

Appendix G

Geotechnical Report

December 18, 2024

Project No. 24105-01

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Subject: Geotechnical Evaluation Report, Proposed Battery Energy Storage System, 53 Via Jero Drive, Goleta, California

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical evaluation for the proposed development of the battery energy storage system (BESS) to be developed at 53 Via Jero Drive in Goleta, California. The purpose of our study was to evaluate the site geotechnical conditions in the context of the proposed grading and development to provide appropriate geotechnical design parameters and recommendations for the proposed project.

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

Sincerely,

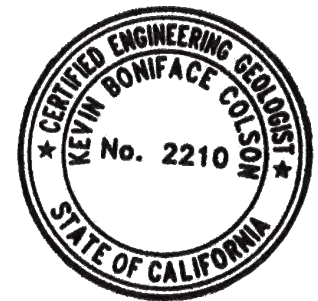
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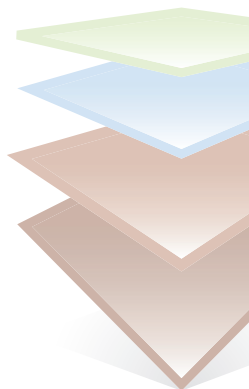


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1.0 INTRODUCTION

LGC Geotechnical has performed a geotechnical evaluation for the proposed battery energy storage system (BESS) to be developed at 53 Via Jero Drive in Goleta, California (Figure 1). Our study included evaluation and analysis of the proposed site plan (Scale, 2024). This report summarizes our findings, conclusions, and preliminary geotechnical design recommendations relative to the proposed grading and development.

1.1 Site and Project Description

The area of proposed development includes approximately 2.1 acres of currently vacant land. The site is relatively flat, sloping from north to south from elevations of approximately 115 feet above mean sea level to elevations of approximately 102 feet above mean sea level. The site is generally bound to the north by railroad tracks, to the west by the Ellwood Generating Station, to the south and east by parking lot areas for a nearby senior assisting facility.

The project will include grading for and construction of a battery storage yard, associated access roads and improvements. We understand that the battery storage yard will consist of an array of 6 foot by 30-foot storage containers with a total load of about 90 kips for each container. A preliminary grading plan was not available, however; site grading is anticipated to include less than approximately 5 to 10 feet of cut and fill. Retaining walls of any significant height are not anticipated.

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

1.2 Geotechnical Evaluation

On November 14, 2024, LGC Geotechnical performed geologic mapping and a subsurface geotechnical evaluation of the subject site consisting of the excavation of five hollow-stem auger borings in order to evaluate onsite geotechnical conditions.

Five borings (HS-1 through HS-3, I-1 and I-2) were excavated using a truck-mounted drill rig equipped with 6-inch and 8-inch-diameter hollow-stem augers to depths ranging from approximately 5 to 51.5 feet below existing grade.

The borings were excavated with a truck-mounted drill rig. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. Driven soil samples were collected by means of the Modified California Drive (MCD) sampler generally obtained at 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound automatic hammer falling 30 inches to advance

the sampler a total depth of 18 inches or until refusal. The raw (no correction factors applied) blow counts for each 6-inch increment of penetration were recorded on the boring logs. In select borings, after removal of the augers the depth of the boring due to caving was measured and is noted on the boring logs. Some settlement of the backfilled borings should be expected.

The results of our geologic mapping and the locations of our excavations are depicted on the Geotechnical Map (Figure 2). The boring Logs are presented in Appendix B.

At the completion of excavation of Infiltration Borings, I-1 and I-2, an infiltration well was constructed within each boring for testing as outlined in the “Preliminary Field Percolation Testing” Section below. At the completion of infiltration testing, the installed pipe was removed, and the resulting void backfilled with native soils.

Please note that some settlement of the backfill may occur over time and the excavations should be topped off as needed.

1.3 Preliminary Field Percolation Testing

Two falling-head field percolation tests (I-1 and I-2) were performed in the approximate locations indicated on the Geotechnical Map (Figure 2). A 3-inch diameter perforated PVC pipe was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils should be followed. If the water level does not meet that criterion, the procedure for fine-grained soils should be followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point. The pre-tests indicated the procedure for fine-grained soils should be followed. The calculated (observed) infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., “Porchet Method”). The observed infiltration rates are provided in Table 1 below and do not include any factors of safety.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Infiltration Test Approximate Depth (ft)	Observed Infiltration Rate (inch/hr.) *
I-1	5	0.0
I-2	5	0.0

*Does not include a factor of safety

It should also be emphasized that infiltration test results are only representative of the location and depth where they are performed. Varying subsurface conditions may exist outside of the test

locations which could alter the calculated infiltration rates indicated above. The percolation tests were performed using relatively clean water free of particulates, silt, etc. Field percolation test data is presented in Appendix B. Refer to further discussion in Section 4.9.

1.4 Laboratory Testing

Representative samples were obtained for laboratory testing during our recent field evaluation. Laboratory testing from included in-situ unit weight and moisture content, Atterberg Limits, fines content, consolidation, direct shear, expansion index, laboratory compaction and corrosion (sulfate, chloride, pH and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 94 pounds per cubic foot (pcf) to 125 pcf, with an average of 112 pcf. Field moisture contents ranged from approximately 6 percent to 71 percent, with an average of approximately 15 percent.
- Atterberg Limit (liquid limit and plastic limit) tests were performed on site soils. Results indicated Plasticity Index values ranging from 5 to 12.
- Two fines content tests indicated fines content (percent passing No. 200 sieve) of approximately 44 and 56 percent.
- Consolidation tests were performed. The deformation versus vertical stress plots are provided in Appendix C.
- Expansion Index (EI) tests indicated EI values ranging from 19 and 64, corresponding to “Very Low” and “Medium” expansion potential.
- A laboratory compaction test resulted in maximum dry density value of 125.5 pcf with optimum moisture content of 10.5 percent.
- A remolded direct shear test was performed. The plot is provided in Appendix C.
- Corrosion testing indicated soluble sulfate contents less than approximately 0.01 percent, chloride content of 134 parts per million (ppm), pH value of 7.5, and minimum resistivity value of 645 ohm-centimeter.

Dry unit weight and moisture content are provided on the boring logs. Laboratory test results are provided in Appendix C.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The site is located in the western portion of the Transverse Ranges. The Transverse Ranges are an east-west trending series of steep mountain ranges and valleys. The western Transverse Ranges consist mainly of variably deformed and faulted marine and nonmarine sedimentary rocks and deposits that range in age from Jurassic to the present. More recently, the local geology has also been influenced by a drop in sea level resulting in a series of abandoned terrace deposits, both marine and non-marine.

2.2 Site-Specific Geology

The geologic materials identified in our study include Quaternary terrace deposits overlying Tertiary Monterey Formation (Minor, 2009 & Dibblee, 1987). Minor amounts of undocumented fill material may be present at the surface of the site. The extent of the undocumented fill has not been mapped due to its thin nature and lack of lateral continuity. The typical onsite characteristics of the materials are described in the following subsections (from youngest to oldest). The approximate lateral extent of the geologic units encountered is presented on the Geotechnical Map (Figure 2). The topographic base utilized for the Geotechnical Map was provided by Scale Microgrids (2024).

The site is not located within a mapped State of California Earthquake Fault-Rupture Hazard Zone per compiled maps released by the CGS (2018), and there are no known active or potentially active faults on-site.

2.2.1 Artificial Fill - undocumented (Not mapped)

Minor amounts of undocumented fill material may be present in the near-surface of the site. The extent of the undocumented fill has not been mapped due to its thin nature and lack of lateral continuity. For the purposes of this report the undocumented fill material is not differentiated from the weathered, upper portion of the terrace deposits.

2.2.2 Quaternary Terrace Deposits (Map Symbol: Qt)

Quaternary terrace deposits were encountered at the surface and extend to a depth of approximately 10 to 15 feet below the surface. The terrace deposits overlie the Tertiary Monterey Formation bedrock. The terrace materials consist of generally hard, sandy, silty clay. In general, the basal portion of the terrace, originally deposited on a wave-cut platform into the Capistrano Formation bedrock, is composed of ancient beach deposits consisting of medium to coarse sand and some gravel.

2.2.3 Tertiary Monterey Formation (Map Symbol – Tm)

Tertiary Monterey Formation bedrock underlies the entire site at depth. This material generally consists of massive to thinly bedded hard sandy and silty claystone and very dense clayey and silty sandstone.

2.3 Geologic Structure

Geologic structure was not identified in the subject site geotechnical evaluation. The terrace deposits are regionally mapped as dipping gently to the west (Minor, 2009). The underlying Monterey Formation materials are mapped as dipping moderately to the south in the canyons to the west of the site. Regionally, the Monterey Formation has been folded into a series of anticlinal and synclinal folds. Several east-west trending faults are also mapped in the region, although not in the immediate vicinity of the site.

2.4 Landslides

The topography of the site and surrounding area is generally flat. Our research and field observations do not indicate the presence of landslides on the site or in the immediate vicinity. Review of regional geologic maps of the area do not indicate the presence of known or suspected landslides in the vicinity of the site. Therefore, the possibility of landslides at the site is considered nil.

2.5 Groundwater

Groundwater was encountered in boring HS-1 at a depth of approximately 35 feet below existing grade (approximate elevation of 71 feet). Historical groundwater information was limited. A site located approximately 800 feet to the northwest was monitored for groundwater elevations from June of 2001 through January of 2011. Readings indicate a high groundwater elevation of approximately 75 feet, about 25 feet below existing grade (California Water Resources Board, 2024).

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

2.6 Seismicity and Faulting

California is located on the boundary between the Pacific and North American Lithospheric Plates. The average motion along this boundary is on the order of 50-mm/yr. in a right-lateral sense. Most of the motion is expressed at the surface along the northwest trending San Andreas Fault Zone with lesser amounts of motion accommodated by sub-parallel faults located predominantly west of the San Andreas including the Elsinore, Newport-Inglewood, Rose

Canyon, and Coronado Bank Faults. Within Southern California, a large bend in the San Andreas Fault north of the San Gabriel Mountains has resulted in a transfer of a portion of the right-lateral motion between the plates into left-lateral displacement and vertical uplift. Compression south and west of the bend has resulted in folding, left-lateral, reverse thrust faulting, and regional uplift creating the east-west trending Transverse Ranges and several east-west trending faults. Further south within the Los Angeles Basin, “blind thrust” faults are believed to have developed below the surface also as a result of this compression, which have resulted in earthquakes such as the 1994 Northridge event along faults with little to no surface expression.

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

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

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

2.6.1 Lurching and Shallow Ground Rupture

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

2.6.2 Liquefaction and Dynamic Settlement

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

The borings primarily encountered terrace deposits over dense/hard bedrock of the Tertiary Monterey Formation. These soils due to their apparent density/consistency (high blow counts) are not considered susceptible to liquefaction and liquefaction-induced settlement. The potential for liquefaction is considered very low.

2.6.3 Lateral Spreading

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

Due to the very low potential for liquefaction, the potential for lateral spreading is considered very low.

2.6.4 Tsunamis and Seiches

Based on the elevation of the site, with respect to sea level, there is a low possibility of damage to the site during a large tsunami event. The site is not located within the Tsunami Inundation Area delineated on the Tsunami Inundation Map for Emergency Planning Dos Pueblos Canyon Quadrangle (CEMA, 2009).

