Appendix H: Geotechnical Evaluation

Appendices

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November 17, 2023 Project No. 23169-01

Mr. Gary Jones
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Subject: ***Preliminary Geotechnical Evaluation and Design Recommendations for the Proposed Residential Development of 1698 and 1700 Greenbriar Lane, City of Brea, Orange County, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation and has provided design recommendations for the proposed residential re-development of the property at 1698 and 1700 Greenbriar Lane, in the City of Brea, Orange County, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed re-development of the property.

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

Respectfully Submitted,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our preliminary geotechnical evaluation for the proposed approximately 9.7-acre residential development located at 1698 and 1700 Greenbriar Lane in the City of Brea, Orange County, California. Refer to the Site Location Map (Figure 1).

The purpose of our study was to provide a preliminary geotechnical evaluation relative to the proposed residential development. As part of our scope of work, we have: 1) reviewed available geotechnical background information including existing geotechnical reports, in-house regional geologic maps, and published geotechnical literature pertinent to the site (Appendix A); 2) performed a limited subsurface geotechnical evaluation of the site consisting of the excavation of seven small-diameter borings ranging in depth from approximately 10 to 51.5 feet below existing ground surface; 3) performed infiltration testing of subsurface soils at three locations; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this preliminary geotechnical summary report presenting our findings, preliminary conclusions and recommendations for the development of the proposed project.

1.2 Project Description

The site is bound to the north by Greenbriar Lane, to the east by Fullerton Creek (aka "Loftus Diversion Channel"), to the south by a commercial development, and to the west by the 57 Freeway. The site is currently a commercial development with several buildings clustered at the west half and a 4-level parking garage within the east half. Parking lots and drive aisles exist throughout the site. A series of small slopes and a maintenance road at the eastern boundary of the property descends from the existing parking lot towards the existing channel bottom that is approximately 20 vertical feet lower than the existing parking lot.

Based on the conceptual site plan by Hunsaker & Associates (Hunsaker, 2023), the proposed improvements include the construction of 183 residential units, interior streets, and associated improvements. A plan that shows the proposed cuts and fills is not available at this time but is assumed to be relatively minor. The proposed residential building structures are anticipated to be relatively light-weight at-grade structures with maximum column and wall loads of approximately 30 kips and 2 kips per linear foot, respectively.

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

1.3 Background

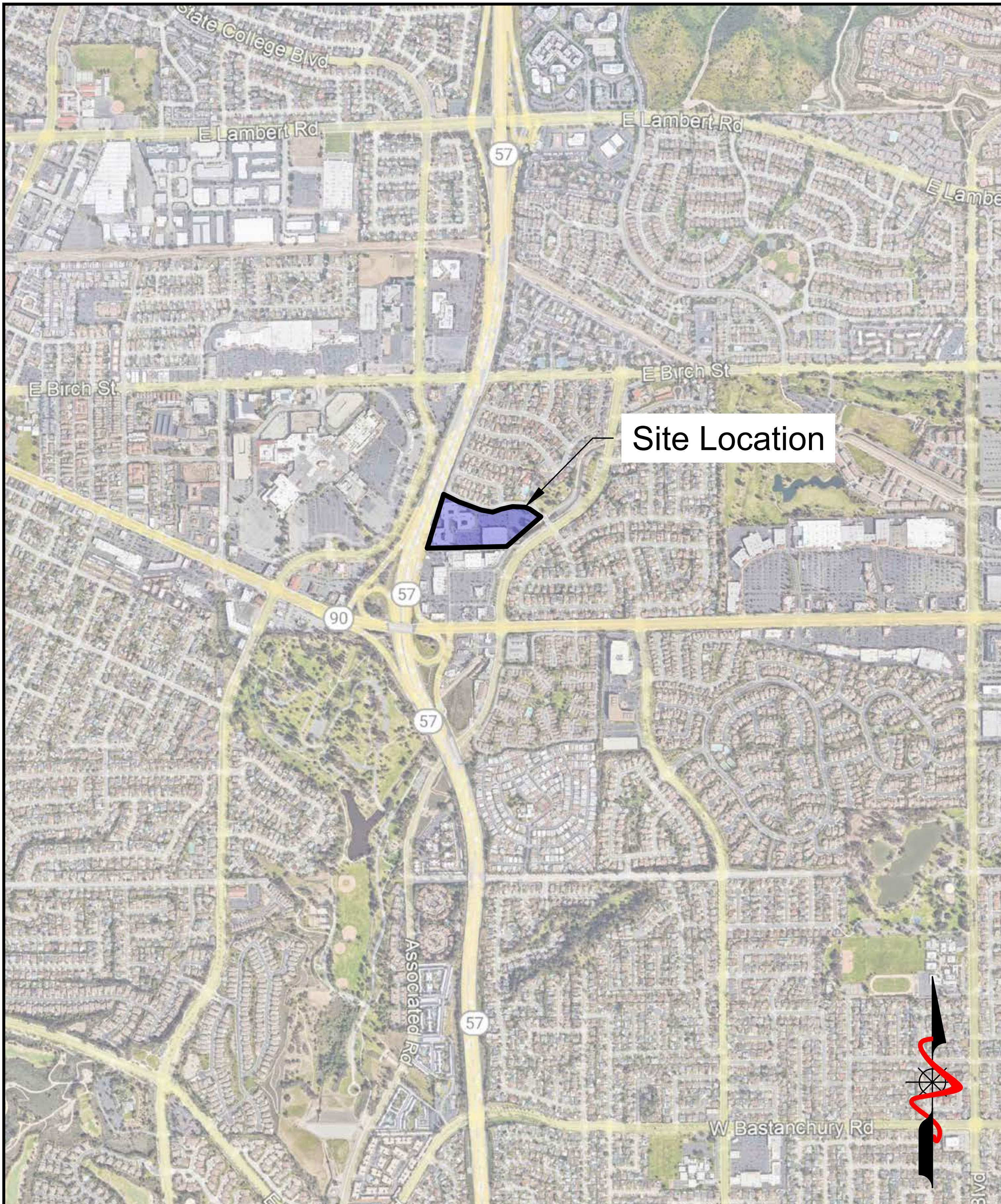
Review of historical aerials indicates that prior to 1952 until 1965 the site consisted of undeveloped rolling hills. By 1972 the shape of the site was formed by adjacent streets, it had trails throughout, and a small south flowing tributary drainage to Fullerton Creek dissected the eastern-most portion of the site. By 1980, the main structures and a parking lot were developed across the entire site to what is seen today. Construction of the large parking structure replaced a portion of the parking lot within the eastern half of the property, the addition occurred between 2006 and 2009.

1.4 Subsurface Geotechnical Evaluation

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

Seven hollow-stem borings (HS-1 through HS-4 and I-1 through I-3) were drilled to depths ranging from approximately 10 to 51.5 feet below existing grade. An LGC Geotechnical staff engineer observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated by 2R Drilling, Inc. under subcontract to LGC Geotechnical using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. Driven soil samples were collected by means of the Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The MCD sampler (2.4-inch ID, 3.0-inch OD) was driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples of the near-surface soils were also collected and logged at select borings for laboratory testing. At the completion of drilling, the borings were backfilled with the native soil cuttings, tamped, and capped with asphalt cold patch. Some settlement of the backfill soils may occur over time.

The approximate locations of our subsurface explorations are provided on the Geotechnical Map (Sheet 1). The boring logs are provided in Appendix B.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

H-6

PROJECT NAME	Lennar - Greenbriar, Brea
PROJECT NO.	23169-01
ENG. / GEOL.	RLD / KTM
SCALE	Not to Scale
DATE	November 2023

1.5 Laboratory Testing

Representative bulk and driven (relatively undisturbed) samples were obtained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, Atterberg Limits, fines content, laboratory compaction, expansion index, consolidation, direct shear, 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 90 pounds per cubic foot (pcf) to 120 pcf, with an average of 107 pcf. Field moisture contents ranged from approximately 4 to 31 percent, with an average of 16 percent.
- Five fines content tests were performed and indicated a fines content (passing No. 200 sieve) ranging from approximately 19 to 95 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as both "coarse and fine-grained."
- Four Atterberg Limit (liquid limit and plastic limit) tests were performed. Results indicated Plasticity Index (PI) values ranging from 'Non-Plastic' to 24.
- One consolidation test was performed. The load versus deformation plot is provided in Appendix C.
- One direct shear test was performed. The plot is provided in Appendix C.
- Expansion potential testing indicated an expansion index value of 55, corresponding to "Medium" expansion potential.
- One laboratory compaction test of a near surface sample indicated a maximum dry density of 118.5 pcf with an optimum moisture content of 11.5 percent.
- Corrosion testing indicated soluble sulfate contents of approximately 0.014 percent, a chloride content of 260 parts per million (ppm), pH of 8.06, and a minimum resistivity of 5,000 ohm-centimeters.

