PRELIMINARY GEOTECHNICAL EVALUATION

for

Mill Creek Drive and Ridge Route Drive
Laguna Hills, California

Prepared For:

Kingsbarn Realty Capital
1645 Village Center Circle, Suite 200
Las Vegas, Nevada 89134

Prepared By:

Langan CA, Inc.
18575 Jamboree Road, Suite 150
Irvine, California 92612

18 February 2026 (revised)
700128701

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Irvine, California 92612

REGISTERED PROFESSIONAL ENGINEER
WILSON W. WONG
NO. 3103
Exp. 9/30/2027
GEOTECHNICAL
STATE OF CALIFORNIA

Wilson Wong

Wilson Wong, GE
Associate Engineer

CERTIFIED ENGINEERING GEOLOGIST
SHAUN H. WILKINS
No. 2665
STATE OF CALIFORNIA

Shaun Wilkins

Shaun Wilkins
Senior Project Geologist

REGISTERED PROFESSIONAL ENGINEER
DIANE M FIORELLI
Diane M. Fiorelli
No. GE 3042
exp 6/30/27
GEOTECHNICAL
STATE OF CALIFORNIA

Diane M. Fiorelli

Diane M. Fiorelli, PE, GE
Principal

18 February 2026 (revised)
700128701

LANGAN

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EXECUTIVE SUMMARY

Langan CA, Inc. has performed a preliminary geotechnical evaluation for the proposed residential development located in Laguna Hills, California. The approximately 18½-acre site is bound by Mill Creek Drive to the west, Ridge Route Drive to the south, the lower portion of the downhill slope to the Veeh Reservoir to the east, and an office building to the northwest. The proposed development consists of 480 housing units consisting of 46 single units, 126 duplexes, 87 triplexes, and 221 multifamily units. The multifamily units will be wrapped around a multi-level parking structure. The project will also include a park, pools, surface parking, and other improvements. All proposed structures are planned to be at-grade.

A portion of the proposed development is within the FEMA Zone A, defined as a having a "1% Annual Chance Flood Hazard without base flood elevation" Therefore, we recommend that the project civil engineer confirm the base flood elevation during design and that the site grades meet the requirement to accommodate the base flood elevation including any required freeboard.

The subsurface conditions beneath the northern portion of the project site generally consist of shallow bedrock associated with the Sespe formation (Ts). In general, the existing Sespe formation (Ts) is considered suitable for support of the proposed structures and other improvements.

The subsurface conditions encountered during our exploration of the southern portion of the site include a variable thickness of fill or residual soil over Sespe formation bedrock. Considering the nature of the proposed construction and the subsurface conditions within the site, we recommend shallow foundations for the proposed apartment buildings and parking structures. However, the use of shallow foundations in areas of fill and residual soil could require overexcavation and recompaction of the fill and residual soil or other ground improvement. The quality and lateral extents of the fill should be characterized further as part of a design-level geotechnical investigation.

On the basis of the soluble sulfate percent, concrete can be designed as exposure class S0 for sulfate exposure. Per ACI 318-A, a minimum specified compressive strength (f'c) of 2,500 pounds per square inch (psi) may be used for foundation element and slabs (ASTM C150). A corrosion expert should be consulted during the design phase for the most economical and effective corrosion protection if ferrous site utilities are required.

The Site is in a seismically active region. The closest fault is approximately 3.5 miles away. The site is not located within a currently established Alquist-Priolo Earthquake Study Zone. The California Building Code should be followed with respect to seismic design.

Table 1 summarizes our preliminary recommendations.

TABLE 1

SUMMARY OF PRELIMINARY RECOMMENDATIONS FOR PROPOSED DEVELOPMENT

Design Item	Recommended Design Parameter
<u>Shallow Foundations</u>	
Allowable Bearing Pressure (for dead plus live loads)	2,500 to 4,500 psf (see Section 6.5.1 for recommendation where footings will span cut-fill transitions)
Minimum Footing Width	12 inches
Minimum Footing Embedment	24 inches or, where closer than 15 to 20 feet from the top of the slope, deepened so that distance to daylight meets the setback requirement (see Section 6.5.1)
Minimum Reinforcement	Structural engineer to design for medium expansion potential
Seismic Design Class (per 2022 CBC and ASCE 7-16)	Site Class D (additional future testing could support Site Class C for some buildings)
<u>Project Site Conditions:</u>	
Expansive Nature of Site Soils	Medium potential
Liquefaction Potential	Low potential
Corrosion Consideration	Sulfate exposure S0
Groundwater Depth	Groundwater was not encountered to the maximum depth explored, approximately 31 feet below existing ground surface (corresponding to approximate elevation 297 feet)
Design Groundwater Elevation	Design Groundwater Elevation 286 feet (NAVD88[1] vertical datum)

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[1] North American Vertical Datum of 1988

PRELIMINARY GEOTECHNICAL EVALAUTION
Mill Creek Drive and Ridge Route Drive
Laguna Hills, California

1.0 INTRODUCTION

As requested, and in accordance with our 2 February 2023 proposal and subsequent authorization by Kingsbarn Realty Capital, LLC, we have completed a preliminary geotechnical investigation for the proposed Mill Creek Drive Apartments located at Mill Creek Drive and Ridge Route Drive in Laguna Hills, Orange County, California (the Site). The purposes of this report are (1) to summarize our understanding of the geological and geotechnical aspects of the Site, (2) to document existing site conditions; (3) to summarize our subsurface investigation and findings; (4) to provide preliminary geotechnical recommendations for the proposed future development; and (5) to provide recommendations for future work in a design-level geotechnical investigation. Recommendations provided herein are in accordance with the 2022 California Building Code (2022 CBC) and the City of Laguna Hills Municipal Code.

2.0 PROJECT DESCRIPTION

2.1 Existing Conditions

The proposed development is located at the northeast corner of Mill Creek Drive and Ridge Route Drive in Laguna Hills, CA. The approximately 18½-acre site is on Orange County Assessor Tax Number (APNs) 588-161-06 to 588-161-10, 588-161-12, 588-161-13, 588-141-11, and 588-141-12. The site is bound by Mill Creek Drive to the west, Ridge Route Drive to the south, the lower portion of the downhill slope to the Veeh Reservoir to the northeast, and existing development to the north as shown on Figure 1, Site Location Map.

Ground surface elevations at the site range from approximate elevation 345 feet[2] to elevation 318 feet, sloping downward toward the south. The northeast portion of the property slopes downward towards the water line of the Veeh Reservoir to the northeast with a maximum height

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[2] Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023.

of approximately 60 feet (as measured from the top of slope to the water line of the reservoir shown in the topographic plan provided by Fuscoe Engineering, the project civil engineer). The slope ratio of this slope varies between approximately 1:1 to 3½:1 (horizontal to vertical).

2.2 Proposed Development

Our understanding of the proposed project is based on a concept site plan dated 16 December 2025[3]. According to the conceptual site plan, we understand Kingsbarn Realty Capital plans to redevelop the site in multiple phases including 480 housing units consisting of 46 single units, 126 duplexes, 87 triplexes, and 221 multifamily units. The multifamily units will be wrapped around a multi-level parking structure. The project will also include a park, pools, surface parking, and other improvements. All of the structures are planned to be at-grade.

According to correspondence with Fuscoe Engineering received 27 March 2023 we understand that, on a preliminary basis, proposed grades will be matched to existing grades and that cuts and fills to the existing building pads will likely be minimal. However, the northeast portion of the project as well as a section of the paseo/fire access lane appears to extend north of the limits of the current building and parking lot pad and, assuming a pad and finished pavement subgrade elevation of approximate el. 345 feet, will require up to 45 feet of new fill to be placed.

3.0 AVAILABLE INFORMATION REVIEW

Information that Langan reviewed included reports, maps, and other publicly available information from the United States Geological Survey (USGS), California Geological Survey (CGS), Orange County, Federal Emergency Management Agency (FEMA), California Geologic Energy Management Division (CalGEM), Groundwater Ambient Monitoring and Assessment Program (GAMA), the City of Laguna Hills, and data from our files.

A summary of the available information reviewed is provided in the sections below.

3.1 Regional and Local Geology

The site is located in Southern California in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is generally characterized by a series of northwest trending mountain

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[3] Conceptual plans titled "Terravista, Laguna Hills, CA" by Urban Arena dated 16 December 2025.

ranges and valleys. These topographic features are roughly parallel to the regional faults branching off of the San Andreas (e.g. Elsinore fault). Basement rook of the Peninsular Ranges is comprised of metamorphic rock with younger granitic intrusions.

Within the Peninsular Ranges, the site is located in the Santa Ana Mountains structural block. The Santa Ana Mountains block is bound to the north and northeast by the Whittier and Elsinore fault zones and to the south and southwest by the Pacific Ocean. The site lies just northeast of the San Joaquin Hills and south of a broad alluvial fan complex associated with the Santa Ana River. The area is characterized by a series of relatively low, rolling hills primarily underlain by Tertiary-aged sedimentary bedrock.

Regional geologic mapping indicates that the subject site is mostly underlain by the Sespe formation, an Eocene to early Miocene (56 to 23 million years ago) continental conglomeratic unit (Morton and Miller, 2006) (Figure 2). The Sespe formation is generally comprised of massive to thickly bedded conglomerate, sandstone, and clayey to silty sandstone. The northeastern limits of the site are within a mapped Quaternary alluvial fan deposit. Regional mapping describes this unit as Holocene and late Pleistocene (present to about 130,000 years) alluvial fan deposits comprised of unconsolidated to moderately consolidated silt and sand with varying pebbles and cobbles.

3.2 Geologic Hazards Review

Our geologic hazard review was performed in general accordance with CGS Special Publication 117A, “Guidelines for Evaluating and Mitigating Seismic Hazards in California,” dated 2008. The following subsections present the results of our hazard review.

3.2.1 Regional Faulting

Recognized and mapped faults that are the highest percentage contributor to the ground motion parameter values for the site are obtained from the USGS Unified Hazard tool. From our review, the fault that contributes the highest percentage to the probabilistic ground motion is the San Joaquin Hills Thrust Fault, located approximately 3.5 miles (5.6 kilometers (km)) northeast of the site.

The Site is located in an active seismic area that has historically been affected by generally high to occasionally very high levels of ground motion. Therefore, the proposed development will probably experience high to occasionally very high levels of ground motion from nearby faults as well as ground motions from other active seismic areas of the southern California region. See Figures 3A and 3B for a map of regional faults and the map legend, respectively.

3.2.2 Regional Seismicity

A search of the web-based USGS Advanced National Seismic System (ANSS) Comprehensive Earthquake Catalog (ComCat), accessed on 5 April 2023, found that 30 earthquakes with magnitudes of 5.0 or greater have occurred within a 100-km radius of the site since 1900. See Figures 3A and 3B for a map of faults and earthquake epicenters and the map legend, respectively

3.2.3 Surface Rupture

Earthquake Fault Zones are regulatory zones delineated by the CGS around known, active faults with the potential to cause surface rupture. The zones average approximately ¼-mile in width. According to the CGS Earthquake Zones of Required Investigation Map, the site is not within a mapped Alquist-Priolo Special Study Zone or a Fault Rupture Study Area. Figure 4 indicates that there are no mapped earthquake fault zones near the site.

3.2.4 Historic High Groundwater

According to CGS Seismic Hazard Zone Report, San Juan Capistrano Quadrangle (2001), the reservoir adjacent to the site has a historical high groundwater at 10 feet below the existing ground surface (bgs). The historical high groundwater map from this seismic hazard zone report is reproduced as Figure 5.

3.2.5 Liquefaction

Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses shear strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. Soil susceptible to liquefaction includes loose to medium-dense sands and gravels, low-plasticity silts, and some low-plasticity clay deposits below the groundwater table. According to CGS Earthquake Zones of Required Investigation, San Juan Capistrano Quadrangle (2001), the northeast section of the site borders a liquefaction hazard zone (Figure 4).

In our investigation, groundwater was not encountered within the maximum explored depth of approximately 31 feet. As discussed in Section 3.2.4, the recorded historic high groundwater data in the reservoir adjacent to the site is about 10 feet bgs. From the depth to historic high groundwater as well as the results of our investigation and engineering judgment, we preliminarily judge that a design groundwater elevation of el. 286 feet is appropriate. Subsurface conditions below the design groundwater elevation are anticipated to consist of Sespe formation bedrock that is not susceptible to liquefaction. Therefore, we preliminarily conclude that the liquefaction potential at the site is low.

3.2.6 Landslides

The CGS Earthquake Zones of Required Investigation, San Juan Capistrano Quadrangle (2001) map, indicates that a portion of the slope along the northern part the site is mapped within a potential earthquake-induced landslide zone (Figure 4). Regional geologic mapping and the CGS Landslide Inventory Map do not indicate the presence of a deep-seated landslide onsite. Please see Section 6.4.1 for conclusions from our preliminary landslide analyses.

3.2.7 Seismically-Induced Ground Deformations

Seismically-induced ground deformations include ground-surface settlement and differential settlement resulting from liquefaction-induced ground deformation and seismic densification of unsaturated sands and gravels from earthquakes. As discussed above, the site borders a liquefaction hazard zone and a portion of the site is mapped within an earthquake-induced landslide zone.

With the exception of the approximately 5½ feet of silty sand fill encountered in LB-3, the soil encountered above the groundwater table at the site generally consists of clayey or silty sandstone or stiff sandy clay to very dense clayey sand that are either cohesive and not subject to seismic densification or sufficiently dense to resist seismic densification. Therefore, we preliminarily conclude that the seismic densification potential at the site is low.

3.2.8 Lateral Spreading

Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a slope, by earthquake and gravitational forces. On the basis of the subsurface conditions encountered, we anticipate that the potential for liquefaction is low. Therefore, we also preliminarily judge that the potential for lateral spreading is low.

3.2.9 Flood Mapping

A review of the FEMA National Flood Hazard Layer, FIRM Panel 060590426J (2009) indicates that the northeast portion of the Site along the slope to the reservoir is located within Zone A, defined as a having a "1% Annual Chance Flood Hazard" without base flood elevation, and the southwest portion of the Site is located within Zone X, defined as having a "0.2% Annual Chance Flood Hazard." See Figure 6 for the FEMA Flood Hazard Map. The Project Civil engineer and architect should address the potential flood hazard as part of the site design.

3.2.10 Tsunami, Seiche, and Dam Inundation

A tsunami is a long, high sea wave caused by an earthquake, submarine landslide, or other disturbances. According to CGS Information Warehouse: Tsunami Hazard Area Map, the site is not within a mapped tsunami inundation hazard zone, and we therefore judge the potential for tsunami to be very low.

A seiche is an oscillation of surface water in an enclosed or semi-enclosed basin such as a lake, bay, or harbor. As noted previously, the property is in close proximity to a large body of water (The Veeh Reservoir, which borders the site to the east). The probability of the site being susceptible to inundation due to seiche is directly related to the height of the development above the water level in the reservoir. The current conceptual plans indicate that development is proposed close to the reservoir atop a plantable MSE wall. However, the exact height of this wall is not indicated. Therefore, the potential for seiche to affect the proposed development should be readdressed during a design-level geotechnical report once development plans have been better defined.

A review of the California Department of Water Resources Division of Safety of Dams California Dam Breach Inundation Maps web-viewer indicates that the site is not located within an inundation zone from a dam breach.

3.2.11 Subsidence

Land subsidence may be induced from withdrawal of oil, gas, or water from wells or from organic decomposition such as in a landfill or marsh area. According to a search on the CalGEM Well Finder online tool, no oil, gas, or geothermal wells are located within approximately 2½ miles of the site. Therefore, the site is not considered to be subject to land subsidence from oil, gas, or water withdrawal from oil wells. From our review of the site geology, the site is not on a former landfill or marsh area.

3.2.12 Expansive Soils

Expansive soils occur when the moisture content in the soil causes swelling or shrinking as a result of cyclic wet/dry weather cycles, installation of irrigation systems, change in landscape plantings, or changes in grading. Swelling and shrinking soils can result in differential movement of structures including floor slabs and foundations, and site work including hardscape, utilities, and sidewalks. The 2022 CBC defines potentially expansive soils as soils with expansion indices (EI) greater than 20. EI testing was performed as part of our investigation, the results of which are discussed in a later section of this report.

3.3 Previous Geotechnical Investigations

As part of our preliminary geotechnical investigation, we requested and reviewed available files for the site from the City of Laguna Hills archives. A geotechnical report for the prior development by EJN & Associates (EJN, 1986) were included with the data transmittal. Boring logs prepared by EJN are included in Appendix A.

From our review of cross sections prepared by EJN, we understand that the following grading was proposed as part of the development in the late 1980's

- the northern portion of the site was primarily cut, with cut depths of approximately 16 feet with shoulder fills up to approximately 12-feet-thick along the downhill slopes to Veeh Reservoir and Mill Creek Drive

- the central portion of the site was a combination of cuts up to 7 feet and fills up to approximately 6 feet

- the southern portion of the site was raised, with fills up to approximately 10 feet that were thickest along the top of the downhill slope to Veeh Reservoir.

According to the compaction reports prepared by EJN, the new fill for the office development was placed as engineered fills at a minimum of 90 percent of ASTM Test Method D1557. Ten-foot-wide keyways were excavated two feet into competent material (as determined by EJN's geotechnical engineer and engineering geologist) and benches were cut and prepared prior to the placement of fill.

3.4 Site History

As part of our preliminary geotechnical investigation, we reviewed historic aerial photos for the site area dated 1938, 1946, 1952, 1963, 1967, 1972, 1980, 1981, 1985, 1987, 1988, 1992 to 2000, 2002 to 2005, 2009, 2010, 2012, 2014, 2016, 2018, and 2020. Pertinent information from these photos are summarized as follows:

1938: The site is undeveloped and occupied by a northwest – southeast trending ridge. The slope down to the reservoir appears generally devoid of vegetation. The area is otherwise undeveloped. Orchards are visible north and east of the reservoir.

1946-1967: The photo depicts similar conditions to the 1938 map. Some trees are visible on the slope to the reservoir. Orchards surround the site

1972: The southeastern tip of the ridge has been removed.

1980: The ridge has been removed. The orchards surrounding the site have been largely replaced by development.

1981-1985: These photos depict similar conditions to the 1980 map. The 1985 photo shows the 23282 and 23272 Mill Creek Drive buildings to the north of the Project HERE site have been constructed.

1987: Some grading of the pads for the 24411 and 23422 Mill Creek Drive buildings appears to have been completed. The slope down to the reservoir is still largely devoid of vegetation

1988: The 24411 and 23422 Mill Creek Drive buildings as well as the appurtenant site improvements along the southern half of the project site have been constructed. Building pads and foundations for the 23382 and 23332 Mill Creek Drive buildings are in progress.

1992: The current office buildings and site improvements have been completed. Vegetation is visible on the slope down to the reservoir.

