GEOTECHNICAL EXPLORATION REPORT
PROPOSED VISTA II PROJECT
CONDOMINIUM DEVELOPMENT
LOT 2 / APN NO. 183-060-84
145 HANNALEI DRIVE
CITY OF VISTA, SAN DIEGO COUNTY
CALIFORNIA 92083

Prepared for

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Leighton and Associates, Inc.
A Leighton Group Company

January 16, 2023

Project No. 13102.001b

Warmington Residential California, Inc.
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Attention: Mr. Matthew Esquivel

Subject: **Geotechnical Exploration Report**
Proposed 5.33-Acre Vista II Condominium Development
APN No. 183-060-78
145 Hannalei Drive
City of Vista, San Diego County, California 92083

In accordance with your request and authorization, Leighton and Associates, Inc. (Leighton) is pleased to present this geotechnical exploration report pertinent to the subject property and planned condominium development project, located on the subject property located within the City of Vista, San Diego County, California (the site). The intent of our services was to evaluate site geotechnical conditions and geologic hazards posing potential impacts to the current residential development project, and determine recommendations for use in project design, remedial grading and construction. We also evaluated conditions of shallow soil infiltration for the purpose of onsite stormwater system design.

Based the findings of our exploration, it is our opinion that development of the project in accordance with current plans is considered generally suitable from a geotechnical standpoint, provided the recommendations contained within this report are implemented during design and construction.

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

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Description

The subject property occupies a marginal valley area within the City of Vista, San Diego County, California (site). For the purpose of this report, we have designated the site as having a centralized global position of 33.180161 degrees North Latitude and -117.127197 degrees West Longitude. The regional site location is depicted on attached Figure 1, *Site Location Map*. The San Diego County Assessor's Office indicates the subject site encompasses 3 contiguous parcels totaling approximately 8.93 acres, under Assessor Parcel Number (APN) APN: 183-060-84). The parcels include Lot 1 [3.52 acres], Lot 2 [5.33 acres] and Lot 3 [0.08 acres]. As further discussed herein, planned residential structures and a majority of appurtenant improvements will be constructed within the boundaries of Lot 2. A minor portion of Lot 1 will be developed as a parking lot and access road.

The parcels form a roughly irregular north-south elongated shape that is divided by a central north-south slope into an upper and lower area of relatively flat relief. Planned condominium structures are cited in the lower east area, and an existing church facility currently occupies the upper west area. The total relief represented by the central slope is on the order of 13 vertical feet, extending between the elevation of 501 and 488 feet above mean sea level (msl). Overall vertical relief across the site as a whole is approximately 52 feet, extending between El. 544 feet (msl) on the northwest, and 475 feet msl on the southeast. The existing and design topography and improvements are depicted on attached Figure 2, *Geotechnical Exploration Map*.

Based on our interpretation of surface geomorphology, notable in a 1964 aerial photo (NETRonline, 1964), the upper church area represents a paleo stream terrace, and that of the central slope an easterly facing stream cut-bank. The lower area of planned condominium development consists of a younger river terrace within a low-lying alluvial plain. By 1971, the lower east site area had been modified by grading to form the existing little league baseball facility, consisting of three diamonds and an adjacent parking lot. To achieve the current site configuration it appears cuts encroached westerly into the terrace, and thin fill was placed across the ballpark areas.

1.2 Purpose and Scope of Work

Our geotechnical services were performed to evaluate general geologic and geotechnical conditions of the property and constraints to proposed development.

The focus was also to determine preliminary geotechnical recommendations for project design, remedial grading and project construction.

The scope of our services consisted of the following tasks:

- Review of 40-scale Preliminary Grading plan prepared by Rick Engineering Company dated September 30, 2022, and a search and review of pertinent literature including on-line and in-house published/unpublished geologic, geo-environmental, topographic, and groundwater reports/maps, and high-altitude aerial photographic imagery available online. Also reviewed was geotechnical information for the neighboring Vista I residential development to the north, currently under development by Century Communities, Inc. A listing of review documents is presented in Appendix A, *References*.

- Site reconnaissance and the demarcation of planned exploratory locations prior to field work. These tasks were performed to screen for the presence/absence of underground public utilities in areas of planned exploration, using Underground Service Alert (USA).

- Advanced four (4) subsurface geotechnical borings on the site to evaluate general soil and bedrock units and collect samples for laboratory testing. Logs of each boring were prepared designating sample locations and the general distribution of subsurface earth units. Boring locations are depicted on an aerial base map along with the current project design, see Figure 2, *Geotechnical Exploration Map*. The logs of our exploratory borings are attached in Appendix B-1, *Logs of Exploratory Borings*. The surface geology of the site and surrounding areas is shown on Figure 3, *Regional Geology Map*.

- Reviewed the logs of three offsite exploratory trench logs, located west of the existing church building along a planned access road associated with the neighboring Vista I construction project. Logs of trenches are attached as Appendix B-2, *Logs of Offsite Exploratory Trenches*.

- Performed two (2) field percolation tests within the lower east area of the site and planned condominium construction. The tests were conducted within in-situ alluvial soils to determine infiltration rates for use by the civil engineer in BMP design. The logs of our field percolation test borings are attached in Appendix B, *Logs of Exploratory Borings*.

- Performed laboratory tests to evaluate engineering properties of site soils. Tests included Expansion Index, Atterberg limits, and Shear Strength. The test results are attached in Appendix C*, Results of Laboratory Testing.*

- Calculated rates of soil infiltration based on percolation test results. Un-factored rates are attached in Appendix D, *Field Percolation Test Results*.

- Evaluated potential geologic hazards including fault rupture, ground lurching, tsunamis, seiches and flooding, liquefaction, landsliding, seismic densification, subsidence and ground fissuring, engineering properties of encountered soils, and groundwater conditions. A map depicting the location of local faults hazards is attached as Figure 4, *Regional Fault and Historic Seismicity Map*. Flood and dam hazard maps are attached as Figure 5, *Flood Hazard Zone Map*. Also attached is a map showing the agency boundaries of potential dam inundation, as Figure 6, *Dam Inundation Map*.

- Geotechnical engineering analysis of collected data including the following:

  - Seismicity: Site-specific seismic coefficients are provided in accordance with the 2022 California Building Code (CBC). The findings of our seismic analysis is presented in Appendix E, *Seismic Analysis*.

  - Corrosivity: Assessment of potential soil corrossivity, relating to its attack of concrete and metal in contact with onsite soils.

  - Foundation: Assessment of foundation type feasibility for planned building structures, and presentation of preliminary geotechnical parameters for foundation and retaining wall design.

  - Pavements: Preliminary assessment of pavement sections for construction of entry drives and paved internal roadways.

  - Grading: Requirements/guidelines for remedial grading, temporary excavations, and geotechnical parameters for design of temporary shoring, including lateral earth pressures. Earthwork specifications are presented as Appendix F, *General Earthwork and Grading Specifications*.

- Assessed rippability of soil and bedrock based on drilling findings, and results of offsite geophysical profiling performed within the neighboring Vista I property.

- Prepared a San Diego County Infiltration Rate Form I-8 to assess the feasibility of on-site infiltration BMP's, attached as Appendix G., *Infiltration Rate Form I-8.*

- Prepared this report, containing geotechnical findings, conclusions and recommendations for remedial grading, foundation design and construction of the residential development. Our report is considered suitable for submittal for review and comment to the building official. Important information concerning the use of this report is attached in Appendix H, *GBA, Important Information About This Geotechnical-Engineering Report.*

1.3 Proposed Development

Our understanding of the project is based on a 40-scale Preliminary Grading plan prepared by Rick Engineering Company, dated September 30, 2022. The plan shows improvements encompassing a majority of the subject property, including 37 detached 2-story residential units on level pads with attached garages. We anticipate building construction will consist of wood framing with light loads supported by conventional spread foundations and either slab-on-grade or post-tensioned floors. Interior and perimeter retaining walls are also planned up to around 11 feet in height, along with appurtenant underground utilities.

Vehicular entry and egress will be via a looping spine road from Hannalei Drive on the south. Alleys will extend off the spine road for garage access. The road will hairpin around the northern end of the site and extend offsite to a planned parking lot planned north of the existing church structure. The road will require a retaining wall on the northwest up to 11 feet high. We expect the road section will consist of asphalt-concrete (AC) over aggregate base. The conceptual layout of the project is shown along with exploration localities on attached Figure 2, Geotechnical *Exploration Map*.

We anticipate construction of building pads and streets will be accomplished via conventional remedial earthwork (grading), involving minor cuts and fills. We understand approximately 10,400 cubic yards of soil will be required to balance the site.

Our grading and foundation design recommendations are considered applicable for the type of construction discussed above. Although undetermined at this time, structural loads for buildings are expected to have a maximum column load of 45 kips and maximum wall load of 2.5 kips per lineal foot.

2.0 FINDINGS

2.1 Literature Review

Our evaluation of site geotechnical conditions included review of geotechnical reports (Leighton, 2021; GeoSoils, 2013) and limited observations of field conditions in October of 2022, associated with grading for the neighboring offsite Vista I condominium development. Pertinent to the subject site are three test pit excavations observed within an existing access road along the west side of the existing church building. The approximate locations of the pits are noted on Figure 2, Geotechnical *Exploration Map*. The logs revealed the road was constructed on a residual soil developed within weathered crystalline bedrock, as shown on attached Appendix B-2, *Logs of Offsite Exploratory Trenches*.

2.2 Field Exploration

Our field exploration included advancement of four (4) hollow stem borings in April of 2021, using a truck-mounted CME-75 drill rig and 8-inch-diameter hollow-stem augers. The borings were spaced roughly evenly across the property in accessible areas. During exploration, disturbed/bulk and undisturbed/ring samples of earth deposits were collected using split-spoon drive samplers. The samples were transported to our laboratory for testing and evaluation. The approximate boring locations are noted on Figure 2, *Geotechnical Exploration Map*. A member of our geotechnical staff (geologist) completed fieldwork including the logging and sampling of borings. The borings were subsequently backfilled with the tailings generated by drilling. Exploratory boring logs are presented in Appendix B-1.

2.3 Field Percolation Testing

Two field percolation tests were conducted in general accordance with City of Vista BMP Design Manual, Appendix D (2016). The tests were performed at depths of 10 and 6 feet bgs, within borings LP-1 and LP-2, respectively.

Percolation test wells consisted of 2-inch-diameter perforated PVC pipe, with perforated sections inserted within zones to be tested and solid PVC pipe above. The borehole annulus was filled with clean sand (#3 Monterey Sand) to approximately 1 foot above the screen zone. After the conclusion of the percolation tests, PVC pipe was removed and the test holes backfilled with soil cuttings.

The falling head procedure was warranted based on soil type and preliminary rates of percolation. This method records a drop in water levels inside the well over the testing period (30-minute intervals). Measured/un-factored infiltration rates were calculated for each well was calculated by dividing the rate of discharge (cubic inches per hour) by the infiltration surface area, or flow area (square inches). Discharge volume was calculated by adding the total volume of water that dropped within the PVC pipe and annulus incorporating a porosity reduction factor to account for the filter pack material.

Copies of our field percolation test data are presented in Appendix D. A summary of infiltration testing results is provided in Section 3.5.5.

2.4 Laboratory Testing

Representative bulk and relatively undisturbed soil samples were subjected to a series of physical laboratory tests within our laboratory. Test types included in-situ moisture and/or density, sieve analyses, Atterberg limits, expansion index, compaction (maximum dry density and optimum moisture), and direct shear testing. The test results aided in forming the basis of our design parameter analyses and recommendations. The results of laboratory testing are presented in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL/GEOLOGIC CONDITIONS

3.1 Regional Geology

The regional locality of the project area lies within a coastal plain region of the Peninsular Ranges Geomorphic Province of California. This province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). It is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous, Tertiary, and Quaternary age sedimentary rocks.

3.2 Site Geology

Published geologic maps indicate the site is underlain entirely at shallow depth by igneous rock (Tonalite), a Cretaceous age crystalline bedrock formation (USGS, 2007). The regional distribution of site geologic units is depicted on Figure 3, *Regional Geology Map*. However, based on the findings of our site borings and other offsite data, we find the geologic conditions differ from that of regional maps. While the upper west area of the site is underlain by weathered crystalline bedrock, the eastern site area is mantled by older alluvium between 6 to 17 feet thick. Deposits of undocumented artificial fill mantle the surface of the east area, logged as approximately 3.5 feet thick within our borings.

The aerial distribution of geologic units on the site is shown on Figure 2, *Geotechnical Exploration Map*. The general distribution of geologic units in the subsurface are documented in the logs of soil borings and offsite trenches attached in Appendix B-1 and B-2, respectively. A brief description of on-site geologic units is presented below.

3.3 Geologic Units

Units of bedrock and soil encountered beneath the site are described as follows.

- *Artificial Fill, Undocumented (Not Mapped):* A thin mantle of artificial fill up to 3.5 feet thick was encountered within the upper portion of each of our four (4) borings. The artificial fill materials encountered generally consist of dark brown to reddish brown, moist to very moist, sand, clayey sand and sandy clay. This unit is classified as undocumented as we expect no record of its placement exists, or is available with agency files. Existing fills on the site will require complete removal and replacement with engineered fill.

- *Quaternary Alluvial Deposits (Qa):* A deposit of Quaternary Alluvium was encountered below the artificial fill in each boring. The alluvium generally consists of a brown to red-brown and orange-brown, moist to very moist, stiff to hard clay, sandy clay and medium dense to dense clayey sand.
- *Cretaceous Tonalite (gr):* The upper surface of the Cretaceous age Tonalite (Kt) has been subjected to significant pedogenesis (soil development), which has effectively destroyed by weathering much of the in-situ properties of the original crystalline igneous rock fabric. The soil mantle contains a horizon of deeply weathered clay on the order of 3 feet thick. With increasing depth, the bedrock exhibits a less and less weathered and harder in-situ granitic texture. Based on the ease of drilling and trenching advancement, the rock remains rippable, slightly harder, and composed of biotite, plagioclase and quartz constituents within the depths explored.

3.4 Surface Water and Groundwater

No evidence of surface ponding was encountered during our field exploration. Nor was any groundwater encountered in our borings, to the maximum depth explored (30.2 feet below the surface). Groundwater is not anticipated to be a constraint to grading or construction of the proposed improvements.

3.5 Engineering Characteristics of On-site Soils

Engineering characteristics of on-site soils are discussed below, based on the results of our laboratory testing of bulk and in-situ representative on-site soils, field observations, and our professional experience on similar sites with similar soils conditions. The results of laboratory testing are provided in Appendix C.

3.5.1 Expansion Potential

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and which shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result.

Two (2) near-surface bulk soil samples obtained during our subsurface exploration were tested for expansion potential. The test results indicate Expansion Index (EI) values of 38 and 53 ("low" to "Medium" potential for expansion). Test results are included in Appendix C of this report.

It is possible the expansion potential of onsite soils may vary across the site, or at depth. As such, additional testing is recommended upon completion of site grading to confirm expansion potential conformance with the data presented in this report. For purposes of this report, and based upon visual characterization of alluvial materials at approximate foundation depth, medium expansion potential of site materials may be considered to support design and verified upon completion of earthwork grading.

3.5.2 Atterberg Limits

We conducted Atterberg Limit tests on samples of alluvium from 2 to 5 feet bgs, and from the weathered zone of the tonalitic bedrock. Test results for samples of alluvium reveal a Liquid Limit between 26 and 34, Plastic Limit between 11 and 12, and Plasticity Index of 14 and 22. This places the upper alluvium deposit in a category of CL. Test results for samples of weathered bedrock revealed a Liquid Limit of 31 and 32, Plastic Limit of 12 and 14, and Plasticity Index of 18 and 19. We do not anticipate the bedrock will be involved in site grading. If it were, once removed, test results indicate is would be classified as a sandy clay (CL) to clayey sand (SC) per USCS.

3.5.3 Shear Strength

Evaluation of shear strength was conducted via performance of laboratory direct shear testing. Test results are included in Appendix C, and as summary graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

3.5.4 Excavation Characteristics

Our findings indicate the deposits of Quaternary Alluvium within the lower east area of the property can easily be excavated using conventional heavy-duty construction equipment. Where weathered granitic bedrock is encountered within the central slope and upper west church pad, we anticipate removals will be shallow and rippability will be feasible using heavy ripping with a dozer. At this time, other than refusal in our borings, the feasibility of deeper bedrock rippability has not been evaluated. Based on the results of exploration and rough grading for the neighboring development to the north, in similar bedrock, the use of blasting is not anticipated within the upper 25 feet of bedrock. We do not anticipate such depths will be required as part of grading. Should any larger cobble or boulder size material be encountered during grading, or generated by bedrock removals, clasts larger than 8 inches in maximum dimension should be placed within non-structural landscape areas or hauled off site.

