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Appendix 4.4A

Geotechnical Evaluation Report

LGC
Geotechnical, Inc.

April 4, 2024 Project No. 22011-01

Mr. Paul McMillan
Engie North America
1360 Post Oak Boulevard
Houston, Texas 77056

Subject: ***Geotechnical Evaluation Report, Compass Battery Energy Storage System, San Juan Capistrano, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical evaluation for the proposed development of the Compass Battery Energy Storage site in San Juan Capistrano, 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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1.0 INTRODUCTION

LGC Geotechnical has performed a geotechnical evaluation for the proposed Compass Battery Energy Storage site in San Juan Capistrano, California (Figure 1). Our study included evaluation and analysis of the proposed grading plan (Sargent & Lundy, 2024) based on data gathered in previous geotechnical evaluations for the site (Appendix A) and data gathered from our recent field evaluation. 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 15.5 acres of land. The project will include grading for and construction of an SDG&E switching station, a battery storage yard, associated access roads and improvements. We understand that the battery storage yard will consist of an array of 8 foot by 20-foot storage containers with a total load of 36 kips for each container. Site grading is anticipated to include up to approximately 9 feet of cut on the west side of the site and minor fill placement on the east side to create a relatively flat site, sloping from west to east from elevations of approximately 215 feet above mean sea level to elevations of approximately 210 feet above mean sea level. The maximum 9-foot-cut in the western portion of the site will include grading of a maximum 3:1 (horizontal to vertical) inclination cut slope.

The proposed development area is located on a portion of the relatively flat Oso Creek abandoned alluvial floodplain and the gently sloping lower portion of a west ascending hillside (Figure 1). The hillside continues to ascend and steepen to the west side of the site, with total height on the order of approximately 420 feet. The incised Oso Creek channel is located east of the development area with channel wall heights of up to approximately 40 feet. The site is located south of Saddleback Church Rancho Capistrano. The area of proposed development has previously been used for agricultural use and is essentially undeveloped. A community garden and associated structures is in the central portion of the site.

The preliminary recommendations given in this report are based upon the provided preliminary layout and grading 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 and building layout when they become available, in order to either confirm or modify the recommendations provided herein.

1.2 Background

Several geotechnical evaluations have been performed on or in the immediate vicinity of the subject site. As part of our study, we have reviewed the pertinent and available reports (Appendix A) and considered the findings, conclusions and recommendations provided. Where appropriate, the subsurface logs and laboratory test results from these previous geotechnical reports are included in Appendix B and C of this report. The approximate locations of the excavations from these studies are included on our Geotechnical Map (Sheet 1).

NMG Geotechnical (2001) performed a geotechnical evaluation of the site in 2001, consisting of excavation, sampling and logging of five small diameter borings, three large diameter borings, and eleven CPT Soundings.

Lowney and Associates performed an evaluation in 2003 consisting of excavation, sampling and logging of one small diameter hollow stem boring and six large diameter borings.

Leighton and Associates, Inc. (2009) performed a geotechnical evaluation in 2009 consisting excavation of four large diameter borings.

Terracon performed a geotechnical evaluation of the site in 2021 consisting of 30 small-diameter borings (Terracon, 2021). The borings (B-1 through 17, B-19 through B-21, B-24 and B-25, and P-1 through P-8) ranged in depth from approximately 5 to 101.5 feet, with borings P-1 through P-8 utilized for field percolation testing.

Geosyntec Consultants, Inc. performed a geomorphic evaluation for the proposed development in 2021 (Geosyntec, 2021). The evaluation included a geomorphic study and review of reports by others. No subsurface evaluation was performed.

A hydrology study was performed for the proposed BESS development, for the portion of the Oso Creek channel adjacent to the site by Chang Consultants (Chang Consultants, 2024).

A soil corrosivity study was performed for the proposed BESS development by HDR (HDR, 2023).

A thermal resistivity analysis was performed for the proposed BESS development by Geotherm USA (Geotherm, 2023).

1.3 <u>Supplemental Geotechnical Evaluation</u>

A supplemental geotechnical evaluation was recently performed by LGC Geotechnical. The evaluation consisted of the geologic mapping of the site, excavation of five large-diameter bucket auger borings, four small-diameter borings and seventeen Cone Penetration Test (CPT) soundings to further evaluate onsite geotechnical conditions. The results of our geologic mapping and the locations of our excavations are depicted on the Geotechnical Map (Sheet 1). The Grading Plan by Sargent and Lundy (2024) was utilized as a base map for the Geotechnical Map (Sheet 1), and for use in constructing Geotechnical Cross-Sections (Sheets 2 - 4). It should be noted that Geotechnical Cross-Sections A-A' through D-D,' G-G' and H-H' are presented herein. Geotechnical Cross-Sections E-E' & F-F' were constructed for a previous, different version of the proposed development plan are no longer relevant and therefore not included herein.

Five large-diameter, bucket-auger borings (B-1 through B-5) were excavated on the site by Roy Brother's Drilling under subcontract to LGC Geotechnical (Sheet 1). The maximum depth of the bucket-auger borings was approximately 98 feet below existing grade. The bucket-auger borings were excavated to evaluate the geologic structure of the landslides on the west side of the site and underlying bedrock materials, and to obtain samples for laboratory testing. The borings were placed at strategic locations in order to supplement the previous field evaluations. Samples were obtained at select and representative depths for laboratory testing. The large-diameter boreholes were surface logged during excavation and downhole logged by an engineering geologist in order to obtain structural geologic information. The borings were subsequently backfilled with a mixture of bentonite chips and drill cuttings (per permit requirements) and tamped for densification to the original ground surface. Some settlement of the borings should be expected. The borings may need to be topped off accordingly.

The four small-diameter (6 & 8-inch), hollow-stem-auger borings (HS-1 through HS-4) were excavated to depths ranging from approximately 90 to 100 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 CPT soundings (CPT-1 through CPT-17) were pushed to depths ranging from approximately 63 to 100 feet below existing grade. The CPT Soundings were pushed to 100 feet or to practical refusal. The CPT soundings were pushed using an electronic cone penetrometer in general accordance with the current ASTM standards (ASTM D5778 and ASTM D3441) using a 30-ton rig. The CPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The interior of the cone penetrometer is instrumented with strain gauges that allow the simultaneous measurement of cone tip and friction sleeve resistance during penetration. The cone penetration assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.8 inches per second while the cone tip resistance and sleeve friction resistance are recorded at approximately every 2 inches and stored in digital form. Seismic cone (shear wave velocity) readings were performed in select soundings. Shear wave velocity readings indicated an average shear wave velocity of approximately 700 feet per second (ft/sec), generally corresponding to Site Class D per Chapter 20 of ASCE 7-16.

The boring and CPT Logs are presented in Appendix B and approximate locations are depicted on the Geotechnical Map, Sheet 1.

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, fines content, Atterberg Limits, fines content, torsional ring shear, direct shear, consolidation, expansion index, laboratory compaction and corrosion (sulfate, chloride, pH and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 81 pounds per cubic foot (pcf) to 119 pcf, with an average of 98 pcf. Field moisture contents ranged from approximately 7 percent to 42 percent, with an average of approximately 24 percent.