2.7 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates

of latitude 34.4314 degrees north and longitude -119.8994 degrees west 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 Table 1 on the following page. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

TABLE 1

Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	C	Chapter 20 of ASCE 7
S _s (Risk-Targeted Spectral Acceleration for Short Periods)	2.364g	From SEAOC, 2024
S ₁ (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.832g	From SEAOC, 2024
F _a (per Table 1613.2.3(1))	1.2	For Simplified Design Procedure of Section 12.14 of ASCE 7, F _a shall be taken as 1.4 (Section 12.14.8.1)
F _v (per Table 1613.2.3(2))	1.4	-
S _{MS} for Site Class C [Note: S _{MS} = F _a S _s]	2.837g	-
S _{M1} for Site Class C [Note: S _{M1} = F _v S ₁]	1.165g	-
S _{DS} for Site Class C [Note: S _{DS} = (2/3) S _{MS}]	1.891g	-
S _{D1} for Site Class C [Note: S _{D1} = (2/3) S _{M1}]	0.777g	-
C _{RS} (Mapped Risk Coefficient at 0.2 sec)	0.874	ASCE 7 Chapter 22
C _{R1} (Mapped Risk Coefficient at 1 sec)	0.870	ASCE 7 Chapter 22

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.95 at a distance of approximately 3.83 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.79 at a distance of approximately 5.51 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be

used for liquefaction potential. The PGA_M for the site is equal to 1.26g (SEAOC, 2024). The design PGA is equal to 0.84g (2/3 of PGA_M).

2.8 Rippability

In general, excavation for foundations and underground improvements should be achievable with the typical grading equipment (scrapers, dozers, backhoes, etc.).

2.9 Oversized Material

Generation of a surplus of oversized material (material greater than 8 inches in maximum dimension) is generally not anticipated during site grading. However, some oversized material may be encountered, which may result in excavation difficulty for narrow excavations. Recommendations are provided for appropriate handling of oversized materials in Appendix D.

2.10 Expansive Soil Characteristics

Expansion Index (EI) testing indicated EI values of 19 and 64, corresponding to “Very Low” and “Medium” expansion potential. Conformational expansion potential testing should be performed at the completion of site grading based on the finish graded conditions.

2.11 Corrosion Potential

Corrosion testing indicated soluble sulfate contents less than approximately 0.01 percent, chloride content of 134 parts per million (ppm), pH value of 7.5, and minimum resistivity value of 645 ohm-centimeter. Site soils are not considered corrosive based on Caltrans guidelines (Caltrans, 2021).

3.0 FINDINGS AND CONCLUSIONS

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

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

- Based on the findings of our evaluation the site is underlain by Quaternary terrace deposits overlying Tertiary Monterey Formation bedrock material.
- The near surface soils on the site are considered potentially compressible and are not considered suitable for the planned improvements in their present condition. Remedial grading of the near surface soils will be required in the proposed development area.
- Groundwater was encountered in boring HS-1 at a depth of approximately 35 feet below existing grade (approximate elevation of 71 feet). A site located approximately 800 feet to the northwest was monitored for groundwater from June of 2001 through January of 2011 indicated a high groundwater elevation of approximately 75 feet, about 25 feet below existing grade. Shallower localized perched groundwater may also be encountered.
- The soils encountered in our borings are not considered susceptible to liquefaction and liquefaction-induced settlement due to their apparent density/consistency, therefore the potential for liquefaction and liquefaction-induced settlement is considered low.
- The subject site is not located within a Fault Rupture Hazard Zone and there are no known active or potentially active faults onsite (CGS, 2018). The proposed development will likely be subjected to strong seismic ground shaking during its design life from one of the regional faults.
- In general, excavation for grading and removals should be achievable with heavy duty construction equipment in good working order. Lighter equipment for use in foundation and utility excavation and equipment for drilling borings may encounter difficult excavation where cobble or oversize material is encountered.
- From a geotechnical perspective, the existing onsite soils are considered suitable material for use as general fill (not retaining wall backfill), provided that they are relatively free from oversized material (larger than 8 inches in maximum dimension), construction debris, and significant organic material. Moisture conditioning will be required to obtain the required compaction.
- Based on our evaluation, site soils are anticipated to have low to medium expansion potential. Mitigation measures are recommended for foundations and site improvements such as concrete flatwork to minimize the impacts of expansive soils.
- Pre-soaking of the subgrade for at-grade foundation slabs and site flatwork is recommended due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking should be accounted for in the construction schedule. (Typically, approximately two weeks). Additional time at the completion of presoaking may be necessary for the surface soils of the pad to dry back sufficiently to be capable of supporting trenching equipment. Alternatives to presoaking may include placing fill at a higher moisture content. Presoaking is not necessary for foundations supported by a deep foundation system (e.g., piles).

- Generation of significant quantities of oversized material (material greater than 8 inches in maximum dimension) is generally not anticipated during site grading.
- Percolation field testing resulted in essentially zero infiltration. The site consists of primarily hard/dense fine-grained soils, these soils have very low permeability and therefore have very low infiltration rates.
- The site contains soils that are not suitable for any required retaining wall backfill due to their fines content and expansion potential, therefore import of sandy soils will be required by the contractor for obtaining suitable backfill soil for planned site retaining walls.
- Based on the findings of our evaluation and analysis provided herein, and provided our recommendations are properly implemented during grading and construction, the proposed development of the site is not anticipated to significantly impact adjacent properties.

Please note that the conclusions and recommendations contained herein are based on preliminary subsurface conditions, which have been interpreted from a limited number of subsurface excavations. These conclusions and recommendations should be verified during site grading and adjusted according to the actual exposed field conditions.

4.0 RECOMMENDATIONS

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

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

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of site preparation, remedial grading, removals of potentially compressible soil, excavation of cut material, fill placement, and slab-on-grade foundation construction. We recommend that earthwork onsite be performed in accordance with the following recommendations, the City of Goleta grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix D. In case of conflict, the following recommendations shall supersede those included as part of Appendix D.

4.1.1 Site Preparation

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

If cesspools or septic systems are encountered, they should be removed in their entirety.

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

4.1.2 Removal Depths and Limits

In the proposed development area, potentially compressible/collapsible materials not removed by the planned design cuts should be excavated to competent material and replaced with compacted fill soils.

In general, remedial grading should be performed so that the near-surface soils are removed to at least 2 feet below existing grade prior to fill placement. Similarly, where not achieved by proposed cuts, the near-surface soils in proposed cut areas should be removed and recompacted within 2 feet of ensuring grade, specific recommendations for the proposed improvement areas are provided below.

Structural Improvements: In order to provide a relatively uniform bearing condition for the planned structural improvements, removals should extend a minimum depth of 5 feet below existing grade or 2 feet below the proposed footings, whichever is greater. In general, the envelope for removals should extend laterally a minimum horizontal distance of 5 feet beyond the edges of the proposed foundation footprints

If the storage containers are supported on a deep foundation system, earthwork removals for a cut condition are generally not required. For areas requiring fill placement, remedial grading should be performed so that the near-surface soils are removed to at least 2 feet below existing grade prior to fill placement to design grade as outlined above.

Retaining/Free-Standing Wall Structures: Removals should extend a minimum of 3 feet below existing grade, or 1-foot below proposed footings, whichever is greater. In general, the envelope for removals should extend laterally a minimum horizontal distance of 3 feet beyond the edges of the proposed foundation footings.

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

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

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Soil conditions should be mapped and frequently checked by a representative of LGC Geotechnical to verify conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained during construction. Excavation safety and protection of adjacent offsite improvements is the responsibility of the contractor.

Surcharge loads (e.g., soil stockpiles, construction equipment, etc.) placed on top of the excavation/temporary slope should not be permitted within a horizontal distance equal to the height of the excavation/temporary slope from the top of the excavation or 5 feet from the top of the slope, whichever is greater, unless the cut is properly shored and designed for the applicable surcharge load. In general, any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation.

Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

4.1.4 Removal Bottoms and Subgrade Preparation

In general, removal bottom areas and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and re-compacted per project recommendations. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.5 Material for Fill

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

From a geotechnical viewpoint, any required import soils for general fill (not retaining wall backfill) should consist of clean, soils of Low expansion potential (expansion index 50 or less based on ASTM D4829) or less and no particles larger than 3 inches in greatest dimension. Source samples of planned importation should be provided to the geotechnical consultant for laboratory testing a minimum of 3 working days prior to any planned importation for required laboratory testing.

Any required retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D422 (or ASTM D1140) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches. The site contains soils that are not suitable for retaining wall backfill due to their clay content and oversize material; therefore, import will be required by the contractor for obtaining suitable retaining wall backfill soil.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the latest requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

4.1.6 Fill Placement and Compaction

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM Test Method D1557). Moisture-conditioning of site soils will be required in order to achieve adequate compaction. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by the geotechnical consultant. Oversized material, as previously defined, should be removed from site fills or be appropriately handled (Appendix D).

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

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

Fill slope faces should also be compacted to project requirements. This may require overbuilding of the slope face and trimming back to design grades. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical.

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

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts

(typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric to prevent the migration of fines into the rock backfill.

4.1.7 Trench and Retaining Wall Backfill and Compaction

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

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

Retaining wall backfill should consist of predominately granular, sandy soils as outlined in Section 4.1.5. The limits of select sandy backfill should extend at minimum $\frac{1}{2}$ the height of the retaining wall or the width of the heel (if applicable), whichever is greater, refer to Figure 3. If the limits of select backfill depicted cannot be achieved due to property line constraints, increased lateral earth pressures should be used as provided in Section 4.8. Retaining wall backfill soils should be compacted in relatively uniform thin lifts to a minimum of 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted. If gap-graded rock is used for retaining wall backfill, refer to above Section 5.1.9.

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

4.1.8 Preliminary Shrinkage and Bulking

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is a preliminary estimate of shrinkage and bulking factors for the various geologic units found onsite. Allowance in the earthwork volumes budget should be made for an estimated ± 5 to 10 percent reduction in volume of in-place undocumented fill and weathered native materials.

It should be stressed that these values are only estimates and that an actual shrinkage factor is extremely difficult to predetermine. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor. Additionally, the onsite geology is very complex, the above estimates are generalized groupings of similar lithologies and should be expected to vary across the site and with depth. The above shrinkage and bulking estimates are intended as an aid for project engineers in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies such as a balance pad should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during grading. Shrinkage and bulking are also expected to vary with variations in survey accuracy during rough grading.

4.2 Preliminary Foundation Recommendations

The proposed battery storage containers and electrical equipment may be supported on reinforced mat foundations, provided earthwork is performed in accordance with the recommendations presented in this report. Shallow foundations should be supported on properly compacted fill.

As an alternative, at your request we are providing geotechnical foundation recommendations for support of the battery storage containers on a deep foundation system consisting of driven steel beams (W8 x 31). The vertical compressive load for each pile is anticipated to be about 7 kips. An alternative to steel driven piles are torque piles.

Please note that the following foundation recommendations are preliminary and must be confirmed by the geotechnical consultant during and at the completion of grading.

Preliminary foundation recommendations are provided in the following sections. The foundation design must be performed by the structural engineer based on the following geotechnical parameters and minimum values provided. If alternative foundation types are requested, LGC Geotechnical can provide geotechnical parameters and recommendations.

