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LGC Valley, Inc.
Geotechnical Consulting

PRELIMINARY GEOTECHNICAL INVESTIGATION, PROPOSED RESIDENTIAL DEVELOPMENT, 23161 MILL CREEK DRIVE, LAGUNA HILLS, CALIFORNIA

Dated: August 15, 2024

Project No. 244006-01

Prepared For:

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Ms. Aylene Chu
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Subject: ***Preliminary Geotechnical Investigation, Proposed Residential Development, 23161 Mill Creek Drive, Laguna Hills, California***

In accordance with your request, LGC Valley, Inc. (LGC) has performed a preliminary geotechnical investigation for the proposed multi-family residential development located at 23161 Mill Creek Drive (Assessor Parcel Number [APN] 588-142-07) in the City of Laguna Hills, California. The purpose of our geotechnical investigation was to evaluate the existing on-site geotechnical conditions relative to the proposed multi-family residential development of the site and to provide geotechnical recommendations applicable to the grading operations and future site construction for the project.

Our geotechnical study included: 1) review of the available geotechnical as-graded documents of the site and adjacent properties, and pertinent geotechnical and geologic reports and maps relative to the general vicinity; 2) a site reconnaissance, geologic mapping, and field exploration consisting of the excavation of six small-diameter borings; 3) laboratory testing of representative on-site soil samples; 4) geotechnical analysis of the collected data; and 5) preparation of this report that includes our findings, conclusions, opinions, and recommendations relative to the grading and development of the site.

Based on the results of our preliminary geotechnical investigation, it is our professional opinion that the proposed site development is feasible from a geotechnical standpoint provided the recommendations included in this report are incorporated into the project plans and specifications and followed during site grading and construction. If you have any questions regarding our report, please contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LGC VALLEY, INC.

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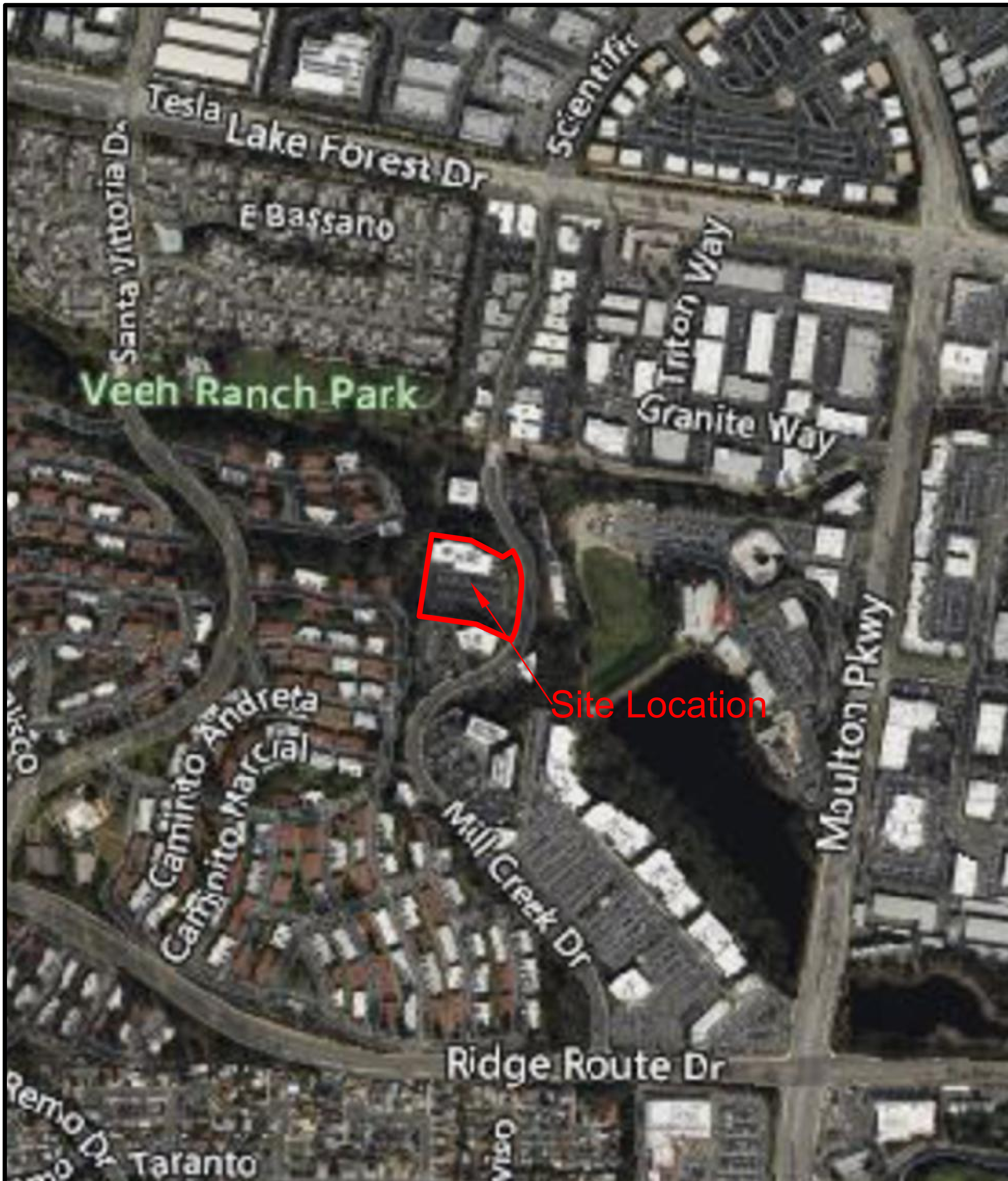
1.0 INTRODUCTION

1.1 Purpose and Scope of Services

The purpose of this preliminary geotechnical investigation was to identify and evaluate the existing geologic and geotechnical conditions at the subject site (Figure 1) and to provide preliminary geotechnical design criteria relative to the proposed multi-family development of the site. Recommendations for site grading, construction, preliminary foundation design for the proposed structures, and other relevant aspects of the proposed development are included herein to address the identified site geotechnical conditions.

Our scope-of-services for preparation of this document included:

- Review of available reports including prior geotechnical investigations, as-graded geotechnical documents, and other geotechnical related documents of the site and adjacent properties; and pertinent published geologic, geotechnical, and seismic reports and maps relative to the general vicinity (Appendix A).
- A site reconnaissance and geologic mapping of the site.
- A subsurface investigation consisting of the excavation, sampling, and logging of six small-diameter hollow-stem borings. The locations of the borings are shown on the Geotechnical Map (Figure 2). The logs of the borings are presented in Appendix B. The test borings were sampled and logged under the supervision of a licensed engineering geologist from our firm. The excavations were performed to evaluate the general characteristics of the subsurface conditions of the site including the classification of site soils and the determination of the depth to competent soil.
- Laboratory testing of representative soil samples obtained during our investigation (Appendix C).
- Review of boring logs from the geotechnical studies previously performed on the site and/or adjacent sites. The applicable boring logs are presented in Appendix D.
- Determination of seismic design parameters based on current building code requirements.
- Geotechnical analyses and evaluation of the data obtained during this study.
- Preparation of this report presenting our findings, conclusions, opinions, and recommendations including the General Earthwork and Grading Specifications for Rough Grading (Appendix E) with respect to the evaluated geotechnical conditions at the site and the proposed development.



Basemap: ESRI World Imagery Basemap, Accessed August 13, 2024

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Figure 1
Site Location Map
23161 Mill Creek Drive
Laguna Hills, California

Project Name	Toll/23161 Mill Creek
Project No.	244006-01
Eng. / Geol.	BIH/RKW
Scale	Not-to-Scale
Date	August 15, 2024

1.2 Site and Project Description

The approximately 2.44-acre site is located on the west side of Mill Creek Drive southwest of the intersection of Mill Creek Drive and Lake Forest Drive in the northwestern portion of the City of Laguna Hills, California (Figure 1). The site is bounded by Mill Creek Drive on the east, existing business developments on the north (23091 Mill Creek Drive) and south (23201 Mill Creek Drive), and a residential development on the west. The site currently consists of one three-story commercial building in the northern portion of the site and a large, paved parking lot in the central and southern portions of the site. Additional improvements include underground utility lines, retaining walls along the south and west sides of the parking lot, landscaping, and associated improvements. A north- to east-facing slope up to approximately 40 to 45 feet in height is present on the north and east sides of the property that descends to the parking lot of 23091 Mill Creek Drive on the north and Mill Creek Drive on the east. An approximately 10 to 15-foot-tall north-facing slope is present along the south side of the property.

Review of available aerial photographs, historical Google Earth images, old topography maps, and geotechnical reports of adjacent properties (Appendix A) indicate that the site (and general vicinity) was rough graded in the late 1970's/early 1980's and the site was precise graded in 1985/1986. The as-graded geotechnical reports and grading plans specific to 23161 Mill Creek were not available at the City of Laguna Hills; and consequently, were not reviewed by us for the purposes of this study.

Prior to the grading of the site, the property was located along the north side of a northwest-southeast trending ridgeline. Mill Creek Drive along the east side of the property was the location of a small tributary canyon that was partially in-filled during the grading of the area. Based on Google Earth images, elevations of the relatively flat portion of the property range from approximately 297 to 301 feet along the north side of the property to 300 to 316 feet along the south side. Based on the original topography, current site grades, and our limited subsurface investigation, we anticipate that the prior grading of the site consisted of a cut-fill condition with shallow fills located across the site (existing parking lot area) and deeper fills along the northern side of the property (and beneath the existing building) with backfill soils behind the on-site retaining walls.

1.3 Proposed Development

We understand that the proposed development will consist of a multi-family townhome residential development consisting of nine 3- to 6-unit structures (totaling 43 units). Preliminary plans show perimeter retaining walls are being considered around the perimeter of the site consisting of a combination of block walls, pile walls, and segmental (MSE) walls ranging in height from 4 to 17 feet. Anticipated associated improvements will include asphalt concrete (AC) streets and parking areas, a gated entrance, Portland cement concrete (PCC) sidewalks, open-space areas/landscaping, underground utility lines, a storm water biofiltration system, and associated improvements.

1.4 *Subsurface Investigation and Laboratory Testing*

Our subsurface investigation was performed on May 8, 2024, and consisted of the excavation, sampling, and logging of six small diameter hollow-stem borings (designated Borings LGC-B-1 through LGC-B-6) excavated to depths ranging from approximately 10.5 to 20.5 feet. All of the borings were extended into competent formational material. The logs of the borings are presented in Appendix B while the approximate locations of the borings are shown on the Geotechnical Map (Figure 1). At least three days prior to the start of our subsurface investigation, the proposed boring excavation locations were staked, Underground Service Alert (USA) notified, and the applicable information provided to them. The USA Dig Alert Number for this project is A241230333.

During the subsurface investigation, representative bulk samples and relatively undisturbed samples were collected for laboratory testing, and were forwarded to EGLAB, Inc. (EGL) and to LGC Valley, Inc. for classification testing. Laboratory testing was performed on representative soil samples and included moisture/density, Atterberg Limit, sieve analysis, consolidation, expansion index, direct shear, remolded direct shear, maximum density, sulfate content, chloride content, pH, and minimum resistivity testing. A summary of the test procedures and printouts of the laboratory test results are presented in Appendix C. The moisture and density test results were presented on the boring logs included in Appendix B.

1.5 *Existing As-Graded Geotechnical Conditions*

Based on our review of available PDF copies of public records obtained from the City of Laguna Hills (see References - Appendix A); a number of different preliminary geotechnical investigations and as-grade reports have been prepared for adjacent properties. However, the preliminary geotechnical investigations and as-grade reports for the subject property (23161 Mill Creek Drive) and the property to the south (23201 Mill Creek Drive) were not available for our review. The findings and conclusions of the available geotechnical reports relative to 23091 Mill Creek Drive (located to the north of the subject site) are discussed below.

2.0 GEOTECHNICAL CONDITIONS

2.1 Site Geology

Based on our subsurface exploration and review of the available previous geotechnical reports for the property and pertinent geologic literature and maps (Appendix A), the bedrock unit at the site is the Tertiary-aged Sespe Formation. The bedrock is overlain by older artificial fills placed during previous development of the site. The approximate extent of the mapped geologic units present on the site is depicted on the Geotechnical Map (Figure 2). A brief description of the geologic units encountered on the site is presented below.

2.1.1 Artificial Older Fill (Afo)

Based on our review of the available geotechnical reports of the site and the general vicinity, it is our understanding that the on-site existing artificial fills (up to approximately 25+ feet in thickness) were placed and documented by Irvine Soils Engineering, Inc. (1980) and G.A. Nicoll and Associates, Inc. (1986); however, these reports were not available for our review.

Existing artificial fills were encountered in our recent geotechnical borings within the existing parking areas across the site and were found to consist of silty to clayey sands that were slightly moist and medium dense. The depths of the encountered fills ranged from approximately 1 to 5 feet in thickness. Based on the available data, the deeper existing fills are located underlying the existing structure and slope located along the northern portion of the site. Due to the existing site improvements and access issues, this deeper fill was not investigated as a part of this study.

Based on our understanding of the placement/documentation of the existing older fills and our observations and testing of the encountered fills on-site, it is our preliminary conclusion that the existing fills are considered suitable for support of the proposed structures, provided remedial removals are performed of any fills disturbed during site demolition. For confirmation purposes, an additional evaluation of the fill soils (after the existing building is demolished and removed) should be performed. This additional evaluation will need to include surface excavation (drilling), sampling, and laboratory testing to determine the competency of the existing fill soils, and suitability for support of the proposed structures. Additional remedial removal or foundation recommendations will be provided at that time, as necessary.

2.1.2 Tertiary-Aged Sespe Formation (Ts)

Based on our review of geologic maps of the site and general vicinity, the site is underlain by massive to thickly bedded non-marine gray to red pebbly sandstones and silty to clayey sandstones with interbedded siltstones and claystones of the Tertiary-aged Sespe Formation (Morton, Hauser, and Ruppert, 1999; and Morton and Miller, 2006). The results of our subsurface investigation indicated that the Sespe Formation soils encountered consisted of fine to coarse sandstone, silty and clayey fine to medium sandstone, and sandy to clayey siltstone with little to no pebbles.

2.2 Geologic Structure

Based on our review of geotechnical documents relative to the property directly north of the site, it appears that a large-diameter boring was excavated in the northern portion of the 23161 Mill Creek Drive property in 1986 by G. A. Nicoll and Associates Inc. (Lawmaster & Co, Inc., 1989). The location of the boring (Boring N-B-2) and the bedding attitudes obtained in the boring are shown on Figure 2. This information was obtained from a geotechnical investigation map of 23091 Mill Creek Drive (Lawmaster, 1989). As indicated, the bedding observed in the Sespe Formation ranges from N68W to N65E dipping 5 to 33 degrees to the south. As a result, bedding is dipping into-the-slope and is considered favorable in regard to slope stability. It should be noted that the geotechnical investigation report by G.A. Nicoll and Associates in 1986 that includes the boring log and the description of the subsurface investigation was not available for our review.

2.3 Landslides

Based on our review of aerial photographs and available geotechnical reports for the site, (Appendix A), no evidence of landsliding or other slope instability conditions were observed on-site or noted in the literature. Consequently, the potential for the existence of landslides is considered insignificant.

2.4 Groundwater

Review of available groundwater related internet web sites (Appendix A) indicates that groundwater is likely greater than 50 feet below the ground surface at the site. Groundwater is relatively shallow (generally less than 20 feet below the ground surface) in the large alluvial valley north and northeast of the site. As a result, groundwater is not expected to have a significant impact to the proposed development (provided the recommendations of this report are implemented during the design, grading, and construction of the proposed site improvements).

2.5 Surface Water and Flooding

Based on our review of the existing site grading plans and topography map of the site, sheet flow across the site is to the southeast. Surface water runoff relative to project design is the purview of the project civil engineer and should be directed away from planned structures. Due to the elevation of the site relative to drainages, flooding at the site is considered insignificant.

2.6 ***Faulting***

The southern California region has long been recognized as being seismically active. The seismic activity results from several active faults that cross the region, all of which are related to the San Andreas transform system, a broad zone of right lateral faults that extend from Baja California to Cape Mendocino. The numerous faults in Southern California include Holocene active, pre-Holocene active, and age-undetermined faults. The definitions of fault activity terms used here are based on those developed for the Alquist-Priolo Special Studies Zone/Earthquake Zone Act of 1972 (CGS, 2018).

