SA GEOTECHNICAL
OPTIMIZED SOIL ENGINEERING

March 4, 2026

Project No. 25093-01

To: Lennar
2000 FivePoint, 3rd Floor
Irvine, California 92618

Attention: Mr. Derek Kegley

Subject: Geotechnical Subsurface Exploration and Preliminary Design Report, Proposed Residential Development, 1 through 5 Polaris Way, Vesting Tentative Tract Map No. 19477, City of Aliso Viejo, California

At your request, SA Geotechnical, Inc. (SA GEO) has conducted geotechnical subsurface exploration and prepared preliminary design recommendations for the subject proposed residential development at 1 through 5 Polaris Way, City of Aliso Viejo, California (Figure 1). The purpose of this study was to evaluate the geotechnical site conditions in light of the proposed residential redevelopment in order to provide a geotechnical summary and preliminary geotechnical recommendations for project design, grading, and construction. Our evaluation included review of background geologic and geotechnical engineering maps and reports for the subject site; a site-specific subsurface exploration; laboratory testing; geotechnical analysis; and preparation of this report. This report has been updated in accordance with the 2025 California Building Code and ASCE 7-22, and supersedes the prior report (SA GEO, 2025).

The subject site was previously mass graded during the late-1980s and remained undeveloped until the late-1990's, when the existing commercial center was constructed. We recovered the geotechnical report documenting the mass grading. Based on our background review and exploration, the mass grading resulted in bedrock at or near existing grades throughout the north/northwestern portion of the site, while the south/southeastern portion was filled to current grades (requiring fills on the order of 2.5 to 47.5 feet thick). The primary geotechnical constraints include the presence of expansive, diatomaceous earth materials (Monterey Formation and existing fill materials), the presence of bedrock at or near existing grades in the northern portion of the site, and the potential for seismic shaking during a strong seismic event. Groundwater was encountered during our exploration in one boring, Boring H-7, at a depth of approximately 51.5 feet.

This report presents our findings, conclusions, and preliminary design recommendations for the proposed residential development. Based on our review, the proposed grading and development is considered geotechnically feasible provided the recommendations in this report are implemented during design, grading, and construction. Additional geotechnical evaluation and analysis will need to be performed once the project plans for grading, utility improvements and foundations are developed. Stormwater infiltration at the subject site is not considered feasible from a geotechnical viewpoint.

References pertinent to the site are included in Appendix A. Boring logs are included in Appendix B. Laboratory test data is included in Appendix C. Seismic design parameters are presented in Appendix D. Slope stability analysis is attached as Appendix E. General earthwork and grading specifications are presented in Appendix F.

If you have any questions regarding this report, please contact our office. We appreciate the opportunity to provide our services.

Respectfully submitted,

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EXECUTIVE SUMMARY

The subject site is underlain by compacted artificial fill materials and bedrock of the Monterey Formation as a result of mass grading in the late-1980s. The north/northwestern portion of the site is underlain by Monterey Formation bedrock at or near existing grades. The south/southeastern portion of the site was brought to current grades with compacted fill (overlying bedrock), ranging in thickness from 2.5 to 47.5 feet thick. The Monterey Formation bedrock generally consisted of white, yellowish brown, gray, olive brown, and pale brown siltstone that was damp to wet, stiff to hard, and diatomaceous. Bedding was observed in the samples to be generally well bedded/laminated and highly variable across the site. The artificial fill materials generally consisted of olive gray, olive brown, reddish brown, yellowish brown, and brown clayey silt, silty clay, clayey sand, and silty sand that was damp to wet and medium dense/stiff to hard. The fine-grained fill materials are generally derived from the Monterey Formation and were found to be diatomaceous.

The primary geotechnical constraints for the proposed site redevelopment include the following:

- Presence of expansive, diatomaceous earth materials (Monterey Formation and existing artificial fill materials);
- The presence of Monterey Formation bedrock at or near existing grades in the north/northwestern portion of the site; and,
- Potential for strong seismic shaking during an earthquake event.

Remedial grading for the site should consist of the removal and recompaction of weathered/disturbed fill materials and near-surface bedrock. In general, recommended remedial removals and/or overexcavations within the proposed building pads are anticipated to be 3 feet below existing grades or below design grades, whichever is lower. Locally, where foundations or large trees are to be removed, remedial removals will need to extend deeper and below disturbed or unsuitable soils. Remedial grading within the proposed drive areas may be limited to removal and recompaction of the upper 1 to 2 feet.

Static groundwater was encountered in one boring, Boring H-7, at a depth of 51.5 feet bgs. In general, groundwater is anticipated to remain more than 30 feet below design grades upon the completion of grading. However, seepage/groundwater may be present along fill/bedrock contacts and at shallow depth locally/seasonally.

Building foundations and slabs should be designed to tolerate a total settlement of 1-inch and differential settlement of ½-inch over a span of 40 feet. Onsite soils are anticipated to have "Medium" to "High" expansion potential at the completion of grading. Onsite soils are anticipated to be corrosive to concrete and metals.

Based on our findings, we conclude that the proposed residential development is feasible from a geotechnical viewpoint, provided it is designed and constructed in accordance with the recommendations presented in this report and any future design/plan review reports. The site is not considered suitable for infiltration of stormwater from a geotechnical viewpoint.

1.0 INTRODUCTION

1.1 Introduction and Scope of Services

At your request, SA Geotechnical, Inc. (SA GEO) has conducted geotechnical exploration, analysis, and review for the subject proposed residential development at 1 through 5 Polaris Way (existing Polaris Place Commercial Center) in the city of Aliso Viejo, California (Figure 1). The purpose of our review was to assess the onsite geologic and geotechnical conditions and provide preliminary recommendations for design, grading, and construction of the proposed improvements. We have reviewed Vesting Tentative Tract Map (VTTM) No. 19477, prepared by C&V Consulting, Inc., dated February 19, 2026, which shows the general building and street layout and provides preliminary street and pad grades. The Preliminary Grading Plan (included as part of the VTTM) and existing site topography were used as the base map for our Geotechnical Map (Plate 1) and Geotechnical Cross-Sections (Plates 2 and 3).

Our scope of services for this study included the following tasks:

- Review of available geologic and geotechnical maps, reports, and data for the subject site and surrounding area, including an archive search of City of Aliso Viejo records. A complete list of references is included in Appendix A.
- Review of available historic aerial photographs dating back to 1938.
- Notification and coordination with DigAlert to identify and clear boring locations of underground utilities.
- Subsurface exploration consisting of excavation, sampling and logging of 13 hollow-stem auger borings (H-1 through H-13) and one hand-auger boring (HA-1) to depths ranging from 11.5 to 66.5 feet below ground surface (bgs).
- Laboratory testing of selected samples to determine engineering properties of onsite soils, including in-situ moisture and density, grain size distribution, Atterberg limits, consolidation, shear strength, maximum density and optimum moisture content, expansion potential, and soluble sulfate content.
- Geotechnical evaluation and analysis of the compiled data regarding the proposed grading and development.
- Evaluation of faulting, seismicity, and seismic and static settlement in accordance with the 2025 California Building Code (CBC).
- Preparation of this report including our findings, conclusions, preliminary recommendations, and accompanying illustrations.
- Consultations with the project team.

SA GEO's expertise and scope of services do not include assessment of potential subsurface environmental contaminants or environmental health hazards.

1.2 Site Location, Existing Conditions, and Prior Mass Grading

The approximately 21.5-acre, roughly rectangular-shaped site encompasses 1 through 5 Polaris Way, Aliso Viejo, California (Figure 1). The site is bound to the north by Aliso Viejo Parkway, SR-73 to the west, and existing residential development to the south and east.

A site-specific topographic map was not available for our review during preparation of this report. Based on our site reconnaissance and review of existing site topography, the site gently slopes to the east/southeast, with grade changes on the order of 25 feet across the site. Descending slopes are present along the eastern, southern, and western perimeters, approximately 10 to 35 feet in height, extending beyond the property boundary. The slopes appear to generally be 2H:1V (horizontal:vertical) and include terrace drains. One ascending slope is located in the northwest corner of the site (SR-73 embankment), up to 25 feet in height. The site is presently developed with five multi-story commercial structures, asphalt concrete paved drive aisle and parking lot, and landscape/hardscape improvements.

As part of our study, we performed a search of City of Aliso Viejo geotechnical archives. The subject site was previously identified as Lots 35 through 38 of Planning Area 24.1, Tentative Tract 12810. We recovered the geotechnical report documenting mass grading of the subject site (Nolte and Associates, 1988). Per our review of the report, the subject site was mass graded by C.W. Poss Incorporated, between January 1987 and September 1988, resulting in the present-day topography and subsurface conditions. Geotechnical observation and testing was performed by Nolte and Associates. The report states that during rough grading, all topsoil, colluvium and highly weathered bedrock were removed prior to any compacted fill placement. Per the report, slopes within the grading limits, including any cut slopes, were buttressed or stabilized and were concluded to have adequate factor of safety to remain stable under normal conditions.

During our site visits, we noticed significant and large cracks (up to 3-4 inches in width) within the existing asphalt concrete pavement at various locations. However, the cracks do not appear to be related to prior fill placement or slope creep/movement as some of the more prominent cracks were present within the northern portion of the site which was cut to grade (underlain by bedrock). The cracks are generally expected to be a result of asphalt concrete shrinkage (over the years), poor maintenance, expansive nature of onsite soils and/or drainage issues.

1.3 Site History

Based on our review of available historic aerial photographs dating back to 1938, the following summarizes our understanding of the site history:

- **1938-1985:** The site and surrounding area were undeveloped natural scrubland. Natural topographically low-lying drainage areas were present in the eastern and southern portions of the site. A northwest trending ridgeline is present through the central portion of the site. The site remained undeveloped during this period except for limited dirt access roads/trails traversing the site and surrounding area.
- **1987-1999:** Mass grading began at the site and surrounding areas to the west and north, encompassing large portions of the City of Aliso Viejo. Additional mass grading details are discussed above (Section 1.2). Grading onsite was completed by 1988 and resulted in grading

of several commercial building pads and interior loop road. The site remained undeveloped from 1988 until 1998, when it was precise graded to the current configuration and building construction was underway. The subject site was under construction during 1998 and 1999.

- **1999-Present:** No significant onsite changes were observed following construction of the existing building and site improvements.

1.4 Proposed Grading and Improvements

Prior to any site development or grading, the existing structures, pavements, utilities, and hardscape will be demolished. Based on our review of the Site Plan, the development is proposed to include construction of 4-story duplex buildings (105 units), 3-story Townhome (108 units), 3-story B2B (100 units), and 3-story Rowtowns (92 units), with a total of 405 units overall. The residential structures are anticipated to consist of wood-framed construction. The project also includes interior streets, a pool/recreation area, retaining/screen walls, and utility improvements to support the development.

Based on our review of the TTM/Preliminary Grading Plan, design cuts and fills on the order of 1 to 8 feet will be required to create a relatively level site, interior streets, and building pads. We do not anticipate significant modification to any perimeter slopes; however, while not currently shown on the preliminary grading plan, we do anticipate that minor retaining walls will be required along the property boundaries, where proposed grades are elevated above the existing top of slopes (see Plates 2 and 3).

1.5 Subsurface Exploration

Our field exploration was performed September 10 through 12, 2025, and included excavation of 13 hollow stem auger borings (H-1 through H-13) and one hand-auger boring (HA-1) to depths ranging from 11.5 to 66.5 feet bgs.

The borings were geologically logged, and samples were obtained at selected intervals. After completion of drilling, the excavations were backfilled with soil cuttings and tamped. Pavement at the boring locations was repaired with cold patch asphalt. The approximate boring locations are shown on Plate 1 (Geotechnical Map). Boring logs are included in Appendix B.

Soil sampling was performed using a Modified California ring sampler. Relatively undisturbed ring samples were obtained from the borings at various depths with a 2.5-inch inside-diameter, split-barrel sampler. The sampler was driven with a 140-pound, automatic trip hammer, free falling 30 inches. The sampling was used to assess the soil beneath the site as well as to obtain a measure of resistance of the soil to penetration (recorded as blows-per-foot on the geotechnical boring logs). Representative bulk samples of onsite soils were also collected from soil cuttings and used for additional soil identification purposes and laboratory testing.

1.6 Laboratory Testing

Representative samples of onsite soils collected during our field exploration were tested to characterize the engineering properties. Laboratory tests performed on selected relatively undisturbed and bulk soil samples included:

- Moisture content and dry density;
- Sieve analysis;
- Atterberg limits;
- Direct shear;
- Consolidation;
- Maximum density and optimum moisture content;
- Expansion potential; and
- Soluble sulfate.

Laboratory tests were conducted in general conformance with applicable ASTM International standards. The laboratory test results are provided in Appendix C.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting and Earth Units

The subject site is located on the eastern foothills of the San Joaquin Hills range, within the Peninsular Ranges geomorphic province of Southern California. The site is mapped by the United States Geologic Survey (2006) as underlain by Tertiary-age sedimentary bedrock of the Monterey Formation. Based on our subsurface exploration and understanding of the site history, the subject site was mass graded in the late-1980's resulting in placement of relatively thick artificial fill materials (over bedrock) in the south/southeastern half of the site. The north/northwestern portion of the site is underlain by shallow Monterey Formation bedrock at or near existing grades. Based on our exploration, alluvium, colluvium, and topsoil appear to have been removed during mass grading, prior to fill placement, consistent with discussions in the recovered report of mass grading (Nolte and Associates, 1988).

The earth unit distribution and our interpretation of the subsurface conditions are shown on the Geotechnical Map (Plate 1) and Geotechnical Cross-Sections A-A', B-B', and C-C' (Plates 2 and 3).

Our subsurface exploration encountered compacted **Artificial Fill (Af)** underlying the site as a result of original mass grading. Approximately 2.5 to 47.5 feet of artificial fill was encountered during drilling, and generally consisted of olive gray, olive brown, reddish brown, yellowish brown, and brown clayey silt, silty clay, clayey sand, and silty sand that was damp to wet and stiff to hard. The fine-grained fill materials are generally derived from the Monterey Formation and were found to be diatomaceous.

We encountered sedimentary bedrock of the **Monterey Formation (Tm)** at/near existing grades (north/northwestern portion of the site) and underlying the fill materials. The Monterey Formation bedrock generally consisted of white, yellowish brown, gray, olive brown, and pale brown siltstone that was damp to wet, stiff to hard, and diatomaceous. Bedding was observed in the samples to be generally well-bedded/laminated and variable across the site. This is also consistent with bedrock discussions in the site mass grading report (Nolte and Associates, 1988) and regional geologic mapping (USGS, 2006).

2.2 Geotechnical Conditions

Based on our geotechnical exploration and laboratory testing data (Appendix C), the geotechnical conditions are generally as follows:

Soil Moisture Content and Dry Density: The collected samples show that dry densities of the diatomaceous Monterey Formation bedrock generally vary from 34.7 to 72.8 pcf with moisture contents in the range of 33.2 to 142.5 percent. Blow counts in Monterey Formation generally ranged from 7 to 73 blows per foot. Please note that lower blow counts were encountered at or near groundwater level (in Boring H-7). Dry densities of the artificial fill material varied from 52.5 to 111.9 pcf with moisture contents in the range of 13.3 to 72.5 percent. Blow counts in the artificial fill material generally ranged from 14 to 70 blows per foot.

Soil Properties: Grain-size distribution tests were conducted on four samples, representative of the Monterey Formation and existing artificial fill material. The Monterey Formation sample had a fines content (passing Sieve No. 200) of 97 percent and the artificial fill samples had fines contents ranging from 70 to 79 percent. The samples were classified as highly plastic silt (MH) or fat clay (CH), per the Unified Soil Classification System (USCS).

Soil plasticity testing was performed on bulk and in-situ samples representative of Monterey Formation and existing fill materials. The samples were classified as highly plastic silt (MH) and clay (CH) with Plasticity Indices ranging from 38 to 52 and Liquid Limits ranging from 73 to 136 percent.

Maximum density testing was performed on three bulk samples representative of the existing artificial fill materials. The testing indicates maximum dry density of the diatomaceous soil ranges from 76.6 to 88.6 pcf with optimum moisture contents ranging from 24.0 to 36.8 percent.

Shear Strength: Direct shear testing was conducted on four relatively undisturbed samples representative of the existing fill materials and the bedrock of Monterey Formation. The following table includes the direct shear test results for various earth units and soil types:

Earth Unit	Soil Type	Ultimate Strengths		Peak Strengths	
		ϕ	C (psf)	ϕ	C (psf)
Fill (Af)	Clayey Silt (MH)	28	100	32	300
	Clayey Silt (MH)	33	50	38	200
	Clayey Silt (MH)	29	150	29	600
Monterey Formation (Tm)	Siltstone (MH)	28	400	28	800

Consolidation: Tests were performed on four samples collected at depths of 2.5, 5, 17.5, and 20 feet bgs, representative of existing fill materials. The testing shows that the fill materials have relatively low compressibility. The samples had negligible (no) collapse upon the addition of water at a load of 3.2 ksf.

Expansion Potential: Three samples were tested to determine the expansion potential of the onsite near-surface fill materials. The samples had Expansion Indices of 64 to 89, indicating "Medium" expansion potential.

Chemical Properties: Soluble sulfate content of the soil was measured in three samples, representative of the artificial fill materials. Soluble sulfate content testing indicates the samples had negligible soluble sulfate content. However, per our experience with the Monterey Formation and review of the laboratory testing from the nearby developments, we anticipate the soluble sulfate content of the onsite soils to be classified as "S2" per Table 19.3.1.1 of ACI-318-14.

2.3 Groundwater

Groundwater was encountered in one boring (Boring H-7) at a depth of 51.5 feet bgs. Historic high groundwater is not mapped by the State for this portion of the San Juan Capistrano Quadrangle (CDMG, 2001a); however, considering the site topography and prior mass grading, we anticipate groundwater to be relatively deep, and not a geotechnical constraint for design or construction.

Groundwater levels may fluctuate on an annual and seasonal basis based on rainfall and variations in future residential irrigation and/or adjacent land use. Perched water within the artificial fill or along earth unit contacts (i.e., fill/bedrock contact) cannot be ruled out and may be encountered during grading or utility construction.

2.4 Regional Faulting and Seismicity

Regional Faults: The site is not located within a fault-rupture hazard zone as defined by the Alquist-Priolo Special Studies Zones Act (CGS, 2018). Also, based on mapping by the State (Jennings and Bryant, 2010), there are no active faults mapped at the site. Regional faults are presented on Figure 3.

Seismicity: Properties in southern California are subject to seismic hazards of varying degrees depending upon the proximity, degree of activity, and capability of nearby faults. These hazards can be primary (i.e., directly related to the energy release of an earthquake) or secondary (i.e., related to the effect of earthquake energy on the physical world). Since there are no active faults at the site, the potential for primary ground rupture is considered very low. The primary seismic hazard for this site is ground shaking during a future earthquake. The maximum moment magnitude for the controlling fault is 7.1 M_W, which would be generated from the San Joaquin Hills Fault. Using the site coordinates of 33.58495 degrees latitude and -117.72402 degrees longitude, the San Joaquin Hills Fault is the closest fault to the site, approximately 2.2 miles to the west.

The site is not located within a mapped potential liquefaction hazard zone per the State's Seismic Hazard Mapping (CDMG, 2001b). The attached Site Location and Seismic Hazards Map (Figure 1) depicts the site relative to the mapped potential seismic hazard zones. Portions of the southerly and westerly perimeter slopes are located within mapped potential earthquake-induced landslide zones; however, considering that the slopes are composed of thick fill materials and/or were stabilized during grading (constructed at 2H:1V inclination), and that design/grading was performed under review of County of Orange, this potential hazard is assumed to have been mitigated as a result of the mass grading. Slope stability was reviewed as part of our study, based on our understanding of the slope grading/construction, and is further discussed in the following section (Section 2.5). Other secondary seismic hazards, such as tsunami and seiche are considered nil due to site elevation and distance from the ocean or other confined bodies of water (CGS, 2021).

2.5 Slope Stability

The site geologic/geotechnical conditions are generally favorable for overall stability of the existing slopes along the southerly portion of the site. Per the geotechnical report documenting mass grading (Nolte and Associates, 1988), slopes within the grading limits, including any cut slopes, were buttressed or stabilized and were concluded to have adequate factor of safety to

remain stable under normal conditions. We prepared cross-sections (Plates 2 and 3) depicting the existing and proposed descending slopes along the site perimeter. Slope stability analysis was performed on cross-section B-B', which represents the approximately 37-foot-high 2H:1V perimeter slope, consisting of fill over bedrock (including the proposed 5-foot retaining wall above). The soil strength parameters were developed during this study per the direct shear test results and our local experience with the earth units.

The following table includes the strength parameters used for slope stability analysis:

Earth Unit	Soil Type	Ultimate Strength		Peak Strength	
		ϕ	C (psf)	ϕ	C (psf)
Fill (Af)	Clayey Silt	27	200	27	200
Monterey Formation (Tm)	Siltstone with Cross Bedding Strength	28	300	28	300
Monterey Formation (Tm)	Siltstone with Along Bedding Strength	18	150	18	150

For pseudo-static analysis, we applied a seismic horizontal coefficient (k_h) of 0.21 in accordance with Special Publication 117A, Guidelines for Evaluation and Mitigating Seismic Hazards in California (CGS, 2008).

The results of the slope stability analysis are presented in Appendix E. The existing slope at the site was found to be grossly stable with a factor of safety of more than 1.5 under static conditions and a factor of safety of 1.1 or greater for pseudo-static conditions.

Please note that the parameters used for slope stability analysis as well as the methodology are considered somewhat conservative since no increase in the shear strengths were applied for pseudo-static analysis (typically a 20 percent increase or peak values are used) and also the bedrock unit is not anticipated to have continuous out of slope bedding.

2.6 Settlement and Foundation Considerations

In general, the anticipated settlements depend upon the building loads, type of foundations, and the geotechnical properties of the supporting subgrade. Considering the subsurface soil conditions, laboratory test data, and anticipated lightly loaded residential structures (3- to 4-story wood-framed), we estimate total settlement (combined static and seismic) to be on the order of 1-inch with differential settlement of ½-inch over a 40-foot span. The settlement estimate assumes remedial grading measures recommended in Section 3.2 of this report are implemented during grading.

2.7 Stormwater Infiltration Feasibility

The southern portion of the site is underlain by variably thick artificial fill overlying fine-grained, expansive bedrock of the Monterey Formation. The remainder of the site is generally underlain by shallow or at-grade bedrock. County of Orange guidelines prohibit stormwater infiltration in compacted fill (OCPW, 2013). Additionally, infiltration of stormwater in bedrock can result in

migration of subsurface water along bedding, faults, or lithologic contacts, resulting in seepage or springs where previously none existed, potentially impacting offsite properties and slopes. Stormwater infiltration is not considered feasible from a geotechnical viewpoint.

2.8 Shrinkage and Bulking

The shrinkage and bulking (reduction or increase in volume of excavated materials upon recompaction and replacement as fill) varies by soil type, earth unit, and location. The volume change primarily depends on in-situ density and the maximum dry density of the soil type. We anticipate that shrinkage/bulking of the bedrock and compacted artificial fill will be relatively minor (negligible). Ground subsidence at the site is estimated to be on the order of 0.1 foot.

2.9 Existing Utilities and Pavement Section

Existing onsite utilities consisting of sewer, water, and gas/electric lines are present, supporting the existing commercial buildings and associated improvements. We anticipate that the existing utilities will be demolished as part of the proposed improvements and that new utilities and service connections will be established.

The existing pavement section was measured for each boring excavated during this study. The pavement section consisted of 3 to 6 inches of asphalt concrete (AC) over 6 to 10 inches of aggregate base (AB).

3.0 CONCLUSION AND PRELIMINARY RECOMMENDATIONS

3.1 General Conclusion and Recommendation

Construction of the proposed residential development, as described herein, is considered geotechnically feasible provided the preliminary recommendations in this report are implemented during design, grading, and construction. Additional geotechnical exploration is not considered necessary provided there are no significant changes to the proposed development. Grading, foundation, utility/street improvements, structural and wall plans for the project should be reviewed by the geotechnical consultant during the design phase. Updated recommendations may be provided once the project plans are finalized and as needed.

The recommendations in this report should be considered minimum and may be superseded by more restrictive requirements of others. In addition to the following recommendations, General Earthwork and Grading Specifications are provided in Appendix F.

3.2 Site Preparation and Earthwork

Site preparation and grading should be performed in accordance with the recommendations herein and the requirements of the City of Aliso Viejo.

3.2.1 Site Demolition and Clearing

Prior to remedial grading, any existing structures and utilities to be abandoned should be demolished. Deleterious materials and debris should be cleared and disposed of offsite. Concrete material may be mixed with onsite soils and placed as compacted fill provided that it is broken into pieces that are smaller than 6 inches in the largest diameter and placed in accordance with Section 3.2.4. Asphalt concrete may also be used in compacted fill materials; however, it should not be placed under the residential pads. Placement of concrete and asphalt as compacted fill should also be approved by the project environmental consultant.

Excavations for the removal of existing utilities and vegetation, including any onsite trees, should be observed by the geotechnical consultant. Large roots, highly organic soils, and utility/pipeline debris should be removed and should not be incorporated into new fills.

Soil that is disturbed as part of excavations or during removal of trees or underground utilities should be evaluated by the geotechnical consultant. Excavations that require backfill should be properly documented and compacted under the observation and testing of the geotechnical consultant in accordance with the recommendations provided in Section 3.2.4.

3.2.2 Protection of Existing Improvements and Utilities

Existing buildings, improvements, and utilities adjacent to the site that are to be protected in place should be located and visually marked prior to grading operations. Excavations adjacent to improvements to be protected in-place or any utility easement should be performed with care, so as not to undermine existing foundations or destabilize the adjacent ground.

Stockpiling of soils more than 5 feet in height at or near existing structures and over utility lines should not be allowed. If deep excavation/removals are required, shoring or other special measures (i.e., setback or laybacks) to provide safety and mitigate the potential for lateral/vertical movements may be required.

3.2.3 Remedial Grading Measures

Remedial grading at the site should consist of removal of highly weathered/disturbed bedrock and compacted fill materials. In general, remedial grading within the proposed building pads is anticipated to consist of removal/overexcavation of soils to a depth of 3 feet below existing or design finish grades (whichever is lower). Remedial grading within the proposed drive areas may be limited to removal and recompaction of the upper 1 to 2 feet provided competent fill materials and/or bedrock is exposed. Locally, deeper removals may be needed (on the order of 4 to 5 feet) where existing foundations, trees, utilities, etc., are planned to be removed.

The geotechnical consultant should review and approve the removal bottoms prior to fill placement and should provide specific recommendations based on actual conditions, as necessary.

Excavations deeper than 4 feet will need to be laid back a minimum inclination of 1H:1V (horizontal to vertical) or provided with shoring. Shallow, unconfined excavations (4 feet or less) may consist of near-vertical excavation. In general, the excavation within the existing bedrock and fill materials should be in accordance with Cal/OSHA requirements for Soil Type "C" (well-bedded siltstone/claystone bedrock) and Type "B" (compacted fill). The contractor's qualified person should verify compliance with Cal/OSHA requirements. Excavations near existing structures (within a 1:1 projection) should be provided with shoring that is designed to support the surcharge load of the existing structure. Otherwise, excavations may be performed in sections (A/B/C slot cuts). The conditions should be reviewed in the field by the project geotechnical consultant. Additional recommendations will be provided based on the actual conditions encountered during excavation and grading, as needed.

3.2.4 Fill Placement

Upon the completion of remedial grading measures, the approved removal/overexcavation bottoms should be scarified a minimum of 6 inches. The removal bottoms and fill materials should be compacted to at least 90 percent of maximum dry density, as determined by ASTM Test Method D1557. Fill materials should be placed in loose lifts no thicker than 8 inches.

Fill should be relatively free of deleterious material. The onsite artificial fill and bedrock materials are considered suitable for re-use as compacted fill provided any deleterious material is removed.

As discussed previously, the onsite artificial fill and bedrock materials are diatomaceous, generally requiring more compaction and support equipment effort. Appropriate equipment should be used and other measures (e.g., mixing, stockpiling, drying) may be needed to achieve the uniform and correct moisture content for fill placement. If the soils become

extremely wet, special measures for mixing and drying may be required that will need to be determined based on the actual field conditions.

Considering the presence of diatomaceous soils at the site, it is important that the geotechnical consultant perform several moisture/density tests (during grading) at lower than typical temperatures in order to establish appropriate maximum density test curves for the materials and correlate the results with the nuclear gauge readings in the field. The moisture content determination should be performed at oven temperatures as low as 90 to 100 degrees Fahrenheit in order to keep the structure of the diatoms intact. In the field, drying under sun to determine the moisture around the soil particles, rather than within the diatoms, should also be explored. Correlations should then be made between the moisture contents determined by slow drying of the materials and nuclear gauge readings in order to better estimate the relative density and optimum moisture content in the field and using a nuclear gauge. Ultimately, placement and compaction of diatomaceous materials is more of an observation and performance evaluation than testing. It is essential that fill be placed and compacted at above optimum moisture content determined in the lab via slow drying process and confirmed in the field. Compaction at higher moisture content (but within compactable range) and placement of fill materials in thinner lifts (6 inches) will result in better performance of the fill materials (i.e. lower compressibility, less expansion). The geotechnical consultant should evaluate the fill placement and compaction effort in the field, and provide additional recommendations, as needed.

Concrete material may be mixed with onsite soils and placed as compacted fill provided it is broken into pieces that are smaller than 6 inches in the largest diameter and in accordance with the below oversize criteria. Asphalt concrete may also be used in compacted fill materials; however, it should not be placed under the residential pads. Placement of concrete and asphalt as compacted fill should be approved by the project environmental consultant.

Oversize material may be placed in fills deeper than 10 feet below design lot/pad grade or below any planned utilities within streets. Rocks larger than 6 inches and smaller than 12 inches in the largest dimension may be placed at depth between 5 and 10 feet below design finish grades. Rocks greater than 3 inches in dimension are typically not suitable for backfill of utility trench excavations (unless accepted by the agency/utility owner). Thus, we recommend that the grading contractor consider placement of rocks larger than 3 inches in dimension outside of proposed utility alignments.

3.2.5 Import

The geotechnical consultant should evaluate and accept any import soils prior to transportation to the subject site. We recommend that import soils have similar or better engineering properties as onsite soils. At minimum, the import materials should have an Expansion Index of less than 90 and a Plasticity Index of less than 30.

3.3 Settlement Potential

The amount of settlement will depend upon the type of foundation(s) selected and future loading by additional fill and structures. Based on the subsurface exploration and laboratory data, our review and analysis, considering the remedial grading recommendations provided in this report are

implemented during grading, and the anticipated structural loads typically associated with the proposed structures (2- to 4-story wood-framed residential units), we estimate that total and differential settlement (combined static and seismic) will be less than 1-inch and ½-inch over a span of 40 feet, respectively.

SA GEO should be provided with the foundation plans and structural loads, once available, to further evaluate the potential for post-construction settlement of the proposed buildings and associated improvements. The parameters provided herein will then be confirmed/updated based on the planned foundation layout and loads and any additional testing/analysis.

3.4 Foundation Design

The slab and foundations should be designed by the project structural engineer based on the proposed structure type and the anticipated loading conditions. The foundation soils are anticipated to have expansive soil conditions ("Medium" to "High" expansion potential) and will be subject to climatic and landscape moisture fluctuations. Post-tensioned slab should be anticipated for the proposed residential buildings. The following foundation recommendations are provided with the assumption that the recommendations included in Section 3.2 of this report are implemented during site grading.

The recommended net allowable bearing capacity for continuous and isolated footings may be calculated based on the following equation:

$q_{all} = 500\ D + 150\ B + 1{,}250$ (but not to exceed 3,500 psf, see below for post-tensioned/mat slabs)

where:
D = embedment depth of footing, in feet
B = width of footing, in feet

Also, the following parameters may be used for design of foundation and slabs:

- Soil unit weight = 100 pcf
- Soil internal friction angle = 27 degrees
- Coefficient of Friction = 0.33
- Subgrade modulus (k) of 50 pci (corrected for large slabs)
- Soil elastic modulus (Es) of 1,000 psi
- Effective Plasticity Index (upper 15 feet) = 50

The allowable bearing capacity of 1,000 psf may be used for design of post-tensioned/mat foundation.

The dead load of concrete below adjacent grades (buried concrete foundations) may be neglected. The allowable bearing pressure and friction coefficient may be increased by one-third for wind and seismic loading.

We recommend that strip and isolated footings for the buildings have a minimum embedment depth of 24 inches below the lowest adjacent grade. Continuous footings should be at least 12 inches wide and isolated column footings should be at least 24 inches wide. The footings of

freestanding and isolated structures, such as walls and pilasters, should have a minimum embedment depth of 24 inches into approved soils.

The following table provides our general guidelines and preliminary recommendations for design of post-tensioned foundations and slabs on expansive soil in accordance with the 2025 California Building Code (CBC) and Post-Tension Institute (PTI) DC 10.5 Edition provisions.

GEOTECHNICAL GUIDELINES
FOR DESIGN OF POST-TENSIONED SLABS

Parameter	Recommendation
Center Lift	
Edge Moisture Variation Distance, e_m	7.50 feet
Center Lift, y_m	1.00 inches
Edge Lift	
Edge Moisture Variation Distance, e_m	4.00 feet
Edge Lift, y_m	1.50 inch
Presaturation, as needed, to obtain the minimum moisture down to the minimum depth	1.3 x optimum down to 18 inches

We recommend that post-tensioned slabs have a thickened edge such that the slab is embedded a minimum of 18 inches below the lowest adjacent grade.

In addition, as indicated in the DC 10.5 Edition of PTI, shape factor calculations should be performed by the project structural engineer in order to determine if strengthening/modification of foundations are necessary. Per PTI guidelines, modifications to the foundations design should be considered if the shape factor (ratio of square of foundation perimeter over foundation area) exceeds 24.

If non-post-tensioned slabs-on-grade and foundations are considered at the site, an effective Plasticity Index of 50 is considered appropriate for the upper 15 feet of soil materials, in accordance with Wire Reinforcement Institute (WRI) method (per the 2025 California Building Code). For non-post-tensioned slabs, we recommend a minimum embedment of 18 inches below the lowest adjacent grade for the perimeter footings. Also, the upper 18 inches of subgrade soil should be pre-saturated to 130 percent of optimum moisture content prior to placement of moisture barrier and concrete.

The foundations and slabs should also be designed to tolerate the total and differential settlements discussed in Section 3.3 of this report.