- Atterberg Limit (liquid limit and plastic limit) tests were performed on site soils. Results indicated Plasticity Index values ranging from approximately 8 to 36.
- Atterberg Limit tests were performed on grab samples of site clay beds. Results indicated Liquid Limit values ranging from approximately 59 to 72.
- Ten fines content tests indicated fines content (percent passing No. 200 sieve) ranging from approximately 44 to 94 percent. Based on the Unified Soils Classification System (USCS), eight of the tested samples are classified as “fine-grained.”
- Two torsional residual ring shear tests were performed on grab samples of site clay landslide rupture materials. The plots are provided in Appendix C.
- Direct shear tests were performed on select samples. The plots are provided in Appendix C.
- Consolidation tests were performed. The deformation versus vertical stress plots are provided in Appendix C.
- Expansion Index (EI) tests indicated EI values ranging from 42 to 73, corresponding to “Low” to “Medium” expansion potential.
- Laboratory compaction tests resulted in maximum dry density values ranging from 112.0 pcf to 118.0 pcf with optimum moisture contents ranging from 11.5 to 13.0 percent.
- Corrosion testing indicated soluble sulfate contents of approximately 0.02 percent or less, chloride contents ranging from 64 to 176 parts per million (ppm), pH values ranging from 7.7 to 8.2 and 7.7, and minimum resistivity values ranging from 1,250 to 1,250 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 on the southwestern border of the Peninsular Ranges at the southeastern-most portion of the Los Angeles Basin. Specifically, the site lies on the central portion of the sedimentary basin known as the Capistrano Embayment, an early Cenozoic seaway, which trended northerly between the Peninsular Ranges and a hypothetical Catalina uplift off the Southern California coast. Locally, the Capistrano Embayment refers to the flat-bottomed structural trough formed by the downward displacement along the west side of the Cristianitos Fault and down-warping along the east side of the San Joaquin Hills. The embayment was subsequently in-filled with marine siltstone and clayey siltstone of the late Miocene to early Pliocene (approximately 5 to 15 million years old) Capistrano Formation. This sedimentary unit, in excess of 3,000 feet thick near the center of the embayment, was uplifted, folded, and eroded in Pliocene and post-Pliocene times (approximately 2 to 3 million years ago) producing the low, rolling ridges observed today. 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 artificial fill, topsoil/colluvium, Quaternary alluvial deposits, Quaternary river terrace deposits, Quaternary older alluvial deposits, Quaternary landslide deposits, and Tertiary Capistrano Formation. 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 (Sheet 1). The topographic base utilized for the Geotechnical Map and Geotechnical Cross-Sections was provided by Sargent & Lundy (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.

Based on our review of the State of California Seismic Hazard Zone Map for the San Juan Capistrano 7.5 Minute Quadrangle (CGS, 2001b), most of the site is located within a potential liquefaction zone and the western border of the site is located within a zone of potential earthquake induced landslides. These maps were prepared by the State to raise awareness of the potential for such hazards and to prompt appropriate investigation to evaluate these potentials on a site-by-site basis.

2.2.1 Artificial Fill by Others (Map Symbol - afo)

Older fill material was observed within localized areas for support of the railroad embankment on the east side of Oso Creek, and in the form of rip-rap armoring along the east bank of Oso Creek. Rip-rap armor on steep creek channel walls consists of variable materials (cobbles to boulders) and thickness is unknown. These materials are not anticipated to be encountered for the proposed site development.

2.2.2 Topsoil/Colluvium (Not Mapped)

A relatively thin veneer of topsoil/colluvium mantles the surface of the majority of the site. The material typically consists of brown to dark brown, dry to slightly moist, medium stiff, sandy silt and clay. These soils are typically porous and contain scattered roots and organics. Topsoil/colluvium is considered potentially compressible and will need to be removed to competent material in areas of proposed development. Topsoil and colluvium were not mapped on the site due to their relatively thin nature and variable lateral extent.

2.2.3 Quaternary Alluvial Deposits (Map Symbol – Qal)

The alluvial deposits encountered during our investigation included both recent alluvial deposits within a portion of the Oso Creek stream channel (not sampled, unknown thickness), and older alluvial deposit that mantles the majority of the site development area. The alluvium that mantles the majority of the site was encountered during the our site evaluation and can be directly observed to form the near-vertical upper portion of the exposed cliff that forms the west channel wall of Oso Creek within the subject area.

As mapped along the upper bank of the stream channel and observed in borings, the material is medium thick bedded, sub horizontal, light gray and dark brown layers of silty sand to sandy silt, and clayey sand to sandy clay. In general, the material was found to be loose to medium dense/stiff, dry to moist, and slightly to moderately indurated, with scattered organics and rootlets.

2.2.4 Quaternary Landslide (Map Symbol – Qls)

The landslide material encountered during our site evaluation consisted of highly variable materials that showed different characteristics between the western hillside portion of the site and the eastern Oso Creek portion of the site.

Landslide material on the west side of the site included thick sections of layered slopewash and organic-rich debris flows from the hillside located west of the site over sheared bedrock block material (similar to the Capistrano Formation parent material). In general, the landslide material consisted of variable colors of sandy silt, silt, clayey silt, siltstone, clayey siltstone, and sandy siltstone, stiff to very stiff, slightly moist to very moist. Landslides along the western side of the site where downhole-logged borings were performed were observed to have a basal rupture surface of very thin, soft, clay over Capistrano Formation siltstone bedrock.

Along the east side of the site, landslide materials were observed to have run out as far as the current alignment of Oso Creek and are interfingered with alluvial deposits in some areas. The landslide material encountered in small-diameter borings and exposed by the more recent Oso Creek channel erosion is highly weathered, variable silty clay to clayey silt, and clay with trace of fine sand. The material is olive brown, gray, or brown with orange oxidation staining, medium stiff to stiff, and moist to wet. Where exposed along the Oso Creek channel, some of the material was observed to have recently moved toward the stream bed as localized rotational failure, labeled as "active." Thick vegetation obscures the exposure of landslide material along the creek channel wall at the southern portion of the site.

2.2.5 *Quaternary Older Alluvial Deposits (Map Symbol – Qalo)*

The older alluvial deposits encountered on site during our investigation were deeply buried by run-out landslide deposits that were in turn capped by relatively young alluvium. The older alluvial deposits were observed to deepen to the south and are interpreted to be syn-depositional with the alluvial deposits of the relatively broad Trabuco Creek that Oso Creek joins at the southern portion of the subject site. The older alluvium was encountered below about 45 feet in depth, within the western half of the development pad (refer to LGC Geotechnical Boring HS-2). In general, the material consisted of sandy clay, sand, and clayey to silty sand, medium stiff/medium dense to very dense, wet, with color ranging from light brown to gray.

2.2.6 *Quaternary River Terrace Deposits of Arroyo Trabuco (Map Symbol – Qtr)*

Described as river terrace deposits of San Juan and Trabuco Creeks on the regional geologic map for San Juan Capistrano (Morton, 1974), this material is mapped just outside the eastern border of the site, below the Interstate 5 Freeway. This unit was encountered during the field mapping portion of our investigation. The river terrace deposit was observed to consist of dense, fine to coarse sand, with an oxidized orange color.