Holocene active faults are those faults that have had surface displacement within Holocene time (approximately the last 11,700 years) and have been included within an Alquist-Priolo Earthquake Zone. Pre-Holocene faults have not moved in the last 11,700 years but may be considered potentially active. Age-undetermined faults are faults where the recency of fault movement has not been determined and may likely be considered inactive faults.

The site is not within the currently established Alquist-Priolo Earthquake Fault Zone for fault rupture hazard (formerly Special Studies Zones for fault rupture hazard). Based on a review of geologic literature, no active faults are known to occur beneath or in the general vicinity of the project site. Accordingly, it appears that there is little probability of surface rupture due to faulting beneath the site. There are, however, several faults located in sufficiently close proximity that movement associated with them could cause significant ground motion at the site.

Regional active faults that occur within the Laguna Hills and southern California area include the Newport-Inglewood fault zone to the west and the San Joaquin Hills blind thrust fault, Whittier-Elsinore, San Jacinto, and San Andreas faults to the east. The closest known active faults to the site are the San Joaquin blind thrust Hills fault located approximately 1 mile (1.7 kilometers) to the northeast, the Newport-Inglewood fault zone located approximately 7 miles (11.5 kilometers) to the southwest, and the Elsinore fault zone located approximately 16.5 miles (26.5 kilometers) to the northeast.

The main seismic parameters to be considered when discussing the potential for earthquake-induced damage are the distances to the causative faults, earthquake magnitudes, and expected ground accelerations. Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region include soil liquefaction and dynamic settlement. Other secondary seismic effects include shallow ground rupture, seiches and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and the causative fault and the on-site geology.

2.7 ***Seismicity and Related Effects***

We have performed site-specific analysis based on these seismic parameters for the site and the on-site geologic conditions. The results of our analysis are discussed in terms of the potential seismic events that could be produced by the maximum probable earthquakes. A maximum probable earthquake is the maximum earthquake likely to occur given the known tectonic framework.

2.7.1 Seismic Design Criteria

Based on the 2022 California Building Code (CBC) the site seismic characteristics for the project were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 CBC. Representative site coordinates for the project site of Latitude 33.625942º N and Longitude -117.735614º W were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (SMS and SM1) and adjusted design spectral response acceleration parameters (SDS and SD1) for Site Class C are provided in the following table.

Table 1

California Building Code Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class (per Chapter 20 of ASCE 7)	C
Risk-Targeted Spectral Acceleration for Short Periods (S_S)	1.223g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)	0.439g
Site Coefficient F_a [per CBC Table 1613.2.3(1)]	1.2
Site Coefficient F_v [per CBC Table 1613.2.3(2)]	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) [Note: $S_{MS} = F_a S_S$]	1.467g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) [Note: $S_{M1} = F_v S_1$]	0.659g
Design Spectral Acceleration for Short Periods (S_{DS}) [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.978g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.439g
Seismic Design Category (per CBC Section 1613.2.5)	D

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake ground motions, Peak Ground Acceleration (PGA) should be used for the geotechnical evaluations. The PGA_M for the site is equal to 0.617g **(**USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.89 at approximately 3.33 km (2.07 mi) from the site would contribute the most to this ground motion (USGS, 2014).

2.7.2 Lurching and Shallow Ground Rupture

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are not likely to be significant where the thickness of soft sediments do not vary appreciably under structures. Although there are several nearby active and potentially active faults, the native soils are dense, and no active faults are known or interpreted to cross the site. Based on this data, it is our opinion that the potential for lurching or shallow rupture at the site is very low to nil.

2.7.3 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 exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Liquefaction is typified by a buildup of pore-water pressure in the affected soil layer to a point where a total loss of shear strength occurs, causing the soil to behave as a liquid. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures.

Based on the anticipated relative density of the on-site soils and the depth to the static groundwater in the area of the proposed development; it is our opinion that the potential for liquefaction impacting the site is nil, and seismically induced settlements are considered to be negligible.

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry, or saturated granular soil. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based on in-situ densities, and soil types, dry sand settlement and induced surface manifestations are not considered an issue at the site.

2.7.4 Tsunamis and Seiches

Due to the elevation of the proposed development at the site with respect to sea level and its distance from large open bodies of water, the potential of seiches and/or tsunami is considered to be very low.

2.8 Slope Stability

Based on the massive to thickly bedded Sespe Formation on the site, favorable (into-the-slope) geologic bedding conditions, and the construction of the proposed segmental geogrid retaining walls along the north and east sides of the property and the block and pile retaining walls along the west and south sides of the property, slope stability is not considered an issue with respect to site development. However, as part of our geotechnical review of the proposed segmental geogrid retaining walls and pile walls, a global slope stability analysis will be performed once the segmental wall designs are completed. The results will be presented in a future retaining wall plan review letter.

2.9 Expansion Potential

Expansion potential testing of representative on-site soil samples indicates that the on-site soils have a very low to low expansion potential (Appendix C). Although some of the near surface soils (Sespe Formation claystone layers) may have a medium to high expansion potential, we would anticipate the mixture and redistribution of site soils will result in a low expansion potential for the site. The as-graded soil conditions of the proposed building pads should be verified with confirmatory observation, sampling, and testing after site grading is completed and prior to the construction of the structures.

2.10 Soluble Sulfate and Corrosivity of the On-Site Soils

Laboratory testing of representative on-site soils indicated the on-site soils tested had soluble sulfate contents that ranged from 0.005 to 0.007 percent (Appendix C). As a result, the on-site soils should be considered to have a negligible degree of corrosivity to concrete and have an Exposure Class of S0 in accordance with ACI 318R-14 Table 19.3.1.1. The as-graded sulfate content of the finish grade soils on the building pads should be verified upon completion of grading.

Minimum resistivity, pH, and chloride content tests were also performed on two representative soil samples obtained during our subsurface investigation. These corrosion tests resulted in a minimum resistivity of 1,300 to 1,700 ohm-centimeters, a pH of 7.82 to 8.15, and a chloride content of 125 to 140 ppm (Appendix C). Based on these results, the on-site soils should be considered moderately corrosive to buried metals in contact with the on-site soils.

2.11 Excavation Characteristics

The site is underlain by near surface soils consisting of silty fine to coarse sands, clayey to fine sandy silts, silty clay, siltstone/claystone, and sandstone layers. It is anticipated that the on-site materials can be excavated with conventional heavy-duty construction equipment and that difficult excavation and/or blasting is not anticipated. In addition, the anticipated site excavation and the proposed construction will not have an adverse impact on the adjacent properties.

2.12 Earthwork Shrinkage and Bulking

The volume change of excavated on-site materials upon recompaction as fill is expected to vary with materials and location. Typically, the surficial soils and bedrock materials vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimate cannot be determined. However, the following factors (based on the results of our subsurface investigation and previous investigations of the site, laboratory testing, geotechnical analysis, and professional experience on nearby sites) are provided in Table 2 as guideline estimates. If possible, we suggest an area where site grades can be adjusted be provided as a balance area.

Table 2

Earthwork Shrinkage and Bulking Estimates

Geologic Unit	Estimated Shrinkage/Bulking
Artificial Fill and upper 1 to 2 feet of the site	3 to 7 percent shrinkage
Sespe Formation	0 to 5 percent bulking

2.12 Stormwater Infiltration/Percolation

Based on our understanding of site development, on-site storm water will flow into proprietary underground biofiltration BMP structures that will treat the storm water and then discharge it into the on-site storm drain system that is connected to the storm drain system in Mill Creek Drive. We also understand that site infiltration is not currently proposed. However, a future percolation/infiltration study may be required to determine the feasibility of storm water subsurface infiltration for the site.

Our limited subsurface investigation of the site that was performed on May 8, 2024 and consisted of the excavation, sampling, and logging of six small diameter hollow-stem borings excavated to depths ranging from approximately 10.5 to 20.5 feet, indicated the near surface soils consist of fill soils and Tertiary-Aged Sespe Formation. As encountered, the Sespe Formation soils consisted of fine to coarse sandstone, silty and clayey fine to medium sandstone, and sandy to clayey siltstone with little to no pebbles. Based on our professional experience with similar soils, infiltration of these sandy soils are likely feasible; however, the results will likely have a relatively low infiltration rate (slightly above the typical minimum required rate of 0.5 inches per hour). Infiltration into the fill soils and along the north, northeast, and east sides of the site are not recommended due to the presence of descending slopes, large retaining walls, and fill soils that will have a reduced factor-of-safety as the result of subsurface groundwater.

Once the final storm water infiltration design is determined for the site, a percolation/infiltration study (in accordance with the City of Laguna Hills/County of Orange requirements) should be performed to determine the actual percolation/infiltration rate(s) at the proposed storm water infiltration locations and appropriate factor-of-safeties.

3.0 CONCLUSIONS

Based on the results of our geotechnical investigation, evaluation, and review; it is our professional opinion that the proposed site development is feasible from a geotechnical standpoint, provided the recommendations included in this report are incorporated into the project plans and specifications, and followed during site grading and construction. Our geotechnical conclusions are as follows:

- Review of available aerial photographs, historical Google Earth images, old topography maps, and geotechnical reports of adjacent properties (Appendix A) indicate that the site (and general vicinity) was rough graded in the early 1980's and the site was precise graded in 1986. The as-graded geotechnical reports and grading plans specific to 23161 Mill Creek were not available at the City of Laguna Hills; and consequently were not reviewed by us.

- Based on our subsurface exploration and review of the available previous geotechnical reports for the property and pertinent geologic literature and maps, the bedrock unit at the site is the Tertiary-aged Sespe Formation that is overlain by artificial older fills. The fill soils are a mixture of silty to clayey sands, silty fine to coarse sands, clayey to fine sandy silts, and silty clay. The Sespe Formation generally consisted of massive to thickly bedded fine to coarse sandstone, silty and clayey fine to medium sandstone, and sandy to clayey siltstone with little to no pebbles. The fill soil layers encountered in our current geotechnical investigation were found to be competent and suitable for support of proposed structures, provided the recommendations of this report are followed during site grading and construction.

- After the removal of the existing buried underground utilities has been made, the trench locations should be surveyed, and the excavations should be replaced with fill soils compacted to a minimum 90 percent relative compaction (based on ASTM D1557). Where these trenches are located within the limits of the proposed multi-family building foundation footprints, special consideration should be made as the majority of the site is cut and the trenches will create cut/fill transition conditions. Mitigation measures may include overexcavation of the upper 5 feet of the pad and replacement with compacted fill or backfilling the trenches with a 2-sack sand/cement slurry.

- The existing on-site soils appear to be suitable material for use as fill provided, they are relatively free of rocks (larger than 8 inches in maximum dimension), organic material, and debris.

- The on-site formational material is anticipated to be massive to thickly bedded. The bedding observed in the Sespe Formation ranges from N68W to N65E dipping 5 to 33 degrees to the south (Lawmaster & Co, Inc., 1989). As a result, bedding is dipping into-the-slope and is considered favorable in regard to slope stability.

- There are no known landslides or other slope instability issues impacting the site.

- Review of available groundwater related internet web sites (Appendix A) indicates that groundwater is likely greater than 50 feet below the ground surface at the site. Groundwater was not encountered during our recent subsurface investigation. Consequently, groundwater is not expected to have an impact on the proposed development.

- Based on the topography of the site, surface water is anticipated to flow across the sheet-graded pad from the west to southeast.

- Due to the elevation of the site relative to existing drainages, flooding at the site is not considered an issue for the site.

- Active or potentially active faults are not known to exist on the site.

- The site is not located within the Alquist-Priolo Earthquake Fault Zone (CGS, 2018). The closest known Holocene active faults to the site are the San Joaquin blind thrust Hills fault located approximately 1 mile (1.7 kilometers) to the northeast, the Newport-Inglewood fault zone located approximately 7 miles (11.5 kilometers) to the southwest, and the Elsinore fault zone located approximately 16.5 miles (26.5 kilometers) to the northeast.

- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults.

- Based on the relatively dense nature of the proposed compacted fill soils and existing formational material and lack of a permanent shallow groundwater condition, liquefaction and/or dynamic settlement at the site is considered very low to nil.

- Due to the elevation of the proposed development at the site with respect to sea level and its distance from large open bodies of water, the potential of seiches and/or tsunami is considered to be nil.

- Based on the massive to thickly bedded Sespe Formation on the site, generally favorable geologic bedding conditions, and the construction of the proposed segmental geogrid retaining walls along the north and west sides of the property, slope stability is not considered an issue with respect to site development. However, as part of our geotechnical review of the proposed geogrid retaining walls and pile walls, a global slope stability analysis will be performed once the wall designs are completed. The results will be presented in a future retaining wall plan review letter.

- Expansion potential testing of representative samples of the on-site soils indicate those soils have a very low to low expansion potential (Appendix C). Although some of the near surface soils may have a medium to high expansion potential, most of the soils on the site have a low expansion potential. The as-graded soil conditions of the proposed building pads should be verified with confirmatory observation, sampling, and testing after site grading is completed and prior to the construction of the structures.

- Laboratory testing of representative on-site soils indicated the on-site soils should be considered to have a negligible degree of corrosivity to concrete and have an Exposure Class of S0 in accordance with ACI 318R-14 Table 19.3.1.1.

- Minimum resistivity, pH, and chloride content tests were also performed on the two representative soil samples obtained during our subsurface investigation and these results indicate the on-site soils should be considered moderately corrosive to buried metals in contact with the on-site soils.

- It is anticipated that the on-site materials can be excavated with conventional heavy-duty construction equipment and that difficult excavation and/or blasting is not anticipated.

- The anticipated site excavation and the proposed construction will not have an adverse impact on the adjacent properties.

- The multi-family residential structures may be designed to be supported by post-tension or mat foundation systems.

- In order to reduce the potential for differential settlement in areas of cut/fill transitions, we recommend the upper 5 feet of the building pad be overexcavated and replaced with properly compacted fill to mitigate the transition condition beneath the proposed structure. This condition is likely along the north and west sides of the property, where existing buried utility lines that are abandoned and the trenches replace with compacted fill, and possibly in the eastern portion of the site along the property boundary.

- Based on our professional experience with similar soils, infiltration of the on-site sandy formational soils are likely feasible; however, a percolation/infiltration study (in accordance with the City of Laguna Hills/County of Orange requirements) should be performed to determine the actual percolation/infiltration rate(s) at the proposed storm water infiltration locations and appropriate factor-of-safeties.

4.0 <u>RECOMMENDATIONS</u>

4.1 <u>Site Earthwork</u>

We anticipate that earthwork at the site will consist of site preparation and remedial grading and construction of site perimeter retaining walls followed by construction of the proposed slab-on-grade type foundations and associated improvements. We recommend that earthwork on-site be performed in accordance with the recommendations herein, the City of Laguna Hills grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the recommendations in the following sections shall supersede those included as part of Appendix E.

4.1.1 <u>Site Preparation</u>

Prior to grading of the area to receive structural fill or engineered structures, the ground surface should be cleared of obstructions, debris, potentially compressible material (such as undocumented fill soils, dry, disturbed, or desiccated documented fill and formational material and stripped of vegetation). Vegetation and debris should be removed and properly disposed of offsite. Holes resulting from the removal of buried obstructions or utilities, which extend below finished site grades, should be replaced with suitable compacted fill material. Areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

4.1.2 <u>Removal and Recompaction</u>

The upper portion of the site is underlain by potentially compressible/collapsible or unsuitable soils (i.e., desiccated existing fills, and formational material) which may settle under the addition of water, under the surcharge of fill, and/or foundation loads. Compressible materials not removed by the planned grading should be excavated to competent material (as determined by the geotechnical consultant) and replaced with compacted fill soils. From a geotechnical perspective, soil that is removed may be placed as fill provided the material is relatively free from rocks (greater than 8-inches in maximum dimension), organic material and construction debris; is moisture-conditioned or dried (as needed) to obtain above-optimum moisture content; and then recompacted prior to additional fill placement or construction.

All disturbed or desiccated fills underlying or within the influence of the proposed development should be removed to competent soils. The actual depth and extent of the required removals should be determined during grading operations by the geotechnical consultant; however, the estimated remedial removals are as follows: we anticipate removals of up to 3 feet below existing grades; however, localized, deeper removals should be anticipated where deemed necessary by the geotechnical consultant based on observations during grading.

4.1.3 ***Cut/Fill Transition Condition***

In order to reduce the potential for differential settlement of proposed buildings located across cut/fill transitions, we recommend the entire cut portion of the transition building pads be overexcavated to a minimum depth of 5 feet and replaced with properly compacted fill to mitigate the transition condition beneath the proposed structure; or the building foundations deepened so that all foundation footings are founded on formational material. Where existing buried utility lines that will be abandoned are located with the limits of the proposed building foundation footprint, special consideration should be made as the majority of the site is likely cut and the trenches will create cut/fill transition conditions. Mitigation measures may include overexcavation of the upper 5 feet of the pad and replacement with compacted fill or backfilling the trenches with a 2-sack sand/cement slurry.