For the design of pole-type foundations (i.e., light poles, shade structures, etc.), an allowable soil-bearing pressure (s_1) of 300 psf/ft may be used for Equation 18-1 (the "pole" equation) of the 2025 California Building Code (CBC) Section 1807.3.2.1 to determine the depth of embedment for the footings, considering level ground conditions. The equation is applicable for designed embedment depths of less than 12 feet for the purpose of computing lateral pressure. Also, for vertical loads on pole-type foundations, an allowable skin friction of 250 pounds per square foot may be used.

Pole foundations should have a minimum embedment of 30 inches below adjacent grades. For cast-in-place pole-type foundations, the vertical end bearing pressure should be neglected.

3.5 Retaining Wall Design and Lateral Earth Pressures

Recommendations for lateral earth pressures for permanent retaining walls and structures with approved onsite drained soils and above groundwater table (drained condition) are as follows:

Conditions	*Level (pcf)*	*2:1 Sloping*
Active	40	60
At-Rest	65	85
Passive	300	170 (sloping down)

The soil parameters provided above are based on a soil internal friction angle of 27 degrees, cohesion of 50 pounds per square foot (psf), and soil unit weight of 100 pcf.

To design an unrestrained retaining wall, such as a cantilever wall, the active earth pressure may be used. For a restrained retaining wall, the at-rest pressure should be used. Passive pressure is used to compute lateral soils resistance developed against lateral structural movement. The passive pressures provided above may be increased by one-third for wind and seismic loads. The passive resistance is taken into account only if it is ensured that the soil against embedded structure will remain intact with time. Future landscaping/planting and improvements adjacent to the retaining walls should also be taken into account in the design of the retaining walls. Excessive soil disturbance, trenches (excavation and backfill), future landscaping adjacent to footings and over-saturation can adversely impact retaining structures and result in reduced lateral resistance.

For sliding resistance, the friction coefficient of 0.33 may be used at the concrete and soil interface. The coefficient of friction may be increased by one-third for wind and seismic loading. The retaining walls may also need to be designed for additional lateral loads if other structures or walls are planned within a 1H:1V projection.

If slopes above the retaining walls are relatively minor (in height), the additional load associated with the soil above the retaining wall and within the active wedge may be calculated as a surcharge load and applied to the retaining wall using a coefficient of active pressure (k_a) of 0.33.

The seismic lateral earth pressure for walls retaining more than 6 feet of soil may be estimated to be an additional 18 pcf for active and at-rest conditions. The earthquake soil pressure has a triangular distribution and is added to the static pressures. For the active and at-rest conditions, the additional earthquake loading is zero at the top and maximum at the base. The seismic lateral earth pressure does not apply to walls retaining less than, or equal to, 6 feet of soil (2025 CBC Section 1803.5.12).

Drainage behind retaining walls (retaining more than 30 inches of soil) should also be provided in accordance with the attached Figure 4. Specific drainage connections, outlets and avoiding open joints should be considered for the retaining wall design.

3.6 Seismic Design Parameters

The following tables summarize the seismic design criteria for the subject site. The seismic design parameters are developed in accordance with ASCE 7-22 and 2025 California Building Code (CBC). Seismicity data including spectral design accelerations for two-period and multi-period structures is attached (Appendix D).

Selected Seismic Design Parameters from 2025 CBC/ASCE 7-22	***Seismic Design Values***	***Reference***
Latitude	33.58495 North	
Longitude	-117.72402 West	
Controlling Seismic Source	San Joaquin Hills	USGS, 2026
Site Class per Table 20.2-1 of ASCE 7-22	D	
Spectral Acceleration for Short Periods (Ss)	1.4 g	SEA/OSHPD, 2026
Spectral Accelerations for 1-Second Periods (S_1)	0.44 g	SEA/OSHPD, 2026
Design Spectral Response Acceleration at Short Periods (S_{DS}) from Equation 11.4-1 of ASCE 7-22	1.1 g	SEA/OSHPD, 2026
Design Spectral Response Acceleration at 1-Second Period (S_{D1}) from Equation 11.4-2 of ASCE 7-22	0.73 g	SEA/OSHPD, 2026
T_S, S_{D1} /S_{DS} per Section 11.4.5 of ASCE 7-22	0.668 sec	SEA/OSHPD, 2026
T_L, Long-Period Transition Period	8 sec	SEA/OSHPD, 2026
Peak Ground Acceleration, the Geometric-Mean Maximum Considered Earthquake (MCE_G) peak ground acceleration for the Site Class, (PGA_M)	0.62 g	SEA/OSHPD, 2026
Seismic Design Category Section 11.6 of ASCE 7-22	D	SEA/OSHPD, 2026

3.7 Slope Stability and Slope Setback

We understand that no significant modifications to the existing slopes are planned as part of this project. The existing slopes are generally composed entirely of artificial fill or artificial fill placed over bedrock (i.e., stabilization fill keyway). The proposed improvements are not anticipated to adversely impact the slope. Based on our slope stability analysis (Appendix E), the existing slope was found to be grossly stable. In general, slopes are considered susceptible to surficial instability in the form of erosion, raveling, and slumping. Special care should be taken to ensure that onsite runoff is controlled and directed offsite in a non-erosive manner and that site drainage, and slopes are maintained over the life of the project.

Footings for structures located above descending slopes should be set back from the slope face in accordance with the minimum requirements of the City of Aliso Viejo and section 1808.7.1 of the 2025 CBC, whichever is greater. The setback distance is measured from the outside edge of the footing bottom along a horizontal line to the face of the slope.

The tables below summarize the minimum setback criteria for structures above descending slopes:

Case A – Building and Retaining Wall Footings Above Descending Slopes	
Slope Height [H] (feet)	*Minimum Setback from Slope face (feet)*
Less than 10	5
10 to 20	½ * H
20 to 30	10
Greater than 30	⅓ * H (maximum of 40')

Case B – Freestanding Wall Footings Above Descending Slopes	
Slope Height [H] (feet)	*Minimum Setback from Slope face (feet)*
Less than 10	5
10 to 20	½ * H
Greater than 20	Maximum of 10

For freestanding walls and other structures that are sensitive to lateral movement (e.g., smooth stucco finish, glass screens, etc.), we recommend that the structural setback requirements in accordance with Case A above be followed or that additional design measures be used to help control the potential for cracking and displacements. Otherwise, typical freestanding walls may have a setback in accordance with Case B.

3.8 Corrosivity

Soluble sulfates exposure in the onsite soils should be classified as "S2" per Table 19.3.1.1 of ACI-318-14. Structural concrete elements in contact with soil include footings and building slabs-on-grade. The flatwork and sidewalk concrete are typically not considered structural elements. Concrete mix for structural elements should be based on the "S2" soluble sulfate exposure class of Table 19.3.2.1 in ACI-318-14. Other ACI guidelines for structural concrete are recommended. Also, onsite soils are anticipated to be corrosive to metals.

3.9 Expansion Potential

At the completion of grading, we anticipate that onsite soils will have "Medium" to "High" expansion potential. The geotechnical recommendations provided in this report, including the design parameters for foundations, slab-on-grade and flatwork improvement should be implemented during design and construction. Updated recommendations will be provided upon additional testing at the completion of grading at the site and as needed.

Homeowners and their design/construction team should be familiar with the recommendations in this report as well as principles described in a useful reference published by the California Geotechnical Engineers Association (CalGeo), titled, "Coexisting with Expansive Soil: An Informational Guide for Homeowners." This free booklet can be downloaded at www.calgeo.org.

3.10 Interior Slab Moisture Mitigation

In addition to geotechnical and structural considerations, the project owner should also consider interior moisture mitigation when designing and constructing slabs-on-grade.

The intended use of the interior space, type of flooring, and type of goods in contact with the floor may dictate the need for, and design of, measures to mitigate potential effects of moisture emission from and/or moisture vapor transmission through the slab. Typically, for human occupied structures, a vapor retarder or barrier is recommended under the slab to help mitigate moisture transmission through slabs. The most recent guidelines by the American Concrete Institute (ACI 302.1R-04) suggest that the vapor retarder be placed directly under the slab (no sand layer). However, the location of the vapor retarder may also be subject to the builder's past successful practice. Placement of 1 or 2 inches of sand over the moisture retardant has been common practice by builders in southern California. Specifying the strength of the retarder to resist puncture and its permeance rating is important. These qualities are not necessarily a function of the retarder thickness. A minimum of 10-mil is typical but some materials, such as 10-mil polyethylene ("Visqueen"), may not meet the desired standards for toughness and permeance.

Vapor retarders, when used, should be installed in accordance with standards such as ASTM E 1643 and/or those specified by the manufacturer.

Concrete mix design and curing are also significant factors in mitigating slab moisture problems. Concrete with lower water/cement ratios results in denser, less permeable slabs that also "dry" faster with regard to when flooring can be installed (reduced moisture emissions quantities and rates). Rewetting of the slab following curing should be avoided since it can result in additional drying time required prior to flooring installation. Proper concrete slab testing prior to flooring installation is also important.

Concrete mix design, the type and location of the vapor retarder should be determined in coordination with all parties involved in the finished product, including the project owner, architect, structural engineer, geotechnical consultant, concrete subcontractors, and flooring subcontractors.

3.11 Exterior Concrete

The driveway, patio slabs, sidewalk and other flatwork elements should be at least 4 inches thick. Driveway approaches should be a minimum of 6 inches thick. We recommend that the concrete be reinforced with No. 3 rebars placed at 24 inches on center both ways. Concrete slabs should be provided with construction or weakened plane control joints at a maximum spacing of 6 feet. The control joints should have a thickness that is ¼ of the total concrete thickness. Upon the placement and compaction of (per Section 3.2 of these recommendations), the upper 18 inches should be pre-saturated to 130 percent of optimum moisture content prior to placement of reinforcement and concrete. We also recommend that 4 inches of granular, non-expansive materials (sand, aggregate base) be placed over the compacted subgrade prior to placement of reinforcement and concrete.

For exterior slabs, the use of a granular sublayer is primarily intended to facilitate presaturation and subsequent construction by providing a better working surface over the saturated soil. It also helps retain the added moisture in the native soil in the event that the slab is not placed immediately.

Exterior concrete elements such as curb and gutter, driveways, sidewalks and patios are susceptible to lifting and cracking when constructed over expansive soils. With expansive soils, the impacts to flatwork/hardscape can be significant, generally requiring removal and replacement of the affected improvements. Please also note that reducing concrete problems is often a function of proper slab design, concrete mix design, placement, and curing/finishing practices. Adherence to guidelines of the American Concrete Institute (ACI) is recommended. Also, the amount of post-construction watering, or lack thereof, can have a very significant impact on the adjacent concrete flatwork.

On projects with expansive soils, additional measures such as thickened concrete edges/footings, subdrains and/or moisture barriers should be considered where planter or natural areas with irrigation are located adjacent to the concrete improvements. Design and maintenance of proper surface drainage is also very important. If the concrete will be subject to heavy loading from cars/trucks or other heavy objects, thicker pavement section will be required the design of which should be performed by the geotechnical consultant, as needed.

3.12 Preliminary Asphalt Concrete Pavement Design

Final structural pavement sections should be based on R-value testing after the completion of grading and utility construction, and in accordance with City of Aliso Viejo requirements. Using an assumed R-value of 5 and estimated traffic indices (TIs), we recommend the following preliminary pavement sections:

Street Location	***Estimated TIs***	***Pavement Section***
Parking Stalls	TI – 4.0	4 inches of AC / 6 inches of AB
General Drives	TI – 5.5	4 inches of AC / 10 inches of AB
AC = Asphalt Concrete, AB = Aggregate Base		

Please note that for two-stage paving operations, we recommend that the final AC cap be a minimum of 0.10 foot thick and the base AC course have a minimum thickness of 0.25 foot.

If pavement repair is needed within the adjacent existing streets, the section should match the existing section and follow standard City guidelines for trench/pavement repair. The new pavement section should at minimum consist of 4 inches of AC over 10 inches of AB unless approved otherwise by the geotechnical consultant and based on evaluation of subgrade soils.

Asphalt concrete pavement should be placed in accordance with the requirements of Sections 301 and 302 of the Standard Specifications of Public Works Construction (the Greenbook). Prior to construction of pavement sections, the subgrade soils should be scarified to a minimum depth of 6

inches, moisture-conditioned as needed, and recompacted in-place to a minimum of 90 percent relative compaction (per ASTM D1557). Subgrade should be firm prior to AB placement.

AB materials can be crushed aggregate base or crushed miscellaneous base in accordance with the Greenbook (Section 200-2). The materials should be free of any deleterious materials. Aggregate base materials should be placed in 6- to 8-inch-thick loose lifts, moisture-conditioned as necessary, and compacted to a minimum of 95 percent relative compaction (per ASTM D1557). Asphalt concrete should also be compacted to a minimum relative compaction of 95 percent.

Unpaved median and parkway areas should be provided with vertical moisture barriers.

3.13 Vehicular PCC Pavements

For trash enclosures, we recommend 6 inches of PCC reinforced with No. 4 rebar at 18 inches on-center, both ways, over 4 inches of aggregate base placed over compacted subgrade. For parking stalls, the section may consist of 4 inches of PCC reinforced with No. 4 rebar at 18 inches on-center, both ways, over 4 inches of aggregate base over compacted subgrade.

The subgrade soils should be scarified to a minimum depth of 6 inches, moisture-conditioned as needed, and recompacted in place to a minimum of 90 percent relative compaction (per ASTM D1557).

Aggregate base materials can be crushed aggregate base or crushed miscellaneous base in accordance with the Greenbook (Section 200-2). The materials should be free of any deleterious materials. Aggregate base materials should be placed in 6- to 8-inch-thick loose lifts, moisture-conditioned as necessary, and compacted to a minimum of 95 percent relative compaction (per ASTM D1557).

3.14 Concrete Paver Section

For concrete pavers within **pedestrian** walkways, we recommend 60-millimeter-thick concrete pavers over approximately 1 inch of sand, over a minimum of 6 inches of AB, over compacted subgrade. This should also accommodate the rare maintenance cart or light truck that might drive up on these areas.

For concrete pavers within **vehicular** drive areas, if any, we recommend 80-millimeter-thick concrete pavers over 6 inches of concrete reinforced with No. 3 rebar at 24 inches on-center, over 4 inches of AB. Alternatively, the concrete low pour slab may be replaced with a minimum of 15 inches of AB over compacted subgrade; or 12 inches of AB over a layer of geotextile (e.g., Mirafi 600X or equivalent), over compacted subgrade.

A concrete band along the perimeter of the pavers should be provided for lateral restraint. The concrete band should, at minimum, be 12 inches deep and deepened to below the depth of disturbed soil, if adjacent to landscape/planter areas.

Also, a drainage system consisting of a 2-inch PVC drainage pipe should be installed at the low end of the concrete low pour at 10-foot intervals through the slab and filled with pea gravel. The PVC

drainage pipe should drain to a minimum of 6-inch-diameter by 12-inch-deep drain pit filled with pea gravel below the pipe. The paver surfaces should slope away from structures a minimum of 2 percent. The concrete underlayment for the drive areas should be provided with weakened plane joints at maximum 6-foot intervals. If the pavers are to be set in mortar or are underlain with AB, this drainage system is not required.

Prior to construction of the paver sections, the subgrade soils should be scarified to a minimum depth of 6 inches, moisture conditioned as needed, and recompacted in place to a minimum of 90 percent relative compaction (per ASTM D1557). The upper 18 inches of subgrade for pedestrian walkway and patio areas should then be presaturated to 130 percent of optimum moisture content. Subgrade for vehicular pavers need not be pre-saturated.

Aggregate base materials can be crushed aggregate base or crushed miscellaneous base in accordance with the Greenbook (Section 200-2). The materials should be free of deleterious materials. Aggregate base materials should be placed in 6- to 8-inch-thick loose lifts, moisture-conditioned as necessary, and compacted to a minimum of 95 percent relative compaction (per ASTM D1557).

3.15 Trench Excavation and Backfill

Excavations should be performed in accordance with the requirements set forth by Cal/OSHA Excavation Safety Regulations (Construction Safety Orders, Section 1504, 1539 through 1547, Title 8, California Code of Regulations). In general, onsite soils may be classified as Type "B" soils for excavations into compacted fill and Type "C" for excavations into bedrock with potentially adverse bedding. Cal/OSHA regulations indicate that, for workmen in confined conditions, the steepest allowable slopes in Type "B" and "C" soils is 1:1 and 1.5H:1V (horizontal to vertical), respectively, for excavations less than 20 feet deep. Where there is no room for these layback slopes, we anticipate that shoring will be necessary. These conditions should also be anticipated for excavations within the streets adjacent to the site. Adequate shoring/shield should be provided, as deemed necessary. Excavations should be reviewed periodically by the contractor's qualified person to confirm compliance with Cal/OSHA requirements.

Onsite soils should be suitable for use as trench backfill. Utility trench backfill should be in accordance with City of Aliso Viejo and/or the governing jurisdiction's specifications. Backfill materials should be compacted to a minimum of 90 percent relative compaction (per ASTM D1557). Rocks greater than 3 inches in largest diameter should generally not be used as trench backfill unless approved by the agency and geotechnical consultant of record. Excavation and backfilling of HDPE pipes (if any) should be in accordance with the manufacturer’s requirement and the Greenbook. Select granular backfill (i.e., clean sand with SE 30 or better) may be used in lieu of native soils but should also be compacted or densified with water jetting and flooding.

Trenches excavated next to structures and foundations should also be properly backfilled and compacted to provide full lateral support and reduce settlement potential.

3.16 Pool and Spa Design

Typical design parameters and recommendations for construction of pools, spas, and decks are provided on Figures 5 and 6 at the rear of text. The pool/spa walls should be designed for an equivalent lateral fluid pressure of 125 psf/ft. For design, the subgrade soils for the pool and spa decks should be considered as having "High" expansion potential. The depth of the cutoff wall adjacent to the pool/spa deck should be a minimum of 24-inches below the top of the deck. If the thickness of the sand layer is increased, the depth of the cut-off wall should also be increased to a minimum of 6-inches below the bottom of the sand layer. The potential settlement associated with construction of pool/spa is anticipated to be minor and within the tolerance of those structures provided that the recommendations included in this report are implemented during design, grading and construction. However, considering the expansive nature of the onsite soils and in order to minimize the differential movement, over-excavation of the subgrade soils below the pool/spa to a depth of 1-foot and replacement with aggregate base should be considered.

If an infinity (weir) edge pool/spa are to be constructed, in order to reduce the post-construction differential movement, we recommend that the subgrade for these structures be over-excavated a minimum of two feet and replaced with two feet of crushed miscellaneous base (CMB) in accordance with the Standard Specifications for Public Works Construction (the Greenbook). CMB should be compacted to a minimum relative compaction of 95 percent per ASTM D1557. Additional recommendations may need to be provided once the plans are available and as needed.

3.17 Groundwater

In general, groundwater is anticipated to remain more than 30 feet below design grades upon the completion of grading. However, shallow, perched groundwater/seepage may occur on an annual and seasonal basis as a result of rainfall, irrigation and/or seepage from adjacent properties, and along earth unit contacts (i.e., artificial fill and bedrock). Groundwater is generally not expected to be encountered during grading or utility construction; however, the presence of locally saturated soils and/or perched water cannot be ruled out, especially during rainy seasons.

3.18 Stormwater Infiltration Feasibility

Considering that the site is underlain by fine-grained artificial fill material overlying expansive, fine-grained bedrock, and the presence of a relatively large perimeter slopes along the eastern and southern property boundaries, infiltration of stormwater is not considered feasible at the site from a geotechnical viewpoint and may contribute to unforeseen geotechnical hazards. Other methods of stormwater treatment, such as detention and biofiltration basins, should be evaluated by the project civil engineer.

3.19 Top of Slope Improvements - Slope Creep and Lateral Fill Extension

Hillside lots are subject to soil phenomena referred to as slope creep and lateral fill extension (LFE). In the past, these have also been called "lot stretching" due to the associated horizontal component of earth movement.

These phenomena are expected to some degree or another with all graded and natural slopes. The exact mechanisms of these natural processes are not entirely understood, but most geotechnical professionals agree that LFE is associated with expansive soil and an increase in the moisture content of the surficial soil over time. As the surficial material gets wetter and the soil expands, the slope area moves outward with surface displacements that can range from fractions of an inch to several inches. At some point, LFE is thought to reach an equilibrium point and movements either cease or greatly diminish. If rear yard improvements are not designed and built with this in mind, tilting, cracks, separations, and other distress may occur.

Slope creep is the result of the pull of gravity on a slope and the tendency of the soil near the top of slope and on the slope face to move downhill very slowly. Creep occurs as the soil expands and contracts with variations in soil moisture and as surficial soils deteriorate from plant growth and/or burrowing activity. The magnitude of slope creep is generally thought to depend on factors such as slope height, steepness, soil type, and the degree of moisture variation. Slope creep may continue indefinitely. The movement of the side yard fencing and tilting of improvements such as pools/spas that are close to the top of slope may be due to slope creep, as well as LFE.

While it is generally not practical or economical to eliminate the effects of LFE and slope creep, many measures can be incorporated into the design and construction of rear yard improvements to mitigate the effects of these phenomena. Some measures include:

- Setting improvements back from the edge (top) of slope,
- Deepening foundations,
- Reinforcing concrete,
- Including "soft" landscape zones in hardscape areas where soil movements will be buffered and have less impact,
- Tying improvements together to resist movements,
- Not tying improvements together (de-coupling) to allow for soil movements without causing damage to the improvements (e.g., expansion joints, flexible connections, slip dowels).
- Using adjustable types of improvements such as concrete pavers and wood fencing,
- Incorporating adjustable hardware for gates hinges, latches, etc.

3.20 Slope Maintenance, Surface Drainage and Irrigation

Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture-related problems and differential movements from soil heave/settlement. Maintenance of proper pad drainage, the design and construction of property improvements in accordance with sound engineering practices, and proper maintenance of landscaping, including regular pad and slope irrigation, should be performed. Irrigation should not be excessive or sporadic; maintaining a relatively constant level of soil moisture around structures and improvements is important. Oversteepening of slopes should be avoided.

Surface drainage should be carefully taken into consideration during grading, landscaping, and building construction. Positive surface drainage should be provided to direct surface water away from structures and slopes and toward the street or suitable drainage devices. Ponding of water adjacent to the structures should not be allowed. Buildings should have roof gutter systems and

the run-off should be directed to parking lot/street gutters by area drainpipes or by sheet flow over paved areas. Paved areas should be provided with adequate drainage devices, gradients, and curbing to prevent run-off flowing from paved areas onto adjacent unpaved areas.

Considering the climatic conditions in southern California and provided that the recommendations included in this report are implemented during grading and construction, a minimum two-percent slope away from structures is considered acceptable and in substantial compliance with the 2025 CBC. Also, swales with one-percent slopes are acceptable from a geotechnical standpoint and are common practice in this locale.

Construction of planter areas immediately adjacent to structures should be avoided if possible. If planter boxes are constructed adjacent to or near buildings, the planters should be provided with controls to prevent excessive penetration of the irrigation water into the foundation and flatwork subgrades. Provisions should be made to drain excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Raised planter boxes may be drained with weepholes. Deep planters (such as palm tree planters) should be drained with below-ground, water-tight drainage lines connected to a suitable outlet. Moisture barriers should also be considered.

It is also important to maintain a consistent level of soil moisture, not allowing the subgrade soils to become overly dry or overly wet. Properly designed landscaping and irrigation systems can help in that regard.

Establishing and maintaining a rodent-control program is recommended to reduce surficial slope deterioration often associated with rodent burrowing. The improvements at or near the tops of slopes may be subject to varying degrees of slope-creep related movements and should be designed with that in mind (also see Section 3.19).

3.21 Additional Subsurface Exploration and Laboratory Testing

Additional subsurface exploration during the design phase of the project is not anticipated provided no significant plan changes occur. Additional laboratory testing should be performed during and upon completion of the grading to confirm/update the design parameters provided herein.

3.22 Review of Future Plans

The project grading, foundation, street improvement, wall, and landscape plans should be reviewed and accepted by the geotechnical consultant prior to grading and construction. Additional recommendations should be provided upon the review of the project plans and as needed.

3.23 Observation and Testing during Grading and Construction

Geotechnical observation and testing should be performed by SA GEO during the following phases of grading and construction:

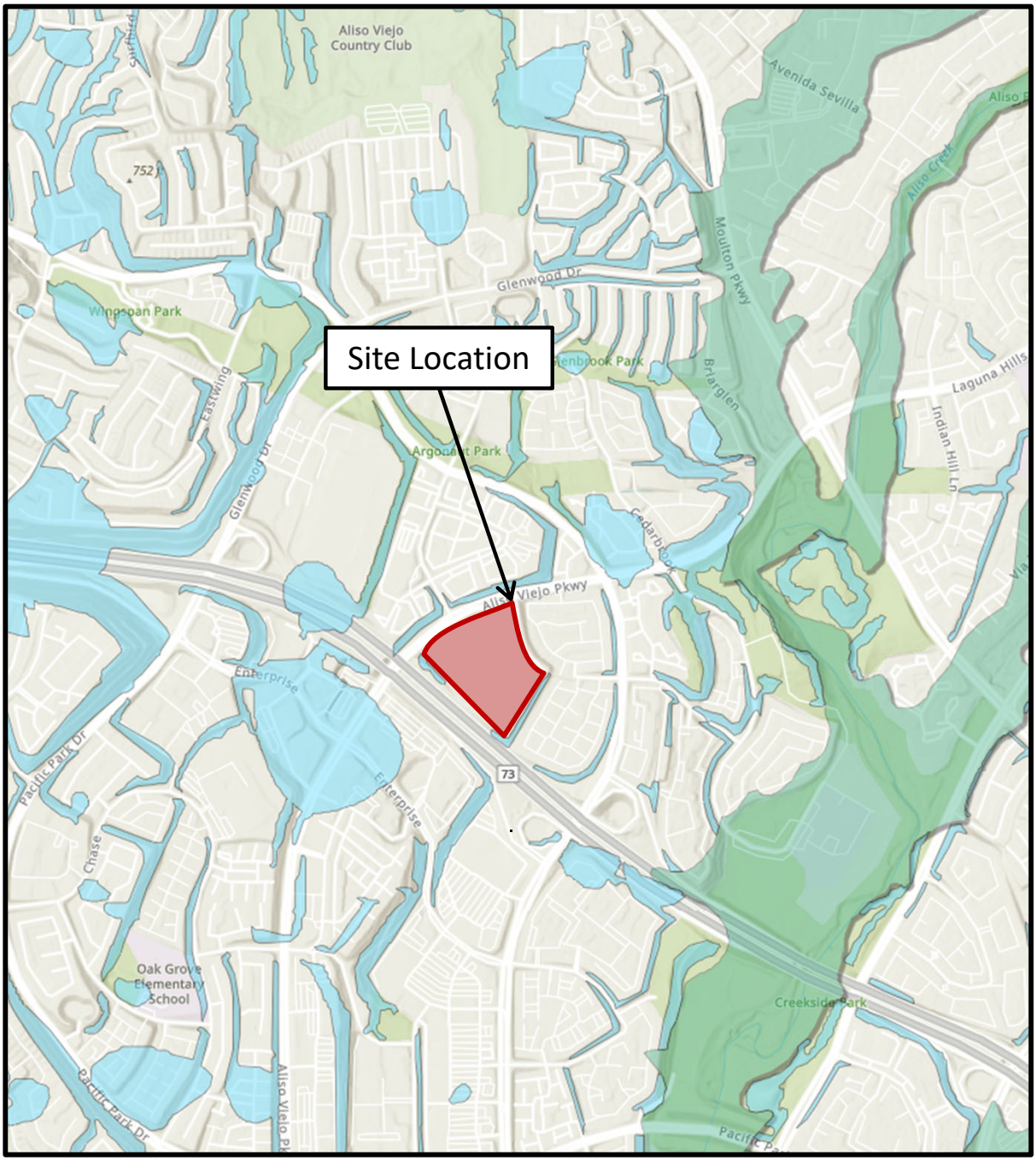
- During site demolition, preparation and clearing;

- During excavations performed for remedial grading and to relocate or remove existing underground improvements and structures;
- During earthwork, including observation and acceptance of remedial removal/overexcavation bottoms and fill placement, including import material (if any);
- Following the completion of grading, in order to verify soil properties for foundations, slab-on-grade and pavement areas;
- Upon completion of any foundation or structural excavation, prior to pouring concrete;
- During slab and flatwork subgrade preparation, including pre-saturation, prior to pouring of concrete;
- During placement of backfill for utility trenches, storm drain, and stormwater filtration devices;
- During placement of backfill for retaining structures, if any;
- During installation and backfill of subdrainage systems, if any; and,
- When any unusual soil conditions are encountered.

4.0 LIMITATIONS

This report has been prepared for the exclusive use of our client, Lennar, within the scope of services requested for the subject property described herein. This report or its contents should not be used or relied upon for other projects or purposes, or by other parties without the acknowledgement of SA GEO and the consultation of a geotechnical professional. The means and methods used by SA GEO for this study are based on local geotechnical standards of practice, care, and requirements of governing agencies. No warranty or guarantee, expressed or implied, is given.

Our findings, conclusions, and recommendations are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in-between exploration points, and can also change over time. Our conclusions and recommendations are, by nature, preliminary and subject to verification and/or modification during grading and construction when more subsurface data is exposed.



Source: Seismic Hazard Zones, San Juan Capistrano Quadrangle (CDMG, 2001b)

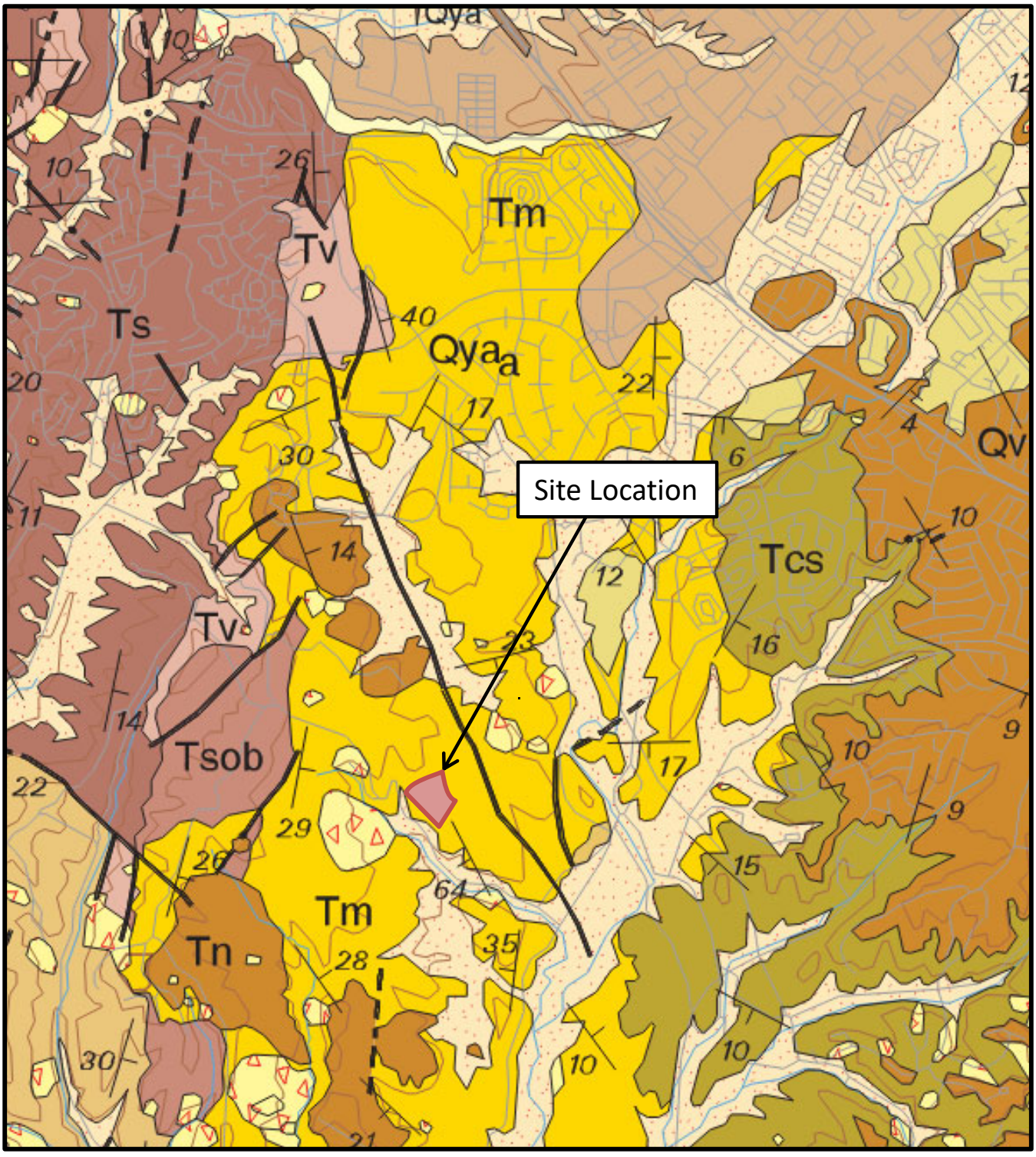
Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resource Code Section 2693(c) would be required.

Earthquake-Induced Landslides
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 Resource Code Section 2693(c) would be required.

