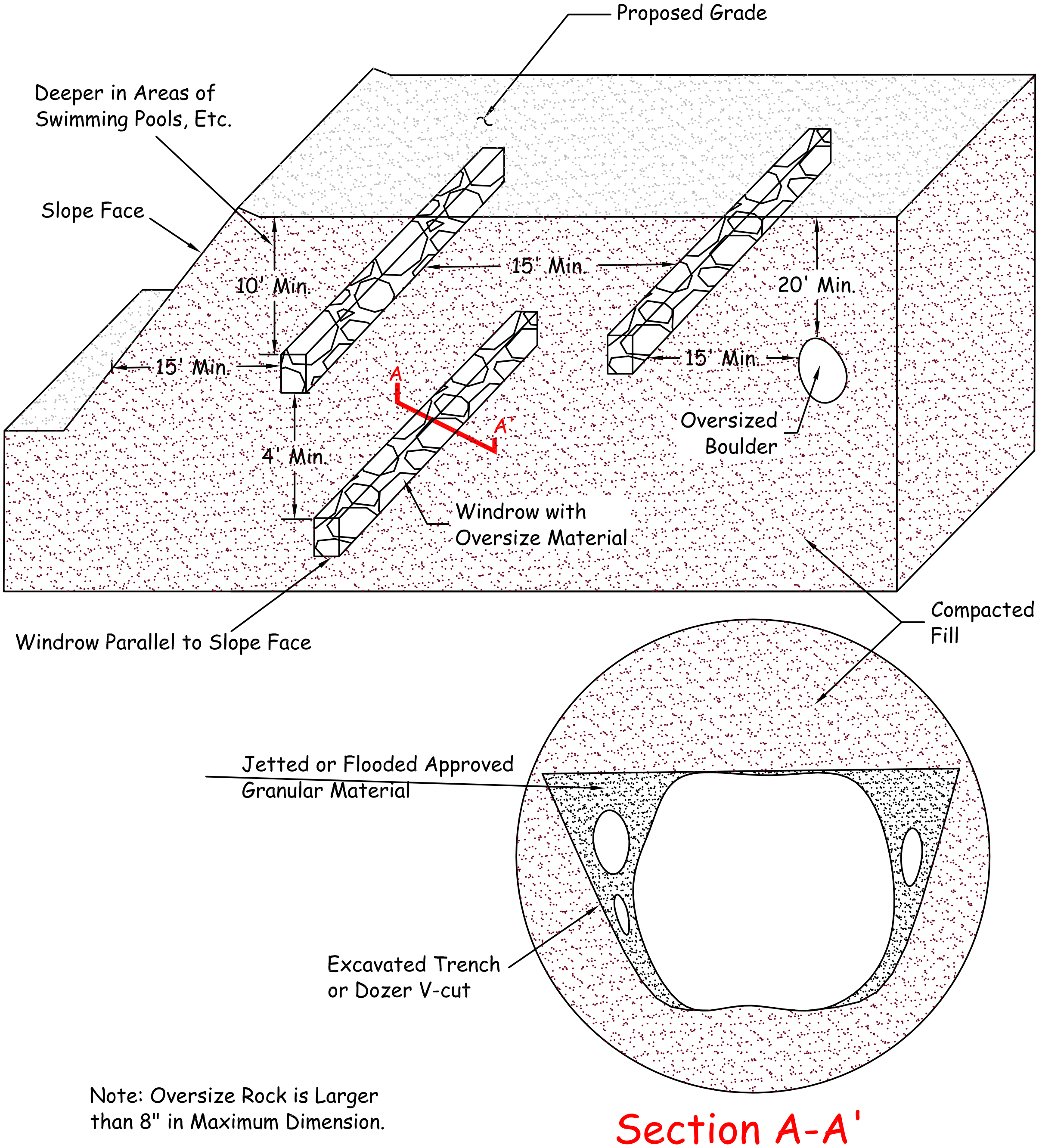
TYPICAL SETTLEMENT PLATE AND RISER



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OVERSIZE ROCK DISPOSAL DETAIL

LEGEND

HS-2

T.D. = 10'

Proposed Location of Hollow Stem Boring by LGC Geotechnical, with Approximate Proposed Depth in Feet

I-1

T.D. = 5'

Proposed Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, with Approximate Proposed Depth in Feet

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

Geotechnical Exploration Location Map

PROJECT NAME	*Great Scott - Lake Forest*
PROJECT NO.	*19035-01*
ENG. / GEOL.	*RLD / KTM*
SCALE	*1" = 50'*
DATE	*September 2023*

SHEET 1 of 1