4.1.4 ***Shrinkage/Bulking***

Based on the encountered site soils; the existing fills and upper 1 to 2 feet of the site is anticipated to shrink while the formational materials are anticipated to bulk. The preliminary estimated shrinkage and bulking factors are presented in Table 2 (in Section 2.12 of this report). The value ranges are preliminary rough estimates which will vary with depth of removal, stripping losses, field conditions at the time of grading, etc. In addition, handling losses is not included in the estimates. If possible, we suggest an area where site grades can be adjusted be provided as a balance area.

4.1.5 ***Temporary Excavation Stability***

In general, all 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. Soil conditions should be mapped and frequently checked by a representative of LGC 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 engineer should be maintained to facilitate construction while providing safe excavations. Excavation safety is the responsibility of the contractor.

Temporary excavations may be cut vertically up to five feet. Excavations over five feet should be slot-cut, shored, or cut no steeper than 1:1 (horizontal to vertical) slope gradient. If a block wall along the western property line adjacent to the existing off -site building is planned, shoring of the western property line will need to be provided to for support of the wall backcut and to maintain lateral support for the off-site structure. Surface water should be diverted away from any exposed cut, and not be allowed to pond on top of the excavations. Temporary cuts should not be left open for an extended period of time. Planned temporary conditions should be reviewed by the geotechnical consultant of record in order to reduce the potential for sidewall failure. The geotechnical consultant may provide recommendations for controlling the length of sidewall exposed.

4.1.6 Temporary Shoring

The following preliminary geotechnical parameters may be utilized by the shoring consultant for design of the temporary shoring system along the southern property boundary, as necessary. Temporary shoring is generally considered to have a service life of two years or less. The geotechnical conditions outside of the perimeter of the proposed site have not been investigated as part of this report. The recommendations provided herein with regard to shoring of the proposed excavation are based on assumed conditions, extrapolated from the data gathered from our review of the subject site and adjacent sites. The shoring designer should independently evaluate the parameters provided and conduct an additional investigation if they consider necessary.

Prior to construction, the contractor should verify underground clearance of any existing utility lines or structures that must be removed or protected in place during construction, or may conflict with any proposed shoring system. Any tieback anchors and/or soil nails that extend beyond the site property limits will require permission from the adjacent property owner. Special attention will be required to protect existing settlement sensitive improvement in close proximity to the proposed excavation, such as any adjacent structures or streets located along the boundary of the site.

Typical cantilever temporary shoring, where deflection of the shoring will not impact the performance of adjacent structures or streets, may be designed using the active equivalent fluid pressures of 40 pounds per square foot (psf) per foot of depth (or pcf) for a level condition, and a 50 pcf for a 2:1 (horizontal to vertical) sloping condition. Braced (i.e. internal bracing -rakers) or tied-back shoring is recommended in areas where the shoring will be located close to existing structures or streets in order to limit shoring defections or required due to the proposed depth of excavation. Braced or tied-back shoring with a level backfill may be designed using an active trapezoidal soil pressure of 24H in pounds per square foot (psf) per foot of depth (or pcf) for level condition and 30 pcf for a 2:1 (horizontal to vertical) sloping condition, where H is equal to the depth in feet of the excavation being shored (shape of the trapezoid should be 0.2H, 0.6H, 0.2H).

Any building, equipment, or traffic loads located within a 1:1 (horizontal to vertical) projection from the base of the shoring should be added to the applicable lateral earth pressure. A minimum additional uniform lateral pressure of 100 psf for the upper 10 feet should be added to the appropriate lateral earth pressures to account for typical vehicle traffic loading. The proposed shoring should be designed for a maximum shoring deflection of up to 1-inch adjacent to the street (non-surcharged condition) and up to a maximum of 0.5-inches adjacent to existing buildings (surcharged condition). Surcharge loads on basement walls and shoring systems from existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation, shoring, and basement should be determined and considered in the design.

If temporary gravity grouted tie-backs are used anchors may be designed using a preliminary bond stress of 400 pounds per square foot (psf), and if pressure/post-grouted tieback anchors are used, anchors may be designed using a preliminary bond stress of up to 2,500 pounds per square foot (psf). However, the tieback designer should make an independent evaluation in order to verify the preliminary bond stress is adequate for site conditions. Tieback bond stress should be verified by field testing. Tieback anchors should be designed, constructed, and tested in accordance with the requirements of the Post-Tensioning Institute (PTI).

For design purposes, tieback should obtain their load-carrying capacity from the soil behind a plane taken to be 3 horizontal feet from the bottom of the shoring facing and inclined at an angle of 60 degrees measured from the horizontal extending to the top of the excavation. Passive resistance of soldier piles (drilled or driven) may be assumed to be an equivalent fluid pressure of 350 pcf to a maximum value of 3,500 psf. The passive earth pressure may be increased by 100 percent for isolated piles. Piles with spacing greater than 3 times of pile diameter can be considered as isolated piles. In order to develop full lateral resistance, firm contact between the soldier pile and undisturbed soils must be assured. For vertical shoring capacity, an allowable skin friction of 500 psf may be used for the portion of pier below the proposed development excavation. End bearing should be neglected. Drilling of shafts for solider piles may require casing or drilling mud to prevent caving.

Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. However, it is recommended that the lagging be designed for the full design active fluid pressure but be limited to a maximum of 400 psf. Therefore, the design lagging pressure should consider a parabolic earth pressure distribution with an active equivalent fluid pressure of 40 pounds per square foot (psf) per foot of depth (or pcf) up to a maximum of 400 psf. The maximum span for lagging for this project should be 10 feet.

The components of the shoring system should be designed by a California licensed structural and/or civil engineer specializing in the design of shoring systems. Field pullout testing should be performed during construction to verify the estimated pullout resistance used in the design and/or post grout tubes should be used to ensure adequate design capacities are obtained. Ultimately, it is the specialty contractor’s responsibility to obtain the required pullout capacity, which may require design and/or field modifications. LGC should review the shoring plans prior to construction to verify that geotechnical recommendations are properly implemented into the project plans.

It is highly recommended that a program of documentation and monitoring be devised and put into practice before the onset of any groundwork. The contractor should establish survey points on the shoring, adjacent streets, and neighboring buildings within 100 feet of the excavation perimeter prior to any excavation. These survey points should be used to monitor the movement of the shoring and existing improvements during construction excavation.

The monitoring program should include, but not necessarily be limited to detailed documentation of the existing improvements, buildings and utilities around the excavation, with particular attention to any distress that is already present prior to the start of work. A licensed surveyor should be retained to establish monuments on the shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project structural (shoring) engineer and LGC for review and evaluation.

4.1.7 Fill Placement and Compaction

From a geotechnical perspective, the on-site soils are suitable for use as compacted fill, provided they are screened of rocks greater than 8-inches in maximum dimension, organic material, and construction debris. Areas prepared to receive structural fill and/or other surface improvements should be scarified to a minimum depth of 6-inches, brought to at least optimum-moisture content, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). 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 generally not exceeding 8-inches in loose thickness. Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant.

If possible, imported soils should contain no materials over 3- to 8-inches in maximum dimension and have a very low to low expansion potential.

4.1.8 Trench Backfill and Compaction

The on-site soils may generally be suitable as trench backfill provided, they are screened of rocks and other material over 3- to 6-inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 8-inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557).

If trenches are shallow and the use of conventional equipment may result in damage to the utilities; clean sand, having sand equivalent (SE) of 30 or greater, should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding and then tamping to ensure adequate compaction. A representative from LGC should observe, probe, and test the backfill to verify compliance with the project specifications.

4.2 Foundation Design

4.2.1 General Foundation Selection

Recommendations for preliminary foundation design and construction are presented herein. Based on the results of representative expansion potential laboratory testing of the representative on-site soils, the proposed structures should be designed for a low or medium expansion potential (i.e., a 20 to 90 Expansion Index). The following post-tension and mat slab foundation recommendations are provided.

The information and recommendations presented in this section are not meant to supersede design by the project structural engineer or civil engineer specializing in the structural design nor impede those recommendations by a corrosion consultant. Should conflict arise, modifications to the foundation design provided herein can be provided.

4.2.2 ***Bearing Capacity***

Shallow foundations may be designed for a maximum allowable bearing capacity of 2,000 lb/ft^2 (gross), for continuous footings a minimum of 18-inches wide and 12-inches deep; and spread footings 24-inches wide and 18-inches deep, into certified compacted fill or competent formational material. A factor of safety greater than 3 was used in evaluating the above bearing capacity value. This value may be increased by 300 psf for each additional foot in depth and 150 psf for each additional foot of width to a maximum value of 4,000 psf.

Lateral forces on footings may be resisted by passive earth resistance and friction at the bottom of the footing. Foundations may be designed for a coefficient of friction of 0.35, and a passive earth pressure of 250 lb/ft^2/ft. The passive earth pressure incorporates a factor-of-safety of greater than 1.5.

All footing excavations should be cut square and level as much as possible, and should be free of sloughed materials including sand, rocks and gravel, and trash debris. Subgrade soils should be pre-moistened for the assumed medium or high expansion potential (to be confirmed at the completion of grading). These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5:1[horizontal to vertical]) conditions only.

Bearing values indicated above are for total dead loads and frequently applied live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

4.2.3 ***Post-Tension Foundation***

Based on our review, the site may be considered suitable for the support of the proposed structure using a post-tensioned slab-on-grade foundation system for the anticipated low to medium expansion potential. The following section summaries our recommendations for the foundation system. Table 3 contains the geotechnical recommendations for the construction of a PT slab-on-grade foundation. The structural engineer should design the foundation system based on these parameters including the foundation settlement as indicated in the following section to the allowable deflection criteria determined by the structural engineer/architect.

As indicated above, the underslab vapor/moisture retarder (i.e., an equivalent capillary break method) may consist of a minimum 15-mil vapor barrier in conformance with ASTM E 1745 Class A material, placed in general conformance with ASTM E1643, underlain by a minimum 1-inch of sand, as needed. The sand layer requirements above the vapor barrier are the purview of the foundation engineer/structural engineer and should be provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction". These recommendations must be confirmed (and/or altered) by the foundation engineer, based upon the performance expectations of the foundation. Ultimately, the design of the moisture retarder system and recommendations for concrete placement and concrete mix design, which will address bleeding, shrinkage, and curling are the purview of the foundation engineer, in consideration of the project requirements provided by the architect and developer. The underslab vapor/moisture retarder described above is considered a suitable alternative in accordance with the Capillary Break Section 4.505.2.1 of the CAL Green code.

Table 3

Preliminary Geotechnical Parameters for Post-Tensioned Foundation Design

Parameter	Value	
Expansion Classification (Assumed to be confirmed at the completion of grading):	Low and Medium Expansion	
Thornthwaite Moisture Index (from Figure 3.3):	-20	
Constant Soil Suction (from Figure 3.4):	PF 3.6	
Center Lift Edge moisture variation distance (from Figure 3.6), e_m: Center lift, y_m:	**Low** 9.0 feet 0.35 inches	**Medium** 9.0 feet 0.5 inches
Edge Lift Edge moisture variation distance (from Figure 3.6), e_m: Edge lift, y_m:	**Low** 5.2 feet 0.65 inches	**Medium** 5.0 feet 1.1 inches
Expansion Potential:	**Low (21-50)**	**Medium (51-90)**
Soluble Sulfate Content for Design of Concrete Mix in Contact with Site Soils in Accordance with American Concrete Institute Standard 318, Section 4.3:	Negligible Exposure (Exposure Class S0)	
Corrosivity of Earth Materials to Ferrous Metals:	Corrosive	
Modulus of Subgrade Reaction, k (assuming presaturation as indicated below):	100 pci (low) 85 pci (medium to high)	

Additional Recommendations:

1. Presaturate slab subgrade to at least 1.2 times optimum moisture to minimum depth of 12 for low expansion potential and at least 1.2 times optimum moisture to a minimum depth of 18 inches below ground surface for medium expansion potential.
2. Install a 15-mil moisture/vapor barrier in direct contact with the concrete (unless superseded by the Structural/Post-tension Engineer*) with minimum 1 inches of sand below the moisture/vapor barrier.
3. Minimum perimeter foundation embedment below finish grade for moisture cut off should be 12 and 18 inches, respectively for low and medium expansion potentials, respectively.
4. Minimum slab thickness should be 5 inches.

* The above sand and moisture/vapor barrier recommendations are traditionally included with geotechnical foundation recommendations although they are generally not a major factor influencing the geotechnical performance of the foundation. The sand and moisture/vapor barrier requirements are the purview of the foundation engineer/corrosion engineer (in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction") and the homebuilder to ensure that the concrete cures more evenly than it would otherwise, is protected from corrosive environments, and moisture penetration of through the floor is acceptable to future homeowners. Therefore, the recommendations provided herein may be superseded by the requirements of the previously mentioned parties.

4.2.4 Mat Foundations

A mat foundation can be used for support of proposed residential buildings. An

allowable soil bearing pressure of 1,500 psf may be used for the design of the mat at the surface under the slab area.
for the design of the mat at the surface under the slab area.

The allowable bearing value is for total dead loads and frequently applied live loads and may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces. A coefficient of vertical subgrade reaction, k, of 85 pounds per cubic inch (pci) may be used to evaluate the pressure distribution beneath the mat foundation.

The magnitude of total and differential settlements of the mat foundation will be a function of the structural design and stiffness of the mat. Based on assumed structural loads, we estimate that total static settlement will be on the order of an inch at the center of the mat foundation. Post construction differential settlement can be taken as one-half of the maximum estimated settlement.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. Foundations may be designed for a coefficient of friction of 0.35.

Coordination with the structural engineer will be required in order to ensure structural loads are adequately distributed throughout the mat foundation to avoid localized stress concentrations resulting in potential settlement. The foundation plan should be reviewed by LGC to confirm preliminary estimated total and differential static settlements.

4.2.5 Foundation Settlement

Based on our current understanding of the project, the results of our site investigation and the recommended remedial grading with shallow foundations embedded into compacted fills or competent bedrock, we estimate the post-construction static settlement of the site to be less than 1-inch with a differential settlement of approximately of 0.5-inches in 30 feet.

For buildings located above the proposed segmental wall, with a portion of the building underlain by the geogrid reinforced soils, the proposed foundation should be designed for a minimum of a medium expansion potential and for a differential settlement of up to 2-inches in 30 feet. Recommendations to reduce the potential for differential settlements in the vicinity of the segmental walls are provided herein in Section 4.4.

4.2.6 Building Clearance and Foundation Setbacks

All building foundations located close to slopes should have a minimum setback per Figure 1808.7.1 of the 2022 CBC. The setback distances should be measured from competent materials on the outer slope face, excluding any weathered and loose materials.

Per the 2022 CBC Section 1808.7.1 and Figure 1808.7.1, building clearance from the toe of an ascending slope should be equal one-half of the total slope height to a maximum setback of 15 feet. Retaining walls may be constructed at the base of the slope to achieve the required building clearances.

Per the 2022 CBC Section 1808.7.2 and Figure 1808.7.1, the building foundation constructed on or near a descending slope should be setback or deepened to provide a minimum footing setback equal to the total height of slope (H) divided by 3 (H/3). The footing setback should be a minimum of 5 feet for slopes up to 15 feet in height and vary up to 40 feet for slopes up to 120 feet in height. The footing setbacks should be measured from the edge of the footing to the competent materials on the outer slope face.

4.3 *Lateral Earth Pressures for Retaining Walls*

The following lateral earth pressures may be used for the design of any future site retaining walls. Due to the expansive nature of some of the on-site clayey formational materials, we recommend site retaining walls be backfilled with either the very low expansive bedrock or sandy soils (with minus 3-inch rock) or approved select soils. Approved select soils should consist of clean, granular soils (less than 15 percent passing the No. 200 sieve) of very low expansion potential (expansion index 20 or less based on UBC. 18-2). The recommended lateral pressures for approved select soils for level or sloping backfill are presented in Table 4.