1993-current: The photo shows conditions generally similar to the 1992 photo.

We also reviewed topographic maps for the site dated 1948 for 2022. The topographic map from 1948 indicates that the boundary of the Veeh Reservoir wraps around the southeastern tip of the ridge along what is now Ridge Route Drive.

4.0 SURFACE AND SUBSURFACE INVESTIGATION

4.1 Langan's Subsurface Investigation

Langan's subsurface investigation consisted drilling of six (6) borings, identified as LB-1 to LB-4, PT-1, and PT-2, to depths ranging from approximately 11½ to 31 feet bgs at the approximate locations shown on Figure 7, Site Plan. Percolation tests were performed in PT-1 and PT-2. Prior to performing our subsurface investigation, the borings were located in the field and DigAlert was contacted to locate and mark public underground utilities at the Site. The borings were drilled on 16 March 2023 by Martini Drilling under the full-time engineering observation of a Langan field engineer.

In LB-1 through LB-4, samples were collected at 2½-foot-intervals within the upper 10 feet using a 3-inch-outer-diameter split barrel California sampler lined with 2.42-inch-inner-diameter brass rings. Below a depth of 10 feet, samples were collected with the California sampler or a 2-inch-outer-diameter standard penetration test (SPT) sampler. Soil samples were visually examined and classified in the field in accordance with the Unified Soil Classification System (USCS). Classifications were confirmed by re-examination at the laboratory. Representative samples were selected for testing. Details regarding the subsurface materials encountered are presented in the boring logs included in Appendix B.

In addition to our subsurface investigation, we also conducted a separate site visit to observe surficial slope conditions. Due to heavy vegetation covering the northerly descending slope toward Veeh Reservoir, site observations for geologic contacts or surficial sliding could not be readily obtained. However, we observed no apparent indicators of existing landslides based on aerial imagery, or field proxies, such as zones of dead vegetation - which can be an indicator of slope movement. Slope stability considerations are discussed further in Section 6.4.1.

4.2 Percolation Testing

Two (2) percolation tests were performed at depths of approximately 10 feet bgs; one in boring PT-1 and another boring PT-2. The percolation tests were performed in general accordance with the methods presented in the Orange County Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Water Quality Management Plans (2013). Percolation test results are included in Appendix C.

4.3 Laboratory Testing

The geotechnical laboratory testing was performed by GeoLogic Associates and included the following tests:

- Moisture Content – ASTM D2216
- Atterberg Limits – ASTM D4318
- Sieve Analysis – ASTM C136
- Direct Shear – ASTM D3080
- Expansion Index – ASTM D4829
- R-Value – ASTM D2844/CTM 301
- Electrical Resistivity – CTM 643
- Sulfate Content – CTM 417
- Chloride Content – CTM 422
- Soil pH – CTM 643

The laboratory test results are included in Appendix D.

5.0 SUBSURFACE CONDITIONS

In general, subsurface conditions below the Site generally consist of shallow bedrock of the Sespe formation (Ts) overlain by up to 17 feet of soil consisting of either fill or native, residual soil. Our interpretation of the subsurface conditions based in the available boring logs (EJN & Associates and Langan) is summarized below.

- **Fill** – As noted above, between 5½ and 13 feet of fill is currently present below portions of the site and was encountered in borings LB-3, LB-4, PT-1, and PT-2. The fill is generally comprised of silty to clayey sand or silt or stiff to very stiff sandy clay. Direct shear laboratory test results on a sample of the fill collected during the current investigation indicates the sandy clay fill has a friction angle of about 30.5 degrees and a cohesion of 150 pounds per square foot (psf).

- **Native soil** – EJN borings 1, 2, 4, 5, and 7 to 9 encountered between 1½ and more than 10 feet of native soil comprised of clay with varying amounts of sand and silt or sand with varying amounts of clay. The native soil encountered in EJN borings 1 and 2 was likely removed as part of the grading for the existing development. The native soil encountered in EJN borings 4, 5, and 7 to 9 likely remained in place as part of grading for the existing development. Langan boring LB-4 encountered approximately three feet of native (residual) soil comprised of stiff sandy clay.

- **Sespe formation (Ts)** – Sespe formation bedrock is present throughout the site. The bedrock consists of red to tannish brown, non-marine clayey to silty sandstone that extends to the maximum explored depth of approximately 31 feet bgs. A corrosion test was performed on bag sample of the Sespe formation collected from boring LB-2. An expansion index test (EI) performed on bag sample of the Sespe formation collected from boring PT-1 indicates that the material has medium expansion potential, with an expansion index of 57. Direct shear laboratory test results on ring samples of the bedrock indicate the material has ultimate friction angles of about 28.5 to 31 degrees with cohesion of 100 to 200 psf. Laboratory test results are included in Appendix B.

- **Groundwater** – Groundwater was not encountered within the maximum explored depth of about 31 feet.

APNs 588-141-11 and 588-141 12 were added to the project after the completion of our subsurface investigation. Though we anticipate that the parcel will be underlain by Sespe formation bedrock, the subsurface conditions should be confirmed as part of a design level geotechnical investigation.

6.0 SEISMIC EVALUATION AND GEOTECHNICAL RECOMMENDATIONS

6.1 Seismic Design Parameters

On the basis of our evaluation of the subsurface conditions at the site, we preliminarily conclude that the site may be characterized as Seismic Site Class D, in accordance with Chapter 20 of ASCE 7-16. As such, the following preliminary seismic design criteria may be used.

Type	Value	Description
S_S	1.222	MCE_R mapped spectral response acceleration at short period
S_1	0.439	MCE_R mapped spectral response acceleration at one-second period
F_a	1.011	Site Amplification Factor at 0.2 second
F_v	2.79	Site Amplification Factor at 1.0 second
S_{MS}	1.236	Site-modified spectral acceleration at short period
S_{M1}	1.225	Site-modified spectral acceleration at one-second period
S_{DS}	0.824	Design earthquake spectral response acceleration at short period

Type	Value	Description
S_{D1}	0.817	Design earthquake spectral response acceleration at one-second period
PGA_M	0.565	MCE geometric mean peak ground acceleration adjusted for site class effects

Notes:

1. Values based on Site Class D.
2. Values of F_v, S_{M1}, and S_{D1} assume the exceptions of ASCE 7-16 Section 11.4.8 are met and have been increased by 50 percent per Supplement 3 of ASCE 7-16.
3. MCE = Maximum Considered Earthquake.
4. MCE_R = Risked-Targeted Maximum Considered Earthquake.

The recommended mapped values of F_v, S_{M1}, and S_{D1} for Site Class D have been increased by 50 percent in accordance with the exception of Section 11.4.8.1 of Supplement No. 3 to ASCE 7-16. If the structural engineer elects not to use this exception in the seismic design approach, we should be notified so that we may develop site-specific response spectra and seismic design criteria in accordance with Chapter 21 of ASCE 7-16.

The design-level geotechnical investigation should include additional subsurface exploration of the site to better understand the subsurface conditions to establish it is appropriate to classify a portion of the site as Site Class C.

6.2 Flood Design

As discussed in Section 3.2.9, a portion of the proposed development is within the FEMA Zone A, defined as a having a "1% Annual Chance Flood Hazard" with base flood elevation or depth not determined but is likely associated with the reservoir spillway elevation. The civil engineer should confirm that the site grades consider the base flood elevation and any required freeboard.

6.3 Groundwater Considerations

As discussed in Section 5.0, groundwater was not encountered within the maximum explored depth of approximately 31 feet during our subsurface investigation. The recorded historic high groundwater noted during our records research data from the site vicinity is about 10 feet bgs. However, we judge that the mapped depth to historic high groundwater likely applies to the reservoir and not to the Project HERE site. Our preliminary geotechnical evaluation did not encounter groundwater above approximately el. 297 feet and the topographic plan provided by Fuscoe indicates that the water level in Veeh Reservoir is approximately el. 283 feet. On the basis of the data from the preliminary geotechnical evaluation and the water level in the reservoir, we judge that a design groundwater elevation of el. 286 feet is appropriate for design.

Some seepage through the Sespe formation bedrock or between the top of bedrock and soil should be anticipated.

6.4 Seismic and Geologic Hazards

During a major earthquake, high to very high ground shaking is expected to occur at the project site. Ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4] and cyclic softening[5], lateral spreading,[6] seismic densification[7], and fault rupture. Each of these conditions has been evaluated based on our literature review, field investigation and analysis, and are discussed in Section 3.2 or in this section.

On the basis of the deaggregation of the probabilistic seismic hazard spectrum from the USGS Unified Hazard Tool, the mean and mode earthquakes for the 2 percent probability of exceedance in 50 years (2,475-year return period) event are 6.71 and 6.89 moment magnitudes, respectively.

6.4.1 Slope Stability

Based on review of historic aerials from 1938 through 2020, there is no apparent history of landsliding onsite. Based on limited site observations above existing slopes, there is no apparent evidence of slope instability.

Records provided by the City of Laguna Hills indicate slope stability analysis for portions of the northerly descending slope was performed by EJN and Associates in March and May of 1986 in support of the existing development and they concluded that the slope is generally stable under static (i.e. non-seismic) conditions.

Since the time of their investigation, updated regulations and standard of practice regarding slope stability and analysis have become more conservative, including analysis for earthquake induced

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[4] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[5] Cyclic softening is a phenomenon in which soil loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced loading, but has sufficient internal cohesion to resist complete liquefaction.

[6] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[7] Seismic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

landsliding, often referred to as a pseudostatic analysis. A quantitative analysis of the slope stability under static and pseudostatic conditions for the downhill slope into Veeh Reservoir should be performed as part of a design-level geotechnical investigation. The supplemental analysis will likely require additional subsurface evaluation including mapping of geologic structure of the bedrock in deep borings and/or on the slope surface so that geologic cross section(s) of the descending slope can be created and evaluated for static and pseudostatic conditions.

6.5 Preliminary Foundation Evaluation and Recommendations

From a geotechnical standpoint, the proposed site development is considered feasible provided the site conditions and geotechnical issues in this report are properly addressed during the design and construction of the proposed improvements. However, additional subsurface information should be gathered during a design-level geotechnical investigation to better characterize the quality, depth, and lateral extents of the fill and to evaluate the stability of the proposed development and associated slopes and/or retaining walls.

From our preliminary discussions with Fuscoe Engineering, we understand that the grades for the proposed development will be at, or near, existing grades, with the exception of the northeastern portion of the project and the paseo/fire access lane, where up to 45 feet fill could be required. If confirmed, we will need to provide supplemental recommendations during final once grading plans are available.

On the basis of our evaluation of the subsurface data and our understanding of the proposed development, foundations are expected to be underlain by Sespe formation, fill, or native soil, which is generally suitable for support of the structure. From our review of the subsurface information as well as our geotechnical exploration, we anticipate the proposed structures can supported on shallow foundations such as spread footings or a mat foundation. Additional foundation information is provided below.

6.5.1 Shallow Foundations

- **Spread Footings or Continuous Footings:** A preliminary bearing value of 2,500 to 4,500 psf may be used for continuous and isolated footings bearing a minimum depth of 24 inches below the lowest adjacent grade and having a minimum width of 12 inches. Transition cut/fill building areas with shallow fill (less than 5 feet thick) should be overexcavated to a minimum depth of 5 feet in order to provide a uniform fill blanket for foundation support. In addition, building footprints underlain by rock should be overexcavated a minimum depth of 5 feet.

Footings nearest the top of the downhill slope to the Veeh Reservoir can be deepened based on the recommendation provided in Section 6.13. Recommended allowable bearing values include both dead and live loads and may be increased by one-third for wind and seismic forces. The parking structures could require partial mat foundations under shear walls to reduce the bearing pressures to the recommended allowable values. Recommendations for mat foundations are provided in Section 6.5.2.

Footing static settlement of approximately one inch and differential settlements of up to ½-inch over 50 feet are anticipated with foundations bearing on appropriately prepared engineered fills. Seismically-induced settlements are not anticipated based on the available subsurface data.

Footing excavations should be performed using a backhoe bucket fitted with a smooth steel plate welded across the bucket teeth to minimize disturbance during excavation and to provide a smooth bearing surface.

The footing subgrades should be firm and unyielding, inspected and approved by a qualified geotechnical engineer prior to steel or concrete placement.

Foundations should be constructed as soon as possible following subgrade approval. The contractor shall be responsible for maintaining the subgrade in its approved condition (i.e. free of water, debris, etc.) until the footing is constructed.

- **Lateral Resistance:** Lateral loads on footings bearing on appropriately prepared engineered fills or bedrock can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along the bases of the footings. We recommend a passive resistance be calculated using a lateral pressure corresponding to an equivalent fluid pressure of 250 pounds per cubic foot (pcf); the upper foot of soil should be ignored unless confined by a concrete slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.25. The passive resistance and base friction values include a factor of safety of about 1.5 and can be used to resist total loads (including wind and/or seismic loads). They may be used in combination without reduction.

6.6 Slabs-on-Grade

The building pads could span transitions between fill and bedrock at the exposed subgrade (pad) level. Even if the fill is compacted based on the recommendations provided in Section 7.2, we anticipate that the compressibility of the fill will differ from the compressibility of bedrock. Therefore, we recommend that the slab on grade be cast over a minimum of 5 feet of new material, which could consist of 56 inches of engineered fill and a four-inch-thick capillary moisture break layer below the floor slab. If the pad subgrade is disturbed during excavation for footings and utilities, it should be re-rolled. Loose, disturbed materials should be excavated, removed, and replaced with engineered fill during final subgrade preparation.

Steel reinforcing and concrete thickness should be designed by the project Structural Engineer for soils with a medium expansion potential (50 < EI < 90). We recommend, at a minimum, using the following recommendations:

- subgrade modulus, K, equal to 120 pounds per cubic inch (pci); and
- 4-inch minimum thickness.

For moisture-sensitive floor areas, a moisture barrier, consisting of a 15-mil polyethylene water vapor retarder over a minimum of four inches of capillary break as required by 2022 CBC Section 1805.4.1, shall be placed between the base course and concrete floor slab. The water vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745 and should be placed in accordance with the requirements of ASTM E1643. The capillary break should consist of open-graded, free draining, virgin material with a gradation that meets the requirements of the 2022 CBC.

6.7 Lateral Earth Pressure

Although grading plans were not available at the time of preparation of this preliminary evaluation, we recommend that any proposed retaining walls be designed to resist soil and surcharge pressures using the parameters below. The project should include the replacement of the crib wall along a portion of the northwest perimeter of the site.

- Ultimate Coefficient of Friction = 0.35
- Soil Unit Weight = 130 pounds per cubic foot (pcf)
- Friction Angle = 28.5 degrees

- Equivalent Fluid Pressure (At-Rest Condition / Restrained Wall): Drained and Above Design Groundwater Level = 70 psf/foot

- Equivalent Fluid Pressure (Active Condition / Unrestrained Wall): Drained and Above Design Groundwater Level = 45 psf/foot

- Hydrostatic Equivalent Fluid Pressure = 62.4 psf/foot

- The design of any retaining walls greater than six feet in height should consider the additional earth pressures caused by seismic ground shaking. The seismic force increment is estimated based on the design earthquake and may be considered to act as a triangular distribution pressure equal to 30 psf/foot. The seismic increment should be added to the active (unrestrained) lateral earth pressures when considered for below grade walls.

- At-rest, active, passive, and seismic thrust increment should be considered to follow a triangular distribution.

- Lateral loads from surcharges on retaining walls may be considered to impart surcharges to the restrained walls using an earth pressure coefficient of ½ for restrained walls presuming a uniform distribution. Surcharge loading from adjacent structures should be considered where the adjacent foundations are supported on the soil above a 1H:1V theoretical influence line projected upwards from the base of the retaining wall.

- Surcharge loading should consider adjacent streets, vehicular traffic, and sidewalks. Where vehicular traffic will pass within 10 feet of retaining walls, temporary traffic loads should be considered in the design of walls. Traffic loads such as fire trucks or cars parked on the street beyond the sidewalk may be modeled by a minimum uniform pressure of 100 psf/foot applied on the upper 10 feet of the walls.

- A wall drainage system, such as uniformly spaced prefabricated drainage panels connected to a toe drain, should be installed behind retaining grade walls to reduce the potential for hydrostatic pressure build up. The toe drain should be sloped to drain into an appropriate outlet. The design of the retaining walls should include the hydrostatic equivalent fluid pressure if they will not be drained.

6.8 Preliminary Pavement Sections

On the basis of the soil conditions present at the Site and estimated traffic volume, preliminary pavement sections are provided in the following table. We selected an R-value of 7 for preliminary design based on the lab test results.

The sections provided herein are for planning purposes only and should be re-evaluated subsequent to site grading. Final pavement sections should be based on actual R-value testing of in-place soils and analysis of anticipated traffic. Preliminary flexible pavement sections for parking areas have been developed with the parameters summarized on the following table.

Pavement Area	**Traffic Index**	**Section Thickness**	
		Asphalt Concrete	**Class 2 Aggregate Base**
Parking Areas with Occasional Trucks	4.0	4 inches	6 inches
Driveways and Truck-Use Areas	6.0	4 inches	12 inches

Aggregate base materials should be Crushed Aggregate Base or Crushed Miscellaneous Base conforming to Section 200-2 of the Standard Specification for Public Works Construction (Greenbook) or Class 2 Aggregate Base conforming to the Caltrans' Standard Specifications. The materials should be moisture conditioned to slightly over the optimum moisture content then compacted to at least 95 percent of ASTM Test Method D1557, latest edition.

Prior to placement of pavement elements, the upper two feet of subgrade soils should be moisture-conditioned to within 2 percent of the optimum moisture content then compacted to at least 95 percent of the laboratory determined maximum dry density. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding compacted soil or aggregate base materials.

Non-vehicular concrete pavements (including exterior slabs and hardscape) should have a nominal thickness of at least 4 inches and be supported on compacted subgrade and at least 6 inches of Class 2 Aggregate Base compacted to at least 90 percent relative compaction. Cold joints or saw cuts in the concrete should be provided at least every 15 feet in each direction.

6.9 Corrosion Considerations

Chemical analyses performed on the near-surface soil are summarized in the following table. A copy of the corrosion test results is provided in Appendix D.

Soil Type	**Boring ID**	**Resistivity (ohm-cm)**	**pH**	**Sulfate (ppm)**	**Chloride (ppm)**
Clayey Sandstone (0 to 5 feet)	LB-2	6,200	7.5	106	9

A corrosion expert should be consulted during the design phase for the most economical and effective corrosion protection if ferrous site utilities are required. On the basis of the soluble sulfate percent, concrete can be designed as exposure class S0 for sulfate exposure. Per ACI 318-A, no special recommendation for cement, but a minimum specified compressive strength (f’c) of 2,500 pounds per square inch (psi) may be used for foundation element and slabs (ASTM C150).