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

2.0 GEOTECHNICAL CONDITIONS

2.1 Geologic Conditions

The subject site is generally located within the eastern-most edge of the Los Angeles sedimentary basin, within the Peninsular Ranges Geomorphic Province of California. The site is more specifically in the area of Brea Canyon, located south of the Whittier Fault and adjacent east-west trending Puente Hills, and east of the Coyote Hills. The Puente Hills to the north have been dissected with a series of drainages that drain across the canyon bottom, and locally combine to form the upper reaches of the San Gabriel River basin. The site is located within the gently to moderately sloping plain starting from the base of the Puente Hills, consisting of older alluvial fan deposits. The older alluvium was further dissected by the main drainages seen today, most of those have been channelized, including the Brea, Fullerton, and Coyote Creeks. The site is located on older alluvial deposits that originally formed the west bank of the Fullerton Creek that runs in a southerly direction adjacent to the eastern end of the subject site. Based on review of historic photographs, the original drainage was formerly naturally flowing southeast along a small, incised tributary to Fullerton Creek that appears to have been filled in as part of development of the subject property.

2.2 Generalized Subsurface Conditions

Based on regional geologic mapping (Dibblee, 2001), the subject site is generally underlain Quaternary Older Alluvium (Map Symbol – Qoa), and relatively limited amounts of older artificial fill placed by others as part of the existing development (Map Symbol – afo). Limits of artificial fill as presented on the Geotechnical Map (Sheet 1) were generally estimated from old topographic maps (Historic Aerials, 2023) and limited observations within on-site borings. Based on review of samples and comparison of historic topography, we estimate that artificial fill was placed in a small north-trending tributary drainage that originally transected the site at the approximate location presented on the Geotechnical Map.

No reports of previous rough grading activities onsite were available for review at this time; however, review of aerial photographs indicates the site was rough graded in the mid to late 1970's. An existing, asphalt-covered parking lot is currently at the top of the eastern slope that gradually descends outside of the subject property to the bottom of the (partially-lined) channel, as much as 20 feet total. A portion of the slope (or all of it) likely consists of artificial fill placed by others.

As indicated in our field exploration logs, the Quaternary Older Alluvium generally consists of silty sand, sandy clay, sand, and silt with clay, medium dense to very dense/very stiff to hard, to the maximum explored depth of approximately 51.5 feet below existing grade. Surficial units including artificial fill placed by others and remnant topsoil were observed to consist of sandy clay, medium stiff to stiff. Materials were generally moist to very moist, becoming wet with depth.

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

2.3 Groundwater

Groundwater was encountered during our recent investigation at a depth of approximately 20 feet below existing ground surface at the eastern side of the site, and approximately 25 feet below ground on the western side of the site. A historic high groundwater depth has not been mapped at the subject site; however, on the eastern portion of the site, we conservatively estimate the groundwater could rise to a depth of approximately 15 feet below existing grade.

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

2.4 Field Infiltration Testing

Three field percolation tests were performed at site per the direction of the project civil engineer, the locations are depicted on Sheet 1 – Geotechnical Map. Test well installation consisted of placing a 3-inch diameter perforated PVC pipe in the excavated 8-inch diameter borehole and backfilling the annulus with crushed rock including the placement of approximately 2 inches of crushed rock at the bottom of the borehole. The infiltration test wells were presoaked the day of installation and testing took place within 24 hours of presoaking. During the pre-test, the water levels in the borings were observed to drop less than 6 inches in 25 minutes for two consecutive readings. Therefore, the test procedure for fine-grained soils or “slow test” was followed. Test well installation and the estimation of infiltration rates were accomplished in general accordance with the guidelines set forth by County of Orange (2013). In general, three-dimensional flow out of the test well (*percolation*), as observed in the field, is mathematically reduced to one-dimensional flow out of the bottom of the test well (*infiltration*). Infiltration tests are performed using relatively clean water, free of particulates, silt, etc. The results of our recent field infiltration testing are presented in Appendix D and summarized in Table 1 below.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Identification	Approx. Depth Below Existing Grade (ft)	Observed Infiltration Rate* (in./hr.)
I-1	10	0.2
I-2	10	0.2
I-3	10	0.1

*Observed Infiltration Rates Do Not Include Factor of Safety.

The tested infiltration rates provided in this report are considered a general representation of the infiltration rates at the location of the proposed infiltration boring. Please note, the testing of infiltration rates is highly dependent upon the materials encountered at the point of testing (i.e., location and depth of testing). Varying subsurface conditions may exist outside of the test location which could alter the calculated infiltration rate. Please refer to Section 4.8 for subsurface water infiltration recommendations.

2.5 <u>Seismic Design Criteria</u>

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (C.B.C) 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 33.9141 degrees north and longitude -117.8791 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 2 on the following page. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.70 at a distance of approximately 8.10 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.62 at a distance of approximately 13.42 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 C.B.C (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 0.864 (SEAOC, 2023). The design PGA is equal to 0.576g (2/3 of PGA_M).

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)	1.812g	From SEAOC, 2023
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.637g	From SEAOC, 2023
F_a (per Table 1613.2.3(1))	1.000	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.700	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.812g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	1.083g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.208g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.722g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.901	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.903	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

2.6 Faulting

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

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

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

2.6.1 Liquefaction and Dynamic Settlement

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

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1998), the subject site is not within a liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of some of the on-site soils and the relatively dense nature of the coarse-grained soils. Therefore, liquefaction potential is considered low.

2.6.2 Lateral Spreading

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

Due to the low potential for shallow liquefaction the potential for lateral spreading is also considered low.

2.7 Oversized Material

Oversized material (material larger than 8 inches in maximum dimension) may be encountered during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix E. If feasible, crushing oversized materials onsite or exporting oversized materials may be considered. Incorporating oversized materials into “rock fills” (windrows, rock blankets or individual rock burial) may be feasible in some of the deeper remedial grading areas if applicable. Special handling recommendations should be provided on a case-by-case basis, if necessary.

2.8 Expansion Potential

Based on the results of previous laboratory testing by others and our recent laboratory testing, site soils have a “Medium” expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

3.0 CONCLUSIONS

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

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

- In general, our borings indicate the site is underlain by primarily by silty sand, sandy clay, sand, and silt with clay, medium dense to very dense/very stiff to hard, to the maximum explored depth of approximately 51.5 feet below existing grade. Surficial units including artificial fill placed by others and remnant topsoil were observed to consist of sandy clay, medium stiff to stiff. The upper approximately 5 to 10 feet of near-surface soils are generally compressible and are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was encountered during our recent investigation at depths of approximately 20 and 25 feet below existing ground surface. We conservatively estimate the historic high groundwater depth to be approximately 15 feet below existing grade which would be above the bottom of the existing channel on the eastern portion of the site.
- The subject site is not located within the State of California Earthquake Fault Zone (Alquist-Priolo). The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The is not located in a State of California Seismic Hazard Zone for liquefaction. Site soils are considered not susceptible to liquefaction due to the fine-grained nature of some of the on-site soils and the relatively dense nature of the coarse-grained soils. Therefore, liquefaction potential is considered low.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “Medium” expansion potential. Mitigation measures are required for foundations and site improvements like concrete flatwork to minimize the impacts of expansive site soils. Final design expansion potential must be determined at the completion of grading.
- Pre-soaking of the subgrade for building slabs will be required due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 2 weeks).
- The site contains soils that are not suitable for 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.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that the on-site earth materials generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.

4.0 PRELIMINARY RECOMMENDATIONS

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

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

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

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of demolition of the existing site improvements, required earthwork removals, subgrade preparation, precise grading and construction of the proposed new improvements, including residential structures, neighborhood amenities, subsurface utilities, interior streets, etc.

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

4.1.1 Site Preparation

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

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

4.1.2 Removal Depths and Limits

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend the near-surface potentially compressible site soils be removed and recompacted. Approximate anticipated removal below existing grades have been estimated and presented on the Geotechnical Map, Sheet 1. Existing older artificial fill within the influence of the proposed building pads should be removed to competent native materials.

We recommend that soils within building pads be removed and recompacted to a minimum of 5 feet below existing grade or to the approximate depths presented on the Geotechnical Map (Sheet 1), whichever is deeper. The envelope for removal and recompaction should extend laterally a minimum distance of 5 feet beyond the edges of the proposed improvements, where possible. Removals along the northern property boundary should be performed efficiently and immediately replaced with properly compacted fill in order to limit the time left open. The contractor should protect the existing property line improvements during grading (e.g., trees, retaining walls, block walls, etc.). In order to promote soil uniformity in areas of design cut, over-excavation shall extend a minimum of 4 feet below finished grade or to the minimum anticipated remedial depths presented on the Geotechnical Map (Sheet 1), whichever is deeper.