Table 4

Lateral Earth Pressures for Retaining Walls

Conditions	**Equivalent Fluid Weight (pcf)**		
	Level Backfill	**2:1 Backfill Sloping Upwards**	**Dynamic Load Increment (pcf) ***
	Approved Select Material	**Approved Select Material**	
Active	35	50	Level -10H^2 2H:1V - 17H^2
At Rest	55	80	16H^2
Passive	250	--	--

* For walls with greater than 6-feet in backfill height, the above seismic earth pressure should be added to the static pressures given in the table above. The seismic earth pressure should be considered as a triangular distribution with the resultant acting at 0.4H in relation to the base of the retaining wall footing (where H is the retained height). The incremental seismic load was determined in general accordance with the standard of practice in the industry for determining earth pressures as a result of seismic events.

For design purposes, the recommended equivalent fluid pressure for each case for walls founded above the static ground water and backfilled with approved select soils is provided in Table 4. The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer. Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated in Figure 3. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water, which contains soluble salts, migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft of depth or pcf. These values do not contain an appreciable factor of safety. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil. For sliding resistance, a friction coefficient of 0.35 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. Refer to Section 4.2.2 for passive resistance and allowable soil bearing.

4.4 Segmental Retaining Wall Recommendations

Segmental retaining walls that may be constructed on the site are anticipated to have a level or up to a 2:1 (horizontal to vertical) sloping backfill above the walls. The zone of influence for geogrid-reinforced modular block walls is defined by a 1:1 (horizontal to vertical) projection from the heel of the bottom geogrid to the finished ground surface overlying the wall. Any building or vehicle loads within this zone should be considered in the wall design.

The following geotechnical parameters presented in Table 5 may be utilized by the wall engineer in design of the on-site segmental walls. Design of segmental retaining walls should be per the National Concrete Masonry Association (NCMA) guidelines (or equivalent guidelines).

Table 5 Design Soil Strength Parameters for Segmental Retaining Walls			
	Cohesion (psf)	**Friction Angle Peak/Ultimate (Degrees)**	**Unit Weight (pcf)**
Infill (Reinforced) Soil	50	35/30	120
Retained (Backfill) Soil	100	35/30	120
Foundation Soil	100	35/30	120

The design acceleration of 0.411g (or 2/3 PGAm), should be used for the proposed design. A Pseudo-Static Coefficient of 0.28g should be used for Slope Stability Analysis. Once the wall designer designs the wall considering external, internal, and local wall stability, LGC will then check the global slope stability. Where global slope stability is the controlling factor, additional geogrid will be added to the design and/or the geogrid will be lengthened, as needed. Thus, the final design is expected to satisfy both the "conventional method" of modular wall design as well as global slope stability.

All excavations should be made in accordance with Cal OSHA, as a general guideline. The backfill soils (having an expansion index less than 30 per UBC. 18-I-B) should be compacted to at least 90 percent relative compaction (based on ASTM Test Methods D2922 and D3017). The walls should be constructed and backfilled as soon as possible after back-cut excavation. Prolonged exposure of back-cut slopes may result in some localized slope instability. Excavation safety is the sole responsibility of the contractor.

The subject walls may be backfilled using the on-site native soils. For closed face walls we recommend a minimum 1-foot-wide drainage gallery be constructed immediately behind the face of the wall using Class II Permeable material and augmented with a perforated 4-inch PVC pipe, or per the wall manufactures specifications. This drainage layer and drain is not a requirement for open faced walls. The remainder of the wall may be backfilled using the on-site native soils. The subject segmental retaining walls should be constructed founded onto competent soils (i.e., compacted fills or competent native soils), or per manufactures specifications. For preliminary purposes, the allowable bearing capacities to be used in the wall design is 1,500 pounds per square foot.

From a geotechnical perspective, the on-site soils are generally suitable for use as compacted fill, provided they are screened of rocks greater than 8 inches in maximum dimension, organic materials, and construction debris. Fill soils should be brought to at least optimum-moisture content, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). 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 generally not exceeding 8 inches in compacted thickness. Placement and compaction of fill should be performed in accordance with local grading ordinances under full-time observation and testing of the geotechnical consultant. The geotechnical consultant shall review and approve all fill materials, including on-site and import materials.

Prior to placement of the geogrid, the surface of the compacted fill shall be prepared such that it has a maximum variation of 6 vertical inches over a distance of 15 feet. Each geogrid layer shall be pulled taut and secured in-place prior to placing backfill material on the geogrid. The geogrid layers shall be continuous and no splice and/or connection system will be accepted. The contractor shall not operate tracked construction equipment directly upon the geogrid reinforcement but shall use rubber tired equipment. All passes with tracked equipment for the purposes of obtaining compaction shall be done in straight lines and shall minimize the turning movements of the equipment to reduce the potential for displacing and/or damaging the geo-grids.

The manufacturer shall provide the owner with quality control testing for each lot of blocks which are shipped to the site. The contractor shall install the block per the manufacturer's recommended procedures.

All excavations should be made in accordance with Cal OSHA, as a general guideline. All excavations should be made at 1:1 inclinations or flatter. Once excavation has been initiated, the segmental retaining wall should be constructed as soon as possible after back-cut excavation. Prolonged exposure of back-cut slopes may result in some localized slope instability. Excavations should be planned so that they are not initiated without sufficient time to backfill them prior to weekends, holidays, or forecasted rain. ***Excavation safety is the sole responsibility of the contractor.***

We recommend the contractors proposed plan of operations be reviewed by this office prior to initiation of work and closely monitored by representatives of this office during excavation and construction.

A backdrain should be installed at the heel of the wall back-cut consisting of a 4-inch PVC pipe surrounded by ¾-inch crushed rock and wrapped in a filter fabric and outletted through the wall face or to another suitable outlet. If water seepage is encountered along the wall back-cut, a continuous chimney drain consisting of a one-foot layer Caltrans Class II permeable material shall be placed at the heel along the back-cut behind the geogrid, as necessary. The chimney drains should be outletted through the backdrain at the heel of the cut. The outlet pipes should be constructed at the low points of the subdrains and have a minimum 2 percent fall to the outlet location. Additional subdrains may be needed if seepage and/or areas of potential seepage are encountered during grading operations.

Positive drainage of surface water away from the base and top of the proposed segmental retaining walls are important. A concrete V-ditch shall be constructed behind the top of each of the proposed walls to prevent surface water from infiltrating the backfill soil. The V-ditch shall be designed and placed by the project civil engineer in accordance with the local codes.

Where proposed residential structures are located above the proposed segmental walls, where the proposed reinforced zones will encroach into the proposed buildings, the following recommendations should be considered in the design and site construction. The fill behind the wall within the reinforced zone should be comprised of on-site sandy granular soils with a low expansion potential. Sandy/granular soils will interlock with the geogrid layers which is expected to reduce the potential for long-term, creep-type lateral deformation of the segmental wall, beyond the initial elastic deformation. As a result, the long-term differential settlement potential behind/above the segmental wall is expected to be reduced. Further, the backcut for wall construction should be excavated at a gradient no steeper than 2:1 (horizontal to vertical). This will help reduce the differential fill thickness beneath the proposed structures. The fills outside the reinforced zone should also consist of soils with relatively low expansion potentials and should be compacted to a minimum 93 percent relative compaction.

4.5 ***<u>Soldier Pile Wall Recommendations</u>***

The following preliminary geotechnical parameters may be utilized by the soldier pile wall consultant for design of the permanent wall system. The recommendations provided herein with regard to the proposed wall design are based on assumed conditions, extrapolated from the data gathered from our site investigation and geotechnical analysis. The wall designer should independently evaluate the parameters provided and conduct an additional investigation if they consider necessary.
based on assumed conditions, extrapolated from the data gathered from our site investigation and geotechnical analysis. The wall designer should independently evaluate the parameters provided and conduct an additional investigation if they consider necessary.

Prior to construction, the contractor should verify underground clearance of any existing utility lines or structures that must be removed or protected in place during construction or may conflict with any proposed foundation system.

Typical cantilever soldier pile wall design, where deflection of the wall will not impact the performance of adjacent structures or streets, may be designed using the active equivalent fluid pressures of 40 pounds per square foot (psf) per foot of depth (or pcf). Restrained walls (with soil nails or tied-back) is recommended to limit defections or required due to the proposed wall height. Restrained or tied-back shoring with a level backfill may be designed using an active trapezoidal soil pressure of 38H in pounds per square foot (psf), where H is equal to the depth in feet of the wall (shape of the trapezoid should be 0.2H, 0.6H, 0.2H) or may be designed using an active triangular soil pressure of 60 pounds per square foot (psf). Any building, equipment, or traffic loads located within a 1:1 (horizontal to vertical) projection from the base of the wall should be added to the applicable lateral earth pressure. A minimum additional uniform lateral pressure of 100 psf for the upper 10 feet should be added to the appropriate lateral earth pressures to account for typical vehicle traffic loading.
the appropriate lateral earth pressures to account for typical vehicle traffic loading.
he appropriate lateral earth pressures to account for typical vehicle traffic loading.

A seismic earth pressure of 17 pcf should be added to the static pressures given in Table 4 above. The seismic earth pressure should be considered as an inverted triangular distribution with the resultant acting at 0.6H in relation to the base of the retaining wall footing (where H is the retained height).

Passive resistance of soldier piles may be assumed to be an equivalent fluid pressure of 350 pcf for level and 150 pcf for sloping down conditions to a maximum value of 3,500 psf. The passive earth pressure may be increased by 100 percent for isolated piles. Piles with spacing greater than 3 times of pile diameter can be considered as isolated piles. In order to develop the full lateral resistance, firm contact between the soldier pile and undisturbed soils must be assured. For vertical capacity, an allowable skin friction of 500 psf may be used for the embedded depth. End bearing should be neglected.

4.6 Preliminary Pavement Recommendations

Preliminary pavement sections utilizing asphalt concrete (AC), Portland Cement Concrete Pavement PCCP), and vehicular concrete pavers, are presented in the following sections.

4.6.1 Asphalt Concrete (AC) Pavement

Based on an assumed R-value of 7 (obtained during the preliminary investigation of an adjacent property), we utilized an assumed R-Value of 7 and recommend the following preliminary minimum street sections for Traffic Indices of 5, 6, and 7 (as indicated in Table 6). These recommendations should be confirmed with R-value testing of representative near-surface soils at the completion of grading. Final AC sections should be confirmed by the project civil engineer based upon the projected Traffic Index. In addition, additional sections can be provided based on other traffic indices.

Table 6

Preliminary AC Pavement Design Sections

Assumed Traffic Index	5	6	7
R-Value Subgrade	7	7	7
AC Thickness	4.0 inches	4.0 inches	4.0 inches
Aggregate Base Thickness	7.0 inches	12.0 inches	15.0 inches

The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction –SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 95 percent. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Aggregate base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

4.6.2 Portland Cement Concrete Pavement

Portland Cement Concrete Pavement (PCCP) may be designed using a minimum of 8-inches of Portland Cement Concrete over 8-inches of compacted aggregate base.

The modulus of rupture of the PCCP should be a minimum of 500 pounds per square inch (psi) at 28 days. Contraction joints should be placed at maximum 15-foot spacing. Where the outer edge of a concrete pavement connects to an asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. In addition, additional sections can be provided based on other desired anticipated traffic loadings.

The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction - SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 95 percent. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

4.6.3 Vehicular Concrete Paver Pavement

Typical vehicular pavers are 3-1/8 inches in thickness and the manufacturers usually recommend that the pavers be underlain by a 1-inch-thick sand layer. Based on ASCE 58-10 for interlocking pavers and considering a Traffic Index (TI) of 6 and an R-Value of 7 for the subgrade soils, we recommend that the vehicular pavers and sand layer be underlain by a minimum of 16-inches of aggregate base.

The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction –SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 95 percent per ASTM- D1557. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

4.7 Nonstructural Concrete Flatwork

Preliminary nonstructural concrete flatwork designs are presented in the following sections. Portland Cement Concrete (PCC) flatwork (such as sidewalks, walkways, patios, entryways, etc.) have a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations because these slabs are typically much thinner than foundation slabs and are not reinforced with the same dynamics as foundation elements. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 7. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will <u>not</u> eliminate all cracking or lifting. Thickening the concrete and/or adding reinforcement will further reduce cosmetic distress.

Table 7 Nonstructural Concrete Flatwork		
	Private Sidewalks	**Patio/Entryways**
Minimum Thickness (in inches)	4	5
Presaturation	Presoak to 12-inches prior to placement	Presoak to 12-inches prior to placement
Reinforcement	--	No. 3 at 24 inches on centers or 6x6 No. 6 by No. 6 Welded Wire Mesh
Crack Control	Saw cut or deep tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep tool joint to a minimum of 1/3 the concrete thickness
Maximum Joint Spacing	5 feet	6 feet
Aggregate Base	--	2 inches

4.8 Corrosivity to Concrete and Metal

The National Association of Corrosion Engineers (NACE) defines corrosion as "a deterioration of a substance or its properties because of a reaction with its environment." From a geotechnical viewpoint, the "environment" is the prevailing foundation soils and the "substances" are the reinforced concrete foundations or various buried metallic elements such as rebar, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. ACI 318R-14 Table 19.3.1.1, provides specific guidelines for the concrete mix design when the soluble sulfate content of the soils exceeds 0.1 percent by weight or 1,000 ppm. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532.

Laboratory testing of representative on-site soils indicated that the on-site soils tested are classified as having a Sulfate Exposure Class of S0 (per Table 19.3.1.2 of the ACI 318-19). As a preliminary recommendation, concrete in contact with on-site soils should be designed in accordance with ACI 318-19 Table 19.3.2.1 for the S0/negligible or not applicable category. It is also our opinion that on-site soil should be considered moderately corrosive to buried metals. Site grading will redistribute the materials, which may result in soils with different corrosion potentials. Therefore, the as-graded soil conditions should be verified with confirmatory sampling and testing during the grading phase of the project.

Despite the minimum recommendation above, LGC is not a corrosion-engineering firm. Therefore, we recommend that after site grading, consultation with a competent corrosion engineer be initiated to evaluate the actual corrosion potential of the site and to provide recommendations to reduce the corrosion potential with respect to the proposed improvements, as necessary. The recommendations of the corrosion engineer may supersede the above requirements.

4.9 Freestanding Walls

Freestanding wall footings should be founded a minimum of 24-inches below the lowest adjacent grade. To reduce the potential for unsightly cracks, we recommend inclusion of construction joints at 10- to 20-foot intervals.

Due to the potential creep of soils, where free standing walls are constructed close to a top-of-slope, some tilt of the wall should be anticipated. To reduce the amount of tilt, a combination of grade beam and caisson foundations may be used to support the wall. The system should consist of a minimum of 12-inch diameter caissons placed at 8 feet maximum on centers, and each 8 feet long and connected together at top with 12-inch by 12-inch grade beam.

4.10 Control of Surface Water and Drainage Control

Positive drainage of surface water away from structures is very important. No water should be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from the building at a gradient of at least 2-percent for a distance of at least 5 feet, and further maintained by a swale or drainage path at a gradient of at least 1-percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes.

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.

4.11 Construction Observation and Testing

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 observation and testing should be performed by the geotechnical consultant during site excavations, subgrade for slab/foundation, backfill of utility trenches, preparation of any subgrade and placement of aggregate base, or when any unusual soil conditions are encountered at the site. Grading plans, foundation plans, and final project drawings should be reviewed by this office prior to construction.