2.2.7 *Tertiary Capistrano Formation (Map Symbol – Tc)*

Tertiary Capistrano Formation bedrock underlies the entire site at depth. This material generally consists of massive to thickly bedded siltstone, very fine sandy siltstone, clayey siltstone, and few thin sand and clay beds and concretionary nodules. Within the upper oxidized (weathered) portion, the material is typically light gray to brown in color and is commonly has gypsum and is iron-stained along joints and fractures. The unoxidized (fresh) portion of the Capistrano Formation is dark gray, very stiff to hard, bedrock.

2.3 *Geologic Structure and Landslides*

The underlying bedrock formation within this geologically complex area is the Capistrano Formation, a generally massive siltstone with few very thin clay beds within the stratigraphy of the formation, which are locally sub horizonal to slightly westward-dipping. Bedding angles within the bedrock as observed in our recent large-diameter borings and recorded by others has a general range between 2 degrees and 7 degrees to the southwest. Uplift and weathering of the bedrock formation commonly produces block-type landslide failures that fail along the very thin clay bedding, creating block-glide type landslides with steep backscarps and gently dipping basal rupture surfaces (Terres, 1992). The hillside to the west of the proposed development area has previously been identified to have numerous landslides as depicted on the regional geologic map and reported in the referenced reports (NMG, 2003 & Leighton, 2009). The presence of landslide on-site has been confirmed with the recent investigation.

As observed in the recent large-diameter borings, landslides have both failed along clay bed(s) as a block-glide and accumulated over time as a series of slopewash pulses and organic-rich debris-flows (up to 65 feet deep in LGC Geotechnical boring B-3 and up to 80 feet deep in LGC Geotechnical boring B-2). Geologic structure of the on-site portion of landslides ranges from lacking structure entirely as gently downslope-dipping slopewash to highly sheared displaced bedrock material. The basal rupture clays of the on-site portions of the landslides were observed to be approximately ¼-inch thick or less, and typically oriented parallel to bedding as observed in numerous borings. A continuous clay bed as observed in LGC Geotechnical's Boring B-3 at 67.5 feet, B-4 at 55 feet, and in borings by others including Leighton's LAB-2 at 58.5 feet and Lowney's LB-15 at 51 feet in depth. The clay bed has been identified at elevations between approximately 180 and 190 feet above sea level along the western property boundary in the toe of slope area.

Based on downhole-logged borings by others excavated on the ascending offsite hillside west of the site, an upper-elevation clay bed observed in NMG Boring B-2 at 21 feet in depth, forms a landslide rupture surface for an offsite landslide. The material below the landslide surface, to a total depth of 203 feet below existing grades, generally lacked landslide features and was interpreted to be bedrock. It is our understanding that the depth was achieved in order to determine whether the western hillside slope could be one very large landslide that may have failed below the elevation of Oso Creek. Instead, the landslide has been modeled as stepped failures. A landslide that occurred on the upper clay is perched in the hillside above the subject site, while a series of landslides failed on the lower-elevation clay bed within the subject site, as discussed in the above paragraph. The lower landslides appear to crosscut the toe of landslides that failed along the upper clay bed. Refer to Cross Sections A-A' and B-B.'

Three different geologic units are modeled within the area underlying the eastern half of the development pad near Oso Creek including older alluvium at depth that was subsequently covered by a chaotic layer of landslide deposits, and both units capped by sub horizontal layers of younger alluvium. Significant downcutting to the current location of Oso Creek bed has occurred in the last 50 years (Geosynetec, 2021) Rotational failures along the west Oso Creek channel walls were mapped and observed to be heavily vegetated, well-defined slumps that likely were saturated and undercut at the toe, by the flowing water of Oso Creek.

No faults are known to transect the site. The closest significant fault to the site is the active offshore portion of the Newport-Inglewood Fault Zone, located approximately 6 miles southwest of the site.

2.4 <u>Groundwater</u>

Perched groundwater seepage was encountered in two of the large-diameter borings drilled by LGC Geotechnical at the northern (B-2) and southern (B-4) portions of the subject site. The seepage was encountered in borings B-2 and B-4 at depths of 63 and 36 feet, respectively. Also, minor amounts of groundwater seepage were observed at a depth of 63 feet in B-1.

Groundwater was encountered in LGC Geotechnical borings HS-1 through HS-4 at depths ranging from approximately 31 feet to 41 feet below existing grade (approximate elevations ranging from 167 feet to 179 feet). Previously, groundwater was encountered in four of the thirty borings ranging in depth from approximately 47 feet to 70 feet below existing grade (approximate elevations ranging from 139 feet to 170 feet) (Terracon, 2021). Historic high groundwater is estimated at 5 feet below grade within the development area (CGS, 2001a).

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.

2.5 <u>Oso Creek Migration</u>

Oso Creek is located on the eastern and a portion of the southern sides of the development area. The Oso Creek channel is deeply incised, with channel wall heights of approximately 40 feet. The channel is migrating and widening towards the proposed development area through both creek erosion and bank collapses, in the form of rotational landslides in the alluvial terrace and older landslide material. Creek migration and incision has been controlled north of the site by construction of a concrete culvert, which flows into a riprap-lined portion of the channel. A riprap embankment has been constructed along much of the eastern bank of the channel across from the development area to mitigate channel migration toward the railroad tracks to the east. According to our review of the geomorphic study for the development, performed by Geosyntec (2021) and a hydrology study by Chang Consultants (2024), the western bank of the creek adjacent to the site is unprotected and is prone to additional erosion, landslides, and migration towards the east side of the subject site. Modeling of the creek for flood events over a projected 100-year period was performed as part of the hydrologic study for the site (Chang Consultants, 2024). Modeling of profiles of the channel adjacent to the site over the 100 years, suggest on the order of 6 to 27 vertical feet of scour may occur (Chang Consultants, 2024). Mitigation against and/or setback from creek erosion and channel migration into the development will be needed for construction of the development as currently proposed.

2.6 <u>Seismicity and Faulting</u>

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. The major active nearby faults that could produce these secondary effects include the onshore and offshore segments of the Newport-Inglewood Fault Zone, located approximately 6 miles southwest of the site. The presence of a blind thrust fault has been interpolated from limited data, to exist at a depth of approximately eight miles below the uplifted local hills; however, the San Joaquin Hills Blind Thrust Fault does not have a known location of surface rupture. 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 site is located within a State of California Seismic Hazard Zone (CDMG, 2002a) for liquefaction potential. The vast majority of the alluvial soils tested are cohesive and not considered to be susceptible to liquefaction based on their Plasticity Index (Bray & Sancio, 2006). However, the data obtained from our field evaluation indicates that the site contains isolated typically relatively thin sandy layers susceptible to liquefaction in the upper 50 feet. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008). Liquefaction analysis was performed based on the seismic criteria (PGA_M) of the 2022 California Building Code (CBC) and an estimated high groundwater depth of 5 feet below existing grade. Estimated total and differential seismic settlement due to liquefaction potential is provided in Table 1 below. Liquefaction calculations are provided in Appendix D.