For minor site structures such as free-standing and screen walls, the removals should extend at least 3 feet beneath the existing grade or 2 feet beneath the base of foundations, whichever is deeper. Within pavement and hardscape areas, removals should extend to a depth of at least 2 feet below the existing grade. Pavement area over-excavation (design cut areas) 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 over-excavation should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements mentioned above.

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

4.1.3 <u>Temporary Excavations</u>

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. <u>Excavation safety is the sole responsibility of the contractor.</u>

Where proposed building structures will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing "ABC" slot cuts. Slot cuts should be backfilled <u>immediately</u> with properly placed compacted fill to finish grade prior to excavation of adjacent slots. Sandy soils are present and should be considered susceptible to caving. Recommendations for ABC slot cuts including dimensions should be provided during grading based on the conditions encountered. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 projection from the bottom of the excavation or 5 feet, whichever is greater. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

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

4.1.4 <u>Removal Bottoms and Subgrade Preparation</u>

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

Removal bottoms, over-excavation 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 oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of soils of "Low" expansion potential (expansion index 50 or less based on American Society for Testing and Materials [ASTM] D 4829), and free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of four working days prior to any planned importation.

Retaining wall backfill should consist of imported sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per ASTM Test Method D1140 (or ASTM D6913/D422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. The site contains soils that are not suitable for retaining wall backfill due to their fines content and expansion potential; therefore, import of soils 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 requirements of Section 200-2 of the most recent version of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) and/or City of Brea requirements.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 1 to 3-inches in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned street areas (i.e., not within building pad areas).

4.1.6 ***Placement and Compaction of Fills***

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

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

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

Aggregate base material should be compacted to at least 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to at least 90 percent relative compaction per ASTM D1557 at 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 the above Section.

Retaining wall backfill should consist of sandy soils as outlined in preceding Section 4.1.5. The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (Figure 2). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

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

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

4.1.8 <u>Shrinkage and Subsidence</u>

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an estimate of shrinkage and bulking factors for the various geologic units found onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

<u>TABLE 3</u>

<u>Estimated Shrinkage and Bulking</u>

Soil Type	Allowance	Estimated Range
Older Artificial Fill	Shrinkage	5% to 15%
Quaternary Older Alluvium	Shrinkage	0% to 10%

Subsidence due to earthwork equipment is expected to be on the order of 0.1 to 0.2 feet. It should be stressed that these values are only estimates and that actual shrinkage factors are extremely difficult to predict. 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 variable; the above estimates are generalized groupings of similar lithologies and should be expected to vary across the site and with depth.

The above shrinkage estimates are intended as an aid for others in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies 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 accuracy of the topographic survey and survey accuracy during rough grading.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability in on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil.

4.2 Slopes

Existing slopes up to a maximum height of approximately 20 feet are anticipated to be both grossly and surficially stable, as long as they are constructed and maintained in accordance with the recommendations herein and the Standard Earthwork and Grading Specifications included in Appendix E.

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. Slopes may be prone to surficial instabilities during periods of heavy rain.

4.2.1 Slope Maintenance Guidelines

It is recommended that any graded slopes be planted with ground cover vegetation as soon as practical to reduce the potential for erosion by reducing runoff velocity. Deep-rooted vegetation that requires little water and is able to survive local climate conditions should also be established to protect against surficial slumping. Under no circumstances should slopes be allowed to be bare of vegetation. Landscape vegetation must not be "trimmed" to root structures leaving no protection of the slopes. Irrigation levels should be kept to the minimum level necessary to establish healthy plant growth. Slopes must not be overwatered. If automatic sprinklers are used, they must be adjusted during periods of rainfall. A landscape professional should be consulted for specific landscape recommendations.

A program for the elimination of burrowing animals in both native and graded slope areas must be established to protect slope stability by reducing the potential for surface water to penetrate into the slope face. Continuous erosion control, rodent control, and maintenance are essential to the long-term stability of all slopes. Trenches excavated on a slope face for utility or irrigation lines and/or for any purpose must be properly backfilled and compacted to project recommendations (refer to Section 4.1.7) to the slope face. Observation/testing and acceptance by the geotechnical consultant during trench backfill are recommended. V-ditches should be inspected and cleared of loose soil and/or debris on a routine basis, especially prior to and during the rainy season.

4.3 Preliminary Foundation Recommendations

Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the residential structures using a conventional or post-tensioned foundation system designed to resist the impacts of expansive soils. Site soils are anticipated to be "Medium" expansion potential (EI of 90 or less per ASTM D4829) and special design considerations from a geotechnical perspective are required. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading.

Preliminary foundation recommendations are provided in the following sections. Recommended soil bearing and estimated settlement due to structural loads are provided in Section 4.4.

4.3.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with the Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2022 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 25
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer
- Minimum Footing Depth: 18 inches below lowest adjacent grade.
- Moisture-condition (presoak) slab subgrade to 120% of optimum moisture content to a minimum depth of 18 inches prior to trenching.

The recommended moisture content should be maintained up to the time of concrete placement.

4.3.2 Provisional Post-Tensioned Foundation Design Parameters

The geotechnical parameters provided herein may be used for post-tensioned slab foundations. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI, 2012) Standard Requirements (PTI DC 10.5), referenced in Chapter 18 of the 2022 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method to resist expansive soils.

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

TABLE 4

Provisional Geotechnical Parameters for Post-Tensioned Foundation Slab Design

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	Medium[1]	Medium[1]
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 9.0 feet 0.5 inch	 9.0 feet 0.6 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 4.7 feet 1.1 inch	 4.7 feet 1.3 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	150 pci	150 pci
Minimum perimeter footing/thickened edge embedment below finish grade	18 inches	6 inches
Perimeter foundation reinforcement	N/A[2]	N/A[2]
Minimum slab thickness	5 inches[2]	8 inches[2]
Presoak (moisture conditioning)	120% of Optimum to 18 inches	120% of Optimum to 18 inches

1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Recommendations for sand below slabs have traditionally been included with geotechnical foundation recommendations, although they are not the purview of the geotechnical consultant. The sand layer requirements are the purview of the foundation engineer/structural engineer and should be provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction".
4. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.

4.3.3 Post-Tensioned Foundation Subgrade Preparation and Maintenance

Moisture conditioning of the subgrade soils is recommended prior to trenching the foundation. 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 recommendations specific to the anticipated site soil conditions, including recommended presoak, are presented in Table 4. The subgrade moisture condition of the building pad soils should be maintained at near-optimum moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the homes.

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

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

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

4.3.4 Slab Underlayment Guidelines

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

4.3.5 Foundation Setback from Top-of-Slope and Bottom-of-Slope

Foundations should be set back from the top and bottom of slopes in accordance with the California Building Code (CBC) and the City of Brea. 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. 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 for additional information. It is the purview of the project civil engineer to implement the appropriate foundation setbacks.

4.4 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 2,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 18 inches below lowest adjacent ground surface. This value may be increased by 400 psf for each additional foot of embedment and 200 psf for each additional foot of foundation width to a maximum value of 3,000 psf. A post-tensioned mat foundation a minimum of 6 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

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

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.3 may be assumed with dead-load forces. For slabs constructed over a moisture retarder, the allowable friction coefficient should be provided by the manufacturer. An allowable passive lateral earth pressure of 225 psf per foot of depth (or pcf) to a maximum of 2,250 psf may be used for 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.

4.5 Lateral Earth Pressures for Retaining Walls

Lateral earth pressures for <u>import soils</u> (sandy soils) meeting indicated project recommendations (Section 4.1.5) are provided below. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented on Table 5 are for backfilled retaining walls using approved select granular soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and a maximum Expansion Index of 20 (per ASTM D-4829). The site contains soils that are not suitable for 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 using the parameters provided in Table 5 below. The retaining wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria.

TABLE 5

<u>Lateral Earth Pressures – Imported Sandy Soils</u>

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

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

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining wall. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. The retaining wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near-surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care and information should be provided to homeowners to maintain these drains. Typical retaining wall drainage is illustrated in Figure 2. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water containing soluble salts migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

If retaining walls greater than 6 feet in height are proposed, the retaining wall designer should contact the geotechnical engineer for specific lateral earth pressure increments based on the configuration of the planned retaining wall structures.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.4. 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.

4.6 Soil Corrosivity

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

Corrosion testing of near-surface bulk samples indicated a soluble sulfate content of approximately 0.014 percent, chloride content of 260 parts per million (ppm), pH of 8.1, and minimum resistivity of 5,000 ohm-centimeters. Based on Caltrans Corrosion Guidelines (Caltrans, 2021), soils are considered corrosive to structural elements if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2 percent) or greater. Based on test results, soils are not considered corrosive using Caltrans criteria. Note that based on minimum resistivity the soils are considered moderately corrosive to metallic improvements. If improvements that may be susceptible to corrosion are proposed, it is recommended that further evaluation by a corrosion engineer be performed.