3.5.5 Infiltration Rates

Infiltration rates were determined in general accordance with the approved Infiltration Rate Assessment Methods, Appendix D, of the County of San Diego BMP Design Manual, adopted by the City of Vista (2020).

Field percolation results were converted into rates of measured infiltration using the Porchet Method by application of the formula below.

$$I_t = \frac{\Delta H * 60 * r}{\Delta t(r+2H_{AVG})}$$

Where:
I_t = calculated infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
r = radius of test hole
H_{AVG} = average head over the time interval, inches

Field-measured infiltration rates at test depths and locations are summarized below in Table 1.

Table 1 – Percolation Test Data and Infiltration Rate Calculations

Test No.	Test Depth (ft)	Soil Type	Field Infiltration Rate (inches/hr)
LP-1	5.5 to 10	CL to SC (Weathered Tonalite)	0.01
LP-2	3 to 6	CL to SC (Alluvium)	0.01

Soil types encountered within the test zone for LP-1, in our boring LB-3 on the western portion of the site, predominately consist of sandy clays to clayey sands. A sieve/hydrometer test performed on a bulk sample recovered from 6 to 9 feet in boring LB-3, adjacent to LP-1, confirm the lean sandy clay (CL) characterization of these soils. The results of the field percolation test at this location yielded a measured infiltration rate of 0.01 in/hr. Soil types within the test zone for LP-2, on the southeastern portion of the site, predominately consist of sandy clays to clayey sands with minor sands locally. The field percolation test at this location also yielded a measured infiltration rate of 0.01 in/hr.

Based on the field percolation testing and the calculated infiltration rates, the tested locations are categorized as “No Infiltration” conditions, as determined by the County of San Diego Infiltration Form I-8, Categorization of Infiltration Feasibility Condition, which has been completed and is presented in Appendix G.

4.0 SEISMIC AND GEOLOGIC HAZARDS

4.1 Regional Tectonic Setting and Seismicity

The site is considered to lie within a seismically active region, as does all of Southern California. During the late Pliocene, several new faults developed in Southern California, creating a new tectonic regime superposed on the flat-lying section of Tertiary and late Cretaceous rocks in the San Diego region.

The principal known onshore faults which collectively account for the majority of seismic hazard in southernmost California are the San Andreas, San Jacinto, Elsinore, Imperial and Rose Canyon faults. The balance of seismic hazard is taken by the offshore zone of faults which include the Coronado Bank, San Diego Trough, and San Clemente faults off of the San Diego. Most of the offshore faults coalesce south of the international border, where they come onshore as the Agua Blanca fault which transects the Baja California peninsula south of Ensenada (Jennings, 2010).

The primary seismic hazard for San Diego is the Rose Canyon fault zone which is located approximately 11.4 miles southwest of the site and is the ‘active’ seismogenic fault considered having the most significant effect at the site from a design standpoint.

4.2 Local Faulting

Our review of available geologic literature (Appendix A) indicates that there are no known active or potentially active faults transecting the site. The subject site is also not located within any State Mapped Earthquake Fault Zones or County of San Diego mapped fault zones. The nearest active fault is the Rose Canyon fault zone located approximately 11.4 miles west of the site (Blake, 2001; USGS, 2008).

The location of major regional faults with surface expression in the site area are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

4.3 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional active faults in Southern California that are mentioned above. The effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California.

4.3.1 Shallow Ground Rupture

As previously discussed, no faults are mapped transecting or projecting toward the site. Therefore, surface rupture hazard due to faulting is considered very low. Ground cracking due to shaking from a seismic event is not considered a significant hazard either, since the site is not located near slopes.

4.3.2 Mapped Seismic Hazard Zones

While the site is not located within a State-mapped Earthquake Fault Zone (EFZ), it is designated by the County of San Diego as occurring within a zone of low liquefaction potential.

4.3.3 Site Class

Per Section 20.3.1 of ASCE 7-16, for structures having fundamental periods of vibration less than 0.5s, Site Class may be determined in accordance to Section 20.3. It is understood that the proposed structures will have a fundamental period less than 0.5 s; therefore, we have utilized a Site Class C for determining spectral acceleration parameters. If it is determined by the structural engineer that the proposed structure has a fundamental period of vibration greater than 0.5 s, a site-specific response analysis will be required.

4.3.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 2 are the spectral acceleration parameters for the project determined in accordance with the 2022 CBC (CBC, 2022) and the SEA/OSHPD Web Application. Since the site has an S_1 value greater than 0.2g a ground motion hazard analysis was also performed according to ASCE 7-16 Section 11.4.8.

Table 2 - 2022 CBC Seismic Coefficients

CBC Categorization/Coefficient	Design Value (g)
Lat: 33.18016, Long: -117.217197	
Site Class C	
Mapped Spectral Response Acceleration at 0.2s Period, S_s	0.913
Mapped Spectral Response Acceleration at 1s Period, S_1	0.337
Short Period Site Coefficient at 0.2s Period, F_a	1.2
Long Period Site Coefficient at 1s Period, F_v	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.095
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.505
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	0.73
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.337

4.4 Secondary Seismic Hazards

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California (Figure 4). The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics.

Accordingly, design of the project should be performed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117A (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. The potential for secondary seismic hazards at the subject site is discussed below.

4.4.1 <u>Fault Rupture</u>

No known traces of faults, either active or inactive, are mapped as transecting the subject site on the published geologic maps we reviewed as part of our site evaluation. Based on our review, the potential for surface rupture due to faulting is negligible.

4.4.2 <u>Ground Lurching</u>

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are likely to be most severe where the thickness of soft sediments varies appreciably under structures. The potential for lurching can be minimized if the potentially compressible soils present on the site are removed and properly compacted in accordance with the recommendations of this report.

4.4.3 <u>Tsunamis, Seiches and Flooding</u>

Due to the site elevation and distance to large bodies of water (inland seas, large rivers, and oceans) from the site, the possibility of tsunamis and/or seismically-induced flooding is considered to be very low.

4.4.4 <u>Liquefaction and Dynamic Settlement</u>

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

San Diego County Hazard Overlay Maps indicate the site has a **<u>low</u>** risk of liquefaction. Additionally, our field observations, blow-count and laboratory data for the clayey Alluvium Deposits, and absence of groundwater, the potential for liquefaction is considered very low.

4.4.5 <u>Landslides</u>

Regional landslide susceptibility mapping by Tan and Giffen (1995) indicates the subject site lies in landslide susceptibility Subarea 3-1, characterized as being "generally susceptible" to landsliding. This refers to the hillside area abutting the property on the west. No ancient landslides or evidence of past slope instability have been mapped on the subject site. The crystalline bedrock underlying offsite slopes is not prone to landsliding. In addition, no evidence of landsliding was encountered during our site investigation. Based on our review of geotechnical literature, site topography, massive nature of crystalline bedrock, and our observations, landsliding is not a considered a constraint to the currently proposed development.

5.0 CONCLUSIONS

The following is a summary of the major geotechnical constraints or opportunities that may affect site development, based on the results of this geotechnical exploration:

- Existing onsite soils are generally suitable for reuse as fill during proposed grading provided they are free of oversize rock, organic material and debris.

- It is expected that decomposed granitic soils, if selectively mined on the site, may be suitable for use as specialized retaining wall backfill.

- Soils at the site surface consist of a clay-rich horizon developed on bedrock and undocumented artificial fill, which are adverse or relatively loose and potentially compressible and/or collapsible. Remedial grading will be required to mitigate this condition. Although removal depths will depend on finish design grades, resultant engineered fill should be established at least 3 feet below the base of proposed foundations. Preliminary grading recommendations are provided within Section 6 of this report.

- Onsite fill and alluvium material within the anticipated influence of remedial grading is considered rippable using conventional heavy grading equipment in good working condition.

- Onsite bedrock material is considered rippable using conventional heavy dozers and single-shank rippers to depths on the order of approximately 25 feet below existing grades. We do not anticipate grading will approach these depths. More definitive rippability studies can be performed to verify this estimate if warranted.

- The site does not occur within an active earthquake fault hazard zone, and the potential for surface fault rupture is considered negligible.

- The County of San Diego designates the liquefaction probability at the site as low. As no groundwater was encountered beneath the site, and alluvial material was predominately clays, the potential for liquefaction is considered negligible.

- The potential for landslides within the crystalline bedrock in offsite adjacent hillside areas is considered negligible.

- The onsite soils within the upper 5 feet generally possess a low to medium expansion potential and negligible sulfate exposure to concrete (Class S0).

- Groundwater was not encountered to depths explored and is not expected to be a constraint during grading, nor a long term impact to improvements.

- The deposits of alluvium and the Weathered Tonalite yield very slow rates of field infiltration, without application of a correction or safety factor. The rates are below a threshold feasible for use of an on-site infiltration BMP system.

6.0 PRELIMINARY RECOMMENDATIONS

6.1 General Discussion

Proposed site development appears feasible from a geotechnical/geologic viewpoint provided our recommendations included in this report are incorporated into design and construction phases of development.

6.2 General Earthwork Considerations

For planning purposes, earthwork should be performed in accordance with the General Earthwork and Grading Specifications attached in Appendix F and the recommendations below. The recommendations contained in Appendix F are general grading specifications provided for typical grading projects and some of the recommendations may not be strictly applicable to this project. Placement and compaction of fill should be performed in accordance with jurisdiction requirements under the fulltime observation and testing of the geotechnical consultant.

6.2.1 Demolition

Prior to grading, all existing improvements including foundation, pavement and underground utility improvements should be demolished and removed along with any heavy vegetation including tees and/or root balls, and the like. Resulting debris and vegetation should be disposed of offsite.

6.2.2 Site Preparation and Remedial Grading

Existing earth deposits underlying the eastern half of the site are expected to consist of older fill and/or weathered alluvium. All existing fill any loose/compressible earth materials (weathered alluvium) will require complete removal within the influence of planned structural improvements (i.e., buildings, WQMP vaults, etc.). The slope area spanning the central area of the site is expected to consist of weathered bedrock. Earth materials generated during remedial grading are expected to be suitable for use as new engineered fill.

The structural influence of new fill is defined as that which extends beyond the perimeter of planned structural improvements at a downward angle away from lowest foundation element, projected at 1:1 (h:v). Removal limits should extend horizontally a minimum of 5 feet from the lower outside edge of footings (including columns connected to the buildings), or a distance equal to the depth of overexcavation below the footings, whichever is more distal. New structural fills are to be placed upon on competent bearing in-situ deposits within removal bottoms.

Where exposed, alluvial materials should possess a minimum in-situ density of 85 percent relative compaction (based on ASTM D1557). Where bedrock is exposed, all residual soil should be removed to competent weathered bedrock. Acceptability of all removal bottoms should be reviewed by the project Geotechnical Engineer or his representative in the field. Removal dimensions and elevations should be documented by the contractor for inclusion within a final geotechnical report of rough grading.

Areas adjacent to existing perimeter improvements, including roadways and buildings, may require special monitoring. Temporary cuts in should be established no steeper than 1:1 (h:v). Cuts in dense formational material can be steeped to no greater than ¾:1 (h:v). If encountered, any friable earth materials may require additional layback or alternative measures.

6.2.3 Cut and Cut/Fill Transition Lots

A possible transition condition may exist between bedrock and alluvium along the existing slope transecting the central area of the site. Should such condition arise, these areas may warrant deeper removal and recompaction in order to reduce potential impacts of statically and seismically induced-settlement. In general, overexcavation of transition conditions should extend across an entire lot, and achieve a minimum depth of 5 feet below pad elevation or one-third of the maximum fill thickness beneath the proposed structure, whichever is deeper. Overexcavation bottoms should be sloped as needed to prevent the accumulation of subsurface water.

The upper eastern area of the site is expected to be underlain entirely by bedrock at the surface. Remedial removals in these areas will require complete removal of clayey residual soils horizons to expose competent weathered bedrock.

6.2.4 Structural Fills

Onsite earth materials (bedrock, alluvium, old fill) are expected to be generally suitable for re-use as compacted fill, provided they are free of oversize rock, debris and organic matter. Although not anticipated, fills placed within 10 feet of finish pad grades or slope faces should contain no rocks over 12 inches in maximum dimension.

Areas to receive structural fill and/or other surface improvements should be scarified to a minimum depth of 8 inches, moisture conditioned to at least optimum moisture content, and recompacted. Fill soils should be placed at a minimum of 90 percent relative compaction (based on ASTM D1557) and at near or above optimum moisture content.

Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant. The optimum thickness of fill lifts to produce a uniformly compacted fill will depend on the material types and compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in thickness.

Fills placed on slopes steeper than 5:1 (horizontal:vertical) should be benched into dense soils (see Appendix F for benching detail). Benching should be of sufficient depth to remove all loose material. A minimum bench height of 2 feet into approved material should be maintained at all times. The outer portion of fill slopes should be either overbuilt by 2 feet (minimum) and trimmed back to the finished slope configuration or compacted in vertical increments of 5 feet (maximum) by a sheepsfoot roller as the fill is placed. The slope face should then be track walked by dozers of appropriate weight to achieve the final configuration and compaction out to the slope face.

6.2.5 Shrinkage and Bulking

Volume changes in excavated onsite earth materials is expected to vary depending on material types, density, insitu moisture content, location, and compaction effort. As the in-place and compacted densities of soil materials are expected to vary, an accurate overall determination of shrinkage and bulking cannot be made. If possible, we recommend site grading include a balance area to accommodate variabilities in import quantity. Based on our experience with similar materials, the following values are provided as guidelines:

Table 3 - Shrinkage and Bulking Estimates

Shrinkage (Older Alluvium and Fill)	Approximately 5 percent
Bulking (Un-weathered Bedrock)	Approximately 8 to 10 percent

6.2.6 Import Soils

If import soils are needed to establish the site design elevations, it should be granular in nature, relatively free of organic material, have an expansion index less than 21 (per ASTM Test Method D4829), and have a low corrosion impact to the proposed improvements. Import soils, if needed, and potential borrow sites should be evaluated by the geotechnical consultant prior to being imported to the site.

6.2.7 Trench Excavation and Backfill

Utility trenches should be backfilled with compacted fill in accordance with the *Standard Specifications for Public Works Construction*, (“Greenbook”), 2018 Edition. Fill material should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557).

Excavation of utility trenches should be performed in accordance with the project plans, specifications and the California Construction Safety Orders (latest Edition). The contractor must be responsible for providing a "competent person" as defined in Article 6 of the California Construction Safety Orders. Contractors should be advised that sandy soils (such as fills generated from the onsite alluvium) could make excavations particularly unsafe. All safety precautions should be properly implemented at all times. In addition, excavations at or near the toe of slopes and/or parallel to slopes may be highly unstable due to the increased driving force and load on the trench wall. Spoil piles from the excavation(s) and construction equipment should be kept away from the sides of the trenches. Leighton does not consult in the area of safety engineering.

6.3 Preliminary Foundation Design Parameters

Our recommendations below are provided as minimum and do not supersede applicable building codes or requirements of the project structural engineer. Grading and foundation plans should be reviewed to confirm foundation design parameters provided below.

6.3.1 Minimum Footing Dimensions and Bearing Capacity – Light Structures

Foundations and slabs should be designed in accordance with structural considerations, the seismic parameters of Section 4.3 and the recommendations presented in the 2022 California Building Code (CBC) or applicable Code at the time of design. In addition, post-tension or other stiffened foundation/slab systems may be considered on this site due to the relatively high potential for seismic activities in this area and associated ground deformation. These foundation systems, combined with the recommended remedial grading, will reduce the potential impact caused by the design level earthquake.

For budgetary and preliminary planning purposes, foundations should be designed for settlements, expansive soils and the following preliminary foundation parameters:

Allowable bearing:	2,000 psf (pounds per square foot) for a minimum 18 inches embedment into compacted fill.
Lateral bearing:	200 psf/foot up to 2,000 psf maximum lateral bearing.
Sliding coefficient:	0.30 for soil against structural concrete.
Effective PI:	15 (estimated)

The bearing pressure value may be increased by 200 psf for each additional foot of embedment and each additional foot of width to a maximum vertical bearing value of 3,000 psf. These bearing values may be increased by one-third when considering short-term seismic or wind loads. A sliding coefficient friction of 0.30 should be applied to the design of foundation elements.