6.10 Site Infiltration

The infiltration rate of the soils at the locations tested is summarized in the table below. The percolation tests suggest the soil and bedrock at tested locations were not permeable. When final drywell or infiltration structure locations are confirmed, supplemental testing may be required to meet North Orange County Percolation Test Requirements. Percolation Test results are included in Appendix C.

Boring ID	Test Depth (feet)	Soil Type	Infiltration Rate (inches/hour)
PT-1	10	CL	0.0
PT-2	10	SC	0.0

All proposed storm water infiltration systems should be located at least 50 feet away from any settlement sensitive structures and the downhill slope to Veeh Reservoir. If the storm water infiltration systems need to be located less than five feet from the edge of the new foundations, we recommend footings adjacent to storm water infiltration systems bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the storm water infiltration system areas. In addition, the passive resistance against the vertical face of the footing should be ignored. The structural engineer will need to confirm that this is acceptable to their foundation design.

6.11 California Building Code Required Slope Setbacks

For buildings on top of a slope, the Orange County Grading Manual requires building foundations be set back a minimum horizontal distance of H/3, where H is the height of the slope, but with a horizontal setback of no more than 40 feet. For this Site, the adjacent slopes (estimated from top of slope to the waterline of the reservoir) vary between approximately 45 and 60 feet tall, so we anticipate that a minimum setback of 15 to 20 feet will be required. Alternatively, the foundations can be deepened such that the horizontal distance between the foundation and the face of the slope (distance to daylight) meets the setback requirement. We note that the estimated setback does not account for the height of the slope below the water line of Veeh Reservoir as shown in the topographic plan provided by Fuscoe.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Site Preparation and Grading

Prior to the commencement of mass excavation and grading, a meeting should be held at the Site with the owner, city inspector, excavation/grading contractor, civil engineer, and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading.

All vegetation and deleterious materials should be disposed of off-site prior to initiation of grading operations.

All surficial units consisting of any soil with roots or loose surficial soil are considered unsuitable for support of the proposed improvements. These materials should be over-excavated to expose competent soils. These over-excavated soils, free of deleterious materials and approved by the geotechnical engineer, may be reused as compacted fill. These surficial soil materials are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment.

Zones of harder bedrock should be anticipated during site preparation. Most of these materials are below optimum moisture content and will require the addition of water to achieve proper compaction.

The geotechnical consultant should be provided with appropriate survey staking during grading to verify that depths and locations of recommended over-excavations have been achieved. Observations of over-excavations should be performed by the geotechnical engineer to verify the anticipated conditions. All excavations should conform to the requirements of CAL/OSHA.

All over-excavation bottoms should be observed by the geotechnical engineer prior to fill placement. Prior to placement of fill material, the over-excavation bottom should be scarified to a depth of at least six inches, moisture conditioned to above optimum moisture content, and proof-rolled.

Any foundation remnants or construction debris encountered within excavations should be fully removed, and any void spaces that may be created should be backfilled with approved compacted structural fill. Private sewage systems, if encountered during grading should be properly removed or abandoned in place in accordance with local codes. If septic systems or seepage pits are encountered, and they are abandoned in-place, they should be pumped clean, backfilled with gravel or clean sand and capped with a minimum of two feet of 2-sack slurry. The top of the slurry cap should be at 5 feet below proposed grade.

Any environmentally unsuitable soils encountered during the excavation process should be properly disposed of off-site in accordance with all state and local regulations.

If removals are limited by existing improvements or property lines, special grading techniques, such as slot cuttings or other acceptable construction methods may be required. Under such conditions, specific recommendations should be provided by the geotechnical consultant during review of final grading plan.

7.2 Fill Material and Compaction Criteria

Fill material (imported or re-used) should be free of organic, and other deleterious materials and have a maximum particle size no greater than 3 inches. Imported fill should contain no more than 12 percent passing the no. 200 sieve by dry weight and have a plasticity index less than 7. Grain-size distributions, Atterberg Limits, maximum dry density, and optimum water content determinations should be made on representative samples of the proposed fill material.

Engineered fill below building foundations and behind retaining walls should be placed in uniform lifts (maximum 8-inches thick) and compacted to at least 95 percent of its maximum dry density at a moisture content above optimum moisture content, as determined by ASTM D1557.

All fill placed below non-vehicular flatwork should be compacted to a minimum of 90 percent of its maximum dry density at a moisture content above optimum moisture content, as determined by ASTM D1557. All fill placed below vehicular pavement or fill thicker than five feet should be compacted to a minimum of 95 percent of its maximum dry density at a moisture content above optimum moisture content, as determined by ASTM D1557. Fill placement should be subject to controlled full-time engineering observation and testing by the geotechnical engineer. Fill material should not be placed in areas where free water is standing or on surfaces which have not been approved by the geotechnical engineer.

Preliminarily, keyways and benches should be anticipated below the fill slopes that will support the north end of Building 4 and the paseo/fire access lane.

7.3 Site Drainage

Proper drainage should be maintained at all times. Ponding or trapping of water in localized areas can cause differing moisture levels in the subsurface soil. Drainage should be directed away from the tops of slopes and excavations. Erosion protection and drainage control measures should be implemented during periods of inclement weather. During rainfall events, backfill operations may need to be restricted to allow for proper moisture control during fill placement.

Although groundwater was not encountered during explorations performed for this study, shallow perched water may be encountered at the Site depending on seasonal rainfall. The Site should be graded to ensure positive drainage away from the locations of the proposed development.

8.0 FUTURE STUDIES AND DESIGN AND CONSTRUCTION PHASE SERVICES

The conclusions and preliminary recommendation provide herein are based on project information provided to date and a limited number of borings. As noted in Section 5.0, APNs 588-141-11 and 588-141 12 were added to the project after the completion of our subsurface investigation. Though we anticipate that the parcel will be underlain by Sespe formation bedrock, the subsurface conditions should be confirmed as part of a design level geotechnical investigation. As part of schematic design, a design-level geotechnical investigation and evaluation should be provided when structural loads are available. The design-level geotechnical investigation should include additional exploratory borings that extend below the proposed foundation level to confirm the subsurface conditions.

During final design we should be retained to consult with the design team as geotechnical questions arise. Technical specifications and design drawings should incorporate Langan's recommendations. When authorized, Langan will assist the design team in preparing specification sections related to geotechnical issues such as earthwork, shallow foundations, backfill and excavation support. Langan should also, when authorized, review the project plans, as well as Contractor submittals relating to materials and construction procedures for geotechnical work, to confirm the designs incorporate the intent of our recommendations.

Langan has investigated and interpreted the site subsurface conditions and developed the foundation design recommendations contained herein and is therefore best suited to perform quality assurance observation and testing of geotechnical-related work during construction. The work requiring quality assurance confirmation and/or special inspections per the Building Code includes, but is not limited to, earthwork, backfill, shallow foundations, and excavation support.

Recognizing that construction observation is the final stage of geotechnical design, quality assurance observation during construction by Langan is necessary to confirm the design assumptions and design elements, to maintain our continuity of responsibility on this project, and allow us to make changes to our recommendations, as necessary. The foundation system and general geotechnical construction methods recommended herein are predicated upon Langan assisting with the final design and providing construction observation services for the Owner. Should Langan not be retained for these services, we cannot assume the role of geotechnical engineer of record, and the entity providing the final design and construction observation services must serve as the engineer of record.

9.0 OWNER AND CONTRACTOR RESPONSIBILITIES

The Contractor is responsible for construction quality control, which includes satisfactorily constructing the foundation system and any associated temporary works to achieve the design intent while not adversely impacting or causing loss of support to neighboring structures. Construction activities that can alter the existing ground conditions such as excavation, fill placement, foundation construction, ground improvement, etc. can also potentially induce stresses, vibrations, and movements in nearby structures and utilities, and disturb occupants of nearby structures. Contractors working at the Site must ensure that their activities will not adversely affect the performance of the structures and utilities and will not disturb occupants of nearby structures. Contractors must also take all necessary measures to protect the existing structures during construction. By using this report, contractors agree that Langan will not be held responsible for any damage to adjacent structures.

10.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the Site inferred from a limited number of borings. Recommendations provided are contingent upon one another and no recommendation should be followed independent of the others.

Any proposed changes in structures or their locations should be brought to Langan's attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata shown on the logs represents conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation, as they may affect our recommendations.

This report has been prepared to assist the owner in their site selection process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities (including underpinning, grouting, stabilization, etc.) on adjacent properties which are beyond the limits of that which is the specific subject of this report.

Environmental issues (such as permitting or potentially contaminated soil and groundwater) are outside the scope of this study and should be addressed in a separate evaluation.

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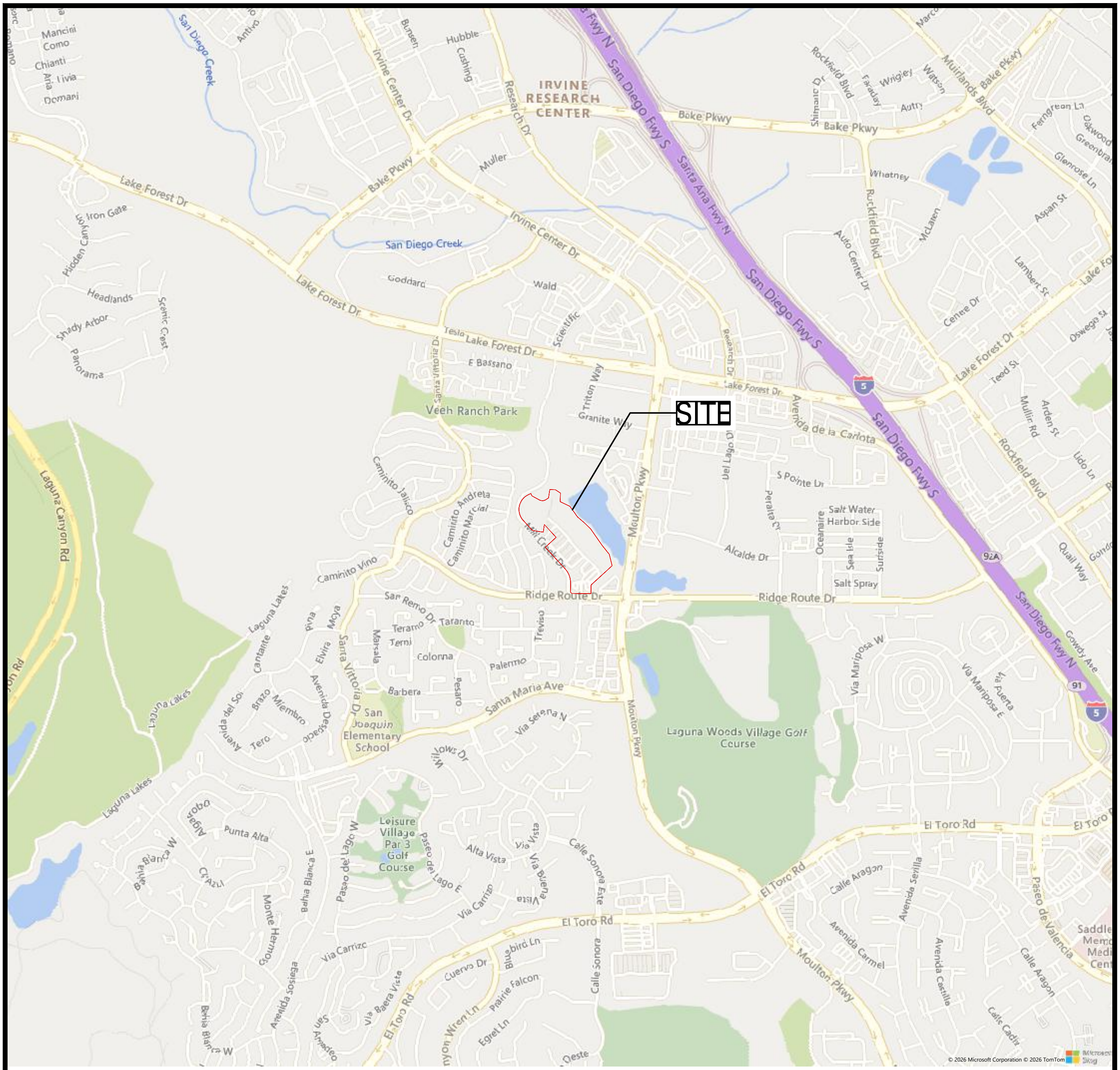
FIGURES



LEGEND:

🟥	SITE LIMITS

2,000 1,000 0 2,000

SCALE IN FEET

REFERENCE: BING MAP ACCESSED ON 18 FEBRUARY 2026.

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Langan CA, Inc.

18575 Jamboree Road, Suite 150
Irvine, CA 92612

T: 949.561.9200 F: 949.561.9201 www.langan.com

Project

PROJECT HERE
MILL CREEK DRIVE AND
RIDGE ROUTE DRIVE

LAGUNA HILLS

ORANGE COUNTY CALIFORNIA

Figure Title

SITE LOCATION MAP

Project No.
700128701

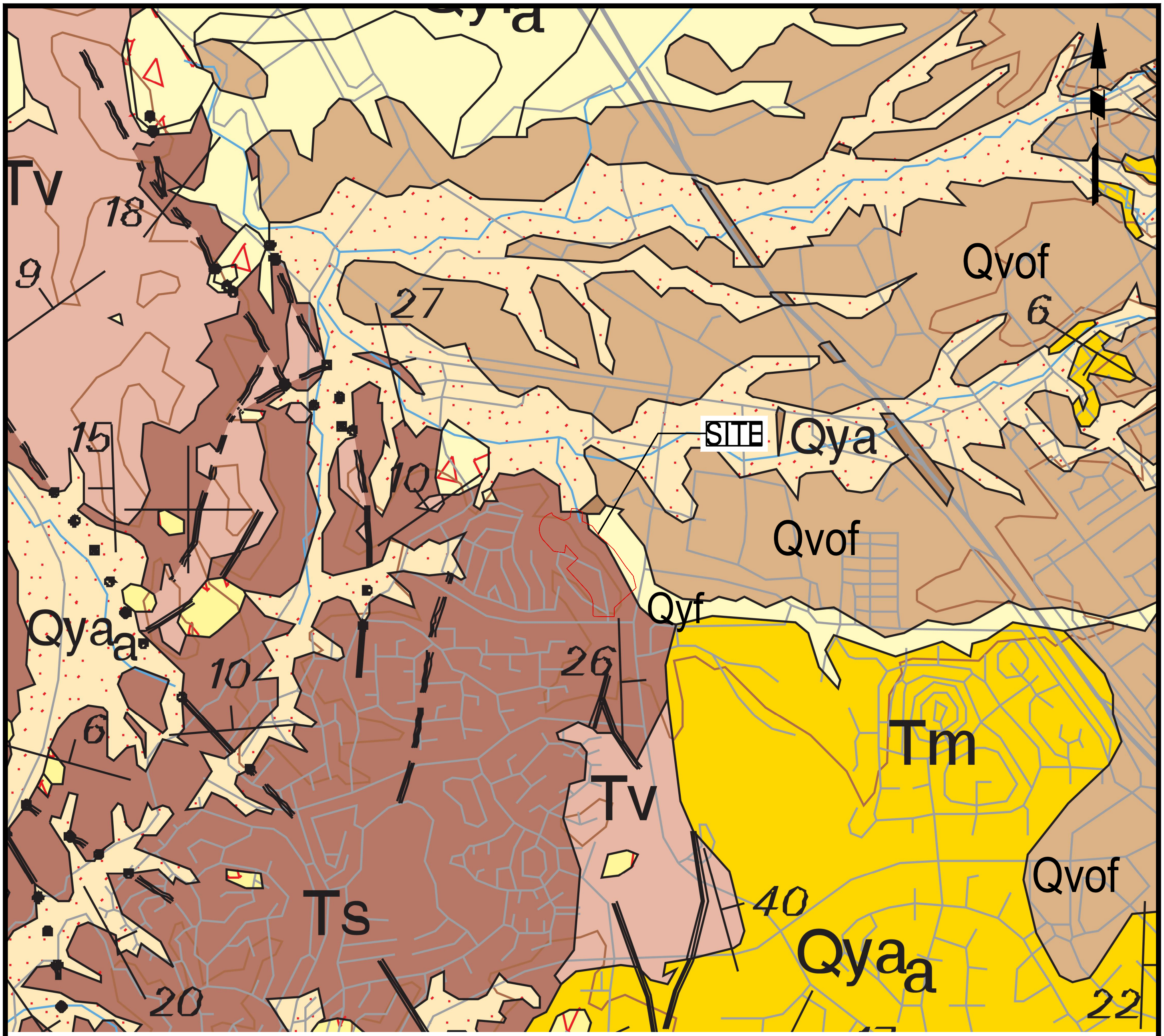
Date
FEBRUARY 2026

Scale
AS SHOWN

Drawn By
JX

Figure No.
1

Filename: \\langan.com\data\IRV\data7\700128701\Project Data_Discipline\Geotechnical\CAD\phase 1\NEW CAD FILES_5-30-2023\4014-002ALT.dwg Date: 2/18/2026 Time: 14:43 User: vramirez Style Table: Langan.stb Layout: Layout1



LEGEND:

━━	SITE LIMITS
Qya	YOUNG AXIAL-CHANNEL DEPOSITS
Qyf	YOUNG ALLUVIAL FAN DEPOSITS
Qvof	VERY OLD ALLUVIAL FAN DEPOSITS
Tm	MONTEREY FORMATION
Tv	VAQUEROS FORMATION
Ts	SESPE FORMATION
══ • • • •	BEDROCK (INACTIVE) FAULT - SOLID WHERE ACCURATELY LOCATED, DASHED WHERE APPROXIMATELY LOCATED OR INFERRED; DOTTED WHERE CONCEALED
10 ┴	STRIKE AND DIP OF BEDS - INCLINED

2,000 1,000 0 2,000

SCALE IN FEET

REFERENCE: GEOLOGIC MAP OF THE SAN BERNARDINO AND SANTA ANA 30' X 60' QUADRANGLES, CALIFORNIA, D.M. MORTON AND F.K. MILLER (2006).

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Langan CA, Inc.