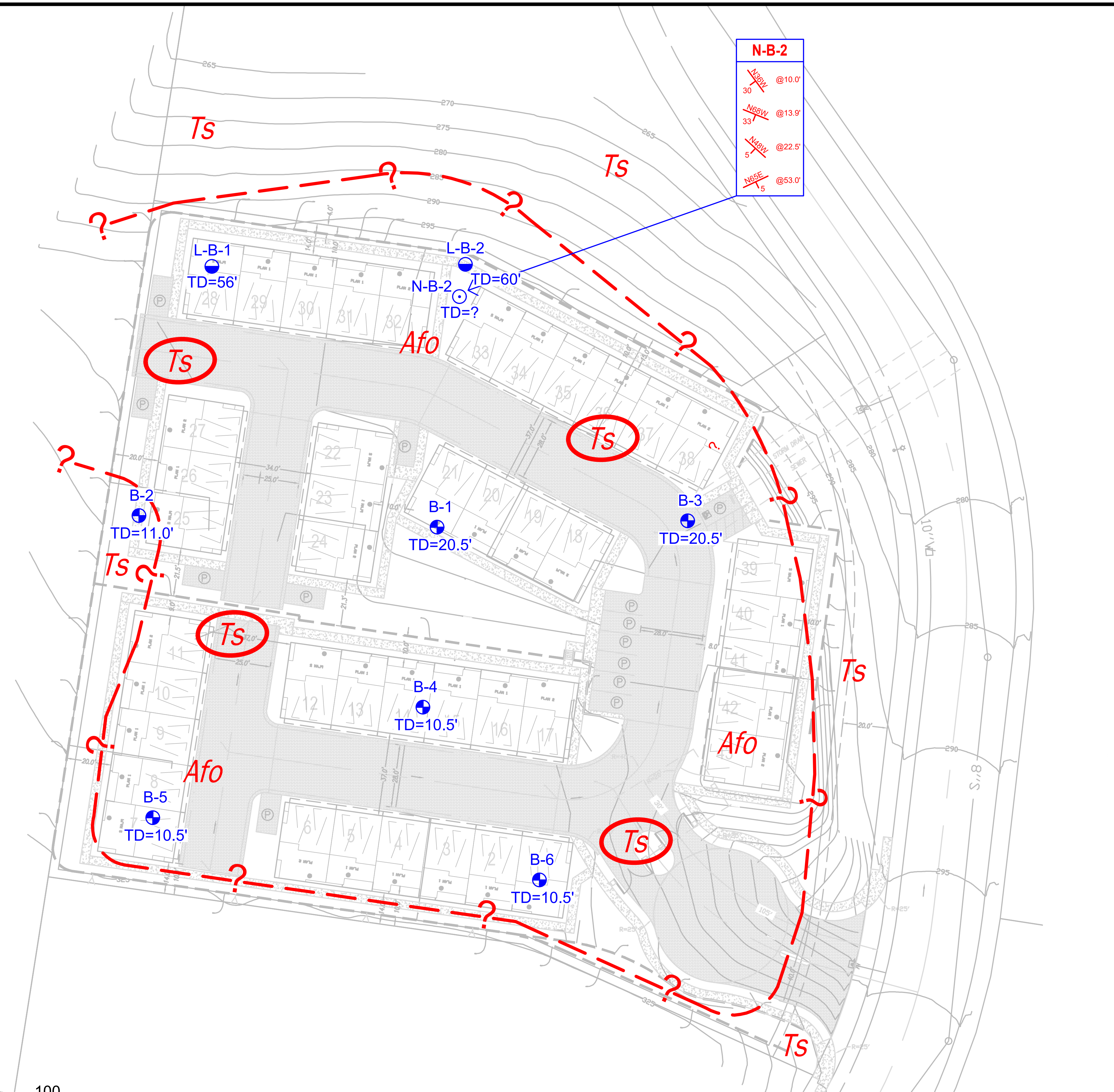
5.0 LIMITATIONS

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

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

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties.

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.



Legend

Afo	Artificial Older Fill
Ts	Tertiary-aged Sespe Formation, circled where buried
	Approximate limits of fill, queried where uncertain
B-6 TD=10.5'	Approximate Boring Location by LGC Valley (2024)
L-B-2 TD=60'	Approximate Boring Location by Lawmaster & Co. (1986)
N-B-2 TD=?	Approximate Boring Location by G. A. Nichol (1986)
N68W 33	Approximate Bedding Attitude

Geotechnical Map

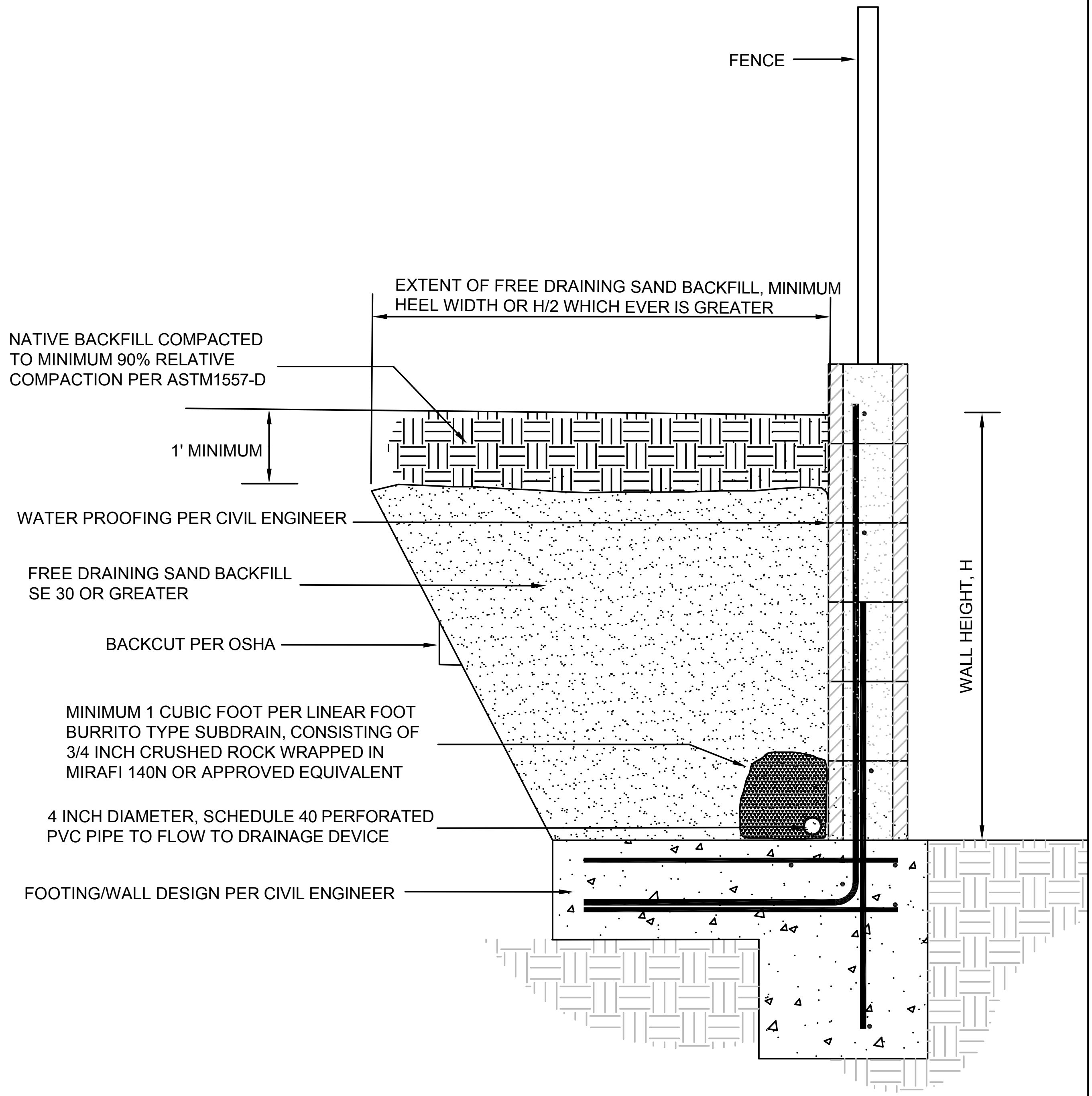
23161 Mill Creek Drive
Laguna Hills, California

LGC

Figure 2

Project Name	Toll/23161 Mill Creek
Project No.	244006-01
Eng. / Geol.	BIH/RKW
Scale	1" = 50'
Date	August 15, 2024

Base Map: 2024-07-30, Site Plan Laguna Hills Mill Creek 2.4 Acre Site, Sheet 1 of 1, by Wilson Mikami Corporation.



LGC

Figure 3: Retaining Wall Detail, Sand Backfill

Project Name	Toll/Laguna Hills
Project No.	244006-01
Eng. / Geol.	BIH/RKW
Scale	Not-To-Scale
Date	August 15, 2024

APPENDIX A

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APPENDIX B

Geotechnical Boring Logs (Current Study)

Geotechnical Boring Log B-1

Date: May 8, 2024	**Page:** 1 of 1
Project Name: Toll/23161 Mill Creek	**Project Number:** 244006-01
Drilling Company: Martini Drilling	**Type of Rig:** Hollow Stem Auger
Drive Weight: 140 pounds	**Drop:** 30 inches **Hole Dia:** 8 inches
Elevation of Top of Hole: +298 Feet	**Hole Location:** See Geotechnical Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: MET Sampled By: MET	Type of Test
298	0							@ 0' 4.5-inches of asphalt concrete over 8-inches of aggregate base	
			B1 @1'-5'				SM	**Artificial Fill, Older (Afo):** @ 1' Silty to clayey fine to medium SAND, very minor gravel; red brown to dark brown, damp, medium dense	COR, AL, EI
293	5		1	50/6"	110.9	6.9	CL/SM	**Sespe Formation (Ts):** @ 5' Silty fine to medium sandy CLAYSTONE to silty fine to coarse SANDSTONE, very minor gravel; dark brown to red brown, damp, hard to very dense	
			2	50/6"	110.5	8.2	SM	@ 6' Silty fine to medium SANDSTONE, very minor gravel; red brown, damp, very dense	
288	10		3	50/2"	114.2	2.0		@ 10' Silty fine to medium SANDSTONE, minor gravel; pale red brown, slightly damp, very dense; sample disturbed - rock in sampler	
			B2 @10'-15'						
283	15		4	50/6"				@ 15' Silty fine to medium SANDSTONE, minor gravel; red brown, damp, very dense	DS
278	20		5	50/4"					
								Total Depth = 20.5 Feet **No Ground Water Encountered** **Backfilled 5/8/2024** **with Native Soil and Concrete Plug**	

LGC

= Ring Sample

= SPT

LGC VALLEY, 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 CONDITIOND ENCOUNTERED.

Geotechnical Boring Log B-2

Date: May 8, 2024	**Page:** 1 of 1
Project Name: Toll/23161 Mill Creek	**Project Number:** 244006-01
Drilling Company: Martini Drilling	**Type of Rig:** Hollow Stem Auger
Drive Weight: 140 pounds	**Drop:** 30 inches **Hole Dia:** 8 inches
Elevation of Top of Hole: +306 Feet	**Hole Location:** See Geotechnical Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: MET Sampled By: MET	Type of Test
306	0						SM	@ 0' 5.5-inches of asphalt concrete over 6-inches of aggregate base	
			1 B1 @1'-6'	50/6"	111.3	14.0		**Sespe Formation (Ts):** @ 1' Silty fine to medium SANDSTONE, very minor gravel; red brown, damp, very dense	
301	5		2	86/8.5"	111.7	12.4		@ 5' Becomes a very silty fine to medium SANDSTONE	
296	10		3	88/9"	116.0	12.2	SM/ML	@ 10' Silty fine SANDSTONE to fine sandy SILTSTONE; red brown to dark red brown, damp, very dense to hard	

Total Depth = 11 Feet
No Ground Water Encountered
Backfilled 5/8/2024
with Native Soil and Concrete Plug

LGC

■ = Ring Sample
☒ = SPT

LGC VALLEY, 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 CONDITIOND ENCOUNTERED.

Geotechnical Boring Log B-3

Date: May 8, 2024	**Page:** 1 of 1
Project Name: Toll/23161 Mill Creek	**Project Number:** 244006-01
Drilling Company: Martini Drilling	**Type of Rig:** Hollow Stem Auger
Drive Weight: 140 pounds	**Drop:** 30 inches **Hole Dia:** 8 inches
Elevation of Top of Hole: +297 Feet	**Hole Location:** See Geotechnical Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: MET Sampled By: MET	Type of Test
297	0							@ 0' 5.5-inches of asphalt concrete over 8-inches of aggregate base	
							SM	**Artificial Fill, Older (Afo):**	
			1 B1 @1'-6'	69	119.3	5.7	SM	@ 1.1' Silty fine to coarse SAND, minor gravel and minor clay; red brown, damp, medium dense	
292	5		2	82	119.6	4.2		**Sespe Formation (Ts):** @ 2.5' Silty fine to coarse SANDSTONE, very minor gravel and minor clay; red brown, damp, dense	
			3	50/6"	119.1	7.5		@ 5' Silty fine to coarse SANDSTONE, very minor gravel; red brown, damp, dense	
287	10		4	50/6"	110.4	5.7		@ 10' Increase in coarse sand	
282	15		5	50/5"					
277	20		6	50/5"				@ 20' Decrease in coarse sand	
								Total Depth = 20.5 Feet **No Ground Water Encountered** **Backfilled 5/8/2024** **with Native Soil & Concrete Plug**	

LGC

■ = Ring Sample

☒ = SPT

Geotechnical Boring Log B-4

Date: May 8, 2024	**Page:** 1 of 1
Project Name: Toll/23161 Mill Creek	**Project Number:** 244006-01
Drilling Company: Martini Drilling	**Type of Rig:** Hollow Stem Auger
Drive Weight: 140 pounds	**Drop:** 30 inches **Hole Dia:** 8 inches
Elevation of Top of Hole: +308 Feet	**Hole Location:** See Geotechnical Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: MET Sampled By: MET	Type of Test
308	0							@ 0' 3-inches of asphalt concrete over 5-inches of aggregate base	EI, RDS, COR
			B1 @1'-3'				SC/SM	**Artificial Fill, Older (Afo):** @ 0.8' Silty fine to medium SAND, very minor gravel; dark red brown, damp, medium dense @ 1.5' Clayey fine to coarse SAND, very minor gravel; dark brown to red brown, damp, medium dense	CN
			1	50/6"	113.4	12.1			
303	5		2	86/9"	113.9	8.8	SM	**Sespe Formation (Ts):** @ 3' Silty fine to coarse SANDSTONE; pale red brown, damp, very dense	
298	10		3	50/6"	110.3	9.4		@ 10' Becomes a very silty fine to coarse SANDSTONE	

Total Depth = 10.5 Feet
No Ground Water Encountered
Backfilled 5/8/2024
with Native Soil and Concrete Plug

LGC

■ = Ring Sample
☒ = SPT

LGC VALLEY, 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 CONDITIOND ENCOUNTERED.

Geotechnical Boring Log B-5

Date: May 8, 2024	**Page:** 1 of 1
Project Name: Toll/23161 Mill Creek	**Project Number:** 244006-01
Drilling Company: Martini Drilling	**Type of Rig:** Hollow Stem Auger
Drive Weight: 140 pounds	**Drop:** 30 inches **Hole Dia:** 8 inches
Elevation of Top of Hole: +317 Feet	**Hole Location:** See Geotechnical Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: MET Sampled By: MET	Type of Test
317	0							@ 0' 8-inches of Asphalt over 5-inches of aggregate base	
			1	52	115.8	13.4	SM	**Artificial Fill, Older (Afo):** @ 1.1' Silty fine to medium SAND, very minor gravel, slightly clayey; red brown, damp, medium dense	CN
312	5		2	85/9"	115.6	9.6	SM	**Sespe Formation (Ts):** @3.5' Silty fine to medium SANDSTONE, very minor gravel; red brown, damp, very dense	
			3	50/6"	111.3	8.4		@7.5' Becomes a silty fine to medium SANDSTONE	
307	10		4	50/6"	105.2	6.5	SW	@10' Fine to coarse SANDSTONE; red brown to gray brown, damp, very dense	

Total Depth = 10.5 Feet
No Ground Water Encountered
Backfilled 5/8/2024
with Native Soil and Concrete Plug

LGC

■ = Ring Sample
☒ = SPT

LGC VALLEY, 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 CONDITIOND ENCOUNTERED.

Geotechnical Boring Log B-6

Date: May 8, 2024	**Page:** 1 of 1
Project Name: Toll/23161 Mill Creek	**Project Number:** 244006-01
Drilling Company: Martini Drilling	**Type of Rig:** Hollow Stem Auger
Drive Weight: 140 pounds	**Drop:** 30 inches **Hole Dia:** 8 inches
Elevation of Top of Hole: +312 Feet	**Hole Location:** See Geotechnical Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION Logged By: MET Sampled By: MET	Type of Test
312	0						SW	@ 0' 8-inches of asphalt concrete over 5-inches of aggregate base	
			1 B1 @2.5'-5'	86/11"	118.1	6.0	SW	**Artificial Fill, Older (Afo):** @ 1.1' Silty fine to coarse gravelly SAND, minor fine cobbles; dark red brown, damp, medium dense	SA, SE
307	5		2	80/8.5"	111.9	9.9	SM	**Sespe Formation (Ts):** @ 2.5' Silty fine to coarse gravelly SANDSTONE, minor fine cobbles; red brown, damp, very dense	
								@ 5' Silty fine to coarse SANDSTONE, very minor gravel; red brown, damp, very dense	
302	10		3	50/6"	105.1	9.8			

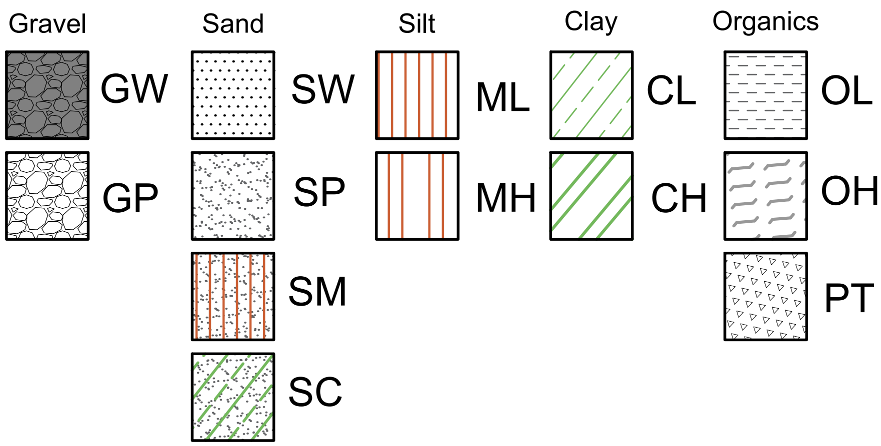
Total Depth = 10.5 Feet
No Ground Water Encountered
Backfilled 5/8/2024
with Native Soil and Concrete Plug

LGC

■ = Ring Sample
☒ = SPT

LGC VALLEY, 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 CONDITIOND ENCOUNTERED.