TABLE 1

Estimated Settlement Due to Liquefaction Potential

Approximate Total Seismic Settlement	**Differential Seismic Settlement**
1-inch	1/2 inch over 50 feet

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.

As discussed above, site alluvial soils are primarily fine-grained and not considered susceptible to liquefaction. Site sandy soils susceptible to liquefaction are generally relatively thin and non-continuous. The potential for lateral spreading is considered 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 Dana Point/San Juan Capistrano Quadrangle (CEMA, 2009).

2.7 Seismic Design Parameters for Site Development

The site contains isolated soils that are susceptible to liquefaction (refer to above Section "Liquefaction and Dynamic Settlement"). The 2022 CBC requires Site Class F designation for sites underlain by potentially liquefiable soils; however, we conclude that Site Class D designation is more appropriate in consideration that potentially liquefiable layers are relatively thin and isolated and negligible seismic settlement is anticipated. It should be noted that the seismic parameters provided herein are not applicable for any structure having a fundamental period of vibration greater than 0.5 second.

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 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 33.534456 degrees north and longitude -117.678844 degrees west were utilized in our analyses. Please note that these coordinates are considered representative of the site for preliminary planning purposes only, however their applicability must be verified with respect to a desired specific location within the site. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 2 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. 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 2

Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**	**Notes/Exceptions**
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.202g	From SEAOC, 2024
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.431g	From SEAOC, 2024
F_a (per Table 1613.2.3(1))	1.019	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))	1.869	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.225g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	0.806g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.817g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.537g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.930	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.933	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16. Site Class F modified to Site Class D, seismic parameters only applicable for structure period ≤ 0.5 second, refer to discussion above.		

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.563g (SEAOC, 2024). The design PGA may be taken as 0.375g (2/3 of PGA_M). A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.68 at a distance of 12.48 km from the site would contribute the most to this ground motion.

2.8 Soil Shear Strength Parameters

The soil shear strength parameters utilized in our slope stability analysis are based on laboratory testing, published shear strength data (CGS, 2002) and engineering judgment. The soil shear strength for the landslide rupture plane is based on the results of torsional ring shear testing of clay rupture surface materials obtained during downhole logging from our recent field evaluation. In addition, numerous Atterberg Limits tests were performed on clay rupture samples to verify similar Liquid Limit characteristics. Previously a ring shear test was performed on a clay rupture surface (NMG, 2001). Where applicable, soil shear strengths were increased for seismic loading conditions. Table 3 below summarizes the static shear strength parameters utilized in our analysis. Laboratory test results are provided in Appendix C.

TABLE 3

Summary of Static Shear Strength Parameters

Description	Soil Unit Weight (pcf)	Friction Angle ϕ (degrees)	Cohesion (psf)
Capistrano Formation (Tc)	120	26	300
Landslide Material/Backscarp (Western Slope)	120	22	250
Basal Rupture Clay Surface	120	14	0
Clay Bed	120	18	0
Compacted Fill	120	26	300
Alluvium	120	24	200
Landslide Material/Alluvium (Oso Creek Channel Wall)	120	24	300

2.10 Slope Stability Analysis

Slope stability analyses were performed based on the proposed design profile. Slope stability analysis was performed using the computer program GEOSTASE, version 4.30.31 (Gregory Geotechnical Software, 2019). Slope stability analysis was performed for static and seismic loading conditions. For seismic loading conditions a horizontal seismic coefficient (Kh) of 0.15 was used. For seismic loading, it is our understanding that 12 inches of seismic displacement is acceptable for the proposed development.

In order to determine an appropriate setback distance from the channel, we have performed slope stability analysis on a hypothetical model of what the western bank of the Oso Creek channel may look like 100 years from now (see Geotechnical Cross-Section C-C'). The model assumes 30-vertical-feet of scour of the Oso Creek channel based on the most extreme scour estimated in the project hydrology report (Chang Consultants, 2024). Similarly, 50-horizontal feet of migration has been assumed for the west bank of the channel eroding into the site. The inclination of the west bank of the Oso Creek channel has been modeled at a 1.5:1 (horizontal to vertical) inclination based on the approximate average inclination of the bank today and anticipated in the modeling performed in the project hydrology report (Chang Consultants, 2024). Based on this cross-sectional model (see Geotechnical Cross-Section C-C'), slope stability analysis was performed for potential rotational surfaces of the hypothetical model of the channel bank.

Provided the development is appropriately setback from the hillside on the west and creek to the east, as recommended herein our analysis indicates slope stability should not be a concern for the proposed development area. Slope stability analysis and a table summarizing the results are provided in Appendix E.

Surficial slope stability analysis was performed for the proposed 3:1 fill slopes assuming a zone of saturation of 4-feet-deep parallel to the slope surface. Analysis indicated a surficial factor of safety greater than 1.5. Refer to Appendix E.

2.11 ***Rippability***

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

2.12 ***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 F.

2.13 ***Expansive Soil Characteristics***

Previous Expansion Index (EI) testing indicated EI values of 39 and 64, corresponding to "Low" and "Medium" expansion potential (Terracon, 2021). Expansion Index (EI) testing from the current evaluation indicated EI values of 42 and 73, corresponding to "Low" and "Medium" expansion potential. Based on lab testing results and our experience with the Capistrano Formation site materials should be anticipated to have "Medium" to "Very High" potential for expansion. Conformational expansion potential testing should be performed at the completion of site grading based on the finish graded conditions.

2.14 ***Corrosion Potential***

Corrosion testing indicated soluble sulfate contents of approximately 0.02 percent or less, chloride contents ranging from 64 to 176 ppm, pH values ranging from 7.7 to 8.2 and 7.7, and minimum resistivity values ranging from 1,250 to 1,250 ohm-centimeter. Previous corrosion testing indicated soluble sulfate contents ranging from approximately less than 0.01 percent to 0.19 percent, chloride contents ranging from not detected to 248 ppm, pH values ranging from 7.0 to 8.1, and minimum resistivity values ranging from 365 ohm-cm to 2,215 ohm-cm (Leighton, 2009 & Terracon, 2021). Site soils are considered corrosive based on Caltrans guidelines (Caltrans, 2021).

A soil corrosivity study was performed by HDR (HDR, 2023). The purpose was to determine the electrical resistivity of the soil for grounding design, and to determine whether the soils are likely to have deleterious impacts on underground piping and concrete structures.

2.15 *Field Percolation Testing*

Previously eight field percolation tests were performed (Terracon, 2021). Calculated infiltration rates for the three tests in the upper 5 feet ranged from 0.41 to 0.97 inches per hour. Calculated infiltration rates for the 5 tests performed from 5 to 10 feet below existing grade ranged from 0.03 (essentially zero infiltration) to 0.18 inches per hour.