Site Location and Seismic Hazard Map

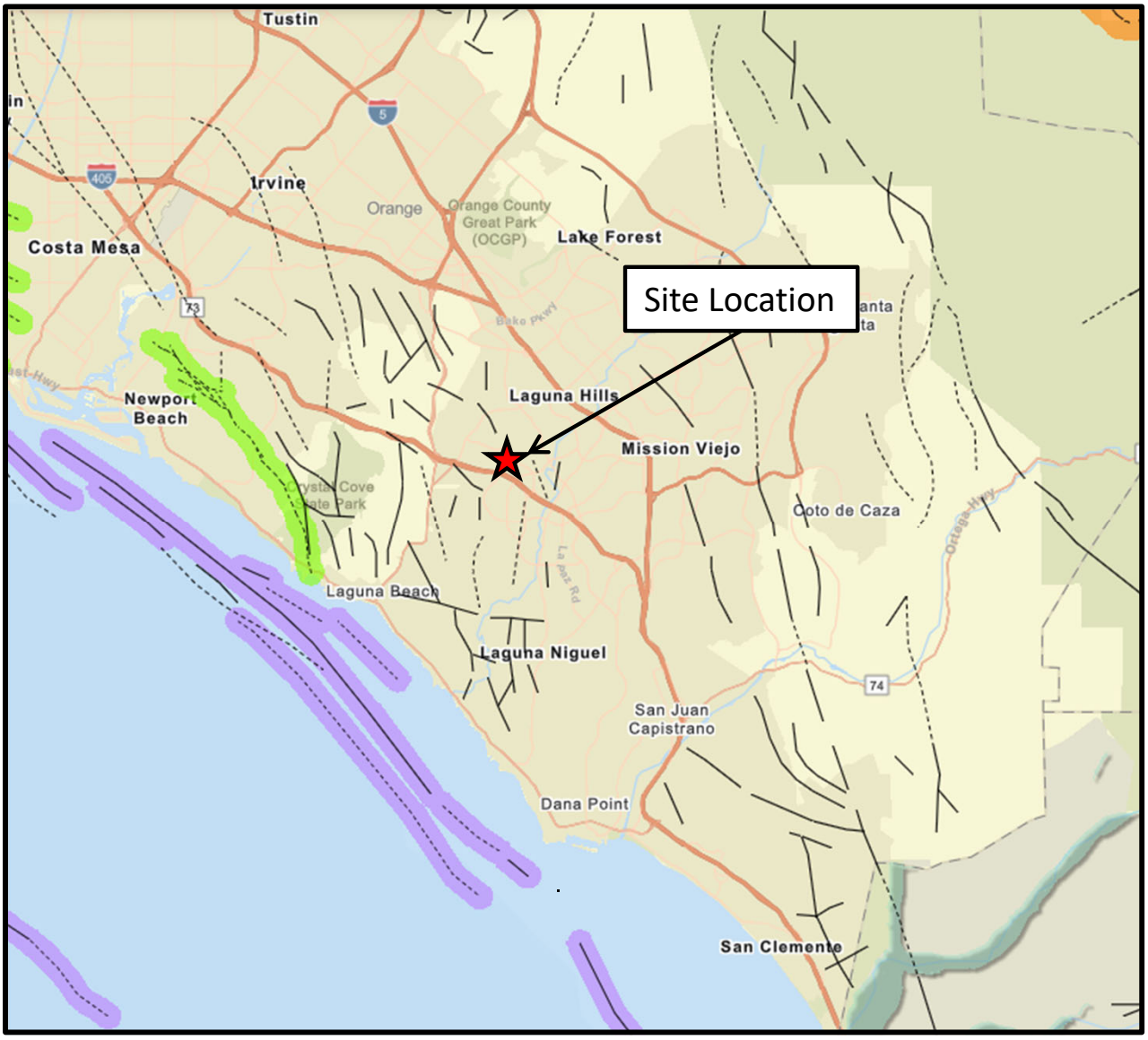
Lennar
Proposed Residential Development
1 through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01
Date: March 4, 2026
Figure 1



Source: Geologic Map of the San Bernardino and Santa Ana 30' X 60' Quadrangles (USGS, 2006)

Regional Geologic Map		
Lennar Proposed Residential Development 1 through 5 Polaris Way Aliso Viejo, California	Project Number: 25093-01 Date: March 4, 2026 Figure 2	S/A



Source: Fault Activity Map of California (Jennings and Bryant, 2010)

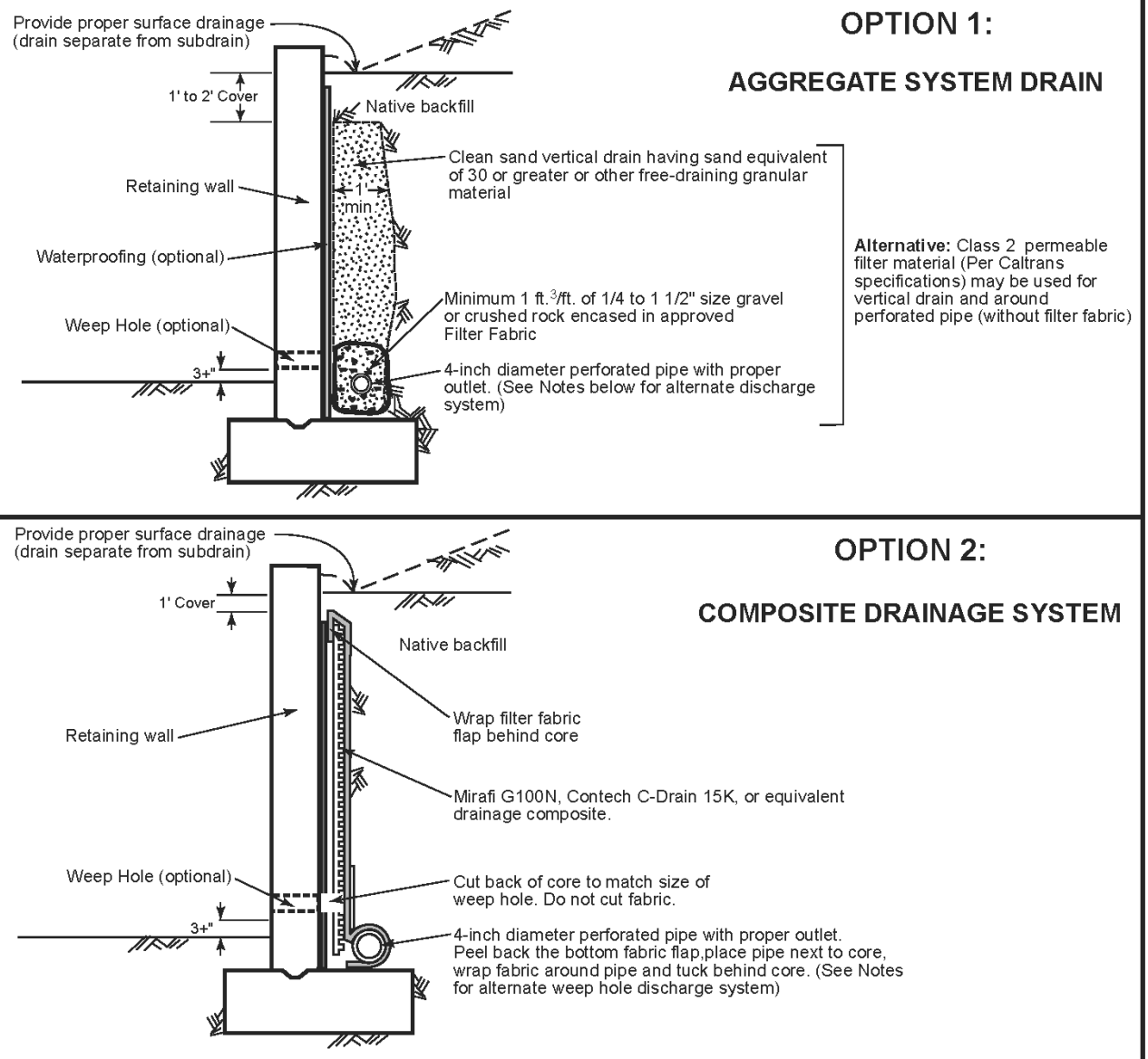
Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

Regional Fault Map	
Lennar Proposed Residential Development 1 through 5 Polaris Way Aliso Viejo, California	Project Number: 25093-01 Date: March 4, 2026 Figure 3



NOTES:

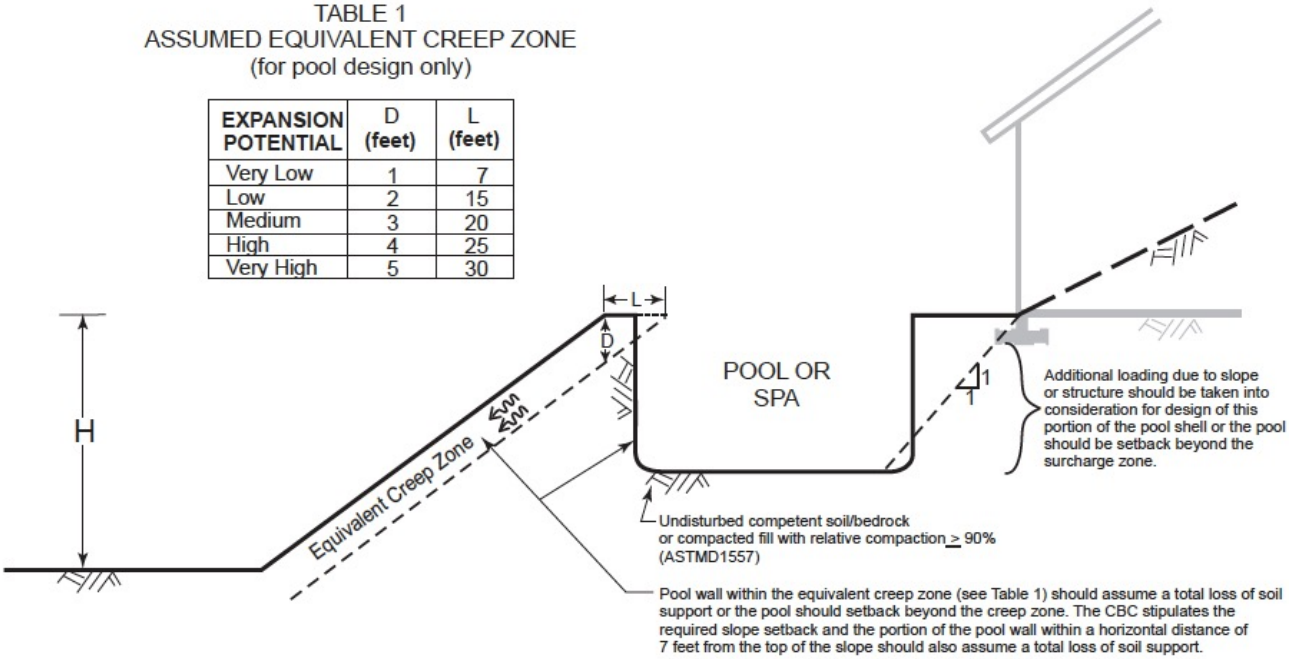
1. PIPE TYPE SHOULD BE PVC OR ABS, SCHEDULE 40 OR SDR35 SATISFYING THE REQUIREMENTS OF ASTM TEST STANDARD D1527, D1785, D2751 , OR D3034.
2. FILTER FABRIC SHALL BE APPROVED PERMEABLE NON-WOVEN POLYESTER, NYLON, OR POLYPROPYLENE MATERIAL.
3. DRAIN PIPE SHOULD HAVE A GRADIENT OF 1 PERCENT MINIMUM.
4. WATERPROOFING MEMBRANE MAY BE REQUIRED FOR A SPECIFIC RETAINING WALL (SUCH AS A STUCCO OR BASEMENT WALL).
5. WEEP HOLES MAY BE PROVIDED FOR LOW RETAINING WALLS (LESS THAN 3 FEET IN HEIGHT) IN LIEU OF A VERTICAL DRAIN AND PIPE AND WHERE POTENTIAL WATER FROM BEHIND THE RETAINING WALL WILL NOT CREATE A NUISANCE WATER CONDITION. IF EXPOSURE IS NOT PERMITTED, A PROPER SUBDRAIN OUTLET SYSTEM SHOULD BE PROVIDED.
6. IF EXPOSURE IS PERMITTED, WEEP HOLES SHOULD BE 2-INCH MINIMUM DIAMETER AND PROVIDED AT 25-FOOT MAXIMUM SPACING ALONG WALL. WEEP HOLES SHOULD BE LOCATED 3+ INCHES ABOVE FINISHED GRADE.
7. SCREENING SUCH AS WITH A FILTER FABRIC SHOULD BE PROVIDED FOR WEEP HOLES/OPEN JOINTS TO PREVENT EARTH MATERIALS FROM ENTERING THE HOLES/JOINTS.
8. OPEN VERTICAL MASONRY JOINTS (I.E., OMIT MORTAR FROM JOINTS OF FIRST COURSE ABOVE FINISHED GRADE) AT 32-INCH MAXIMUM INTERVALS MAY BE SUBSTITUTED FOR WEEP HOLES.
9. THE GEOTECHNICAL CONSULTANT MAY PROVIDE ADDITIONAL RECOMMENDATIONS FOR RETAINING WALLS DESIGNED FOR SELECT SAND BACKFILL.

Retaining Wall Drainage Detail

Figure 4

TABLE 1
ASSUMED EQUIVALENT CREEP ZONE
(for pool design only)

EXPANSION POTENTIAL	D (feet)	L (feet)
Very Low	1	7
Low	2	15
Medium	3	20
High	4	25
Very High	5	30



Note: This Detail Is Typical And Subject To Change By the Geotechnical Consultant. Construction Should be in Accordance With Section 1808.7.3 of the 2022 CBC

1) The pool walls should be designed to account for soil expansion forces. Where constructed in soils possessing "medium" expansion potential, the pool walls should be designed for an equivalent lateral fluid pressure of 100 lbs./cu. ft. For soils possessing "high" or "very high" expansion potential, the pool walls should be designed for and equivalent lateral fluid pressure of 125 lbs./cu. ft. The actual expansiveness of soils exposed in pool excavations should be evaluated upon completion of the excavation as pool subgrade soils are exposed.

2) Pools and spas should conform to setback criteria pertaining to slopes as established in the California Building Code. The CBC indicates that the pool should be setback from descending slopes a minimum distance of one-sixth of the descending slope height (20 feet maximum). In addition, that portion of the pool wall within a horizontal distance of 7 feet from the top of the slope shall be capable of supporting the water in the pool without soil support. In addition, we recommend that the portion of the pool wall within the equivalent soil creep zone (Table 1) should also assume a total loss of soil support. Where pools are planned near toes-of-slopes and/or structures, appropriate surcharge loads should be incorporated into design and construction.

3) Pool/spa excavations exposing bedrock should be evaluated by a qualified geotechnical consultant to determine the need for special design to account for bedding plane surcharges. If encountered, the pool walls should be designed to support any daylight bedding. The bedding plane surcharge value will vary depending upon bedding angle, rock type, and strength.

4) In order to provide uniform conditions, the pool excavation may need to be overexcavated a depth of 3 feet below bottom of pool and replaced to pool subgrade with compacted fill. As an alternative, the reinforcing steel in the area of a transition area may be increased to account for the differences in engineering properties and the potential differential behavior.

5) Consideration should be given to incorporating provisions for mitigating subsurface water. Whereas pool excavations may be free of water at the time of construction, future irrigation could result in the development of perched water zones which could affect subsurface improvements. Heavy-duty pipes and couplings should be used for the pool plumbing system to minimize leaking which may produce additional pressures on the pool shell. In addition, installation of a pressure valve in the pool bottom may be considered to mitigate potential build-up of pressure.

6) In general, all below grade improvements must be constructed by qualified professionals utilizing appropriate designs which account for the on-site (lot) geotechnical and geologic conditions. Observation/testing should be performed by a geotechnical consultant during pool/spa excavation to verify exposed soil conditions are consistent with the assumed design conditions.

Swimming Pool Detail

Figure 5

NOTE: RECOMMENDATIONS DEPICTED ON THIS DETAIL ARE TYPICAL AND SUBJECT TO CHANGE BY THE GEOTECHNICAL CONSULTANT OR ARCHITECT/DESIGNER

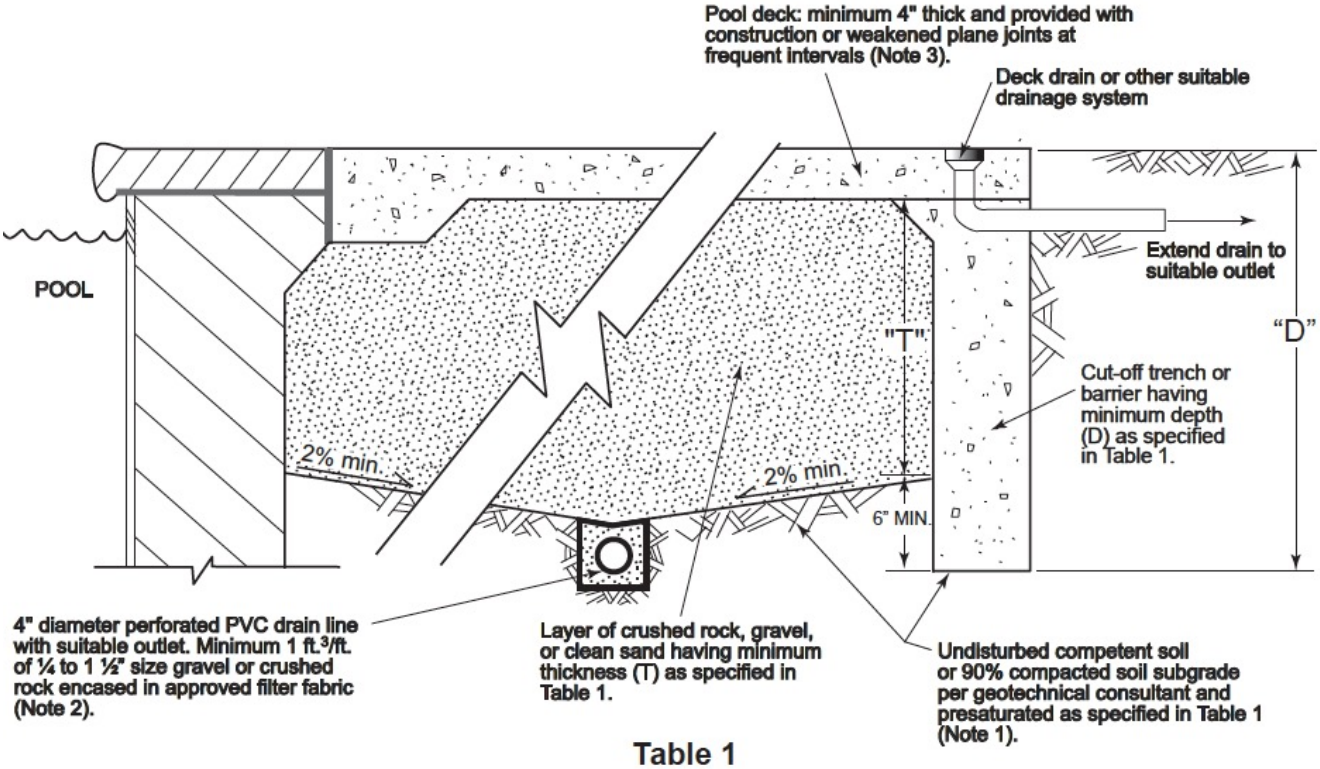


Table 1

Expansion Potential	Min. Depth of Cut-off "D"	Min. Thickness of Sand Back-fill "T"	Min. Depth of Presaturation	Min. Presaturation Moisture Content
Low	12"	4"	6"	120% of Optimum
Medium	18"	6"	12"	130% of Optimum
High	24"	12"	18"	140% of Optimum
Very High	30"	18"	24"	140% of Optimum

Notes:

1) To reduce the potential for excessive cracking due to expansive soil forces, pool deck concrete slabs should be a minimum of 4 inches thick and provided with construction or weakened plane joints at frequent intervals (e.g., every 6 feet or less). Slabs should be underlain by a layer of crushed rock, gravel, or clean sand having a minimum thickness as indicated in Table 1. This layer is not required for very low expansion potential subgrades. For very low expansion potential subgrade, water spraying the subgrade prior to pouring concrete is considered adequate otherwise, the subgrade should be presaturated to the minimum depth and minimum moisture content (as a percentage of optimum moisture content) indicated in Table 1. Presoaking should be observed, tested, and accepted by a geotechnical consultant prior to placement of concrete.

2) The Subgrade below pool decks should have a drain line consisting of 4-inch diameter perforated pipe (PVC Schedule 40, SDR 35, Armco A2000 PVC, or approved equivalent), surrounded by approved gravel which is wrapped with filter fabric (Mirafi 140N, or approved equivalent) provided below the sand layer. One line of subdrain around the swimming pool area is generally sufficient. The drain pipe should have a gradient of 1 percent minimum.

3) All concrete has a tendency to crack and cracks in concrete can be caused by many different factors. When constructing concrete decks, it is important that the ground on which these improvements are to rest be properly prepared, including moisture conditioning. Slab thickness, location of joints, reinforcement, and concrete mixture must also be appropriate for the intended use. Proper placement, finishing, and curing of concrete are also very important factors in minimizing cracking. Reinforcement of slabs may also be considered to further reduce unsightly cracking especially for high or very high expansion potential areas. Adjacent landscaping should provide appropriate surface and subdrainage to minimize overwetting the soils.

Swimming Pool Deck Detail

Figure 6

Appendix A

APPENDIX A

REFERENCES

California Division of Mines and Geology (CDMG), 2001a, Seismic Hazard Zone Report for the San Juan Capistrano 7.5-Minute Quadrangle, Orange County, California, Seismic Hazard Zone Report 053.

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APPENDIX A (Cont'd)

REFERENCES

U.S. Geological Survey, 2006, Geologic Map of the San Bernardino and Santa Ana 30' X 60' Quadrangles, California, dated 2006, Open File Report 2006-1217.

U.S. Geological Survey, 2026, Unified Hazard Tool, NSHM 2014 Dynamic Deaggregation Program; web site address: https://earthquake.usgs.gov/hazards/interactive/; Date Accessed: February 20, 2026.

Appendix B

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*GRAVEL AND GRAVELLY SOILS* MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	*CLEAN GRAVELS* (LITTLE OR NO FINES)	**GW**	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	**GM**	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	*CLEAN SANDS* (LITTLE OR NO FINES)	**SW**	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	**SM**	SILTY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*SILTS AND CLAYS*	LIQUID LIMIT LESS THAN 50	**ML**	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			**PT**	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Sampler and Symbol Descriptions

- Modified California sample (D-#)
- Standard Penetration Test (S-#)
- Shelby tube sample (T-#)
- Large bulk sample (B-#)
- Small bulk sample (SB-#)
- Approximate depth of groundwater during drilling
- Approximate depth of static groundwater

Note: Number of blows required to advance driven sample 12 inches (or length noted).

Laboratory and Field Test Abbreviations

AL	Atterberg limits (plasticity)
CC	Chemical Testing incl. Soluble Sulfate
CN	Consolidation
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Analysis (Sieve, Hydro. and/or -No. 200)
MD	Maximum Density and Optimum Moisture
RV	Resistance Value (R-Value)
SE	Sand Equivalent
UU	Unconsolidated Undrained Shear Strength

Notes:

1. Soil classifications are based on the Unified Soil Classification System and include color, moisture, and relative density or consistency. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate. Bedrock descriptions are based on visual classification and include rock type, moisture, color, grain size, strength, and weathering.
2. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were drilled. They are not warranted to be representative of subsurface conditions at other locations or times.

S/A

KEY TO LOG OF BORING

Date(s)Drilled: 9-10-2025	**Logged By:** LZ/PA	**H-1**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5”	Page 1 of 1
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30”)	
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 11.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 445
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 3” AC over 6" AB.			
				Monterey Formation (Tm)			
	B-1		MH	@ 1-2.5': Olive brown to grayish brown clayey SILTSTONE, moist, laminated, diatomaceous.			B-1 @ 1-5’
	D-1	17	MH	@ 2.5’: Olive brown to pale gray clayey SILTSTONE, stiff, moist, laminated, diatomaceous.	59.3	64.0	
5	D-2	18	MH	@ 5’: Olive brown to pale gray clayey SILTSTONE, stiff, moist, laminated, diatomaceous.	54.4	65.7	
	D-3	29	MH	@ 7.5’: Pale brown to white clayey SILTSTONE, very stiff, damp to moist, laminated, diatomaceous.	52.1	62.8	
10	D-4	29	MH	@ 10’: Pale brown to white clayey SILTSTONE, very stiff, damp to moist, laminated, diatomaceous.	49.7	68.7	
15				Notes: Total Depth: 11.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
20							
25							
30							

Log Of Boring

Lennar
Proposed Residential Development
1 Through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Date(s)Drilled: 9-11-2025	**Logged By:** LZ	**H-2**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	Page 1 of 1
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 11.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 442
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 3" AC over 8" AB.			
			SM	**Artificial Fill (Af)** @ 1-2.5': Pale brown to dark brown silty fine SAND, moist.			
	D-1	17	MH	**Monterey Formation (Tm)** @ 2.5': Pale gray to white clayey SILTSTONE, stiff, damp to moist, laminated, highly diatomaceous, trace FeO staining.	115.4	38.8	GS, AL
5	D-2	35	MH	@ 5': Pale gray to white clayey SILTSTONE, very stiff, damp, laminated, highly diatomaceous, trace FeO staining.	112.6	39.2	
	D-3	23	MH	@ 7.5': Pale gray to white clayey SILTSTONE, very stiff, damp, highly diatomaceous, laminated, abundant dendritic MnO staining.	142.5	34.7	
10	D-4	26	MH	@ 10': Pale brown to pale gray clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO staining and microfossils.	69.2	68.4	
15				Notes: Total Depth: 11.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
20							
25							
30							

Log Of Boring

Lennar
Proposed Residential Development
1 Through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Date(s)Drilled: 9-10-2025	**Logged By:** LZ	**H-3**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	Page 1 of 1
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 16.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 439
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 5" AC over 7" AB.			
				Artificial Fill (Af)			
	B-1		CH	@ 1-2.5': Olive brown to dark brown silty CLAY, moist to very moist, trace fine to coarse gravel.			B-1 @ 1-5' GS, AL, EI, CC
	D-1	31	CH	@ 2.5': Mottled pale brown, olive brown, reddish brown, silty CLAY, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	50.4	69.1	
5	D-2	27	MH	@ 5': Pale brown to brown clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	45.9	75.4	
	D-3	18	MH	@ 7.5': Pale brown to brown clayey SILT, stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	41.1	72.7	
10	D-4	30	MH	**Monterey Formation (Tm)** @ 10': Olive brown to pale gray clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO staining.	33.9	72.8	
	D-5	17	MH	@ 12.5': Olive brown to pale gray clayey SILTSTONE, stiff, damp, laminated, diatomaceous, trace FeO staining.	72.3	57.1	
15	D-6	33	MH	@ 15': Pale gray to white clayey SILTSTONE, very stiff, damp to moist, laminated, diatomaceous, trace FeO staining and dendritic MnO staining.	62.6	58.1	
20				Notes: Total Depth: 16.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
25							
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Appendix B

Date(s)Drilled: 9-12-2025	**Logged By:** LZ	**H-4**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	Page 1 of 1
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 16.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 435
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
				Surface: 6" AC over 9" AB.			
0			MH	**Artificial Fill (Af)** @ 1-2.5': Pale brown to dark brown clayey SILT, moist, abundant bedrock fragments.			
	B-1						B-1 @ 1-5' GS, AL, MD, EI, CC
	D-1	28	MH	@ 2.5': Mottled pale brown, pale gray, gray brown clayey SILT, very stiff, damp, diatomaceous, abundant ½- to 1-inch bedrock fragments.	33.3	71.9	
5	D-2	48	MH	@ 5': Mottled pale brown, pale gray, grayish brown clayey SILT, hard, damp, diatomaceous, abundant ½- to 1-inch bedrock fragments.	36.0	66.4	
	D-3	44	MH	@ 7.5': Pale gray to white clayey SILT, damp, very stiff, diatomaceous, abundant ½- to 1-inch bedrock fragments.	32.6	52.5	
10	D-4	41	MH	**Monterey Formation (Tm)** @ 10': Pale gray to white clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, some caliche stringers, trace FeO staining.	37.1	64.1	
	D-5	35	MH	@ 12.5': Pale gray to white clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, some caliche stringers, trace FeO staining.	39.8	60.0	
15	D-6	52	MH	@ 15': White clayey SILTSTONE, hard, damp, diatomaceous, laminated, some FeO staining.	33.2	64.2	
20				Notes: Total Depth: 16.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
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Date(s)Drilled: 9-11-2025	**Logged By:** LZ	**H-5**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	Page 1 of 1
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 16.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 437
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 3" AC over 6" AB.			
	B-1		SM	**Artificial Fill (Af)** @ 1-2.5': Pale brown to reddish brown clayey fine SAND, moist, 6 to 8-inch rounded cobbles, some wires/debris.			B-1 @ 1-5'
	D-1	19	MH	@ 2.5': Mottled pale brown and reddish brown clayey SILT, stiff, moist, diatomaceous, trace organics.	48.9	64.7	
5	D-2	16	MH	**Monterey Formation (Tm)** @ 5': Pale brown to pale gray clayey SILTSTONE, stiff, damp to moist, laminated, diatomaceous.	47.4	69.4	
	D-3	24	MH	@ 7.5': Pale brown to pale gray clayey SILTSTONE, very stiff, moist, laminated, diatomaceous.	59.4	54.7	
10	D-4	30	MH	@ 10': Pale gray to white clayey SILTSTONE, very stiff, moist, diatomaceous, laminated, some FeO staining.	53.8	58.1	
	D-5	29	MH	@ 12.5': Pale gray to white clayey SILTSTONE, very stiff, moist, diatomaceous, laminated, some FeO staining, trace microfossils.	59.7	56.3	
15	D-6	24	MH	@ 15': Pale gray to white clayey SILTSTONE, very stiff, damp to moist, diatomaceous, laminated, abundant caliche stringers, some FeO staining, microfossils.	45.7	67.4	
20				Notes: Total Depth: 16.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
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Date(s)Drilled: 9-12-2025	**Logged By:** LZ	**H-6**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	Page 1 of 1
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 26.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 429.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
				Surface: 4.5" AC over 8" AB.			
0	B-1		SC	**Artificial Fill (Af)** @ 1-2.5': Brown clayey fine SAND, moist, abundant fine to coarse gravel.			B-1 @ 1-5'
	D-1	18	MH	@ 2.5': Mottled pale brown to brown and grayish brown clayey SILT, stiff, very moist, diatomaceous, some bedrock fragments and fine to coarse gravel.	44.4	70.3	
5	D-2	18	MH	@ 5': Mottled pale brown to brown and grayish brown clayey SILT, stiff, diatomaceous, very moist, some bedrock fragments and fine to coarse gravel.	64.4	62.7	
	D-3	18	MH	@ 7.5': Mottled pale gray and brown clayey SILT, stiff, very moist, diatomaceous, some bedrock fragments and fine to coarse gravel.	57.3	63.8	
10	D-4	19	MH	@ 10': Mottled pale gray and brown clayey SILT, stiff, very moist, diatomaceous, some bedrock fragments and gravel.	53.5	67.1	
	D-5	24	MH	@ 12.5': Mottled brown, pale brown, gray, and yellowish brown clayey SILT, very stiff, wet, diatomaceous, some fine to coarse gravel, abundant ½- to 1-inch bedrock fragments.	56.4	67.5	
15	D-6	29	MH	**Monterey Formation (Tm)** @ 15': Pale gray to white clayey SILTSTONE, very stiff, moist, laminated, diatomaceous, some FeO staining, some fossils.	102.4	47.4	
20	D-7	15	MH	@ 20': Pale gray to white clayey SILTSTONE, stiff, moist, laminated, diatomaceous, some caliche stringers, trace clay beds ~3mm thick, trace FeO staining.	70.1	53.4	
25	D-8	30	MH/CH	@ 25': Pale brown to pale gray SILTSTONE/CLAYSTONE, very stiff, moist, laminated, diatomaceous, interbedded, dip ~20°.	43.6	69.8	
				Notes: Total Depth: 26.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
30							

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Date(s)Drilled: 9-11-2025	**Logged By:** LZ	**H-7**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 66.5
Approximate Groundwater Depth: Groundwater at 51.5 feet.		**Approximate Ground Surface Elevation (ft)** 425.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
					Surface: 6" AC over 10" AB.			
0				SM	**Artificial Fill (Af)** @ 1-2.5': Pale brown to brown silty SAND, damp, slightly diatomaceous, trace bedrock fragments.			
	D	D-1	34	SM	@ 2.5': Mottled pale brown to yellowish brown silty SAND, medium dense, moist, slightly diatomaceous, trace bedrock fragments.	11.9	91.6	
5	D	D-2	36	MH	@ 5': Mottled pale brown to yellowish brown clayey SILT, very stiff, moist, diatomaceous, trace bedrock fragments.	40.2	74.7	
	D	D-3	25	MH	@ 7.5': Mottled pale brown to yellowish brown clayey SILT, very stiff, moist, diatomaceous, trace bedrock fragments.	46.7	66.6	
10	D	D-4	31	MH	@ 10': Mottled pale brown to reddish brown, clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	57.9	59.5	DS
	D	D-5	29	MH	@ 12.5': Mottled pale brown to reddish brown clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	54.8	60.1	
15	D	D-6	34	MH	@ 15': Mottled pale gray to white clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	56.5	61.5	
	D	D-7	33	MH	@ 17.5': Mottled pale gray to white clayey SILT, very stiff, damp to moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	46.5	65.6	
20	D	D-8	40	MH	@ 20': Mottled pale gray to white clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	56.6	58.4	
	D	D-9	37	MH	@ 22.5': Mottled pale gray to white clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	50.5	60.1	
25	D	D-10	37	MH	@ 25': Mottled pale gray to white clayey SILT, very stiff, damp to moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	49.1	60.0	
	D	D-11	39	MH	@ 27.5': Dark brown to pale gray to white clayey SILT, very stiff, damp, diatomaceous, some ½-inch bedrock fragments.	31.3	77.1	
30								

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Date(s)Drilled: 9-11-2025	**Logged By:** LZ	**H-7**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 66.5
Approximate Groundwater Depth: Groundwater at 51.5 feet.		**Approximate Ground Surface Elevation (ft)** 425.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
30	D-12	58	MH	@ 30': Mottled pale gray, white, and brown clayey SILT, hard, damp to moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	30.4	78.4	
	D-13	70	CH	@ 32.5': Mottled brown to dark brown silty CLAY, very moist, hard, diatomaceous, some bedrock fragments.	30.3	85.3	
35	D-14	43	CH	@ 35': Mottled brown to dark brown silty CLAY, very moist, very stiff, slightly diatomaceous, some bedrock fragments.	29.1	84.5	
	D-15	58	CH	@ 37.5': Mottled brown to dark brown silty CLAY, very moist, hard, slightly diatomaceous, some bedrock fragments.	28.2	80.9	
40	D-16	40	CH	@ 40': Mottled brown to dark brown silty CLAY, wet, very stiff, diatomaceous, some bedrock fragments.	36.0	79.6	
	D-17	42	CH	@ 42.5': Mottled brown to dark brown silty CLAY, wet, very stiff, diatomaceous, some bedrock fragments.	31.4	82.0	
45	D-18	14	CH	@ 45': Mottled pale brown and yellowish brown silty CLAY, stiff, wet, diatomaceous, trace bedrock fragments.	46.2	70.1	
	D-19	8	MH	**Monterey Formation (Tm)** @ 47.5': Pale brown to yellowish brown clayey SILTSTONE, medium stiff, wet, diatomaceous, highly weathered, massive, some pores and caliche stringers.	56.7	64.8	
50	D-20	7	MH ▽	@ 50': Pale brown to yellowish brown clayey SILTSTONE, medium stiff, saturated, diatomaceous, highly weathered, massive, some pores and caliche stringers.	56.9	65.3	Groundwater at 51.5 feet.
	D-21	9	MH	@ 52.5': Pale brown to yellowish brown clayey SILTSTONE, medium stiff, saturated, diatomaceous, highly weathered, massive, some pores and caliche, trace FeO staining.	66.8	60.0	
55	D-22	20	MH	@ 55': Pale brown to yellowish brown clayey SILTSTONE, stiff, saturated, diatomaceous, highly weathered, massive, abundant FeO staining, some pores and caliche stringers.	83.3	51.4	
	D-23	28	MH	@ 57.5': Pale brown to reddish brown clayey SILTSTONE, very stiff, saturated, poorly bedded, diatomaceous, some FeO staining.	92.1	46.6	
60							

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Date(s)Drilled: 9-11-2025	**Logged By:** LZ	**H-7**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 66.5
Approximate Groundwater Depth: Groundwater at 51.5 feet.		**Approximate Ground Surface Elevation (ft)** 425.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
60	D-24	44	MH	@ 60': Pale gray to white clayey SILTSTONE, very stiff, saturated, laminated, diatomaceous, trace FeO staining.	107.0	42.3	
	D-25	17	MH	@ 62.5': Olive brown to brown clayey SILTSTONE, stiff, saturated, laminated, diatomaceous.	66.0	60.8	
65	D-26	29	MH	@ 65': Pale brown to gray clayey SILTSTONE, very stiff, saturated, laminated, diatomaceous, trace FeO staining.	107.9	45.1	
70				Notes: Total Depth: 66.5 Feet. Groundwater at 51.5 Feet. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
75							
80							
85							
90							

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Date(s)Drilled: 9-10-2025	**Logged By:** LZ/PA	**H-8**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	Page 1 of 2
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 36.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 433.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 4" AC over 7" AB.			
			SM	**Artificial Fill (Af)** @ 1-2.5': Brown to dark brown silty fine SAND, damp, some construction debris.			
	D-1	31	SM	@ 2.5': Mottled pale brown to reddish brown silty fine SAND, medium dense, moist, trace coarse gravel.	14.2	108.2	
5	D-2	27	SM	@ 5': Pale brown to pale gray silty SAND, medium dense, moist, some bedrock fragments.	15.9	91.8	
	D-3	40	MH	@ 7.5': Mottled pale brown, dark brown, and yellowish brown clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments, trace coarse angular gravel.	41.8	77.0	
10	D-4	29	MH	@ 10': Mottled pale brown, dark brown, and yellowish brown clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments, trace coarse angular gravel.	33.4	88.4	
	D-5	21	MH	@ 12.5': Mottled pale brown, dark brown, and yellowish brown clayey SILT, stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments, trace coarse angular gravel.	61.4	60.2	
15	D-6	24	MH	@ 15': Mottled pale brown, dark brown, and white clayey SILT, very stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	68.4	58.6	
20	D-7	17	MH	@ 20': Mottled pale brown, dark brown, white clayey SILT, stiff, very moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	66.6	57.8	
25	D-8	22	MH	**Monterey Formation (Tm)** @ 25': Yellow brown to white clayey SILTSTONE, stiff, damp, laminated, diatomaceous, some dendritic MnO staining.	61.6	51.6	
30							

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Date(s)Drilled: 9-10-2025	**Logged By:** LZ/PA	**H-8**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	Page 2 of 2
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 36.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 433.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
30	D-9	27	MH/CH	@ 30': Pale brown clayey SILTSTONE/CLAYSTONE, very stiff, moist, laminated, diatomaceous, interbedded.	65.5	57.2	
35	D-10	51	MH/CH	@ 35': Pale brown clayey SILTSTONE/CLAYSTONE, hard, moist, laminated, diatomaceous, interbedded.	106.0	43.1	
40				Notes: Total Depth: 36.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
45							
50							
55							
60							

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Date(s)Drilled: 9-10-2025	**Logged By:** LZ
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5”
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30”)
Sampling Method(s): Modified California	
Approximate Groundwater Depth: Groundwater not encountered.	
Comments: Hand auger upper 2.5' for utility clearance.	