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

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

4.7 *Control of Surface Water and Drainage Control*

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

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

4.8 *Subsurface Water Infiltration*

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

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating 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.

Per the County of Orange Guidelines (2013), infiltration of stormwater is not required when the factored infiltration rate (observed infiltration rate with safety factor applied) is less than 0.3 inches per hour. The infiltration rates presented in Table 1, with or without the safety factor applied, are lower than the minimum infiltration rate requirements from the County.

Based on results of field infiltration testing indicating low infiltration rates, very stiff clays and dense silty sands and sands encountered at depth, and shallow groundwater levels, we strongly recommend against the intentional infiltration of stormwater into the subsurface soils.

4.9 *Preliminary Asphalt Concrete Pavement Sections*

The following provisional minimum asphalt concrete (AC) street sections are provided in Table 6 on the following page for Traffic Indices (TI) of 5.0, and 6.0. These sections are based on an assumed R-value of 10. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final pavement sections should be confirmed by the project civil engineer based upon the final design Traffic Index. The City of Brea requires a minimum pavement section of 4 inches asphalt concrete over 6 inches aggregate base for alleys and local streets. Refer to the minimum pavement section recommendations below in accordance with the City of Brea. The final Traffic Index is determined by the Civil Engineer or City Engineer. We are not responsible for selecting a design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 6

Preliminary Asphalt Concrete Pavement Section Options

Assumed Traffic Index	5.0 (or less)	6.0
R -Value Subgrade	10	10
AC Thickness	4.0 inches	5.0 inches
Aggregate Base Thickness	7.5 inches	9.5 inches

Due to anticipated heavy construction traffic during installation of utilities and home construction, we recommend that the total thickness (base course and capping course) of AC be placed at essentially the same time. Allowing heavy construction traffic loading on only the base course of the AC will increase the potential for pavement distress. It should be noted that construction traffic such as concrete trucks will likely exceed traffic loading after completion of construction.

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

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

4.10 Nonstructural Concrete Flatwork

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

TABLE 7

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

	Community Sidewalks (≤6 feet wide)	**Private Drives**	**Patios/Walkways** (adjacent to homes or flatwork >6 feet wide)	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 (nominal)	5 (full)	5 (full)	City/Agency Standard
Presoaking	Wet down	Presoak to 12 inches	Presoak to 12 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	—	City/Agency Standard

To reduce the potential for driveways to separate from the garage slab, the builder may elect to install dowels to tie these two elements together. Similarly, future homeowners should consider the use of dowels to connect flatwork to the foundation.

4.11 Geotechnical Plan Review

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

4.12 Geotechnical Observation and Testing During Construction

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

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

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

5.0 LIMITATIONS

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

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

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

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

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

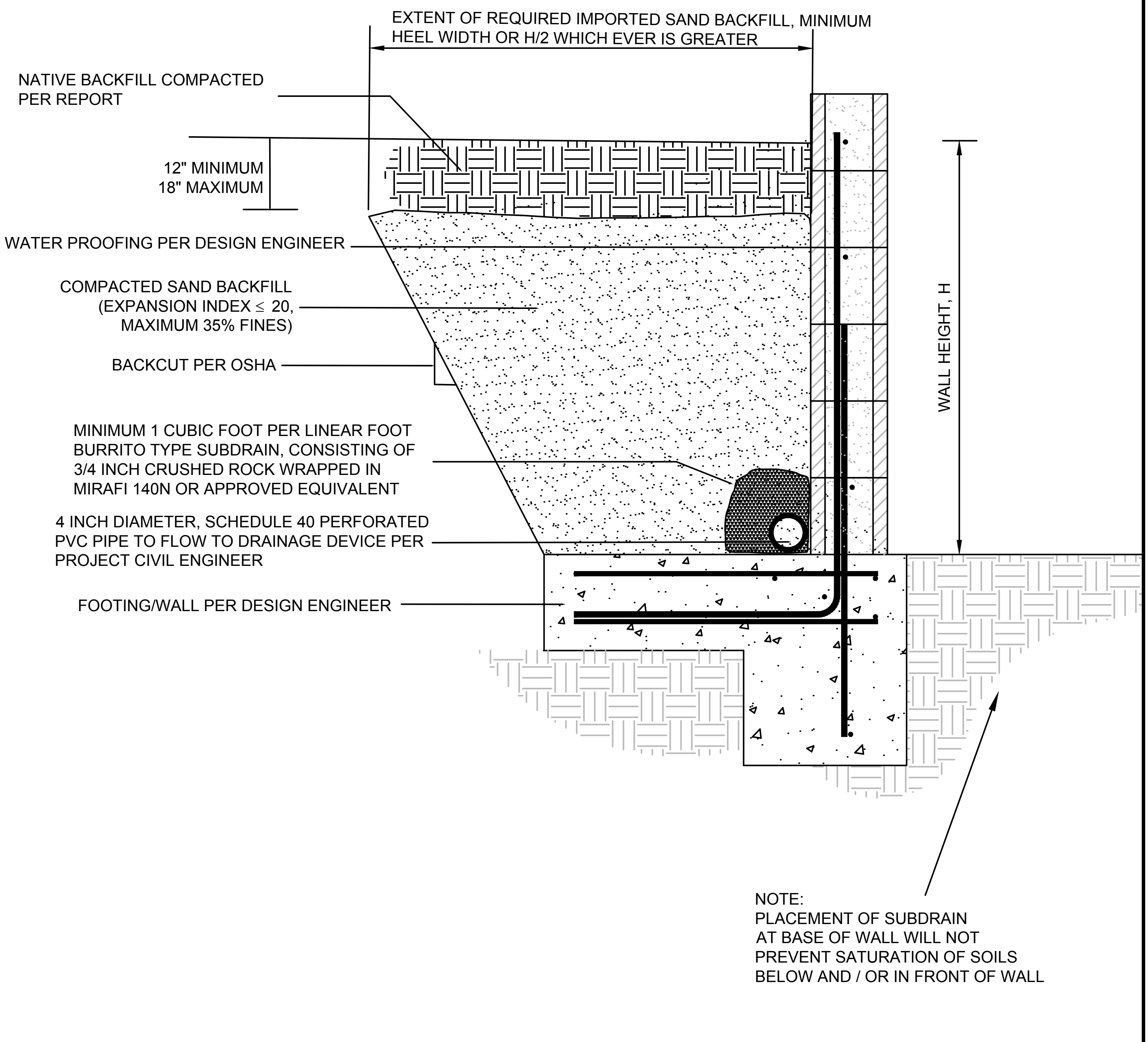


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FIGURE 2
Retaining Wall Backfill Detail

PROJECT NAME	Lennar - Greenbriar, Brea
PROJECT NO.	23169-01
ENG. / GEOL.	RLD / KTM
SCALE	Not to Scale
DATE	November 2023

Appendix A
References

APPENDIX A

<u>References</u>

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16, Third Printing, 2017.

American Society for Testing and Materials (ASTM), Volume 04.08 Soil and Rock (I): D420 – D5876.

Bray, J.D., and Sancio, R. B., 2006, Assessment of Liquefaction Susceptibility of Fine-Grained Soils, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, pp. 1165-1177, dated September 2006.

California Building Standards Commission, 2022, California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2022.

California Department of Transportation (Caltrans), 2021, Corrosion Guidelines, Version 3.2, dated May 2021.

California Division of Mines and Geology (CDMG), 1998, State of California Seismic Hazard Zones, La Habra Quadrangle, Official Map, scale: 1:24,000, Release Date: April 15, 1998.

________, 2001, State of California Seismic Hazard Zone Report for the La Habra 7.5-Minute Quadrangle, Los Angeles and Orange Counties, California, Seismic Hazard Zone Report 009, Revised 2001.

California Geological Survey [CGS], 2008, California Geological Society Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California.

________, 2018, California Geological Survey website, Interactive Fault Map: <u>http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html</u>.

County of Orange, 2013, Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs), dated December 20, 2013.

Dibblee, 2001, Geologic Map of Whittier and La Habra Quadrangles (Western Puente Hills), Los Angeles and Orange Counties, California, by Thomas Dibblee, Jr.

Historic Aerials, 2023, viewed November 7, 2023, Aerials viewed from: 1952 through 2020, <u>https://www.historicaerials.com/</u>.

Hunsaker & Associates, Inc., 2023, Greenbriar - Conceptual Grading Plan, 183 Unit, Brea, CA, dated November 13, 2023.

NCEER, 1997, "Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", T. L. Youd and I. M. Idriss Editors, Technical Report NCEER-97-0022, NCEER, Buffalo, NY.

Post-Tensioning Institute (PTI), 2012, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12.

Southern California Earthquake Center (SCEC), 1999, "Recommended Procedure for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigation Liquefaction Hazards in California", Edited by Martin, G.R., and Lew, M., dated March 1999.