4.2.1 Mat Foundation Pad Design and Construction

A mat foundation can be used for support of the proposed battery storage and equipment pads. Mat foundations can be useful in helping distribute structural loads, to span local irregularities in the supporting capacity of the foundation soils, and to reduce the impact of expansive soils.

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

$$k = 100[(B+1)/2B]^2$$

k = modulus of vertical subgrade reaction, pounds per cubic inch (pci)

B = mat foundation width (feet)

The mat slab should have a thickened perimeter edge with a minimum embedment of 12-inches below lowest adjacent grade. Mat slabs are to be supported on compacted fill soils properly prepared in accordance with the recommendations provided in this report. Minimum slab reinforcement should be determined by the structural engineer based on the imposed loading, crack control, etc.

It is recommended that subgrade soils below slabs be moisture conditioned in order to maintain the recommended moisture content up to the time of concrete placement. The mat slab subgrade soils should be presoaked/moisture conditioned to 120 percent of optimum moisture content to a depth of 18 inches prior to trenching the mat foundation systems. The moisture content of the slab subgrade should be verified by the geotechnical engineer within 1 to 2 days prior to concrete placement. In addition, this moisture content should be maintained around the immediate perimeter of the slab during construction and installation of the battery storage equipment and electrical equipment.

4.2.2 Shallow Foundation Design Parameters

Provided our earthwork recommendations are appropriately implemented, the mat foundation may be designed using an allowable soil bearing pressure of 1,500 pounds per square foot (psf). Spread and continuous footings may be designed with an allowable soil bearing pressure of 2,500 pounds per square foot (psf) provided the minimum footing width is 12 inches and minimum embedment is 18 inches below the adjacent grade. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by $\frac{1}{3}$ for short duration loading (i.e., wind or seismic loads). This increase is based on a reduced factor of safety for short duration loading.

In utilizing the above-mentioned allowable bearing capacity based on the estimated structural loading and provided our earthwork recommendations are implemented, foundation settlement due to estimated structural loads is anticipated to be on the order of 1-inch or less. Differential static settlement may be taken as half of the static settlement (i.e., $\frac{1}{2}$ -inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.25 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 330 pcf (maximum of 3,300 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive

resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.2.3 Preliminary Deep Foundation Design Parameters

To resist axial dead and live compressive loads, an allowable skin friction of 400 pounds per square foot (psf) may be used. The upper 2 feet of skin friction should be neglected. Allowable compressive capacity is based on a factor of safety of 2. Allowable skin friction should be reduced to 300 psf for any required resistance to uplift loading conditions. Piles should be spaced at a minimum on-center spacing of three times the width. The pile capacities are based on soil resistance only and may be further limited by the pile-head connection details and structural material strength.

For piles spaced a minimum of 5.5 pile widths on-center, an allowable passive pressure of 400 pcf may be used for passive resistance. The provided passive pressure is based on an arching factor of 2 (e.g., 200 pcf x 2) and should be limited to a maximum of 12 times the value provided above (e.g., 400 pcf to a maximum of 4,800 psf). The provided passive pressure is only applicable for the upper 20 feet (above groundwater) and level (5 horizontal feet to 1-foot vertical or flatter) soil conditions. If any of these or other conditions exist, LGC Geotechnical should be contacted for updated specific recommendations. Reduced passive pressures are applicable for closer spacings. Passive resistance should be neglected for the upper 2 feet of the pile. Allowable passive pressure values are based on a static factor of safety of 1.5 and may be increased by one-third for short-duration (i.e., wind and seismic) loading. The structural designer should incorporate appropriate factors of safety/load factors in their design. If requested, lateral pile capacity based on deflection criteria will be provided.

4.2.4 Foundation Subgrade Preparation and Maintenance

Moisture conditioning of the subgrade for equipment pad slabs will be required due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 2 weeks). The subgrade moisture condition of the equipment pad soils should be maintained at the recommended moisture content (refer to Section 4.2.1) up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to completion of the equipment pad construction. Presoaking is not necessary for foundations supported by a deep foundation system (e.g., piles).

4.2.5 Slab Underlayment Guidelines

Slab underlayment guidelines are typically provided for residential or commercial structures where moisture sensitive flooring may be installed. Therefore, the following

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

4.3 Lateral Earth Pressures for Retaining Wall Design

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

The following lateral earth pressures are presented on Table 2 below for approved select granular soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and Very Low expansion potential (EI of 20 or less per ASTM D4829). The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria.

TABLE 2

Lateral Earth Pressures – Select Sandy Backfill

Conditions	Equivalent Fluid Unit Weight (pcf)
	Level Backfill
	Approved Soils
Active	35
At-Rest	55

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed, refer to Figure 3. Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall

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

Per Section 1803.5.12 of the 2022 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. Conventional retaining walls greater than 6 feet in height are not anticipated. If required, a seismic lateral earth pressure increment will be provided.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3.2. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.4 Soil Corrosivity

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

Corrosion testing indicated soluble sulfate contents less than approximately 0.01 percent, chloride content of 134 parts per million (ppm), pH value of 7.5, and minimum resistivity value of 645 ohm-centimeter. Site soils are not considered corrosive based on Caltrans guidelines (Caltrans, 2021).

Based on laboratory sulfate test results, the near surface soils are designated to a class "S0" per ACI 318, Table 19.3.1.1 with respect to sulfates.

4.5 Non-structural Concrete Flatwork

Nonstructural concrete flatwork (such as walk-ways, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 3. These guidelines will reduce the potential for irregular cracking and promote cracking along control joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 3

Nonstructural Concrete Flatwork for Low/Medium Expansion Potential

	Flatwork	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	5 (full)	City/Agency Standard
Presoaking	Presoak to 12 inches	City/Agency Standard
Reinforcement	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of $\frac{1}{3}$ the concrete thickness	City/Agency Standard
Maximum Joint Spacing	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	City/Agency Standard

4.6 Freestanding Walls

To reduce the potential for unsightly cracks, due to differential settlement or expansive soils, we recommend the inclusion of control joints at a maximum of 15-foot on center for any proposed freestanding walls. This spacing may be altered by the wall designer based upon the wall reinforcement. If the soil-moisture content below the wall foundation varies significantly, some wall movement should be expected; however, this movement is unlikely to cause more than cosmetic distress.

4.7 Surface Drainage and Landscaping

4.7.1 Precise Grading

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed foundations be sloped away from the proposed foundations and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of foundations. Where the proposed development geometry necessitates that the drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained

system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

4.8 Subsurface Water Infiltration

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

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

Field testing resulted in essentially zero infiltration. The site consists of primarily hard/dense fine-grained soils, these soils have very low permeability and very low infiltration rates. In general, we do not recommend the intentional infiltration of storm water.

4.9 Pile Driving

Hard driving is anticipated, refer to the boring logs provided in Appendix B. Piles should be driven at least to the specified depth and the nominal resistance (at least 2 times the design vertical load) should be checked with the pile-driving formula given in Section 49-2.01 of the Caltrans Standard Specifications (2018). However, if the specified depth is reached without achieving the required nominal resistance, pile driving should continue until bearing is attained. In this case, it may be prudent to allow the pile to “set up” before continuing the driving. The selected pile-driving hammer should be able to deliver sufficient energy to drive the piles at a penetration rate of not less than 1/8-inch per blow at the required nominal resistance. Predrilling and vibratory hammers are generally not allowed for pile installation.

Vibrations associated with pile driving may cause distress to nearby existing improvements. The contractor should be informed about the potential concerns of any existing improvements so they can use equipment and/or implement measures that will reduce the impact of vibration and noise associated with pile driving. We recommend that existing nearby improvements be documented prior to pile driving, refer to Section 4.11 below. LGC Geotechnical should provide geotechnical observation during pile driving operations.

4.10 Pre-Construction Documentation and Construction Monitoring

It is highly recommended that a program of documentation and monitoring be devised and put into practice before the onset of any groundwork or pile driving. LGC Geotechnical can perform

these services at your request. This should include, but not necessarily be limited to, detailed documentation of the existing improvements, buildings, and utilities around the area of proposed excavation, with particular attention to any distress that is already present prior to the start of work. Subsequent readings should be scheduled consistent with the program of work.

4.11 Geotechnical Plan Review

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

4.12 Geotechnical Observation and Testing During Construction

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

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

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

5.0 LIMITATIONS

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

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

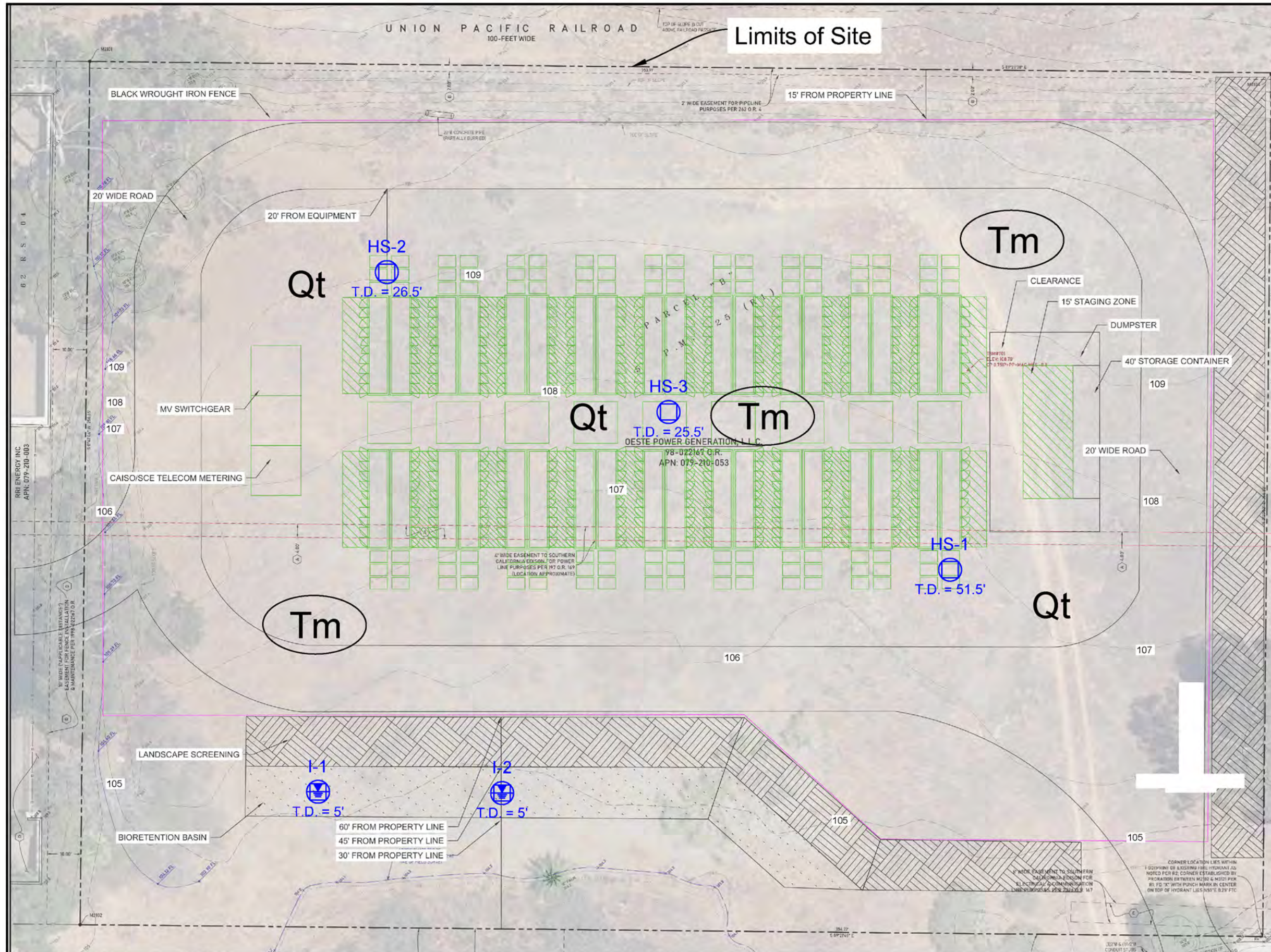
The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings and conclusions presented in this report can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