18575 Jamboree Road, Suite 150
Irvine, CA 92612

T: 949.561.9200 F: 949.561.9201 www.langan.com

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Figure Title

REGIONAL
GEOLOGIC MAP

Project No.
700128701

Date
FEBRUARY 2026

Scale
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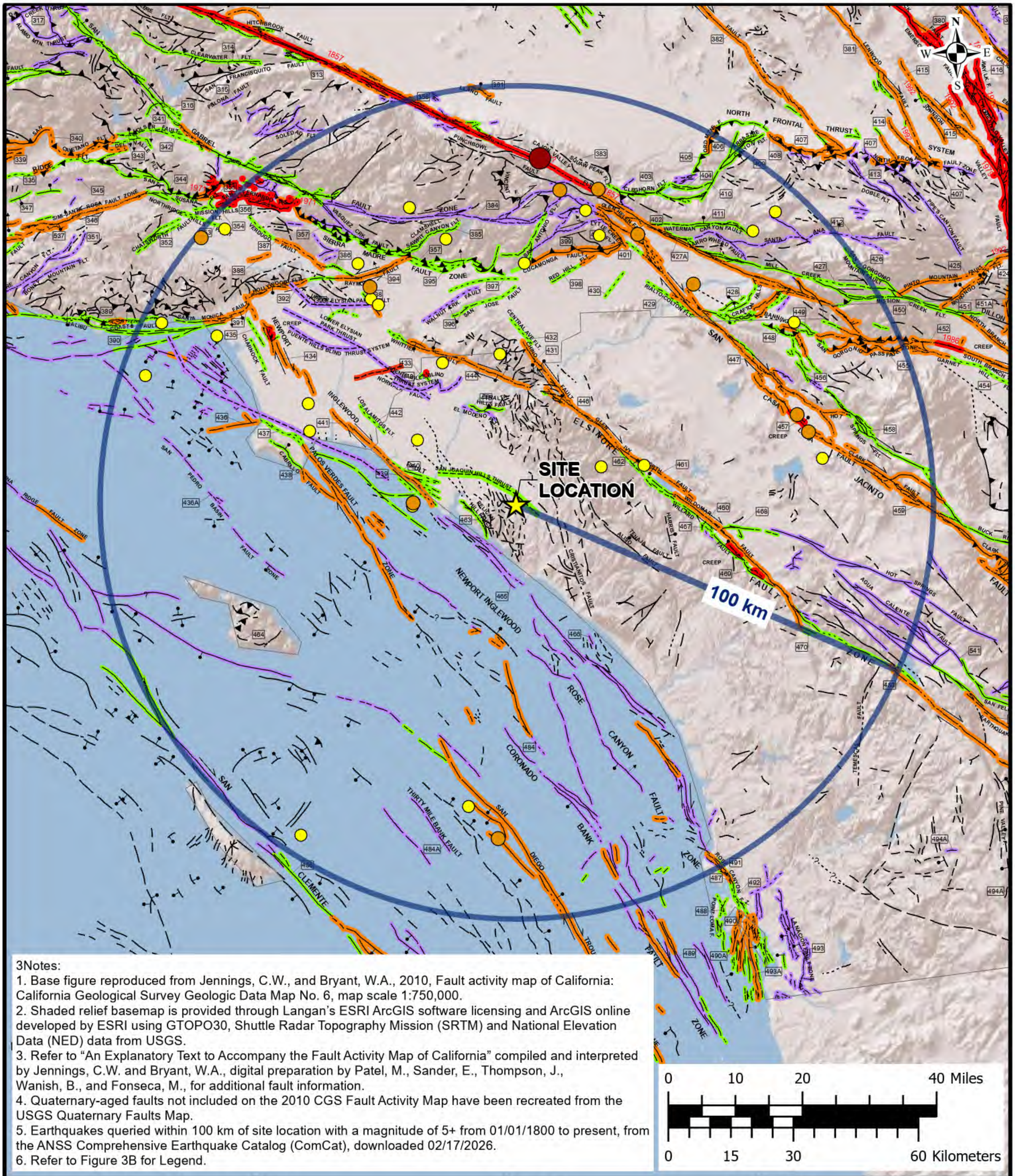
Figure No.
2



3Notes:

1. Base figure reproduced from Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California: California Geological Survey Geologic Data Map No. 6, map scale 1:750,000.
2. Shaded relief basemap is provided through Langan's ESRI ArcGIS software licensing and ArcGIS online developed by ESRI using GTOPO30, Shuttle Radar Topography Mission (SRTM) and National Elevation Data (NED) data from USGS.
3. Refer to "An Explanatory Text to Accompany the Fault Activity Map of California" compiled and interpreted by Jennings, C.W. and Bryant, W.A., digital preparation by Patel, M., Sander, E., Thompson, J., Wanish, B., and Fonseca, M., for additional fault information.
4. Quaternary-aged faults not included on the 2010 CGS Fault Activity Map have been recreated from the USGS Quaternary Faults Map.
5. Earthquakes queried within 100 km of site location with a magnitude of 5+ from 01/01/1800 to present, from the ANSS Comprehensive Earthquake Catalog (ComCat), downloaded 02/17/2026.
6. Refer to Figure 3B for Legend.

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Langan CA, Inc.

18575 Jamboree Road, Suite 150
Irvine, CA 92612

T: 949.561.9200 F: 949.561.9201 www.langan.com

Project	**PROJECT HERE** MILL CREEK DRIVE AND RIDGE ROUTE DRIVE LAGUNA HILLS ORANGE COUNTY CALIFORNIA
Figure Title	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS**
Project No.	700128701
Date	FEBRUARY 2026
Scale	1 inch = 20 miles
Drawn By	AC
Figure	3A

Path: \\langan.com\data\IRV\data7\700128701\Project Data\ArcGIS\APRX\LagunaHill_FaultFigs.aprx Date: 2/18/2026 User: Toulie Time: 2:55 PM

LEGEND:

★ Site Location

Fault Age

🟥 Historic

🟧 Holocene

🟩 Late Quaternary

🟪 Early Quaternary

⬛ Pre-Quaternary Fault

☐ 100 km Search Radius

Earthquake Epicenter

● Magnitude 5.0 to 5.9

● Magnitude 6.0 to 6.9

● Magnitude 7.0 to 7.4

● Magnitude 7.5 to 8.0

Fault Symbols

Bar and ball on downthrown side (relative or apparent).

⇌ Relative or apparent direction of lateral movement.

↑ Direction of dip.

▼ Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened.

491 Numbers refer to annotations listed in the appendices of the accompanying report.

-·-·- Structural discontinuity (offshore) separating differing Neogene structural domains.

▨ Brawley Seismic Zone.

Fault Classification

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:
(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.
(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.
(c) displaced survey lines.

1906 ▶
1906 ▷
A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

▶1906◀ Date bracketed by triangles indicates local fault break.

1906 No triangle by date indicates an intermediate point along fault break.

CREEP Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

1906 ■
1906 ■
Square on fault indicates where fault creep slippage has occurred that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

LANGAN
Langan CA, Inc.

18575 Jamboree Road, Suite 150
Irvine, CA 92612

T: 949.561.9200 F: 949.561.9201 www.langan.com

Project
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RIDGE ROUTE DRIVE

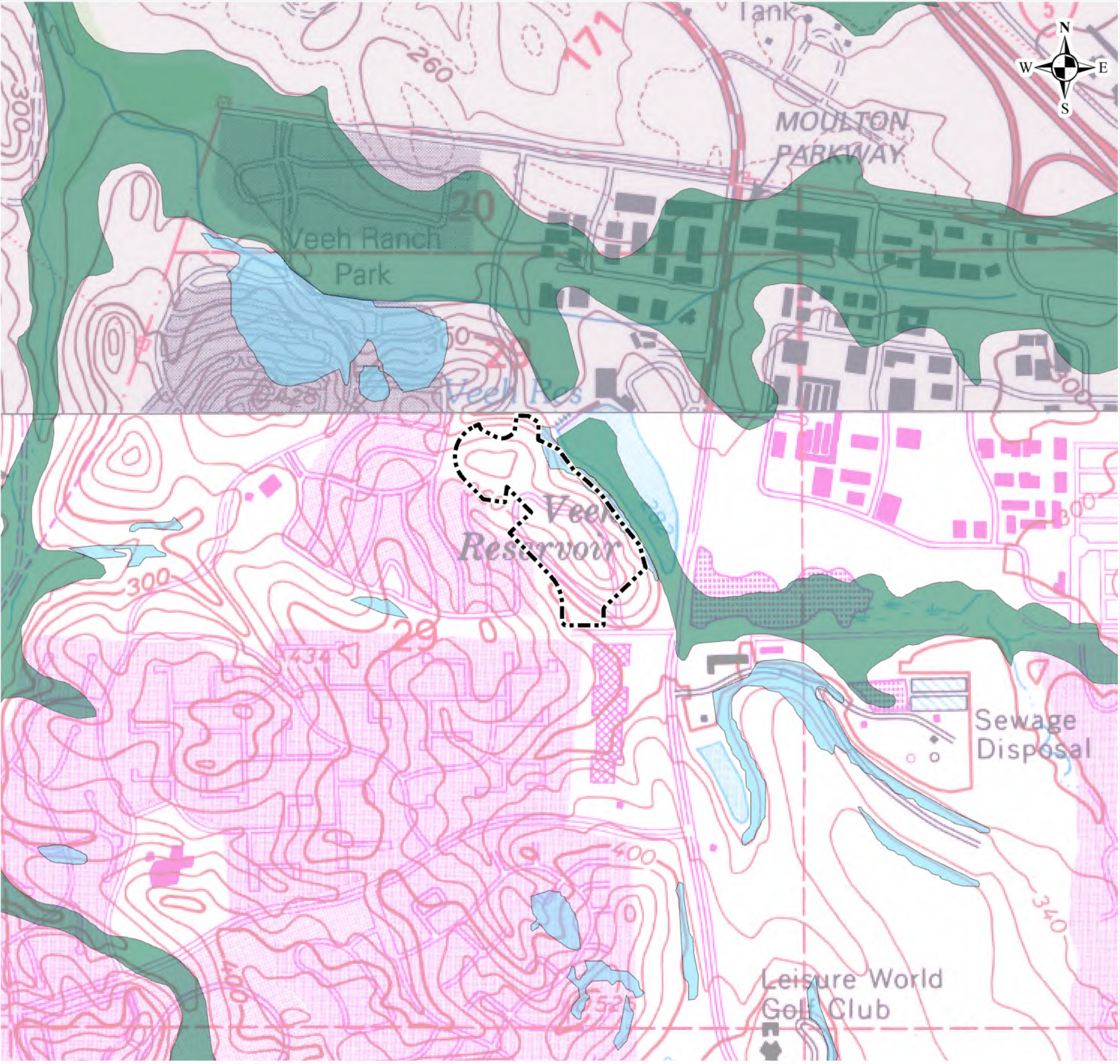
LAGUNA HILLS

ORANGE COUNTY CALIFORNIA

Figure Title
MAJOR FAULTS AND EARTHQUAKE EPICENTERS MAP LEGEND

Project No.
700128701
Date
FEBRUARY 2026
Scale
NOT TO SCALE
Drawn By
AC

Figure
3B



Legend

⬚ Approximate Site Location

🟩 **Liquefaction Zone** Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required

🟦 **Landslide Zone** Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Notes:
1. USGS Historical Topographic basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online, National Geographic Society, i-cubed.
2. Landslide and liquefaction data provided by the CGS.
3. All features shown are approximate.

1,000 0 1,000
SCALE IN FEET

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MILL CREEK DRIVE AND
RIDGE ROUTE DRIVE

LAGUNA HILLS

ORANGE COUNTY CALIFORNIA

Figure Title

SEISMIC HAZARD ZONES MAP

Project No.
700128701

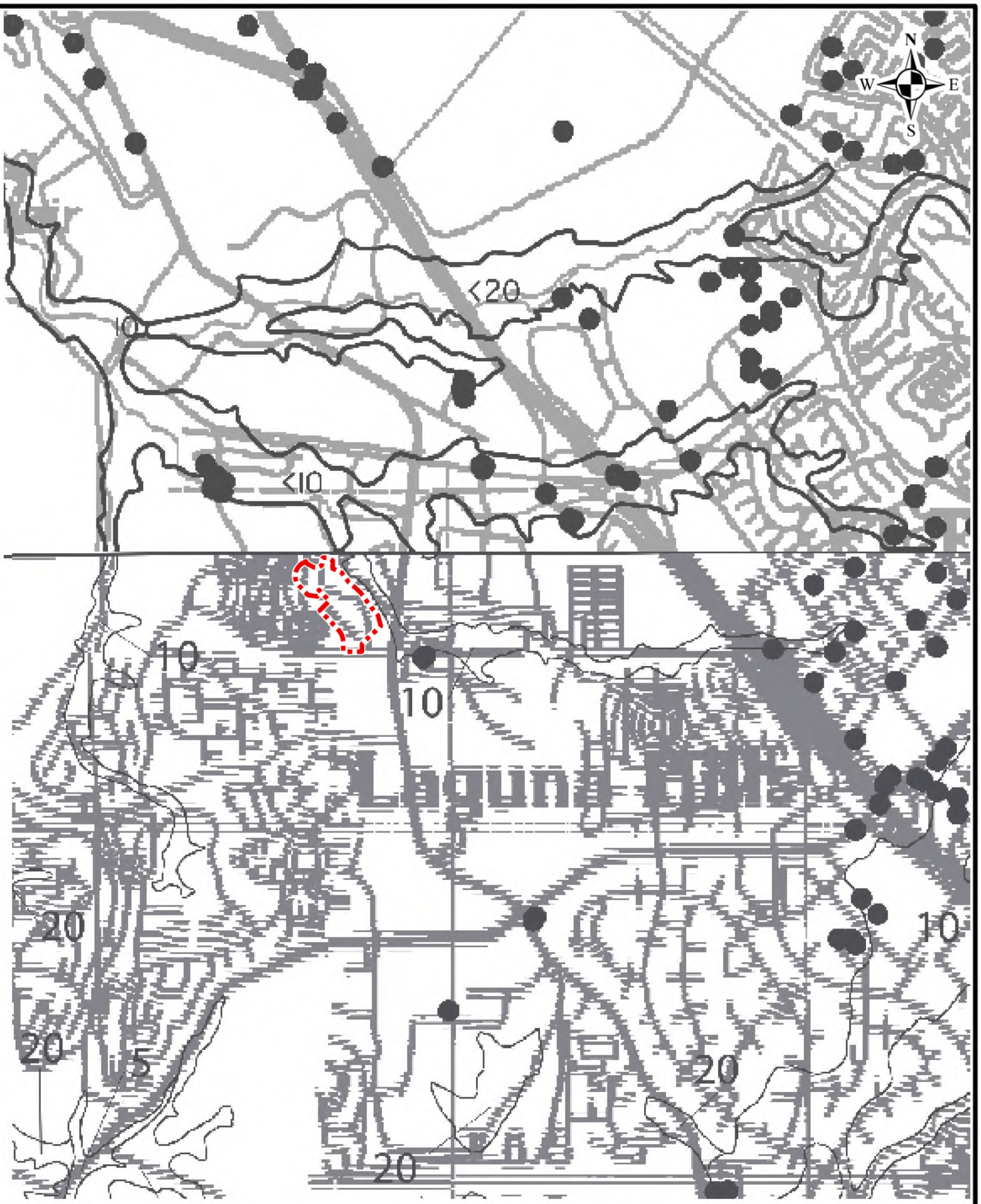
Date
FEBRUARY 2026

Scale
1" = 1,000'

Drawn By
OG

Figure
4

Path: \\langan.com\data\IRV\data7\700128701\Project Data\ArcGIS\APRX\LagunaHill_FaultFigs.aprx



Legend

▭ Approximate Site Location

● Geotechnical bore holes used in liquefaction evaluation

20 Depth to groundwater (in feet)

Notes:
1. Seismic Hazard Zone Reports for San Juan Capistrano Plate and Lake Forest (El Toro) Plate provided by California Department of Conservation Division of Mines and Geology.
2 All features shown are approximate.

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Figure Title

HISTORICAL HIGH GROUNDWATER MAP

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Date
FEBRUARY 2026

Scale
1" = 2,500'

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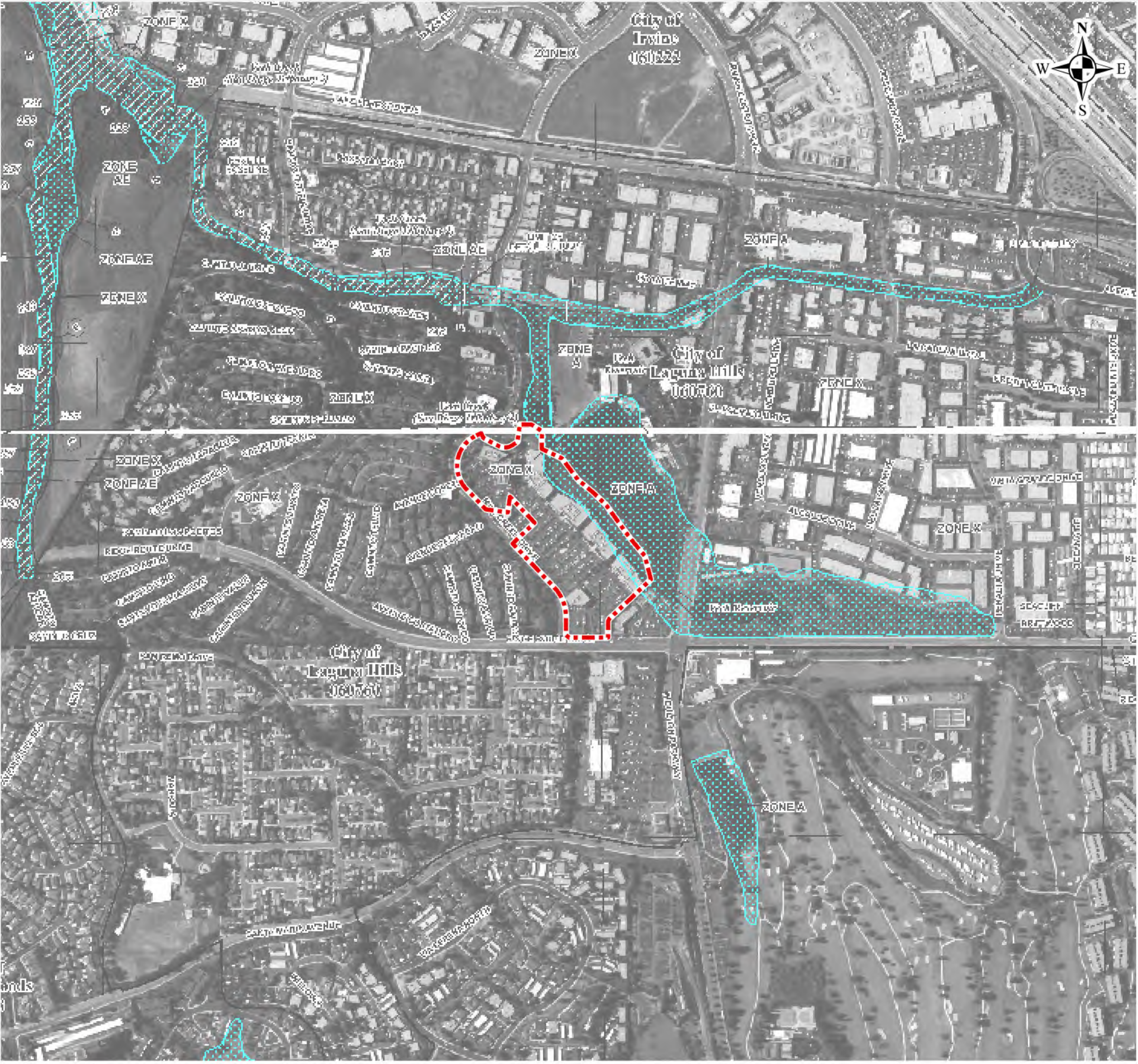
OG

Figure
5

Path: \\langan.com\data\IRV\data7\700128701\Project Data\ArcGIS\APRX\LagunaHill_FaultFigs.aprx



Legend

▨ SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.