Key to Boring Logs



Symbol	**Laboratory Test**
SA	Sieve Analysis
H	Hydrometer Anlysis
SHA	Sieve & Hydrometer Analysis
-200	Percent Passing #200 Sieve
AL	Atterburge Limits
MAX	Maximum Density
DS	Undisturbed Direct Shear
RDS	Remolded Direct Shear
SE	Sand Equivalent
EI	Expansion Index
P	Permeability
CN	Consolidation
COL	Collapse
UC	Unconfined Compression
S	Sulfate Content
pHR	pH & Resistivity
COR	Corrosion Suite (pH, Resistivity, Chloride, Sulfate)
RV	R-Value

APPENDIX C

Laboratory Testing Procedures and Test Results (Current Study)

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 the results are presented on the following pages. LGC has reviewed the laboratory test data, procedures, and results with respect to the subject site, concurs with, and accepts responsibility as geotechnical engineer of record for their work (laboratory testing).

Soil Classification: Soils were classified according to the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. This system relies on the Atterberg limits and grain size distribution of a soil. The soil classifications (or group symbol) are shown on the laboratory test data and boring logs.

Atterberg Limits: The liquid and plastic limits ("Atterberg limits") were determined in accordance with ASTM Test Method D4318 for engineering classification of fine-grained material and presented on the following table:

Sample Location	Sample Description	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
B-1 #B1 @ 1-5'	Silty to clayey fine to medium SAND	27	15	12

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

Sample Location	Sample Description	Chloride Content (ppm)	Potential Degree of Chloride Attack*
B-1 #B1 @ 1-5'	Silty to clayey fine to medium SAND	125	Negligible
B-4 #B1 @ 1-3'	Clayey fine to medium SAND	140	Negligible

* Extrapolation from California Test Method 532, Method for Estimating the Time to Corrosion of Reinforced Concrete Substructures and previous experience.

Consolidation: Consolidation tests were performed on selected, relatively undisturbed ring samples (per Modified ASTM Test Method D2435). Samples (2.42 inches in diameter and 1 inch in height) were placed in a consolidometer and increasing loads were applied. The samples were 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 curves are presented on the attached figures at the end of this appendix.

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

Sample Location	Sample Description	Peak/Ultimate Friction Angle (degrees)	Peak/Ultimate Apparent Cohesion (psf)
B-1 #4 @15'	Undisturbed	29/29	278/96
B-4 #B1 @ 1-3'	Clayey fine to medium SAND (Remolded)	30/30	227/198

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

Sample Location	Sample Description	Expansion Index	Expansion Potential
B-1 #B1 @ 1-5'	Silty to clayey fine to medium SAND	14	Very Low
B-4 #B1 @ 1-3'	Clayey fine to medium SAND	38	Low

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-4 #B1 @ 1-3'	Clayey fine to medium SAND	133.5	8.0

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

Grain Size Distribution/Sieve Analysis: 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. The portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D422 (CTM 202). The sieve analysis curve is presented on the attached figure at the end of this appendix. The percent passing the #200 sieve is presented on the following table:

Sample Location	Sample Description	Percent Passing #200 Sieve
B-6 #B1 @2.5-5'	Silty fine to coarse gravelly SAND	26.5

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. As a result of soil's resistivity decreases corrosivity increases. The results are presented in the table below:

Sample Location	Sample Description	pH	Minimum Resistivity (ohms-cm)	Potential Degree of Corrosivity*
B-1 #B1 @ 1-5'	Silty to clayey fine to medium SAND	7.82	1,700	Moderately Corrosive
B-4 #B1 @ 1-3'	Clayey fine to medium SAND	8.15	1,300	Moderately Corrosive

* NACE Corrosion Basics, 1984.

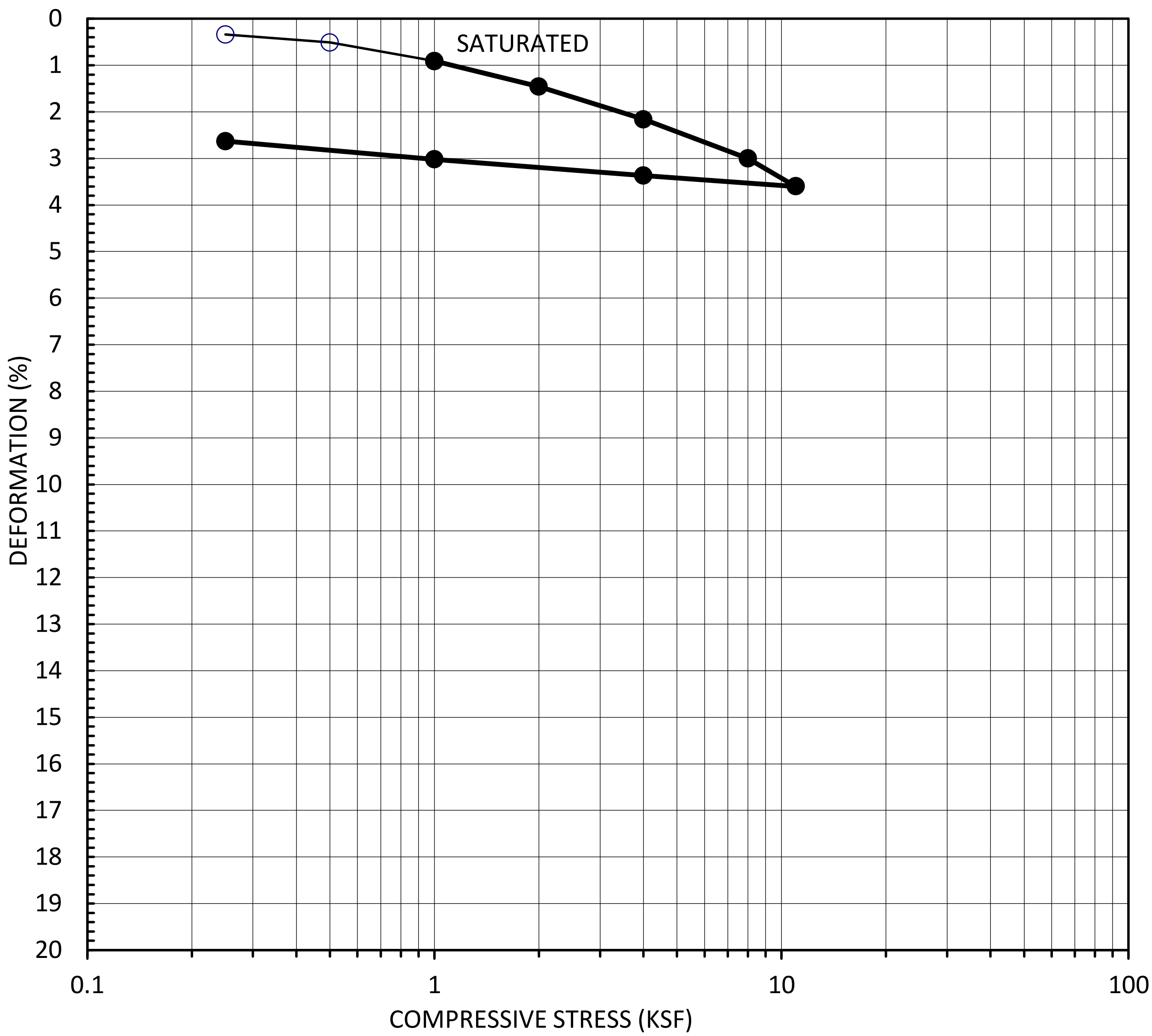
Sand Equivalent: The sand equivalent (SE) of selected samples was determined in accordance with ASTM D2419. The sand equivalent results are used to determine the applicability of material for use as backfill and to assess whether flooding or jetting is a suitable compaction method. The results are presented in the table below:

Sample Location	Sample Description	Sand Equivalent Value
B-6 #B1 @2.5-5'	Silty fine to coarse gravelly SAND	14

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below:

Sample Location	Sample Description	Sulfate Content (% by weight)	Exposure Class (Exposure Severity)*
B-1 #B1 @ 1-5'	Silty to clayey fine to medium SAND	0.007	S0 (Not Applicable)
B-4 #B1 @ 1-3'	Clayey fine to medium SAND	0.005	S0 (Not Applicable)

* Per ACI 318-19 Table 19.3.2.1 (ACI, 2019).



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-4	1	2.5	SM	12.1	115.1	0.463

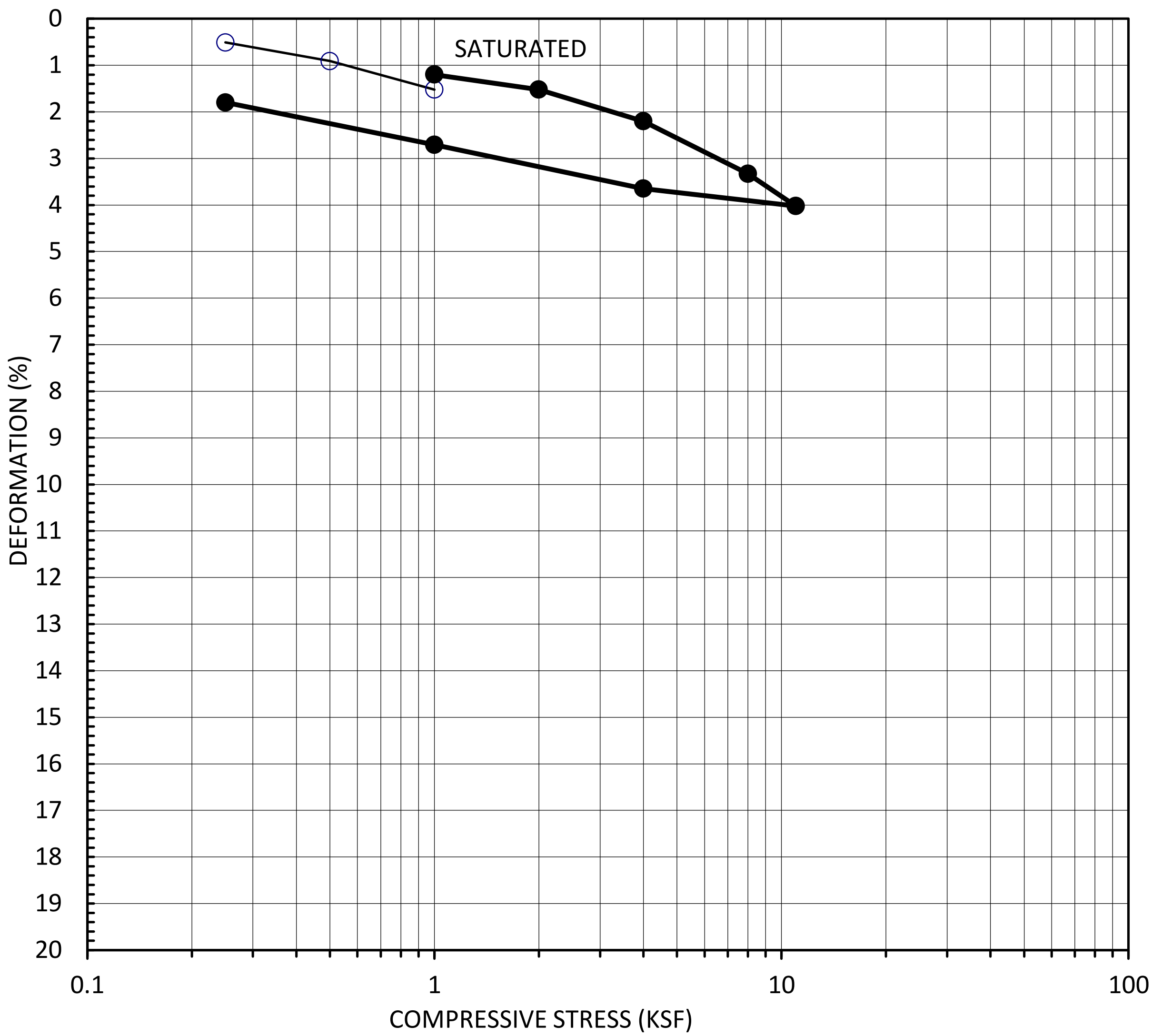
Project Name:
Toll / Mill Creek 2

EGLAB, INC.

Client:	LGC Valley, Inc.
Job No.:	244006-01
EGLAB Project No.:	24-059-009

CONSOLIDATION

08/24 (ASTM D2435) Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-5	1	2.5	CL/SC	13.4	118.4	0.423

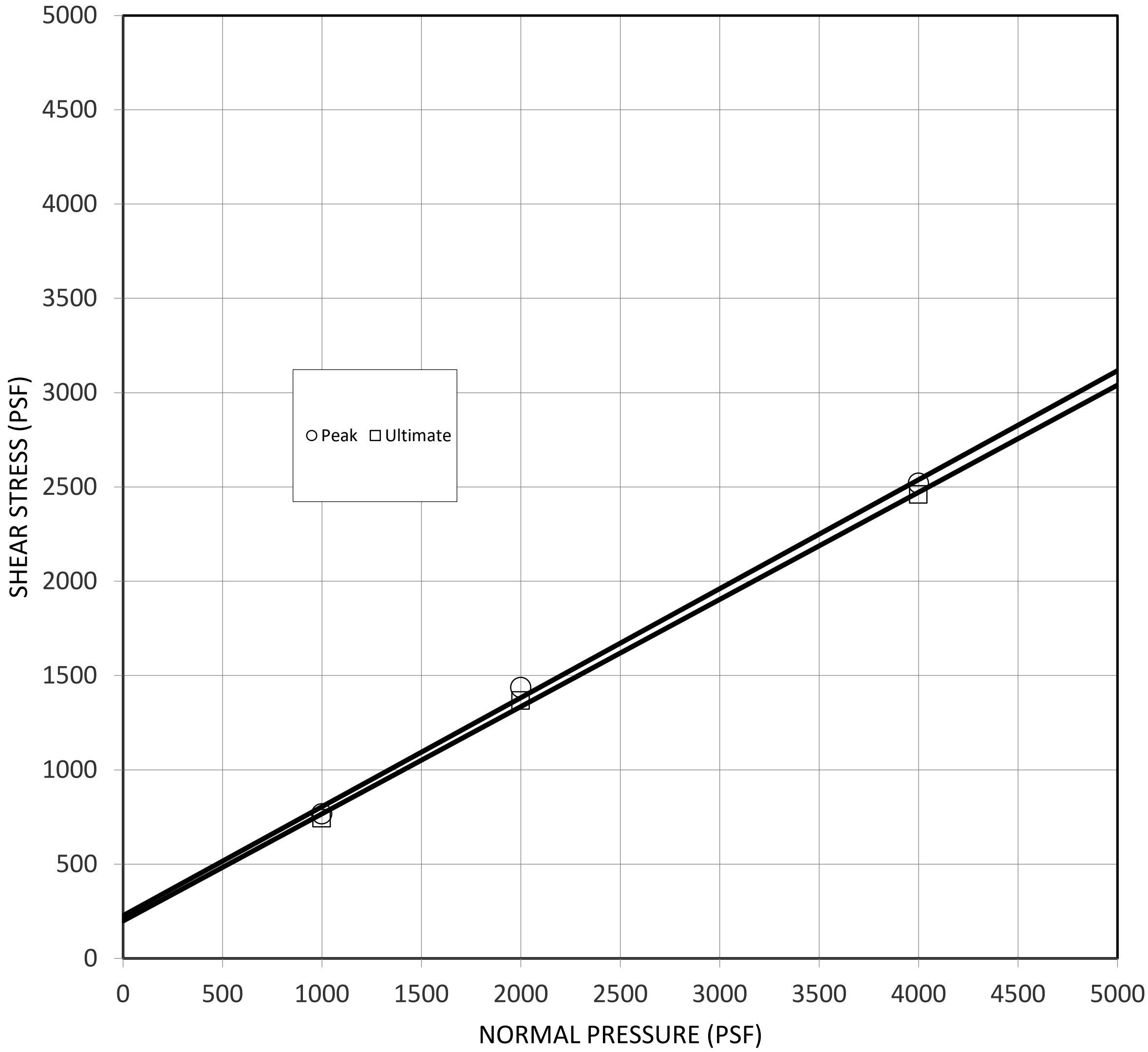
Project Name:
Toll / Mill Creek 2

EGLAB, INC.