Conventional footings/slab may be enhanced by structurally tying the slabs-on-grade to the perimeter and interior footings as designed by the project Structural Engineer.

6.3.2 Interior Floor Slab Design

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a very low expansion potential. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

- *Subgrade Moisture Conditioning:* The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing the moisture vapor retarder, steel or concrete.

- *Moisture Retarder:* A minimum of 10-mil moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder. The moisture barrier may be placed directly on subgrade provided gravel or other protruding objects that could puncture the moisture retarder are removed from the subgrade prior to placement. A heavier vapor retarder (such as 15 mil Stego Wrap) placed directly on prepared subgrade may also be used. Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

- *Concrete and Structural Design Thickness:* Slabs-on-grade should be designed by the structural engineer, but should be at least 5 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced slabs) should be No. 3 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing.

Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

6.4 Settlement

Settlement of onsite fill materials is expected to occur during and within 90 days following fill placement. However, following the placement of fill and construction of structures, additional settlement may occur due to: (a) new footing/foundation loads, (b) compression within the fill due to the effects of landscaping irrigation, (c) compression of the left-in-place alluvial soils, and (d) dynamic settlements below the removal depths. The settlements below are general estimates/guidelines and should be further verified based on actual structural loads/footing size and additional subsurface investigation performed for individual buildings.

6.4.1 Settlement from Building Loads

Settlement resulting from buildings located on compacted fill soils (minimum 90 percent relative compaction) is not expected to be significant and is estimated to be less than 1 inches (total) and ½ inch (differential) in 30 feet or the least dimension of the building, whichever is a greater distortion. The majority of settlement associated with building loads is anticipated to occur during construction as the load is applied. These settlements and angular distortions are for imposed building loads and do not include compression within the fill itself, underlying alluvium nor dynamic settlements.

6.4.2 Dynamic Settlement

The earth materials underlying the site consist of hard bedrock and dense alluvium that is not expected to undergo any significant amount of earthquake-induced settlement during the design seismic event. Planned remedial grading is also expected to reduce the effects and magnitude of the earthquake-induced settlement.

6.5 Footing Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural footings (retaining and decorative walls, building footings, etc.). This distance is measured from the outside bottom edge of the footing horizontally to the slope face (or to the face of a retaining wall) and should be a minimum of H/2, where H is the slope height (in feet). The setback should not be less than 7 feet and need not be greater than 10 feet for manufactured fill slopes and 20 feet for tall natural descending slopes.

Note that the soils within the structural setback area possess poor lateral stability and improvements (such as retaining walls, sidewalks, fences, pools, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a pier and grade-beam foundation system to support the improvement. The deepened footing should meet the setback as described above.

6.6 Lateral Resistance and Earth Pressures for Retaining Wall Design

Preliminary lateral earth pressure values for level or sloping backfill are presented in Table 4 below. The provided values are recommended for planning of walls backfilled with very low expansive soils (EI ≤ 21 per ASTM Test Method D4829). Leighton should review the retaining wall plans and update design parameters as needed. The following recommendations are applicable for short retaining walls (height under 6 feet).

Table 4 - Lateral Earth Pressures[1]

Conditions	Equivalent Fluid Weight (pcf)	
	Level Backfill[2]	2:1 Slope Backfill
Active	40	70
At-Rest	60	95
Passive[3]	350 (Maximum of 3 ksf)	200 (sloping down)

Notes:
(1) Assumes drained condition
(2) Assumes a level condition behind and in front of wall foundation of project.
(3) Maximum passive pressure = 3,000 psf, level conditions.

Unrestrained (yielding) cantilever walls should be designed for the active equivalent fluid weight value provided they are free draining. In the design of walls restrained from movement at the top (non-yielding) such as basement or elevator pit/utility vaults, the at-rest equivalent fluid weight value should be used.

Foundations placed in properly compacted fill soils may be designed using a coefficient of friction of 0.30 (total frictional resistance equals coefficient of friction [concrete on soil] times the dead load). A design passive resistance value of 350 psf per foot of depth (with a maximum lateral passive value of 3,000 psf) may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance. The total lateral resistance value may be increased by one-third when considering loads of short duration such as wind or seismic forces.

These above equivalent fluid pressures do not include the effect of earthquake surcharges in the backfill. Based on recent studies (Sitar, et. al., 2013), a uniform pressure distribution of 11H (psf) or incremental earth pressures of 24 pounds-per-cubic-foot (pcf) may be considered to estimate seismic lateral pressures acting against retaining walls. These pressures need only to be applied to walls supporting more than 6 feet of level backfill per the 2022 California Building Code.

Should a sloping backfill other than a 2 to 1 (horizontal to vertical) be constructed above the wall (or a backfill is loaded by an adjacent surcharge load), the equivalent fluid weight values provided above should be evaluated on an individual case basis by the geotechnical engineer. All retaining wall structures should be provided with appropriate drainage and waterproofing. The outlet pipe should be sloped to drain to a suitable outlet.

Foundations for retaining walls in competent formational soils or properly compacted fill should be embedded at least 18 inches below lowest adjacent grade with proper footing setback. At this depth, an allowable vertical bearing capacity of 2,000 psf may be assumed. This vertical bearing value may be increased 200 psf per foot of additional depth to a maximum bearing of 3,000 psf for retaining walls.

Where applicable, basement walls or cantilever retaining walls (less than 6 feet in height) should be designed for lateral earth pressures as described in this section. The magnitude of these pressures depends on the amount that the wall can yield horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance.

A seismic increment load of 28 pcf should be added to the active case when checking seismic stability of walls over 6 feet tall.

6.7 Site Soil Corrosivity

6.7.1 Ferrous Corrosivity

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February, 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table 5.

Table 5 - Relationship between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures. If buried metallic materials are to be used onsite, then pH and Resistivity tests should be performed during grading to evaluate the corrosivity of onsite soils.

6.7.2 <u>Sulfate Attack</u>

Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland Cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Potentially high sulfate content could also cause corrosion of the structural concrete. In accordance with American Concrete Institute (ACI 318-08), Table 6 below provides exposure categories based on water-soluble sulfate in soil. Our testing indicate site soil have a negligible exposure to concrete.

Table 6 - Relationship between Sulfate Concentration and Exposure Category

Sulfate In Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	Exposure
0-150	0.00 - 0.10	Negligible
150-1,500	0.10 - 0.20	Moderate[1]
1,500-10,000	0.20 - 2.00	Severe
>10,000	Over 2.00	Very Severe

[1]= seawater

6.7.3 Corrosivity Test Results

To evaluate corrosion potential of soils sampled from this site, we tested a bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below

Table 7 - Results of Corrosivity Testing

Boring Number	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
LB-3	0 to 5	111	220	8.4	630

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

The results are discussed as follows.

- *Sulfate Exposure:* Based on our previous experience and Table 19.3.1.1 of ACI 318-14, in our opinion, sulfate exposure should be considered “negligible” with an Exposure Class S0 for soils samples at the site. Based on Table 19.3.2.1 of ACI 318-14, for this Exposure Category S0, there are no mix-design restrictions for sulfate exposure other than f'_c (28-day compressive strength) of at-least (≥) 2,500 pounds-per-square-inch (psi) for structural concrete. Note that this is based solely on tested site soils.

- *Ferrous Corrosivity:* As shown above, minimum soil resistivity of 630 ohm-centimeters was measured in our laboratory test. Based on resistivity correlations presented above, corrosion potential to buried steel should be characterized as “very severely corrosive” at the site.

Ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site soils.

6.8 Drainage

All drainage should be directed away from structures, pavements and tops of slopes by means of approved permanent/temporary drainage devices. Adequate storm drainage of any proposed pad should be provided to avoid wetting of foundation and pavement subgrade soils. Irrigation adjacent to buildings should be avoided wherever possible. Pavements should be separated from irrigated areas by embedded concrete curbs extending below pavement base. Below grade stormwater percolation or retention facilities (if planned) should be located a minimum distance of 15 feet from structural footings and 25 feet from descending slopes. Provisions to prevent the lateral migration of subsurface saturation below adjacent structures or into descending slopes may be needed.

6.9 Preliminary Pavement Design

The following preliminary pavement design recommendations are intended for planning purposes only. Our recommendations are based on an assumed R-Value of 20.

Table 8 - Preliminary Asphalt Pavement Section Thickness

TI	Asphaltic-Concrete (AC) Thickness (inches)	Class 2 Aggregate Base (AB) Thickness (inches)
5.0	3.0	8.0
6.0	4.0	9.0
7.0	4.0	12.0

Appropriate Traffic Index (TI) data should be selected by the project civil engineer or traffic engineering consultant and appropriate R-value of the subgrade soils will need to be determined after completion of rough grading to finalize the pavement design. Final pavement sections should be in general accordance with local, county and industry standards. The Caltrans pavement section design calculations were based on a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Portland cement concrete should be used, rather than asphalt, in point and impact load areas such as loading docks and trash truck bin loading areas.

Subgrade soils in the upper 12 inches should be properly compacted to at least 95 percent relative compaction (ASTM D1557) and should be moisture-conditioned to optimum or slightly above optimum, and kept in this condition until the pavement section is constructed. Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density as determined by ASTM D1557. Asphalt concrete and aggregate base should conform to *Caltrans Standard Specifications* (Latest Edition) or Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book).

For preliminary planning purposes, fire lanes may be constructed of Portland Cement Concrete (PCC) with a minimum thickness of 6.0 inches over 4 inches of Class 2 Aggregate Base assuming light axle loads and an average daily truck traffic (ADTT) of less than 500. For medium/heavy axle loads and an ADT of 500 or more, Leighton should be consulted to provide additional recommendations. Class 2 aggregate base should be compacted at 95 percent relative compaction. The upper 6 inches of the underlying subgrade soils should also be compacted to at least 95 percent relative compaction (ASTM D1557).

The above PCC sections should be re-evaluated following the provision of the precise grading plans. We recommend that the ADT be confirmed by the project civil designer or traffic consultant prior to completion of the project.

Additional details should be added to the plans indicating the pavement thickness transitions, pavement joint dowels, expansion joints and saw-cut joints. Use of concrete cutoff or edge barriers should be considered at the perimeter of the common parking or driveway areas when they are adjacent to either open (unfinished) or landscaped areas.

7.0 GEOTECHNICAL REVIEW

Geotechnical review is of paramount importance in engineering practice. The poor performance of many foundation and earthwork projects have attributed to inadequate construction review. We recommend that Leighton be provided the opportunity to review the following items.

7.1 Plans and Specifications

Leighton should review the project rough-grading plans as well as foundation and specifications prior to release for bidding and construction. Such review is necessary to evaluate whether the geotechnical recommendations have been effectively incorporated in plans and other construction documents. Review findings should be reported in writing by the geotechnical engineer. Additional subsurface evaluation cannot be precluded dependent upon the results of a detailed plan review.

7.2 Future Investigations

It is our opinion that the existing studies completed for this site are sufficient for design of the current development. Based on our future review of Tentative Tract Map or specific foundation/grading plans, additional studies cannot be precluded to address certain areas substantially differing from the current plans.

7.3 Construction Review

Field observation and testing should be performed by Leighton during grading and construction. It is possible that the subsurface conditions exposed during construction may vary from those encountered in our exploratory borings and other explorations. Reasonably continuous construction observation and review during site grading and foundation installation allows for evaluation of the actual soil conditions and the ability to provide appropriate revisions during grading and construction, if required.

Site preparation, removal of unsuitable soils, geologic observation and mapping of all earthwork excavations, laboratory approval of imported earth materials, fill placement, foundation installation and other site geotechnical-related operations should be observed and tested by Leighton. Additional laboratory tests of subsurface materials to confirm compacted dry density and moisture content, consolidation potential corrosion potential, expansion potential, and resistance value (R-value) should be performed during or prior to grading.

8.0 LIMITATIONS

This preliminary report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions and recommendations presented in this report are based on the assumption that we (Leighton Consulting, Inc.) will provide geotechnical observation and testing during construction as the Geotechnical Engineer of Record for this project. Please refer to Appendix H, GBA's *Important Information About This Geotechnical-Engineering Report*, prepared by the Geoprofessional Business Association (GBA) presenting additional information and limitations regarding geotechnical engineering studies and reports.

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Project: 13102.001R	Eng/Geol: JLH
Scale: 1 " = 2,000 '	Date: January 2023

Base Map: ESRI ArcGIS Online 2022
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Proposed Vista II Condominium Development
145 Hannalei Drive
Vista, California

FIGURE 1

Leighton

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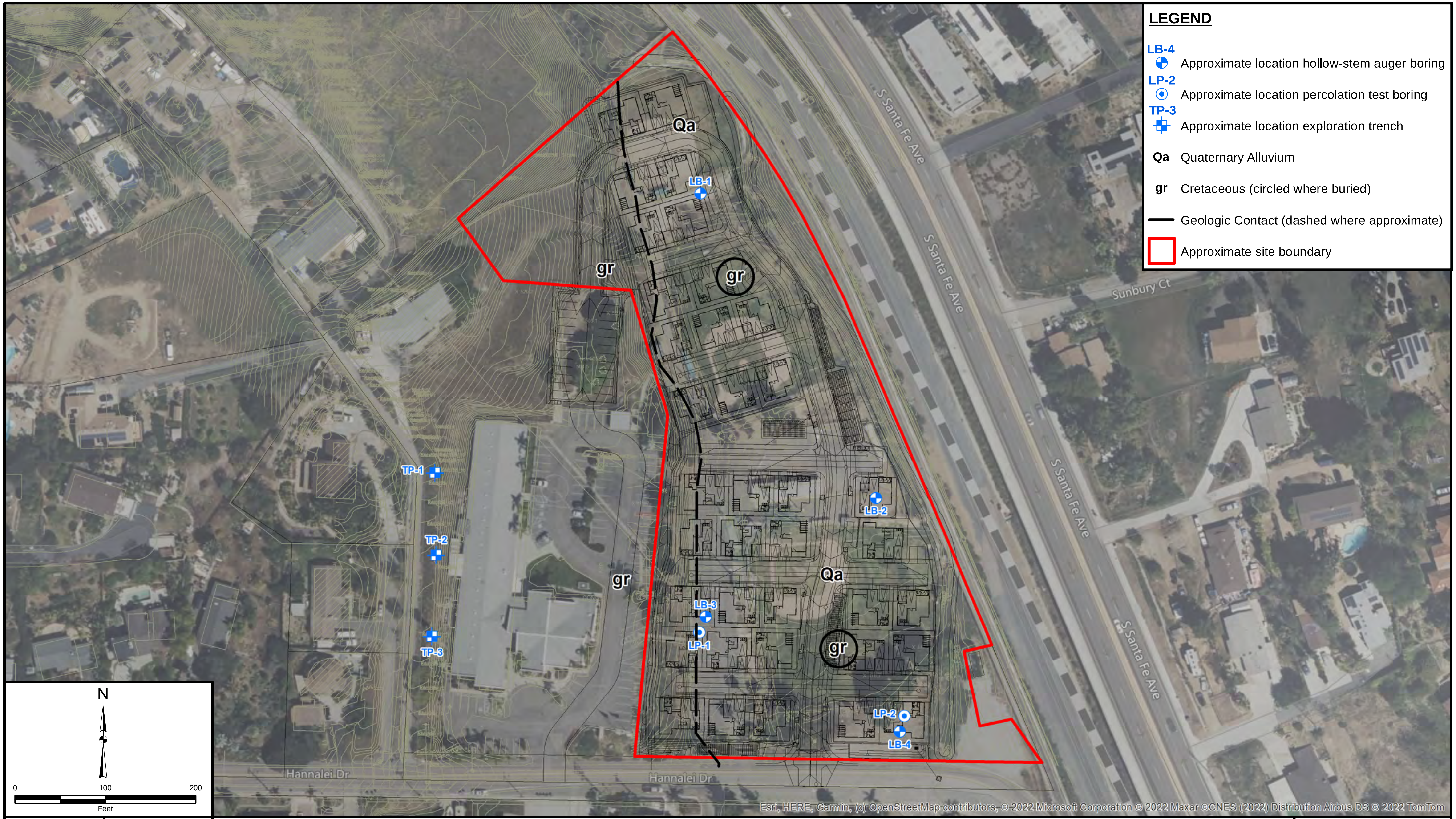