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



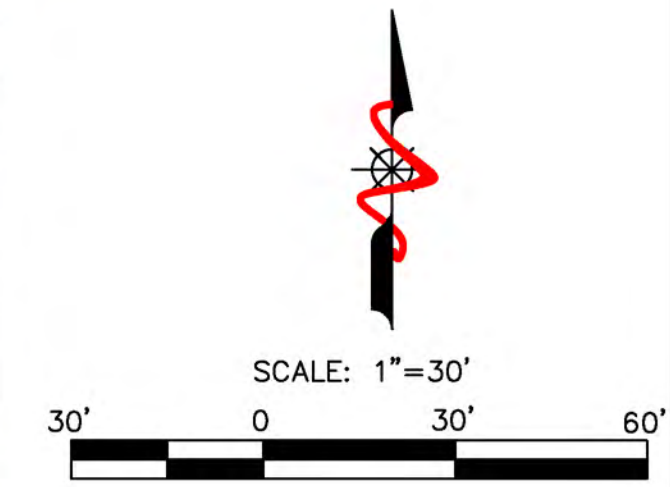
FIGURE 1
Site Location Map

PROJECT NAME	ERM - 53 Via Jero Dr
PROJECT NO.	24105-01
ENG. / GEOL.	BTZ / KBC
SCALE	Not to Scale
DATE	December 2024



LEGEND

Qt	Quaternary Terrace Deposits
Tm	Tertiary Monterey Formation, Circled Where Buried
HS-3	Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet
T.D. = 25.5'	
I-2	Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet
T.D. = 5'	



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Figure 2
Geotechnical Map

PROJECT NAME	ERM - 53 Via Jero Drive
PROJECT NO.	24105-01
ENG. / GEOL.	BTZ / KBC
SCALE	1" = 30'
DATE	December 2024

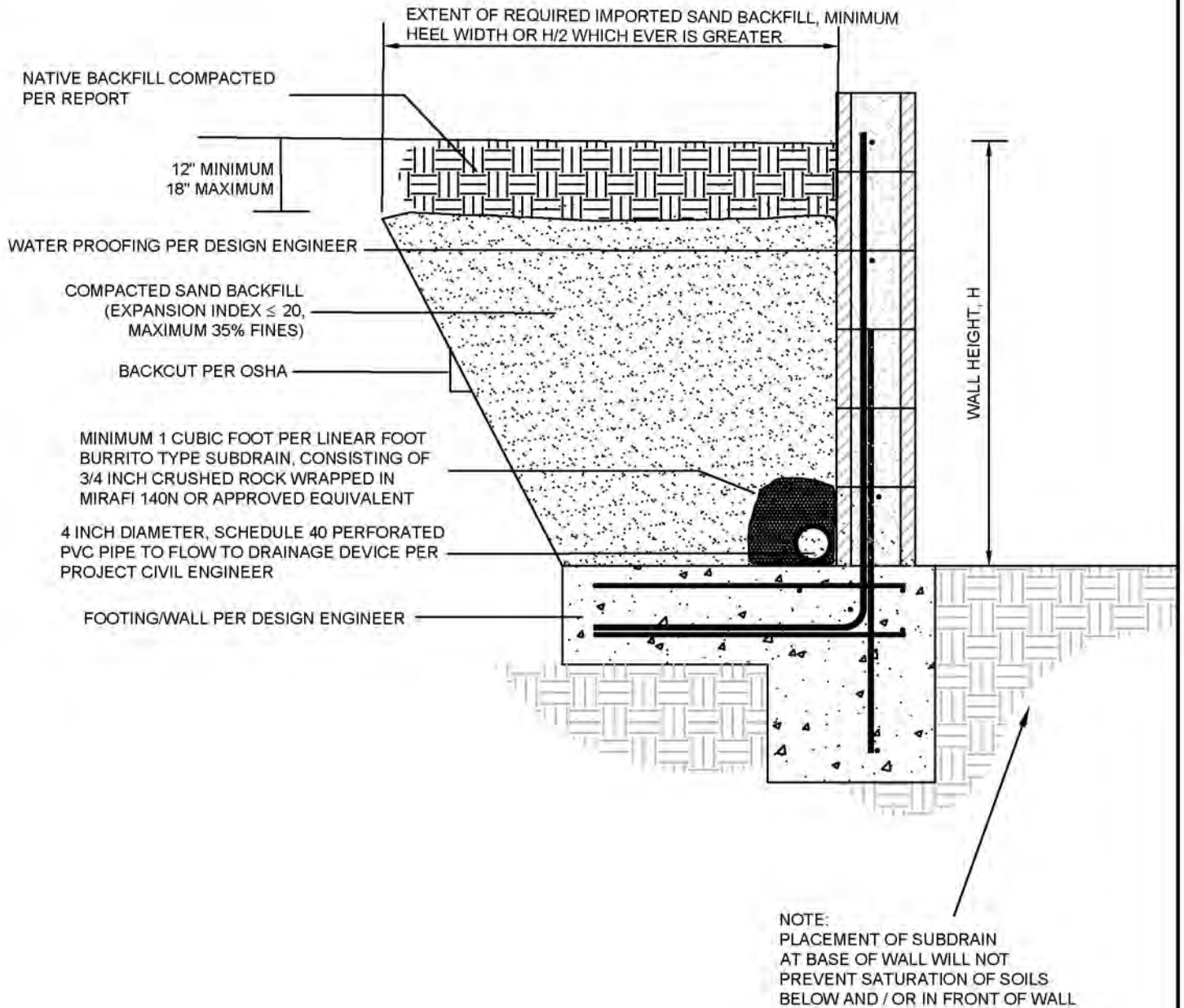


FIGURE 3
Retaining Wall
Backfill Detail

PROJECT NAME	ERM - 53 Via Jero Dr
PROJECT NO.	24105-01
ENG. / GEOL.	BTZ / KBC
SCALE	Not to Scale
DATE	December 2024

Appendix A
References

APPENDIX A

References

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Appendix B
Boring Logs

Geotechnical Boring Log Borehole HS-1

Date: 11/14/2024	Drilling Company: Choice Drilling
Project Name: ERM - 53 Via Jero Dr	Type of Rig: CME 95
Project Number: 24105-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~106' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
105	0							<u>Quaternary Terrace Deposits - (Qt)</u> @ 0' - Dry Vegetation	CR
100	5	B-1	R-1	50/6"	102.9	11.0	CL	@ 5' - Sandy CLAY: light brown, slightly moist, hard	
			R-2	45 50/4"	110.4	9.9		@ 7.5' - Sandy CLAY: light brown, slightly moist, hard	
95	10		R-3	50/6"	94.0	10.6	CL-ML	@ 10' - Sandy Silty CLAY: brown, slightly moist, hard	
90	15		R-4	30 50/6"	101.8	19.8	CL	<u>Tertiary Monterey Formation - (Tm)</u> @ 15' - CLAY with Sand: brown, very moist, hard	
85	20		SPT-1	8 33 50/5"		13.2	SC	@ 20' - Clayey SAND: brown, very moist, very dense	#200
80	25		R-5	24 50/6"	110.6	6.4	SM	@ 25' - Silty SAND: brown, slightly moist, very dense	
	30								

	THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.	<table style="width: 100%;"> <tr> <td>SAMPLE TYPES:</td> <td>TEST TYPES:</td> </tr> <tr> <td>B BULK SAMPLE</td> <td>DS DIRECT SHEAR</td> </tr> <tr> <td>R RING SAMPLE (CA Modified Sampler)</td> <td>MD MAXIMUM DENSITY</td> </tr> <tr> <td>G GRAB SAMPLE</td> <td>SA SIEVE ANALYSIS</td> </tr> <tr> <td>SPT STANDARD PENETRATION TEST SAMPLE</td> <td>S&H SIEVE AND HYDROMETER</td> </tr> <tr> <td></td> <td>EI EXPANSION INDEX</td> </tr> <tr> <td></td> <td>CN CONSOLIDATION</td> </tr> <tr> <td></td> <td>CR CORROSION</td> </tr> <tr> <td></td> <td>AL ATTERBERG LIMITS</td> </tr> <tr> <td></td> <td>CO COLLAPSE/SWELL</td> </tr> <tr> <td></td> <td>RV R-VALUE</td> </tr> <tr> <td></td> <td>#200 % PASSING # 200 SIEVE</td> </tr> </table>	SAMPLE TYPES:	TEST TYPES:	B BULK SAMPLE	DS DIRECT SHEAR	R RING SAMPLE (CA Modified Sampler)	MD MAXIMUM DENSITY	G GRAB SAMPLE	SA SIEVE ANALYSIS	SPT STANDARD PENETRATION TEST SAMPLE	S&H SIEVE AND HYDROMETER		EI EXPANSION INDEX		CN CONSOLIDATION		CR CORROSION		AL ATTERBERG LIMITS		CO COLLAPSE/SWELL		RV R-VALUE		#200 % PASSING # 200 SIEVE
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Geotechnical Boring Log Borehole HS-1

Date: 11/14/2024	Drilling Company: Choice Drilling
Project Name: ERM - 53 Via Jero Dr	Type of Rig: CME 95
Project Number: 24105-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~106' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
75	30		SPT-2	16 19 38		14.4	CL-ML	@ 30' - Sandy Silty CLAY: brown, moist, hard	
70	35		R-6	16 32 50	111.0	18.8	CL	@ 35' - CLAY: brown, moist, hard	
65	40		SPT-3	15 16 16		19.0	CL-ML	@ 40' - Sandy Silty CLAY: brown, very moist, hard	
60	45		R-7	42 50/6"				@ 45' - No Recovery	
55	50		SPT-4	5 7 11		71.4	CL	@ 50' - CLAY: dark brown, wet, very stiff	
50	55							Total Depth = 51.5' Perched Groundwater Between 35' - 40' Caving: Hole Measured Approximately 51.5' after Removal of the Augers Backfilled with Cuttings on 11/14/2024	




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

<p>SAMPLE TYPES:</p> <ul style="list-style-type: none"> B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE <p> GROUNDWATER TABLE</p>	<p>TEST TYPES:</p> <ul style="list-style-type: none"> DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE
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Geotechnical Boring Log Borehole HS-2