▨ OTHER FLOOD AREAS

ZONE X — Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

☐ OTHER AREAS

ZONE X — Areas determined to be outside the 0.2% annual chance floodplain.

———	1% annual chance floodplain boundary
———	0.2% annual chance floodplain boundary
— — —	Floodway boundary
– —— —— –	Zone D boundary
··············	CBRS and OPA boundary
▨	Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

Notes:
1. Flood Firm Panels for San Juan Capistrano Plate and Lake Forest (El Toro) Plate provided by Federal Emergency Management Agency (FEMA).
2 All features shown are approximate.

1,000 0 1,000
SCALE IN FEET

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MILL CREEK DRIVE AND
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ORANGE COUNTY CALIFORNIA

Figure Title
FEMA FLOOD MAP

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Date
FEBRUARY 2026
Scale
1" = 1,000'
Drawn By
OG

Figure
6

Path: \\langan.com\data\IRV\data7\700128701\Project Data\ArcGIS\APRX\LagunaHill_FaultFigs.aprx

LEGEND:

▬ ▬ ▬	SITE LIMITS
────	WATER LINE OF VEEH RESERVOIR AT THE TIME OF THE SURVEY
LB-1 21'	APPROXIMATE BORING LOCATION WITH DEPTH IN FEET. (LANGAN 2023)
PT-1 11.5'	APPROXIMATE PERCOLATION TEST LOCATION WITH DEPTH IN FEET. (LANGAN 2023)
B-12 14'	APPROXIMATE BORING LOCATION WITH DEPTH IN FEET (EJN & ASSOCIATES, 1986)

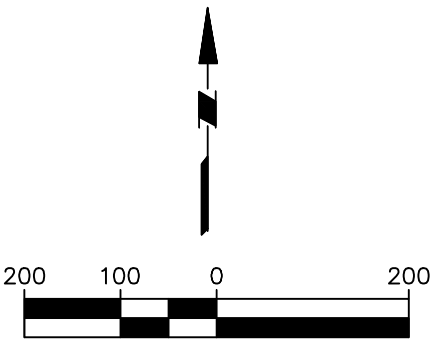
GEOLOGIC UNITS:

fill	ARTIFICIAL FILL
Ts	BEDROCK - SESPE FORMATION, CIRCLED WHERE BURIED
—?—	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, QUERIED WHERE UNCERTAIN

NOTES:

SITE PLAN BASED ON:

1. TOPOGRAPHIC AND BOUNDARY PLANS TITLED "BASEMAP OF 23272, 32382 MILL CREEK DRIVE, LAGUNA HILLS, CALIFORNIA" DATED 30 MAY 2023 FROM FUSCOE ENGINEERING.
2. CONCEPTUAL DEVELOPMENT PLAN TITLED "OVERALL SITE PLAN, SP1" DATED 16 DECEMBER 2025 PROVIDED BY KINGSBARN REALITY CAPITAL.
3. GEOTECHNICAL REPORT TITLED "PRELIMINARY SOILS INVESTIGATION, TENTATIVE PARCEL MAP. NO. 85-367, ORANGE COUNTY, CALIFORNIA" DATED 18 MARCH 1986 BY EJN & ASSOCIATES.



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MILL CREEK DRIVE AND
RIDGE ROUTE DRIVE

LAGUNA HILLS
ORANGE COUNTY CALIFORNIA

Figure Title

SITE PLAN

Project No. 700128701
Date FEBRUARY 2026
Scale AS SHOWN
Drawn By CDC

Figure No. 7

APPENDIX A

BORING LOGS FROM PREVIOUS INVESTIGATIONS

Major Divisions			Group Symbols		Soil Description
COARSE GRAINED SOIL (More than 50% material larger than the #200 sieve)	GRAVEL (More than 50% material larger than #4 sieve)	Clean GRAVEL (Less than 5% fines)	GW	░	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	▒	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL With Fines (More than 12% fines)	GM	▥	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	▨	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND (More than 50% material smaller than #4 sieve)	Clean SAND (Less than 5% fines)	SW	░	Well graded sands, gravelly sands, little or no fines.
			SP	░	Poorly graded sands or gravelly sands, little or no fines.
		SAND With Fines (More than 12% fines)	SM	▥	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	▨	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOIL (More than 50% material smaller than the #200 sieve)	SILT & CLAY (Liquid limit less than 50)		ML	▥	Inorganic silts, sandy or clayey silts Low to no plasticity.
			CL	▨	Inorganic clay, sandy or silty clay. Low to medium plasticity.
			OL	▦	Organic silt or organic silty clay. Low to medium plasticity.
	SILT & CLAY (Liquid limit more than 50)		MH	▥	Inorganic silts, diatomaceous or micaceous fine sandy or silty soils.
			CH	▨	Inorganic clays of high plasticity, Fat clays.
			OH	▨	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOIL			PT	▤	Peat and other highly organic soils.

PARTICLE SIZE LIMITS

(Sieve Openings in mm.) .074 .425 2.00 4.17 19.0 75.0 300.0

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

(U.S. Standard Sieve Sizes) #200 #40 #10 #4 .75 in. 3 in. 12 in.

Relative Density

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

Consistency

CLAYS AND PLASTIC SILTS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

*Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1 - 3/8 inch I.D.) split spoon (ASTM D-1586).

+Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

Surface Elevation: ≈ 360.0'

Relative Compaction %	Max. Density (pcf)	Dry Density (pcf)	Moisture (%)	Penetration (N)	Depth (ft)	Material Symbol	Unified Soil Classification	Comments:
						▨		Sandy Silty CLAY - Brownish Red Moist, Firm
					1	▨		
	127.6				Bulk	▨		
					2	▨		
87.9	127.6	112.2	17.3		Ring	▨	CL	
					3	▨		
					4	▨		
					5	▨		Fill
		107.1	19.4		Ring	▨		CLAY-Red/Brown Very Moist, Stiff Some Silt Native
					6	▨		
					7	▨		
					8	▨		Material gets very stiff Density decreases @ 9 Feet
					9	▨		
					10	▨		
		109.5	19.7		Ring	▨	CL	Firm Color changes @ ≈ 11 Feet Some Sand, probable Native Sespe Formation
					11	▨		
					12	▨		
					13	▨		
					14	▨		Gets Very Stiff
					15	▨		
		111.4	16.5		Ring	▨		
					16			End of Boring @ 16 Feet No Groundwater No Caving
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 1

Groundwater: None

Exploratory Boring Number: 1

EJN & Associates | Date: 3/24/86 | Job No.: 86-155-1 | Appendix D1

Surface Elevation: ≈362.0'

Comments:

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments
					1	▨		**Silty CLAY-** Red/Brown Moist, Firm
					2	▨		
		107.2	16.5		Ring	▨	CL	
					3	▨		
					4	▨		
					5	▨		
		109.7	16.3		Ring	▨		**Silty CLAY-** Dark Red/Brown Moist, Soft to Firm
					6	▨		
					7	▨	CL	
					8	▨		Fill
					9	░		**Clayey Silty SAND-** Light Red Moist, Soft to Firm (probable Sespe Formation) Native
					10	░		
		123.8	11.0		Ring	░	SM	
					11	░		Gets Stiff
					12	░		
					13			End of Boring @ 13 feet No Groundwater No Caving
					14			
					15			
					16			
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 13 Feet

Groundwater: None

Exploratory Boring Number: 2

EJN & Associates | Date: 3/24/86 | Job No.: 86-155-1 | Appendix D2

Surface Elevation:≈ 343.0'

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
						▨		Silty CLAY- Light Red/Brown Moist, Soft
					1	▨		
					2	▨		Gets Firm
					3	▨		
					4	▨		
					5	▨		
		113,4	17,0		Ring	▨	CL	Gets Stiff
					6	▨		
					7	▨		
					8	▨		
					9	▨		
					10	▨		Fill
		110,9	18,2		Ring	▨		
					11	⁙		Silty SAND- Light Red Moist, Very Stiff to Hard (Sespe Formation)
					12	⁙	SM	Native
					13	⁙		
					14			
					15			
					16			
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 13 1/2 Feet

Groundwater: None

Exploratory Boring Number: 3

EJN & Associates | Date: 3/24/86 | Job No.: 86-156-1 | Appendix D3

Surface Elevation: ≈ 332'

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
						▨		**Sandy CLAY** - Red Moist, Soft
					1	▨		
	120.3				Bulk	▨		
					2	▨		
					Bulk	▨		
					3	▨	CL	
85.4	120.3	102.7	18.3		Rings	▨		Very Soft
					4	▨		
					5	▨		Fill
					6	▨		
						░	SM	**Silty SAND** - Gray Moist, Firm
					7	▨		**Silty CLAY** - Red Moist, Stiff Native
					8	▨		
					9	▨		
					10	▨		
		109.2	16.5		Rings	▨		
					11	▨	CL	
					12	▨		
					13	▨		
					14	▨		
					15	▨		Gets Sandier
					16	░	SM	**Silty SAND**- Pinkush Gray Moist, Very Stiff (Sespe Formation)
					17	░		
					18			End of Boring @ 18 Feet No Groundwater No Caving
					19			
					20			

Logged By: RW

Depth of Boring: 18 Feet

Groundwater: None

Exploratory Boring Number: 4

EJN & Associates	Date: 3/24/86	Job No.: 86-155-1	Appendix D4

Surface Elevation: ≈ 330.0'

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
					1	▨		**Silty CLAY**- Red/Brown Native Moist, Firm
					2	▨		
					3	▨	CL	
		114.3	17,2		Ring	▨		
					4	▨		Gets Stiff
					5	▨		
					6	░		**Silty SAND**- Gray Moist, Stiff
					7	░	SM	
		121,3	8,4		Ring	░		
					8	░		
					9	▨	CL	**Silty CLAY**- Pink Moist, Very Stiff
					10			End of Boring @ 10 Feet No Groundwater No Caving
					11			
					12			
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 10 Feet

Groundwater: None

Exploratory Boring Number: 5

EJN & Associates | Date: 3/25/86 | Job No.: 86-155-1 | Appendix D5

Surface Elevation: 310

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
					1	░	SM	**Silty SAND**- Redish Brown Firm to Stiff Modreatly Dry
					2	░		
					3	░		
		117.6	7.3		Ring	░		
					4	░		
					5	▨	CL	**Sandy Clay**- Brown/Red Very Moist, Stiff
					6	▨		
					7	▨		
					8	▨		**GETS Stiff**
					9	▨		??
					10	▨		Some mechanical debris found from 5 to 10 feet
		105.1	19.7		Ring	▨		
					11	▨		Fill
					12	▨	CL	**Silty Clayey Sand**- Pinkish Gray Probably silty sand from Sespe Formation
					13	▨		Native
					14	▨		
					15			End of Boring @15 feet No Groundwater No Caving
					16			
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 15

Groundwater: None

Exploratory Boring Number: 6

EJN & Associates | Date: 3/24/86 | Job No.: 86-155-1 | Appendix D 6

Surface Elevation: 300

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
						⁙		**Silty Sand**- Red/Brown Moist,Firm
					1	⁙		
	122.8				Bulk	⁙		
					2	⁙		
					Bulk	⁙	SM	
					3	⁙		
94.0	122.8	115.4	9.3		Ring	⁙		Fill
					4	⁙		
					5	▥		**Clayey Silt**- Brown/Red
					6	▥		Moist ,Stiff Native
					7	▥		
					8	▥		
					9	▥		Some Sand
					10	▥	ML	
		106.5	15.7		Ring	▥		Gets Very Stiff
					11	▥		
					12	▥		Cobbles
					13	▥		
					14	▥		
					15	⁙		**Silty Sand**-Pink/Brown
					16	⁙	SM	Moist,Very Stiff Sespe Formation
					17	⁙		
					18			Refusal @ 18feet due toCobble No Groundwater No Cavitation
					19			
					20			

Logged By: RW

Depth of Boring: 18 Feet

Groundwater: None

Exploratory Boring Number: 7

EJN & Associates | Date:3/25/86 | Job No.: 86-155-1 | Appendix D 7

Surface Elevation: 306

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
					1 2 3	░	SM	**Silty Sand**-Red/Brown Moderate Dry Firm to Stiff Some Clay Fill
					4 5 6	▨	SC	**Calyey Sand**-Pink/White Moist,Very Stiff Sespe Formation Native
					7 8 9 10 11 12 13 14 15 16 17 18 19 20			Refusal @ 7 feet No Groundwater No Caving

Logged By: RW

Depth of Boring: 7 Feet

Groundwater: None

Exploratory Boring Number: 8

EJN & Associates | **Date: 3/25/86** | **Job No.: 86-155-1** | **Appendix D 8**

Surface Elevation: 330 | **Comments:**

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments
					1	⋯		**Silty Sand-Red/Brown** Moist,Dense Sand Gets Whitish Grey
	124.3				Bulk	⋯		
					2	⋯		
					Bulk	⋯		
					3	⋯	CL	
					4	⋯		Fill
93.2	124.3	115.8	9.0		Ring	⋯		
					5			
					6	▨		**Clayey Silty Sand** Moist,Very Dense Native
					7	▨	CL	
					8	▨		
					9	▨		
		109.5	8.8		Ring	▨		
					10			
					11			End of Boring @ 10 feet No Groundwater No Caving
					12			
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 10 Feet

Groundwater: None

Exploratory Boring Number: 9

EJN & Associates | **Date: 3/25/86** | **Job No.: 86-155-1** | **Appendix D 9**

Surface Elevation: 329

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
					1 2 3 4 5	▨	SM	Cayey Silty Sand Red/Brown Moist,Stiff Gets Very Stiff Fill ?Where Native
					6 7 8 9 10 11 12 13 14 15 16 17 18 19 20			Refusal @ 5 feet due to Tight Materi l ??? No Groundwater No Caving

Logged By: RW

Depth of Boring: 5 Feet

Groundwater: None

Exploratory Boring Number: 10

EJN & Associates | Date:3/25/86 | Job No.:86-155-1 | Appendix D 10

Surface Elevation: 341 | **Comments:**

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
	121.3				1 Bulk	▨	SC	Clayey SAND-Red/Brown Very Moist,Stiff
					2 Bulk	▨		
					3	▨		
90.8	121.3	110.1	9.8		4 Ring	▨		Fill
					5	▨	CL	Sandy Loam-Top Soil,Roots & Grass
					6	▨	CL	CLAY-Very Dark Gray Very Moist, Firm to Stiff Native
					7	▨		
					8	▨		
		123.7	7.1		9 Ring	▨	SM	Clayey Silt Sand-Red/Brown Moist,Very Stiff Sespe Formation
					10			
					11			
					12			End OF Boring @10 Feet No Groundwater No Caving
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 10

Groundwater: None

Exploratory Boring Number: 11

EJN & Associates | Date: 3/25/86 | Job No.: 86-155-1 | Appendix D 11

Surface Elevation: 365 | **Comments:**

RELATIVE COMPACTION %	MAX. DENSITY (pcf)	DRY DENSITY (pcf)	MOISTURE (%)	PENETRATION (N)	DEPTH (ft)	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	Comments:
						▩		Clayey Sand-Reddish Brown
					1	▩		Moist,Stiff
					2	▩	SC	
		109.3	8.2		3 Ring	▩		
					4	▩		Fill
					5	▨		Clay-Very Dark Gray
					6	▨		Very Moist,Very Stiff Native
					7	▨	CL	
		100.3	21.5		8 Ring	▨		
					9	░		Silty Sand-Red/Brown
					10	░		Stiff Moist
					11	░	SM	
					12	░		Gets Very Stiff
		115.3	7.9		13 Ring	░		
					14			REfusal @ 14 Feet due to
					15			Dense Material
								No Groundwater
					16			No Caving
					17			
					18			
					19			
					20			

Logged By: RW

Depth of Boring: 14 Feet

Groundwater: None

Exploratory Boring Number: 12

EJN & Associates | Date: 3/25/86 | Job No.: 86-155-1 | Appendix D 12

APPENDIX B

BORING LOGS FROM CURRENT INVESTIGATION

LANGAN

Log of Boring **LB-1** Sheet 1 of 2

Project Project HERE		Project No. 700128701	
Location Mill Creek Drive and Ridge Route Drive		Elevation and Datum 344.5 feet (NAVD 88)	
Drilling Company Martini Drilling		Date Started 03/16/2023	Date Finished 03/16/2023
Drilling Equipment CME75 Truck-Mounted Drill Rig #1		Completion Depth 21 ft	Rock Depth <1 ft
Size and Type of Bit 8-inch O.D. Hollow Stem Auger		Number of Samples	Disturbed 7 / Undisturbed - / Core -
Casing Diameter (in) -	Casing Depth (ft) -	Water Level (ft.)	First ▽ - / Completion ▼ - / 24 HR. ▼ -
Casing Hammer - / Weight (lbs) -	Drop (in) -	Drilling Foreman Jeff Razer	
Sampler 2-inch O.D. Split Spoon; 3-inch O.D. Cal Mod.		Field Engineer Julia Xu	
Sampler Hammer Automatic / Weight (lbs) 140	Drop (in) 30		

Elev. (ft)	Sample Description	Depth Scale	Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft) 10 20 30 40	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+344.9								
+344.5	5 inches of asphalt.	0						Bulk sample collected from 0 to 5 feet.
+344.0	6 inches of aggregate base.							
	Bedrock - Sespe Formation (Ts)	2						
	Red, clayey SANDSTONE, moist.	4	S-1	CR	18	25 36 34	70	Cohesion = 100 psf, friction angle = 31 deg
	Red, clayey SANDSTONE, moist.	6	S-2	CR	6	50/6	50/6	
	Red, clayey SANDSTONE, moist.	8	S-3	CR	5	50/5	50/5	
	Red, clayey SANDSTONE, moist.	10	S-4	CR	5	50/5	50/5	
		12						
		14						
	Tannish red, SANDSTONE, moist.	16	S-5	SS	15	25 42 50/3	92/8	
		18						
	Tannish red SANDSTONE, moist.	20	S-6	CR	4	50/4	50/4	Drill rig encountered refusal at 21 feet.
+323.9	End of boring at 21 feet. No groundwater encountered. Boring backfilled with bentonite grout mixture	22						
	Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023.	24						
	Notes: LL - Liquid limit PL - Plastic limit PI - Plasticity index (the difference between the liquid limit and the plastic limit) MC - Moisture content DD - Dry density	26 28 30						