LEGEND

LB-4 ●	Approximate location hollow-stem auger boring
LP-2 ○	Approximate location percolation test boring
TP-3 ✚	Approximate location exploration trench
Qa	Quaternary Alluvium
gr	Cretaceous (circled where buried)
—	Geologic Contact (dashed where approximate)
🟥	Approximate site boundary

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Scale: 1 " = 100 '	Date: January 2023

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Thematic Information: Leighton
Author: Leighton Geomatics (btran)

GEOTECHNICAL EXPLORATION MAP

Proposed Vista II Condominium Development
145 Hannalei Drive
Vista, California

FIGURE 2

Leighton

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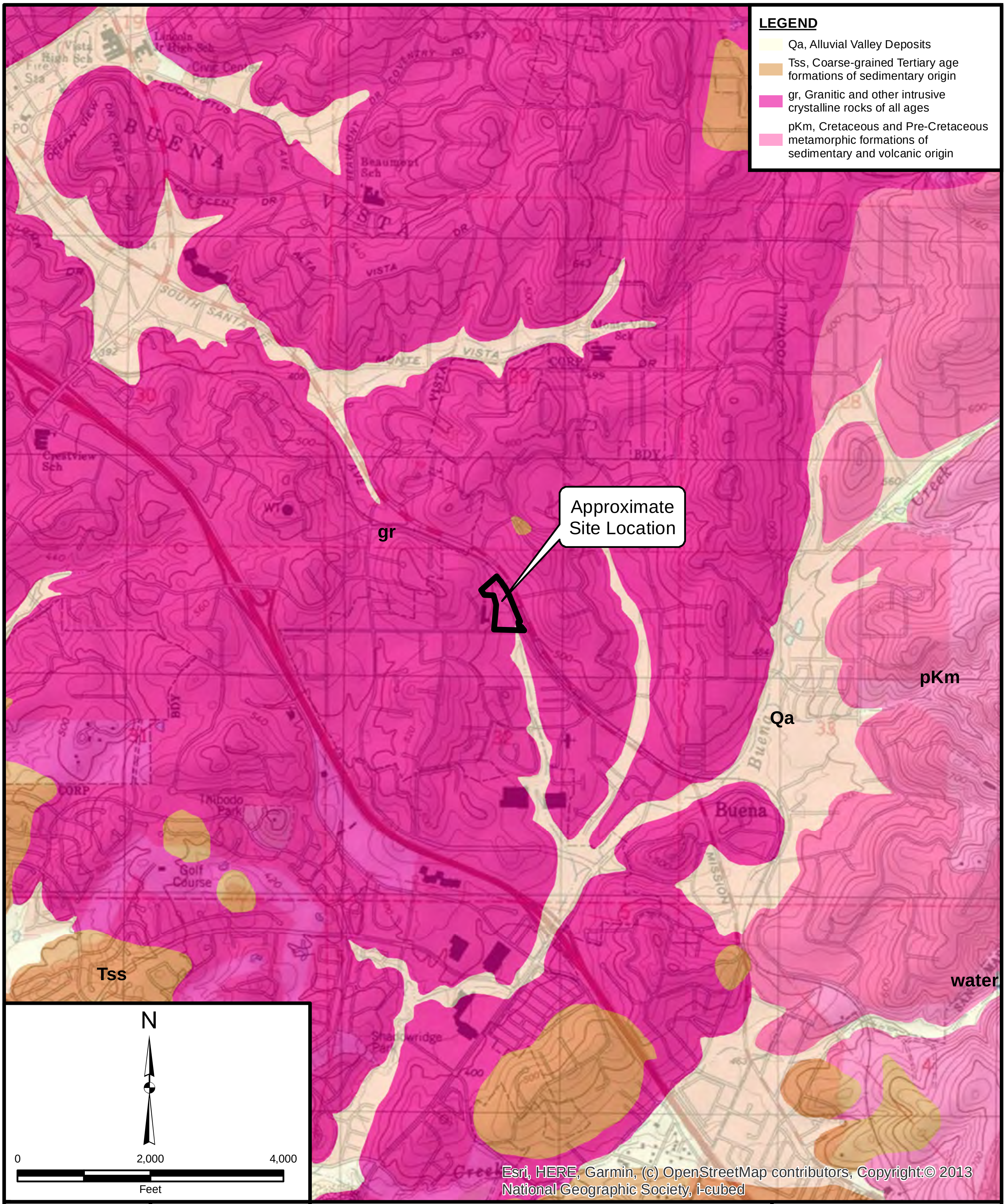

LEGEND

- Qa, Alluvial Valley Deposits
- Tss, Coarse-grained Tertiary age formations of sedimentary origin
- gr, Granitic and other intrusive crystalline rocks of all ages
- pKm, Cretaceous and Pre-Cretaceous metamorphic formations of sedimentary and volcanic origin

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Project: 13102.001R	Eng/Geol: JLH
Scale: 1 " = 2,000 '	Date: January 2023

Base Map: ESRI ArcGIS Online 2022
Thematic Information: Leighton, USGS
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Proposed Vista II Condominium Development
145 Hannalei Drive
Vista, California

FIGURE 3

Leighton

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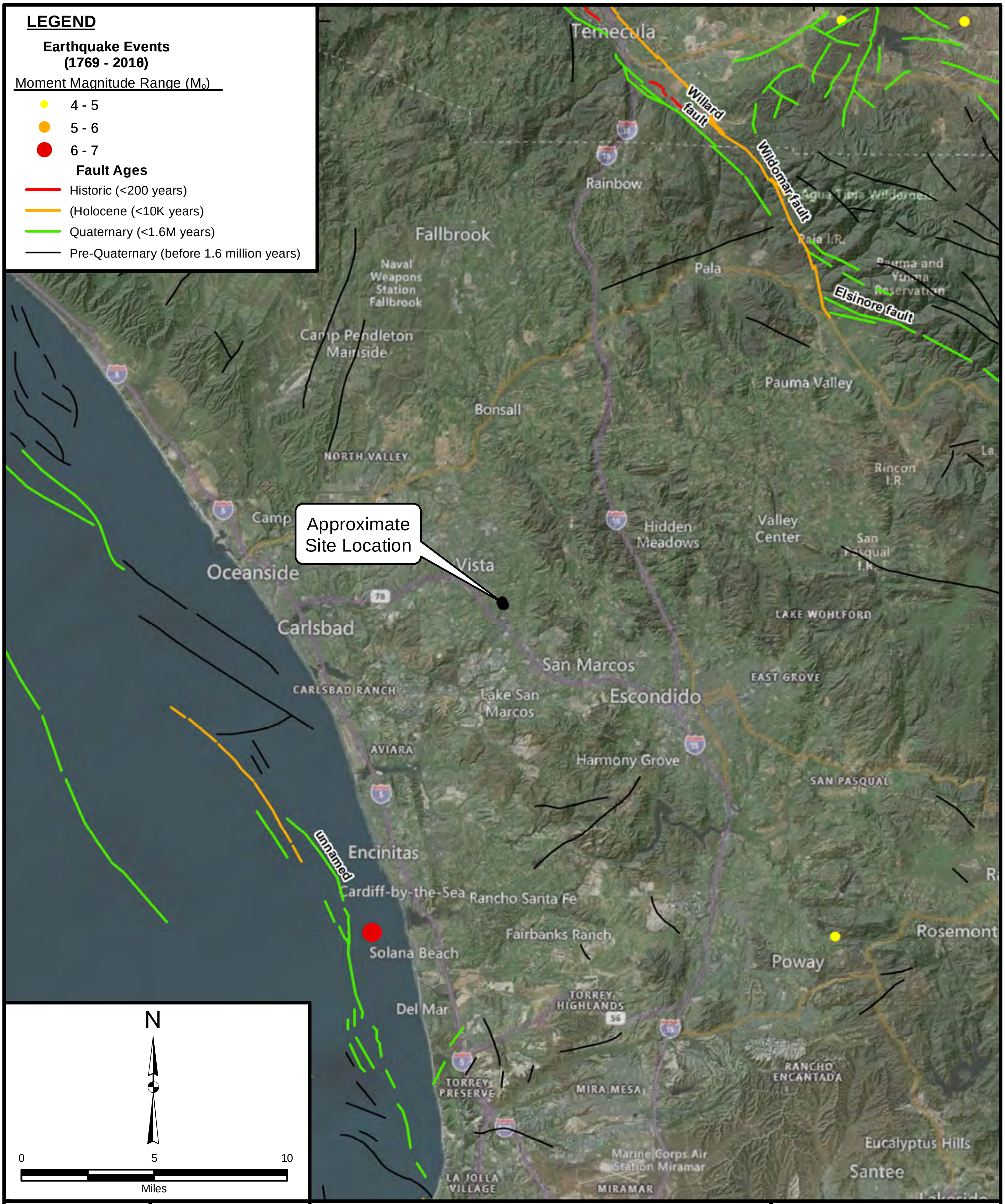

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Eng/Geol: JLH

Scale: 1 " = 5 miles

Date: January 2023

Base Map: ESRI ArcGIS Online 2022
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (btran)

REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

Proposed Vista II Condominium Development
145 Hannalei Drive
Vista, California

FIGURE 4

Leighton

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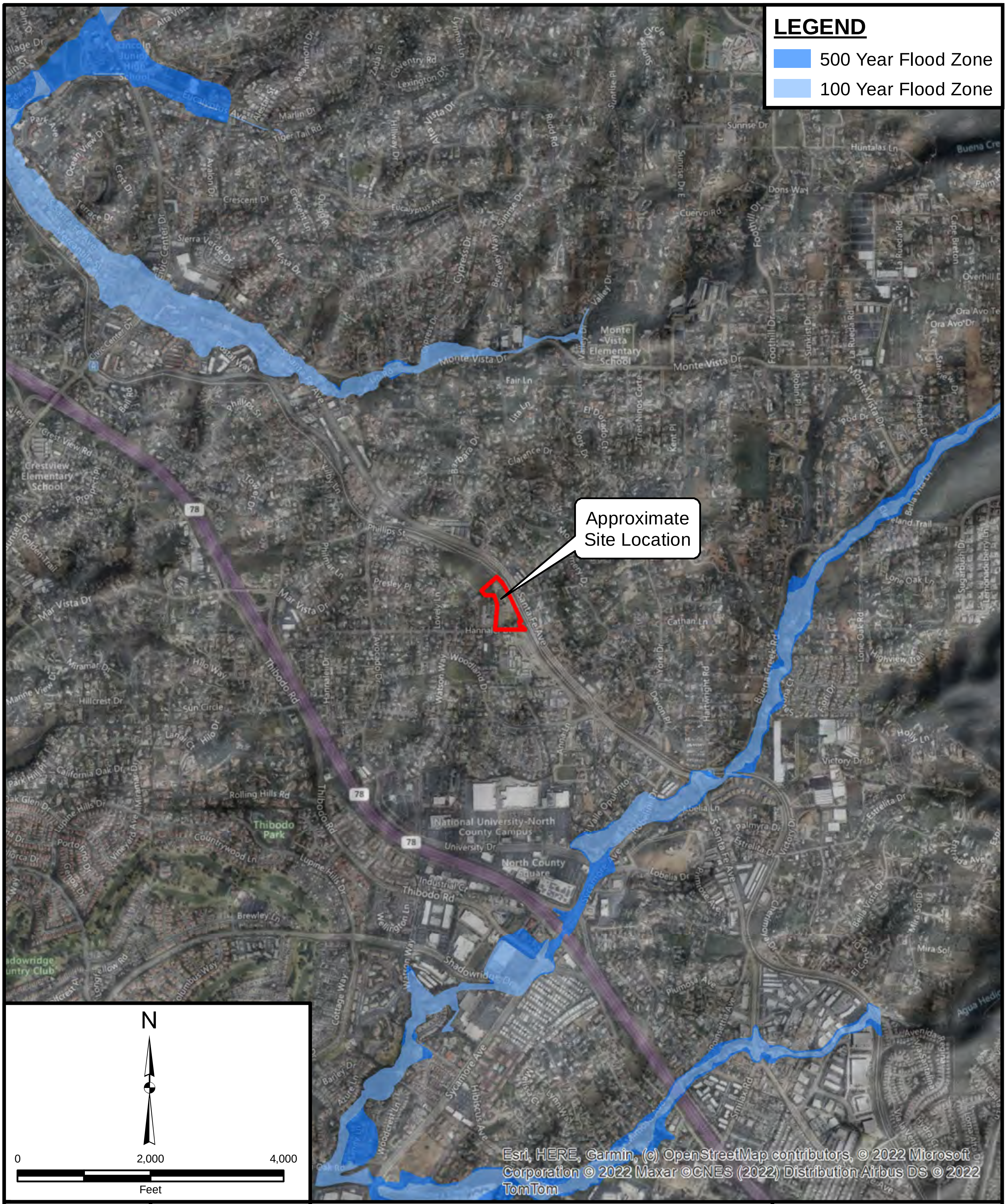

Project: 13102.001R
Eng/Geol: JLH
Scale: 1 " = 2,000 '
Date: January 2023

Base Map: ESRI ArcGIS Online 2022
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (btran)

FLOOD HAZARD ZONE MAP

Proposed Vista II Condominium Development
145 Hannalei Drive
Vista, California

FIGURE 5

Leighton

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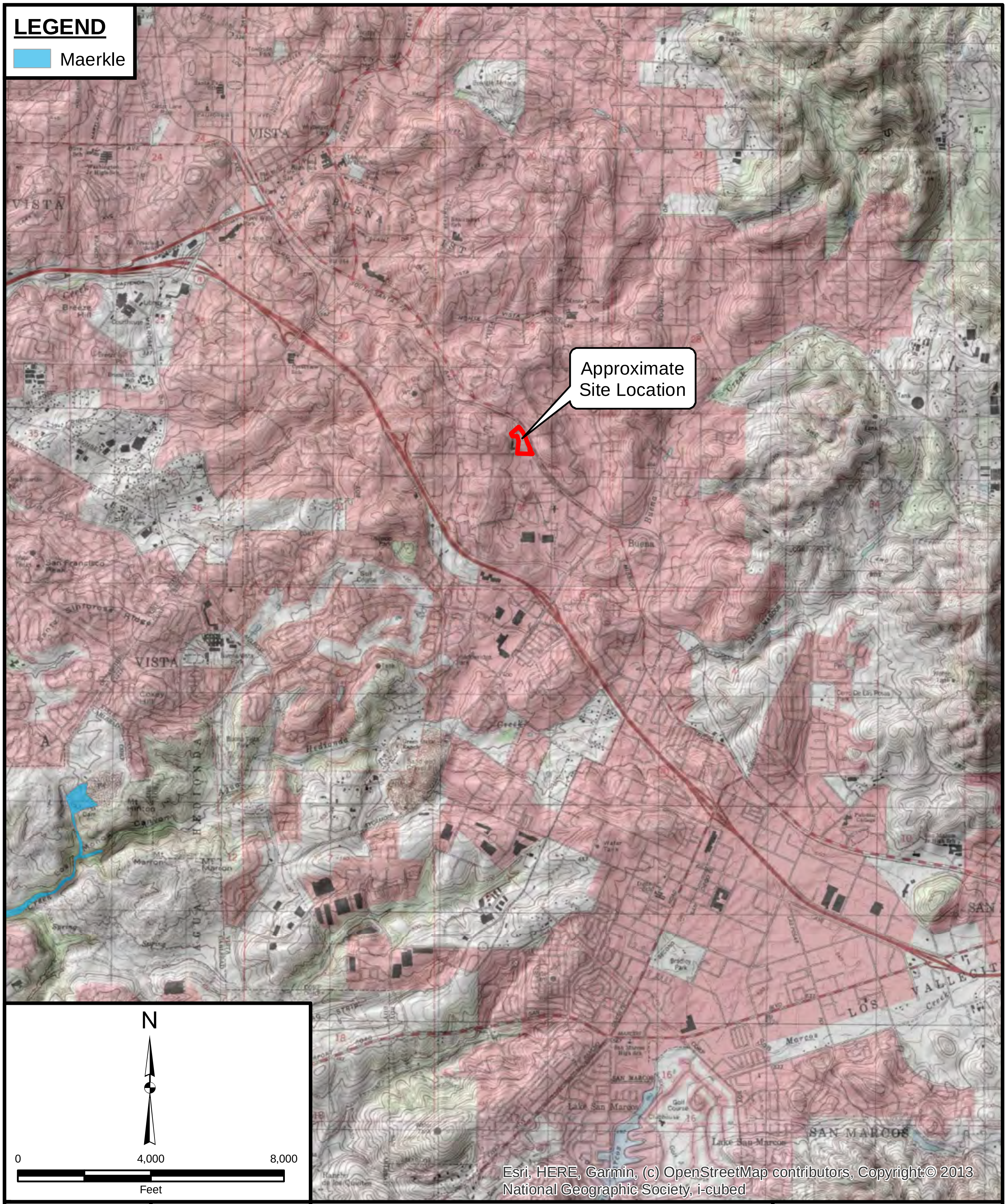