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 topsoil/colluvium, Quaternary alluvial deposits, Quaternary older alluvial deposits, Quaternary landslide deposits, and Tertiary Capistrano 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.
- A large ancient landslide complex underlies the western portion of the site and the ascending hillside to the west of the site.
- The landslide complex consists of failed material derived from the Tertiary-age Capistrano Formation.
- The landslide rupture surface(s) were found to primarily consist of plastic clay, up to approximately ¼-inch-thick.
- Modeling of the landslides indicates that the landslide complex on the west side of the site is a series of block-type failures.
- Slope stability analysis indicates that the large landslide complex is relatively stable with respect to the proposed development. The proposed development should be planned so that it does not cut into the landslide complex, potentially reducing its stability. Slope stability analyses of the generated geotechnical cross-sections through the landslide complex, were performed to determine a sufficient setback distance for the proposed development, which would not detrimentally impact the stability of the landslide complex. The setback line is depicted on the project on the west side of the development area on Geotechnical Map. It should be noted that the preparation of the proposed grading plan and the determination of the setback line, was an iterative process between LGC Geotechnical and the project civil engineer. The slope stability analysis presented herein and setback line determined are specific to and consideration with the currently proposed grading plan. Should changes to the grading plan be proposed, LGC Geotechnical must review the revisions, to confirm the site stability will be maintained.
- The western bank of Oso Creek adjacent to the site is unprotected and is prone to erosion, landslides and migration towards the east side of the development area. Mitigation against creek erosion and channel migration into the development can be achieved by setting back from the creek per the recommendations provided herein. In order to determine an appropriate setback distance from the channel, we have performed slope stability analysis on a hypothetical model of the western bank of the Oso Creek channel after 100 years of projected additional erosion. Based on this model, a setback line has been established west of the Oso Creek channel where the site will have at least a 1.5 static factor of safety for potential rotational surfaces of the hypothetical model of the bluff. Structural improvements should not be planned east of the setback line.

- Groundwater is not expected to significantly impact the proposed grading and development of the subject site. Deeper excavation should anticipate encountering groundwater at depth. Groundwater has been encountered at the subject site during the previous and subject evaluation as shallow as 36 feet below the ground surface. Historic high groundwater is estimated at 5 feet below grade within the development area (CGS, 2001a). Shallower localized perched groundwater may also be encountered.
- Based on our review of the State of California Seismic Hazard Zone Map for the San Juan Capistrano 7.5 Minute Quadrangle (CGS, 2001b), most of the site is located within a potential liquefaction zone. Based on lab testing, the majority of site alluvial soils are cohesive and not considered susceptible to liquefaction. Subsurface data indicates that isolated sandy layers are susceptible to liquefaction and liquefaction-induced settlement. Our analysis indicates approximately 1-inch of seismically-induced settlement may occur at the site during a significant earthquake. Differential dynamic settlement may be taken as 1/2 inch over a horizontal span of 50 feet.
- Based on our review of the State of California Seismic Hazard Zone Map for the San Juan Capistrano 7.5 Minute Quadrangle (CGS, 2001b), the western border of the site is located within a zone of potential earthquake induced landslides. The proposed development is adequately set back from the hillside on the west and creek to the east with respect to slope stability.
- 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 and experience in the area, site soils are anticipated to have high 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 three weeks for high expansion). 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.
- Based on experience in the area and laboratory test results, site soils are considered to be corrosive based on Caltrans guidelines (Caltrans, 2021).
- Generation of significant quantities of oversized material (material greater than 8 inches in maximum dimension) is generally not anticipated during site grading.
- Previous field testing resulted in infiltration rates ranging from 0.03 inches per hour (essentially zero infiltration) to 0.97 inches per hour (Terracon, 2021). The development site will consist of compacted fill over 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 San Juan Capistrano grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix F. In case of conflict, the following recommendations shall supersede those included as part of Appendix F.

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 (minimum ultimate compressive strength of 100 psi at 28 days). 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. Similarity, 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

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.

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 Medium expansion potential (expansion index 90 or less based on ASTM D4829) or less and no particles larger than 6 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 F).

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 ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater, refer to Figure 2. 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 0 to 10 percent reduction in volume of in-place landslide material (Qls), topsoil and alluvium.

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.

4.1.9 Fill Slopes

Design fill slopes at the site are anticipated to be both grossly and surficially stable as designed, as long as they are constructed in accordance with the Standard Earthwork and Grading Specifications included in Appendix F. Fill slopes should be constructed with a maximum slope ratio of 2:1 (horizontal to vertical). Slope faces should also be compacted to minimum project specifications. 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.

Fill slopes should be constructed at least equipment width wide (approximately 10 horizontal feet). Where design grades will result in “sliver” fills, thinner than 10 feet, the slopes should be constructed as stability fill slopes as described herein.

4.1.10 Existing Native Slopes

Natural slopes will be left in their existing condition above and below portions of the site. These slopes will be subject to “natural” phenomena such as erosion, sloughing and surficial instabilities. It is impossible to predict where or when this may happen. Should erosion or slippage occur, it should be promptly repaired. Paramount in reducing the potential for either erosion or slippage is to properly maintain these slopes (refer to Section 4.7).

4.2 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 presented on Table 4 are 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 import sandy soil backfill criteria. Refer to Figure 2.

TABLE 4

Lateral Earth Pressures – Approved Sandy Soils

Conditions	**Equivalent Fluid Unit Weight (pcf)**	**Equivalent Fluid Unit Weight (pcf)**
	Level Backfill	**2:1 Sloped Backfill**
	Approved Sandy Soils	**Approved Sandy Soils**
Active	35	55
At-Rest	55	80

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed, refer to Figures 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 Sections 4.3.2 and 4.3.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.3 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. Foundations should be supported on properly compacted fill. 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.

4.3.1 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 50 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:

$k = 50[(B+1)/2B]^2$
k = modulus of vertical subgrade reaction, pounds per cubic inch (pci)
B = mat foundation width (feet)

A minimum mat slab (pad) thickness of 8 inches is recommended for the equipment pads. Actual mat slab thickness should be determined by the structural engineer based on the parameters provided herein. The mat slab should have a thickened perimeter edge with a minimum embedment of 18-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 140% of optimum moisture content to a depth of 30 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.

4.3.2 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 ⅓ 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. ½-inch over a horizontal span of 40 feet).

4.3.3 Lateral Load Resistance

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 220 psf per foot of depth (or pcf) to a maximum of 2,200 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 300 pcf (maximum of 3,000 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.3.4 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 4.3.1) 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.

4.3.5 Slab Underlayment Guidelines

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.

4.3.6 Foundation Setback from Top-of-Slope & Bottom- of-Slope

Foundations should have adequate setback from top and bottom of slopes. Per the 2022 CBC, the minimum top-of-slope setback is H/3, with a maximum required setback of 40 feet, where H is the total height of the slope. This distance is measured horizontally from the outside bottom edge of the footing to the slope face. Deepened footings or drilled piers may be used to obtain the required top-of-slope setback. The minimum bottom-of-slope setback is H/2, with a maximum required setback of 15 feet. Refer to Chapter 18 of the 2022 CBC.

Foundation setback criteria should be reviewed based on the finalized grading plans and foundation design.