Log of Boring **LB-1** Sheet 2 of 2

Project Project HERE	Project No. 700128701
Location Mill Creek Drive and Ridge Route Drive	Elevation and Datum 344.5 feet (NAVD 88)

MATERIAL SYMBOL	Elev. (ft)	Sample Description	Depth Scale	Sample Data					Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
				Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft) 10 20 30 40	
	+314.9	AC - Asphalt concrete AB - Aggregate base NP - Non-plastic qu - Unconfined compressive strength	30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 67.5						

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Log of Boring **LB-2** Sheet 1 of 2

Project	Project HERE	Project No.	700128701
Location	Mill Creek Drive and Ridge Route Drive	Elevation and Datum	342.7 feet (NAVD 88)
Drilling Company	Martini Drilling	Date Started	03/16/2023
		Date Finished	03/16/2023
Drilling Equipment	CME75 Truck-Mounted Drill Rig #1	Completion Depth	30.7 ft
		Rock Depth	<1 ft
Size and Type of Bit	8-inch O.D. Hollow Stem Auger	Number of Samples	Disturbed 9 / Undisturbed - / Core -
Casing Diameter (in)	-	Water Level (ft.)	First ▽ - / Completion ▼ - / 24 HR. ▼ -
Casing Depth (ft)	-	Drilling Foreman	Jeff Razer
Casing Hammer	-	Field Engineer	Julia Xu
Weight (lbs)	-		
Drop (in)	-		
Sampler	2-inch O.D. Split Spoon; 3-inch O.D. Cal Mod.		
Sampler Hammer	Automatic		
Weight (lbs)	140		
Drop (in)	30		

Elev. (ft)	Sample Description	Depth Scale	Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft) 10 20 30 40	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+343.8								
+343.4	5 inches of asphalt.	0	B-1	BAG				Bulk sample collected from 0 to 5 feet. Soil corrosivity tests
+342.9	6 inches of aggregate base.							
	Bedrock - Sespe Formation (Ts)							
	Reddish tan, silty SANDSTONE, moist.		S-1	CR	4	50/4	50/4	
	Red SANDSTONE, trace clay, trace silt, moist.		S-2	CR	5	50/5	50/5	
	Red SANDSTONE, trace clay, trace silt, moist.		S-3	CR	5	50/5	50/5	
	Red SANDSTONE, trace clay, trace silt, moist.	10	S-4	CR	5	50/5	50/5	
	Red SANDSTONE, trace clay, trace silt, moist.		S-5	SS	10	30 50/4	50/4	
	Red SANDSTONE, trace clay, trace silt, moist.	20	S-6	CR	5	50/5	50/5	
	Red silty SANDSTONE, moist.		S-7	SS	17	24 34 50/5	84/11	

Depth Scale: 0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30

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Log of Boring **LB-2** Sheet 2 of 2

Project Project HERE	Project No. 700128701
Location Mill Creek Drive and Ridge Route Drive	Elevation and Datum 342.7 feet (NAVD 88)

Elev. (ft)	Sample Description	Depth Scale	Sample Data Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft) 10 20 30 40	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+313.8		30						
+313.1	Red silty SANDSTONE, moist.		S-8	CR	8	36 50/2	50/2	

End of boring at 30.7 feet.
No groundwater encountered.
Boring backfilled with bentonite grout mixture

Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023.

Notes:
LL - Liquid limit
PL - Plastic limit
PI - Plasticity index (the difference between the liquid limit and the plastic limit)
MC - Moisture content
DD - Dry density
AC - Asphalt concrete
AB - Aggregate base
NP - Non-plastic
qu - Unconfined compressive strength

Depth scale: 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 67.5

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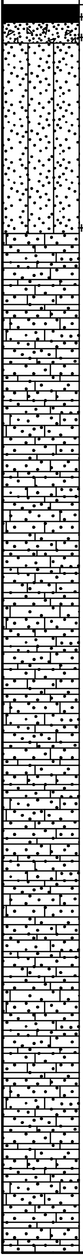
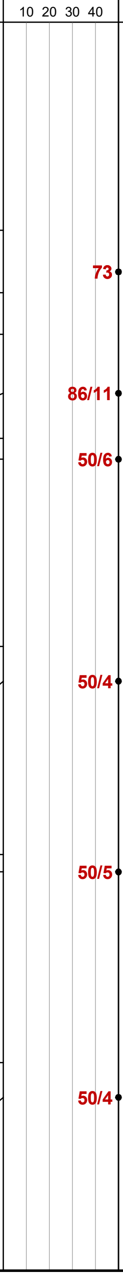
Log of Boring **LB-3** Sheet 1 of 2

Project Project HERE		Project No. 700128701	
Location Mill Creek Drive and Ridge Route Drive		Elevation and Datum 333.5 feet (NAVD 88)	
Drilling Company Martini Drilling		Date Started 03/16/2023	Date Finished 03/16/2023
Drilling Equipment CME75 Truck-Mounted Drill Rig #1		Completion Depth 30.3 ft	Rock Depth 5.5 ft
Size and Type of Bit 8-inch O.D. Hollow Stem Auger		Number of Samples: Disturbed 8	Undisturbed - ; Core -
Casing Diameter (in) -	Casing Depth (ft) -	Water Level (ft.): First ▽ -	Completion ▼ - ; 24 HR. ▼ -
Casing Hammer -; Weight (lbs) -	Drop (in) -	Drilling Foreman Jeff Razer	
Sampler 2-inch O.D. Split Spoon; 3-inch O.D. Cal Mod.		Field Engineer Julia Xu	
Sampler Hammer Automatic; Weight (lbs) 140	Drop (in) 30		



Elev. (ft)	Sample Description	Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft)	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+333.7							
+333.3	5 inches of asphalt.						Hand Augered top 5 feet. Bulk sample collected from 0 to 5 feet.
+332.8	6 inches of aggregate base.						
	FILL						
+328.2	Tannish red, silty fine SAND, trace clay, (SM), moist.						
	Bedrock - Sespe Formation (Ts) Reddish brown, silty SANDSTONE, moist	S-1	CR	18	29 35 38	73	MC = 9.4% DD = 120 pcf
	Reddish brown, clayey SANDSTONE, trace silt, moist.	S-2	CR	17	19 36 50/5	86/11	Cohesion = 200 psf, friction angle = 28.5 deg
	Reddish brown, clayey SANDSTONE, trace silt, moist.	S-3	CR	6	50/6	50/6	
	Reddish brown, clayey SANDSTONE, trace silt, moist.	S-4	SS	10	35 50/4	50/4	
	Tannish brown, SANDSTONE, trace clay, trace silt, moist.	S-5	CR	5	50/5	50/5	
	Tannish brown, SANDSTONE, trace clay, trace silt, moist.	S-6	SS	10	26 50/4	50/4	

Depth Scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30



Project	Project No.
Project HERE	700128701
Location	**Elevation and Datum**
Mill Creek Drive and Ridge Route Drive	333.5 feet (NAVD 88)

MATERIAL SYMBOL	Elev. (ft)	Sample Description	Depth Scale	Sample Data					Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
				Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft) 10 20 30 40	
	+303.7								
∴	+303.5	Tannish brown, SANDSTONE, trace clay, trace silt, moist.	30	S-7	CRIII	3	50/3	50/3	
		End of boring at 30.3 feet. No groundwater encountered.							
		Boring backfilled with bentonite grout mixture	32						
		Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023.	34						
		Notes: LL - Liquid limit PL - Plastic limit	36						
		PI - Plasticity index (the difference between the liquid limit and the plastic limit) MC - Moisture content	38						
		DD - Dry density AC - Asphalt concrete AB - Aggregate base NP - Non-plastic	40						
		qu - Unconfined compressive strength	42						
			44						
			46						
			48						
			50						
			52						
			54						
			56						
			58						
			60						
			62						
			64						
			66						
			67.5						

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Log of Boring **LB-4** Sheet 1 of 2

Project: Project HERE

Project No.: 700128701

Location: Mill Creek Drive and Ridge Route Drive

Elevation and Datum: 327.5 feet (NAVD 88)

Drilling Company: Martini Drilling

Date Started: 03/16/2023

Date Finished: 03/16/2023

Drilling Equipment: CME75 Truck-Mounted Drill Rig #1

Completion Depth: 30.8 ft

Rock Depth: 18 ft

Size and Type of Bit: 8-inch O.D. Hollow Stem Auger

Number of Samples: Disturbed 8; Undisturbed -; Core -

Casing Diameter (in): -

Casing Depth (ft): -

Water Level (ft.): First ▽ -; Completion ▼ -; 24 HR. ▼ -

Casing Hammer: -

Weight (lbs): -

Drop (in): -

Drilling Foreman: Jeff Razer

Sampler: 2-inch O.D. Split Spoon; 3-inch O.D. Cal Mod.

Sampler Hammer: Automatic

Weight (lbs): 140

Drop (in): 30

Field Engineer: Julia Xu

Elev. (ft)	Sample Description	Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft)	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+327.4							
+327.0	5 inches of asphalt.						
+326.2	10 inches of aggregate base.						
	FILL						
	Very stiff, reddish brown, sandy CLAY, fine to medium sand, trace silt, (CL), moist.	S-1	CR	18	4 10 14	24	Cohesion = 150 psf, friction angle = 30.5 deg
	Very stiff, reddish brown, sandy CLAY, fine to medium sand, trace silt, (CL), moist.	S-2	CR	18	4 10 16	26	
	Stiff, reddish brown, sandy CLAY, fine to medium sand, trace silt, (CL), moist.	S-3	CR	18	6 9 12	21	q_u=1.5 tsf (PP)
	Very stiff, reddish brown and dark grayish brown mottled, sandy CLAY, fine to medium sand, trace silt, (CL), moist.	S-4	CR	18	6 11 15	26	q_u=2.25 tsf (PP) MC = 14.8% DD = 116 pcf
+312.4	**Residual Soil** Stiff, reddish brown, sandy CLAY, fine to medium sand, trace silt, (CL), moist, homogenous, in-situ weathering.	S-5	SS	18	3 4 8	12	q_u=1.25 tsf (PP) LL = 37, PL = 20, PI = 17
+309.4	**Bedrock - Sespe Formation (Ts)**						
	Red, clayey SANDSTONE, trace silt, moist.	S-6	CR	9	27 50/3	50/3	
	Red, clayey SANDSTONE, trace silt, moist.	S-7	SS	12	17 50/6	50/6	

Depth Scale: 0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30

N-Value scale: 10 20 30 40

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Log of Boring **LB-4** Sheet 2 of 2

Project Project HERE	Project No. 700128701
Location Mill Creek Drive and Ridge Route Drive	Elevation and Datum 327.5 feet (NAVD 88)

MATERIAL SYMBOL	Elev. (ft)	Sample Description	Depth Scale	Sample Data					Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
				Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft) 10 20 30 40	
▨	+297.4 +296.7	Red, clayey SANDSTONE, trace silt, moist.	30	S-8	CR	9	40 50/3	50/3 ●	
		End of boring at 30.8 feet. No groundwater encountered. Boring backfilled with bentonite grout mixture Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023. Notes: LL - Liquid limit PL - Plastic limit PI - Plasticity index (the difference between the liquid limit and the plastic limit) MC - Moisture content DD - Dry density AC - Asphalt concrete AB - Aggregate base NP - Non-plastic qu - Unconfined compressive strength	32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 67.5						

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Log of Boring **PT-1** Sheet 1 of 1

Project Project HERE	Project No. 700128701
Location Mill Creek Drive and Ridge Route Drive	Elevation and Datum 320 feet (NAVD 88)
Drilling Company Martini Drilling	Date Started 03/16/2023 Date Finished 03/16/2023
Drilling Equipment CME75 Truck-Mounted Drill Rig #1	Completion Depth 11.5 ft Rock Depth -
Size and Type of Bit 8-inch O.D. Hollow Stem Auger	Number of Samples: Disturbed 4; Undisturbed -; Core -
Casing Diameter (in) - Casing Depth (ft) -	Water Level (ft.): First ▽ -; Completion ▼ -; 24 HR. ▼ -
Casing Hammer - Weight (lbs) - Drop (in) -	Drilling Foreman Jeff Razer
Sampler 2-inch O.D. Split Spoon; 3-inch O.D. Cal Mod.	Field Engineer Julia Xu
Sampler Hammer Automatic Weight (lbs) 140 Drop (in) 30	

Elev. (ft)	Sample Description	Depth Scale	Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft)	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+320.7		0						
+320.2	6 inches of asphalt.							Bulk sample collected from 0 to 5 feet.
+319.5	9 inches of aggregate base.							
	FILL	2	B-1	BAG				R-value = 12 EI = 57
		4						
	Stiff, red, fine to medium sandy CLAY, trace silt, trace fine gravel, (CL), moist.	6	S-1	SS	18	2 4 5	9	LL = 35, PL = 18, PI = 17
	Stiff, red, fine to medium sandy CLAY, trace fine gravel, (CL), moist.	8	S-2	SS	18	3 4 5	9	
	Stiff, tannish red, fine to medium sandy CLAY, (CL), moist.	10	S-3	SS	18	3 5 8	13	
+309.2	End of boring at 11.5 feet. No groundwater encountered. Boring backfilled with bentonite grout mixture	12						

Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023.

Notes:
LL - Liquid limit
PL - Plastic limit
PI - Plasticity index (the difference between the liquid limit and the plastic limit)
MC - Moisture content
DD - Dry density
AC - Asphalt concrete
AB - Aggregate base
NP - Non-plastic
qu - Unconfined compressive strength

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Log of Boring **PT-2** Sheet 1 of 1

Project Project HERE		Project No. 700128701	
Location Mill Creek Drive and Ridge Route Drive		Elevation and Datum 343 feet (NAVD 88)	
Drilling Company Martini Drilling		Date Started 03/16/2023	Date Finished 03/16/2023
Drilling Equipment CME75 Truck-Mounted Drill Rig #1		Completion Depth 11.5 ft	Rock Depth 8 ft
Size and Type of Bit 8-inch O.D. Hollow Stem Auger		Number of Samples	Disturbed 4 Undisturbed - Core -
Casing Diameter (in) -	Casing Depth (ft) -	Water Level (ft.)	First ▽ - Completion ▼ - 24 HR. ▼ -
Casing Hammer - Weight (lbs) - Drop (in) -		Drilling Foreman Jeff Razer	
Sampler 2-inch O.D. Split Spoon; 3-inch O.D. Cal Mod.			
Sampler Hammer Automatic Weight (lbs) 140 Drop (in) 30		Field Engineer Julia Xu	

Elev. (ft)	Sample Description	Depth Scale	Sample Data Number	Type	Recov. (in)	Penetr. resist BL/6in	N-Value (Blows/ft)	Remarks (Drilling Fluid, Depth of Casing, Fluid Loss, Drilling Resistance, etc.)
+344.0		0						
+343.6	5 inches of asphalt.							Bulk sample collected from 0 to 5 feet.
+343.1	6 inches of aggregate base.							
	FILL	2	B-1	BAG				
		4						
	Very dense, tannish red, clayey fine to medium SAND, (SC), moist.	6	S-1	SS	18	21 30 35	65	
+336.0	Very dense, reddish tan, clayey fine to medium SAND, (SC), moist.	8	S-2	SS	18	10 29 50	79	
	Bedrock - Sespe Formation (Ts) Tannish red, clayey SANDSTONE, moist.							
	Tannish red, clayey SANDSTONE, moist.	10	S-3	SS	18	16 30 40	70	MC = 9.6%
+332.5	End of boring at 11.5 feet. No groundwater encountered. Boring backfilled with bentonite grout mixture	12						

Depth Scale: 14, 16, 18, 20, 22, 24, 26, 28, 30

N-Value scale: 10 20 30 40

Elevations referenced to NAVD88 vertical datum based on topographic plan titled "Basemap of 23272 & 23282 Mill Creek Drive, Laguna Hills, California" by Fuscoe Engineering dated 19 April 2023.

Notes:
LL - Liquid limit
PL - Plastic limit
PI - Plasticity index (the difference between the liquid limit and the plastic limit)
MC - Moisture content
DD - Dry density
AC - Asphalt concrete
AB - Aggregate base
NP - Non-plastic
qu - Unconfined compressive strength

APPENDIX C

PERCOLATION TEST RESULTS

PERCOLATION TEST DATA SHEET

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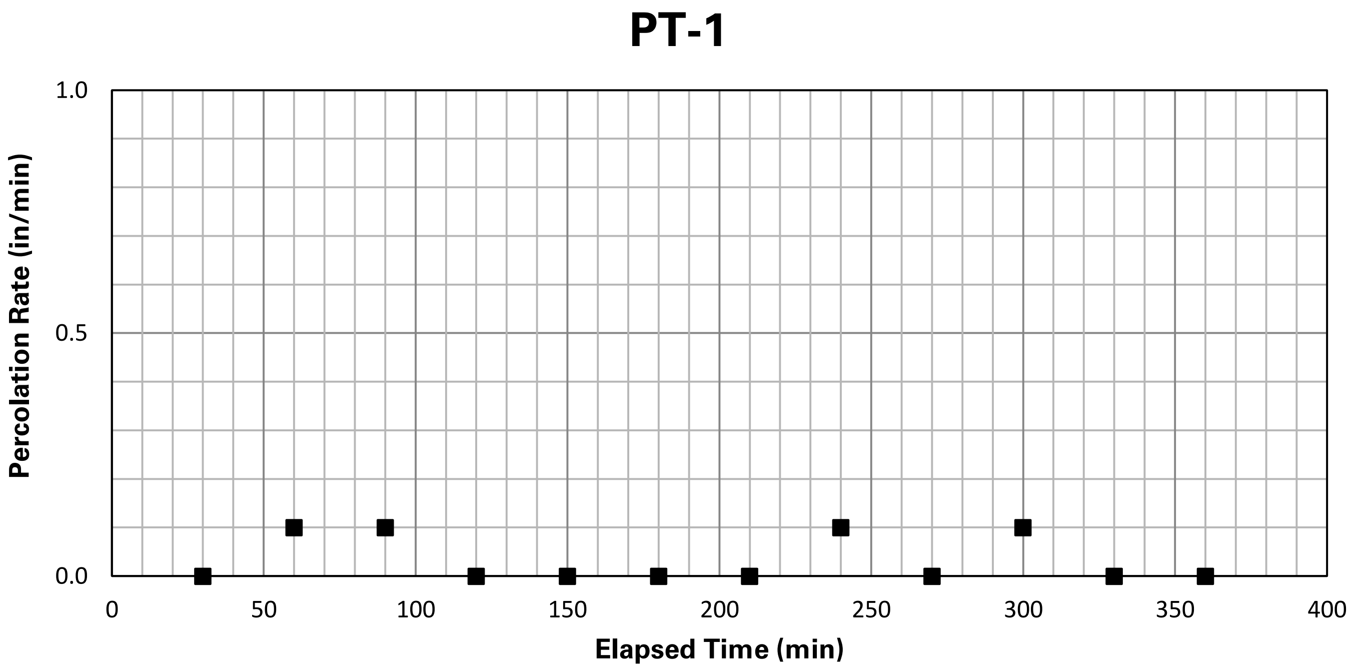
Project:	Project HERE - Laguna Hills	Project No.:	700128701	Date of Test:	3/16/2023
Test Hole No.:	PT-1	Tested By:	RF		
Depth of Test Hole (ft):	10	USCS Soil Classification:		Sandy CLAY (CL)	
Casing Depth (ft):	10.0' PVC Pipe; with 5 ft screen	Test Hole Diameter (in):		8	