Date: 11/14/24	Drilling Company: Choice Drilling
Project Name: ERM - 53 Via Jero Dr	Type of Rig: CME 95
Project Number: 24105-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~109' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							Quaternary Terrace Deposits - (Qt) @ 0' - Dry Vegetation	MD DS EI CR
	5	B-1	R-1	13 28 50/5"	118.5	10.0	CL	@ 5' - Sandy CLAY: light brown, slightly moist, hard	
	7.5		R-2	12 18 29	110.0	15.5		@ 7.5' - Sandy CLAY to CLAY: orangish brown, moist, hard	CN
	10		R-3	24 31 50/5.5"	125.4	7.0	SM	@ 10' - Silty SAND with Gravel: brown, slightly moist, very dense	
	15		SPT-1	10 18 22		14.2	CL-ML	Tertiary Monterey Formation - (Tm) @ 15' - Sandy Silty CLAY: brown, moist, hard	#200 AL
	20		R-4	34 50/4"	120.3	6.5	SM	@ 20' - Silty SAND: brown, slightly moist, very dense	
	25		SPT-2	12 17 20		9.5	CL-ML	@ 25' - Sandy Silty CLAY: light brown, slightly moist, hard	
	30							Total Depth = 26.5' Groundwater Not Encountered Caving: Hole Measured Approximately 24' after Removal of Augers Backfilled with Cuttings on 11/14/2024	

	THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.	<table border="0"> <tr> <td>SAMPLE TYPES:</td> <td>TEST TYPES:</td> </tr> <tr> <td>B BULK SAMPLE</td> <td>DS DIRECT SHEAR</td> </tr> <tr> <td>R RING SAMPLE (CA Modified Sampler)</td> <td>MD MAXIMUM DENSITY</td> </tr> <tr> <td>G GRAB SAMPLE</td> <td>SA SIEVE ANALYSIS</td> </tr> <tr> <td>SPT STANDARD PENETRATION TEST SAMPLE</td> <td>S&H SIEVE AND HYDROMETER</td> </tr> <tr> <td></td> <td>EI EXPANSION INDEX</td> </tr> <tr> <td></td> <td>CN CONSOLIDATION</td> </tr> <tr> <td></td> <td>CR CORROSION</td> </tr> <tr> <td></td> <td>AL ATTERBERG LIMITS</td> </tr> <tr> <td></td> <td>CO COLLAPSE/SWELL</td> </tr> <tr> <td></td> <td>RV R-VALUE</td> </tr> <tr> <td></td> <td>#200 % PASSING # 200 SIEVE</td> </tr> </table>	SAMPLE TYPES:	TEST TYPES:	B BULK SAMPLE	DS DIRECT SHEAR	R RING SAMPLE (CA Modified Sampler)	MD MAXIMUM DENSITY	G GRAB SAMPLE	SA SIEVE ANALYSIS	SPT STANDARD PENETRATION TEST SAMPLE	S&H SIEVE AND HYDROMETER		EI EXPANSION INDEX		CN CONSOLIDATION		CR CORROSION		AL ATTERBERG LIMITS		CO COLLAPSE/SWELL		RV R-VALUE		#200 % PASSING # 200 SIEVE
SAMPLE TYPES:	TEST TYPES:																									
B BULK SAMPLE	DS DIRECT SHEAR																									
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	RV R-VALUE																									
	#200 % PASSING # 200 SIEVE																									

Geotechnical Boring Log Borehole HS-3

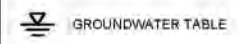
Date: 11/14/24	Drilling Company: Choice Drilling
Project Name: ERM - 53 Via Jero Dr	Type of Rig: CME 95
Project Number: 24105-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~107' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
105	0							Quaternary Terrace Deposits - (Qt) @ 0' - Dry Vegetation	
100	5		R-1	17 26 29	114.6	14.9	CL-ML	@ 5' - Silty CLAY: dark brown, moist, hard	AL CN
95	10		R-2	14 27 28	120.6	13.6	CL	Tertiary Monterey Formation - (Tm) @ 7.5' - Sandy CLAY: dark brown, moist, hard	
90	15		R-3	13 30 28	122.5	10.3		@ 10' - Sandy CLAY: grayish brown, slightly moist, hard	
85	20		R-4	12 20 40	109.9	19.9		@ 15' - CLAY with Sand: brown, very moist, hard	
80	25		SPT-1	28 29 31		7.0		@ 20' - Sandy CLAY: brown, slightly moist, hard	
75	25		R-5	50/6"	111.3	6.5	CL SM	@ 25' - Sandy CLAY transitioning to Silty SAND with Gravel: brown, slightly moist, hard/very dense	
70	25.5							Total Depth = 25.5' Groundwater Not Encountered Caving: Hole Measured Approximately 25.5' after Removal of Augers Backfilled with Cuttings on 11/14/2024	



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

SAMPLE TYPES:	TEST TYPES:
B BULK SAMPLE	DS DIRECT SHEAR
R RING SAMPLE (CA Modified Sampler)	MD MAXIMUM DENSITY
G GRAB SAMPLE	SA SIEVE ANALYSIS
SPT STANDARD PENETRATION TEST SAMPLE	S&H SIEVE AND HYDROMETER
	EI EXPANSION INDEX
	CN CONSOLIDATION
	CR CORROSION
	AL ATTERBERG LIMITS
	CO COLLAPSE/SWELL
	RV R-VALUE
	#200 % PASSING # 200 SIEVE



Geotechnical Boring Log Borehole I-1

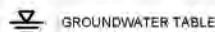
Date: 11/14/2024	Drilling Company: Choice Drilling
Project Name: ERM - 53 Via Jero Dr	Type of Rig: CME 95
Project Number: 24105-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~105' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry Vegetation	
100	5							Total Depth = 5' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 11/14/24 Backfilled with Cuttings on 11/15/24	
95	10								
90	15								
85	20								
80	25								
75	30								



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

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



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

Geotechnical Boring Log Borehole I-2

Date: 11/14/2024	Drilling Company: Choice Drilling
Project Name: ERM - 53 Via Jero Dr	Type of Rig: CME 95
Project Number: 24105-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~104' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry Vegetation	
100	5							Total Depth = 5' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 11/14/24 Backfilled with Cuttings on 11/15/24	
	10								
95	15								
90	20								
85	25								
80	30								



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

SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE GROUNDWATER TABLE	TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE
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Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Via Jero
Project Number: 24105-01
Date: 11/15/2024
Boring Number: I-1

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

*measured at time of test

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

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	6:52	7:17	25.0	1.20	1.21	0.01	No
2	7:17	7:42	25.0	1.21	1.21	0.00	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D _o (feet)	Final Depth to Water, D _f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate (in/hr)
1	7:42	8:12	30.0	1.21	1.21	0.00	0.0
2	8:12	8:42	30.0	1.21	1.21	0.00	0.0
3	8:42	9:12	30.0	1.21	1.21	0.00	0.0
4	9:12	9:42	30.0	1.21	1.22	0.01	0.0
5	9:42	10:12	30.0	1.22	1.22	0.00	0.0
6	10:12	10:42	30.0	1.22	1.22	0.00	0.0
7	10:42	11:12	30.0	1.22	1.23	0.01	0.0
8	11:12	11:42	30.0	1.23	1.23	0.00	0.0
9	11:42	12:12	30.0	1.23	1.24	0.01	0.0
10	12:12	12:42	30.0	1.24	1.24	0.00	0.0
11	12:42	13:12	30.0	1.24	1.25	0.01	0.0
12	13:12	13:42	30.0	1.25	1.25	0.00	0.0

Measured Infiltration Rate (No Factor of Safety)	0.0
Feasibility Factor of Safety	2.0
Measured Infiltration Rate (With Factor of Safety)	0.0

Sketch:

Notes:



Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Via Jero
Project Number: 24105-01
Date: 11/15/2024
Boring Number: I-2

Test hole dimensions (if circular)

Boring Depth (feet)*: 5
 Boring Diameter (inches): 8
 Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet): _____
 Pit Length (feet): _____
 Pit Breadth (feet): _____

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	6:54	7:19	25.0	0.89	0.89	0.00	No
2	7:19	7:44	25.0	0.89	0.89	0.00	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D _o (feet)	Final Depth to Water, D _r (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate (in/hr)
1	7:44	8:14	30.0	0.89	0.89	0.00	0.0
2	8:14	8:44	30.0	0.89	0.89	0.00	0.0
3	8:44	9:14	30.0	0.89	0.9	0.01	0.0
4	9:14	9:44	30.0	0.9	0.9	0.00	0.0
5	9:44	10:14	30.0	0.9	0.91	0.01	0.0
6	10:14	10:44	30.0	0.91	0.91	0.00	0.0
7	10:44	11:14	30.0	0.91	0.92	0.01	0.0
8	11:14	11:44	30.0	0.92	0.92	0.00	0.0
9	11:44	12:14	30.0	0.92	0.93	0.01	0.0
10	12:14	12:44	30.0	0.93	0.93	0.00	0.0
11	12:44	13:14	30.0	0.93	0.94	0.01	0.0
12	13:14	13:44	30.0	0.94	0.94	0.00	0.0

Measured Infiltration Rate (No Factor of Safety)

0.0

Feasibility Factor of Safety

2.0

Measured Infiltration Rate (With Factor of Safety)

0.0

Sketch:

Notes:

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019



Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Test Results

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

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

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

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-2 @ 7.5 ft	29	17	12	CL
HS-2 @ 15 ft	23	18	5	CL-ML
HS-3 @ 5 ft	21	16	5	CL-ML

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

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 20 ft	Clayey Sand	44
HS-2 @ 15 ft	Sandy Silty Clay	56

APPENDIX C (Cont'd)

Laboratory Test Results

Expansion Index: The expansion potential of selected representative samples was evaluated by the Expansion Index Test per ASTM D4829. The results are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 1-5 ft	64	Medium
HS-2 @ 1-5 ft	19	Very Lot

* Per ASTM D4829

Consolidation: Consolidation tests were performed per ASTM D2435. Samples (2.4 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 provided in this Appendix.

Direct Shear: A direct shear test was performed on a select remolded (90 percent relative compaction) sample, which were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plot is provided in this Appendix.

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2 @ 0-5 ft	Brown Sandy Silty Clay	125.5	10.5

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The test results are presented in the table below.