Esri, HERE, Garmin, (c) OpenStreetMap contributors, Copyright © 2013 National Geographic Society, i-cubed

Project: 13102.001R	Eng/Geol: JLH
Scale: 1 " = 4,000 '	Date: January 2023

Base Map: ESRI ArcGIS Online 2022
Thematic Information: Leighton, CGS
Author: Leighton Geomatics (btran)

DAM INUNDATION MAP

Proposed Vista II Condominium Development
145 Hannalei Drive
Vista, California

FIGURE 6

Leighton

Map Saved as V:\Drafting\13102\001\Maps\13102-001_F06_DIM_2022-10-28.mxd on 5/6/2021 3:39:13 PM

APPENDIX A

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APPENDIX A

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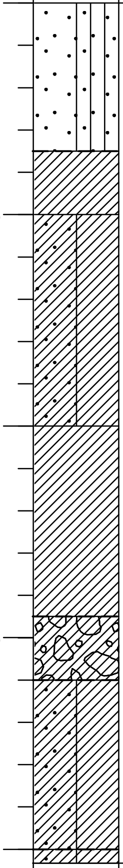
APPENDIX B-1

Logs of Exploratory Borings

GEOTECHNICAL BORING LOG LB-1

Project No.	13102.001	**Date Drilled**	4-26-21
Project	Warmington Vista - Santa Fe	**Logged By**	KMD
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~497'
Location	See Figure 2, Exploration Location Map	**Sampled By**	KMD



Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Bulk Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								SP-SM	@0': Coarse Decomposed Granitics at surface. **Artificial Fill, undocumented (Afu):** SAND with silt, reddish brown, moist to very moist, fine to coarse sand, some asphalt fragments.	
				B-1							AL
									CL	**Quaternary Alluvium (Qa):** @3.5': Sandy CLAY, dark brown, moist, fine sand.	
	5			R-1		7 13 18	115	15	SC-CL	@5': Interbedded dark brown Sandy CLAY and light orangeish brown Clayey SAND, moist, very stiff/medium dense, predominantly fine sand, some medium sand, trace coarse sand.	
	10			R-2		9 21 31	115	16	CL-SM	@10': Sandy CLAY, medium to dark brown, moist, hard, faintly laminated, high sand comment in some laminations, fine to medium sand, few coarse sand, trace charcoal.	
				B-2							AL, SA
	15			R-3		15 50/5"	114	14	SC-CL	**Decomposed Granitics (DG):** @15': Clayey SAND to SAND with clay, reddish brown, moist, very dense, predominantly fine sand, with beds of medium to coarse sand, very weathered fine granitic gravels at basal contact.	
									SC-CL	**Cretaceous Tonalite (Kt), heavily weathered zone:** @16': Clayey SAND to Sandy CLAY, reddish brown, very moist, medium-grained, heavily fractured (clay-filled fractures).	
	20			R-4		50/4"			SC-CL	**Cretaceous Tonalite (Kt):** @20': Clayey SAND to Sandy CLAY, light grey, moist, hard, coarser grained. **Total Depth: 20.33' - Practical Refusal on Hard Bedrock** **No Groundwater Encountered During Drilling** **Boring Backfilled with Soil Cuttings to Ground Surface.**	
	25										
	30										

SAMPLE TYPES:

- **B BULK SAMPLE**
- **C CORE SAMPLE**
- **G GRAB SAMPLE**
- **R RING SAMPLE**
- **S SPLIT SPOON SAMPLE**
- **T TUBE SAMPLE**

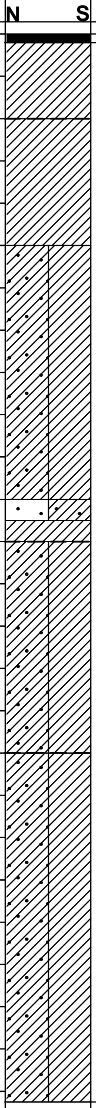
TYPE OF TESTS:

- **-200 % FINES PASSING**
- **AL ATTERBERG LIMITS**
- **CN CONSOLIDATION**
- **CO COLLAPSE**
- **CR CORROSION**
- **CU UNDRAINED TRIAXIAL**
- **DS DIRECT SHEAR**
- **EI EXPANSION INDEX**
- **H HYDROMETER**
- **MD MAXIMUM DENSITY**
- **PP POCKET PENETROMETER**
- **RV R VALUE**
- **SA SIEVE ANALYSIS**
- **SE SAND EQUIVALENT**
- **SG SPECIFIC GRAVITY**
- **UC UNCONFINED COMPRESSIVE STRENGTH**

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-2

Project No.	13102.001	**Date Drilled**	4-26-21
Project	Warmington Vista - Santa Fe	**Logged By**	KMD
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~481'
Location	See Figure 2, Exploration Location Map	**Sampled By**	KMD



Elevation Feet	Depth Feet	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							CL	@0': 2-inches Asphalt over Subgrade. **Artificial Fill, undocumented (Afu):** @0.2': Sandy CLAY, very dark brown, moist, plastic, fine sand.	
			B-1	Bulk				CL	**Quaternary Alluvium (Qa):** @2': Sandy CLAY, brown to dark brown, moist, plastic, fine sand.	AL, EI
	5		R-1	Driven	7 7 19	123	14		@5': Sandy CLAY to CLAY with sand, dark brown, very moist, very stiff, predominantly fine sand, some medium sand, plastic, regions with some to many carbonate veins.	
	10		R-2	Driven	8 16 21	114	17		@10': Sandy CLAY, dark reddish brown, very moist, very stiff, fine sand. @11': Thin interbeds of SAND with clay to Clayey SAND.	
								CL		
			B-2	Bulk				SC-CL	**Cretaceous Tonalite (Kt), heavily weathered zone:** @12': Sandy CLAY to Clayey SAND, greenish grey, moist, predominantly fine-grained, silty.	SA
	15		R-3	Driven	50/5"			SC-CL	@15': Sandy CLAY to Clayey SAND, light brownish grey, moist, very weathered, fine grained.	
									Cretaceous Tonalite (Kt):	
	20		R-4	Driven	50/2"			SC-CL	@20': Sandy CLAY to Clayey SAND, dark reddish brown, moist, hard, less weathered.	
	25		R-5	Driven	50/3"				@25': Sandy CLAY to Clayey SAND, hard, less weathered. **Total Depth: 25.25' - Practical Refusal on Hard Bedrock** **No Groundwater Encountered During Drilling** **Boring Backfilled with Soil Cuttings and Ground Surface Patched with Cold-Mix Asphalt.**	
	30									

SAMPLE TYPES:

B	BULK SAMPLE
C	CORE SAMPLE
G	GRAB SAMPLE
R	RING SAMPLE
S	SPLIT SPOON SAMPLE
T	TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-3

Project No. 13102.001
Project Warmington Vista - Santa Fe
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer
Location See Figure 2, Exploration Location Map

Date Drilled 4-26-21
Logged By KMD
Hole Diameter 8"
Ground Elevation ~487'
Sampled By KMD

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Bulk Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								SC	@0': 2-inches Asphalt over Subgrade.	
									CL	**Artificial Fill, undocumented (Afu):** @0.2': Clayey SAND, reddish brown, moist, fine to coarse sand.	
				B-1						**Quaternary Alluvium (Qa):** @1': Sandy CLAY, brown to dark brown, moist, plastic, fine sand.	
	5			R-1		11 50/6"	111	13		@5': Sandy CLAY, brown to dark brown, moist, fine sand, sharp contact below.	
				B-2					SC-CL	**Cretaceous Tonalite (Kt), heavily weathered zone:** @6': Sandy CLAY to Clayey SAND, orangeish brown, moist, heavily weathered, medium to coarse grained, significant weathering of mafic minerals to clay.	AL, DS Mx, SA
	10			R-2		50/3"	123	4		@10': Grey-brown, weathered, fine-grained, foliated.	
									SC-CL	**Cretaceous Tonalite (Kt):**	
	15			R-3		19 50/1"				@15': Sandy CLAY to Clayey SAND, grey with few orangeish FeO-stained zones, less weathered, fine to medium grained.	
	20			R-4		50/3"				@20': Greyish brown, moist, harder, medium to coarse grained.	
										Total Depth: 20.25' - Practical Refusal on Hard Bedrock **No Groundwater Encountered During Drilling** **Boring Backfilled with Soil Cuttings and Ground Surface Patched with Cold-Mix Asphalt.** **Percolation Well Installed Adjacent to LB-3, Testing 6' to 10' bgs.**	
	25										
	30										

SAMPLE TYPES:

- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG LB-4

Project No.	13102.001	**Date Drilled**	4-26-21
Project	Warmington Vista - Santa Fe	**Logged By**	KMD
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~476'
Location	See Figure 2, Exploration Location Map	**Sampled By**	KMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								SP	**Artificial Fill, undocumented (Afu):** @0': SAND, light olive, very moist, fine to medium sand.	
									CL	**Quaternary Alluvium (Qa):** @1.5': CLAY with sand to Sandy CLAY, dark brown, very moist, fine sand, few medium sand, grading slightly sandier by 5'.	
				B-1							
	5			R-1		4 10 10	115	11	SC-CL	@5': Interbedded Sandy CLAY and Clayey SAND, brown to dark brown, moist, stiff/medium dense, predominantly fine sand, some medium to coarse sand.	AL, EI
	10			R-2		10 22 33	123	14	CL	@10': CLAY with sand, dark brown, moist to very moist, hard, predominantly fine sand, few medium sand, massive, plastic.	
	15			R-3		8 16 30	118	13	SP-SM CL	@15': Few thin interbeds of SAND with silt, reddish brown, dense, mafic-rich, fine sand.	
										Cretaceous Tonalite (Kt), heavily weathered zone:	
				B-2							
	20			R-4		23 50/4"			SC-CL	@20': Sandy CLAY to Clayey SAND, grey-brown, very weathered, high mafic content, medium-grained.	
	25			R-5		38 50/5"				@25': Strongly oxidized (FeO-stained) zones, olive-grey CLAY-filled fractures.	
									SC-CL	**Cretaceous Tonalite (Kt):**	
	30										

SAMPLE TYPES:

B	BULK SAMPLE
C	CORE SAMPLE
G	GRAB SAMPLE
R	RING SAMPLE
S	SPLIT SPOON SAMPLE
T	TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING
AL	ATTERBERG LIMITS
CN	CONSOLIDATION
CO	COLLAPSE
CR	CORROSION
CU	UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
H	HYDROMETER
MD	MAXIMUM DENSITY
PP	POCKET PENETROMETER
RV	R VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
SG	SPECIFIC GRAVITY
UC	UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-4

Project No. 13102.001
Project Warmington Vista - Santa Fe
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer
Location See Figure 2, Exploration Location Map

Date Drilled 4-26-21
Logged By KMD
Hole Diameter 8"
Ground Elevation ~476'
Sampled By KMD

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Bulk	Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30	▨		R-6		▪	50/2"				@30': Sandy CLAY to Clayey SAND, less weathered and fractured.	
	35											
	40											
	45											
	50											
	55											
	60											

Total Depth: 30.2' - Practical Refusal on Hard Bedrock
No Groundwater Encountered During Drilling
Boring Backfilled with Soil Cuttings to Ground Surface

Percolation Well Installed Adjacent to LB-4, Testing 3' to 6' bgs.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

APPENDIX B-2

Logs of Offsite Exploratory Trenches

Log of Trench: TP-1

Project Name: Vista II
Logged by: AS
Project Number: 13201.001b
Elevation: 519.00
Equipment: Excavator
Location/Grid: NA

Earth Material Description: This Soil and/or Rock Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

			Engineering Properties			
Geologic Attitudes	**Earth Materials Exposed On:**	**Geologic Unit**	**Unified Soil Classification**	**Sample Number**	**Moisture (%)**	**Density (pcf)**
	COLLUVIUM/TOPSOIL: @ 0-1': CLAY; dark brown, slightly moist, slightly plastic, stiff, scattered caliche stingers	Qcol				
	RESIDUAL SOIL IN BEDROCK: @ 1-3': SILTY CLAY; medium red-brown, stiff, moist, plastic					
	CRETACEOUS GRANITE: @ 3-5': Igneous bedrock, reddish brown, dry, severely weathered, easily friable to coarse and fine grained sand texture, minor interstitial clay, soft, massive	Kgr				

Graphical Representation
Total Depth = 5 feet
Scale: 1 inch = 2 feet
Surface Slope: 0°
Trend: West/East

***** This log is a part of a report by Leighton and should not be used as a stand-alone document.*****

Log of Trench: TP-2

Project Name: Vista II
Project Number: 13201.001b
Equipment: Excavator

Logged by: AS
Elevation: 514.00
Location/Grid: NA

Earth Material Description: This Soil and/or Rock Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

			Engineering Properties			
Geologic Attitudes	**Earth Materials Exposed On:**	**Geologic Unit**	**Unified Soil Classification**	**Sample Number**	**Moisture (%)**	**Density (pcf)**
	COLLUVIUM/TOPSOIL: @ 0-2': CLAY; dark brown, slightly moist, slightly plastic, stiff, rootlets, scattered caliche stingers	Qcol				
	RESIDUAL SOIL IN BEDROCK: @ 2-4.5': SILTY CLAY; medium red-brown, stiff, moist, plastic					
	CRETACEOUS GRANITE: @ 4.5-5': Igneous bedrock, reddish brown, dry, severely weathered, easily friable to coarse and fine grained sand texture, minor interstitial clay, soft, massive	Kgr				

Graphical Representation
Total Depth = 5 feet

Scale: 1 inch = 2 feet **Surface Slope:** 0° **Trend:** West/East

***** This log is a part of a report by Leighton and should not be used as a stand-alone document.*****

Log of Trench: TP-3

Project Name: Vista II **Project**
Number: 13201.001b
Equipment: Excavator

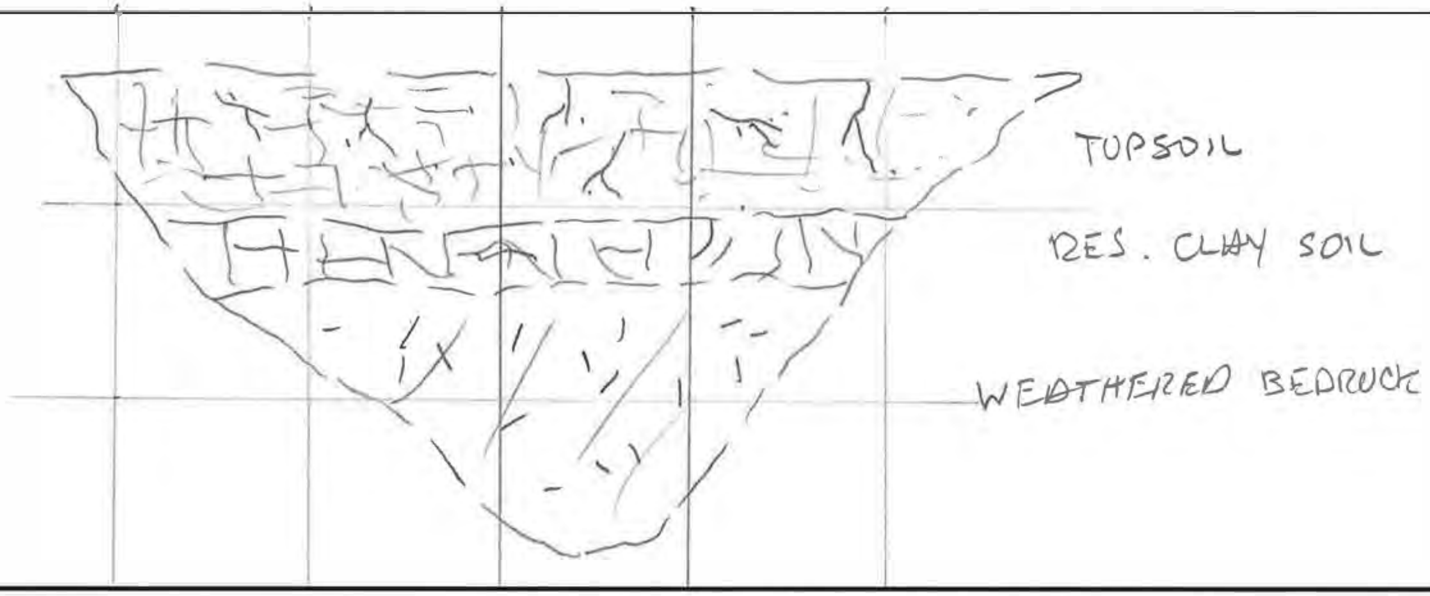
Logged by: AS
Elevation: 510.00
Location/Grid: NA