Trial No.	Date	Time of Measurement	Initial Depth to Water (Feet)	Time of Measurement	Final Depth to Water (Feet)	Time Interval (min)	Change in Water Level (Feet)	Percolation Rate (in/min)	Infiltration Rate (in/hr)
Pre-Soak #1	3/16/2023	8:00 AM	4.30	8:30 AM	4.40	30	0.10	0.04	
Pre-Soak #2	3/16/2023	8:30 AM	4.30	9:00 AM	4.40	30	0.10	0.04	
1	3/16/2023	9:30 AM	4.30	10:00 AM	4.30	30	0.00	0.00	
2	3/16/2023	10:00 AM	4.30	10:30 AM	4.40	30	0.10	0.04	
3	3/16/2023	10:30 AM	4.30	11:00 AM	4.40	30	0.10	0.04	
4	3/16/2023	11:00 AM	4.30	11:30 AM	4.30	30	0.00	0.00	
5	3/16/2023	11:30 AM	4.30	12:00 PM	4.30	30	0.00	0.00	
6	3/16/2023	12:00 PM	4.30	12:30 PM	4.30	30	0.00	0.00	
7	3/16/2023	12:30 PM	4.30	1:00 PM	4.30	30	0.00	0.00	
8	3/16/2023	1:00 PM	4.30	1:30 PM	4.40	30	0.10	0.04	
9	3/16/2023	1:30 PM	4.30	2:00 PM	4.30	30	0.00	0.00	
10	3/16/2023	2:00 PM	4.30	2:30 PM	4.40	30	0.10	0.04	
11	3/16/2023	2:30 PM	4.30	3:00 PM	4.30	30	0.00	0.00	
12	3/16/2023	3:00 PM	4.30	3:30 PM	4.30	30	0.00	0.00	0.0

Comments:

1. Percolation test was performed in accordance with the Orange County - Technical Guidance Document dated 28 September 2017.
2. Infiltration Rate was calculated using Porchet Method.
3. Per the procedures for shallow percolation tests in non-sandy soils, a minimum of twelve measurements were taken in 30-minute intervals for six hours after sandy soil criteria was not met.
4. Weather: Cloudy, 62°F
5. Measurements were collected from the Top of PVC Pipe



1. Percolation test PT-1 was performed approximately 10 feet below existing grade.
2. Refer to Figure 1 for percolation test location.

LANGAN

Langan Engineering & Environmental Services, Inc.
18575 Jamboree, Suite 150
Irvine, CA 92612
P: 949.255.8640 F: 949.255.8641 www.langan.com

Project

Project HERE
Mill Creek Drive and
Ridge Route Drive

LAGUNA HILLS
ORANGE COUNTY CALIFORNIA

Title

PERCOLATION TEST RESULTS
PT-1

Project No.
700128701
Date
March 2023
Scale
N/A
Prepared By:
RF

PERCOLATION TEST DATA SHEET

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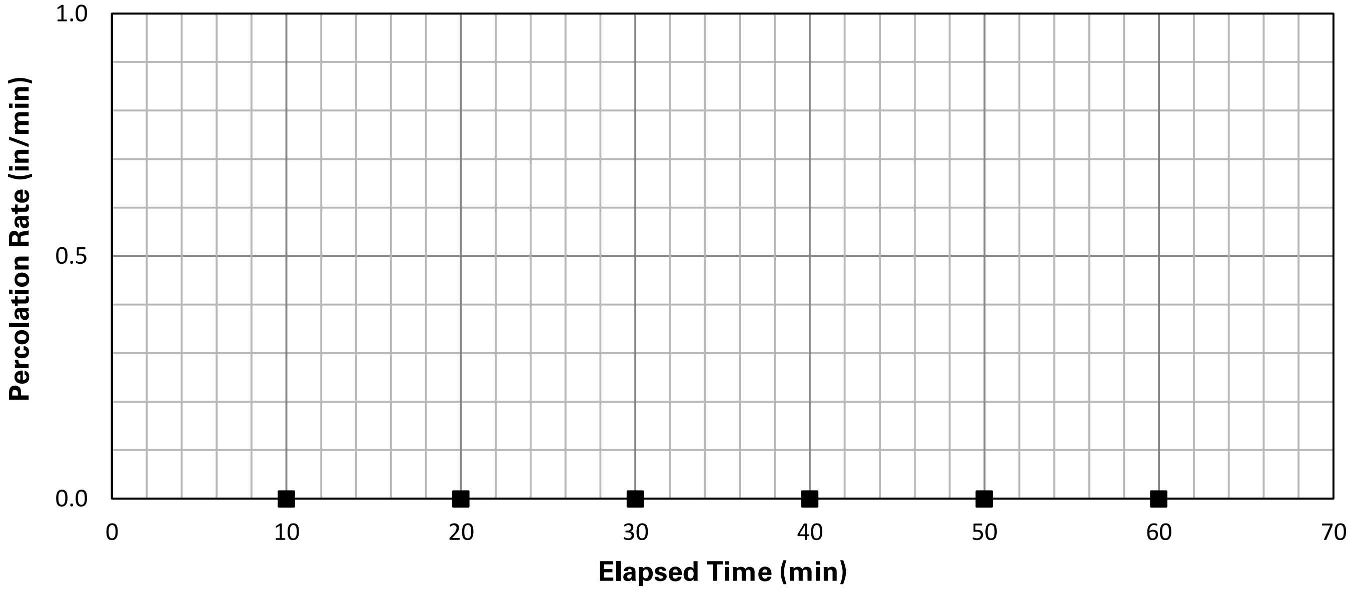
Project:	Project HERE - Laguna Hills	Project No.:	700128701	Date of Test:	3/16/2023
Test Hole No.:	PT-2	Tested By:	DJJ		
Depth of Test Hole (ft):	10	USCS Soil Classification:	Clayey SAND (SC)		
Casing Depth (ft):	10.0' PVC Pipe; Perforated entire length of pipe	Test Hole Diameter (in):	8		

Trial No.	Date	Time of Measurement	Initial Depth to Water (Feet)	Time of Measurement	Final Depth to Water (Feet)	Time Interval (min)	Change in Water Level (Feet)	Percolation Rate (in/min)	Infiltration Rate (in/hr)
Pre-Soak #1	3/16/2023	8:00 AM	3.70	8:30 AM	3.70	30	0.00	0.00	
Pre-Soak #2	3/16/2023	8:30 AM	3.70	9:00 AM	3.70	30	0.00	0.00	
1	3/16/2023	9:30 AM	3.70	10:00 AM	3.70	30	0.00	0.00	
2	3/16/2023	10:00 AM	3.70	10:30 AM	3.70	30	0.00	0.00	
3	3/16/2023	10:30 AM	3.70	11:00 AM	3.70	30	0.00	0.00	
4	3/16/2023	11:00 AM	3.70	11:30 AM	3.70	30	0.00	0.00	
5	3/16/2023	11:30 AM	3.70	12:00 PM	3.70	30	0.00	0.00	
6	3/16/2023	12:00 PM	3.70	12:30 PM	3.70	30	0.00	0.00	
7	3/16/2023	12:30 PM	3.70	1:00 PM	3.70	30	0.00	0.00	
8	3/16/2023	1:00 PM	3.70	1:30 PM	3.70	30	0.00	0.00	
9	3/16/2023	1:30 PM	3.70	2:00 PM	3.70	30	0.00	0.00	
10	3/16/2023	2:00 PM	3.70	2:30 PM	3.70	30	0.00	0.00	
11	3/16/2023	2:30 PM	3.70	3:00 PM	3.70	30	0.00	0.00	
12	3/16/2023	3:00 PM	3.70	3:30 PM	3.70	30	0.00	0.00	0.0

Comments:

1. Percolation test was performed in accordance with the Orange County - Technical Guidance Document dated 20 December 2013.
2. Infiltration Rate was calculated using Porchet Method.
3. Per the procedures for shallow percolation tests in non-sandy soils, a minimum of twelve measurements were taken in 30-minute intervals for six hours after sandy soil criteria was not met.
4. Weather: Cloudy, 62°F
5. Measurements were collected from the Top of PVC Pipe

PT-2



1. Percolation test PT-2 was performed approximately 10 feet below existing grade.
2. Refer to Figure 1 for percolation test location.

LANGAN

Langan Engineering & Environmental Services, Inc.
18575 Jamboree, Suite 150
Irvine, CA 92612
P: 949.255.8640 F: 949.255.8641 www.langan.com

Project

Project HERE
Mill Creek Drive and Ridge Route Drive

LAGUNA HILLS
ORANGE COUNTY CALIFORNIA

Title

PERCOLATION TEST RESULTS
PT-2

Project No.
700128701

Date
March 2023

Scale
N/A

Prepared By:
RF

APPENDIX D

LABORATORY TEST RESULTS

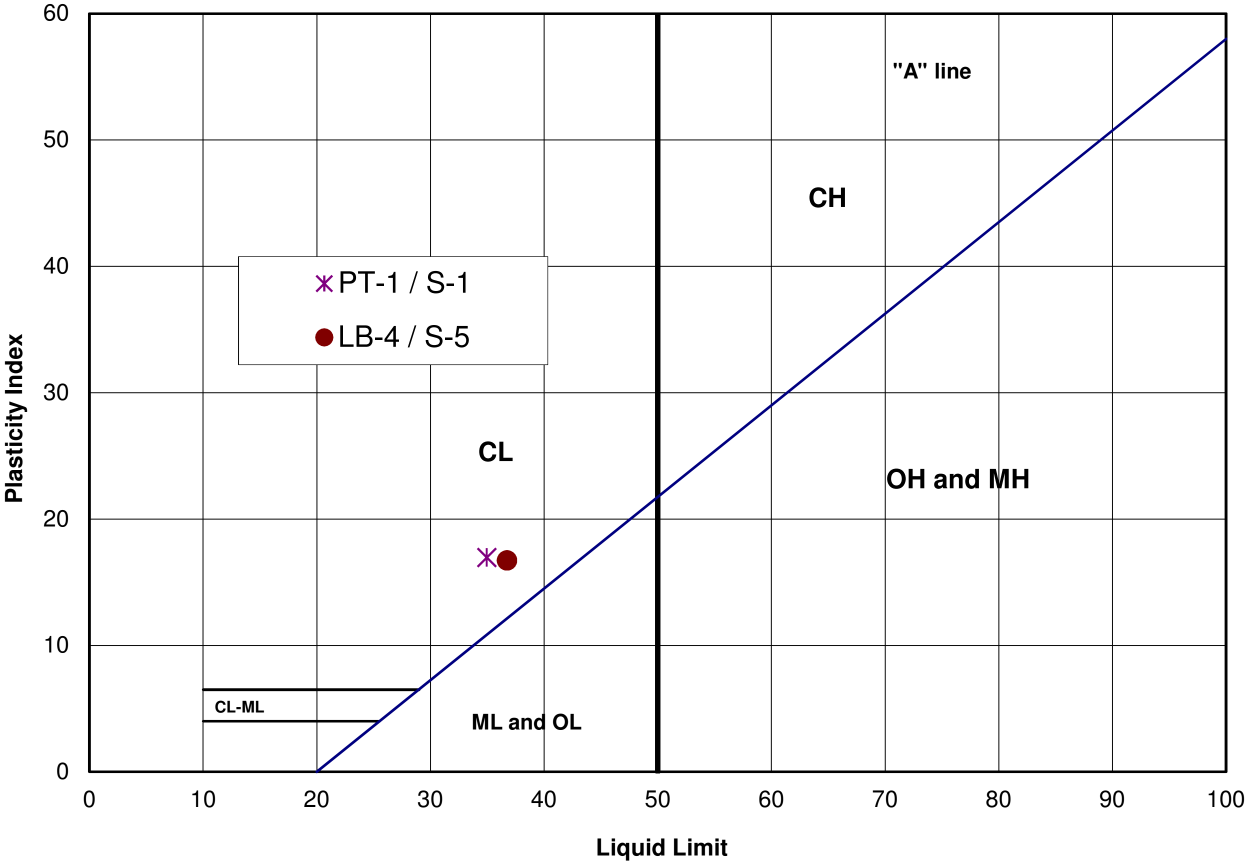
MOISTURE DENSITY TESTS

PROJECT Langan # 700128701 **JOB NO.** 2012-0057 **BY** LD **DATE** 03/28/23

Sample No.	**PT-2 / S-3**	**LB-3 / S-1**	**LB-4 / S-3**					
Depth (ft)	**10.0**	**5.0**	**7.5**					
Testing								
Soil Type	Brown, Clayey Sand	L. Gray, Silty Sand	Brown, Sandy Clay					
Wet+Tare		809.7	1023.4					
Tare		4	5					
Wet Weight	126.4	162.7	163.2					
Dry Weight	115.3	148.7	142.2					
Wet density		131.1	133.0					
% Water	9.6	9.4	14.8					
Dry Density	0.0	119.8	115.9					
O.B.Press(psf)								
Sample No.								
Depth (ft)								
Testing								
Soil Type								
Wet+Tare								
No. Ring								
Wet Weight								
Dry Weight								
Wet density								
% Water								
Dry Density								
O.B.Press(psf)								

Geo-Logic
ASSOCIATES

PLASTICITY INDEX _ ASTM D4318



Sample	Depth	LL	PL	PI	USCS	Material Description
PT-1 / S-1	5'	35	18	17	CL	
LB-4 / S-5	15'	37	20	17	CL	

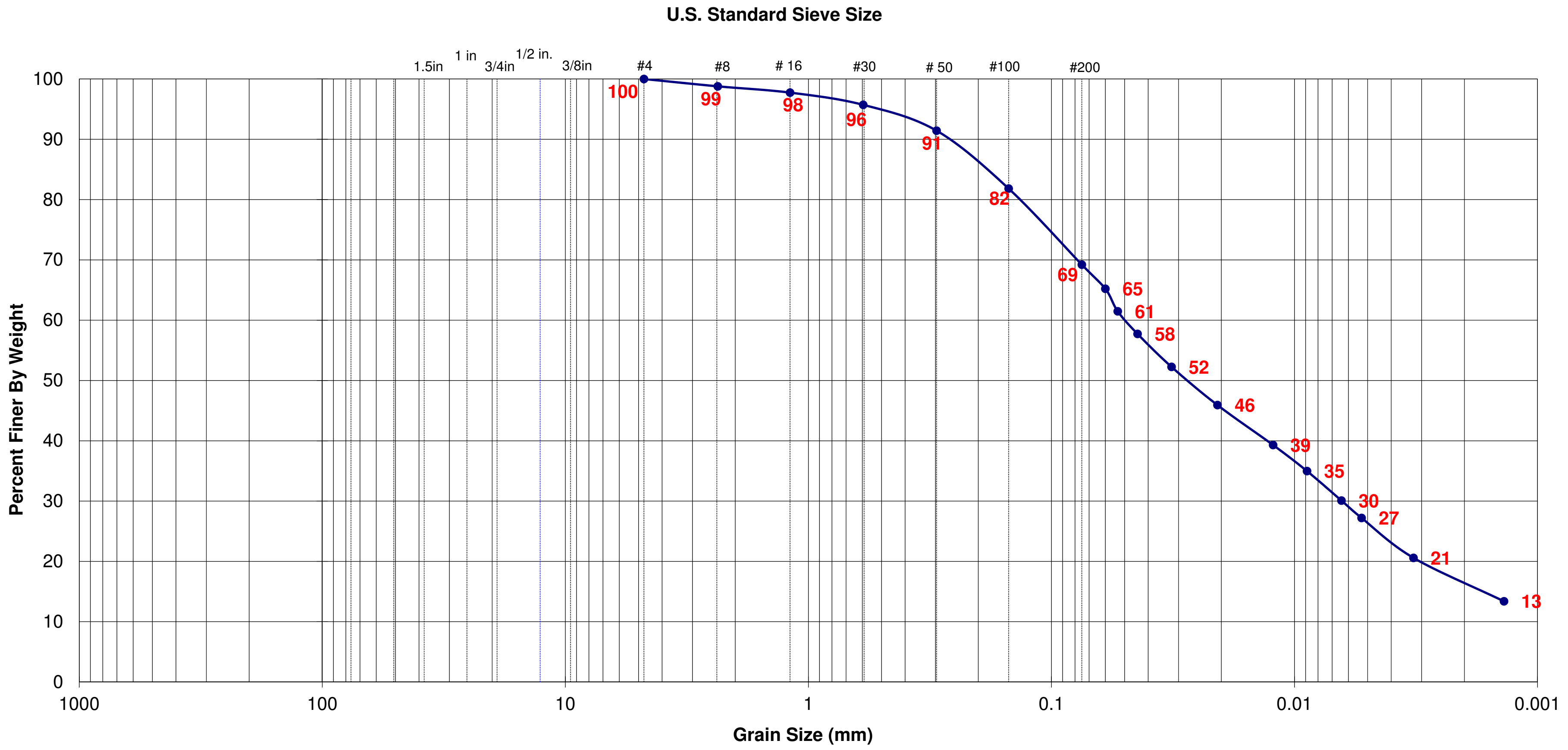
Job Name: Langan # 700128701

Date: 3-28-23

Job No.: 2012-0057

GRAIN SIZE ANALYSIS - ASTM D 422

Date: 3/28/23



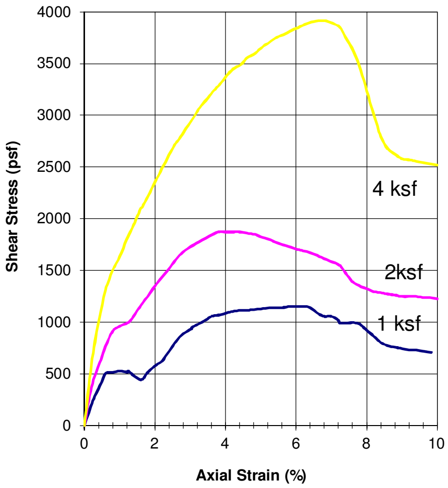
Boring / Sample No.	Initial Dry Density (pcf)	Initial Moist. (%)	Test Dry Density (pcf)	% Passing No. 200 Sieve	Sand Equivalent (ASTM D2419)	LL	PL	PI	Unified Soil Class.	Description
PT-1 / S-2									**CL**	