Structural Engineers Association of California (SEAOC), 2023, Seismic Design Maps, Retrieved November 3, 2023, from https://seismicmaps.org/.

United States Geological Survey (USGS), 2014, Unified Hazard Tool, Dynamic: Conterminous U.S. 2014 (update) (v4.2.0), Retrieved November 3, 2023, from: https://earthquake.usgs.gov/hazards/interactive/.

Woodley Architectural Group, Inc., 2023, Concept Site Plan, Option "A", Mercury Site, Lennar, dated August 14, 2023.

Appendix B
Field Exploration Logs & Infiltration Data

Geotechnical Boring Log Borehole HS-1

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~339' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 3.5" Asphalt over 5" Base	
								Quaternary Older Alluvium (Qoa)	
		B-1	R-1	14 25 26	122.1	11.3	CL	@ 2.5' - Sandy CLAY: brown, slightly moist, hard, trace of gravel	
335	5		R-2	16 20 25	116.0	15.4		@ 5' - Sandy CLAY: brown, moist, hard, trace of gravel	
			SPT-1	5 11 14		14.3		@ 7.5' - Sandy CLAY: brown, moist, hard	
330	10		R-3	16 24 36	113.7	11.8		@ 10' - Sandy CLAY: brown, moist, hard	
325	15		R-4	9 26 45	110.1	10.2	SM to ML	@ 15' - Silty SAND to Sandy SILT: light brown, moist, very dense to hard, fine to medium grained sand	
320	20		SPT-2	9 15 25		7.8	SM	@ 20' - Silty SAND: light brown, moist, very dense, fine	-#200
315	25	▽		8 21 40				@ 25' - No Recovery: Groundwater Encountered	
310	30								

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

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

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

Last Edited: 10/20/2022

Geotechnical Boring Log Borehole HS-1

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~339' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-5	8 18 25	96.3	29.0	SM	@ 30' - Silty SAND: brown, wet, dense	AL -#200
305	35								
300	40		R-6	7 16 21	90.1	31.4	CL	@ 40' - CLAY: light brown, wet, hard, trace of gravel	AL -#200
295	45								
290	50		R-7	5 21 24	91.8	30.6		@ 50' - CLAY: brown, wet, hard, few gravel	-#200
285	55							Total Depth = 51.5' Groundwater Encountered @ 25 feet Backfilled with Cuttings on 10/11/2022 and patched with asphalt	
280	60								

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- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~332' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 2.5" Asphalt over 5" Base	
330								**Quaternary Older Alluvium (Qoa)**	
			SPT-1	2 3 4		18.3	CL	@ 2.5' - Sandy CLAY: dark brown, very moist, stiff	AL
	5		R-1	6 14 20	115.4	15.4		@ 5' - Sandy CLAY: dark brown, moist, very stiff	
325			SPT-2	4 4 6		16.9		@ 7.5' - Sandy CLAY: brown, moist, stiff	
	10		R-2	3 14 19	105.7	14.5	ML	@ 10' - SILT with Sand: light brown, moist, very stiff, trace of gravel	
320									
	15		SPT-3	5 12 15		4.5	SM	@ 15' - Silty SAND: light brown, slightly moist, dense	
315									
	20		R-3	14 21 24	95.5	10.4		@ 20' - Silty SAND: reddish brown, moist, dense	
310								Total Depth = 21.5' Groundwater Not Encountered Backfilled with Cuttings on 10/11/2022 and patched with asphalt	
	25								
305									
	30								
295									

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

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

Last Edited: 10/20/2022

Geotechnical Boring Log Borehole HS-3

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~333' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 3" Asphalt over 6" Base	
								Quaternary Older Alluvium (Qoa)	
330			SPT-1	7 17 22		15.0	CL	@ 2.5' - Sandy CLAY: reddish brown, moist, hard	
	5	B-1	R-1	14 27 43	113.6	10.5	SM	@ 5' - Silty SAND: reddish brown, moist, very dense, trace of gravel	
325			SPT-2	6 14 8		16.5		@ 7.5' - Silty SAND: light brown, very moist, medium dense	
	10		R-2	11 30 45	106.1	4.4		@ 10' - Silty SAND: yellowish brown, dry, very dense, trace of gravel	
320									
	15		SPT-3	14 17 27		8.6		@ 15' - Silty SAND: light brown, moist, very dense, trace of gravel	
315									
	20		R-3	11 20 40	98.1	21.6	SP-SC to SM	@ 20' - SAND with Clay to Silty SAND: light brown, very moist, dense	
310								Total Depth = 21.5' Groundwater Not Encountered Backfilled with Cuttings on 10/11/2022 and patched with asphalt	
	25								
305									
	30								

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

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

Last Edited: 10/20/2022

Geotechnical Boring Log Borehole HS-4

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~327' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 3.5" Asphalt over 5" Base	MD EI DS CR -#200
325								**Artificial Fill Placed by Other (afo)**	
			SPT-1	3 4 4		15.6	CL	@ 2.5' - Sandy CLAY: reddish brown, stiff, trace of gravel	
	5	B-1	R-1	4 7 13	111.0	15.1		@ 5' - CLAY: yellowish brown, moist, very stiff	CN
320								**Quaternary Older Alluvium (Qoa)**	
			SPT-2	4 6 8		16.3	CL	@ 7.5' - Sandy CLAY: brown, moist, very stiff	
	10		R-2	5 11 15	114.5	15.0		@ 10' - Sandy CLAY: mottled dark and reddish brown, moist, very stiff	
315									
	15		SPT-3	4 7 7		21.1		@ 15' - Sandy CLAY: brown, very moist, very stiff	
310									
	20		R-3	8 12 18	105.8	22.1		@ 20' - CLAY: light brown, very moist, very stiff groundwater encountered at 20 feet	
305								Total Depth = 21.5' Groundwater Encountered at 20 feet Backfilled with Cuttings on 10/11/2022 and patched with asphalt	
	25								
300									
	30								

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

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

Last Edited: 10/20/2022

Geotechnical Boring Log Borehole I-1

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~325' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 3.5" Asphalt over 5" Base	
								Quaternary Older Alluvium (Qoa)	
			SPT-1	7 7 7		9.7	SM	@ 2.5' - Silty SAND: light brown, moist, medium dense	
320	5		SPT-2	8 12 16		16.6	CL	@ 5' - Sandy CLAY: brown, moist, hard	
			SPT-3	4 7 10		17.1		@ 7.5' - CLAY with Sand: light brown, moist, very stiff	
315	10		R-1	8 21 25	111.0	18.0		@ 10' - Sandy CLAY: light brown, very moist, hard	
310	15							Total Depth = 10' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel installed Pipe Removed and Backfilled with Cuttings on 10/12/2023	
305	20								
300	25								
295	30								

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

Last Edited: 10/20/2022

Geotechnical Boring Log Borehole I-2

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~333' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 3.5" Asphalt over 5" Base	
								Artificial Fill Placed by Others (afo)	
330			SPT-1	2 3 3		18.1	CL	@ 2.5' - Sandy CLAY: brown, very moist, medium stiff	AL
	5		SPT-2	4 7 8		17.3		@ 5' - Sandy CLAY: dark brown, moist, very stiff	
								Quaternary Older Alluvium (Qoa)	
325			SPT-3	2 4 8		17.3	CL	@ 7.5' - Sandy CLAY: brown, moist, very stiff	
	10		R-1	4 6 13	108.6	20.3		@ 10' - Sandy CLAY: mottled brown and reddish brown, very moist, very stiff	
320								Total Depth = 10' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel installed Pipe Removed and Backfilled with Cuttings on 10/12/2023	
	15								
315									
	20								
310									
	25								
305									
	30								

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

Last Edited: 10/20/2022

Geotechnical Boring Log Borehole I-3

Date: 10/11/2023	**Drilling Company:** 2R Drilling
Project Name: Lennar - Greenbriar, Brea	**Type of Rig:** Truck Mounted
Project Number: 23169-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~328' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By KTM

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - 3.5" Asphalt over 5" Base	
								Artificial Fill Placed by Others (afo)	
325			SPT-1	3 3 4		18.5	CL	@ 2.5' - Sandy CLAY: brown, very moist, stiff	
								Quaternary Older Alluvium (Qoa)	
	5		SPT-2	3 3 6		20.6	CL	@ 5' - Sandy CLAY: brown, very moist, stiff, trace of small gravel	
320			R-1	4 6 10	105.6	22.2		@ 7.5' - Sandy CLAY: dark gray, very moist, stiff, rootlets	
	10		R-2	5 8 12	110.9	17.9		@ 10' - Sandy CLAY: dark brown, moist, very stiff	
315								Total Depth = 10' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel installed Pipe Removed and Backfilled with Cuttings on 10/12/2023	
	15								
310									
	20								
305									
	25								
300									
	30								

Geotechnical, Inc.