H-9

Total Depth Drilled (ft)	11.5
Approximate Ground Surface Elevation (ft)	439

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
				Surface: 3” AC over 9" AB.			
0			CL	**Artificial Fill (Af)** @ 1-2.5': Pale brown to brown silty CLAY, damp, abundant bedrock fragments.			
	D-1	42	MH	**Monterey Formation (Tm)** @ 2.5’: Olive brown to pale gray clayey SILTSTONE, very stiff, moist, laminated, diatomaceous, trace FeO staining.	70.0	50.7	
5	D-2	47	MH	@ 5’: Pale brown to white clayey SILTSTONE, hard, damp to moist, laminated, diatomaceous, trace FeO staining.	80.4	45.5	
	D-3	50/ 6"	MH	@ 7.5’: Pale brown to white clayey SILTSTONE, hard, damp to moist, laminated, diatomaceous, trace FeO staining and calcite filled fractures.	92.7	30.5	
10	D-4	73	MH	@ 10’: White clayey SILTSTONE, hard, damp to moist, laminated, diatomaceous, FeO stained microfossils.	88.0	35.1	
15				Notes: Total Depth: 11.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
20							
25							
30							

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Date(s)Drilled: 9-10-2025	**Logged By:** LZ	**H-10**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	Page 1 of 1
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 16.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 434.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 3" AC over 6" AB.			
				Artificial Fill (Af)			
	B-1		MH	@ 1-2.5': Brown fine sandy/clayey SILT, moist.			
	D-1	33	MH	@ 2.5': Mottled pale brown to brown clayey SILT, very stiff, moist, diatomaceous, abundant ½-inch bedrock fragments.	68.6	57.5	B-1 @ 1-5' GS, AL, MD, EI, CC
5	D-2	31	MH	@ 5': Mottled pale brown to brown clayey SILT, very stiff, moist, diatomaceous, abundant ½-inch bedrock fragments.	50.1	65.6	AL, DS
	D-3	24	MH	@ 7.5': Mottled pale brown to brown clayey SILT, very stiff, moist, diatomaceous, abundant ½-inch bedrock fragments, trace FeO staining.	55.5	64.8	CN
10	D-4	29	MH	@ 10': Pale brown to pale gray clayey SILT, very stiff, moist, diatomaceous, abundant ½-inch bedrock fragments.	64.7	59.6	
	D-5	80	MH	@ 12.5': UPPER: Dark brown clayey SILT, hard, very moist, diatomaceous, abundant ½-inch bedrock fragments.	29.5	83.3	
			MH	**Monterey Formation (Tm)**			
15	D-6	69		LOWER: White clayey SILTSTONE, hard, damp to moist, laminated, diatomaceous.	52.5	48.9	
			MH	@ 15': White clayey SILTSTONE, hard, damp to moist, laminated, diatomaceous, trace FeO staining.			
20				Notes: Total Depth: 16.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
25							
30							

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Date(s)Drilled: 9-10-2025	**Logged By:** LZ	**H-11**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	Page 1 of 1
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 16.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 430.5
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 3" AC over 6" AB.			
				Artificial Fill (Af)			
	B-1		SM	@ 1-2.5': Pale brown to dark brown silty fine SAND, damp to moist, trace fine to coarse gravel, trace bedrock fragments.			B-1 @ 1-5'
	D-1	36	SM	@ 2.5': Mottled pale brown to brown silty fine SAND, medium dense, moist, some bedrock fragments.	13.3	111.9	
5	D-2	35	MH	**Monterey Formation (Tm)** @ 5': Pale brown to pale gray clayey SILTSTONE, very stiff, damp, laminated, vertical dipping beds, diatomaceous.	59.2	56.9	
	D-3	37	MH	@ 7.5': Pale brown to pale gray clayey SILTSTONE, very stiff, damp, laminated, subhorizontal beds, diatomaceous.	81.8	45.8	
10	D-4	28	MH	@ 10': White clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO and MnO staining.	63.0	49.0	DS
	D-5	50	MH	@ 12.5': White clayey SILTSTONE, hard, damp, laminated, diatomaceous, trace FeO and MnO staining.	56.0	61.3	
15	D-6	39	MH	@ 15': White clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO and MnO staining.	48.9	57.7	
20				Notes: Total Depth: 16.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
25							
30							

Log Of Boring

Lennar
Proposed Residential Development
1 Through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Date(s)Drilled: 9-10-2025	**Logged By:** LZ	**H-12**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	Page 1 of 1
Sampling Method(s): Modified California		**Total Depth Drilled (ft)** 16.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 432
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0				Surface: 3" AC over 6" AB.			
				Artificial Fill (Af)			
			MH	@ 1-2.5': Mottled dark brown to pale brown clayey SILT, damp, some bedrock fragments.			
	D-1	43	MH	@ 2.5': Mottled dark brown to pale brown clayey SILT, very stiff, damp, diatomaceous, abundant ½- to 1-inch bedrock fragments.	44.5	60.1	
5	D-2	41	MH	@ 5': Mottled dark brown to pale brown clayey SILT, very stiff, damp, diatomaceous, abundant ½- to 1-inch bedrock fragments.	39.6	74.7	
	D-3	32	MH	@ 7.5': Mottled dark brown to pale brown clayey SILT, very stiff, damp, diatomaceous, abundant ½- to 1-inch bedrock fragments.	52.0	64.6	
10	D-4	35	MH	**Monterey Formation (Tm)** @ 10': Pale brown to pale gray clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO staining.	49.5	59.0	
	D-5	36	MH	@ 12.5': Pale brown and olive brown to gray clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO staining.	75.8	50.5	
15	D-6	22	MH	@ 15': Pale brown and olive brown to gray clayey SILTSTONE, very stiff, damp, laminated, diatomaceous, trace FeO staining.	52.1	64.8	
20				Notes: Total Depth: 16.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
25							
30							

Log Of Boring

Lennar
Proposed Residential Development
1 Through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Date(s)Drilled: 9-12-2025	**Logged By:** LZ	**H-13**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	Page 1 of 2
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 41.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 428
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
				Surface: 3" AC over 8" AB.			
0			MH	**Artificial Fill (Af)** @ 1-2.5': Mottled pale brown and reddish brown clayey SILT, moist.			
	D-1	21	MH	@ 2.5': Mottled pale brown, white, and grayish brown clayey SILT, stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	57.3	62.7	DS
5	D-2	18	MH	@ 5': Mottled pale brown, white, and grayish brown clayey SILT, stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	49.4	66.7	CN
	B-1						B-1 @ 5-10' MD
	D-3	17	MH	@ 7.5': Mottled pale brown, white, and grayish brown clayey SILT, stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments.	62.8	64.4	
10	D-4	17	MH	@ 10': Mottled pale brown and reddish brown clayey SILT, stiff, moist, diatomaceous, abundant ½- to 1-inch bedrock fragments, trace FeO staining.	67.4	54.9	
	D-5	16	MH	@ 12.5': Mottled pale brown and reddish brown clayey SILT, stiff, very moist, diatomaceous, abundant ½- to 1-inch bedrock fragments, trace FeO staining.	72.5	54.0	CN
15	D-6	16	MH	@ 15': Mottled pale brown and reddish brown clayey SILT, stiff, very moist, diatomaceous, abundant ½- to 1-inch bedrock fragments, trace FeO staining.	63.7	60.0	
20	D-7	15	MH	@ 20': Mottled pale brown, white, and gray clayey SILT, stiff, very moist, diatomaceous, abundant angular ½- to 1-inch bedrock fragments.	54.1	62.6	CN
25	D-8	14	MH	@ 25': Mottled pale brown, white, and gray clayey SILT, stiff, very moist, diatomaceous, abundant angular ½- to 1-inch bedrock fragments.	63.9	57.8	
30							

Log Of Boring

Lennar
Proposed Residential Development
1 Through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Date(s)Drilled: 9-12-2025	**Logged By:** LZ	**H-13**
Drilling Method: Hollow Stem	**Drill Bit Size/Type:** 10.5"	Page 2 of 2
Drill Rig Type: CME 85	**Hammer Type:** Auto (140lbs @ 30")	
Sampling Method(s): Bulk, Modified California		**Total Depth Drilled (ft)** 41.5
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 428
Comments: Hand auger upper 2.5' for utility clearance.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
30	D-9	22	MH	@ 30': Mottled dark brown and pale brown clayey SILT, stiff, very moist to wet, diatomaceous, abundant 1- to 2-inch bedrock fragments.	40.0	75.4	
35	D-10	27	MH	**Monterey Formation (Tm)** @ 35': Pale brown to white clayey SILTSTONE, very stiff, moist, laminated, dipping ~30°, diatomaceous, some FeO staining, trace microfossils.	78.2	51.6	
40	D-11	35	MH	@ 40': Brown to grayish brown clayey SILTSTONE, very stiff, moist, laminated, diatomaceous, some FeO staining.	56.4	65.7	
45				Notes: Total Depth: 41.5 Feet. Groundwater Not Encountered. Backfilled with Cuttings and Tamped. Patched with Cold Patch Asphalt.			
50							
55							
60							

Log Of Boring

Lennar
Proposed Residential Development
1 Through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Date(s) Drilled: 09-12-2025	**Logged By:** AZ	**HA-1**
Drilling Method: Hand Auger	**Drill Bit Size/Type:** 3" Hand Auger	Page 1 of 1
Drill Rig Type: Manual	**Hammer Type:**	
Sampling Method(s):		**Total Depth Drilled (ft)** 13
Approximate Groundwater Depth: Groundwater not encountered.		**Approximate Ground Surface Elevation (ft)** 432
Comments: 13 Feet East of Top of Slope.		

Depth (ft)	SAMPLES Type Number	SAMPLES Blows Per Foot	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	REMARKS
0			ML	**Artificial Fill (Af)** @ 0-6': Mottled pale brown to grayish brown clayey SILT, loose to medium stiff, moist, some bedrock fragments.			
5			ML	@ 6-13': Mottled pale brown to grayish brown clayey SILT, medium stiff to stiff, moist, some bedrock fragments.			
10							
15				Notes: Total Depth: 13 Feet (Refusal on Rock Fragment). Groundwater Not Encountered. Backfilled With Cuttings.			
20							

Log Of Boring

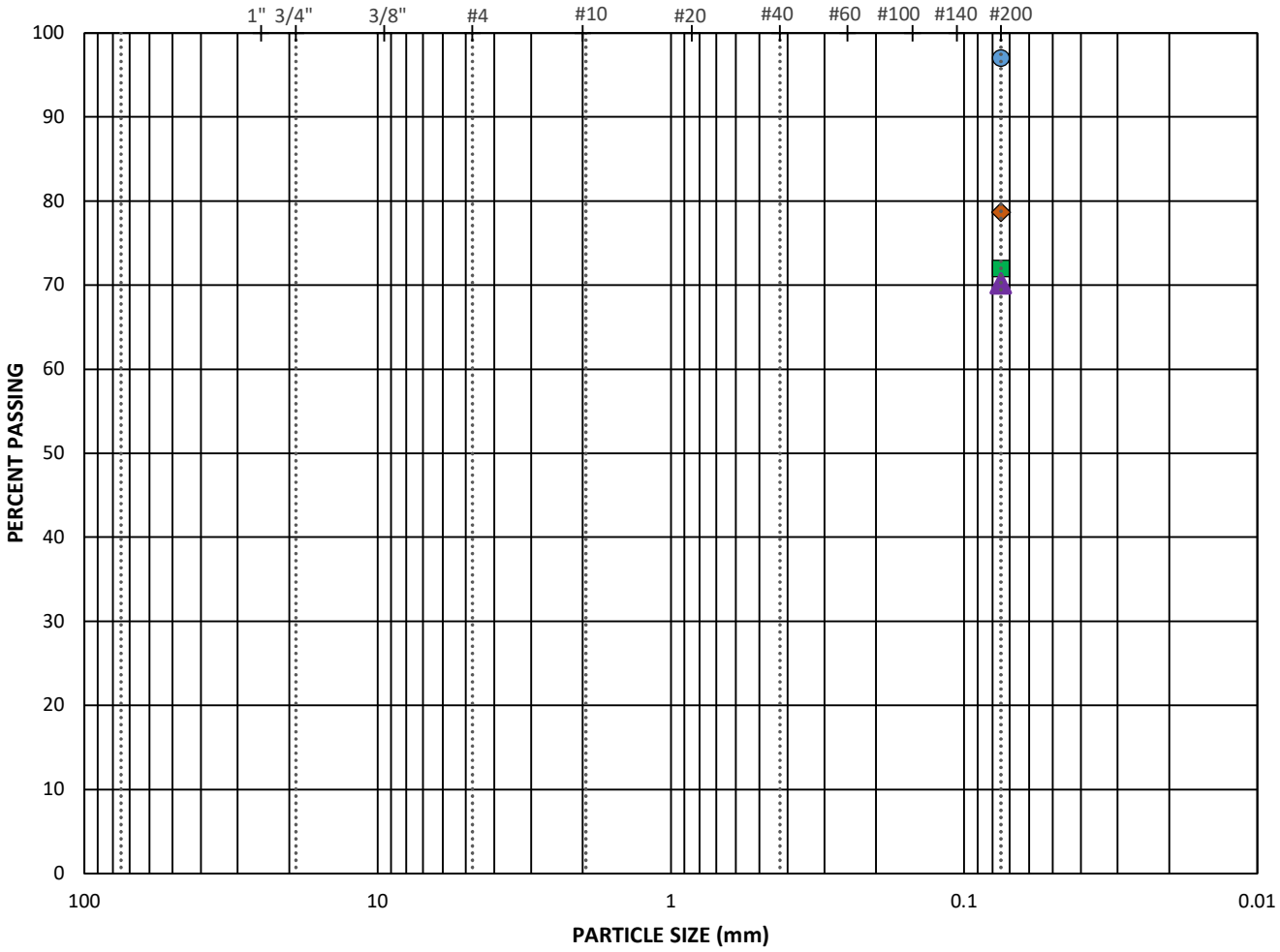
Lennar
Proposed Residential Development
1 through 5 Polaris Way
Aliso Viejo, California

Project Number: 25093-01

Appendix B

Appendix C

GRAVEL	SAND			SILT OR CLAY
	COARSE	MEDIUM	FINE	

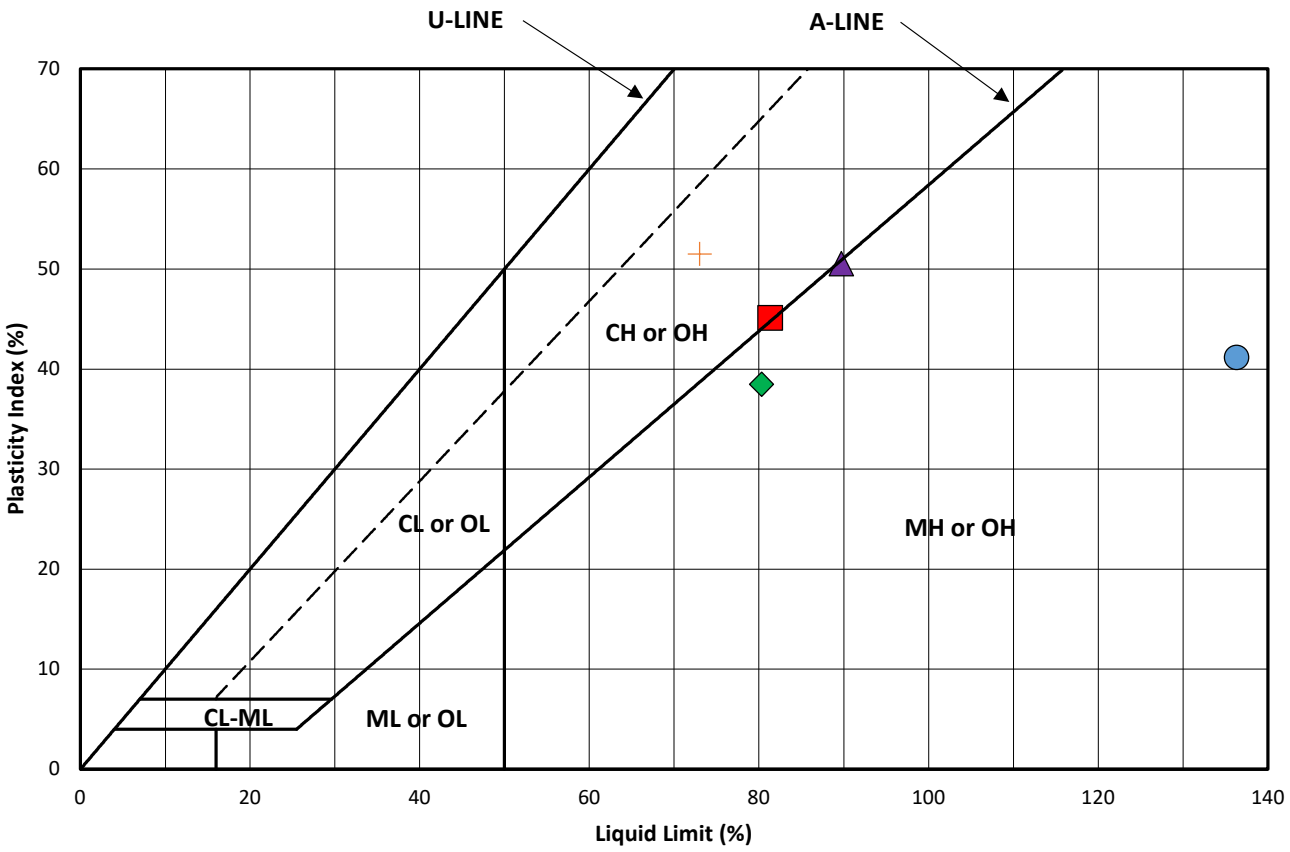


Symbol	Boring Number/ Location	Sample ID	Depth (ft)	Passing #200 (%)	USCS
●	H-2	D-1	2.5	97	MH
■	H-3	B-1	1-5	72	CH
◆	H-4	B-1	1-5	79	MH
▲	H-10	B-1	1-5	70	MH

PARTICLE SIZE DISTRIBUTION RESULTS
(ASTM D6913 and/or ASTM D1140)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025



Symbol	Boring Number/ Location	Sample ID	Depth (ft)	LL	PI	USCS
●	H-2	D-1	2.5	136	41	MH
■	H-3	B-1	1-5	81	45	CH
◆	H-4	B-1	1-5	80	38	MH
▲	H-10	B-1	1-5	90	51	MH
+	H-10	D-2	5	73	52	CH

ATTERBERG LIMITS TEST RESULTS (ASTM D4318)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Project No.: SAG-25-027

Tested by: GA

Checked by: KL

Date: 9/16/2025

Client: SA Geotechnical, Inc

Project Name: I-5 Polaris Way

Project Number: 25093-01

Boring No.: H-7

Sample No.: D4

Sample Type: Undisturbed Ring

Depth (ft): 10

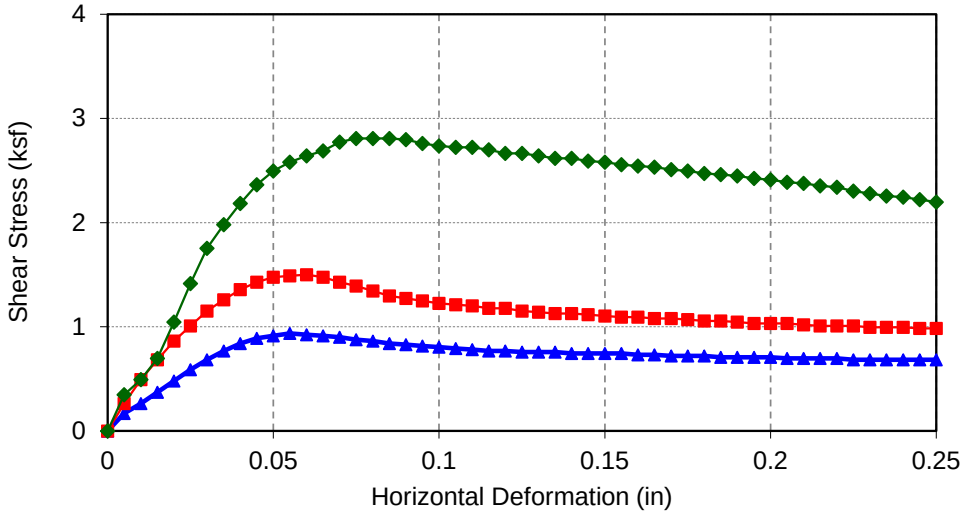
Soil Description: Olive Brown, Sandy/Clayey Silt (MH)

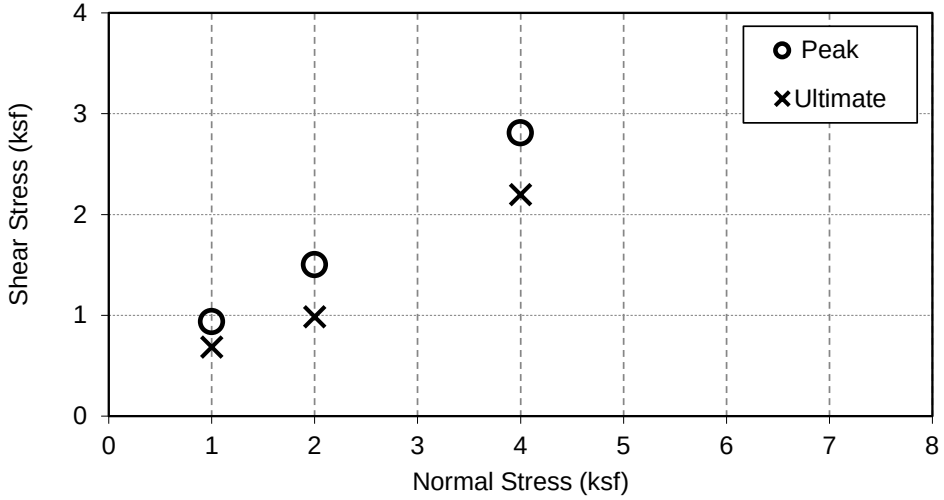
Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	O	**0.94**	**1.50**	**2.81**
Shear Stress @ End of Test (ksf)	X	**0.68**	**0.98**	**2.20**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	1.0132	1.0012	1.0012
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**51.0**	**51.0**	**51.0**
Final Moisture Content (%)	**75.4**	**60.3**	**58.5**
Dry Density (pcf)	**59.6**	**63.7**	**64.7**





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Project No.: SAG-25-027

Tested by: GA

Checked by: KL

Date: 9/16/2025

Client: SA Geotechnical, Inc

Project Name: I-5 Polaris Way

Project Number: 25093-01

Boring No.: H-10

Sample No.: D2

Sample Type: Undisturbed Ring

Depth (ft): 5

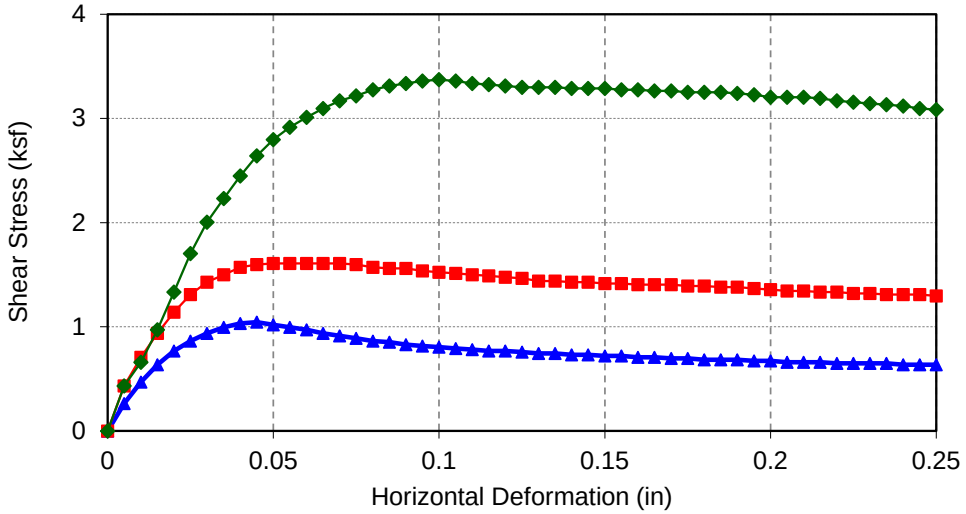
Soil Description: Olive Brown, Sandy/Clayey Silt (MH)

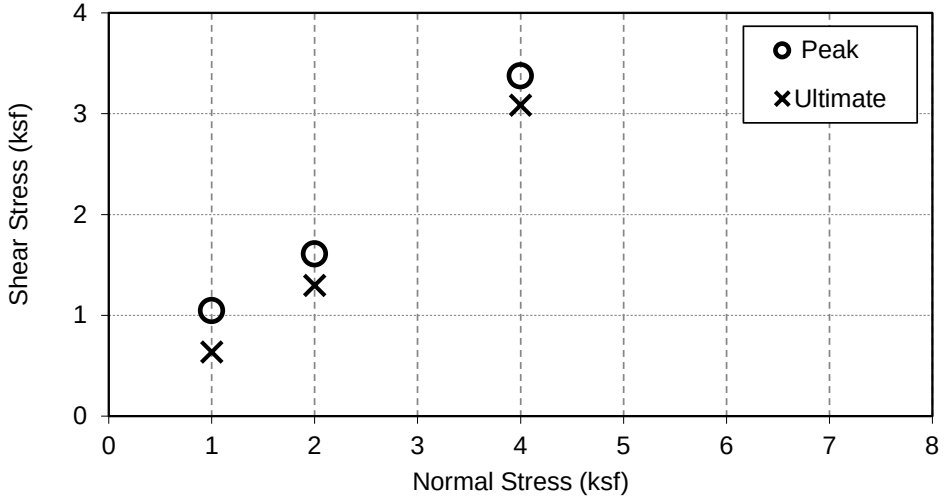
Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	O	**1.04**	**1.61**	**3.37**
Shear Stress @ End of Test (ksf)	X	**0.64**	**1.30**	**3.08**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	1.0075	1.0058	0.9911
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**52.0**	**52.0**	**52.0**
Final Moisture Content (%)	**73.8**	**64.1**	**60.2**
Dry Density (pcf)	**62.0**	**64.5**	**64.6**





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Project No.: SAG-25-027

Tested by: GA

Checked by: KL

Date: 9/16/2025

Client: SA Geotechnical, Inc

Project Name: I-5 Polaris Way

Project Number: 25093-01

Boring No.: H-11

Sample No.: D4

Sample Type: Undisturbed Ring

Depth (ft): 10

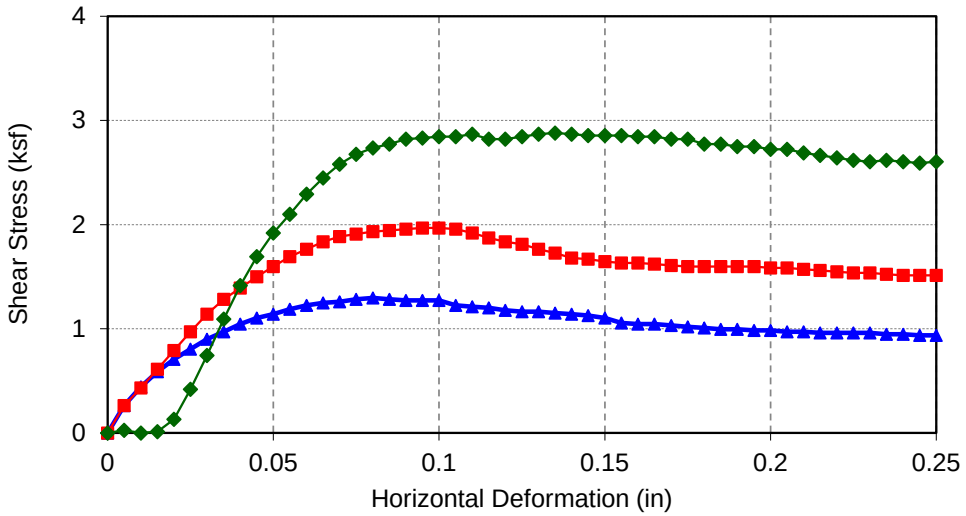
Soil Description: Olive Brown, Sandy/Clayey Silt (MH)

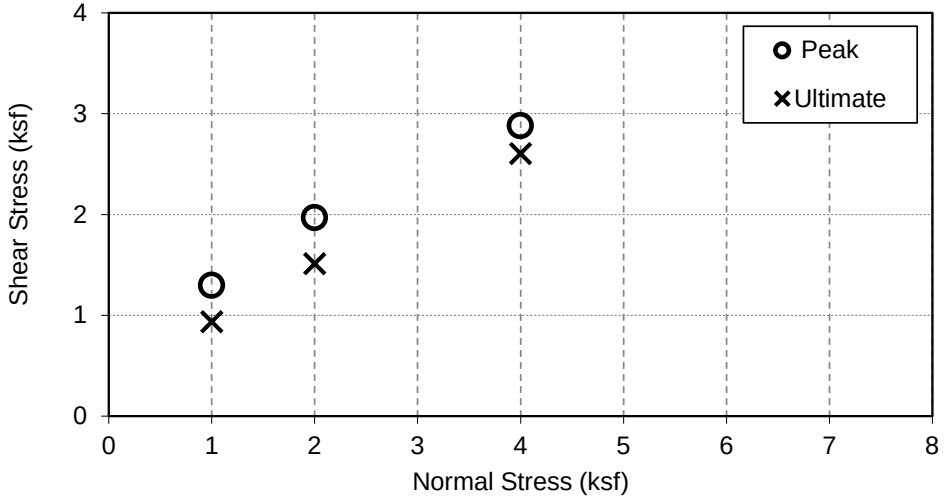
Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**1.30**	**1.97**	**2.88**
Shear Stress @ End of Test (ksf)	**X**	**0.94**	**1.51**	**2.60**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9996	0.9974	0.9910
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**56.6**	**56.6**	**56.6**
Final Moisture Content (%)	**85.6**	**87.1**	**83.1**
Dry Density (pcf)	**49.6**	**51.0**	**53.7**





HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Project No.: SAG-25-027

Tested by: GA

Checked by: KL

Date: 9/16/2025

Client: SA Geotechnical, Inc

Project Name: I-5 Polaris Way

Project Number: 25093-01

Boring No.: H-13

Sample No.: D1

Sample Type: Undisturbed Ring

Depth (ft): 2.5

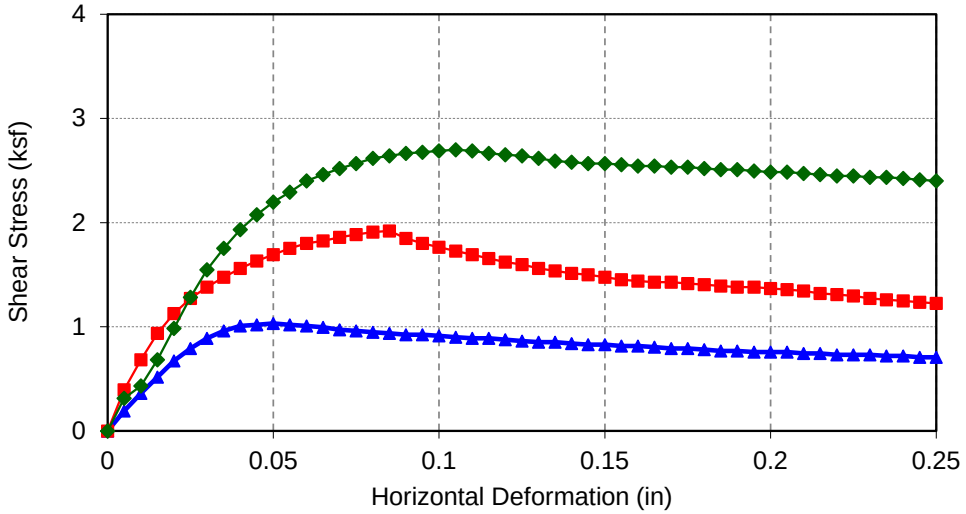
Soil Description: Grayish Brown, Sandy/Clayey Silt (MH)

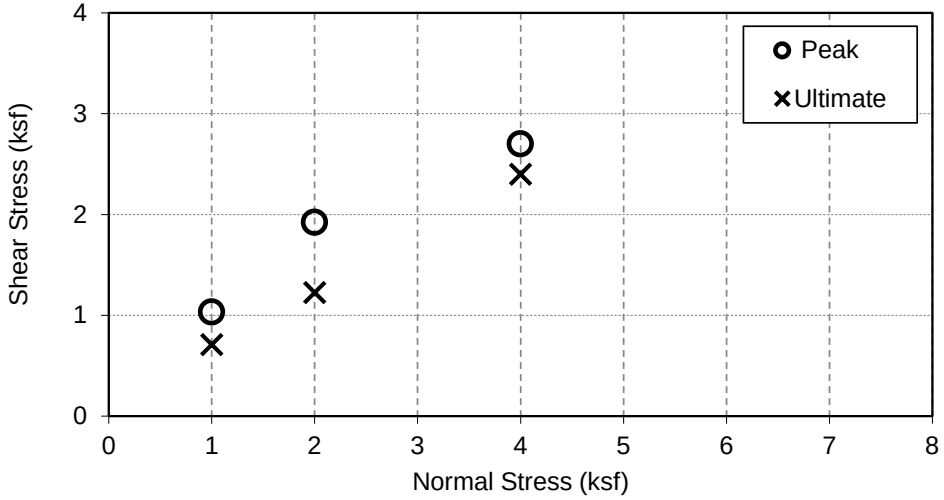
Type of test: **Consolidated, Drained**

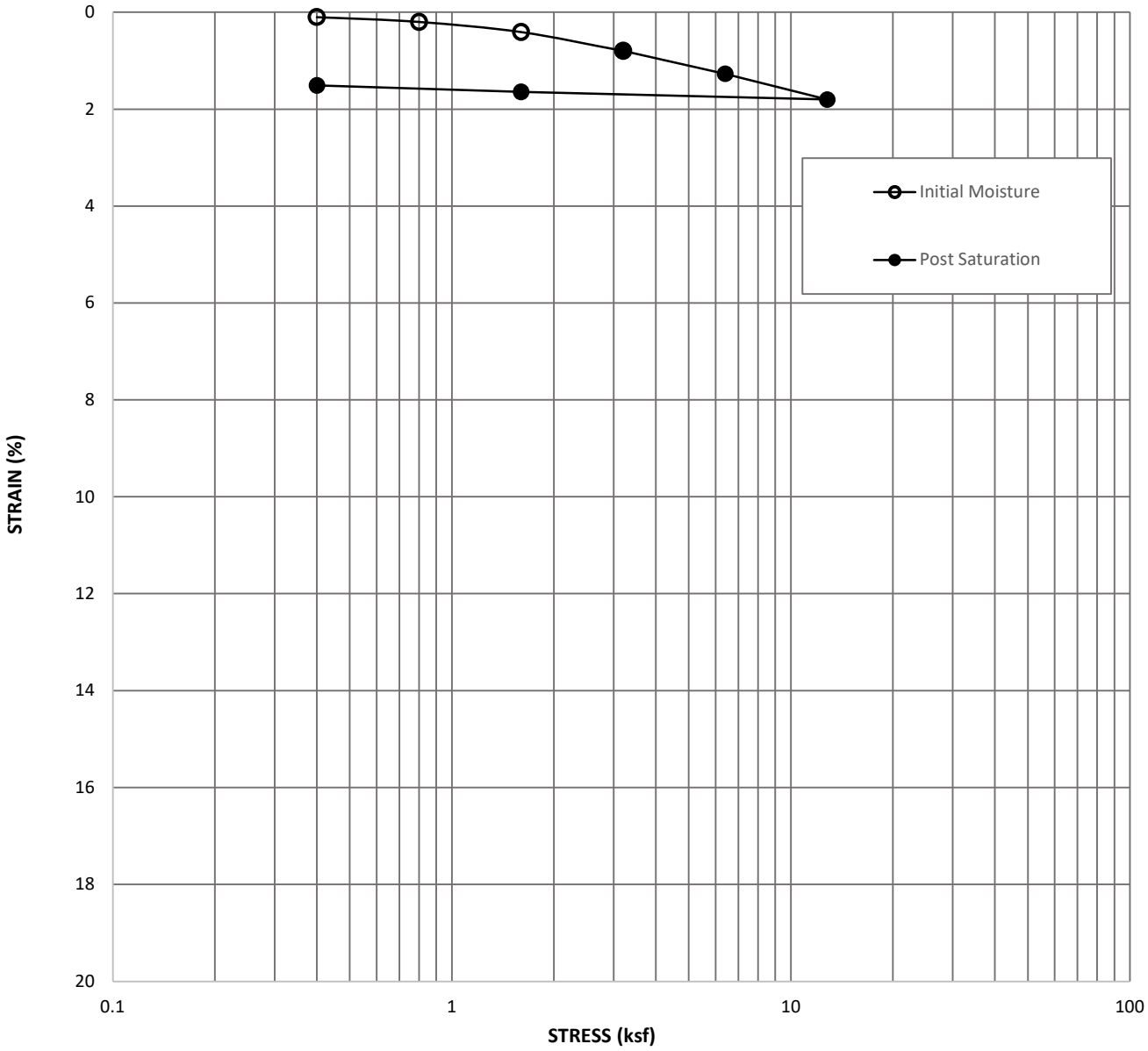
Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**1.03**	**1.92**	**2.70**
Shear Stress @ End of Test (ksf)	**X**	**0.71**	**1.22**	**2.40**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9917	0.9902	0.9766
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**47.8**	**47.8**	**47.8**
Final Moisture Content (%)	**71.6**	**64.7**	**54.4**
Dry Density (pcf)	**64.8**	**66.5**	**68.7**







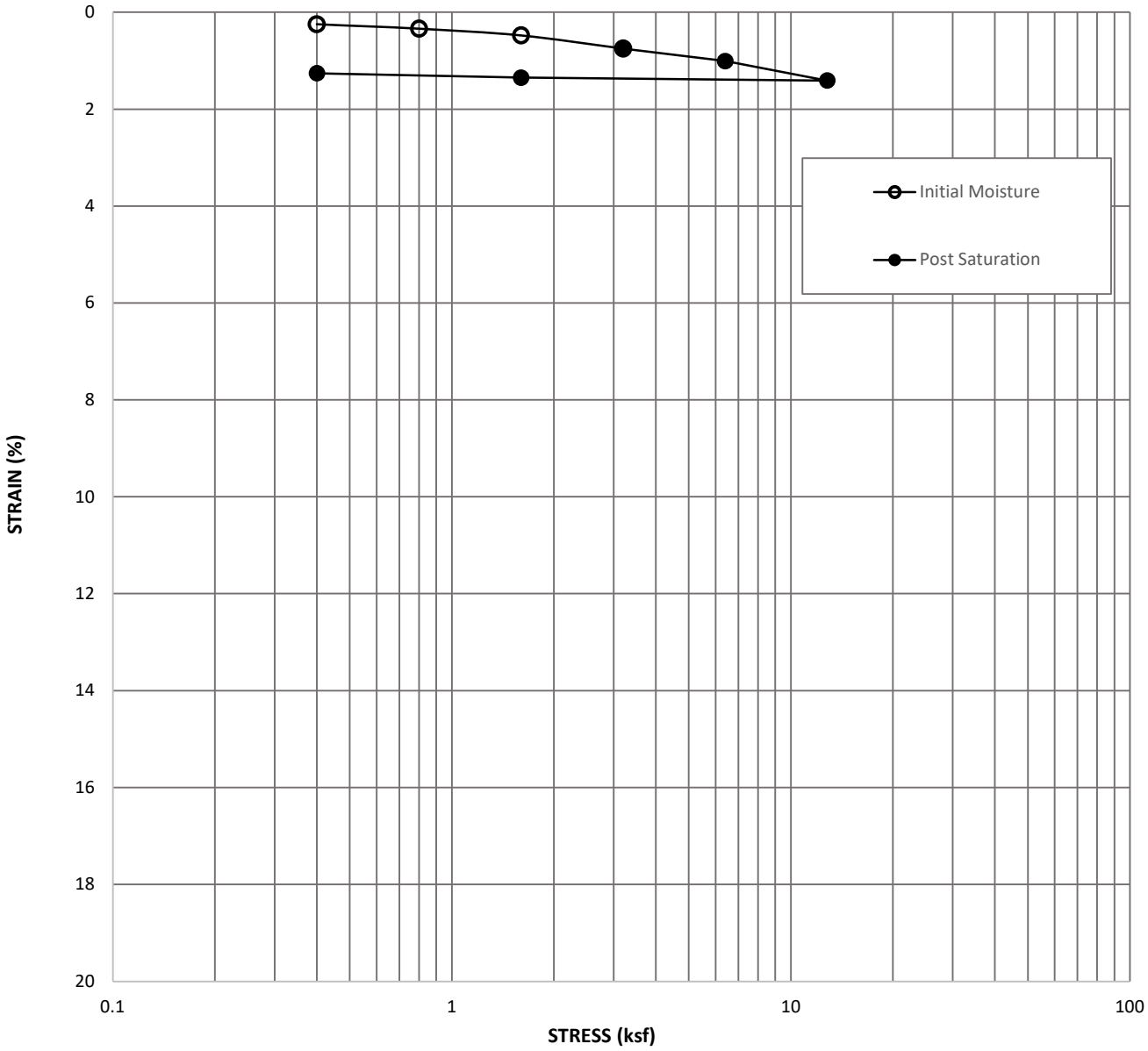
Boring No./Location: H-13	Sample ID: D-5	Depth: 17.5 ft	USCS: MH
Sample Description:	Olive Gray SILT (Af)		

Test Stage	Moisture Content (%)	Dry Density (pcf)	Saturation (%)	Void Ratio
Initial	72.5	54.0	91.5	2.179
Final	73.7	63.9	100.0	1.685

CONSOLIDATION TEST RESULTS (ASTM D2435)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025



Boring No./Location: H-13	Sample ID: D-7	Depth: 20 ft	USCS: MH
Sample Description:	Olive Gray SILT (Af)		

Test Stage	Moisture Content (%)	Dry Density (pcf)	Saturation (%)	Void Ratio
Initial	52.2	61.3	79.7	1.801
Final	83.1	65.8	100.0	1.610

CONSOLIDATION TEST RESULTS (ASTM D2435)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025

CONSOLIDATION TEST

ASTM D2435

Client :	SA Geotechnical, Inc	*HAI Project No.:*	SAG-25-028
Project Name:	I-5 Polaris Way	*Tested by:*	GA
Project Number:	25093-01	*Checked by:*	KL
Boring No.:	H-10	*Date:*	09/16/25
Sample No.:	D-3		
Type of Sample:	Undisturbed ring		
Depth (ft):	2.5		
Soil Description:	Olive, Sandy/Clayey Silt (MH)		

Initial Total Weight	Final Total Weight	Final Dry Weight
(g)	(g)	(g)
118.61	120.06	76.28

			Initial Conditions	Final Conditions
Height	H	(in)	1.000	0.967
Height of Solids	H_s	(in)	0.377	0.377
Height of Water	H_w	(in)	0.563	0.583
Height of Air	H_a	(in)	0.059	0.007
Dry Density		**(pcf)**	**63.4**	**67.7**
Water Content		**(%)**	**55.5**	**57.4**
Saturation		**(%)**	**90.5**	**98.8**

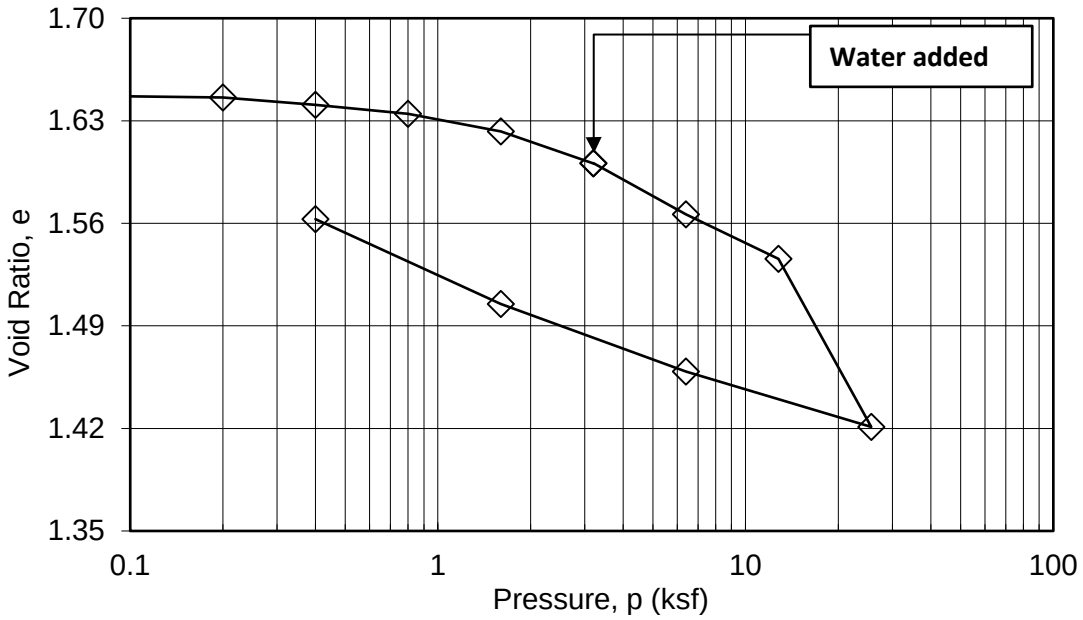
* Saturation is calculated based on Gs= 2.69

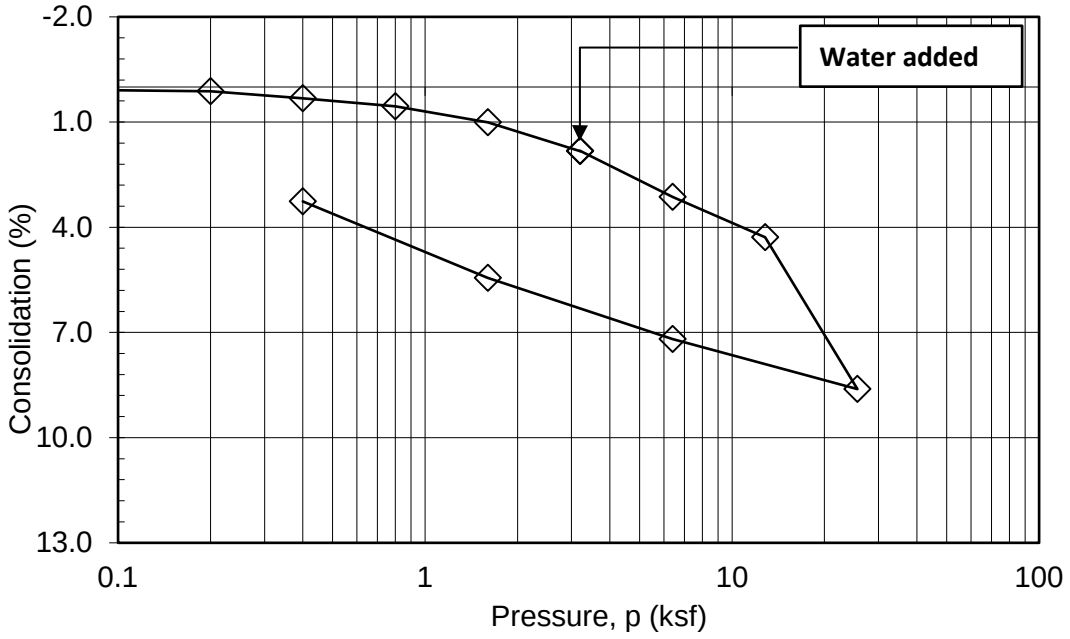
Load (ksf)	**δH (in)**	**H (in)**	**Voids (in)**	**e**	**Consol. (%)**	**a_v (ksf^{-1})**	**M_v (ksf^{-1})**	**Comment**
0.01	-------	1.0000	0.623	1.649	0			
0.2	0.0013	0.9988	0.621	1.646	0.1	1.7E-02	6.6E-03	
0.4	0.0032	0.9968	0.619	1.641	0.3	2.6E-02	9.8E-03	
0.8	0.0055	0.9945	0.617	1.635	0.5	1.5E-02	5.8E-03	
1.6	0.0101	0.9900	0.612	1.623	1.0	1.5E-02	5.7E-03	
3.2	0.0183	0.9818	0.604	1.601	1.8	1.4E-02	5.2E-03	
3.2	0.0182	0.9818	0.604	1.601	1.8	Water Added		
6.4	0.0314	0.9687	0.591	1.566	3.1	1.1E-02	4.2E-03	
12.8	0.0429	0.9572	0.580	1.536	4.3	4.8E-03	1.9E-03	
25.6	0.0862	0.9139	0.536	1.421	8.6	9.0E-03	3.7E-03	
6.4	0.0719	0.9282	0.551	1.459	7.2	Unloaded		
1.6	0.0545	0.9456	0.568	1.505	5.4			
0.4	0.0326	0.9674	0.590	1.563	3.3			

CONSOLIDATION TEST

ASTM D2435

Client :	SA Geotechnical, Inc	*HAI Project No.:*	SAG-25-028
Project Name:	I-5 Polaris Way	*Tested by:*	GA
Project Number:	25093-01	*Checked by:*	KL
Boring No.:	H-10	*Date:*	09/16/25
Sample No.:	D-3		
Type of Sample:	Undisturbed ring		
Depth (ft):	2.5		
Soil Description:	Olive, Sandy/Clayey Silt (MH)		





CONSOLIDATION TEST

ASTM D2435

Client :	SA Geotechnical, Inc	*HAI Project No.:*	SAG-25-028
Project Name:	I-5 Polaris Way	*Tested by:*	GA
Project Number:	25093-01	*Checked by:*	KL
Boring No.:	H-13	*Date:*	09/16/25
Sample No.:	D2		
Type of Sample:	Undisturbed ring		
Depth (ft):	5		
Soil Description:	Olive, Sandy/Clayey Silt (MH)		

Initial Total Weight	Final Total Weight	Final Dry Weight
(g)	(g)	(g)
122.42	121.59	78.79

			Initial Conditions	Final Conditions
Height	H	(in)	1.000	0.959
Height of Solids	H_s	(in)	0.388	0.388
Height of Water	H_w	(in)	0.581	0.570
Height of Air	H_a	(in)	0.031	0.001
Dry Density		**(pcf)**	**65.4**	**71.2**
Water Content		**(%)**	**55.4**	**54.3**
Saturation		**(%)**	**95.0**	**99.9**

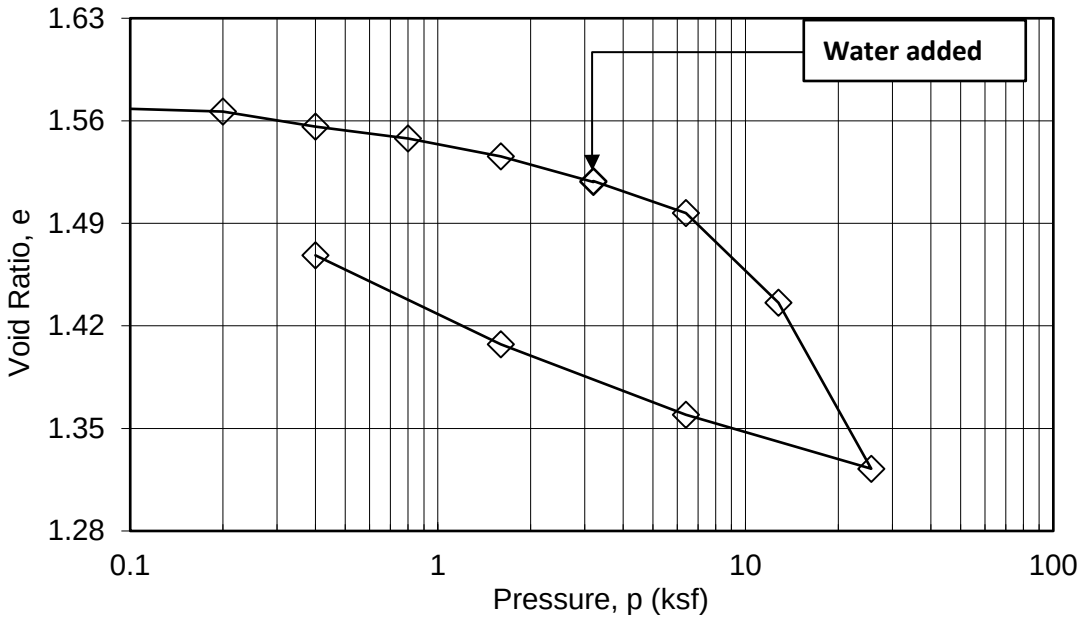
* Saturation is calculated based on Gs= 2.70

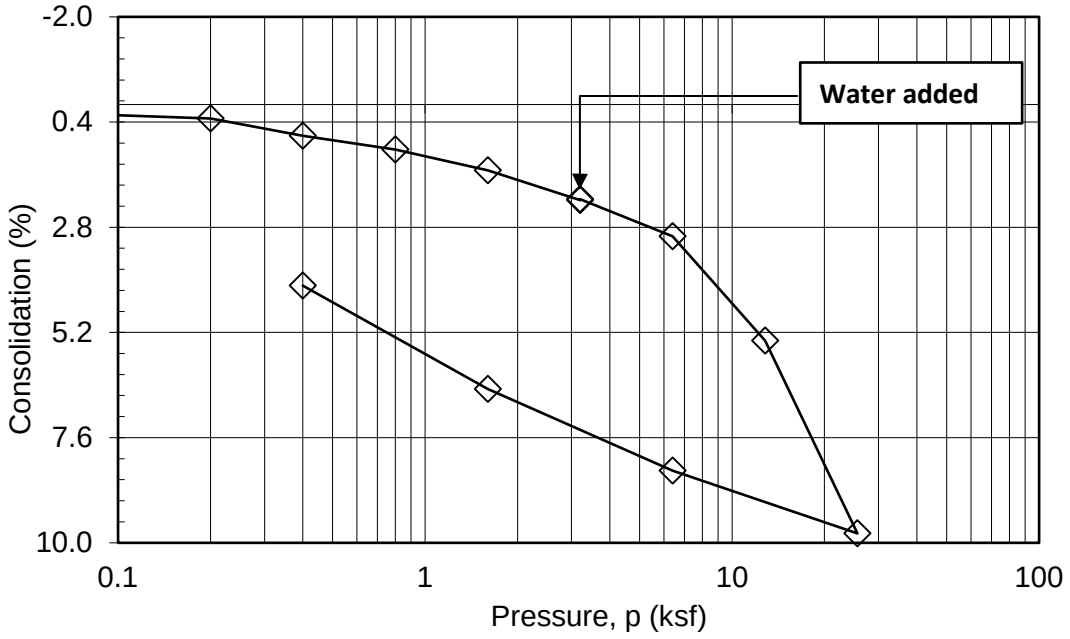
Load (ksf)	**δH (in)**	**H (in)**	**Voids (in)**	**e**	**Consol. (%)**	**a_v (ksf^{-1})**	**M_v (ksf^{-1})**	**Comment**
0.01	-------	1.0000	0.612	1.574	0			
0.2	0.0032	0.9969	0.608	1.566	0.3	4.3E-02	1.7E-02	
0.4	0.0071	0.9929	0.604	1.556	0.7	5.1E-02	2.0E-02	
0.8	0.0103	0.9897	0.601	1.548	1.0	2.1E-02	8.1E-03	
1.6	0.0150	0.9850	0.597	1.536	1.5	1.5E-02	6.0E-03	
3.2	0.0218	0.9782	0.590	1.518	2.2	1.1E-02	4.3E-03	
3.2	0.0216	0.9784	0.590	1.519	2.2	Water Added		
6.4	0.0301	0.9700	0.582	1.497	3.0	6.8E-03	2.7E-03	
12.8	0.0539	0.9462	0.558	1.436	5.4	9.6E-03	3.9E-03	
25.6	0.0979	0.9022	0.514	1.323	9.8	8.8E-03	3.8E-03	
6.4	0.0836	0.9165	0.528	1.359	8.4	Unloaded		
1.6	0.0649	0.9351	0.547	1.407	6.5			
0.4	0.0413	0.9587	0.570	1.468	4.1			

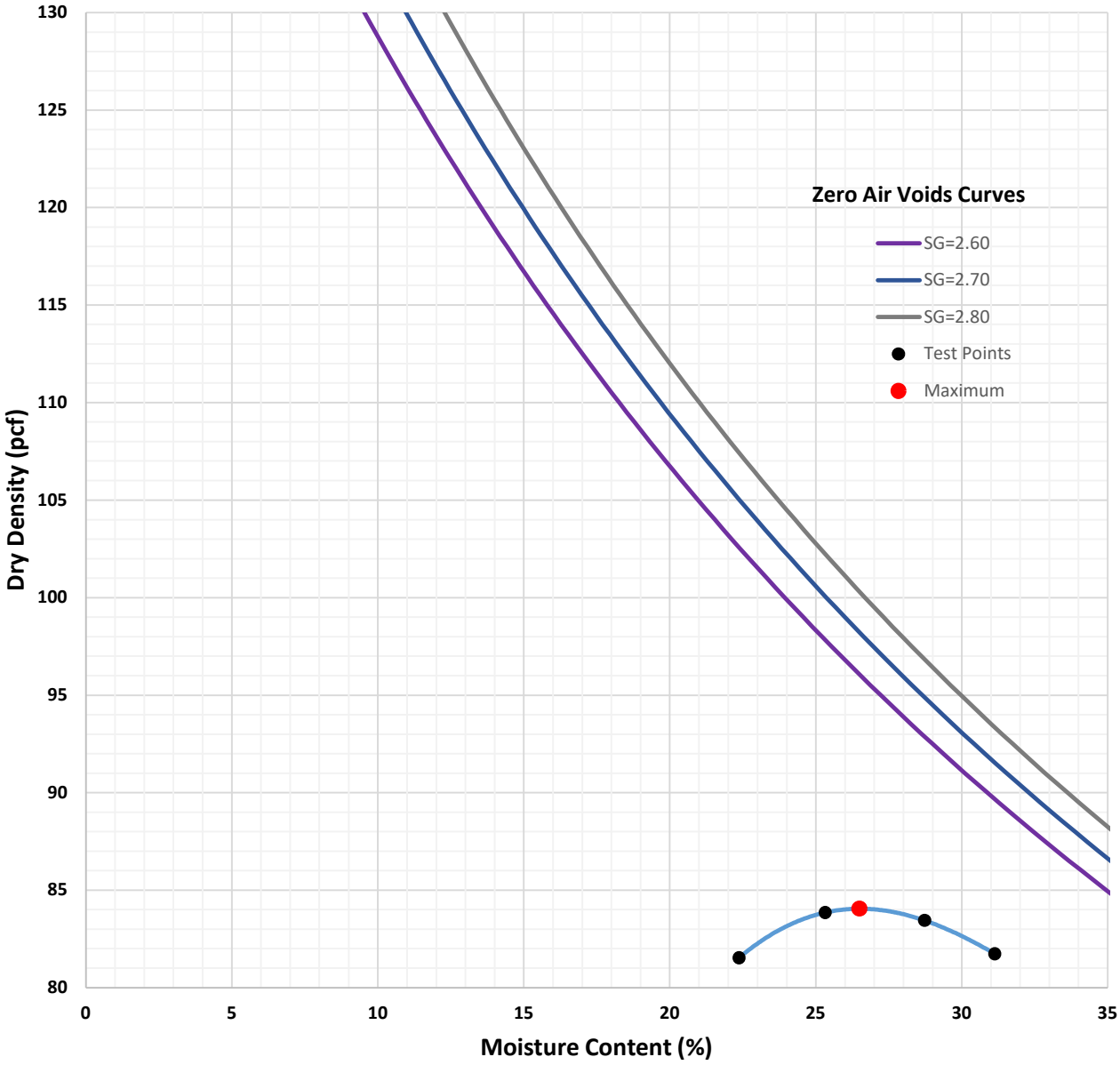
CONSOLIDATION TEST

ASTM D2435

Client :	SA Geotechnical, Inc	*HAI Project No.:*	SAG-25-028
Project Name:	I-5 Polaris Way	*Tested by:*	GA
Project Number:	25093-01	*Checked by:*	KL
Boring No.:	H-13	*Date:*	09/16/25
Sample No.:	D2		
Type of Sample:	Undisturbed ring		
Depth (ft):	5		
Soil Description:	Olive, Sandy/Clayey Silt (MH)		







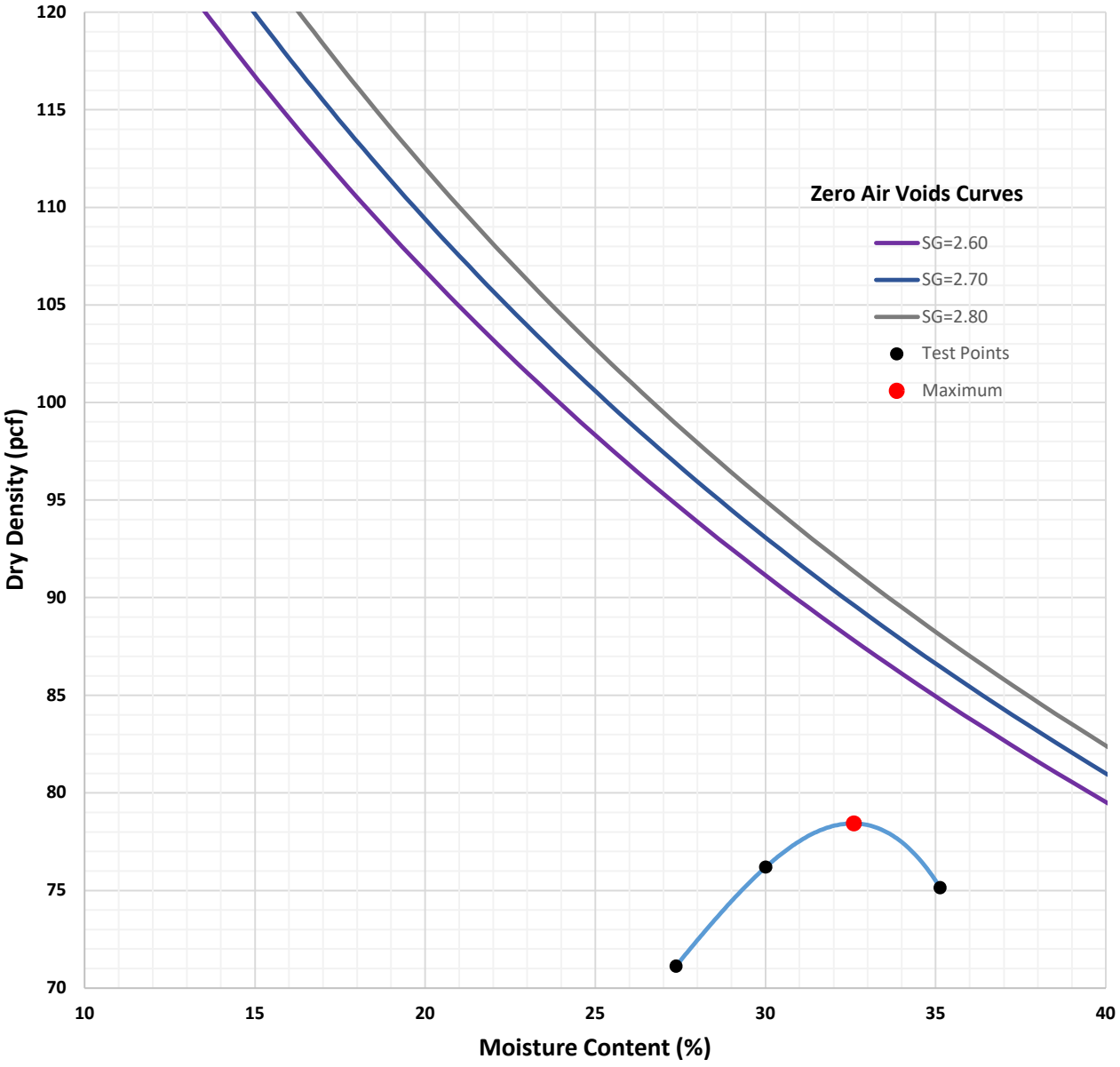
Curve No.	1		
Maximum Dry Density (pcf)	84.0	**Corrected Maximum Dry Denisty (pcf)**	88.6
Opt. Moisture Content (%)	26.5	**Corrected Opt. Moisture Content (%)**	24.0
No. 4 Oversize Particles (%)	10.3	**Specific Gravity (Oversize)**	2.7

Boring No./Location: H-4	**Sample ID: B-1**	**Depth: 1-5 ft**	**USCS: MH**
Sample Description:	Pale Gray to White SILT (Af)		
Comments:			

COMPACTION TEST RESULTS (ASTM D1557/D4718)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025