Earth Material Description: This Soil and/or Rock Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Geologic Attitudes	Earth Materials Exposed On:	Geologic Unit	Engineering Properties			
			Unified Soil Classification	Sample Number	Moisture (%)	Density (pcf)
	COLLUVIUM/TOPSOIL: @ 0-1.5': CLAY; dark brown, slightly moist, slightly plastic, stiff, rootlets, scattered caliche stingers	Qcol				
	RESIDUAL SOIL IN BEDROCK: @ 1.5-2': SILTY CLAY; medium red-brown, stiff, moist, plastic					
	CRETACEOUS GRANITE: @ 2-5': Igneous bedrock, reddish brown, dry, severely weathered, easily friable to coarse and fine grained sand texture, minor interstitial clay, soft, massive	Kgr				

Graphical Representation
Total Depth = 5 feet

Scale: 1 inch = 2 feet **Surface Slope:** 0° **Trend:** West/East



***** This log is a part of a report by Leighton and should not be used as a stand-alone document.*****

APPENDIX C

Results of Laboratory Testing

Leighton

DIRECT SHEAR TEST

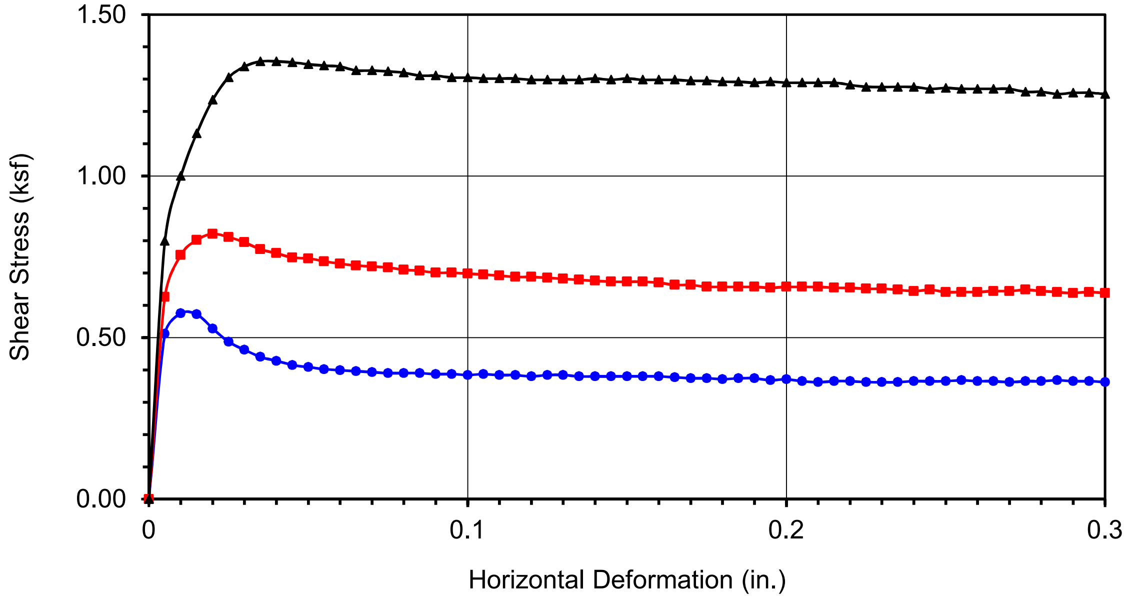
Consolidated Drained - ASTM D 3080

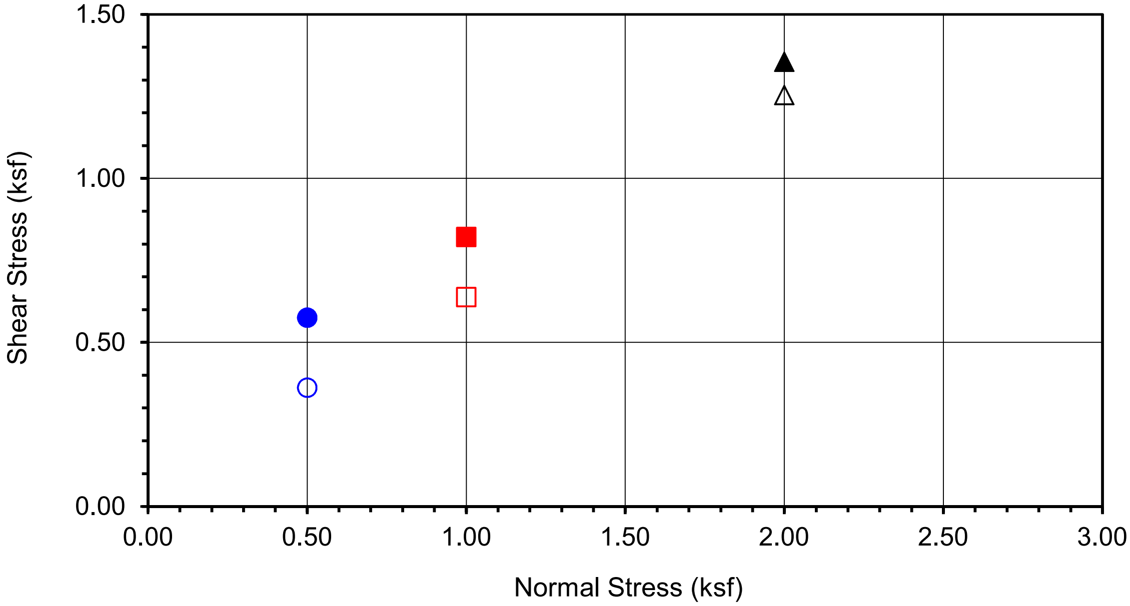
Project Name: Warmington Vista - Hannalei
Project No.: 13102.001
Boring No.: LB-3
Sample No.: B-2
Soil Identification: Olive sandy lean clay s(CL)

Tested By: G. Bathala
Checked By: A. Santos
Sample Type: 90% Remold
Depth (ft.): 6-9

Date: 05/17/21
Date: 05/24/21

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	205.81	205.36	205.53
Weight of Ring(gm):	45.38	44.60	44.26
Before Shearing			
Weight of Wet Sample+Cont.(gm):	178.20	178.20	178.20
Weight of Dry Sample+Cont.(gm):	169.35	169.35	169.35
Weight of Container(gm):	62.98	62.98	62.98
Vertical Rdg.(in): Initial	0.2473	0.2304	0.0000
Vertical Rdg.(in): Final	0.2341	0.2259	-0.0054
After Shearing			
Weight of Wet Sample+Cont.(gm):	223.66	226.28	221.29
Weight of Dry Sample+Cont.(gm):	200.68	203.76	199.91
Weight of Container(gm):	53.74	56.92	52.54
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-3**
Sample No.	**B-2**
Depth (ft)	**6-9**

Sample Type:

90% Remold

Soil Identification:
Olive sandy lean clay s(CL)

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.575	■ 0.821	▲ 1.355
Shear Stress @ End of Test (ksf)	○ 0.362	□ 0.638	△ 1.254
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.32	8.32	8.32
Dry Density (pcf)	123.2	123.4	123.8
Saturation (%)	61.0	61.4	62.2
Soil Height Before Shearing (in.)	1.0132	1.0045	0.9946
Final Moisture Content (%)	15.6	15.3	14.5

Leighton

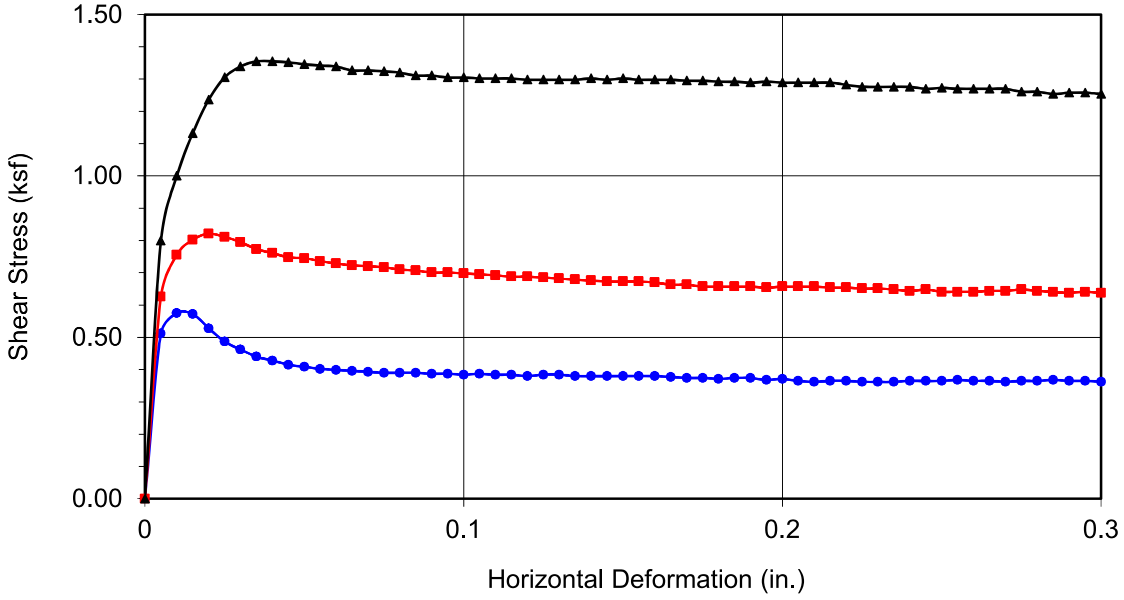
DIRECT SHEAR TEST RESULTS

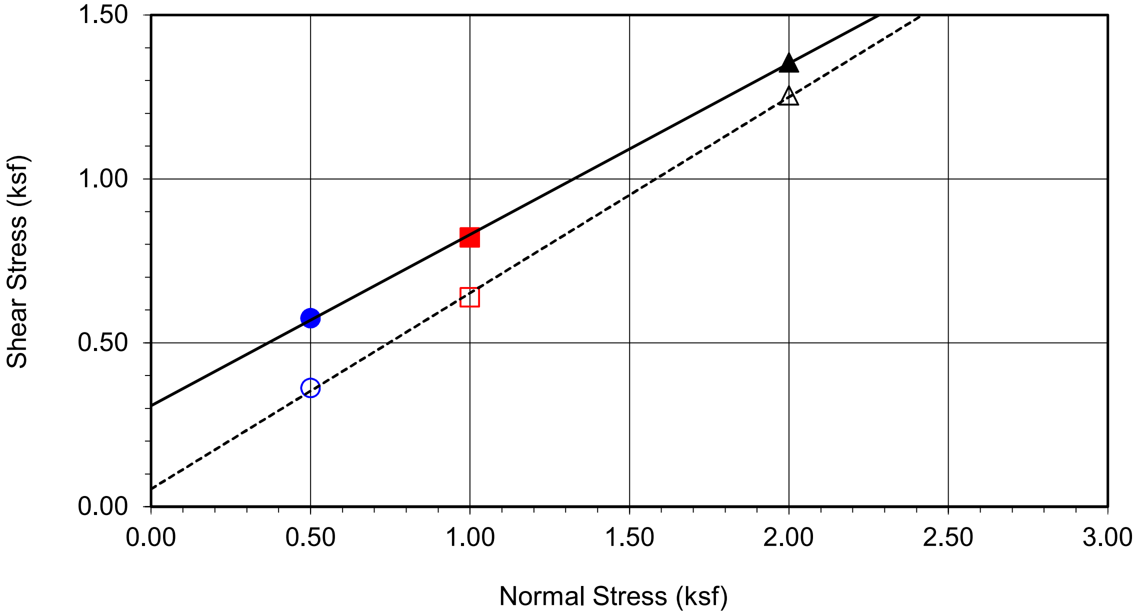
Consolidated Drained - ASTM D 3080

Project No.: 13102.001

Warmington Vista - Hannalei

05-21





Boring No.	**LB-3**
Sample No.	**B-2**
Depth (ft)	**6-9**
Sample Type:	90% Remold

Soil Identification:
Olive sandy lean clay s(CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	308	28
Ultimate	54	31

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.575	■ 0.821	▲ 1.355
Shear Stress @ End of Test (ksf)	○ 0.362	□ 0.638	△ 1.254
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.32	8.32	8.32
Dry Density (pcf)	123.2	123.4	123.8
Saturation (%)	61.0	61.4	62.2
Soil Height Before Shearing (in.)	1.0132	1.0045	0.9946
Final Moisture Content (%)	15.6	15.3	14.5

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13102.001

Warmington Vista - Hannalei

05-21

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Warmington Vista - Hannalei	Tested By:	S. Felter	Date:	05/11/21
Project No.:	13102.001	Checked By:	A. Santos	Date:	05/24/21
Boring No.:	LB-2	Depth (ft.):	2-5		
Sample No.:	B-1				
Soil Identification:	Dark brown sandy lean clay s(CL)				

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0375
Wt. Comp. Soil + Mold (g)	592.10	428.60
Wt. of Mold (g)	190.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	798.10	618.70
Dry Wt. of Soil + Cont. (g)	725.50	555.55
Wt. of Container (g)	0.00	190.10
Moisture Content (%)	10.01	17.28
Wet Density (pcf)	121.3	124.6
Dry Density (pcf)	110.2	106.3
Void Ratio	0.529	0.587
Total Porosity	0.346	0.370
Pore Volume (cc)	71.6	79.4
Degree of Saturation (%) [S_{meas}]	**51.0**	79.5

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
05/11/21	8:40	1.0	0	0.4925
05/11/21	8:50	1.0	10	0.4920
Add Distilled Water to the Specimen				
05/11/21	13:18	1.0	268	0.5240
05/12/21	6:33	1.0	1303	0.5300
05/12/21	7:36	1.0	1366	0.5300

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**38**

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: Warmington Vista - Hannalei
Tested By: S. Felter
Date: 05/11/21
Project No.: 13102.001
Checked By: A. Santos
Date: 05/24/21
Boring No.: LB-4
Depth (ft.): 2-5
Sample No.: B-1
Soil Identification: Dark brown sandy lean clay s(CL)

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0540
Wt. Comp. Soil + Mold (g)	591.30	439.80
Wt. of Mold (g)	191.50	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	798.10	631.30
Dry Wt. of Soil + Cont. (g)	725.50	554.95
Wt. of Container (g)	0.00	191.50
Moisture Content (%)	10.01	21.01
Wet Density (pcf)	120.6	125.9
Dry Density (pcf)	109.6	104.0
Void Ratio	0.538	0.621
Total Porosity	0.350	0.383
Pore Volume (cc)	72.4	83.6
Degree of Saturation (%) [S_{meas}]	**50.2**	91.4

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
05/11/21	9:15	1.0	0	0.6610
05/11/21	9:25	1.0	10	0.6620
Add Distilled Water to the Specimen				
05/11/21	13:17	1.0	232	0.7100
05/12/21	6:32	1.0	1267	0.7150
05/12/21	7:34	1.0	1329	0.7150

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**53**

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MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Warmington Vista - Hannalei
Tested By: J. Gonzalez
Date: 05/11/21
Project No.: 13102.001
Checked By: A. Santos
Date: 05/12/21
Boring No.: LB-3
Depth (ft.): 6-9
Sample No.: B-2
Soil Identification: Olive sandy lean clay s(CL)