Sample Location	Sulfate Content (ppm)	Sulfate Content (%)
HS-1 @ 1-5 ft	46	<0.01
HS-2 @ 1-5 ft	68	<0.01

APPENDIX C (Cont'd)

Laboratory Test Results

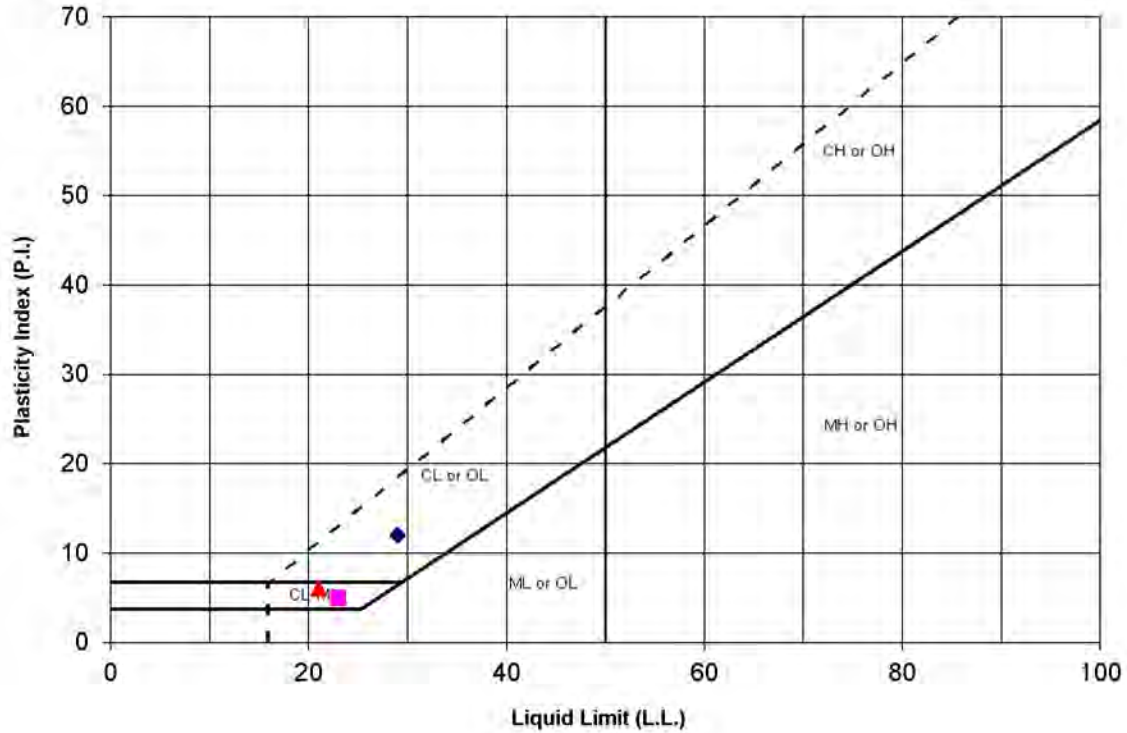
Chloride Content: Chloride content was tested per CTM 422. The results are presented below.

Sample Location	Chloride Content (ppm)
HS-1 @ 1-5 ft	134

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

Sample Location	pH	Minimum Resistivity (ohm-cm)
HS-1 @ 1-5 ft	7.5	645

PLASTICITY CHART - CLASSIFICATION OF FINE-GRAINED SOILS



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	HS-2	R-2	7.5'	-	29	17	12	CL
■	HS-2	SPT-1	15'	56	23	18	5	CL-ML
▲	HS-3	R-1	5'	-	21	16	5	CL-ML



ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 24105-01
Date: Dec-24

53 Via Jero Dr.

Location	Sample No.	Depth (ft)	Molding Moisture Content (%)	Initial Dry Density (pcf)	Final Moisture Content (%)	Expansion Index	Expansion Classification ¹
HS-1	B-1	0-5'	9.5	111.1	23.1	64	Medium
HS-2	B-1	0-5'	8.5	115.3	16.6	19	Very Low

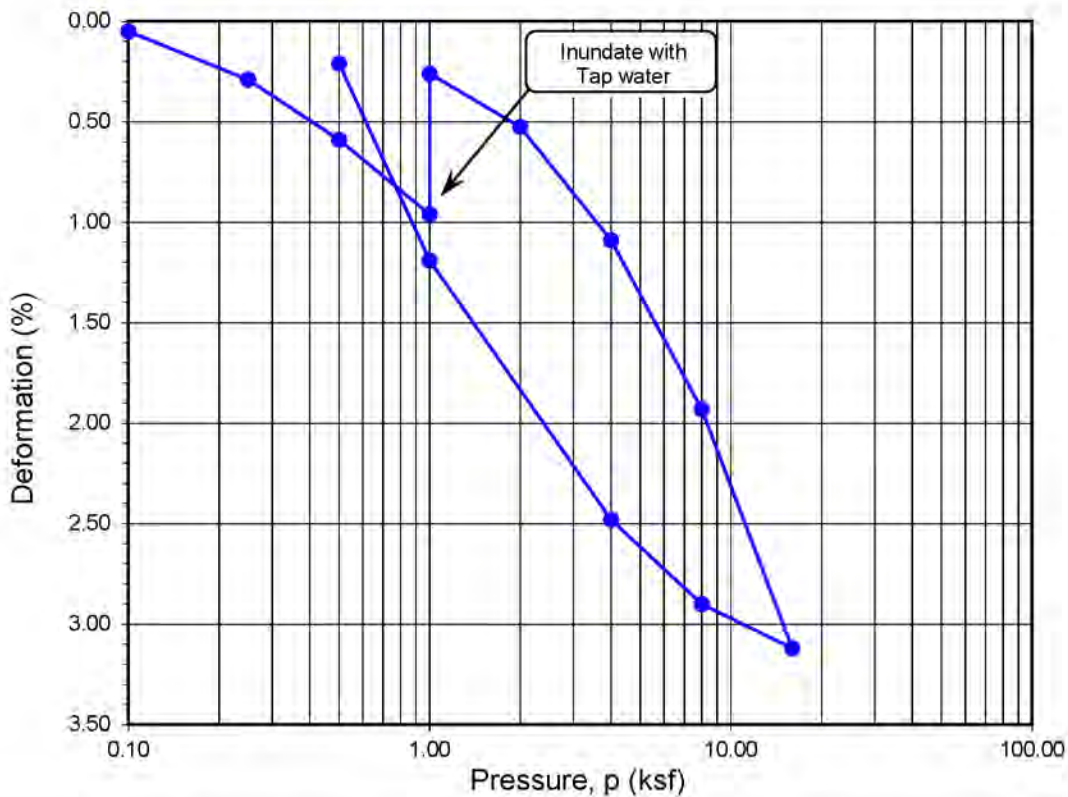
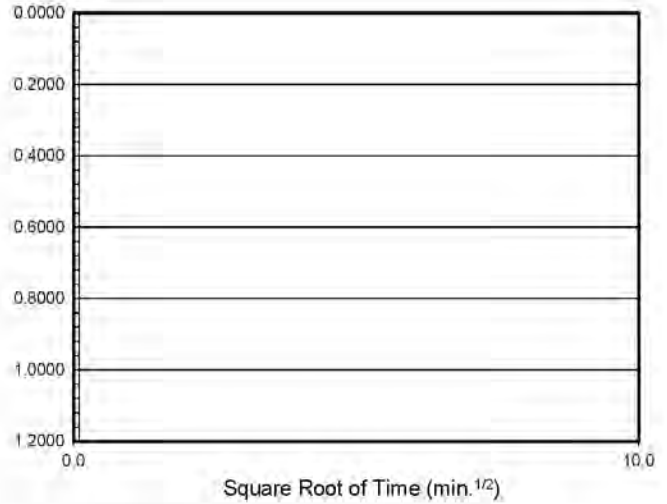
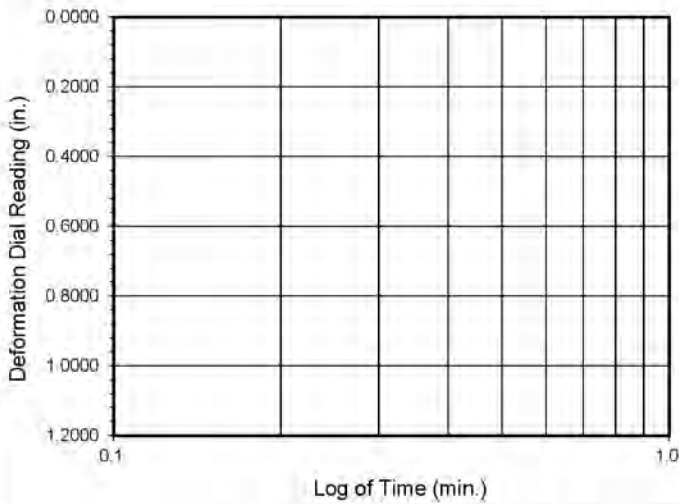


EXPANSION INDEX
(ASTM D 4829)

Project Number: 24105-01
Date: Dec-24

53 Via Jero Dr.

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-2	R-2	7.5	14.8	19.9	110.8	108.0	0.521	0.518	77	96

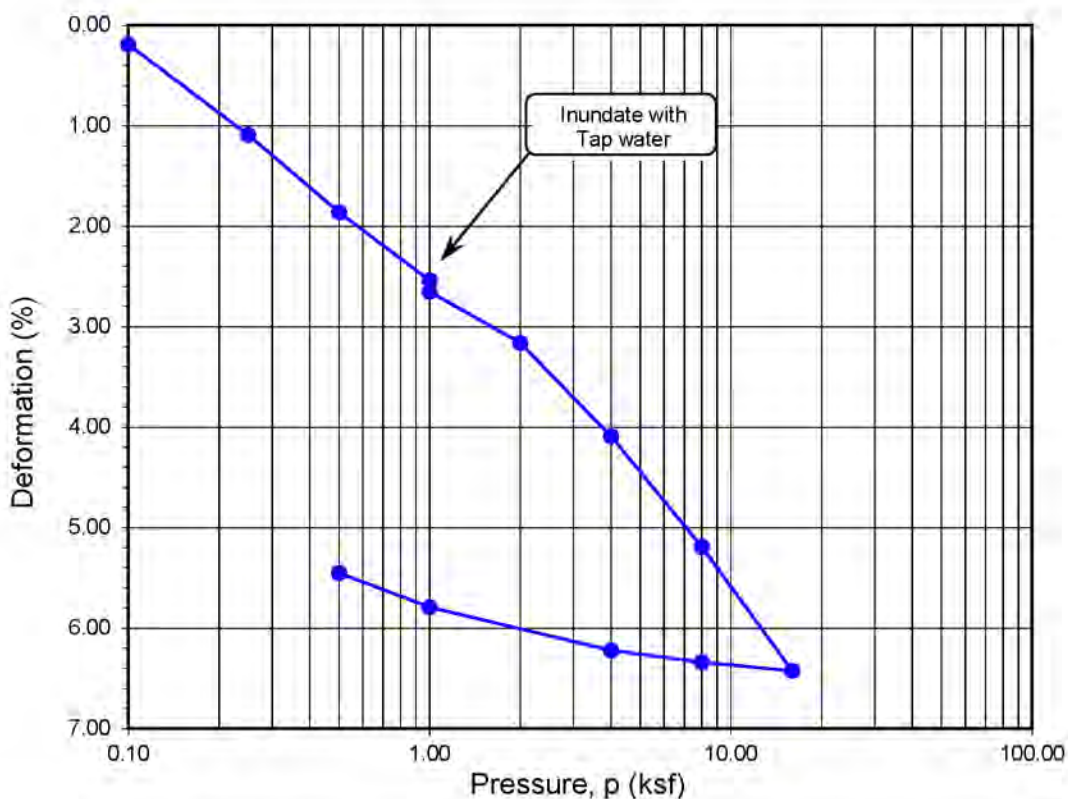
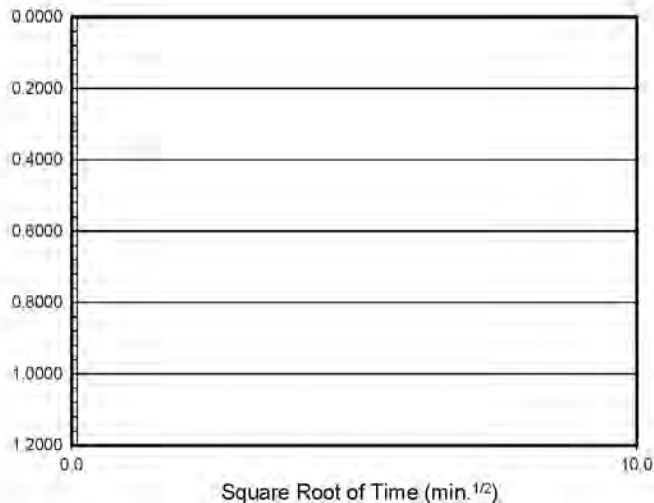
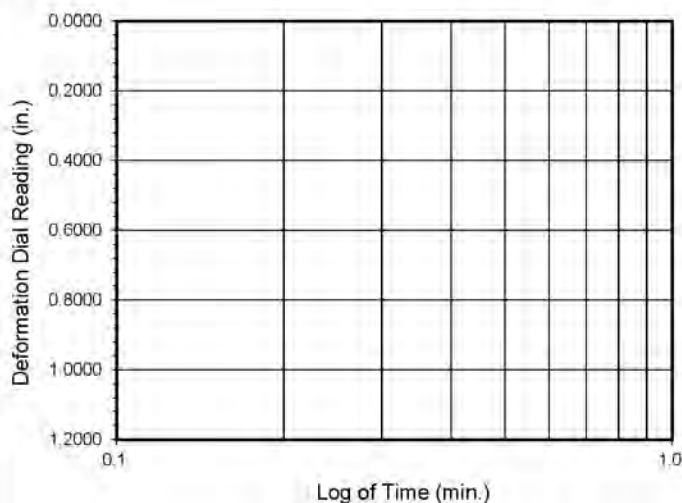
Soil Identification: Yellowish brown lean clay (CL)