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Last Edited: 10/20/2022

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

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

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

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-4 @ 1-5 feet	55	Medium

* ASTM D4829

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

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 20 feet	Silty Sand	24
HS-1 @ 30 feet	Silty Sand	19
HS-1 @ 40 feet	Clay	95
HS-1 @ 50 feet	Clay	87
HS-4 @ 1-5 feet	Sandy Clay	53

Laboratory Testing Procedures and Test Results

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

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-1, R-5 @ 30 ft	NP	NP	NP	NP
HS-1, R-6 @ 40 ft	48	26	22	CL
HS-2, SPT-1 @ 2.5 ft	40	16	24	CL
I-2, SPT-1 @ 2.5 ft	36	18	18	CL

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

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

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-4 @ 1-5 feet	Light Brown Sandy Clay	118.5	11.5

Laboratory Testing Procedures and Test Results

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented below.

Sample Location	Chloride Content, ppm
HS-4 @ 1-5 feet	260

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

Sample Location	Sulfate Content (ppm)	Sulfate Exposure Class *
HS-4 @ 1-5 feet	136	S0

*Based on ACI 318R-14, Table 19.3.1.1

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

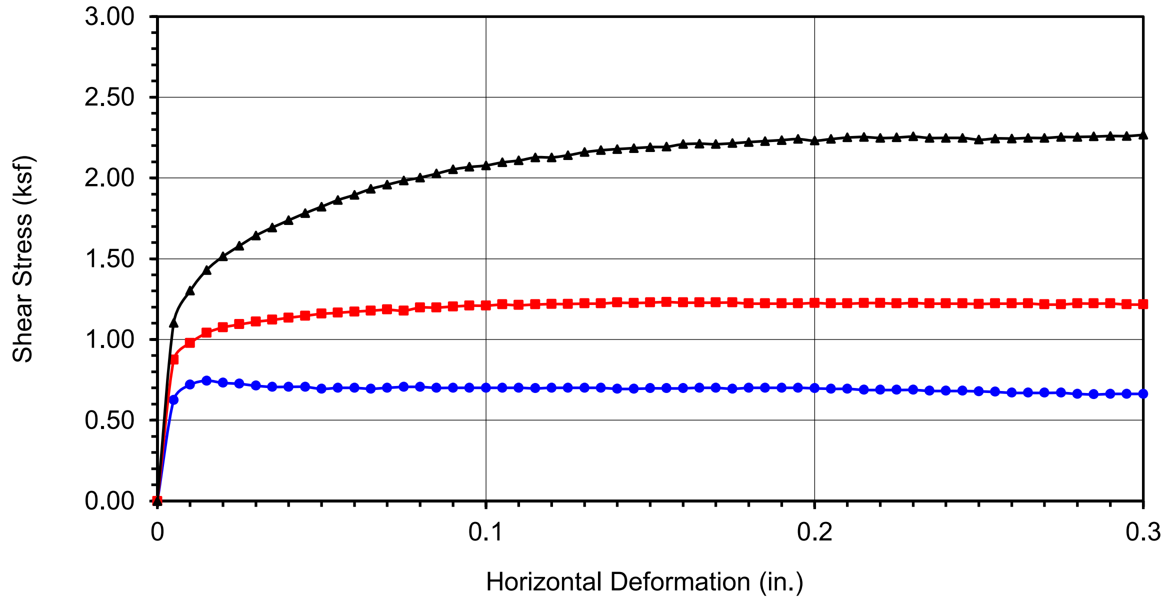
Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-4 @ 1-5 feet	8.06	5000

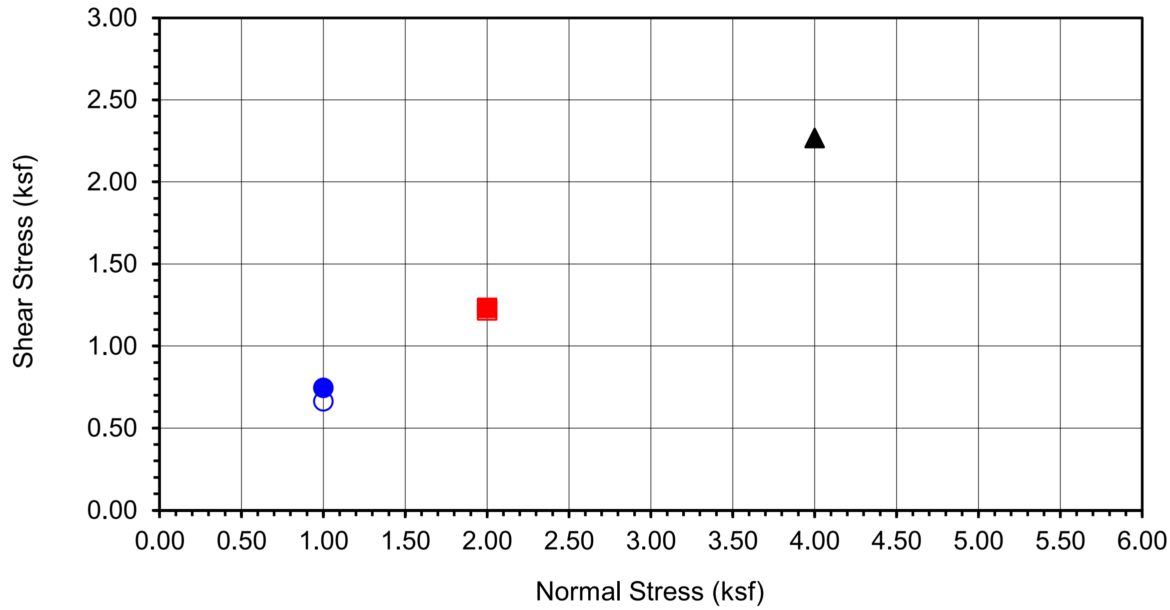
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Lennar - Greenbriar Lane	Tested By:	G. Bathala	Date:	10/23/23
Project No.:	23169-01	Checked By:	J. Ward	Date:	10/30/23
Boring No.:	HS-4	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	1-5		
Soil Identification:	Light brown sandy lean clay s(CL)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	189.19	188.82	189.70
Weight of Ring(gm):	45.46	44.52	45.30
Before Shearing			
Weight of Wet Sample+Cont.(gm):	160.48	160.48	160.48
Weight of Dry Sample+Cont.(gm):	149.52	149.52	149.52
Weight of Container(gm):	55.48	55.48	55.48
Vertical Rdg.(in): Initial	0.0000	0.2541	0.2483
Vertical Rdg.(in): Final	0.0024	0.2535	0.2683
After Shearing			
Weight of Wet Sample+Cont.(gm):	215.18	205.82	205.80
Weight of Dry Sample+Cont.(gm):	190.32	182.80	184.64
Weight of Container(gm):	62.62	55.54	57.44
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**HS-4**
Sample No.	**B-1**
Depth (ft)	**1-5**

<u>Sample Type:</u>

90% Remold

<u>Soil Identification:</u>

Light brown sandy lean clay s(CL)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.745	■ 1.232	▲ 2.267
Shear Stress @ End of Test (ksf)	○ 0.663	□ 1.217	△ 2.267
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.65	11.65	11.65
Dry Density (pcf)	107.1	107.5	107.6
Saturation (%)	54.8	55.4	55.5
Soil Height Before Shearing (in.)	1.0024	1.0006	0.9800
Final Moisture Content (%)	19.5	18.1	16.6