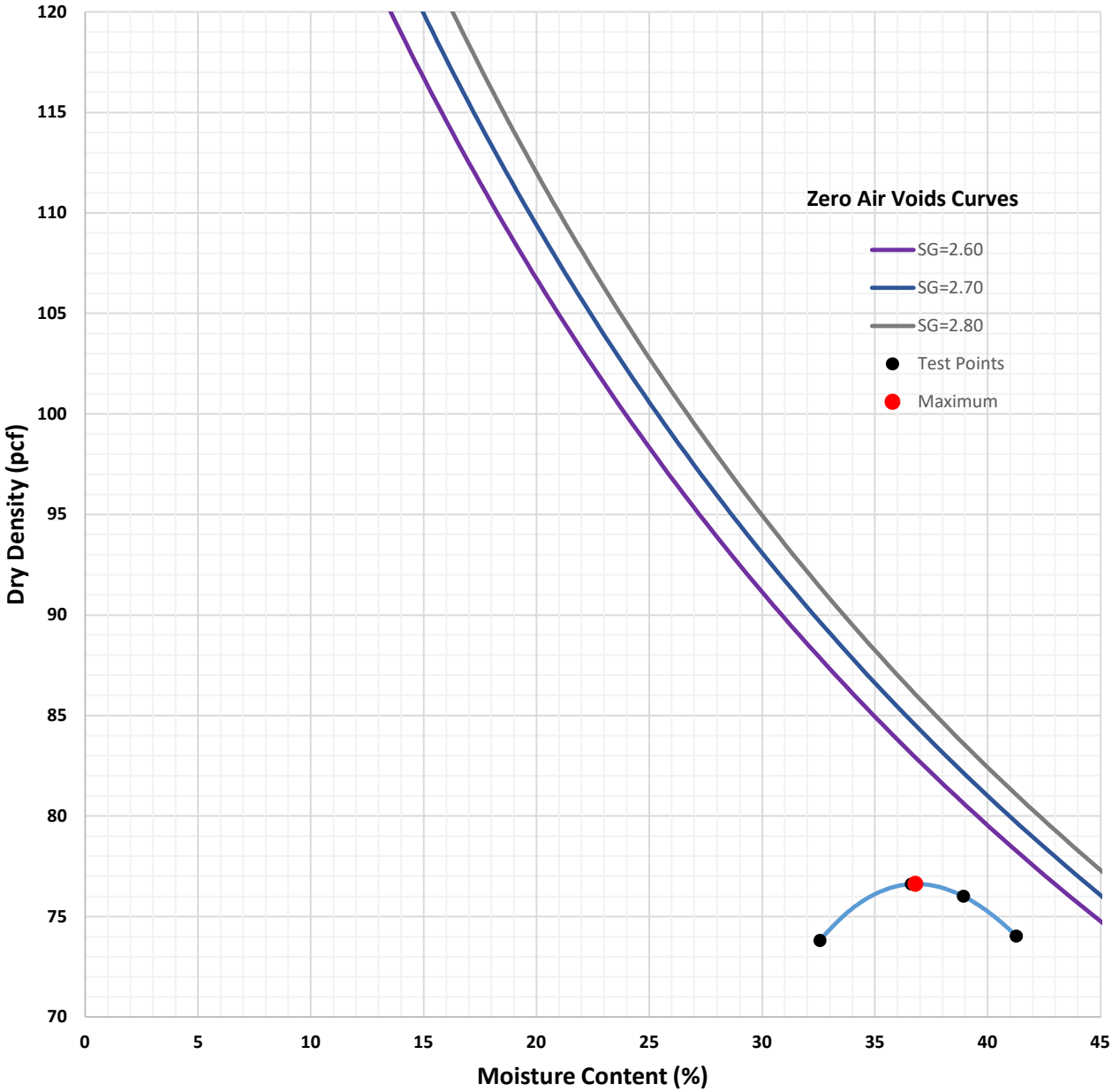
Curve No.			2
Maximum Dry Density (pcf)	78.4	**Corrected Maximum Dry Denisty (pcf)**	84.0
Opt. Moisture Content (%)	32.6	**Corrected Opt. Moisture Content (%)**	28.8
No. 4 Oversize Particles (%)	12.5	**Specific Gravity (Oversize)**	2.7

Boring No./Location: H-10	**Sample ID: B-1**	**Depth: 1-5 ft**	**USCS: MH**
Sample Description:	Olive Gray SILT (Af)		
Comments:			

COMPACTION TEST RESULTS (ASTM D1557/D4718)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025



Maximum Dry Density (pcf)	76.6
Optimum Moisture Content (%)	36.8
Curve No.	3

Boring No./Location: H-13	**Sample ID: B-1**	**Depth: 5-10 ft**	**USCS: CH**
Sample Description:	Olive Gray Silty CLAY (Af)		
Comments:			

COMPACTION TEST RESULTS (ASTM D1557)

Lennar - Polaris Place Commercial Center
1-5 Polaris Way
Aliso Viejo, California

Project No. 25093-01
Date: September 2025

Sample	Compacted Moisture (%)	Compacted Dry Density (pcf)	Expansion Index	Expansion Potential	Soluble Sulfate (%)	Sulfate Exposure
H-3 B-1 1-5 ft CH	19.5	85.3	89	Medium	0.030	S0
H-4 B-1 1-5 ft MH	23.1	78.8	64	Medium	0.043	S0
H-10 B-1 1-5 ft MH	27.5	61.1	70	Medium	0.030	S0

Table 1. ASTM D4829 Classification of Expansion Potential of Soils Based on EI

Expansion Index, EI	**Potential Expansion**
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

Table 1. ACI C318-08 Sulfate Exposure Classes

Severity of Potential Exposure	**Water Soluble Sulfate in Soil, % Mass**
S0	$SO_4 < 0.10$
S1	$0.10 \leq SO_4 \leq 0.20$
S2	$0.20 \leq SO_4 \leq 2.00$
S3	$SO_4 > 2.0$

EXPANSION INDEX AND SOLUBLE SULFATES TEST RESULTS
(ASTM D4829, ASTM C1580)

Ohalloran Residence 2430 Holly Lane Newport Beach, California	Project No. 24XXX-XX Date:

Appendix D

Announcement
ASCE 7-22 is now available.

Latitude, Longitude: 33.58495, -117.72402



Date	2/20/2026, 8:30:12 AM
Design Code Reference Document	ASCE7-22
Risk Category	II
Site Class	D

Type	Value	Description (Data)
S_S	1.4	The MCE_R spectral response acceleration at 0.2 seconds for Site Class BC, in units of g.
S_1	0.44	The MCE_R spectral response acceleration at 1 second for Site Class BC, in units of g.
S_{MS}	1.64	S_{MS} = 1.5 x S_{DS}, the Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class.
S_{M1}	1.1	S_{M1} = 1.5 x S_{D1}, the MCE_R spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class.
S_{DS}	1.1	The design spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class, in units of g.
S_{D1}	0.73	The design spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class, in units of g

Type	Value	Description (Data Contd.)
SDC	D	Seismic design category
PGA_M	0.62	PGA_M, the Geometric-Mean Maximum Considered Earthquake (MCE_G) peak ground acceleration for the user-specified Site Class, in units of g
T_S	0.668	T_S = S_{D1}/S_{DS}, in seconds, for construction of the two-period design spectrum
T_0	0.134	T_0 = 0.2 x T_S, in seconds, for construction of the two-period design response spectrum
T_L	8	T_L, the long-period transition period, in seconds, for construction of the two-period design response spectrum

Type	Value	Description (Underlying Data and Metadata)
PGA_{uh}	0.62	Probabilistic uniform-hazard (2%-in-50-years), geometric-mean peak ground acceleration, in units of g.
PGA_{84th}	0.99	Deterministic 84th-percentile, geometric-mean peak ground acceleration (without deterministic lower limit), in units of g.
V_{S30}	260	The shear-wave velocity used for the user-specified Site Class, in units of m/s
Spatial Interpolation Method	linearloglinear	Identifier for spatial interpolation method used to obtain values for location of interest from underlying gridded values: "linearloglinear" for bilinear of natural logarithm of values.
PGA_{dFloor}	0.53	Deterministic lower limit peak ground acceleration (PGA_G) for the user-specified Site Class, in units of g.
riskTargetedSpectrum		Probabilistic risk-targeted, maximum direction response spectrum (for 1%-in-50-years collapse risk)
eightyFourthSpectrum		Deterministic 84th-percentile, maximum-direction response spectrum (without deterministic lower limit)

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Multi Period

Period T(s)	Design Spectrum Ordinates (g)	MCE_R Spectrum Ordinates (g)
0	0.47	0.71
0.01	0.48	0.72
0.02	0.48	0.72
0.03	0.49	0.73
0.05	0.56	0.84
0.075	0.7	1.05
0.1	0.84	1.26
0.15	1.02	1.53
0.2	1.13	1.69
0.25	1.18	1.78
0.3	1.22	1.83
0.4	1.19	1.78
0.5	1.12	1.69
0.75	0.9	1.34
1	0.73	1.1
1.5	0.51	0.76
2	0.38	0.57
3	0.24	0.36
4	0.17	0.25
5	0.13	0.19
7.5	0.078	0.12
10	0.053	0.079

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Two Period

Period T(s)	Design Spectrum Ordinates (g)	MCE_R Spectrum Ordinates (g)
0	0.44	0.66
0.025	0.56	0.84
0.05	0.68	1.03
0.1	0.93	1.39
0.13	1.1	1.64
0.15	1.1	1.64
0.2	1.1	1.64
0.25	1.1	1.64
0.3	1.1	1.64
0.35	1.1	1.64
0.4	1.1	1.64
0.45	1.1	1.64
0.5	1.1	1.64
0.55	1.1	1.64
0.6	1.1	1.64
0.65	1.1	1.64
0.67	1.1	1.64
0.7	1.05	1.57
0.75	0.98	1.46
0.8	0.91	1.37
0.85	0.86	1.29
0.9	0.81	1.22
0.95	0.77	1.16

Period T(s)	Design Spectrum Ordinates (g)	MCE_R Spectrum Ordinates (g)
1	0.73	1.1
1.05	0.7	1.05
1.1	0.67	1
1.15	0.64	0.95
1.2	0.61	0.91
1.25	0.59	0.88
1.3	0.56	0.84
1.35	0.54	0.81
1.4	0.52	0.78
1.45	0.5	0.76
1.5	0.49	0.73
1.55	0.47	0.71
1.6	0.46	0.69
1.65	0.44	0.67
1.7	0.43	0.65
1.75	0.42	0.63
1.8	0.41	0.61
1.85	0.4	0.59
1.9	0.39	0.58
1.95	0.38	0.56
2	0.37	0.55
2.05	0.36	0.54
2.1	0.35	0.52
2.15	0.34	0.51
2.2	0.33	0.5
2.25	0.33	0.49
2.3	0.32	0.48
2.35	0.31	0.47
2.4	0.3	0.46
2.45	0.3	0.45
2.5	0.29	0.44
2.55	0.29	0.43
2.6	0.28	0.42
2.65	0.28	0.41
2.7	0.27	0.41
2.75	0.27	0.4
2.8	0.26	0.39
2.85	0.26	0.39
2.9	0.25	0.38
2.95	0.25	0.37
3	0.24	0.37
3.05	0.24	0.36
3.1	0.24	0.35
3.15	0.23	0.35
3.2	0.23	0.34
3.25	0.23	0.34
3.3	0.22	0.33
3.35	0.22	0.33
3.4	0.22	0.32
3.45	0.21	0.32

Period T(s)	Design Spectrum Ordinates (g)	MCE_R Spectrum Ordinates (g)
3.5	0.21	0.31
3.55	0.21	0.31
3.6	0.2	0.3
3.65	0.2	0.3
3.7	0.2	0.3
3.75	0.2	0.29
3.8	0.19	0.29
3.85	0.19	0.29
3.9	0.19	0.28
3.95	0.19	0.28
4	0.18	0.27
4.05	0.18	0.27
4.1	0.18	0.27
4.15	0.18	0.26
4.2	0.17	0.26
4.25	0.17	0.26
4.3	0.17	0.26
4.35	0.17	0.25
4.4	0.17	0.25
4.45	0.16	0.25
4.5	0.16	0.24
4.55	0.16	0.24
4.6	0.16	0.24
4.65	0.16	0.24
4.7	0.16	0.23
4.75	0.15	0.23
4.8	0.15	0.23
4.85	0.15	0.23
4.9	0.15	0.22
4.95	0.15	0.22
5	0.15	0.22
5.05	0.14	0.22
5.1	0.14	0.22
5.15	0.14	0.21
5.2	0.14	0.21
5.25	0.14	0.21
5.3	0.14	0.21
5.35	0.14	0.21
5.4	0.14	0.2
5.45	0.13	0.2
5.5	0.13	0.2
5.55	0.13	0.2
5.6	0.13	0.2
5.65	0.13	0.19
5.7	0.13	0.19
5.75	0.13	0.19
5.8	0.13	0.19
5.85	0.13	0.19
5.9	0.12	0.19
5.95	0.12	0.18

Period T(s)	Design Spectrum Ordinates (g)	MCE_R Spectrum Ordinates (g)
6	0.12	0.18
6.05	0.12	0.18
6.1	0.12	0.18
6.15	0.12	0.18
6.2	0.12	0.18
6.25	0.12	0.18
6.3	0.12	0.17
6.35	0.12	0.17
6.4	0.11	0.17
6.45	0.11	0.17
6.5	0.11	0.17
6.55	0.11	0.17
6.6	0.11	0.17
6.65	0.11	0.17
6.7	0.11	0.16
6.75	0.11	0.16
6.8	0.11	0.16
6.85	0.11	0.16
6.9	0.11	0.16
6.95	0.11	0.16
7	0.1	0.16
7.05	0.1	0.16
7.1	0.1	0.15
7.15	0.1	0.15
7.2	0.1	0.15
7.25	0.1	0.15
7.3	0.1	0.15
7.35	0.1	0.15
7.4	0.099	0.15
7.45	0.098	0.15
7.5	0.098	0.15
7.55	0.097	0.15
7.6	0.096	0.14
7.65	0.096	0.14
7.7	0.095	0.14
7.75	0.094	0.14
7.8	0.094	0.14
7.85	0.093	0.14
7.9	0.093	0.14
7.95	0.092	0.14
8	0.091	0.14
8.05	0.09	0.14
8.1	0.089	0.13
8.15	0.088	0.13
8.2	0.087	0.13
8.25	0.086	0.13
8.3	0.085	0.13
8.35	0.084	0.13
8.4	0.083	0.12
8.45	0.082	0.12

Period T(s)	Design Spectrum Ordinates (g)	MCE_R Spectrum Ordinates (g)
8.5	0.081	0.12
8.55	0.08	0.12
8.6	0.079	0.12
8.65	0.078	0.12
8.7	0.077	0.12
8.75	0.076	0.11
8.8	0.076	0.11
8.85	0.075	0.11
8.9	0.074	0.11
8.95	0.073	0.11
9	0.072	0.11

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Probabilistic & Deterministic

Period T(s)	Risk Targeted Spectrum Ordinates (g)	84th Spectrum Ordinates(g)
0	0.71	1.2
0.01	0.72	1.2
0.02	0.72	1.2
0.03	0.73	1.19
0.05	0.84	1.26
0.075	1.05	1.46
0.1	1.26	1.66
0.15	1.53	1.98
0.2	1.69	2.32
0.25	1.78	2.61
0.3	1.83	2.86
0.4	1.78	3.01
0.5	1.69	2.93
0.75	1.34	2.46
1	1.1	2.07
1.5	0.76	1.47
2	0.57	1.11
3	0.36	0.69
4	0.25	0.46
5	0.19	0.33
7.5	0.12	0.17
10	0.079	0.11

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.58495

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

-117.72402

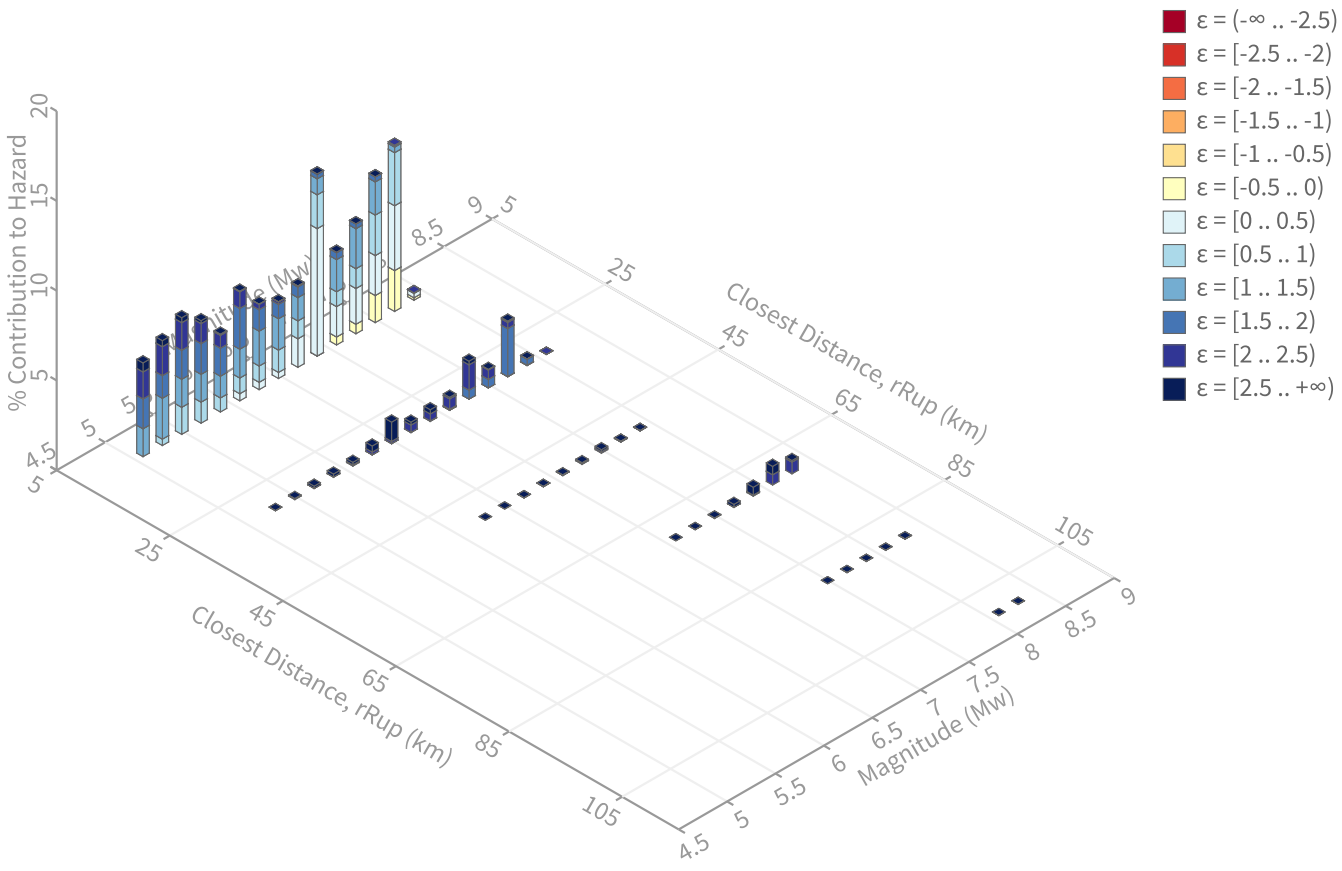
Site Class

259 m/s (Site class D)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.61699579 g

Recovered targets

Return period: 2887.1113 yrs
Exceedance rate: 0.00034636697 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (over all sources)

m: 6.63
r: 11.88 km
ε_0: 1.2 σ

Mode (largest m-r bin)

m: 6.89
r: 4.76 km
ε_0: 0.41 σ
Contribution: 10.14 %

Mode (largest m-r-ε_0 bin)

m: 6.89
r: 3.77 km
ε_0: 0.19 σ
Contribution: 7.09 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							29.10
San Joaquin Hills [2]		3.58	7.11	0.23	117.713°W	33.600°N	30.95	14.96
Newport-Inglewood (Offshore) [2]		11.07	7.30	1.17	117.806°W	33.513°N	223.49	5.25
Elsinore (Glen Ivy) rev [1]		29.59	7.32	2.12	117.502°W	33.776°N	43.93	2.26
UC33brAvg_FM31	System							25.91
San Joaquin Hills [2]		3.58	7.52	0.19	117.713°W	33.600°N	30.95	9.26
Newport-Inglewood (Offshore) [2]		11.07	7.30	1.17	117.806°W	33.513°N	223.49	5.52
Oceanside alt1 [6]		7.29	7.00	0.52	117.802°W	33.514°N	222.70	3.50
Elsinore (Glen Ivy) rev [1]		29.59	7.30	2.13	117.502°W	33.776°N	43.93	2.21
UC33brAvg_FM31 (opt)	Grid							22.58
PointSourceFinite: -117.724, 33.607		5.63	5.67	1.07	117.724°W	33.607°N	0.00	4.10
PointSourceFinite: -117.724, 33.607		5.63	5.67	1.07	117.724°W	33.607°N	0.00	4.10
PointSourceFinite: -117.724, 33.652		8.83	5.68	1.56	117.724°W	33.652°N	0.00	1.77
PointSourceFinite: -117.724, 33.652		8.83	5.68	1.56	117.724°W	33.652°N	0.00	1.77
PointSourceFinite: -117.724, 33.688		11.29	5.94	1.73	117.724°W	33.688°N	0.00	1.66
PointSourceFinite: -117.724, 33.688		11.29	5.94	1.73	117.724°W	33.688°N	0.00	1.66
UC33brAvg_FM32 (opt)	Grid							22.41
PointSourceFinite: -117.724, 33.607		5.63	5.66	1.07	117.724°W	33.607°N	0.00	4.06
PointSourceFinite: -117.724, 33.607		5.63	5.66	1.07	117.724°W	33.607°N	0.00	4.06
PointSourceFinite: -117.724, 33.652		8.83	5.68	1.56	117.724°W	33.652°N	0.00	1.73
PointSourceFinite: -117.724, 33.652		8.83	5.68	1.56	117.724°W	33.652°N	0.00	1.73
PointSourceFinite: -117.724, 33.688		11.29	5.94	1.73	117.724°W	33.688°N	0.00	1.65
PointSourceFinite: -117.724, 33.688		11.29	5.94	1.73	117.724°W	33.688°N	0.00	1.65

Appendix E

Summary of Slope Stability Analysis
Cross-Section B-B'

Filename	Description	Factor of Safety (FS)	
		Static	*Pseudostatic*
02, 02p	Global Stability of the Design Slope, Block Search through Bedrock with along-bedding strengths	1.53	1.25

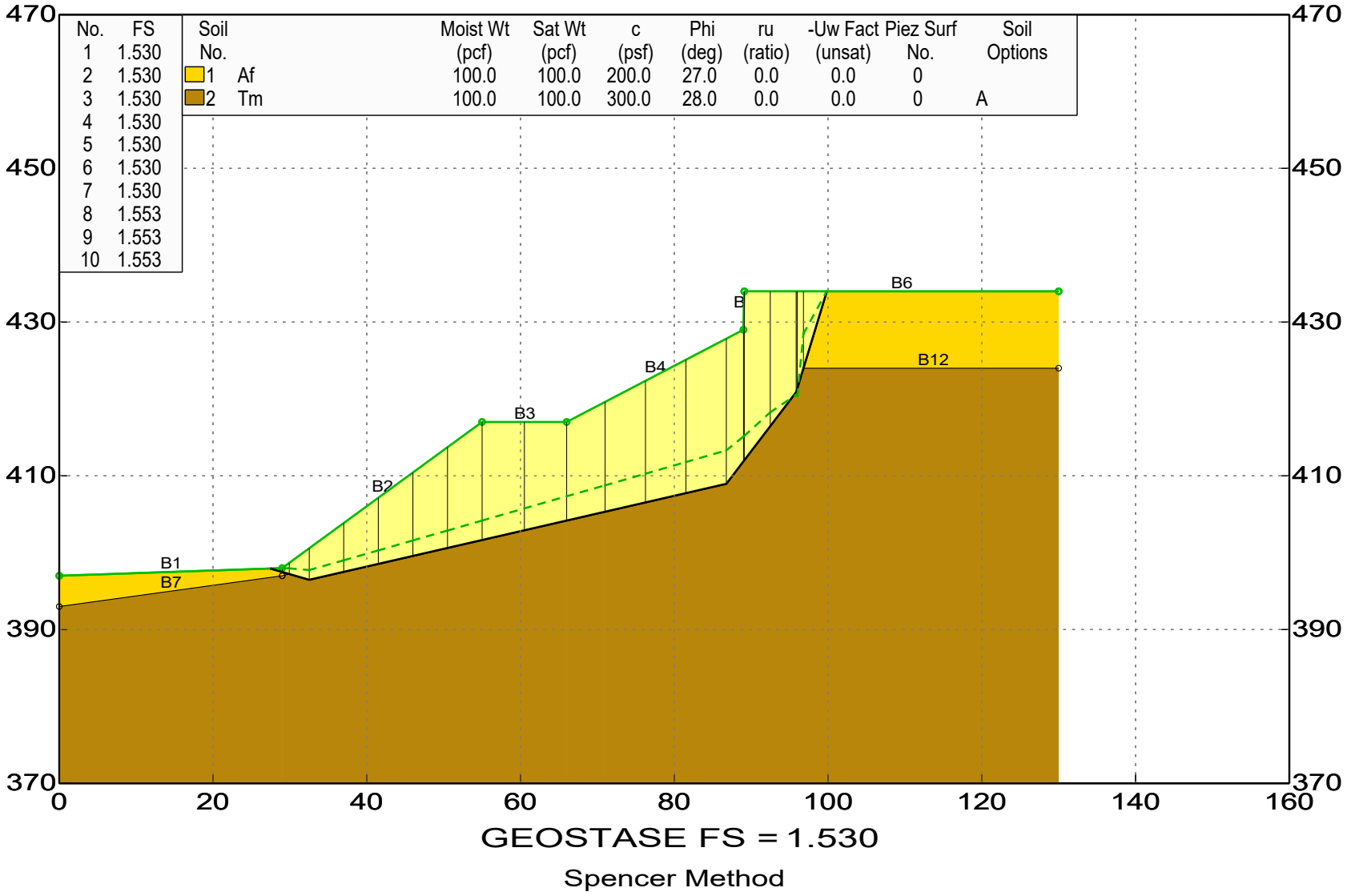
Project No.: 25093-01

Project Name: 1-5 Polaris Way, Aliso Viejo

Lennar, 1-5 Polaris Way, Aliso Viejo
B-B' - Global Stability of Design Slope -Block Search

SA Geotechnical / RS

\03.gsd



GEOSTASE FS = 1.530

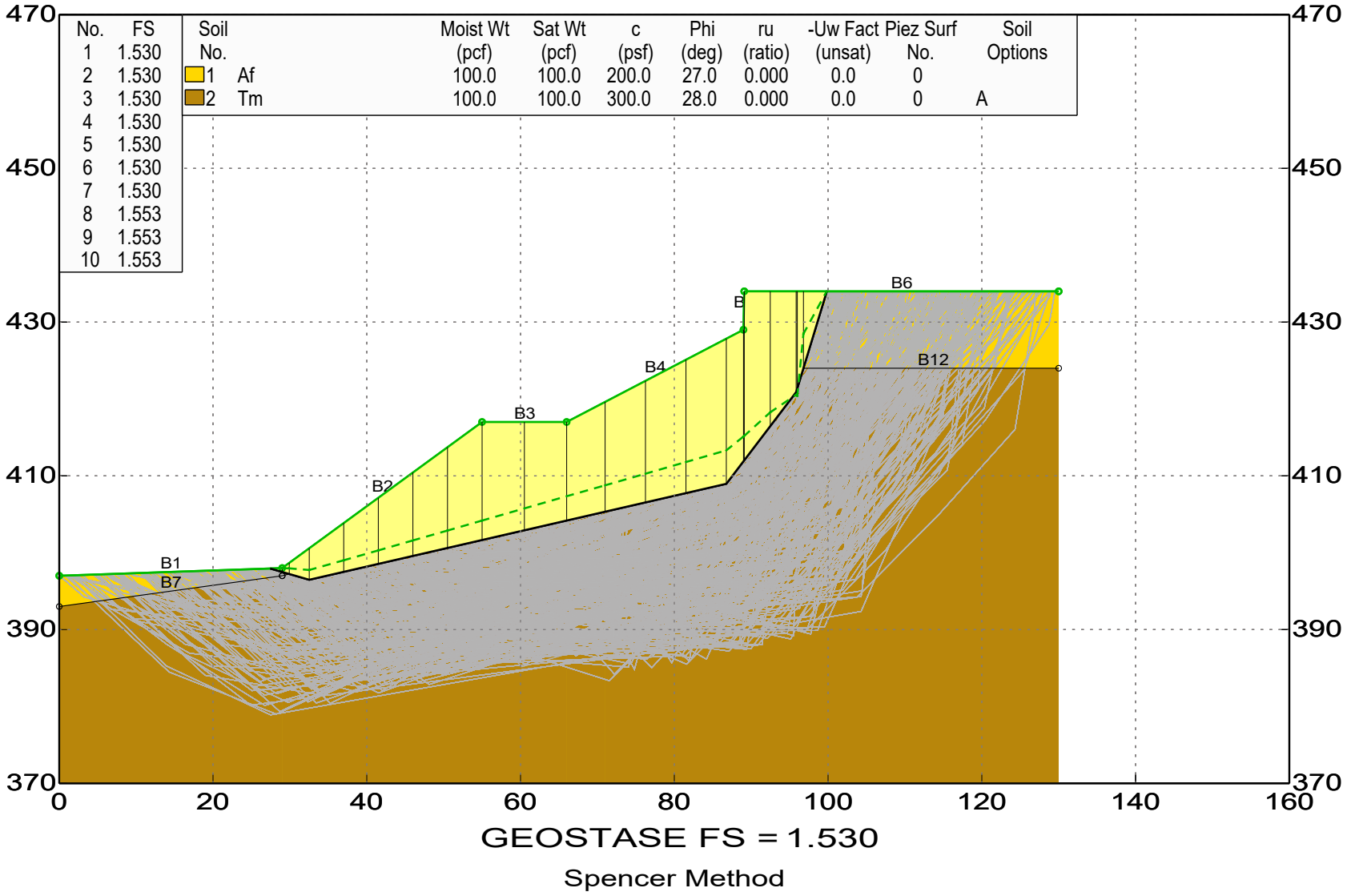
Spencer Method

GEOSTASE
Slope Stability Analysis

Lennar, 1-5 Polaris Way, Aliso Viejo
B-B' - Global Stability of Design Slope -Block Search

SA Geotechnical / RS

\03.gsd



GEOSTASE FS = 1.530

Spencer Method

GEOSTASE
Slope Stability Analysis

*** GEOSTASE(R) ***

** GEOSTASE(R) (c)Copyright by Garry H. Gregory, Ph.D.,P.E.,S.E.,BC.GE **

** Current Version 4.30.32-Double Precision, Sept 2023 **

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****

SLOPE STABILITY ANALYSIS SOFTWARE (2D & 3D)

Simplified Bishop, Simplified Janbu, or General Equilibrium (GE) Options.

(Spencer, Morgenstern-Price, USACE, and Lowe & Karafiath)

Including Pier/Pile, Planar Reinf, Nail, Tieback, Line Loads

Applied Forces, Fiber-Reinforced Soil (FRS), Distributed Loads

Nonlinear Undrained Shear Strength, Curved Strength Envelope,

Anisotropic Strengths, Water Surfaces, 3-Stage Rapid Drawdown

2- or 3-Stage Pseudo-Static & Simplified Newmark Seismic Analyses.

****

Analysis Date: 3/ 3/ 2026

Analysis Time:

Analysis By: SA Geotechnical / RS

Input File Name: C:\Users\mreza\SA Geotechnical, Inc\SA Geotechnical - Projects\2025\25093-01 Lennar - Polaris Place, Aliso Viejo\Engineering\Sted\B-B'\03.gsd

Output File Name: C:\Users\mreza\SA Geotechnical, Inc\SA Geotechnical - Projects\2025\25093-01 Lennar - Polaris Place, Aliso Viejo\Engineering\Sted\B-B'\03.OUT

Unit System: English

PROJECT: Lennar, 1-5 Polaris Way, Aliso Viejo

DESCRIPTION: B-B' - Global Stability of Design Slope -Block Search

PROFILE BOUNDARY DATA

6 Surface Boundaries
12 Total Boundaries

Boundary No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Soil Type Below Bnd
1	0.000	397.000	29.000	398.000	1
2	29.000	398.000	55.000	417.000	1
3	55.000	417.000	66.000	417.000	1
4	66.000	417.000	89.000	429.000	1
5	89.000	429.000	89.100	434.000	1
6	89.100	434.000	130.000	434.000	1
7	0.000	393.000	29.000	397.000	2
8	29.000	397.000	37.000	399.000	2
9	37.000	399.000	66.000	409.000	2
10	66.000	409.000	71.000	411.000	2
11	71.000	411.000	96.000	424.000	2
12	96.000	424.000	130.000	424.000	2

User Specified X-Origin = 0.000(ft)

User Specified Y-Origin = 370.000(ft)

SURFACE BOUNDARY POINT COORDINATES
(Information Only-Not Used Directly)

Point No.	X-COORD	Y-COORD
1	0.000	397.000
2	29.000	398.000
3	55.000	417.000
4	66.000	417.000
5	89.000	429.000
6	89.100	434.000
7	130.000	434.000

SUBSURFACE BOUNDARY POINT COORDINATES
(Information Only-Not Used Directly)

Point No.	X-COORD	Y-COORD
8	0.000	393.000
9	29.000	397.000
10	37.000	399.000
11	66.000	409.000
12	71.000	411.000
13	96.000	424.000
14	130.000	424.000

PROFILE BOUNDARY SLOPES AND ANGLES

BND No.	Slope	Angle(Deg.)
1	0.034	1.975
2	0.731	36.158
3	0.000	0.000
4	0.522	27.553
5	50.000	88.854
6	0.000	0.000
7	0.138	7.853
8	0.250	14.036
9	0.345	19.026
10	0.400	21.801
11	0.520	27.474
12	0.000	0.000

MOHR-COULOMB SOIL PARAMETERS

2 Type(s) of Soil Defined

Soil Number and Description	Moist Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Ratio(ru)	(-Uw) Fact.	Water Surface No.	Water Option
1 Af	100.0	100.0	200.00	27.00	0.000	0.0	0	0
2 Tm	100.0	100.0	300.00	28.00	0.000	0.0	0	0

ANISOTROPIC STRENGTH PARAMETERS

1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	300.00	28.00
2	30.0	150.00	18.00
3	90.0	300.00	28.00

A Non-Circular Zone Search Has Been Selected For Analysis
Using Random Generation Within Specified Zones.