ONE-DIMENSIONAL CONSOLIDATION
 PROPERTIES of SOILS
 ASTM D 2435

Project No.: 24105-01

ERM - Via Jero

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-3	R-1	5	14.4	13.9	116.8	122.5	0.443	0.364	88	100

Soil Identification: Dark brown silty clay (CL-ML)

ONE-DIMENSIONAL CONSOLIDATION
 PROPERTIES of SOILS
 ASTM D 2435

Project No.: 24105-01

ERM - Via Jero

DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: ERM - Via Jero
Project No.: 24105-01
Boring No.: HS-2
Sample No.: B-1
Soil Identification: Brown sandy silt/clay (CL/ML)

Tested By: G. Bathala
Checked By: J. Ward
Sample Type: Bulk
Depth (ft.): 0-5

Date: 12/02/24
Date: 12/10/24

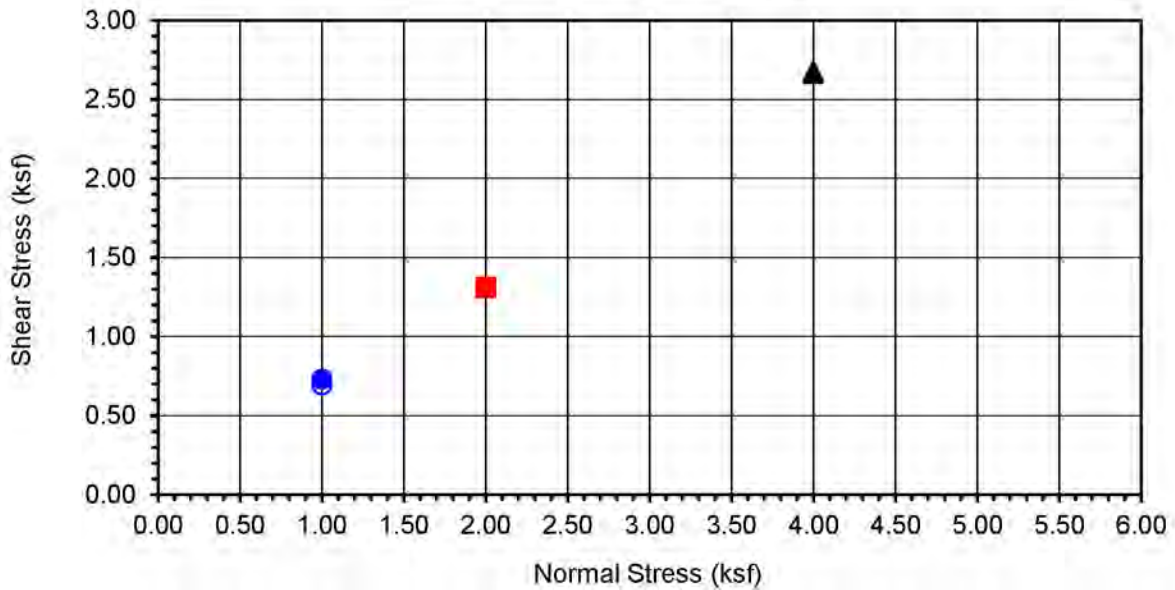
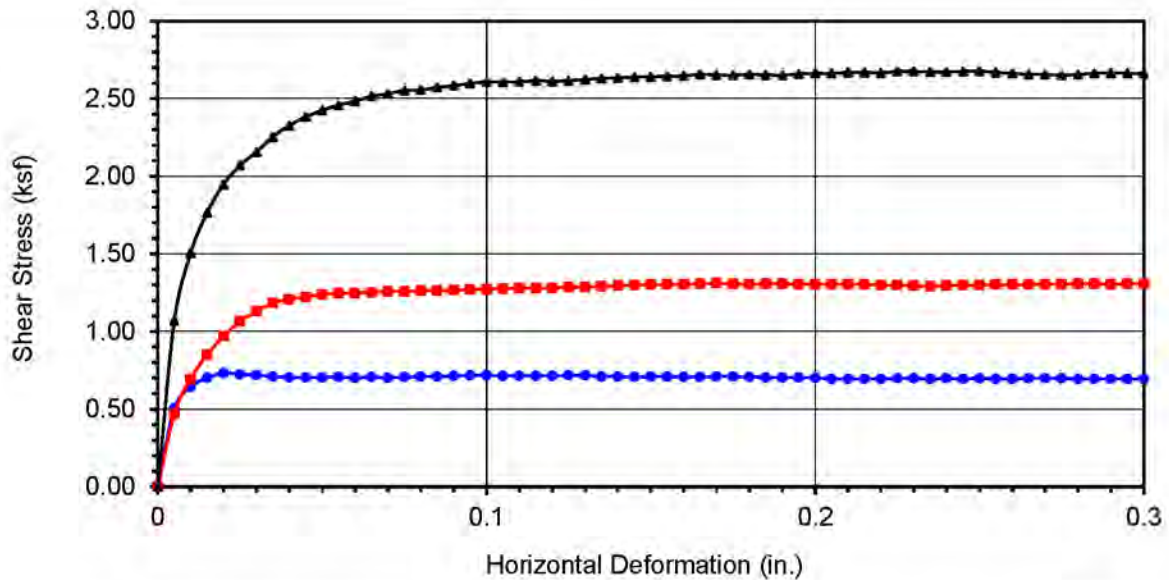
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	194.04	194.95	192.43
Weight of Ring(gm):	44.26	45.06	42.34

Before Shearing

Weight of Wet Sample+Cont.(gm):	182.00	182.00	182.00
Weight of Dry Sample+Cont.(gm):	170.02	170.02	170.02
Weight of Container(gm):	58.30	58.30	58.30
Vertical Rdg.(in): Initial	0.0000	0.2451	0.2795
Vertical Rdg.(in): Final	-0.0119	0.2585	0.3017

After Shearing

Weight of Wet Sample+Cont.(gm):	213.16	219.68	208.49
Weight of Dry Sample+Cont.(gm):	192.48	200.02	189.43
Weight of Container(gm):	58.10	66.84	55.15
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	HS-2
Sample No.	B-1
Depth (ft)	0-5
<u>Sample Type:</u>	
Bulk	
<u>Soil Identification:</u>	
Brown sandy silt/clay (CL/ML)	

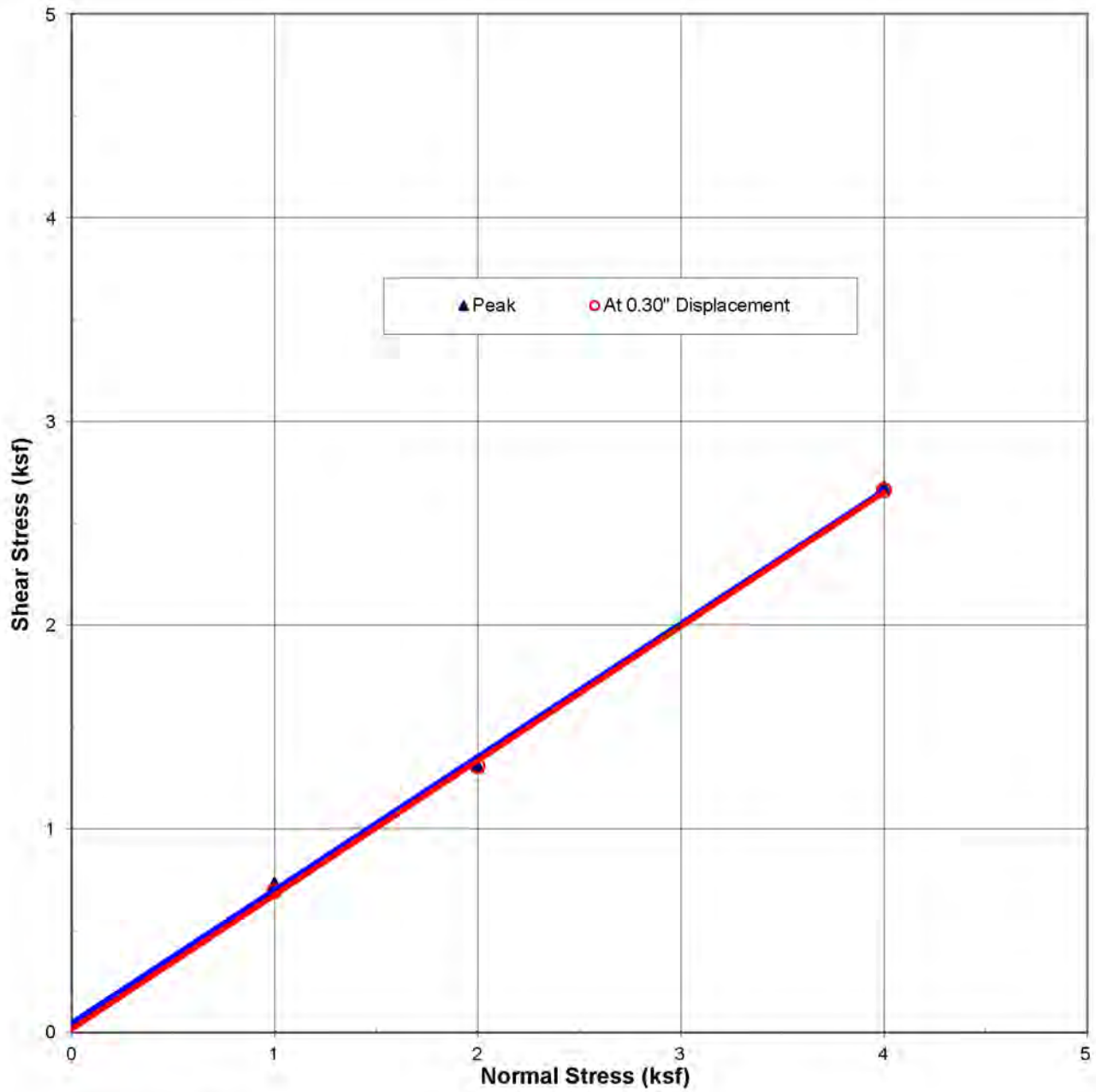
Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 0.732	■ 1.311	▲ 2.678
Shear Stress @ End of Test (ksf)	○ 0.695	□ 1.308	△ 2.663
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.72	10.72	10.72
Dry Density (pcf)	112.5	112.6	112.7
Saturation (%)	58.1	58.2	58.5
Soil Height Before Shearing (in.)	0.9881	0.9866	0.9778
Final Moisture Content (%)	15.4	14.8	14.2