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 of approximately 0.02 percent or less, chloride contents of 118 parts per million (ppm), pH values of 8.2 and 7.7, and minimum resistivity values of 1,345 ohm-centimeter and 1,250 ohm-centimeter. Previous corrosion testing indicated soluble sulfate contents ranging from approximately less than 0.01 percent to 0.19 percent, chloride contents ranging from not detected to 248 ppm, pH values ranging from 7.0 to 8.1, and minimum resistivity values ranging from 365 ohm-cm to 2,215 ohm-cm (Leighton, 2009 & Terracon, 2021). Site soils are considered corrosive based on Caltrans guidelines (Caltrans, 2021).

Based on laboratory test results and our experience in the area, onsite soils should be designated sulfate exposure class of “S2” per ACI 318, Table 19.3.1.1. This must be verified based on as-graded conditions.

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 5. 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 5

Nonstructural Concrete Flatwork for High 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.)	8 x 8	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	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 Slope Maintenance Guidelines

We recommend that graded slopes be properly landscaped with deep-rooted drought-tolerant, slope stabilizing vegetation as soon as possible to minimize the potential for erosion and/or other instabilities. Slopes should not be allowed to be bare of vegetation. Landscape vegetation should not be "trimmed" to root structures leaving no protection of the slopes.

Irrigation at the site should be kept at the minimum level to support plant growth, overwatering must be avoided. Future landowners/property managers should be made aware that even though the site has been developed in accordance with the local standard of practice that includes a subdrain system, improper maintenance and particularly significant overwatering or poor surface drainage could possibly lead to a buildup in localized groundwater levels. This may result in nuisance type water-related issues to foundations, flatwork, walls, landscaping improvements, etc., and in extreme cases a decrease in the stability of slopes. To help reduce the potential for excessive erosion of graded slopes we recommend that protective measures be implemented in accordance with the latest City of San Juan Capistrano grading ordinances and other governing codes. Design of surface drainage provisions are within the purview of the project civil engineer.

Subdrains and v-ditches must be properly maintained, and their outlets kept free draining and clear of any potential obstructions. Routine maintenance should be performed, especially prior to and during the rainy season. Failure to properly maintain these elements may result in slope failures, slumps, excessive erosion, localized saturated zones, nuisance type water issues, etc.

Any future trenches excavated on a slope face for utility or irrigation lines and/or for any purpose should be properly backfilled and compacted to the slope face. Observation/testing and acceptance by the geotechnical consultant during trench backfill are recommended.

A program for the elimination of burrowing animals in both native and graded slope areas must be established and properly maintained to protect slope stability by reducing the potential for surface water to penetrate into the soil. Continuous erosion control, rodent control, and maintenance are essential to the long-term stability of all slopes.

4.8 Surface Drainage and Landscaping

4.8.1 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.

4.9 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.

Previous field testing resulted in infiltration rates ranging from 0.03 inches per hour (essentially zero infiltration) to 0.97 inches per hour. The site consists of primarily fine-grained clayey soils, which have very low to negligible rates of infiltration. In general, we do not recommend the intentional infiltration of storm water. If desired, additional evaluation should be performed, specific to the proposed location of infiltration. Any infiltration system would have to prevent the lateral migration of the infiltrated water which could decrease the existing stability of the proposed development.

4.10 Geotechnical Plan Review

Project plans (e.g., grading, foundation, retaining wall, 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.

4.11 Geotechnical Observation and Testing During Construction

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

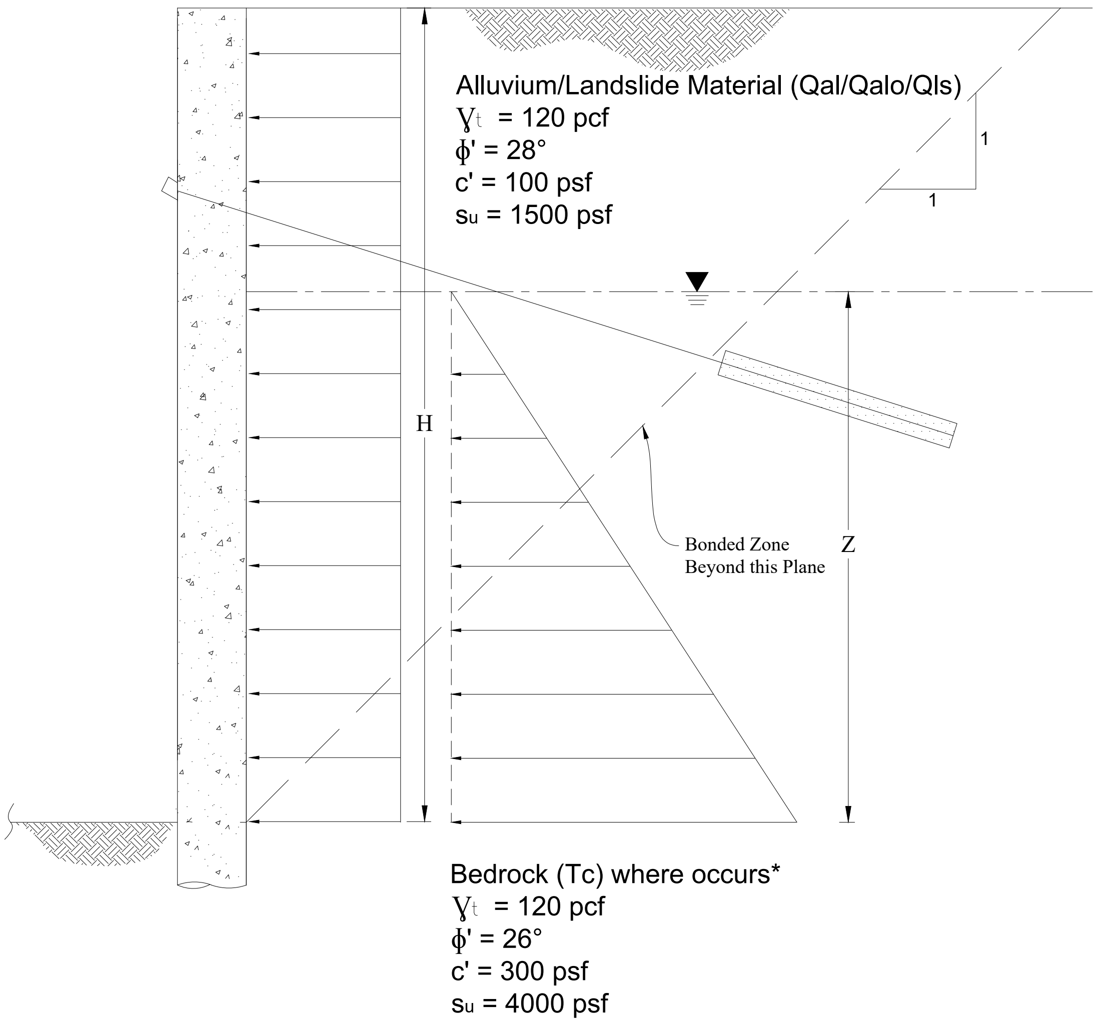
This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical 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	Compass Bess - SJC
PROJECT NO.	22011-01
ENG. / GEOL.	BTZ / KBC
SCALE	NTS
DATE	April 2024



STATIC HORIZONTAL STRESS (psf) = $30H\left(1-0.52\frac{Z}{H}\right)+62.4Z$

WHERE:

H = Total Retained Height (ft)

Z = Height of Water Measured From Bottom of Retaining Structure (ft)

NOTES:

1. The earth pressures shown are based on level retained conditions
2. Applicable seismic and surcharge loading to be added, refer to text discussion

**Refer to Cross Section F-F' for Geologic Stratigraphy*

LGC Geotechnical, Inc.