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 23169-01

Lennar - Greenbriar Lane

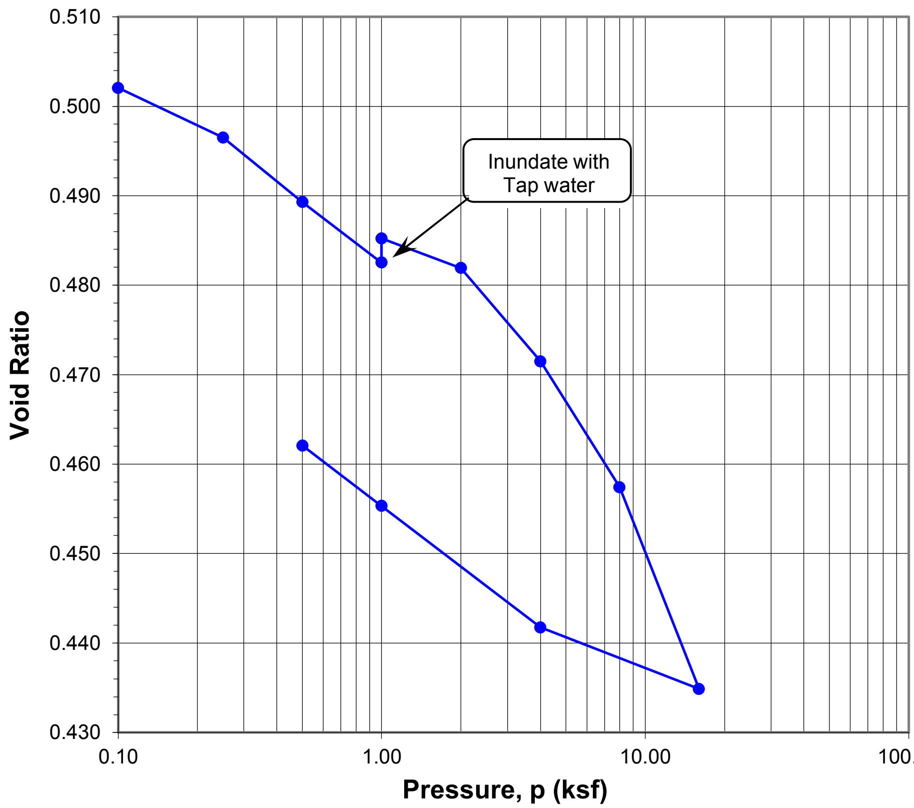
10-23

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Lennar - Greenbriar Lane	Tested By: GB/JD	Date: 10/19/23
Project No.:	23169-01	Checked By: J. Ward	Date: 10/30/23
Boring No.:	HS-4	Depth (ft.): 5.0	
Sample No.:	R-1	Sample Type:	Ring
Soil Identification:	Yellowish brown lean clay (CL)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	199.81
Weight of Ring (g):	44.98
Height after consol. (in.):	0.9729
Before Test	
Wt. of Wet Sample+Cont. (g):	229.65
Wt. of Dry Sample+Cont. (g):	208.82
Weight of Container (g):	68.09
Initial Moisture Content (%)	14.801
Initial Dry Density (pcf)	112.2
Initial Saturation (%):	79
Initial Vertical Reading (in.)	0.0714
After Test	
Wt. of Wet Sample+Cont. (g):	240.79
Wt. of Dry Sample+Cont. (g):	219.56
Weight of Container (g):	39.71
Final Moisture Content (%)	15.74
Final Dry Density (pcf):	115.3
Final Saturation (%):	92
Final Vertical Reading (in.)	0.1022
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43



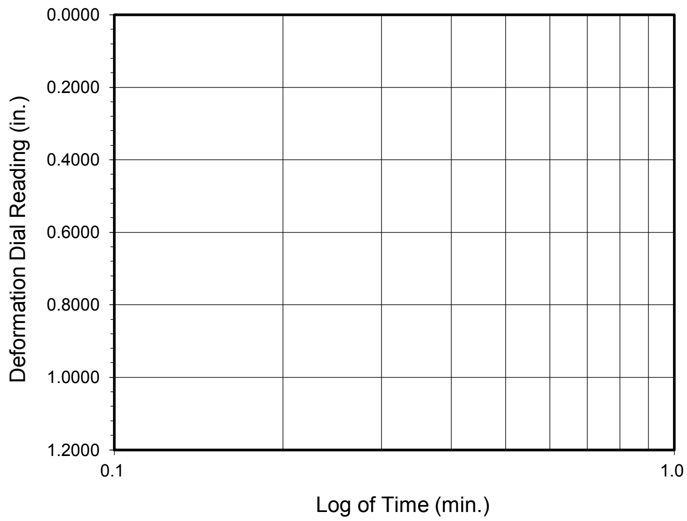
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.0719	0.9995	0.00	0.05	0.502	0.05
0.25	0.0760	0.9954	0.04	0.46	0.496	0.42
0.50	0.0813	0.9901	0.09	0.99	0.489	0.90
1.00	0.0865	0.9849	0.16	1.51	0.483	1.35
1.00	0.0847	0.9867	0.16	1.33	0.485	1.17
2.00	0.0877	0.9837	0.24	1.63	0.482	1.39
4.00	0.0956	0.9759	0.33	2.42	0.471	2.09
8.00	0.1061	0.9653	0.45	3.47	0.457	3.02
16.00	0.1223	0.9491	0.57	5.09	0.435	4.52
4.00	0.1168	0.9547	0.47	4.54	0.442	4.07
1.00	0.1070	0.9644	0.40	3.56	0.455	3.16
0.50	0.1022	0.9692	0.37	3.08	0.462	2.71

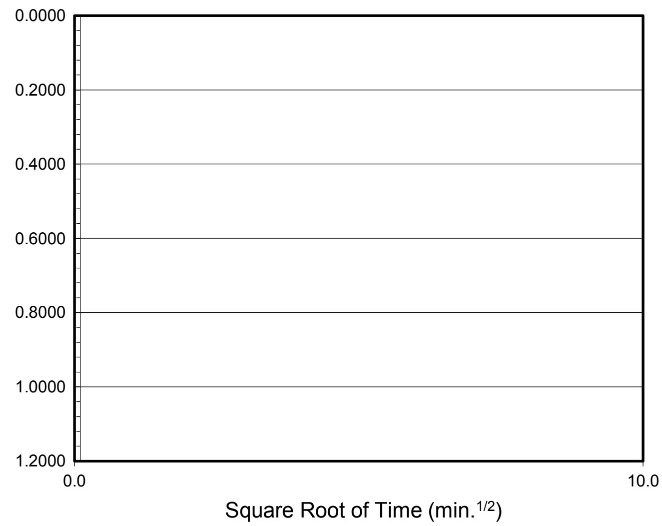
Time Readings

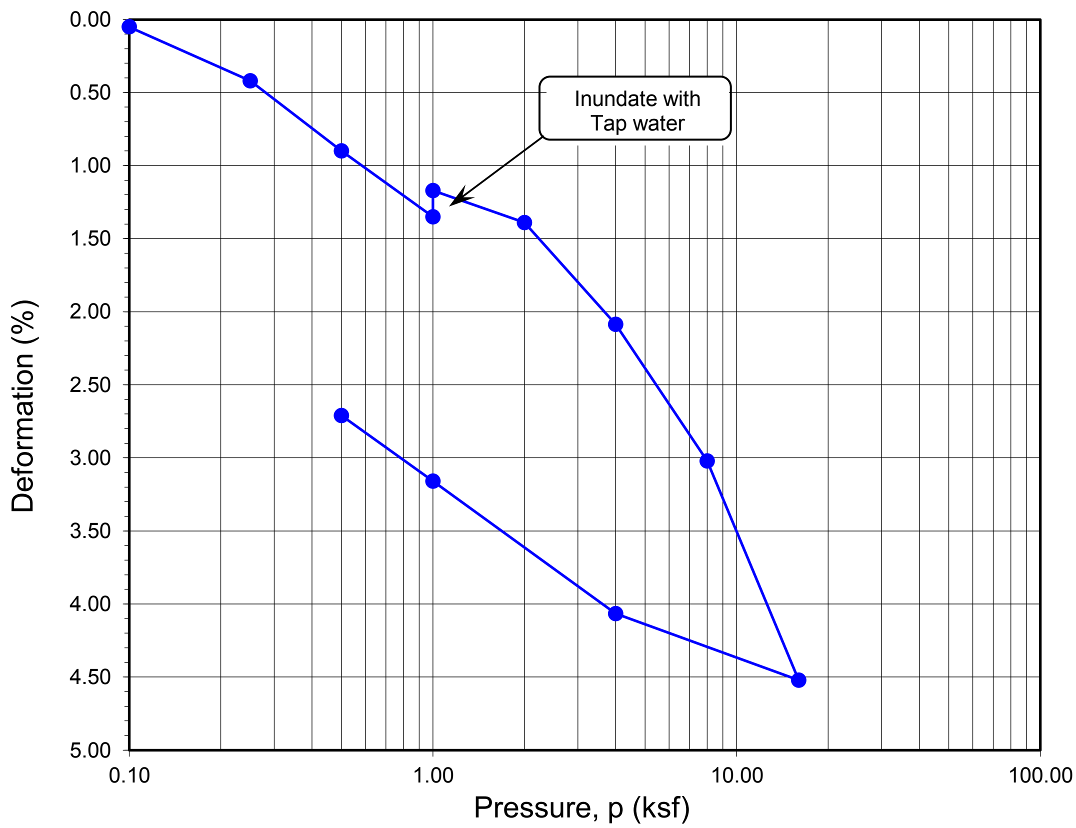
Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