Preparation Method: ☒ Moist ☐ Dry

☒ Mechanical Ram ☐ Manual Ram

Mold Volume (ft³) 0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3862	4070	4032			
Weight of Mold (g)	1850	1850	1850			
Net Weight of Soil (g)	2012	2220	2182			
Wet Weight of Soil + Cont. (g)	455.0	453.0	380.0			
Dry Weight of Soil + Cont. (g)	433.1	422.7	347.6			
Weight of Container (g)	39.4	38.3	39.4			
Moisture Content (%)	5.56	7.88	10.51			
Wet Density (pcf)	133.2	147.0	144.5			
Dry Density (pcf)	126.2	136.2	130.7			

Maximum Dry Density (pcf) 136.6 **Optimum Moisture Content (%)** 8.4

PROCEDURE USED

☒ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

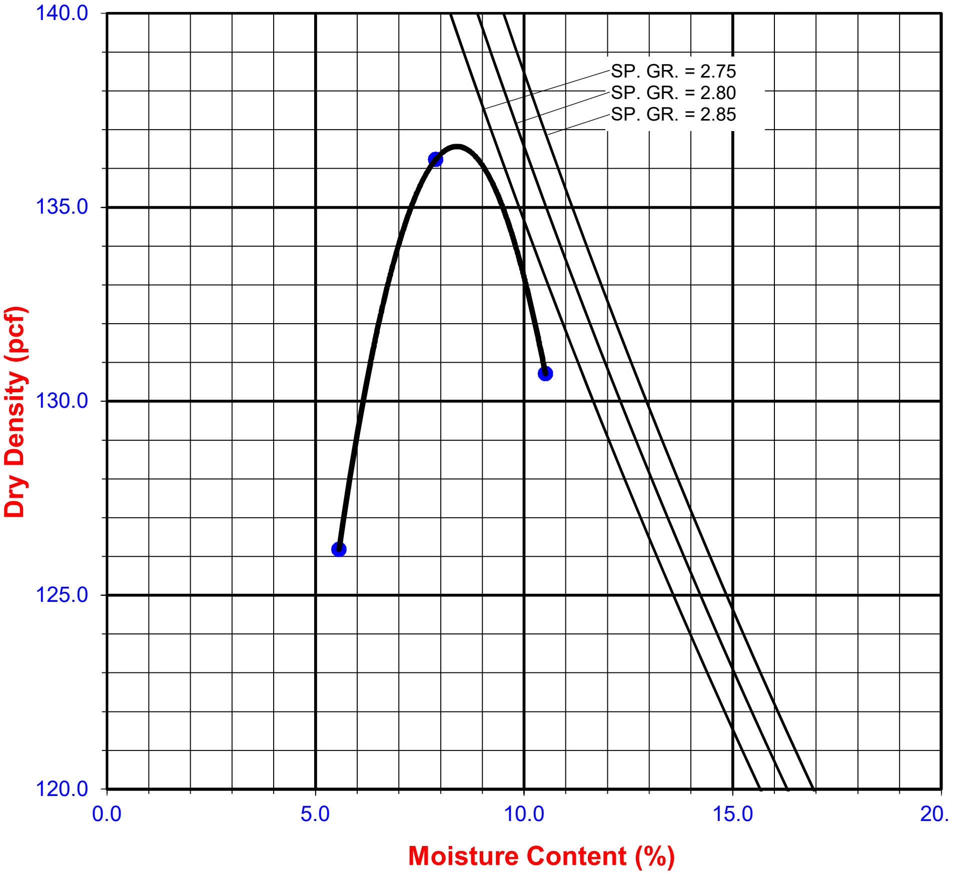
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MX LB-3, B-2 @ 6-9

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ATTERBERG LIMITS

ASTM D 4318

Project Name: Warmington Vista - Hannalei
Tested By: Y. Nguyen
Date: 05/13/21
Project No. : 13102.001
Input By: G. Bathala
Date: 05/24/21
Boring No.: LB-1
Checked By: A. Santos
Sample No.: B-1
Depth (ft.) 2-5
Soil Identification: Dark brown lean clay with sand (CL)s

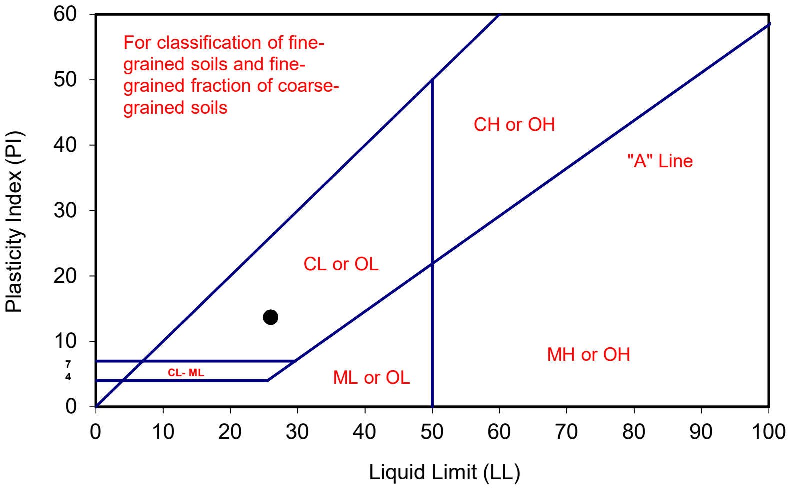
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			28	23	18	
Wet Wt. of Soil + Cont. (g)	9.62	9.71	21.04	21.96	21.38	
Dry Wt. of Soil + Cont. (g)	8.68	8.75	16.94	17.54	16.93	
Wt. of Container (g)	1.00	1.00	1.03	1.10	1.02	
Moisture Content (%) [Wn]	12.24	12.39	25.77	26.89	27.97	

Liquid Limit	**26**
Plastic Limit	**12**
Plasticity Index	**14**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 4.38

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



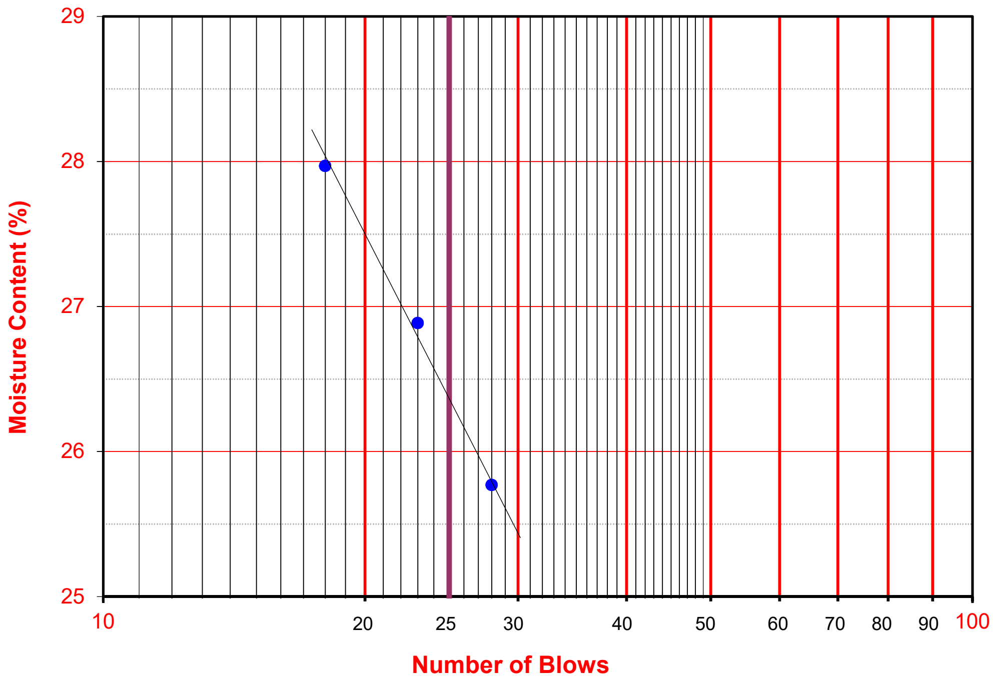
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name: Warmington Vista - Hannalei
Tested By: Y. Nguyen
Date: 05/14/21
Project No. : 13102.001
Input By: G. Bathala
Date: 05/24/21
Boring No.: LB-1
Checked By: A. Santos
Sample No.: B-2
Depth (ft.) 14-17
Soil Identification: Olive clayey sand (SC)

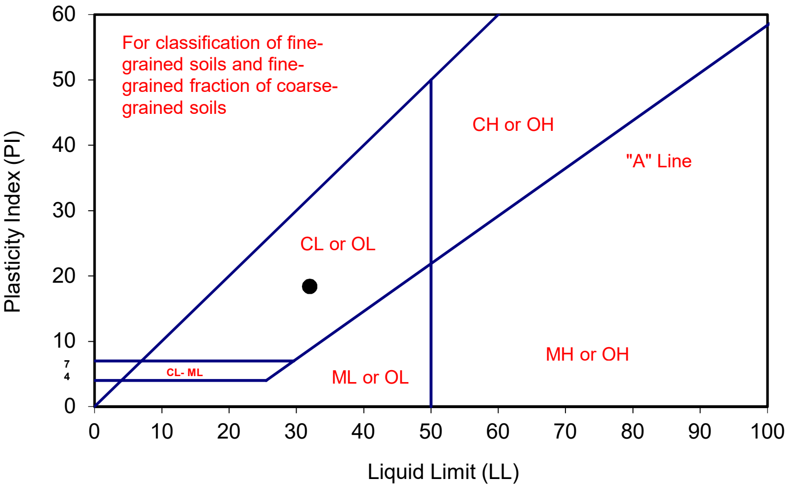
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	25	19	
Wet Wt. of Soil + Cont. (g)	9.72	9.61	22.03	22.22	21.48	
Dry Wt. of Soil + Cont. (g)	8.69	8.58	17.22	17.11	16.37	
Wt. of Container (g)	1.05	1.06	1.12	1.04	1.05	
Moisture Content (%) [Wn]	13.48	13.70	29.88	31.80	33.36	

Liquid Limit	**32**
Plastic Limit	**14**
Plasticity Index	**18**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 8.76

One - Point Liquid Limit Calculation

$LL = W_n(N/25)^{0.121}$



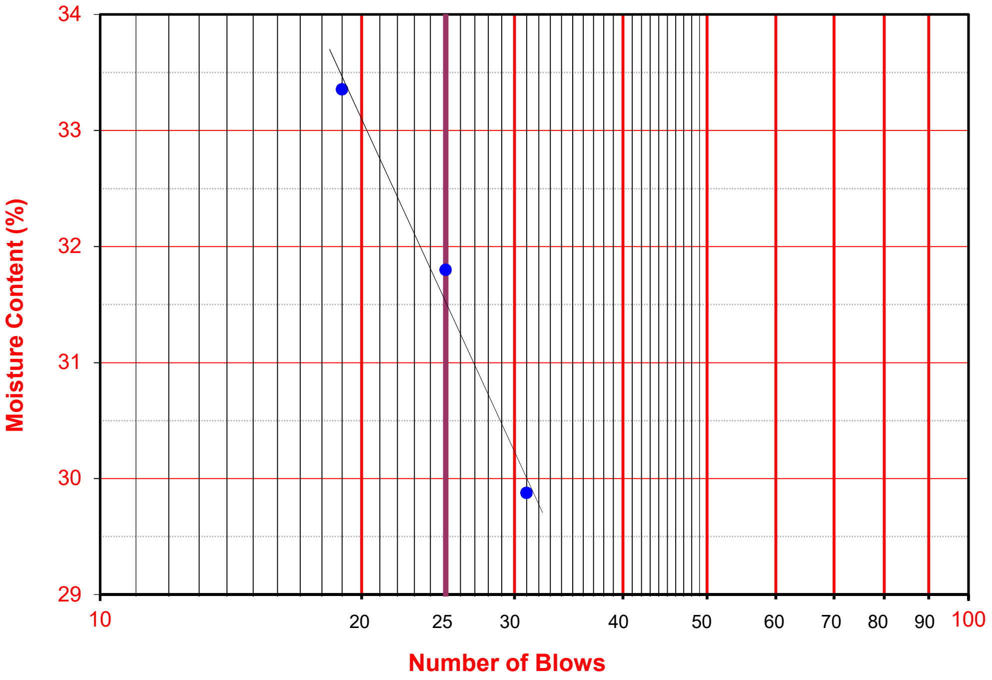
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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ATTERBERG LIMITS

ASTM D 4318

Project Name: Warmington Vista - Hannalei
Tested By: Y. Nguyen
Date: 05/13/21
Project No. : 13102.001
Input By: G. Bathala
Date: 05/24/21
Boring No.: LB-2
Checked By: A. Santos
Sample No.: B-1
Depth (ft.) 2-5
Soil Identification: Dark brown sandy lean clay s(CL)

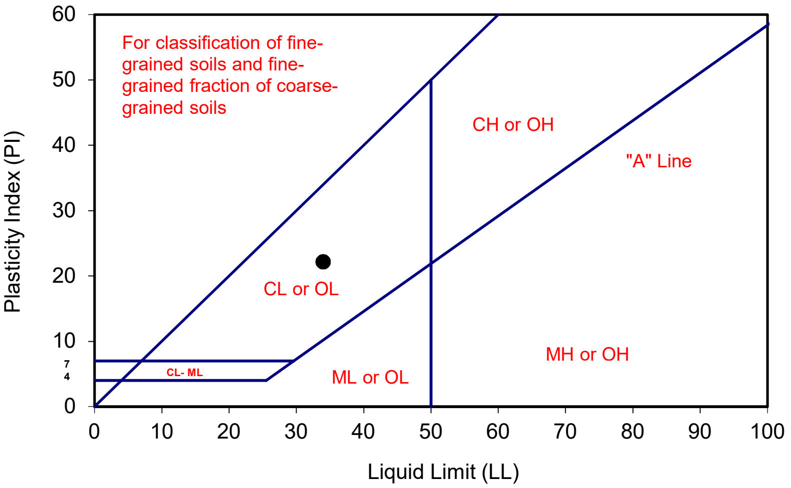
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	26	21	
Wet Wt. of Soil + Cont. (g)	9.37	9.50	21.96	22.18	21.47	
Dry Wt. of Soil + Cont. (g)	8.48	8.61	16.79	16.82	16.16	
Wt. of Container (g)	1.05	1.00	1.06	1.08	1.04	
Moisture Content (%) [Wn]	11.98	11.70	32.87	34.05	35.12	

Liquid Limit	**34**
Plastic Limit	**12**
Plasticity Index	**22**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 10.22

One - Point Liquid Limit Calculation

$$LL = Wn(N/25)^{0.121}$$



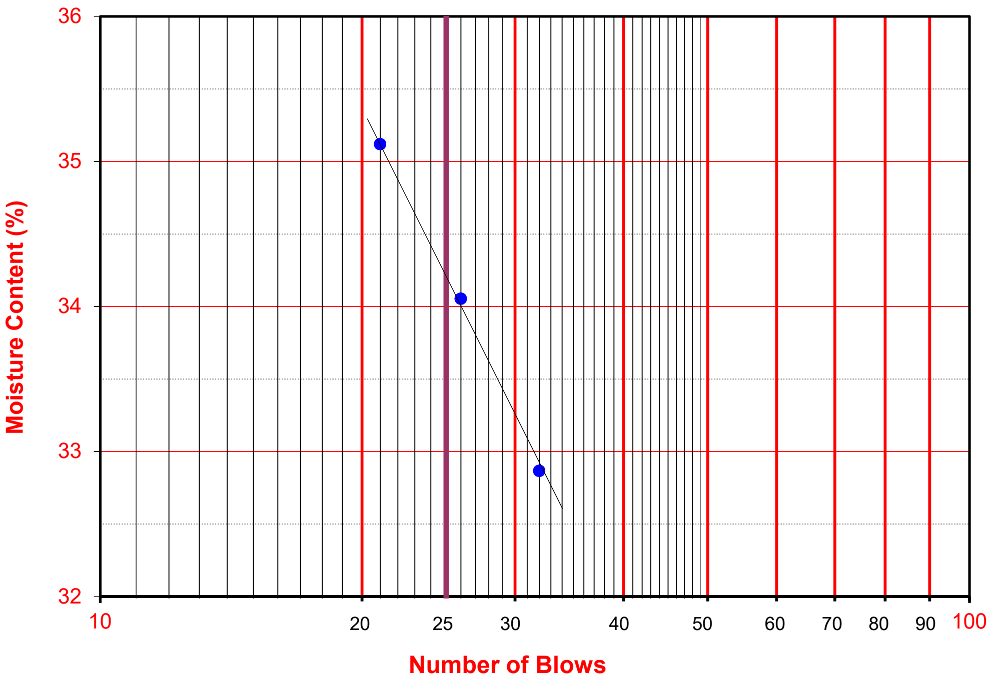
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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ATTERBERG LIMITS