FIGURE 2

Lateral Earth Pressures for Braced Eastern Retaining Wall Structure

PROJECT NAME	Compass BESS - SJC
PROJECT NO.	22011-01
ENG. / GEOL.	BTZ / KBC
SCALE	NTS
DATE	April 2024

Appendix A
References

APPENDIX A

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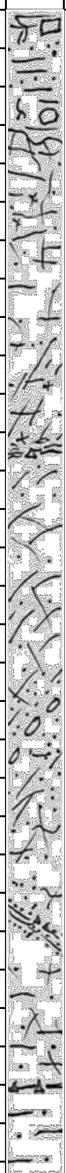
Appendix B
Boring Logs

Geotechnical Bucket Auger Log B-1

Date : 7/27/2022 Page 1 of 3	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 233 ' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

Logged by MJG/KTM

Sampled by MJG/KTM



Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0								**@ 0 to 44' - <u>Quaternary Landslide (Qls)</u>**	
									@ 0' - Dried grass and dry light brown, SILT	
									@ 0' to 1' - Concrete debris	
230									@ 1 to 4' - Mixed Sandy SILT, some trace clay, soil is mottled gray with tan, some white mineralization, @3' - Artificial Fill, topsoil and SILTSTONE clasts, light gray and brown, trace orange.	
	5								@ 4' - Grades to highly fractured siltstone, gray with orange, moist, medium dense, abundant white gypsum crystals. Some inclusions of clay, and fine sand lenses	
225										
			SH/RS: N18W 68W						@ 9' - Sub-horizontal sand lenses (undulatory), off set by R.S. Zone: closely spaced gypsum fabric	
	10			R-1	Push				@ 10' - SILT and Lean CLAY: light gray overall, iron oxide, gypsum crystals	
				G-2					@ 11.5' - Rupture Surface Subhorizontal, sample of G-2 of clay with SILT, organics, moist, soft. Surface is clean but anastomosed, overlies material similar to above	
220										
	15									
215										
									@ 19' - sheared fabric with small offset	
	20		GB: N28W 30S	R-2	Push				@ 20' - CLAY: light gray, slightly moist, some iron oxide and gypsum, medium to low plasticity	
									@ 22' - Qls material, chaotic, gray mixed with tan, iron oxide, gypsum, pockets of fine sand	
210										
			N40E 29NW & N60E 26NW						@ 24' - rupture surface, oxidized high plasticity gray clay with fine white sand pinched along surface, 5" thick zone of 1/2" thick sand lenses, dense with iron oxide. Becomes dark gray below, unoxidized	
	25									
205			RS: N50W 7W	G-3					@ 28' - light gray CLAY 1/2" thick planar, some siltstone	

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

SAMPLE TYPES:

B	BULK SAMPLE
R	RING SAMPLE
G	GRAB 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

Last Edited: 8/11/2022

Geotechnical Bucket Auger Log B-1

Date : 7/27/2022	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 233' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

Logged by MJG/KTM
Sampled by MJG/KTM

Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	60			R-6	7	91.0	29.1	ML	@ 60' - SILTSTONE: dark gray, hard to very hard, wet, vague concreted zones with tight, short, sub planar, faint slicken lines, lacks clay. Decrease moisture to very moist.	
168										
	65								During drilling, buckets are not saturated after 65ft	
163										
	70								@ 68' - Concretion, approximately 1 foot thick, lacks clay above and below. Standing water at bottom may be from seepage above	
158									**Total Depth = 71' (Practical Refusal)** **Groundwater Encountered @41ft and @69ft** **Backfilled & Tamped with native soil and bentonite layers on 7/28/2022**	
	75									
153										
	80									
148										
	85									
143										

Geotechnical Bucket Auger Log B-2

Date : 7/28/2022 Page 1 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 275' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

Logged by MJG/KTM

Sampled by MJG/KTM

Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0								**@ 0' to 84' - <u>Quaternary Landslide (Qls)</u>**	
									@ 0' - SILT: light gray, dry, loose Shift to SILT with lean CLAY, darker brownish gray, dry to slightly moist @ 0' to 3' - Sandy SILT: mottled light brown, dry to slightly moist, very stiff, some fine sand,(colluvial deposit)	
272										
	5									
267										
	10			R-1	1	104.3	7.9	ML	@ 10' - Sandy SILT: light brown, slightly moist, loose to medium stiff, some trace clay, fine sand @ 11' - Change in color to a lighter brown silt, undulatory contact, slightly moist	
262										
	15									
257										
	20		C: N40W, 26SW	R-2	Push	109.8	15.4	ML	@ 20' - Sandy SILT: reddish brown, very moist, medium stiff. Contact attitude, lighter brown, clay layer with darker brown clay below, planar	
252										
	25								@ 24' - 1ft thick layer of clay and silt, dark brown with numerous pods of soil in lighter brown below, some organics present, pulses of slopewash	
247										

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SAMPLE TYPES:

B	BULK SAMPLE
R	RING SAMPLE
G	GRAB 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

Last Edited: 10/13/2022

Geotechnical Bucket Auger Log B-2

Date : 7/28/2022 Page 2 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 275' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

Logged by MJG/KTM

Sampled by MJG/KTM

Elevation (ft)	Depth (ft)	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-3	Push				@ 30' - SILT with CLAY: some gray and black mottled Qls, very light, brownish gray overall	DS
242									
	35							@ 35' - Chaotic silt and clay, tan to gray overall with small patches of oxidation, very light brown, near vertical pond deposit laminations of fine sand	
237		RS: Subhorizontal						@ 38' - Subhorizontal clay bed (1/2" thick) non-continuous, possible internal rupture surface	
	40		R-4	3	103.6	21.6	CL	@ 40' - CLAY: light olive brown, mottled, very stiff, very moist, mixed iron oxide and some magnesium oxide	
232									
		CB: N75W, 38S						@ 44' - Clay Bed attitude, strands of gray plastic CLAY, very moist, remnant bedrock structure, very faint	
	45								
		RS: Subhorizontal						@ 47' - Subhorizontal rupture surface, numerous similar features to 56'	
227								@ 49' - Iron oxide band around hole	
	50		R-5	Push	96.8	26.0	CL	@ 50' - Sandy CLAY: olive brown with iron oxide, slightly stiff, wet	
								@ 52' - Band of black CLAY around hole, mottled with gray CLAY, medium plasticity	
222									
	55							@ 56' - Concretion and lens of black organic rich clay (1/4 of hole), below is gray mottled, silt and siltstone, extremely hammered/pulverized, stiff, very moist, abundant pods of iron oxide, calcium carbonate nodules, organic/carbon pods	
217									