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 24105-01

ERM - Via Jero

12-24



Tested Sample:
HS-2 at 0-5 ft

Peak:	At 0.30" Displacement:
33.2 Degrees	33.4 Degrees
0.05 ksf	0.02 ksf

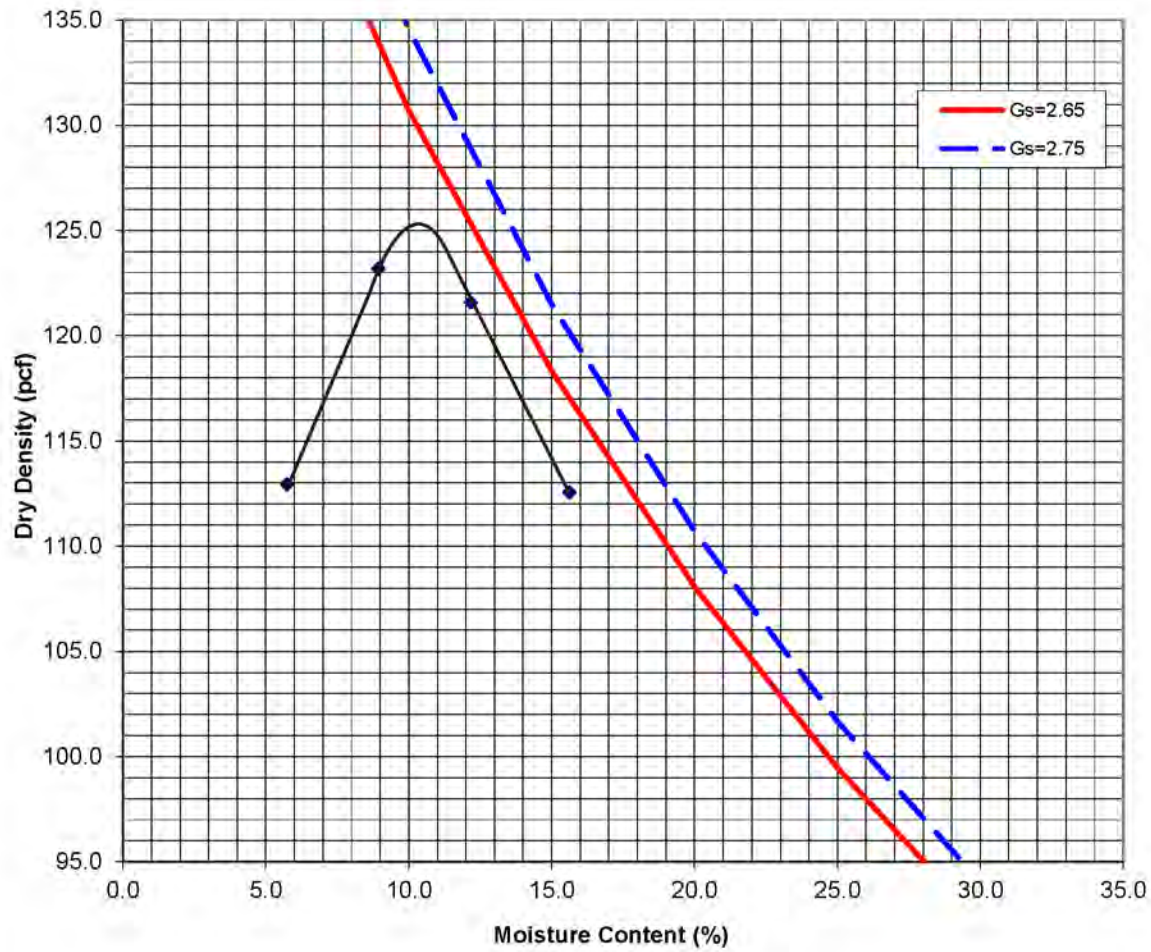
Remolded to 90% Relative Compaction



DIRECT SHEAR PLOT

Project Number: 24105-01
Date: Dec-24

ERM - Via Jero



Location:	Sample No.:	Depth (ft)	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2	B-1	0-5'	Brown sandy SILT/CLAY (CL/ML)	125.5	10.5



LABORATORY COMPACTION
(ASTM D 1557)

Project Number: 24105-01
Date: Dec-24

53 Via Jero Dr.

**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: ERM - Via Jero Tested By : KJ/GEB Date: 11/21/24
 Project No. : 24105-01 Checked By: J. Ward Date: 12/10/24

Boring No.	HS-1			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Yellowish brown CL			
Wet Weight of Soil + Container (g)	130.14			
Dry Weight of Soil + Container (g)	125.18			
Weight of Container (g)	77.75			
Moisture Content (%)	10.46			
Weight of Soaked Soil (g)	100.59			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	400			
Crucible No.	300			
Furnace Temperature (°C)	860			
Time In / Time Out	6:15/7:00			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	58.5132			
Wt. of Crucible (g)	58.5122			
Wt. of Residue (g) (A)	0.0010			
PPM of Sulfate (A) x 41150	41.15			
PPM of Sulfate, Dry Weight Basis	46			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO ₃ Soln. Used in Titration (C)	0.8			
PPM of Chloride (C -0.2) * 100 * 30 / B	120			
PPM of Chloride, Dry Wt. Basis	134			

pH TEST, DOT California Test 643

pH Value	7.46			
Temperature °C	20.8			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: ERM - Via Jero
 Project No. : 24105-01
 Boring No.: HS-1
 Sample No. : B-1

Tested By : G. Berdy Date: 11/22/24
 Checked By: J. Ward Date: 12/10/24
 Depth (ft.) : 0-5

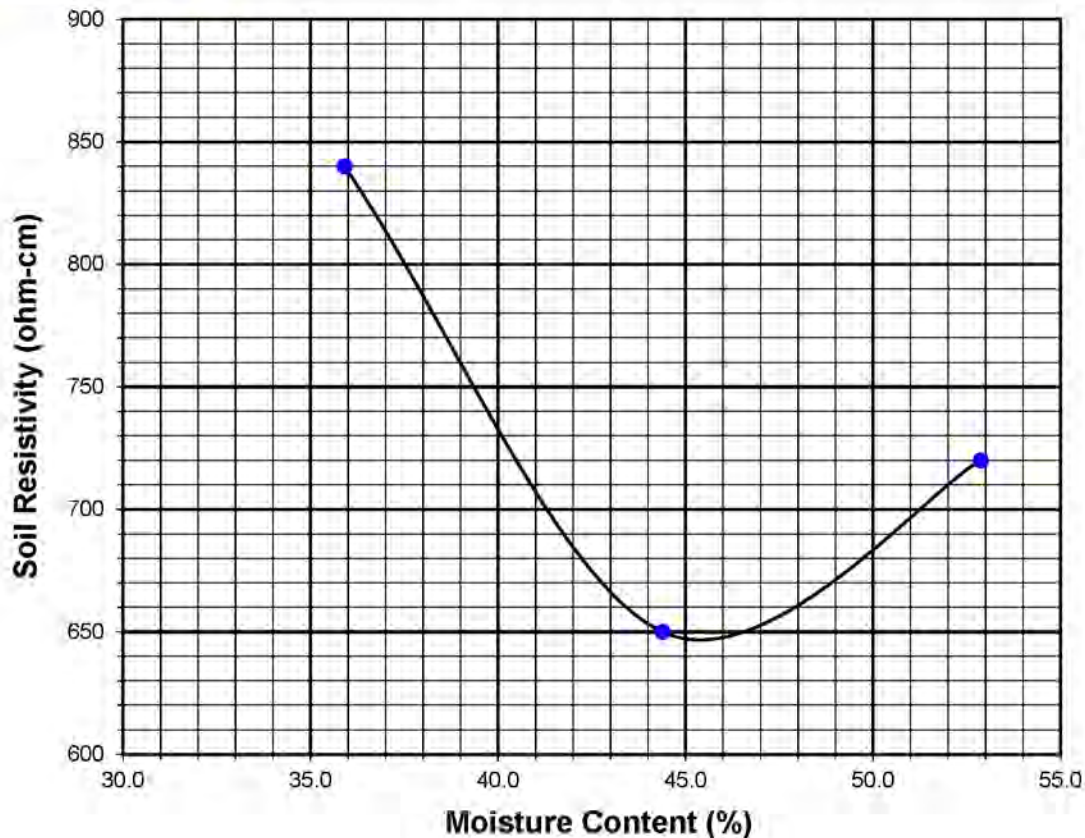
Soil Identification:* Yellowish brown CL

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	35.91	840	840
2	40	44.39	650	650
3	50	52.87	720	720
4				
5				

Moisture Content (%) (Mci)	10.46
Wet Wt. of Soil + Cont. (g)	130.14
Dry Wt. of Soil + Cont. (g)	125.18
Wt. of Container (g)	77.75
Container No.	
Initial Soil Wt. (g) (Wt)	130.21
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
645	45.4	46	134	7.46	20.8



**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: ERM - Via Jero Tested By : KJ/GEB Date: 11/21/24
 Project No. : 24105-01 Checked By: J. Ward Date: 12/10/24

Boring No.	HS-2			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Brown sandy CL/ML			
Wet Weight of Soil + Container (g)	113.45			
Dry Weight of Soil + Container (g)	108.65			
Weight of Container (g)	57.45			
Moisture Content (%)	9.37			
Weight of Soaked Soil (g)	100.65			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	401			
Crucible No.	304			
Furnace Temperature (°C)	860			
Time In / Time Out	6:15/7:00			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	65.4367			
Wt. of Crucible (g)	65.4352			
Wt. of Residue (g) (A)	0.0015			
PPM of Sulfate (A) x 41150	61.73			
PPM of Sulfate, Dry Weight Basis	68			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)				
ml of AgNO ₃ Soln. Used in Titration (C)				
PPM of Chloride (C -0.2) * 100 * 30 / B				
PPM of Chloride, Dry Wt. Basis	N/A			

pH TEST, DOT California Test 643

pH Value	N/A			
Temperature °C				

Appendix D
General Earthwork and Grading Specifications
for Grading

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

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

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

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

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

2.2 Processing

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

2.3 Over-excavation

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

2.4 Benching

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

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 **Import**

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

4.0 **Fill Placement and Compaction**

4.1 **Fill Layers**

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

4.2 **Fill Moisture Conditioning**

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

4.3 **Compaction of Fill**

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

4.4 **Compaction of Fill Slopes**

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

4.5 **Compaction Testing**

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 The Contractor shall follow all OSHA 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.