Consol HS-4, R-1 @ 5

Time Readings



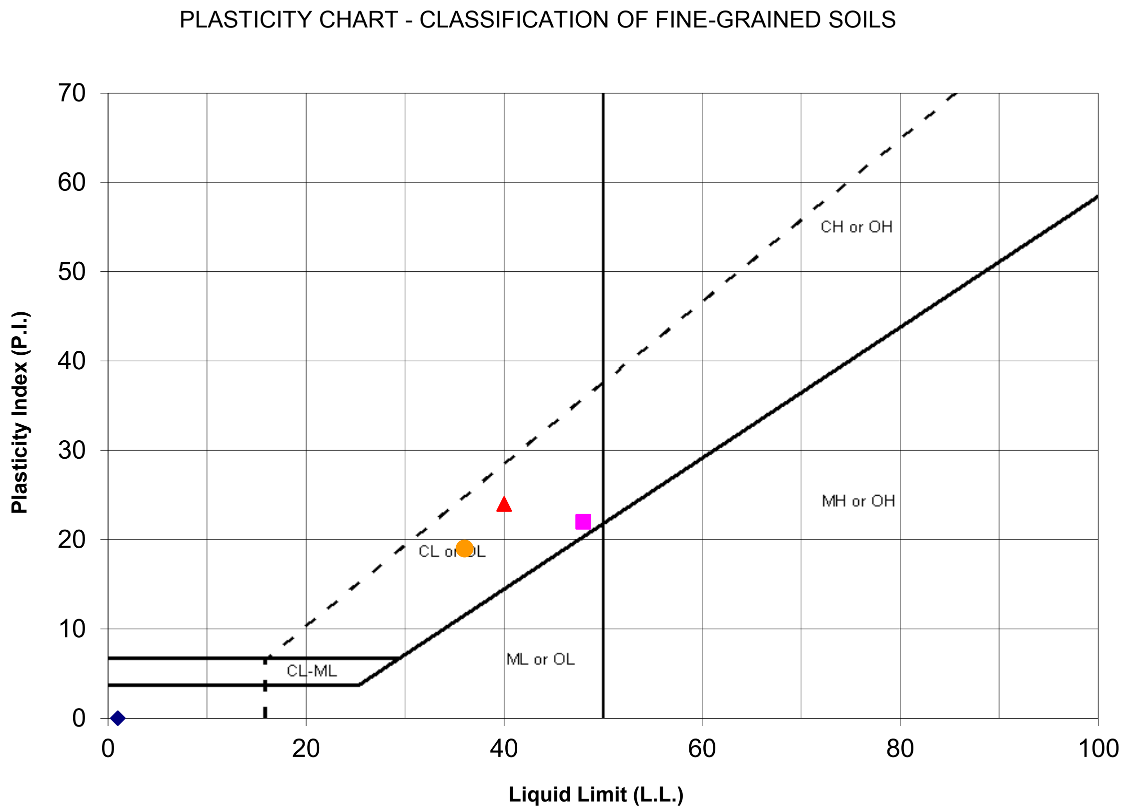




Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-4	**R-1**	**5**	**14.8**	**15.7**	**112.2**	**115.3**	**0.503**	**0.462**	**79**	**92**

Soil Identification: Yellowish brown lean clay (CL)

	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS **ASTM D 2435**	Project No.: 23169-01 Lennar - Greenbriar Lane 10-23



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	**HS-1**	**R-5**	**30'**	**19**	**NP**	**NP**	**NP**	**NP**
■	**HS-1**	**R-6**	**40'**	**95**	**48**	**26**	**22**	**CL**
▲	**HS-2**	**SPT-1**	**2.5'**	**-**	**40**	**16**	**24**	**CL**
●	**I-2**	**SPT-1**	**2.5'**	**-**	**36**	**18**	**18**	**CL**

LGC Geotechnical, Inc.

ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 23169-01
Date: Oct-23

Greenbriar Lane

Appendix D
Infiltration Testing Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lennar - Greenbriar Lane
Project Number: 23169-01
Date: 10/12/2023
Boring Number: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	10
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

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

Minimum test Head (D_o):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

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

*Pre-Test (Sandy Soil Criteria)**

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:18	8:43	25.0	8.25	8.38	0.13	No
2	8:44	9:09	25.0	8.31	8.44	0.13	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	9:10	9:40	30.0	8.20	8.30	0.1	0.2
2	9:41	10:11	30.0	8.20	8.30	0.1	0.2
3	10:12	10:42	30.0	8.25	8.36	0.1	0.2
4	10:43	11:13	30.0	8.26	8.36	0.1	0.2
5	11:14	11:44	30.0	8.24	8.35	0.1	0.2
6	11:45	12:15	30.0	8.25	8.36	0.1	0.2
7	12:16	12:46	30.0	8.19	8.30	0.1	0.2
8	12:47	13:17	30.0	8.21	8.30	0.1	0.2
9	13:18	13:48	30.0	8.17	8.29	0.1	0.2
10	13:49	14:19	30.0	8.22	8.31	0.1	0.2
11	14:20	14:50	30.0	8.20	8.29	0.1	0.2
12	14:51	15:21	30.0	8.21	8.30	0.1	0.2

Calculated Infiltration Rate (No factors of safety)	**0.2**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Orange County 12/20/2013

Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lennar - Greenbriar Lane
Project Number: 23169-01
Date: 10/12/2023
Boring Number: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	10
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

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

Minimum test Head (D_o):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

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

*Pre-Test (Sandy Soil Criteria)**

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:21	8:46	25.0	7.98	8.21	0.23	No
2	8:47	9:12	25.0	7.91	8.13	0.22	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	9:13	9:43	30.0	7.84	8.03	0.2	0.3
2	9:44	10:14	30.0	7.82	8.00	0.2	0.3
3	10:15	10:45	30.0	7.87	8.06	0.2	0.3
4	10:46	11:16	30.0	7.98	8.13	0.2	0.3
5	11:17	11:47	30.0	7.92	8.05	0.1	0.2
6	11:48	12:18	30.0	7.86	8.01	0.1	0.3
7	12:19	12:49	30.0	7.74	7.90	0.2	0.3
8	12:50	13:20	30.0	7.90	8.04	0.1	0.3
9	13:21	13:51	30.0	7.92	8.06	0.1	0.3
10	13:52	14:22	30.0	7.87	8.00	0.1	0.2
11	14:23	14:53	30.0	7.89	8.02	0.1	0.2
12	14:54	15:24	30.0	7.93	8.06	0.1	0.2

Calculated Infiltration Rate (No factors of safety)	**0.2**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Orange County 12/20/2013
Spreadsheet Revised on: 10/26/2016

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: Lennar - Greenbriar Lane
Project Number: 23169-01
Date: 10/12/2023
Boring Number: I-3

Test hole dimensions (if circular)	
Boring Depth (feet)*:	10
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

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

Minimum test Head (D_o):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

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

*Pre-Test (Sandy Soil Criteria)**

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:25	8:50	25.0	7.32	7.46	0.14	No
2	8:51	9:16	25.0	7.28	7.39	0.11	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Calculated Infiltration Rate(in/hr)
1	9:17	9:47	30.0	7.40	7.49	0.1	0.1
2	9:48	10:18	30.0	7.35	7.43	0.1	0.1
3	10:19	10:49	30.0	7.35	7.46	0.1	0.2
4	10:50	11:20	30.0	7.33	7.38	0.0	0.1
5	11:21	11:51	30.0	7.23	7.34	0.1	0.2
6	11:52	12:22	30.0	7.14	7.23	0.1	0.1
7	12:23	12:53	30.0	7.14	7.23	0.1	0.1
8	12:54	13:24	30.0	7.19	7.29	0.1	0.1
9	13:25	13:55	30.0	7.21	7.30	0.1	0.1
10	13:56	14:26	30.0	7.22	7.30	0.1	0.1
11	14:27	14:57	30.0	7.25	7.33	0.1	0.1
12	14:58	15:28	30.0	7.28	7.36	0.1	0.1

Calculated Infiltration Rate (No factors of safety)	**0.1**
Factor of Safety	**2.0**
Calculated Infiltration Rate (With Factor of Safety)	**0.1**

Sketch:

Notes:

Based on Guidelines from: Orange County 12/20/2013

Spreadsheet Revised on: 10/26/2016

LGC Geotechnical, Inc.

Appendix E
General Earthwork and Grading Specifications

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

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

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

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

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

2.2 Processing

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

2.3 Over-excavation

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

2.4 Benching

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

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 Import

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

4.0 Fill Placement and Compaction

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

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

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

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

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

LEGEND

afo	Artificial Fill Placed by Others
Qoa	Quaternary Older Alluvium, Circled Where Buried
10'	Estimated Remedial Grading Depth in Feet
HS-1 T.D. = 51.5'	Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet
I-1 T.D. = 10'	Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet
? ?	Approximate Location of Geologic Contact, Queried Where Uncertain
	Approximate Limits of This Report

*Conceptual Plan by Hunsaker, 2023

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Sheet 1
Geotechnical Map

PROJECT NAME	Lennar - Greenbriar, Brea
PROJECT NO.	23169-01
ENG. / GEOL.	RLD/KTM
SCALE	1" = 40'
DATE	November 2023

SHEET 1