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Geotechnical Bucket Auger Log B-2

Date : 7/28/2022 Page 3 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 275' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

Logged by MJG/KTM

Sampled by MJG/KTM

Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	60			R-6	3	106.1	20.8	ML	@ 60' - SILT with Sand to Siltstone: olive yellow, wet, stiff, mottled white mineralization	
212										
	65									
207									@ 68' - Minor seepage, small concretion	
	70			R-7	2	97.3	26.2	ML	@ 70' - Clayey SILT to SILTSTONE: olive with mottled iron oxide, wet, stiff	
202										
	75									
									@ 77' - Pods of organic material aligned to the southwest at 55 degrees	
197										
	80		C: N24E, 24NW	R-8	4	92.7	30.1	ML	@ 80' - Contact attitude, material changes to Clayey SILT: olive brown mottled with iron oxide transitioning to a darker gray, wet, stiff	
192										
			RS: N35E, 42SE	G-1 G-2					@ 84' - Rupture Surface, CLAY, 1/2" thick dark gray, soft, moist, polished base, grab sample taken.	AL
	85								**@ 84' to T.D. - <u>Tertiary Capistrano Formation (Tc)</u>** Siltstone to fine Sandy SILTSTONE, dark gray, very stiff to hard, very moist to wet	
187										

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Geotechnical Bucket Auger Log B-2

Date : 7/28/2022 Page 4 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 275' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

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Sampled by MJG/KTM

Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	90		Sh: 30E, 55SE	R-9	11	90.3	26.0	ML	@ 90' - Sandy SILTSTONE: dark grayish brown, wet, hard, Shear attitude, very faint fabric of tight paper thin Tectonic Shear	
182										
	95									
				R-10	12	87.7	28.8	ML	@ 97' - Sandy SILTSTONE: dark grayish brown, wet, hard	
177									**Total Depth = 98'** **Very Minor Seepage @68'** **Backfilled & Tamped with native soil and bentonite layers** **7/29/2022**	
	100									
172										
	105									
167										
	110									
162										
	115									
157										

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Geotechnical Bucket Auger Log B-3

Date : 8/03/2022 Page 1 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 255' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

Logged by MJG/KTM

Sampled by MJG/KTM

Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0								**@ 0' to 67.5' - <u>Quaternary Landslide (Qls)</u>** @ 0 to 2' - Silt with some CLAY: Dry and loose at surface, light gray to light brown, topsoil and artificial fill @ 2' - Moderately gray brown, Fine Sandy SILT, dry, hard, mottled Silt and Clay, gypsum, iron oxide, very stiff, dry to slightly moist, light yellowish, brown and gray mottled, varied (pulses of material below)	
251										
	5									
246										
	10			R-1	1	102.6	20.2	CL	@ 10' - Sandy CLAY: olive yellow, mottled, some gypsum and iron oxide, very moist, medium stiff, light gray silt, some sand lenses, trace clay, iron oxide, some concretions, mottle Clay grades into Silt (Gray to light gray, iron oxide)	
241			Sh: N48W, 45W						@ 14' - Clay-lined Shear attitude(about 1/2" thick) and undulating, clay lined shear, light gray Siltstone (Qls) over Colluvium	
	15									
									@ 17' - Fracture	
236										
	20		GB: N10W, 36W	R-2	1	101.7	15.3	ML	@ 20' - Sandy SILT: light olive yellow with mottled oxidation, very moist, medium stiff. Generalized Bedding Attitude, sand lense (few non continuous ext. fractured/offset sand lenses) iron oxide	
231										
	25		C: N40W, 38W						@ 25' - Contact attitude, organic silt layer	
226										

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Last Edited: 10/13/2022

Geotechnical Bucket Auger Log B-3

Date : 8/03/2022 Page 2 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 255' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

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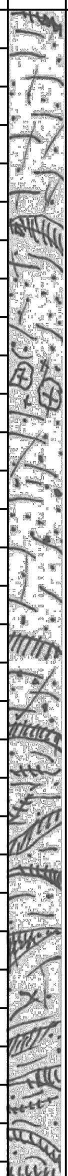

Elevation (ft)	Depth (ft)	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30	C: N38W, 25SW	R-3	2	107.1	19.0	CL	@ 30' - Sandy CLAY: olive gray mottled, some gypsum and iron oxide, very moist, low plasticity. Contact attitude, undulating clay band, possible rupture surface	
221									
	35								
		RS: N38W, 30SW	G-1					@ 36' - Rupture Surface attitude, undulatory clay band, approximately 2" to 3" thick dark brown, soft, moist, rupture zone. Grab sample taken.	
216									
	40		R-4	1				@ 40' - Lean CLAY: mottled gray, some iron oxide, slightly moist. Concretions and gypsum, overlies light gray, clayey Siltstone, stiff, very moist, iron oxide and gypsum pods (lacks open fractures)	DS
211									
	45								
								@ 47' - Top of zone of layered slopewash with organic-rich layers alternating light gray and dark gray with greenish brown zones of small sand pods, abundant gypsum pods, jarosite, well indurated	
206									
	50		R-5	4	100.3	24.0	ML	@ 50' - Sandy SILT: olive gray overall with some light gray mottling, wet, stiff, some gypsum	
201									
	55							@ 55' - SILT and CLAY: chaotic, light and dark gray, gypsum, some iron oxide	
196									

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Geotechnical Bucket Auger Log B-3

Date : 8/03/2022 Page 3 of 4	**Drilling Company :** Roy Brothers
Project Name : Compass BESS - San Juan Capistrano	**Type of Rig :** EZ Bore
Project Number : 22011-01	**Drop :** 12" **Hole Diameter :** 24"
Elevation of Top of Hole : ~ 255' MSL	**Drive Weight :** 0' - 25' = 4900 lbs. 25' - 50' = 3400 lbs. 50' - 75' = 2200 lbs. 75' - 100' = 1200 lbs.
Hole Location : See Geotechnical Map	

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Elevation (ft)	Depth (ft)	Graphic Log	Attitudes	Sample Number	Blow Count	Dry Density(pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	60			R-6	1	104.2	20.2	CL	@ 60' - Sandy CLAY: olive gray mottled, very moist, low plasticity, flecks of gypsum in a band	
									@ 62' - Flecks of gypsum in a band, about 4-inches wide	
191										
	65								@ 65' - Base of slopewash layers	
			RS: N25W, 6SW	G-2					@ 67.5' - Rupture surface attitude, planar, continuous, 1/2" clay dark gray, soft, moist, and plastic, grab sample taken	AL
									@ 67.5' to T.D. Tertiary Capistrano Formation (Tc)	
186										
	70			R-7	13	92.8	28.1	CL/ML	@ 70' - Clayey SILTSTONE: dark olive gray, wet, slightly hard to hard	DS
181										
	75									
176										
	80			R-8	20	94.5	23.8	ML	@ 80' - SILTSTONE: grayish brown, very moist, hard to very hard	
171										
	85									
									@ 87' - Trace fossils, fine sand - filled burrows	
166										

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