2 Zones Defined For Generation Of Non-Circular Surfaces

3000 Trial Surfaces Have Been Generated.

Length Of Line Segments For Active And Passive Portions Of
Non-Circular Zone Search = 15.00(ft)

Zone No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Height (ft)
1	24.00	387.00	70.00	396.00	18.00
2	71.00	396.00	105.00	404.00	26.00

The Spencer Method Was Selected for FS Analysis.

Selected fx function = Constant (1.0)

SELECTED CONVERGENCE PARAMETERS FOR SPENCER METHOD:
Initial estimate of FS = 1.500
FS tolerance = 0.000001000

```
Initial estimate of theta(deg) =  15.00
Theta tolerance(radians) =  0.0001000
Minimum theta(deg) = -45.00 ; Maximum theta(deg) =  45.00
Theta convergence Step Factor = 5000.00
Maximum number of iterations = 50
Allowable negative side force = -1000.0(lbs)
Maximum force imbalance  = 100.000000(lbs)
Maximum moment imbalance = 100.000000 (ft/lbs)

Selected Lambda Coefficient =  1.00

Specified Tension Crack Water Depth Factor =   0.000

Total Number of Trial Surfaces Attempted = 3000

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 50 Iterations.


Number of Trial Surfaces with Non-Converged FS = 484

Number of Trial Surfaces With Valid FS = 2516


Percentage of Trial Surfaces With Non-Converged and/or
Non-Valid FS Solutions of the Total Attempted =  16.1 %

Statistical Data On All Valid FS Values:
   FS Max =  16.591   FS Min =   1.530   FS Ave =   3.478
   Standard Deviation =    1.847   Coefficient of Variation =   53.09 %


Critical Surface is Sequence Number     205 of Those Analyzed.


*****BEGINNING OF DETAILED GEOSTASE OUTPUT FOR CRITICAL SURFACE FROM A SEARCH*****


  Iter.     Theta         FS         FS
```

No.	(deg) (fx=1.0)	(Moment)	(Force)	Lambda	Delta FS
1	15.0000	1.579305	1.512116	0.268	0.6718924E-01
2	19.9500	1.503891	1.536456	0.363	0.3256494E-01
3	18.3344	1.533788	1.528243	0.331	0.5545156E-02
4	18.5697	1.529811	1.529422	0.336	0.3895399E-03
5	18.5875	1.529506	1.529511	0.336	0.4970182E-05
6	18.5873	1.529510	1.529510	0.336	0.7940740E-08

Factor Of Safety For The Preceding Specified Surface = 1.530
Theta (fx = 1.0) = 18.59 Deg Lambda = 0.336

The Spencer Method Has Been Selected For Analysis.

Selected fx function = Constant (1.0)

SELECTED CONVERGENCE PARAMETERS FOR ANALYSIS METHOD:
Initial estimate of FS = 1.500
FS tolerance = 0.000001000
Initial estimate of theta(deg) = 15.00
Theta tolerance(radians) = 0.0001000
Minimum theta(deg) = -45.00 ; Maximum theta(deg) = 45.00
Theta convergence Step Factor = 5000.00
Maximum number of iterations = 50
Maximum force imbalance = 100.000000(lbs)
Maximum moment imbalance(if Applicable) = 100.000000 (ft/lbs)

Selected Lambda Coefficient = 1.00

Tension Crack Water Force = 0.00(lbs)

Specified Tension Crack Water Depth Factor = 0.000 (0.00% Filled)

Depth of Tension Crack (zo) at Side of Last Slice = 0.000(ft)

Depth of Water in Tension Crack = 0.000(ft)

Theoretical Tension Crack Depth = 6.527(ft)

NOTE: In Table 1 following, when a tension crack with water is present on the first slice (right facing slope) or on the last slice (left facing slope), the "side force" in the tension crack is set equal to the water pressure resultant.

*** Table 1 - Line of Thrust(if applicable) and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	h/H	Side Force (lbs)	fx	Force Angle (Deg)	Vert. Shear Force(lbs)
1	29.00	397.98	0.967	372.77	1.000	18.59	118.8
2	29.92	397.96	0.505	655.29	1.000	18.59	208.9
3	32.51	397.75	0.310	2203.34	1.000	18.59	702.3
4	37.00	399.02	0.237	2626.19	1.000	18.59	837.1
5	41.50	400.30	0.204	3032.24	1.000	18.59	966.5
6	46.00	401.58	0.185	3420.55	1.000	18.59	1090.3
7	50.50	402.88	0.173	3791.12	1.000	18.59	1208.4
8	55.00	404.18	0.165	4143.94	1.000	18.59	1320.9
9	60.50	405.77	0.202	4570.40	1.000	18.59	1456.8
10	66.00	407.34	0.246	5009.04	1.000	18.59	1596.6
11	71.00	408.77	0.241	5406.94	1.000	18.59	1723.5
12	76.26	410.28	0.236	5811.50	1.000	18.59	1852.4
13	81.51	411.79	0.233	6201.94	1.000	18.59	1976.9
14	86.77	413.31	0.231	6578.28	1.000	18.59	2096.8
15	89.00	415.10	0.188	4988.86	1.000	18.59	1590.2
16	89.10	415.19	0.144	4908.73	1.000	18.59	1564.7
17	92.48	418.27	0.103	2159.20	1.000	18.59	688.2
18	95.86	420.55	0.000-	266.89	1.000	18.59	85.1
19	96.00	420.40	0.000-	197.04	1.000	18.59	62.8
20	96.80	428.49	0.449	-74.37	1.000	18.59	-23.7
21	99.83	434.00	0.000	0.00	1.000	18.59	0.0

NOTE: A value of 0.000- for h/H indicates that the line of thrust is at or below the lower boundary of the sliding mass. A value of 1.000+ for h/H indicates that

the line of thrust is at or above the upper boundary of the sliding mass.

Table 2 - Geometry Data on the 21 Slices

Slice No.	Width (ft)	Height (ft)	X-Cntr (ft)	Y-Cntr-Base (ft)	Y-Cntr-Top (ft)	Alpha (deg)	Beta (deg)	Base Length (ft)
1	1.56	0.25	28.22	397.72	397.97	-16.11	1.97	1.62
2	0.92	0.97	29.46	397.36	398.34	-16.11	36.16	0.96
3	2.59	2.76	31.21	396.86	399.62	-16.11	36.16	2.70
4	4.49	5.21	34.75	397.00	402.21	12.95	36.16	4.61
5	4.50	7.46	39.25	398.03	405.49	12.95	36.16	4.62
6	4.50	9.71	43.75	399.07	408.78	12.95	36.16	4.62
7	4.50	11.97	48.25	400.10	412.07	12.95	36.16	4.62
8	4.50	14.22	52.75	401.14	415.36	12.95	36.16	4.62
9	5.50	14.71	57.75	402.29	417.00	12.95	0.00	5.64
10	5.50	13.45	63.25	403.55	417.00	12.95	0.00	5.64
11	5.00	13.55	68.50	404.76	418.30	12.95	27.55	5.13
12	5.26	15.04	73.63	405.94	420.98	12.95	27.55	5.39
13	5.26	16.58	78.88	407.14	423.72	12.95	27.55	5.39
14	5.26	18.11	84.14	408.35	426.46	12.95	27.55	5.39
15	2.23	18.00	87.88	410.42	428.42	52.70	27.55	3.68
16	0.10	19.55	89.05	411.95	431.50	52.70	88.85	0.17
17	3.38	19.76	90.79	414.24	434.00	52.70	0.00	5.58
18	3.38	15.33	94.17	418.67	434.00	52.70	0.00	5.58
19	0.14	12.88	95.93	421.12	434.00	73.15	0.00	0.49
20	0.80	11.32	96.40	422.68	434.00	73.15	0.00	2.76
21	3.03	5.00	98.31	429.00	434.00	73.15	0.00	10.45

Table 2A - Coordinates of Slice Points Defining the Slip Surface

Point No.	X-Pt (ft)	Y-Pt (ft)

1	27.440403	397.946221
2	29.000000	397.495685
3	29.919844	397.229961
4	32.509888	396.481750
5	37.000000	397.514142
6	41.500000	398.548806
7	46.000000	399.583471
8	50.500000	400.618136
9	55.000000	401.652801
10	60.500000	402.917391
11	66.000000	404.181981
12	71.000000	405.331608
13	76.256470	406.540205
14	81.512939	407.748801
15	86.769409	408.957397
16	89.000000	411.885867
17	89.100000	412.017154
18	92.479224	416.453627
19	95.858449	420.890100
20	96.000000	421.357537
21	96.800202	424.000000
22	99.828445	434.000000



Table 3 - Force and Pore Pressure Data On The 21 Slices (Excluding Reinforcement)

Slice No.	Weight (lbs)	Ubeta Force Top (lbs)	Ubeta Stress Top (psf)	Ualpha Force Bot (lbs)	Pore Pressure (psf)	-Uw (psf)	Earthquake Force Horiz (lbs)	Earthquake Force Vert (lbs)	Distributed Load (psf)
1	39.3	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
2	89.5	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
3	715.6	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
4	2338.3	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
5	3356.5	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
6	4370.7	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
7	5384.9	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00

8	6399.1	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
9	8093.2	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
10	7397.7	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
11	6773.8	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
12	7907.9	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
13	8714.2	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
14	9520.4	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
15	4014.3	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
16	195.5	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
17	6678.9	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
18	5179.7	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
19	182.3	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
20	905.9	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00
21	1514.1	0.0	0.0	0.0	0.00	0.00	0.0	0.0	0.00

TOTAL WEIGHT OF SLIDING MASS = 89771.76(lbs)

EFFECTIVE WEIGHT OF SLIDING MASS = 89771.76(lbs)

TOTAL AREA OF SLIDING MASS = 897.72(ft2)

TABLE 4 - SOIL STRENGTH & SOIL OPTIONS DATA ON THE 21 SLICES

Slice No.	Soil Type	Cohesion (psf)	Phi(Deg)	Options
1	1	200.00	27.00	
2	1	200.00	27.00	
3	2	300.00	28.00	A
4	2	150.00	18.00	A
5	2	150.00	18.00	A
6	2	150.00	18.00	A
7	2	150.00	18.00	A
8	2	150.00	18.00	A
9	2	150.00	18.00	A
10	2	150.00	18.00	A
11	2	150.00	18.00	A
12	2	150.00	18.00	A

13	2	150.00	18.00	A
14	2	150.00	18.00	A
15	2	300.00	28.00	A
16	2	300.00	28.00	A
17	2	300.00	28.00	A
18	2	300.00	28.00	A
19	2	300.00	28.00	A
20	2	300.00	28.00	A
21	1	200.00	27.00	

SOIL OPTIONS:
A = ANISOTROPIC SHEAR STRENGTH
C = CURVED STRENGTH ENVELOPE (TANGENT PHI & C)
F = FIBER-REINFORCED SOIL (FRS)
M = INDEPENDENT MULTI-STAGE SHEAR STRENGTH
N = NONLINEAR UNDRAINED SHEAR STRENGTH
R = RAPID DRAWDOWN OR RAPID LOADING (SEISMIC) SHEAR STRENGTH
S = SHANSEP UNDRAINED SHEAR STRENGTH
NOTE: Phi and C in Table 4 are modified values based on specified Soil Options (if any).

TABLE 5 - Total Base Stress Data on the 21 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Total Normal/Vert. Stress Ratio
1	-16.11	28.22	1.62	154.00	25.22	6.107
2	-16.11	29.46	0.96	257.81	97.33	2.649
3	-16.11	31.21	2.70	581.88	276.27	2.106
4	12.95	34.75	4.61	503.63	520.76	0.967
5	12.95	39.25	4.62	717.08	745.89	0.961
6	12.95	43.75	4.62	930.76	971.27	0.958
7	12.95	48.25	4.62	1144.45	1196.65	0.956
8	12.95	52.75	4.62	1358.14	1422.03	0.955
9	12.95	57.75	5.64	1405.03	1471.49	0.955

10	12.95	63.25	5.64	1285.13	1345.03	0.955
11	12.95	68.50	5.13	1294.35	1354.76	0.955
12	12.95	73.63	5.39	1436.24	1504.40	0.955
13	12.95	78.88	5.39	1581.67	1657.80	0.954
14	12.95	84.14	5.39	1727.10	1811.19	0.954
15	52.70	87.88	3.68	902.92	1799.65	0.502
16	52.70	89.05	0.17	990.07	1954.85	0.506
17	52.70	90.79	5.58	1002.20	1976.46	0.507
18	52.70	94.17	5.58	753.10	1532.81	0.491
19	73.15	95.93	0.49	224.69	1287.62	0.174
20	73.15	96.40	2.76	175.19	1132.12	0.155
21	73.15	98.31	10.45	36.20	500.00	0.072

TABLE 5A - Total Base Force Data on the 21 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Force (lbs)	Total Vert. Force (lbs)	Total Normal/Vert. Force Ratio
1	-16.11	28.22	1.62	249.99	39.33	6.357
2	-16.11	29.46	0.96	246.85	89.53	2.757
3	-16.11	31.21	2.70	1568.72	715.55	2.192
4	12.95	34.75	4.61	2320.35	2338.27	0.992
5	12.95	39.25	4.62	3311.05	3356.51	0.986
6	12.95	43.75	4.62	4297.73	4370.72	0.983
7	12.95	48.25	4.62	5284.40	5384.93	0.981
8	12.95	52.75	4.62	6271.08	6399.14	0.980
9	12.95	57.75	5.64	7929.30	8093.20	0.980
10	12.95	63.25	5.64	7252.65	7397.67	0.980
11	12.95	68.50	5.13	6640.62	6773.78	0.980
12	12.95	73.63	5.39	7746.52	7907.86	0.980
13	12.95	78.88	5.39	8530.92	8714.15	0.979
14	12.95	84.14	5.39	9315.33	9520.45	0.978
15	52.70	87.88	3.68	3323.87	4014.28	0.828
16	52.70	89.05	0.17	163.39	195.48	0.836
17	52.70	90.79	5.58	5589.14	6678.90	0.837
18	52.70	94.17	5.58	4199.93	5179.72	0.811

19	73.15	95.93	0.49	109.74	182.26	0.602
20	73.15	96.40	2.76	483.70	905.93	0.534
21	73.15	98.31	10.45	378.24	1514.12	0.250

TABLE 6 - Effective and Base Shear Stress Data on the 21 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Effective Normal Stress (psf)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	-16.11	28.22	1.62	154.00	278.46	182.06
2	-16.11	29.46	0.96	257.81	331.36	216.65
3	-16.11	31.21	2.70	581.88	609.39	398.42
4	12.95	34.75	4.61	503.63	313.64	205.06
5	12.95	39.25	4.62	717.08	382.99	250.40
6	12.95	43.75	4.62	930.76	452.42	295.80
7	12.95	48.25	4.62	1144.45	521.85	341.19
8	12.95	52.75	4.62	1358.14	591.29	386.58
9	12.95	57.75	5.64	1405.03	606.52	396.55
10	12.95	63.25	5.64	1285.13	567.56	371.08
11	12.95	68.50	5.13	1294.35	570.56	373.03
12	12.95	73.63	5.39	1436.24	616.66	403.18
13	12.95	78.88	5.39	1581.67	663.91	434.07
14	12.95	84.14	5.39	1727.10	711.17	464.97
15	52.70	87.88	3.68	902.92	780.09	510.03
16	52.70	89.05	0.17	990.07	826.43	540.32
17	52.70	90.79	5.58	1002.20	832.88	544.54
18	52.70	94.17	5.58	753.10	700.43	457.94
19	73.15	95.93	0.49	224.69	419.47	274.25
20	73.15	96.40	2.76	175.19	393.15	257.04
21	73.15	98.31	10.45	36.20	218.45	142.82

TABLE 6A - Effective and Base Shear Force Data on the 21 Slices

Slice	Alpha	X-Coord.	Base	Effective	Available	Mobilized

No. *	(deg)	Slice Cntr (ft)	Leng. (ft)	Normal Force (lbs)	Shear Force (lbs)	Shear Force (lbs)
1	-16.11	28.22	1.62	249.99	452.05	295.55
2	-16.11	29.46	0.96	246.85	317.27	207.43
3	-16.11	31.21	2.70	1568.72	1642.89	1074.13
4	12.95	34.75	4.61	2320.35	1445.02	944.76
5	12.95	39.25	4.62	3311.05	1768.44	1156.21
6	12.95	43.75	4.62	4297.73	2089.03	1365.82
7	12.95	48.25	4.62	5284.40	2409.62	1575.42
8	12.95	52.75	4.62	6271.08	2730.21	1785.02
9	12.95	57.75	5.64	7929.30	3422.91	2237.91
10	12.95	63.25	5.64	7252.65	3203.06	2094.17
11	12.95	68.50	5.13	6640.62	2927.24	1913.84
12	12.95	73.63	5.39	7746.52	3326.04	2174.58
13	12.95	78.88	5.39	8530.92	3580.91	2341.21
14	12.95	84.14	5.39	9315.33	3835.78	2507.85
15	52.70	87.88	3.68	3323.87	2871.70	1877.53
16	52.70	89.05	0.17	163.39	136.39	89.17
17	52.70	90.79	5.58	5589.14	4644.86	3036.83
18	52.70	94.17	5.58	4199.93	3906.21	2553.89
19	73.15	95.93	0.49	109.74	204.87	133.94
20	73.15	96.40	2.76	483.70	1085.48	709.69
21	73.15	98.31	10.45	378.24	2282.41	1492.25

Average Effective Normal Stress = 950.5145(psf)
Average Available Shear Strength = 538.5657(psf)
Total Length of Failure Surface = 89.6499(ft)

INPUT PARAMETERS FOR SIMPLIFIED 3D ANALYSIS

Composite Phi (deg)= 29.536
3D Depth (Vertical difference between ends of Slip Surface) = 36.05(ft)
L/D(1): L = 36.05(ft) L/D(2): L = 72.11(ft) L/D(3): L = 108.16(ft)
L/D(4): L = 144.22(ft) L/D(5): L = 180.27(ft) L/D(6): L = 216.32(ft)

SUM OF MOMENTS = -0.371324E-04 (ft/lbs);Imbalance (Fraction of Total Weight) = -0.4136314E-09
SUM OF FORCES = 0.247563E-07 (lbs);Imbalance (Fraction of Total Weight) = 0.2757692E-12

Sum of Available Shear Forces =   48282.36(lbs)

Sum of Mobilized Shear Forces =   31567.21(lbs)

FS Balance Check: FS = 1.529510

**** END OF GEOSTASE OUTPUT ****

Lennar, 1-5 Polaris Way, Aliso Viejo
B-B' - Global Stability of Design Slope -Block Search - Pseudo Static

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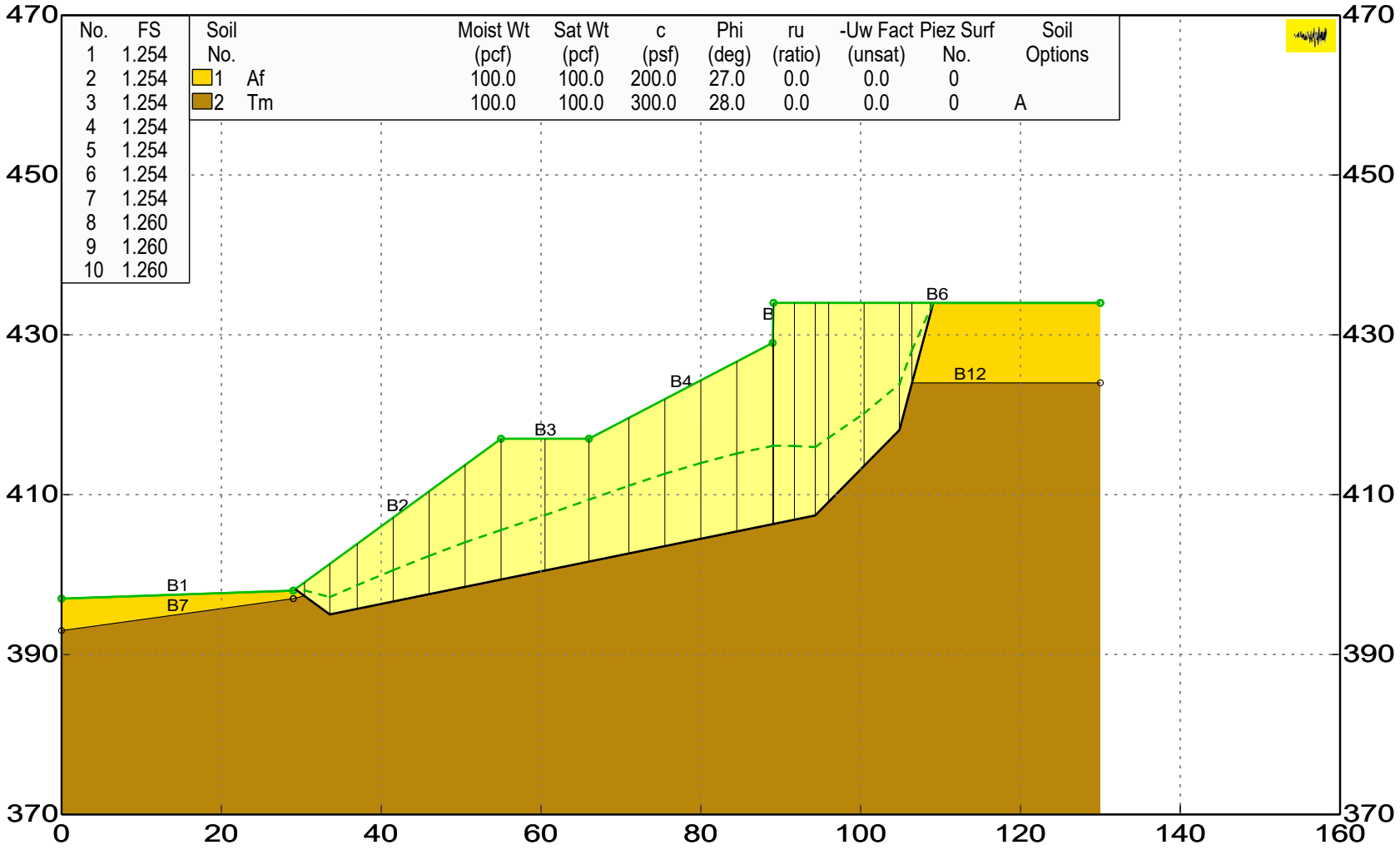
\03p.gsd



GEOSTASE FS = 1.254

Spencer Method

kh = 0.21000

GEOSTASE Slope Stability Analysis

PLATE C.1

Lennar, 1-5 Polaris Way, Aliso Viejo
B-B' - Global Stability of Design Slope -Block Search - Pseudo Static

SA Geotechnical / RS

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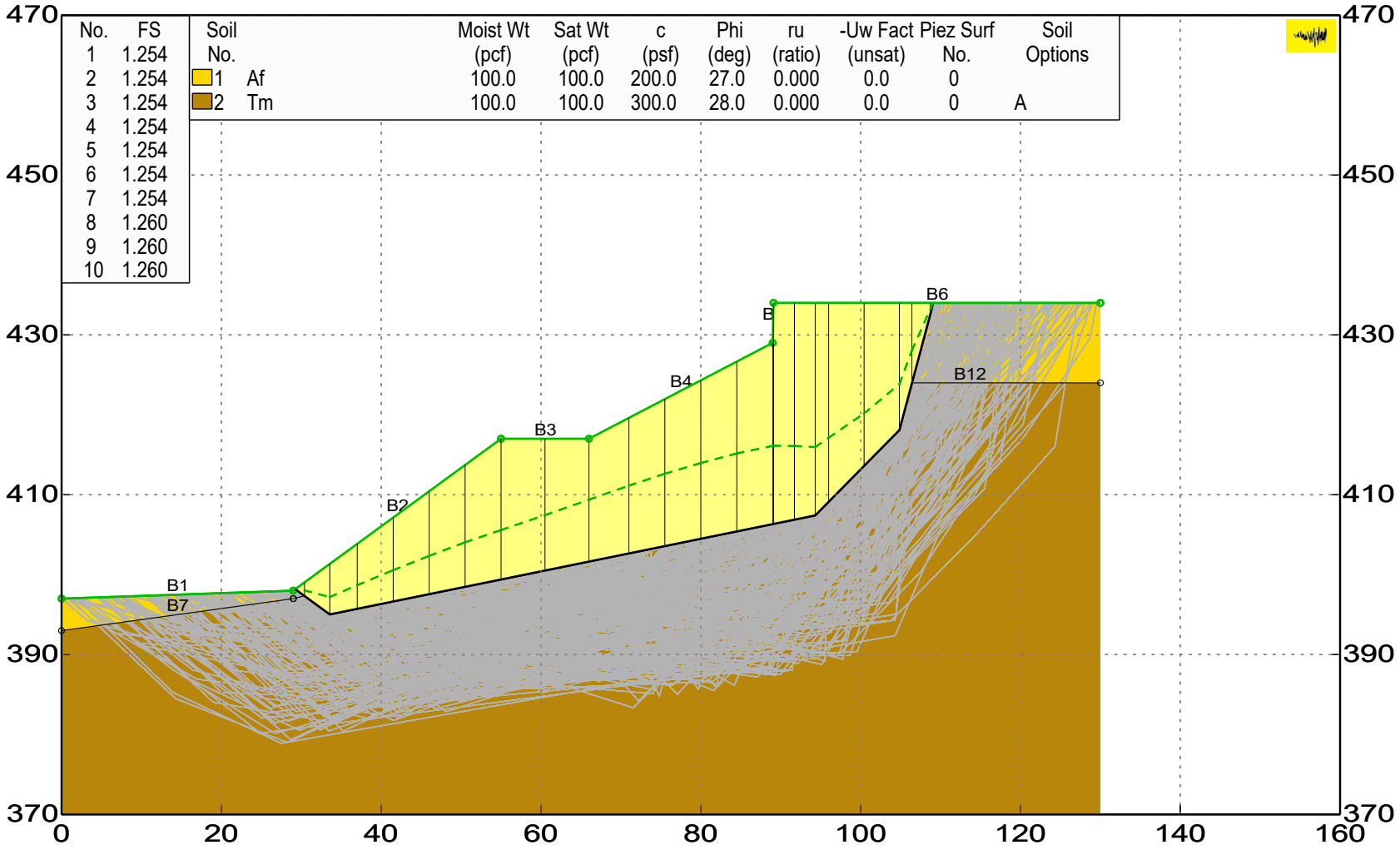

GEOSTASE FS = 1.254

Spencer Method

kh = 0.21000

GEOSTASE Slope Stability Analysis

PLATE C.2

*** GEOSTASE(R) ***

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** Current Version 4.30.32-Double Precision, Sept 2023 **

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SLOPE STABILITY ANALYSIS SOFTWARE (2D & 3D)

Simplified Bishop, Simplified Janbu, or General Equilibrium (GE) Options.

(Spencer, Morgenstern-Price, USACE, and Lowe & Karafiath)

Including Pier/Pile, Planar Reinf, Nail, Tieback, Line Loads

Applied Forces, Fiber-Reinforced Soil (FRS), Distributed Loads

Nonlinear Undrained Shear Strength, Curved Strength Envelope,

Anisotropic Strengths, Water Surfaces, 3-Stage Rapid Drawdown

2- or 3-Stage Pseudo-Static & Simplified Newmark Seismic Analyses.

****

Analysis Date: 3/ 3/ 2026

Analysis Time:

Analysis By: SA Geotechnical / RS

Input File Name: C:\Users\mreza\SA Geotechnical, Inc\SA Geotechnical - Projects\2025\25093-01 Lennar - Polaris Place, Aliso Viejo\Engineering\Sted\B-B'\03p.gsd

Output File Name: C:\Users\mreza\SA Geotechnical, Inc\SA Geotechnical - Projects\2025\25093-01 Lennar - Polaris Place, Aliso Viejo\Engineering\Sted\B-B'\03p.OUT

Unit System: English

PROJECT: Lennar, 1-5 Polaris Way, Aliso Viejo

DESCRIPTION: B-B' - Global Stability of Design Slope -Block Search - Pseudo Static

PROFILE BOUNDARY DATA

6 Surface Boundaries
12 Total Boundaries

Boundary No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Soil Type Below Bnd
1	0.000	397.000	29.000	398.000	1
2	29.000	398.000	55.000	417.000	1
3	55.000	417.000	66.000	417.000	1
4	66.000	417.000	89.000	429.000	1
5	89.000	429.000	89.100	434.000	1
6	89.100	434.000	130.000	434.000	1
7	0.000	393.000	29.000	397.000	2
8	29.000	397.000	37.000	399.000	2
9	37.000	399.000	66.000	409.000	2
10	66.000	409.000	71.000	411.000	2
11	71.000	411.000	96.000	424.000	2
12	96.000	424.000	130.000	424.000	2

User Specified X-Origin = 0.000(ft)

User Specified Y-Origin = 370.000(ft)

SURFACE BOUNDARY POINT COORDINATES
(Information Only-Not Used Directly)

Point No.	X-COORD	Y-COORD
1	0.000	397.000
2	29.000	398.000
3	55.000	417.000
4	66.000	417.000
5	89.000	429.000
6	89.100	434.000
7	130.000	434.000

SUBSURFACE BOUNDARY POINT COORDINATES
(Information Only-Not Used Directly)

Point No.	X-COORD	Y-COORD
8	0.000	393.000
9	29.000	397.000
10	37.000	399.000
11	66.000	409.000
12	71.000	411.000
13	96.000	424.000
14	130.000	424.000

PROFILE BOUNDARY SLOPES AND ANGLES

BND No.	Slope	Angle(Deg.)
1	0.034	1.975
2	0.731	36.158
3	0.000	0.000
4	0.522	27.553
5	50.000	88.854
6	0.000	0.000
7	0.138	7.853
8	0.250	14.036
9	0.345	19.026
10	0.400	21.801
11	0.520	27.474
12	0.000	0.000

MOHR-COULOMB SOIL PARAMETERS

2 Type(s) of Soil Defined

Soil Number and Description	Moist Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Ratio(ru)	(-Uw) Fact.	Water Surface No.	Water Option
1 Af	100.0	100.0	200.00	27.00	0.000	0.0	0	0
2 Tm	100.0	100.0	300.00	28.00	0.000	0.0	0	0

ANISOTROPIC STRENGTH PARAMETERS

1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	300.00	28.00
2	30.0	150.00	18.00
3	90.0	300.00	28.00

SEISMIC (EARTHQUAKE) DATA

Specified Peak Ground Acceleration Coefficient (PGA) = 0.210(g)
Default Velocity = 1.050(ft) per second
Specified Horizontal Earthquake Coefficient (kh) = 0.21000(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
(NOTE:Input Velocity = 0.0 will result in default Peak
Velocity = 2 times(PGA) times 2.5 fps or 0.762 mps)
Specified Seismic Pore-Pressure Factor = 0.000
Horizontal Seismic Force is Applied at Center of Gravity of Slices

A Non-Circular Zone Search Has Been Selected For Analysis
Using Random Generation Within Specified Zones.

2 Zones Defined For Generation Of Non-Circular Surfaces

3000 Trial Surfaces Have Been Generated.

Length Of Line Segments For Active And Passive Portions Of
Non-Circular Zone Search = 15.00(ft)

Zone	X - 1	Y - 1	X - 2	Y - 2	Height

No.	(ft)	(ft)	(ft)	(ft)	(ft)
1	24.00	387.00	70.00	396.00	18.00
2	71.00	396.00	105.00	404.00	26.00

The Spencer Method Was Selected for FS Analysis.

Selected fx function = Constant (1.0)

SELECTED CONVERGENCE PARAMETERS FOR SPENCER METHOD:
Initial estimate of FS = 1.500
FS tolerance = 0.000001000
Initial estimate of theta(deg) = 15.00
Theta tolerance(radians) = 0.0001000
Minimum theta(deg) = -45.00 ; Maximum theta(deg) = 45.00
Theta convergence Step Factor = 5000.00
Maximum number of iterations = 50
Allowable negative side force = -1000.0(lbs)
Maximum force imbalance = 100.000000(lbs)
Maximum moment imbalance = 100.000000 (ft/lbs)

Selected Lambda Coefficient = 1.00

Specified Tension Crack Water Depth Factor = 0.000

Total Number of Trial Surfaces Attempted = 3000

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 50 Iterations.

Number of Trial Surfaces with Non-Converged FS = 1255

Number of Trial Surfaces With Valid FS = 1745

Percentage of Trial Surfaces With Non-Converged and/or
Non-Valid FS Solutions of the Total Attempted = 41.8 %

Statistical Data On All Valid FS Values:

FS Max = 12.512 FS Min = 1.254 FS Ave = 2.298
Standard Deviation = 1.089 Coefficient of Variation = 47.40 %

Critical Surface is Sequence Number 226 of Those Analyzed.

*****BEGINNING OF DETAILED GEOSTASE OUTPUT FOR CRITICAL SURFACE FROM A SEARCH*****

Iter. No.	Theta (deg) (fx=1.0)	FS (Moment)	FS (Force)	Lambda	Delta FS
1	15.0000	1.262001	1.061142	0.268	0.2008587E+00
2	19.9500	1.288210	1.133967	0.363	0.1542430E+00
3	36.3254	0.000000	0.791484	0.735	0.7914835E+00
4	22.6202	1.287745	1.191967	0.417	0.9577771E-01
5	24.1021	1.271746	1.231968	0.447	0.3977755E-01
6	25.1545	1.241462	1.264419	0.470	0.2295677E-01
7	24.7694	1.255644	1.252128	0.461	0.3516379E-02
8	24.8207	1.254018	1.253734	0.463	0.2839759E-03
9	24.8252	1.253872	1.253875	0.463	0.3669885E-05
10	24.8251	1.253874	1.253874	0.463	0.2041870E-07

Factor Of Safety For The Preceding Specified Surface = 1.254
Theta (fx = 1.0) = 24.83 Deg Lambda = 0.463

The Spencer Method Has Been Selected For Analysis.

Selected fx function = Constant (1.0)

SELECTED CONVERGENCE PARAMETERS FOR ANALYSIS METHOD:
Initial estimate of FS = 1.500
FS tolerance = 0.000001000
Initial estimate of theta(deg) = 15.00
Theta tolerance(radians) = 0.0001000

Minimum theta(deg) = -45.00 ; Maximum theta(deg) = 45.00
Theta convergence Step Factor = 5000.00
Maximum number of iterations = 50
Maximum force imbalance = 100.000000(lbs)
Maximum moment imbalance(if Applicable) = 100.000000 (ft/lbs)

Selected Lambda Coefficient = 1.00

Tension Crack Water Force =   0.00(lbs)

Specified Tension Crack Water Depth Factor = 0.000 (0.00% Filled)

Depth of Tension Crack (zo) at Side of Last Slice =   0.000(ft)

Depth of Water in Tension Crack =   0.000(ft)

Theoretical Tension Crack Depth =   6.527(ft)

NOTE: In Table 1 following, when a tension crack with water is present on the first slice (right facing slope) or on the last slice (left facing slope), the "side force" in the tension crack is set equal to the water pressure resultant.