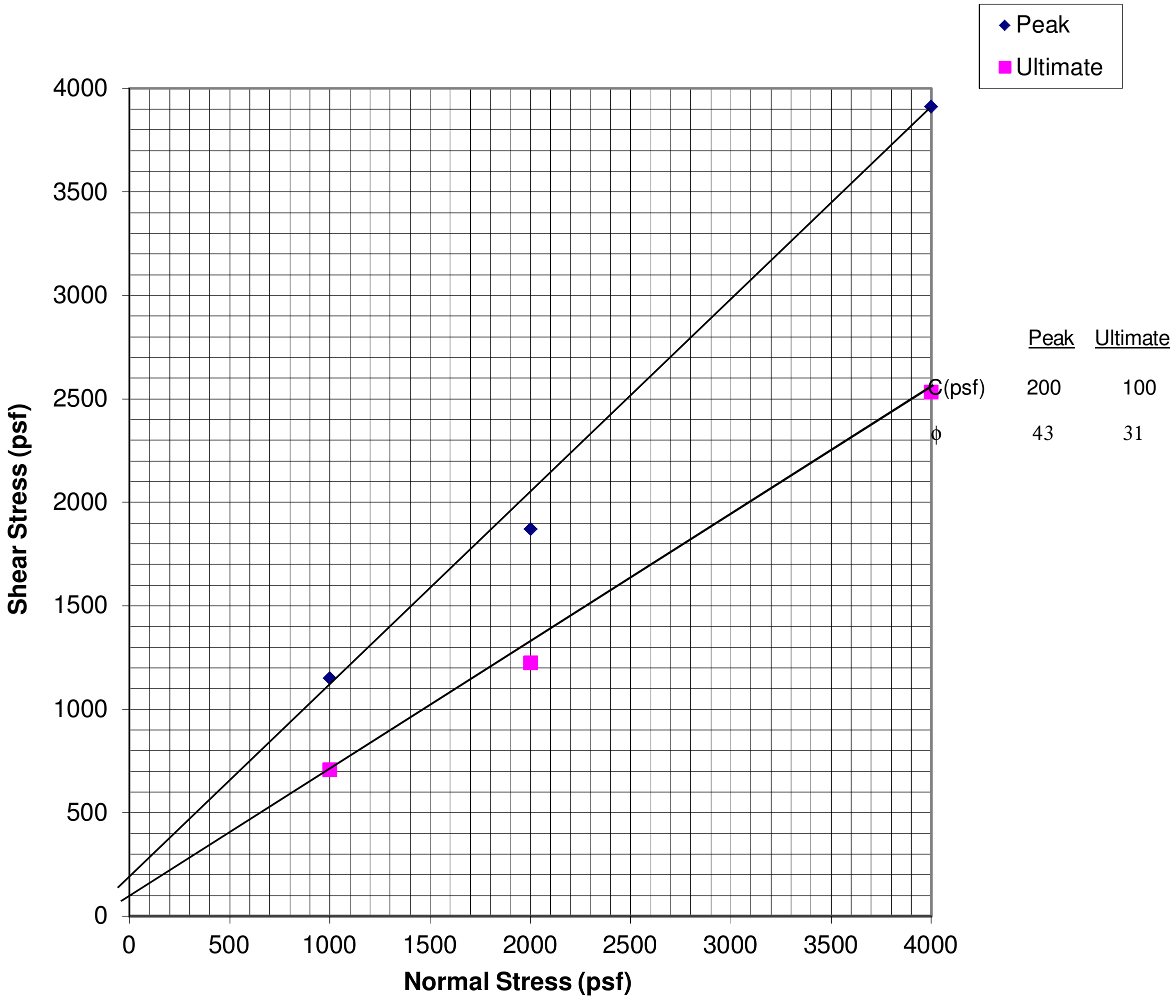
Geo-Logic ASSOCIATES

DIRECT SHEAR
ASTM D3080

PROJECT: Langan # 700128701 Date: 3/27/2023
GLA JOB NO.: 2012-0057
SAMPLE : LB-1 / S-1
SAMPLE TYPE: Undisturbed
DESCRIPTION: Clayey Sand

Specimen No.	1	2	3
Normal Stress, psf	1000	2000	4000
Peak Stress, psf	1152	1872	3912
Displacement, % strain	5.84	3.8	6.64
Ultimate Stress, psf	708	1224	2532
Displacement, % strain	10	10	10
Initial Dry Density, pcf	127.2	127.2	127.2
Initial Water Content,%	9.9	9.9	9.9
Final Water Content,%	18.0	18.0	18.0
Strain Rate, in/min.	0.0084	0.0084	0.0084





GeoLogic Associates

DIRECT SHEAR
ASTM D3080

PROJECT: Langan # 700128701

Date: 3/27/2023

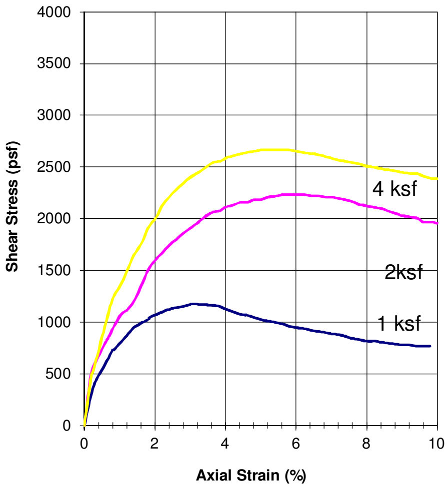
GLA JOB NO.: 2012-0057

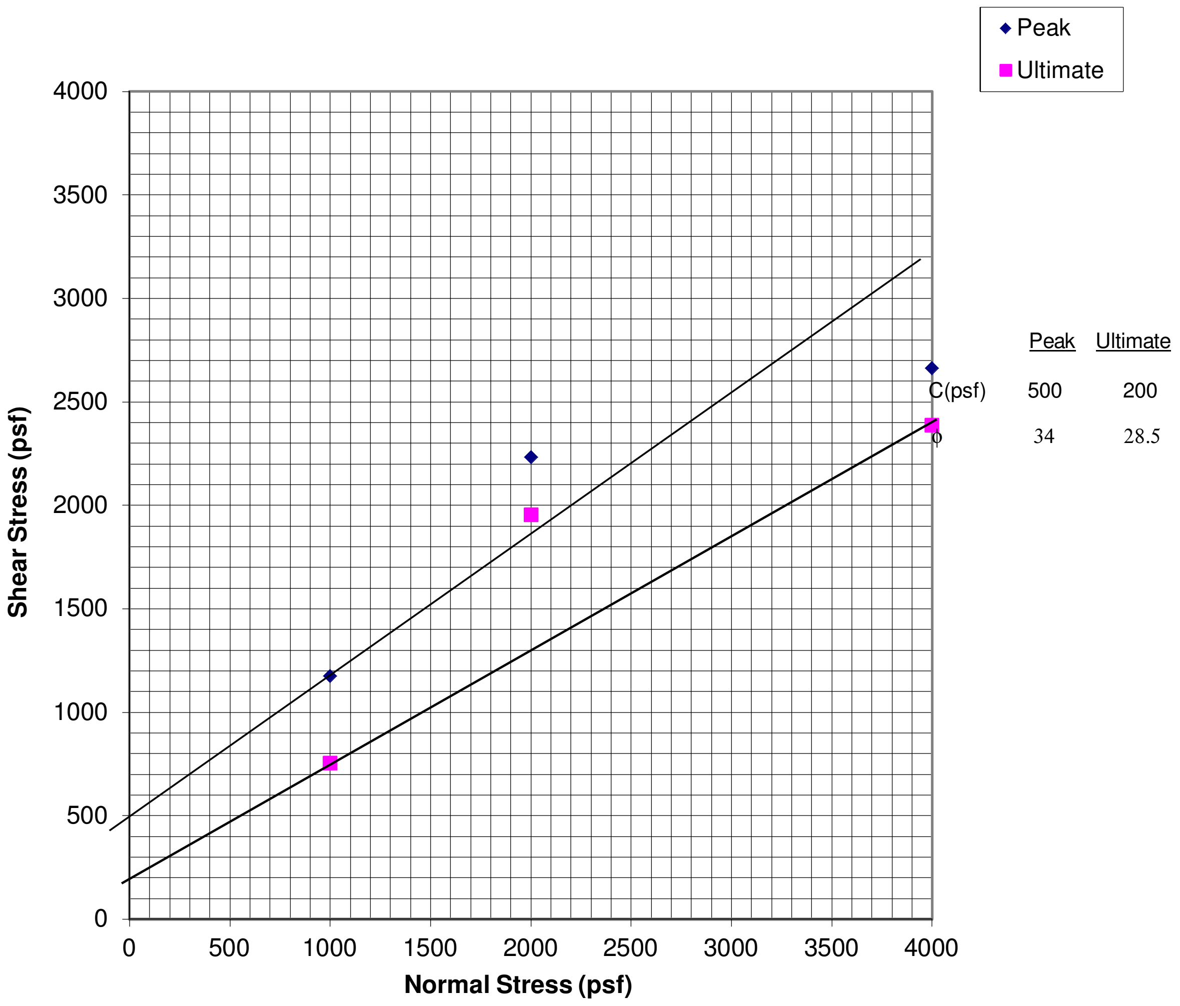
SAMPLE : LB-3 / S-2

SAMPLE TYPE: Undisturbed

DESCRIPTION: Silty Clay

Specimen No.	1	2	3
Normal Stress, psf	1000	2000	4000
Peak Stress, psf	1176	2232	2664
Displacement, % strain	3.04	5.6	5
Ultimate Stress, psf	756	1956	2388
Displacement, % strain	10	10	10
Initial Dry Density, pcf	116.2	116.2	116.2
Initial Water Content,%	15.2	15.2	15.2
Final Water Content,%	18.5	18.5	18.5
Strain Rate, in/min.	0.0084	0.0084	0.0084



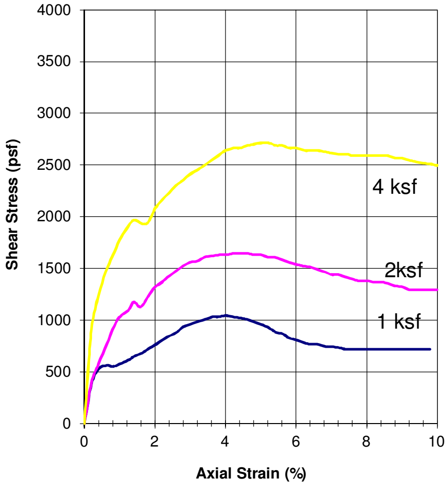


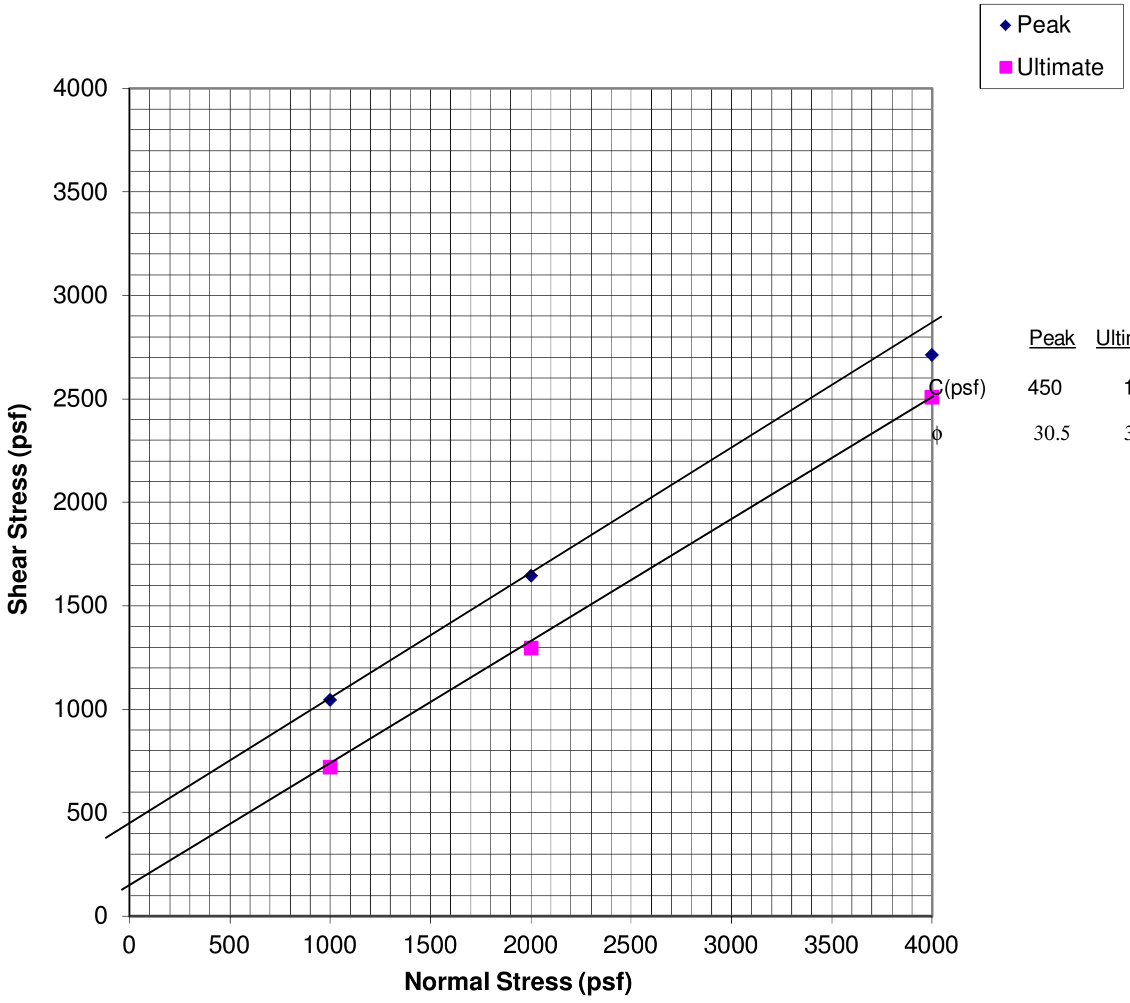
	Peak	Ultimate
C(psf)	500	200
φ	34	28.5

DIRECT SHEAR
ASTM D3080

PROJECT:	Langan # 700128701	Date:	3/27/2023
GLA JOB NO.:	2012-0057		
SAMPLE :	LB-4 / S-1		
SAMPLE TYPE:	Undisturbed		
DESCRIPTION:	Silty Clay		

Specimen No.	1	2	3
Normal Stress, psf	1000	2000	4000
Peak Stress, psf	1044	1644	2712
Displacement, % strain	4	4.2	5
Ultimate Stress, psf	720	1296	2508
Displacement, % strain	10	10	10
Initial Dry Density, pcf	114.3	114.3	114.3
Initial Water Content,%	15.2	15.2	15.2
Final Water Content,%	17.6	17.6	17.6
Strain Rate, in/min.	0.0084	0.0084	0.0084





	Peak	Ultimate
C(psf)	450	150
ϕ	30.5	30.5

'R' VALUE CA 301

Client: Langan Engineering

Date: 3/28/23

By: LD

Client's Job No.: **700128701**

Sample No.: PT-1 / B-1

GLA Reference: 2001-064

Soil Type: Brown, Silty Clay

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	**140**	**70**	**100**	
Initial Moisture Content	%	**14.2**	**14.2**	**14.2**	
Water Added	ml	**40**	**70**	**55**	
Moisture at Compaction	%	18.0	20.9	19.4	
Sample & Mold Weight	gms	**3200**	**3158**	**3167**	
Mold Weight	gms	**2106**	**2092**	**2098**	
Net Sample Weight	gms	1094	1066	1069	
Sample Height	in.	**2.501**	**2.529**	**2.482**	
Dry Density	pcf	112.3	105.7	109.3	
Pressure	lbs	**6360**	**2135**	**3700**	
Exudation Pressure	psi	506	170	295	
Expansion Dial	x 0.0001	**100**	**25**	**54**	
Expansion Pressure	psf	433	108	234	
Ph at 1000lbs	psi	**54**	**70**	**62**	
Ph at 2000lbs	psi	**118**	**140**	**129**	
Displacement	turns	**4.04**	**5.85**	**4.62**	
R' Value		18	6	12	
Corrected 'R' Value		**18**	**6**	**12**	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	**12**
By Epansion Pressure :	**7**
TI = 5	

EXPANSION INDEX - UBC 18-2 & ASTM D 4829-88

PROJECT Langan # 700128701

JOB NO. 2012-0057

Sample PT-1 / B-1 By LD

Sta. No.

Soil Type Brown, Silty Clay

Date	Time	Dial Reading	Wet+Tare	602.4
3/25/2023	16:20	0.2392	Tare	219.4
		H2O	Net Weight	383
3/26/2023	10:00	0.1825	% Water	11.5
			Dry Dens.	104.1
			% Max	
			Wet+Tare	653.3
			Tare	219.4
			Net Weight	433.9
INDEX	57	5.7%	% Water	26.3

Sample By

Sta. No.

Soil Type

Date	Time	Dial Reading	Wet+Tare	
			Tare	
			Net Weight	
			% Water	
			Dry Dens.	
			% Max	
			Wet+Tare	
			Tare	
			Net Weight	
INDEX			% Water	

Sample By

Sta. No.

Soil Type

Date	Time	Dial Reading	Wet+Tare	
			Tare	
			Net Weight	
			% Water	
			Dry Dens.	
			% Max	
			Wet+Tare	
			Tare	
			Net Weight	
INDEX			% Water	

Sample By

Sta. No.

Soil Type

Date	Time	Dial Reading	Wet+Tare	
			Tare	
			Net Weight	
			% Water	
			Dry Dens.	
			% Max	
			Wet+Tare	
			Tare	
			Net Weight	
INDEX			% Water	

Langan # 700128701

SOIL TEST RESULTS

Job No. 2012-0057

SAMPLE NO.:	LB-2 / B-1				
DESCRIPTION	Silty Sand				
DIRECT SHEAR TEST (type)					
Initial Moisture Content %					
Dry Density (pcf)					
Normal Stress (psf)					
Peak Shear Stress (psf)					
Ultimate Shear Stress (psf)					
Cohesion (psf)					
Internal Friction Angle (degrees)					
EXPANSION TEST UBC STD 18-2					
Initial Dry Density (pcf)					
Initial Moisture Content %					
Final Moisture Content %					
Pressure (psf)					
Expansion Index Swell %					
CORROSIVITY TEST					
Resistivity (CTM643) (ohm-cm)	6200				
pH (CTM643)	7.5				
CHEMICAL TESTS					
Soluble Sulfate (CTM 417) (ppm)	106				
Chloride Content (CTM 422) (ppm)	9				
Wash #200 Sieve (ASTM-1140) %					
Sand Equivalent (ASTM D2419)					