Client:	LGC Valley, Inc.
Job No.:	244006-01
EGLAB Project No.:	24-059-009

CONSOLIDATION

08/24 (ASTM D2435) Figure



Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-1	4	15.0	Ring	SM	○	227	30
					□	198	30

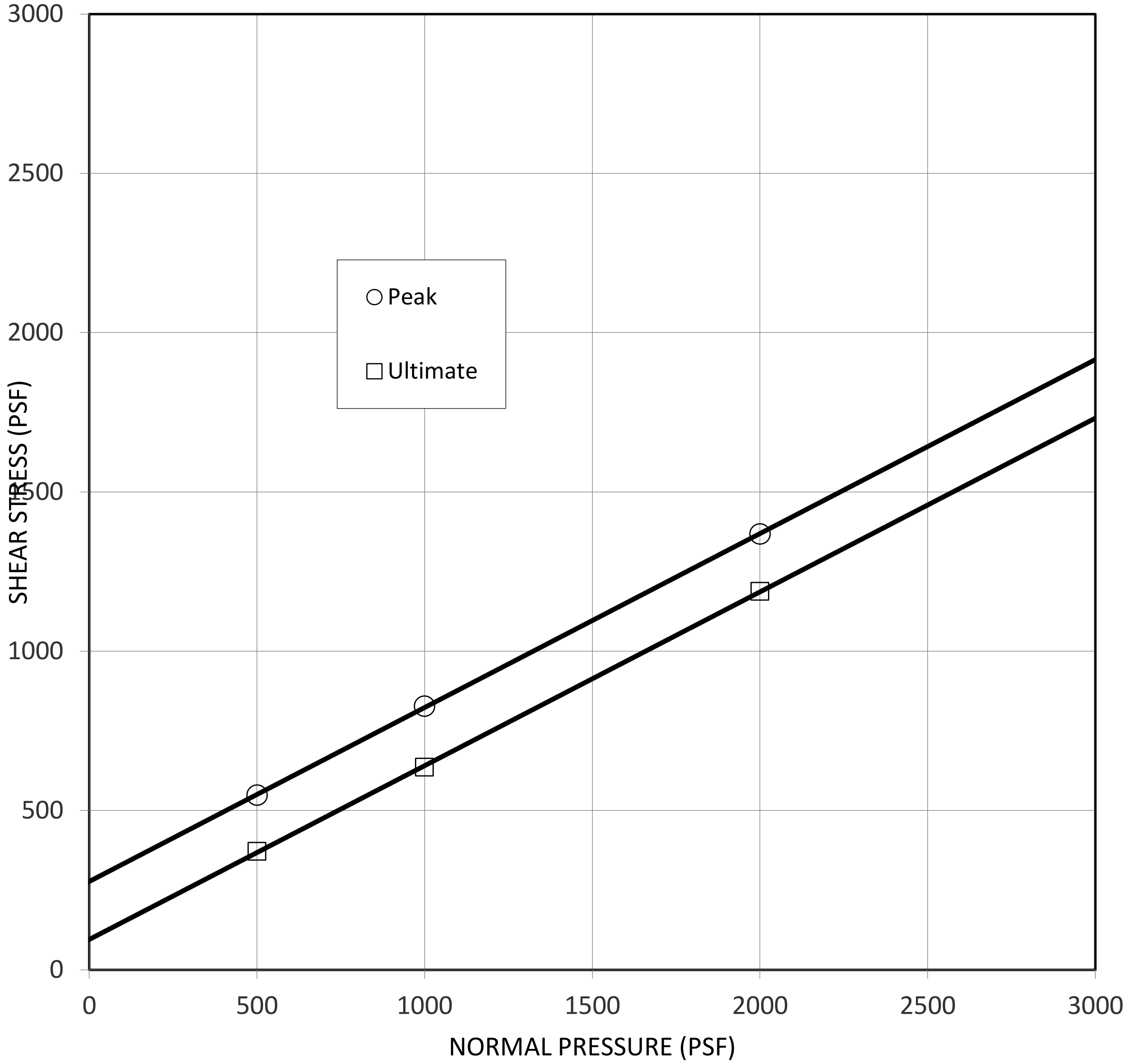
Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)	γ_d (pcf)	S (%)
1000	7.9	21.8	102.8	92
2000	7.9	21.3	103.5	92
4000	7.9	20.3	104.8	90

EGLAB, INC.

Project Name:
Toll / Mill Creek 2
Client: LGC Valley, Inc.
Project No.: 244006-01
EGLAB Project No.: 24-059-009

DIRECT SHEAR

08/24 (ASTM D3080) Figure



Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-4	B1	1.0-3.0	Bulk	SC	○	278	29
					□	96	29

Note: Sample was remolded to **90 %** maximum relative density and optimum moisture

Maximum Dry Density: **133.5 pcf** Optimum Moisture: **8.0 %**

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)
500	7.8	19.5
1000	7.8	19.0
2000	7.8	18.1

EGLAB, INC.

Project Address:
Toll / Mill Creek 2

Client: LGC Valley, Inc.
Project No.: 244006-01
EGLAB Project No.: 24-059-009

DIRECT SHEAR

08/24 (ASTM D3080) Figure

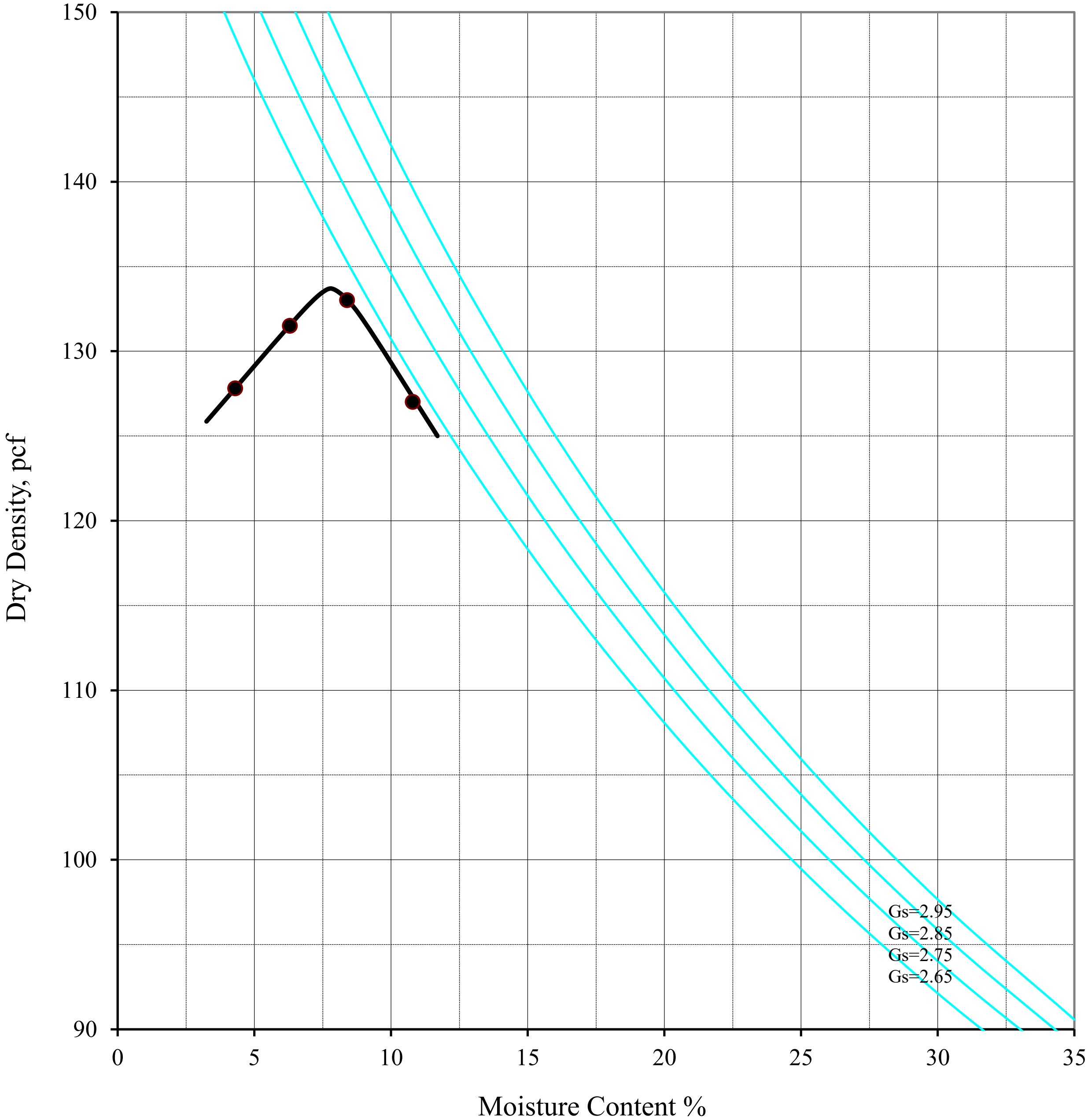
SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-6	B1	2.5-5.0	Bulk	SC	N/A	N/A

Gravel:	23.3%
Sand:	50.2%
Fine:	26.5%

EGLAB, INC.

Project Name:
Toll / Mill Creek 2
Client Job No.: 244006-01
Client Name: LGC Valley, Inc.
EGLAB Project No.: 24-059-009

GRAIN SIZE
DISTRIBUTION CURVE
08/07/24 (ASTM D422) FIGURE



Method "A"

Maximum Dry Density =	**133.5**	pcf
Optimum Moisture Content =	**8.0**	%

Boring No:	B-4
Sample:	B1
Depth :	1.0-3.0 feet
Description :	Clayey sand (SC), reddish brown, trace of gravel

EGLAB, INC.

Modified Proctor

(ASTM D1557)

Project Name:	Toll / Mill Creek 2	
Client Name:	LGC Valley, Inc.	
Job No.:	244006-01	
EGLAB Project No.:	24-059-009	
Date :	Aug-24	Figure

APPENDIX D

Previous Geotechnical Borings Logs (by Others)

TEST BORING LOG

B-1 Depth	Soil Classification	% Moisture	Dry Density, pcf	U.S.C.
0	FILL: Brown/reddish brown CLAYEY SAND			SC
3'	NATURAL: Brown/reddish brown SANDY SILT, some clay (32/50/18) @ 5'			ML
5'		10	125	
10'		15	113	
13.5'	Light reddish brown SILTY FINE-MEDIUM SAND			SM
15'		6	111	
17.5'	BEDROCK: Brown/reddish brown CLAYSTONE, weathered			
20'		20	112	
24'				
25'	Brown/reddish brown SILSTONE (30/56/14) @ 25'	11	112	
30'		13	114	
33'	Reddish brown/gray SANDSTONE			
35'		10	115	
40'		9	121	
45'	Light gray SANDSTONE	9	111	
50'		7	111	
56'	NO GROUNDWATER NO CAVING			

LEGEND:

▮ - Undisturbed core sample

(%Sand/% Silt/% Clay) @ sample depth

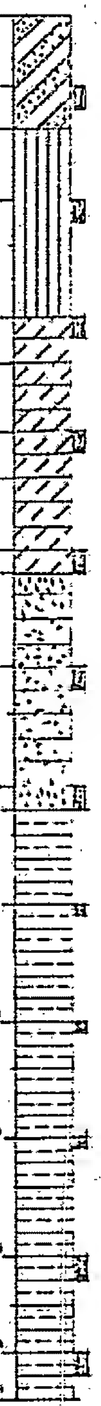
U.S.C. - Unified Soil Classification System Group Symbol

PLATE B-1

FILE NO. 85-10789
H. V. LAWMASTER & CO., INC.

TEST BORING LOG

B-2



Depth	Soil Classification	% Moisture	Dry Density, pcf	U.S.C.
0	FILL: Brown/reddish brown CLAYEY SAND with silt			SC
3'		11	115	
5'	NATURAL: Brown/reddish brown SANDY SILT, some clay (32/50/18) @ 8'			ML
8'		12	112	
13'	BEDROCK: Brown/reddish brown CLAYSTONE	10	121	
15'				
18'		11	119	
20'				
23'		12	102	
24'	Light grayish brown SANDSTONE			
28'		10	102	
33'		14	114	
34'	Yellowish brown SILTSTONE/SANDSTONE			
38'		5	111	
43'		6	114	
48'		10	115	
53'		8	116	
58'		11	113	
60'				

NO GROUNDWATER
NO CAVING

PLATE B-2

FILE NO. 85-10789
H. V. LAMMASTER & CO., INC.

APPENDIX E

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). No fill lift shall contain more than 10 percent organic matter. Nesting of 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 overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free from oversize material and the working surface is reasonably uniform, flat, and free from uneven features that would inhibit uniform compaction.

2.3 ***Overexcavation:*** In addition to removals and overexcavations 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 overexcavated 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 overexcavated 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 from 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 oversized 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 Section 3.1. 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 D1557-91).

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 D1557-91). 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 sheeps-foot 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 D1557-91.

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.

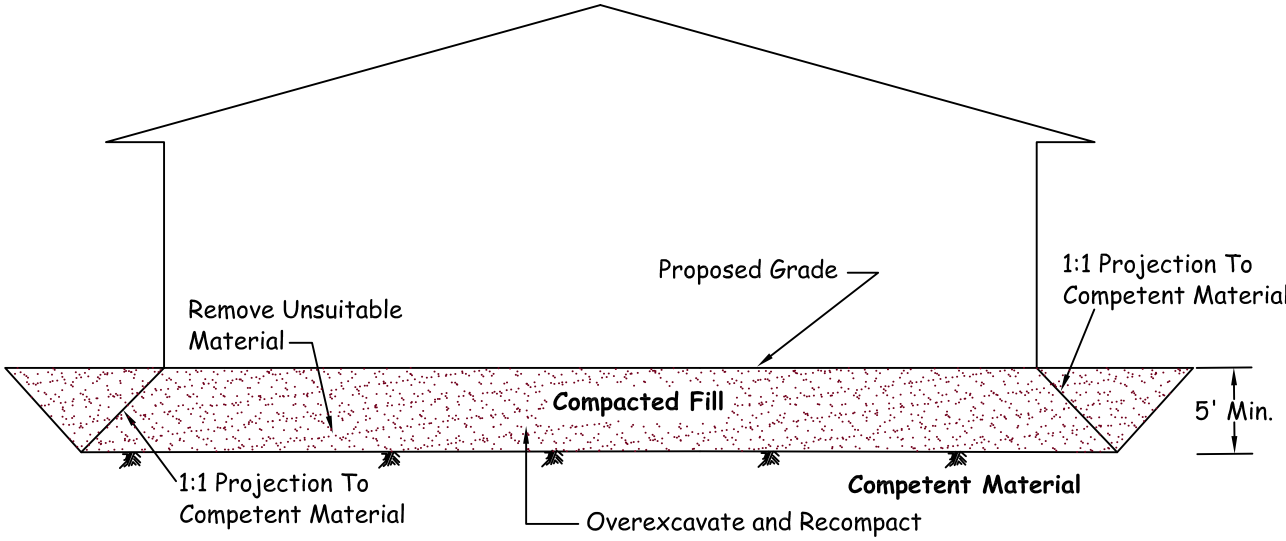
7.3 The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

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.