*** Table 1 - Line of Thrust(if applicable) and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	h/H	Side Force (lbs)	fx	Force Angle (Deg)	Vert. Shear Force(lbs)
1	30.40	398.03	0.408	2355.85	1.000	24.83	989.1
2	33.57	397.20	0.344	19839.95	1.000	24.83	8329.8
3	37.00	398.68	0.364	19857.95	1.000	24.83	8337.4
4	41.50	400.56	0.373	19714.14	1.000	24.83	8277.0
5	46.00	402.35	0.372	19380.48	1.000	24.83	8136.9
6	50.50	404.04	0.365	18856.94	1.000	24.83	7917.1
7	55.00	405.59	0.352	18143.54	1.000	24.83	7617.6
8	60.50	407.43	0.419	17210.50	1.000	24.83	7225.8
9	66.00	409.38	0.504	16387.29	1.000	24.83	6880.2

10	71.00	411.14	0.500	15618.15	1.000	24.83	6557.3
11	75.50	412.62	0.492	14805.03	1.000	24.83	6215.9
12	80.00	413.98	0.478	13877.39	1.000	24.83	5826.4
13	84.50	415.17	0.459	12835.22	1.000	24.83	5388.9
14	89.00	416.12	0.432	11678.53	1.000	24.83	4903.2
15	89.10	416.13	0.354	11647.12	1.000	24.83	4890.0
16	91.71	416.10	0.340	10723.52	1.000	24.83	4502.3
17	94.32	415.97	0.321	9824.69	1.000	24.83	4124.9
18	96.00	417.12	0.322	7865.72	1.000	24.83	3302.4
19	100.43	420.17	0.321	3483.29	1.000	24.83	1462.5
20	104.85	423.85	0.361	246.14	1.000	24.83	103.3
21	106.42	428.06	0.406	-341.13	1.000	24.83	-143.2
22	108.71	433.22	0.443	-218.40	1.000	24.83	-91.7
23	109.06	434.00	0.000	0.00	1.000	24.83	0.0

NOTE: A value of 0.000- for h/H indicates that the line of thrust is at or below the lower boundary of the sliding mass. A value of 1.000+ for h/H indicates that the line of thrust is at or above the upper boundary of the sliding mass.

Table 2 - Geometry Data on the 23 Slices

Slice No.	Width (ft)	Height (ft)	X-Cntr (ft)	Y-Cntr-Base (ft)	Y-Cntr-Top (ft)	Alpha (deg)	Beta (deg)	Base Length (ft)
1	1.14	0.84	29.83	397.77	398.60	-36.23	36.16	1.42
2	3.17	3.99	31.98	396.19	400.18	-36.23	36.16	3.93
3	3.43	7.21	35.28	395.38	402.59	11.53	36.16	3.51
4	4.50	9.30	39.25	396.19	405.49	11.53	36.16	4.59
5	4.50	11.67	43.75	397.10	408.78	11.53	36.16	4.59
6	4.50	14.04	48.25	398.02	412.07	11.53	36.16	4.59
7	4.50	16.42	52.75	398.94	415.36	11.53	36.16	4.59
8	5.50	17.04	57.75	399.96	417.00	11.53	0.00	5.61
9	5.50	15.92	63.25	401.08	417.00	11.53	0.00	5.61
10	5.00	16.15	68.50	402.15	418.30	11.53	27.55	5.10
11	4.50	17.66	73.25	403.12	420.78	11.53	27.55	4.59
12	4.50	19.09	77.75	404.04	423.13	11.53	27.55	4.59

13	4.50	20.52	82.25	404.96	425.48	11.53	27.55	4.59
14	4.50	21.95	86.75	405.88	427.83	11.53	27.55	4.59
15	0.10	25.15	89.05	406.35	431.50	11.53	88.85	0.10
16	2.61	27.38	90.41	406.62	434.00	11.53	0.00	2.67
17	2.61	26.85	93.02	407.15	434.00	11.53	0.00	2.67
18	1.68	25.73	95.16	408.27	434.00	45.41	0.00	2.39
19	4.43	22.63	98.21	411.37	434.00	45.41	0.00	6.31
20	4.43	18.14	102.64	415.86	434.00	45.41	0.00	6.31
21	1.57	12.95	105.64	421.05	434.00	75.10	0.00	6.10
22	2.29	5.70	107.57	428.30	434.00	75.10	0.00	8.90
23	0.35	0.70	108.89	433.30	434.00	76.09	0.00	1.44

Table 2A - Coordinates of Slice Points Defining the Slip Surface

Point No.	X-Pt (ft)	Y-Pt (ft)
1	29.254772	398.186179
2	30.396882	397.349220
3	33.565552	395.027161
4	37.000000	395.727763
5	41.500000	396.645731
6	46.000000	397.563698
7	50.500000	398.481665
8	55.000000	399.399633
9	60.500000	400.521593
10	66.000000	401.643553
11	71.000000	402.663517
12	75.500000	403.581484
13	80.000000	404.499452
14	84.500000	405.417419
15	89.000000	406.335387
16	89.100000	406.355786
17	91.711560	406.888525
18	94.323120	407.421265
19	96.000000	409.122041

20	100.427209	413.612340
21	104.854419	418.102640
22	106.423960	424.000000
23	108.712272	432.598052
24	109.059461	434.000000

Table 3 - Force and Pore Pressure Data On The 23 Slices (Excluding Reinforcement)

Slice No.	Weight (lbs)	Ubeta Force Top (lbs)	Ubeta Stress Top (psf)	Ualpha Force Bot (lbs)	Pore Pressure (psf)	-Uw (psf)	Earthquake Force Horiz (lbs)	Earthquake Force Vert (lbs)	Distributed Load (psf)
1	95.5	0.0	0.0	0.0	0.00	0.00	20.0	0.0	0.00
2	1264.4	0.0	0.0	0.0	0.00	0.00	265.5	0.0	0.00
3	2477.5	0.0	0.0	0.0	0.00	0.00	520.3	0.0	0.00
4	4186.6	0.0	0.0	0.0	0.00	0.00	879.2	0.0	0.00
5	5253.4	0.0	0.0	0.0	0.00	0.00	1103.2	0.0	0.00
6	6320.1	0.0	0.0	0.0	0.00	0.00	1327.2	0.0	0.00
7	7386.8	0.0	0.0	0.0	0.00	0.00	1551.2	0.0	0.00
8	9371.7	0.0	0.0	0.0	0.00	0.00	1968.0	0.0	0.00
9	8754.6	0.0	0.0	0.0	0.00	0.00	1838.5	0.0	0.00
10	8075.4	0.0	0.0	0.0	0.00	0.00	1695.8	0.0	0.00
11	7947.0	0.0	0.0	0.0	0.00	0.00	1668.9	0.0	0.00
12	8590.5	0.0	0.0	0.0	0.00	0.00	1804.0	0.0	0.00
13	9233.9	0.0	0.0	0.0	0.00	0.00	1939.1	0.0	0.00
14	9877.4	0.0	0.0	0.0	0.00	0.00	2074.2	0.0	0.00
15	251.5	0.0	0.0	0.0	0.00	0.00	52.8	0.0	0.00
16	7149.9	0.0	0.0	0.0	0.00	0.00	1501.5	0.0	0.00
17	7010.8	0.0	0.0	0.0	0.00	0.00	1472.3	0.0	0.00
18	4314.3	0.0	0.0	0.0	0.00	0.00	906.0	0.0	0.00
19	10020.0	0.0	0.0	0.0	0.00	0.00	2104.2	0.0	0.00
20	8032.1	0.0	0.0	0.0	0.00	0.00	1686.7	0.0	0.00
21	2032.3	0.0	0.0	0.0	0.00	0.00	426.8	0.0	0.00
22	1304.6	0.0	0.0	0.0	0.00	0.00	274.0	0.0	0.00
23	24.3	0.0	0.0	0.0	0.00	0.00	5.1	0.0	0.00

TOTAL WEIGHT OF SLIDING MASS = 128974.63(lbs)

EFFECTIVE WEIGHT OF SLIDING MASS = 128974.63(lbs)

TOTAL AREA OF SLIDING MASS = 1289.75(ft2)

TABLE 4 - SOIL STRENGTH & SOIL OPTIONS DATA ON THE 23 SLICES

Slice No.	Soil Type	Cohesion (psf)	Phi(Deg)	Options
1	1	200.00	27.00	
2	2	300.00	28.00	A
3	2	150.00	18.00	A
4	2	150.00	18.00	A
5	2	150.00	18.00	A
6	2	150.00	18.00	A
7	2	150.00	18.00	A
8	2	150.00	18.00	A
9	2	150.00	18.00	A
10	2	150.00	18.00	A
11	2	150.00	18.00	A
12	2	150.00	18.00	A
13	2	150.00	18.00	A
14	2	150.00	18.00	A
15	2	150.00	18.00	A
16	2	150.00	18.00	A
17	2	150.00	18.00	A
18	2	300.00	28.00	A
19	2	300.00	28.00	A
20	2	300.00	28.00	A
21	2	300.00	28.00	A
22	1	200.00	27.00	
23	1	200.00	27.00	

SOIL OPTIONS:
A = ANISOTROPIC SHEAR STRENGTH
C = CURVED STRENGTH ENVELOPE (TANGENT PHI & C)
F = FIBER-REINFORCED SOIL (FRS)

M = INDEPENDENT MULTI-STAGE SHEAR STRENGTH
N = NONLINEAR UNDRAINED SHEAR STRENGTH
R = RAPID DRAWDOWN OR RAPID LOADING (SEISMIC) SHEAR STRENGTH
S = SHANSEP UNDRAINED SHEAR STRENGTH
NOTE: Phi and C in Table 4 are modified values based on specified Soil Options (if any).

TABLE 5 - Total Base Stress Data on the 23 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Total Normal/Vert. Stress Ratio
1	-36.23	29.83	1.42	1518.77	83.58	18.172
2	-36.23	31.98	3.93	4194.47	399.04	10.511
3	11.53	35.28	3.51	664.07	721.38	0.921
4	11.53	39.25	4.59	847.73	930.36	0.911
5	11.53	43.75	4.59	1056.05	1167.41	0.905
6	11.53	48.25	4.59	1264.38	1404.46	0.900
7	11.53	52.75	4.59	1472.70	1641.51	0.897
8	11.53	57.75	5.61	1527.56	1703.94	0.896
9	11.53	63.25	5.61	1428.96	1591.74	0.898
10	11.53	68.50	5.10	1449.47	1615.08	0.897
11	11.53	73.25	4.59	1582.11	1766.01	0.896
12	11.53	77.75	4.59	1707.77	1909.00	0.895
13	11.53	82.25	4.59	1833.43	2051.98	0.893
14	11.53	86.75	4.59	1959.09	2194.97	0.893
15	11.53	89.05	0.10	2240.72	2515.44	0.891
16	11.53	90.41	2.67	2436.12	2737.78	0.890
17	11.53	93.02	2.67	2389.30	2684.51	0.890
18	45.41	95.16	2.39	1286.41	2572.83	0.500
19	45.41	98.21	6.31	1122.31	2263.28	0.496
20	45.41	102.64	6.31	884.27	1814.25	0.487
21	75.10	105.64	6.10	92.08	1294.87	0.071
22	75.10	107.57	8.90	-2.65	570.10	-0.005
23	76.09	108.89	1.44	-117.34	70.10	-1.674

TABLE 5A - Total Base Force Data on the 23 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Force (lbs)	Total Vert. Force (lbs)	Total Normal/Vert. Force Ratio
1	-36.23	29.83	1.42	2150.50	95.46	22.529
2	-36.23	31.98	3.93	16477.61	1264.42	13.032
3	11.53	35.28	3.51	2327.69	2477.54	0.940
4	11.53	39.25	4.59	3893.35	4186.64	0.930
5	11.53	43.75	4.59	4850.11	5253.36	0.923
6	11.53	48.25	4.59	5806.87	6320.08	0.919
7	11.53	52.75	4.59	6763.63	7386.80	0.916
8	11.53	57.75	5.61	8574.61	9371.66	0.915
9	11.53	63.25	5.61	8021.15	8754.58	0.916
10	11.53	68.50	5.10	7396.62	8075.41	0.916
11	11.53	73.25	4.59	7266.12	7947.05	0.914
12	11.53	77.75	4.59	7843.23	8590.48	0.913
13	11.53	82.25	4.59	8420.34	9233.92	0.912
14	11.53	86.75	4.59	8997.45	9877.36	0.911
15	11.53	89.05	0.10	228.69	251.54	0.909
16	11.53	90.41	2.67	6493.10	7149.89	0.908
17	11.53	93.02	2.67	6368.31	7010.76	0.908
18	45.41	95.16	2.39	3072.49	4314.33	0.712
19	45.41	98.21	6.31	7077.04	10020.02	0.706
20	45.41	102.64	6.31	5576.02	8032.07	0.694
21	75.10	105.64	6.10	561.92	2032.35	0.276
22	75.10	107.57	8.90	-23.62	1304.56	-0.018
23	76.09	108.89	1.44	-169.47	24.34	-6.964

TABLE 6 - Effective and Base Shear Stress Data on the 23 Slices

Slice No.	Alpha (deg)	X-Coord. Slice Cntr	Base Leng.	Effective Normal Stress	Available Shear Strength	Mobilized Shear Stress

*		(ft)	(ft)	(psf)	(psf)	(psf)
1	-36.23	29.83	1.42	1518.77	973.85	776.68
2	-36.23	31.98	3.93	4194.47	2530.24	2017.94
3	11.53	35.28	3.51	664.07	365.77	291.71
4	11.53	39.25	4.59	847.73	425.44	339.30
5	11.53	43.75	4.59	1056.05	493.13	393.29
6	11.53	48.25	4.59	1264.38	560.82	447.27
7	11.53	52.75	4.59	1472.70	628.51	501.25
8	11.53	57.75	5.61	1527.56	646.33	515.47
9	11.53	63.25	5.61	1428.96	614.30	489.92
10	11.53	68.50	5.10	1449.47	620.96	495.23
11	11.53	73.25	4.59	1582.11	664.06	529.61
12	11.53	77.75	4.59	1707.77	704.89	562.17
13	11.53	82.25	4.59	1833.43	745.72	594.73
14	11.53	86.75	4.59	1959.09	786.55	627.29
15	11.53	89.05	0.10	2240.72	878.05	700.27
16	11.53	90.41	2.67	2436.12	941.54	750.91
17	11.53	93.02	2.67	2389.30	926.33	738.78
18	45.41	95.16	2.39	1286.41	984.00	784.76
19	45.41	98.21	6.31	1122.31	896.74	715.18
20	45.41	102.64	6.31	884.27	770.17	614.24
21	75.10	105.64	6.10	92.08	348.96	278.30
22	75.10	107.57	8.90	-2.65	198.65	158.43
23	76.09	108.89	1.44	-117.34	140.21	111.82

TABLE 6A - Effective and Base Shear Force Data on the 23 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Effective Normal Force (lbs)	Available Shear Force (lbs)	Mobilized Shear Force (lbs)
1	-36.23	29.83	1.42	2150.50	1378.93	1099.73
2	-36.23	31.98	3.93	16477.61	9939.82	7927.29
3	11.53	35.28	3.51	2327.69	1282.09	1022.50
4	11.53	39.25	4.59	3893.35	1953.93	1558.31
5	11.53	43.75	4.59	4850.11	2264.80	1806.24

6	11.53	48.25	4.59	5806.87	2575.67	2054.17
7	11.53	52.75	4.59	6763.63	2886.54	2302.10
8	11.53	57.75	5.61	8574.61	3628.05	2893.47
9	11.53	63.25	5.61	8021.15	3448.22	2750.05
10	11.53	68.50	5.10	7396.62	3168.75	2527.17
11	11.53	73.25	4.59	7266.12	3049.81	2432.31
12	11.53	77.75	4.59	7843.23	3237.32	2581.86
13	11.53	82.25	4.59	8420.34	3424.83	2731.40
14	11.53	86.75	4.59	8997.45	3612.35	2880.95
15	11.53	89.05	0.10	228.69	89.61	71.47
16	11.53	90.41	2.67	6493.10	2509.54	2001.43
17	11.53	93.02	2.67	6368.31	2468.99	1969.09
18	45.41	95.16	2.39	3072.49	2350.20	1874.35
19	45.41	98.21	6.31	7077.04	5654.67	4509.76
20	45.41	102.64	6.31	5576.02	4856.56	3873.24
21	75.10	105.64	6.10	561.92	2129.57	1698.40
22	75.10	107.57	8.90	-23.62	1767.44	1409.58
23	76.09	108.89	1.44	-169.47	202.51	161.51

Average Effective Normal Stress = 1295.3139(psf)
Average Available Shear Strength = 687.0639(psf)
Total Length of Failure Surface = 98.7975(ft)

INPUT PARAMETERS FOR SIMPLIFIED 3D ANALYSIS

Composite Phi (deg)= 27.942
3D Depth (Vertical difference between ends of Slip Surface) = 35.81(ft)
L/D(1): L = 35.81(ft) L/D(2): L = 71.63(ft) L/D(3): L = 107.44(ft)
L/D(4): L = 143.26(ft) L/D(5): L = 179.07(ft) L/D(6): L = 214.88(ft)

Average Elevation of Point of Application of kh on Sliding Mass = 413.807(ft)

SUM OF MOMENTS = -0.441330E-03 (ft/lbs);Imbalance (Fraction of Total Weight) = -0.3421833E-08
SUM OF FORCES = -.116552E-03 (lbs);Imbalance (Fraction of Total Weight) = -0.9036825E-09

Sum of Available Shear Forces = 67880.20(lbs)

Sum of Mobilized Shear Forces = 54136.40(lbs)

FS Balance Check: FS = 1.253874

**** END OF GEOSTASE OUTPUT ****

Appendix F

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 GENERAL

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

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

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

During grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all keyway bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of subgrade and fill materials and perform adequate relative compaction testing of fill to determine the attained level of compaction and assess if, in their opinion, if the work was performed in substantial compliance

with the geotechnical report(s) and these specifications. The Geotechnical Consultant shall provide test results to the owner on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with applicable grading codes, the project plans, and these specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork planned for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

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

2.0 PREPARATION OF FILL AREAS

2.1 Clearing and Grubbing: Areas to be excavated and filled shall be cleared and grubbed. Vegetation, such as brush, grass, roots, and other deleterious material, man-made structures, and similar debris shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant. Borrow areas shall be cleared and grubbed to the extent necessary to provide a suitable fill material.

Concrete fragments that are free of reinforcing street may be placed in fills, provided they are placed in accordance with Section 3 and 4. Earth fill material

shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent organic matter. Nesting of organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, etc.) have chemical constituents that are considered hazardous waste. As such, the indiscriminate dumping or spillage of such fluids may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

The Geotechnical Consultant shall not be responsible for the identification or analysis of potentially hazardous materials; however, if observations, odors, or soil discoloration are suspect, the Geotechnical Consultant may request from the owner the termination of grading operations until such materials are deemed not hazardous as defined by applicable laws and regulations.

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

2.3 **Processing:** Ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Ground that is not satisfactory shall be removed/overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction. After scarification, the surface should be moisture conditioned, as necessary, to achieve the proper moisture content and compacted in accordance with Section 4 of these specifications.

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

2.5 **Benching:** Fills to be placed on ground sloping steeper than 5H:1V (horizontal to vertical units) shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for fill placement.

3.0 FILL MATERIAL

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

3.2 **Oversize:** Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or other underground construction.

3.3 **Import:** If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1 and/or requirements defined in the project geotechnical report(s). The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before import begins so that suitability can be determined, and appropriate laboratory tests performed.

4.0 FILL PLACEMENT AND COMPACTION

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

4.2 **Fill Moisture Conditioning:** Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with ASTM International (ASTM Test Method D1557).

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

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

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

4.5 **Frequency of Compaction Testing:** Tests shall be taken at intervals required by the governing agency and as deemed necessary by the Geotechnical Consultant in order to adequately qualify the fill material. In general, it should be anticipated that tests will be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill, unless recommended otherwise by the Geotechnical Consultant. In addition, test(s) shall be taken on slope faces and/or each 10 feet of vertical height of slope as deemed necessary by the Geotechnical Consultant. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.6 **<u>Compaction Test Locations</u>:** The Geotechnical Consultant shall document the approximate elevation and location of each compaction test. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided. Alternatively, GPS units may be used to determine the approximate location/coordinates of the field density tests.

5.0 SUBDRAIN INSTALLATION

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and standard details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys. The Contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The Contractor is responsible for the performance of subdrains.

6.0 EXCAVATION

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

7.0 TRENCH BACKFILLS

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

7.2 Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum 90 percent of maximum from 1 foot above the top of the conduit to the surface, except in traveled ways (see Section 7.6 below).

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

7.4 Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill, unless required differently by the governing agency or the Geotechnical Consultant.

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

7.6 Trench backfill in the upper foot measured from finish grade within existing or future traveled way, shoulder, and other paved areas (or areas to receive pavement) should be placed to a minimum 95 percent relative compaction.



LEGEND

EARTH UNITS - CIRCLED WHERE BURIED

Af	ARTIFICIAL FILL
Tm	MONTEREY FORMATION

SYMBOLS - LOCATIONS ARE APPROXIMATE, QUERIED WHERE UNCERTAIN

GEOLOGIC CONTACT, DOTTED WHERE BURIED

OTHER SYMBOLS - LOCATIONS ARE APPROXIMATE

● H-13 T.D.41.5' Af@0-35' Tm@35'+ — HOLLOW-STEM AUGER BORING LOCATION, SHOWING TOTAL DEPTH, DEPTH TO EARTH UNITS, AND DEPTH TO GROUNDWATER IN FEET

● HA-1 T.D.13' Af@0-13' — HAND AUGER BORING LOCATION, SHOWING TOTAL DEPTH AND DEPTH TO EARTH UNITS IN FEET

C — C' GEOTECHNICAL CROSS-SECTION

GRAPHIC SCALE

60 0 30 60 120

(IN FEET)
1 inch = 60 ft.

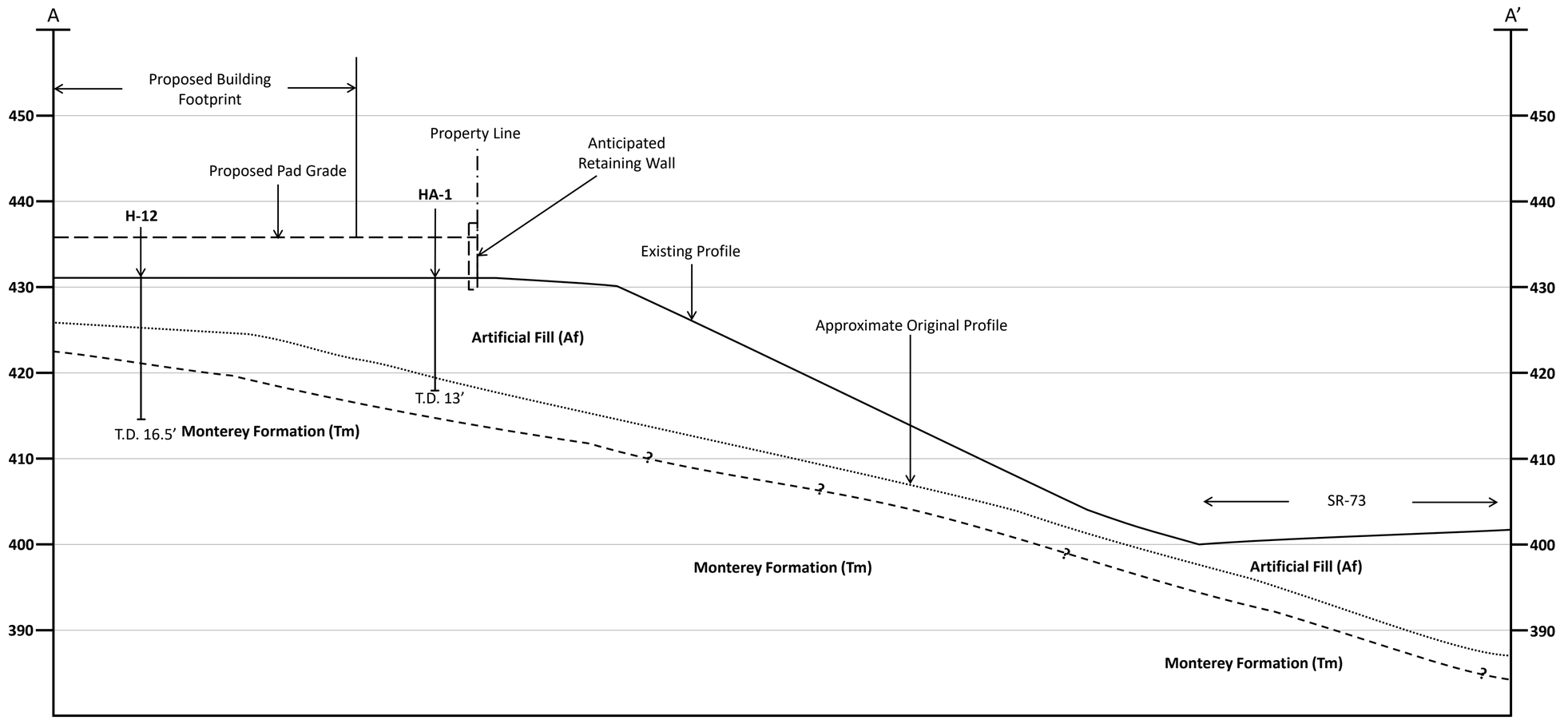
PLATE 1

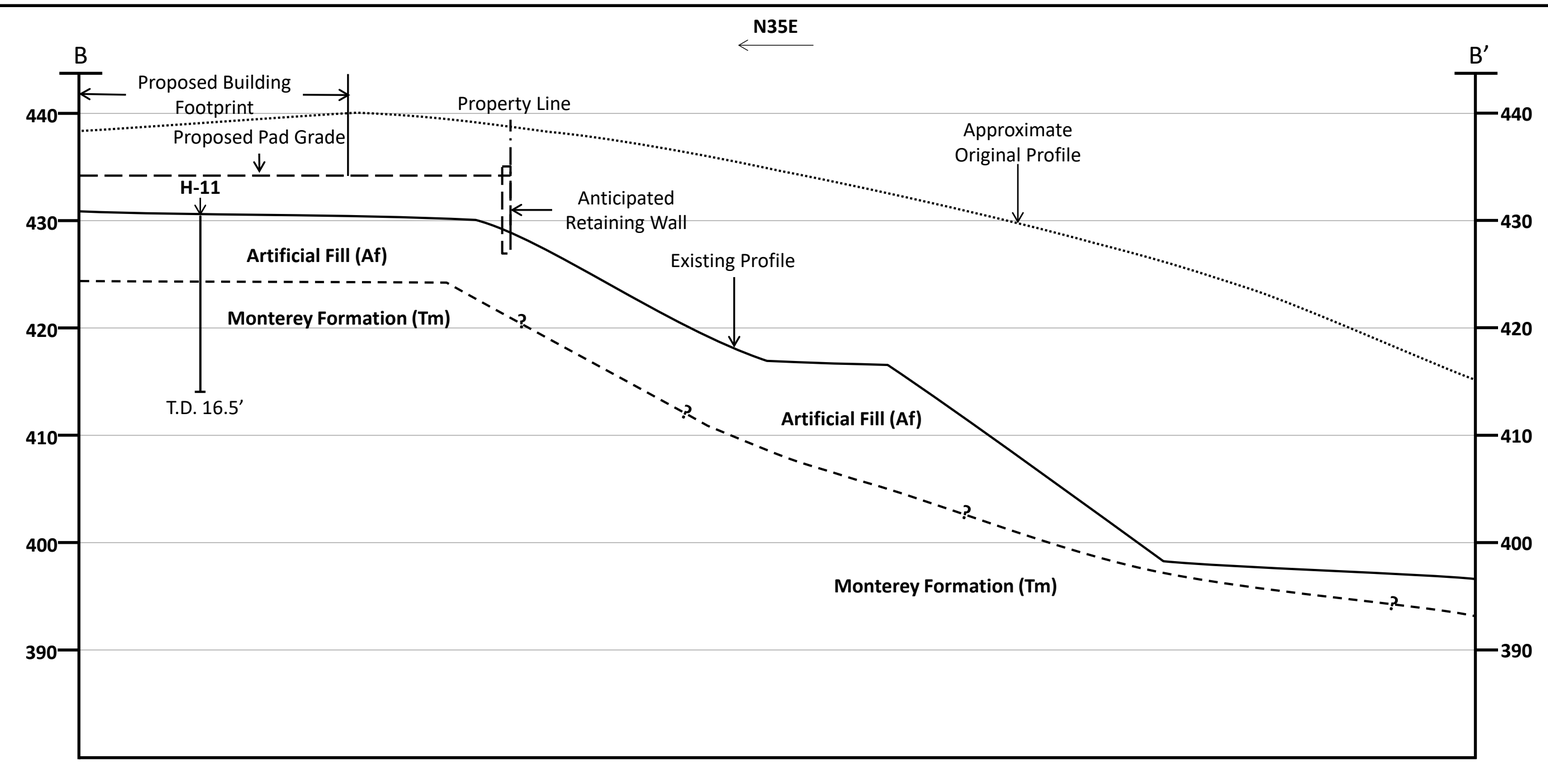
GEOTECHNICAL MAP
PROPOSED RESIDENTIAL DEVELOPMENT
1 THROUGH 5 POLARIS WAY
ALISO VIEJO, CALIFORNIA

Project No.: 25093-01 By: RS/AZ
Project Name: Lennar/Polaris Place
Date: 3/4/2026 SCALE: 1" = 60'

S/A

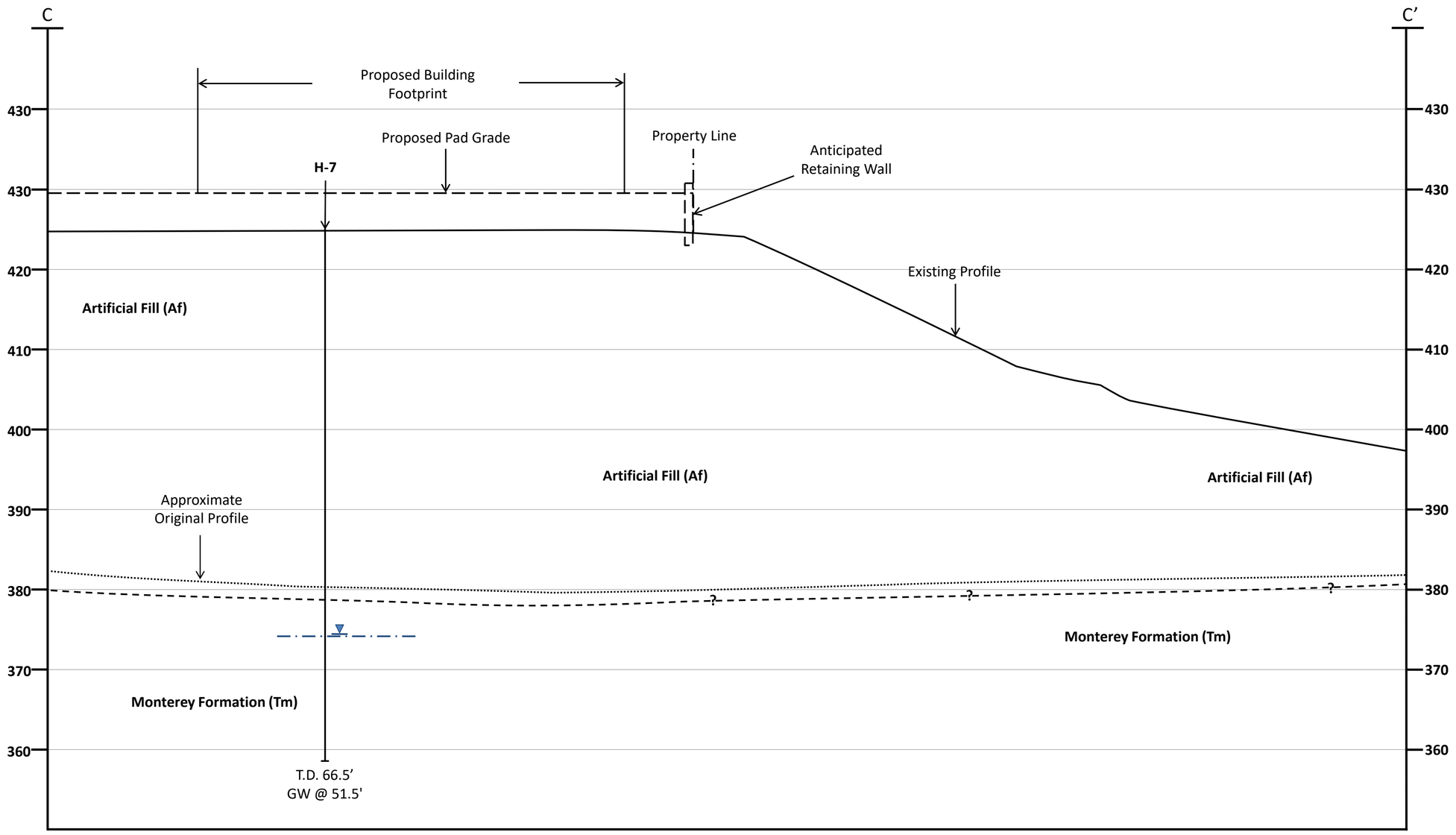
Drawing: P:\2025\25093-01 Lennar - Polaris Place, Aliso Viejo\Drafting_MAP\25093-01 GeoMap Plate.dwg Layout: 1 Last Saved: Tue Mar 03, 2026 - 12:28pm Last Plotted: Tue Mar 03, 2026 - 3:27pm By: Anthony Zepeda





Scale: 1 inch=10 feet

Geotechnical Cross-Sections A-A' and B-B'	
Lennar Proposed Residential Development 1 Through 5 Polaris Way Aliso Viejo, California	Project Number: 25093-01 Date: March 4, 2026 Plate 2



N60W

Scale: 1 inch=10 feet

Geotechnical Cross-Section C-C'	
Lennar Proposed Residential Development 1 Through 5 Polaris Way Aliso Viejo, California	Project Number: 25093-01 Date: March 4, 2026 Plate 3