Cut Lot

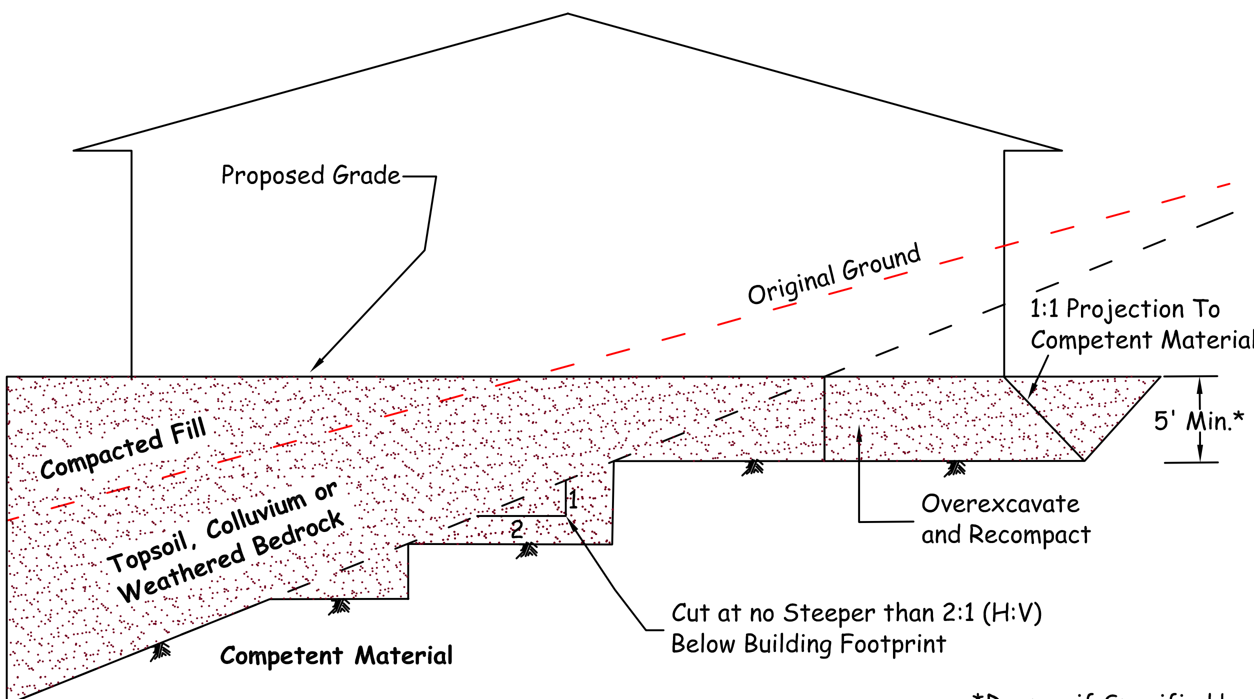
(Exposing Unsuitable Soils at Design Grade)



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

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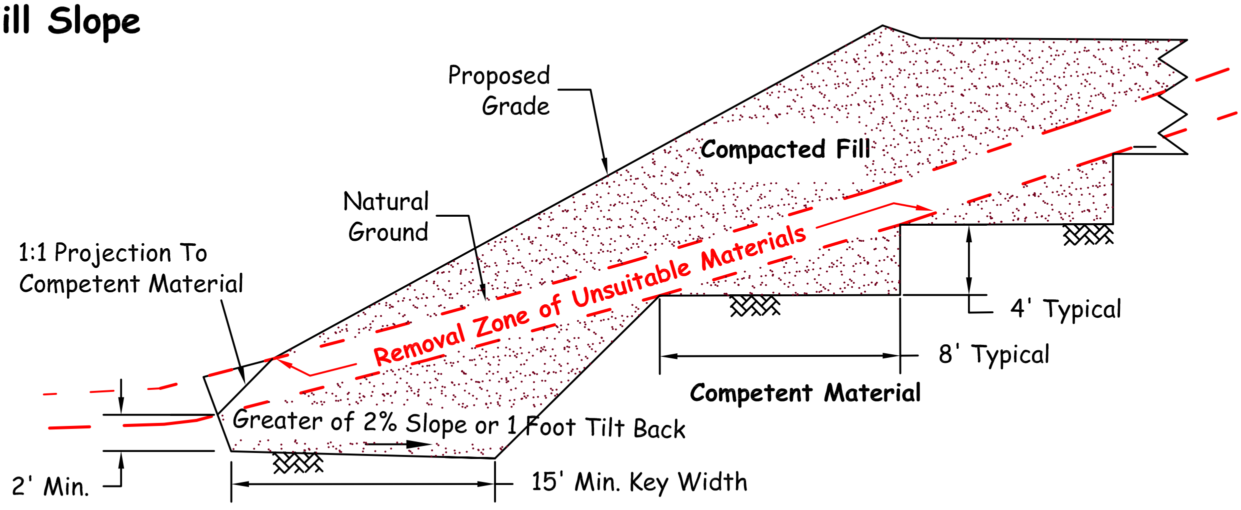
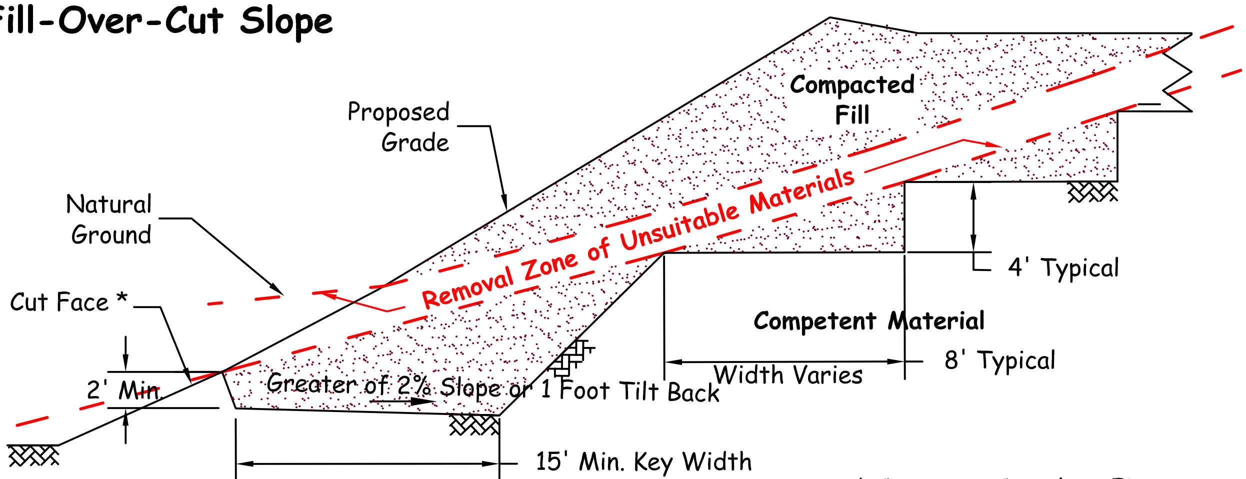
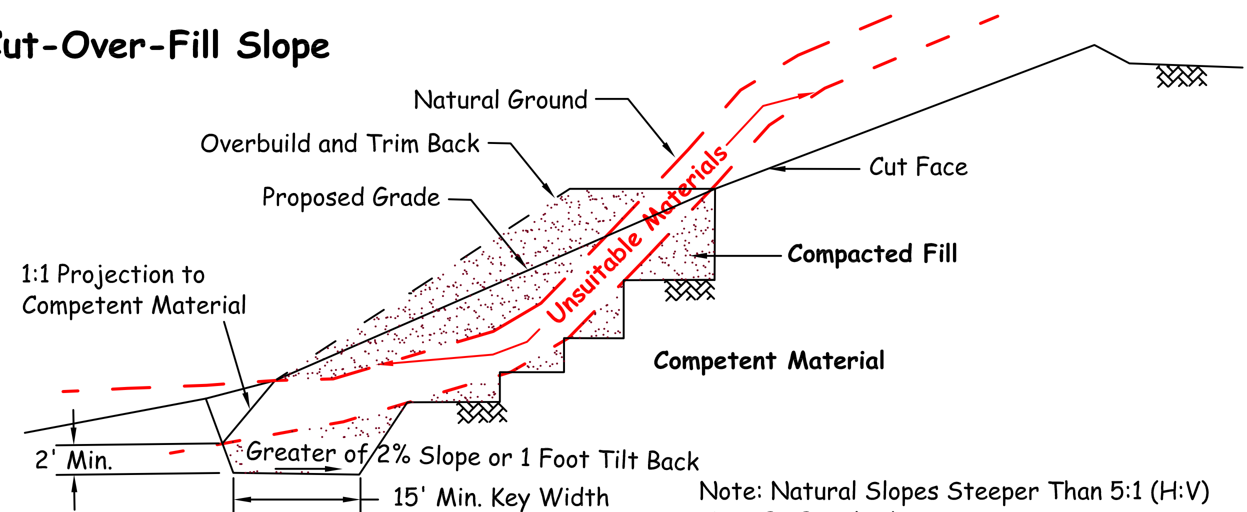
Fill Slope



Fill-Over-Cut Slope



Cut-Over-Fill Slope



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

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KEYING AND BENCHING

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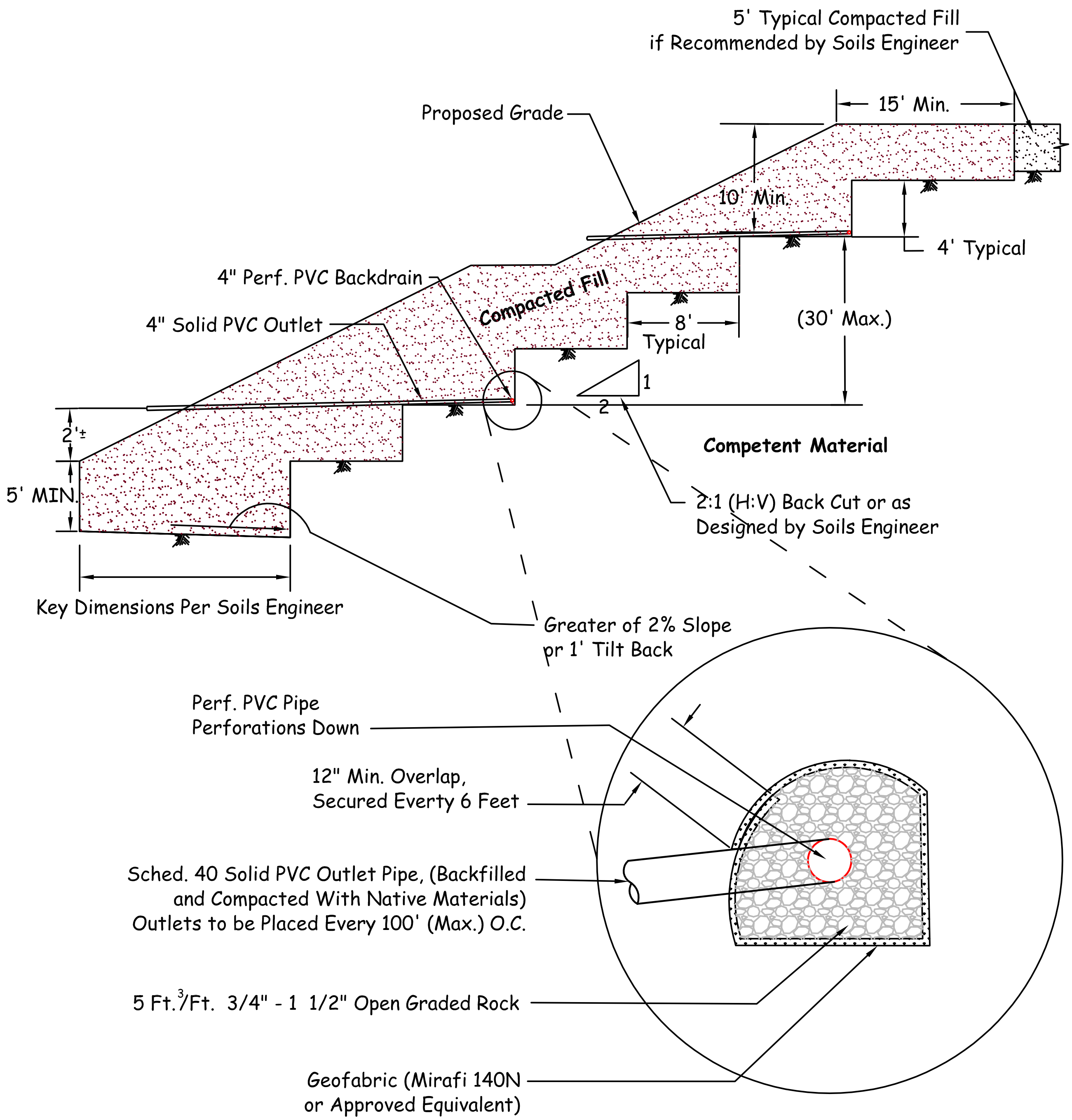

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TYPICAL BUTTRESS DETAIL

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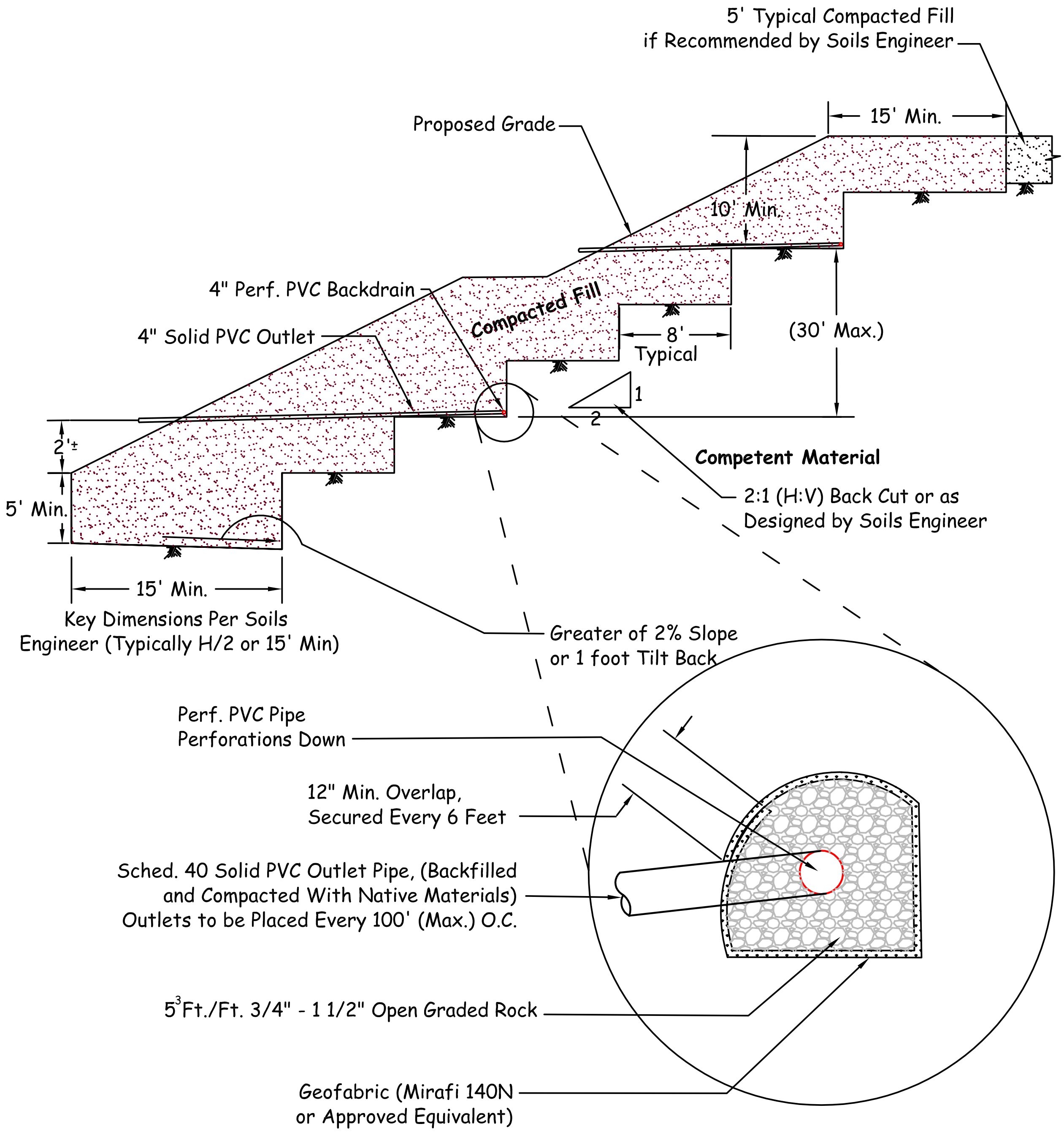

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TYPICAL STABILIZATION FILL DETAIL

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