ASTM D 4318

Project Name: Warmington Vista - Hannalei
Tested By: Y. Nguyen
Date: 05/14/21
Project No. : 13102.001
Input By: G. Bathala
Date: 05/24/21
Boring No.: LB-3
Checked By: A. Santos
Sample No.: B-2
Depth (ft.) 6-9
Soil Identification: Olive sandy lean clay s(CL)

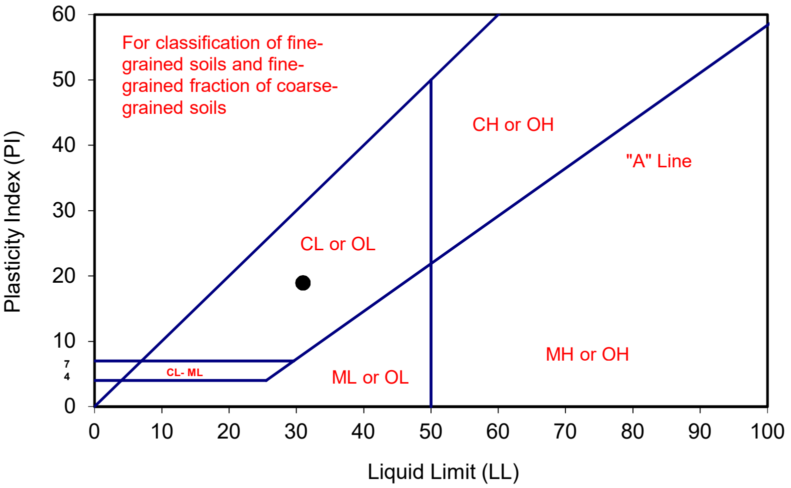
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			29	23	16	
Wet Wt. of Soil + Cont. (g)	9.53	9.53	20.30	20.65	20.88	
Dry Wt. of Soil + Cont. (g)	8.61	8.62	15.84	15.97	15.96	
Wt. of Container (g)	1.04	1.04	1.06	1.04	1.09	
Moisture Content (%) [Wn]	12.15	12.01	30.18	31.35	33.09	

Liquid Limit	**31**
Plastic Limit	**12**
Plasticity Index	**19**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 8.03

One - Point Liquid Limit Calculation

$$LL = W_n(N/25)^{0.121}$$



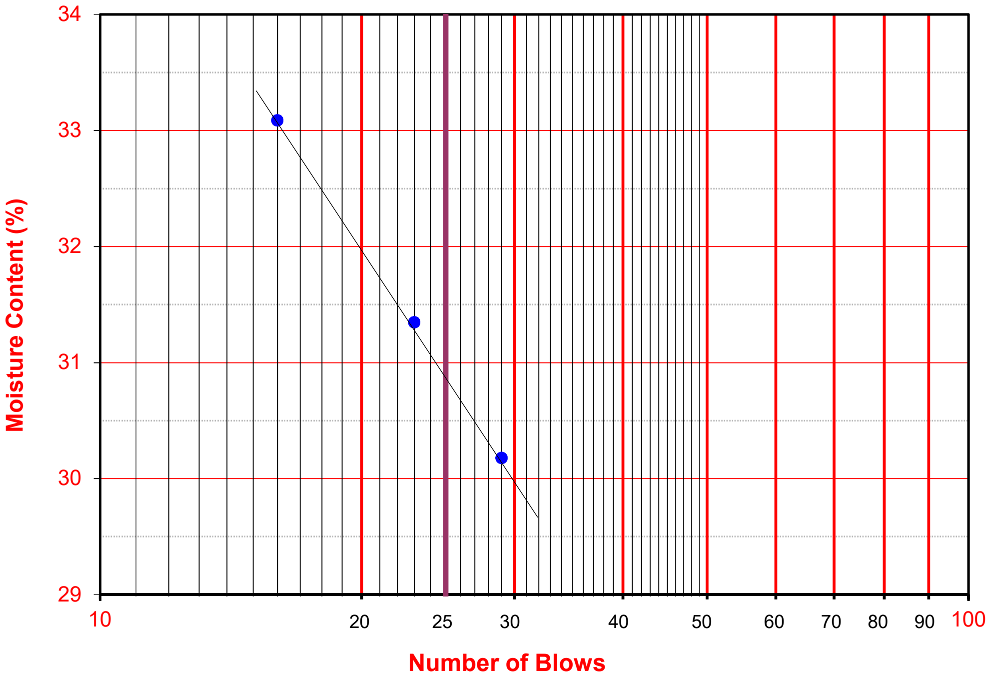
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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ATTERBERG LIMITS

ASTM D 4318

Project Name: Warmington Vista - Hannalei
Tested By: Y. Nguyen
Date: 05/13/21
Project No. : 13102.001
Input By: G. Bathala
Date: 05/24/21
Boring No.: LB-4
Checked By: A. Santos
Sample No.: B-1
Depth (ft.) 2-5
Soil Identification: Dark brown sandy lean clay s(CL)

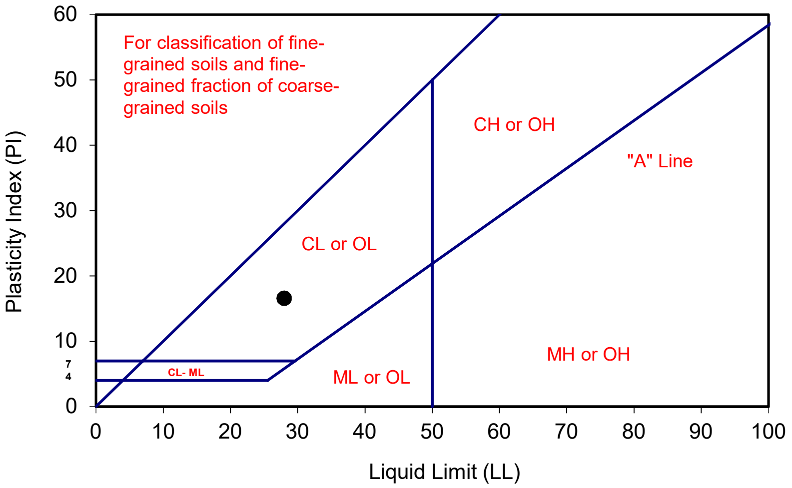
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			30	25	19	
Wet Wt. of Soil + Cont. (g)	9.31	9.32	22.31	21.13	22.99	
Dry Wt. of Soil + Cont. (g)	8.46	8.47	17.81	16.78	18.12	
Wt. of Container (g)	1.01	1.06	1.06	1.04	1.02	
Moisture Content (%) [Wn]	11.41	11.47	26.87	27.64	28.48	

Liquid Limit	**28**
Plastic Limit	**11**
Plasticity Index	**17**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 5.84

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



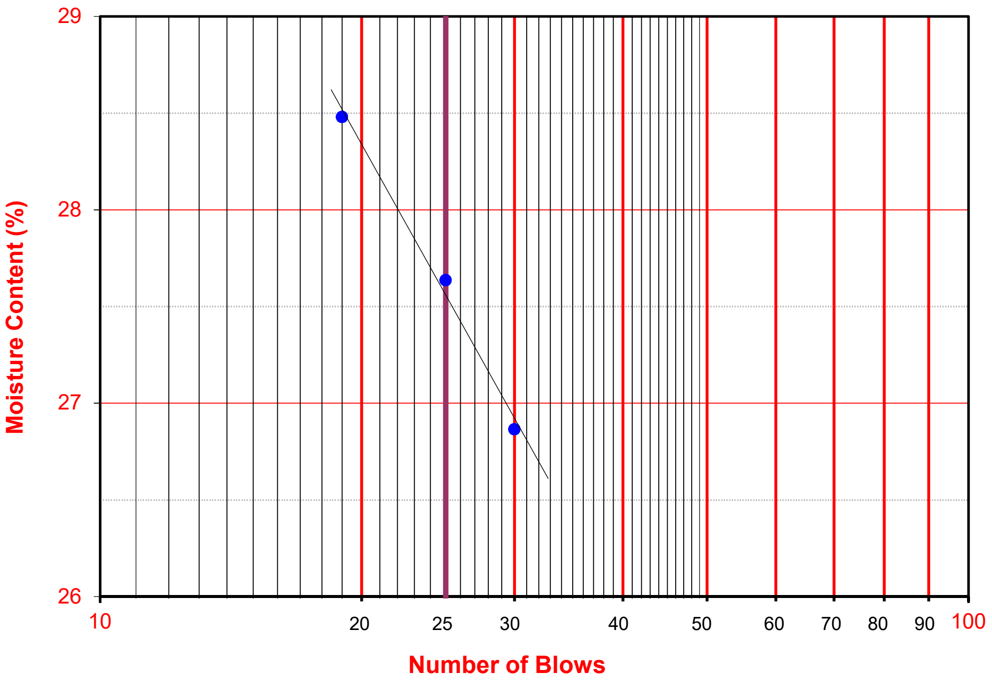
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: Warmington Vista - Hannalei
Tested By: GB/GEB
Date: 05/10/21
Project No.: 13102.001
Checked By: A. Santos
Date: 05/24/21
Boring No.: LB-1
Sample No.: B-2
Depth (feet): 14-17
Soil Identification: Olive clayey sand (SC)

% Gravel	**0**	**Soil Type**
% Sand	**59**	**SC**
% Fines	**41**	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	117.68	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	116.54	106.30
Wt.of Air-Dry Soil + Cont. (g)	750.04	Wt. of Container No.___ (g)	1.00	57.16	74.92
Wt. of Container	245.49	Moisture Content (%)	0.00	1.92	
Dry Wt. of Soil (g)	504.55	Wt. of Dry Soil (g)			31.38

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	0.00	100.0
No. 4	0.00	100.0
No. 10	18.43	96.3
Pan		

Sieve after Hydrometer & Wet Sieve

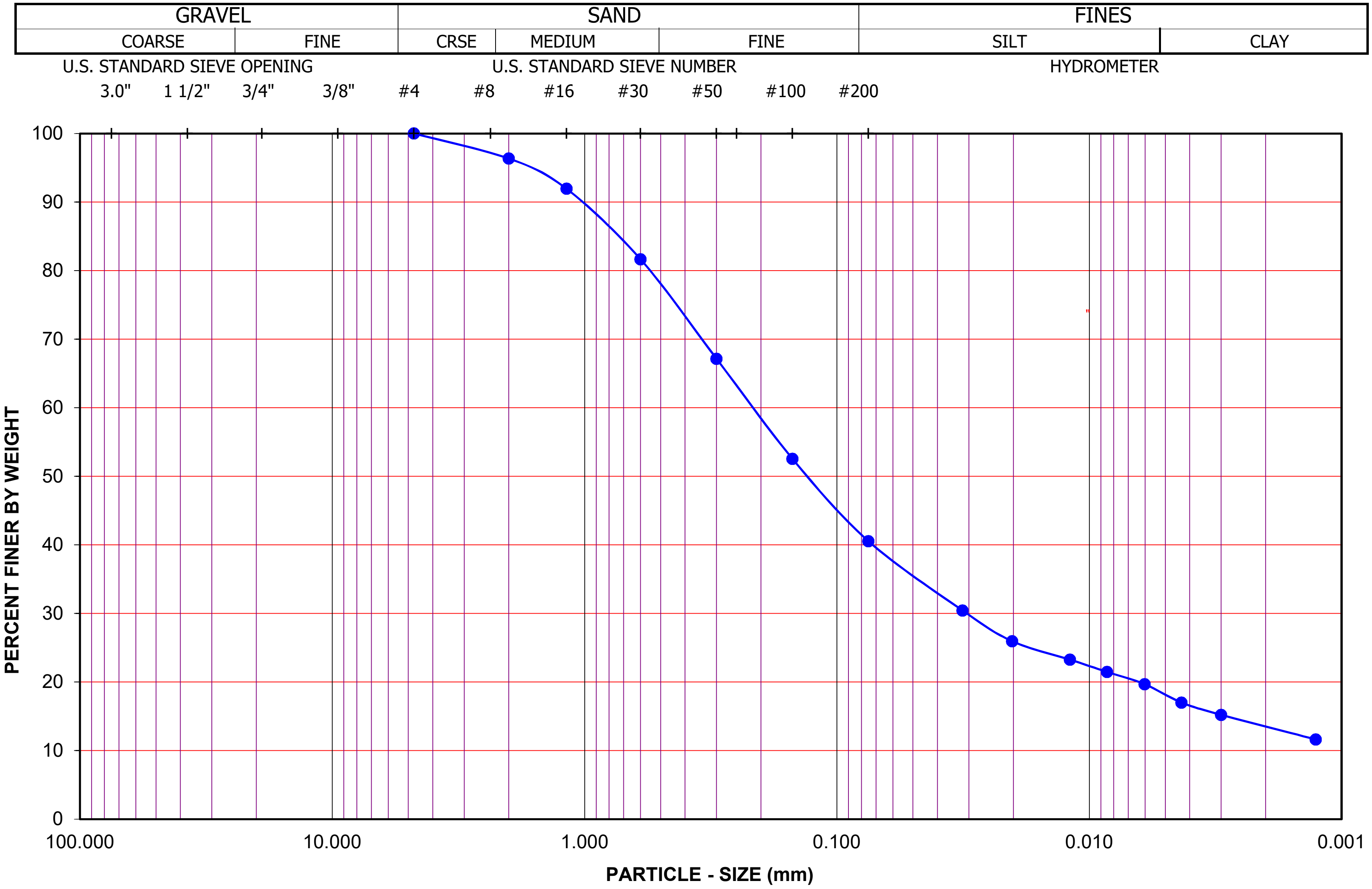
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	96.3
No. 16	2.44	95.4	91.9
No. 30	8.14	84.7	81.7
No. 50	16.17	69.7	67.2
No. 100	24.26	54.5	52.6
No. 200	30.91	42.1	40.6
Pan			

Hydrometer

Wt. of Air-Dry Soil (g) 54.40
Wt. of Dry Soil (g) 53.38

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
12-May-21	7:26	0		9.0			
	7:28	2	23.9	9.0	26.0	30.4	0.0318
	7:31	5	24.0	9.0	23.5	26.0	0.0202
	7:41	15	23.9	9.0	22.0	23.3	0.0119
	7:56	30	23.8	9.0	21.0	21.5	0.0085
	8:26	60	23.6	9.0	20.0	19.7	0.0060
	9:26	120	23.5	9.0	18.5	17.0	0.0043
	11:36	250	23.9	9.0	17.5	15.2	0.0030
13-May-21	7:26	1440	23.2	9.0	15.5	11.6	0.0013



Project Name: Warmington Vista - Hannalei

Project No.: 13102.001

Boring No.: LB-1

Sample No.: B-2

Depth (feet): 14-17

Soil Type : SC

Soil Identification: Olive clayey sand (SC)

GR:SA:FI : (%) 0 : 59 : 41

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

May-21

Leighton

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: Warmington Vista - Hannalei
Tested By: GB/GEB
Date: 05/10/21
Project No.: 13102.001
Checked By: A. Santos
Date: 05/24/21
Boring No.: LB-2
Sample No.: B-2
Depth (feet): 12-14
Soil Identification: Olive clayey sand (SC)

% Gravel	**0**	**Soil Type**
% Sand	**73**	**SC**
% Fines	**27**	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	147.37	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	146.38	149.56
Wt.of Air-Dry Soil + Cont. (g)	593.26	Wt. of Container No.___ (g)	1.00	39.14	76.42
Wt. of Container	88.88	Moisture Content (%)	0.00	0.92	
Dry Wt. of Soil (g)	504.38	Wt. of Dry Soil (g)			73.14

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	0.00	100.0
No. 4	0.00	100.0
No. 10	26.00	94.8
Pan		

Sieve after Hydrometer & Wet Sieve

U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	94.8
No. 16	7.20	92.8	88.0
No. 30	22.28	77.8	73.8
No. 50	41.06	59.0	56.0
No. 100	58.61	41.5	39.4
No. 200	71.47	28.7	27.2
Pan			

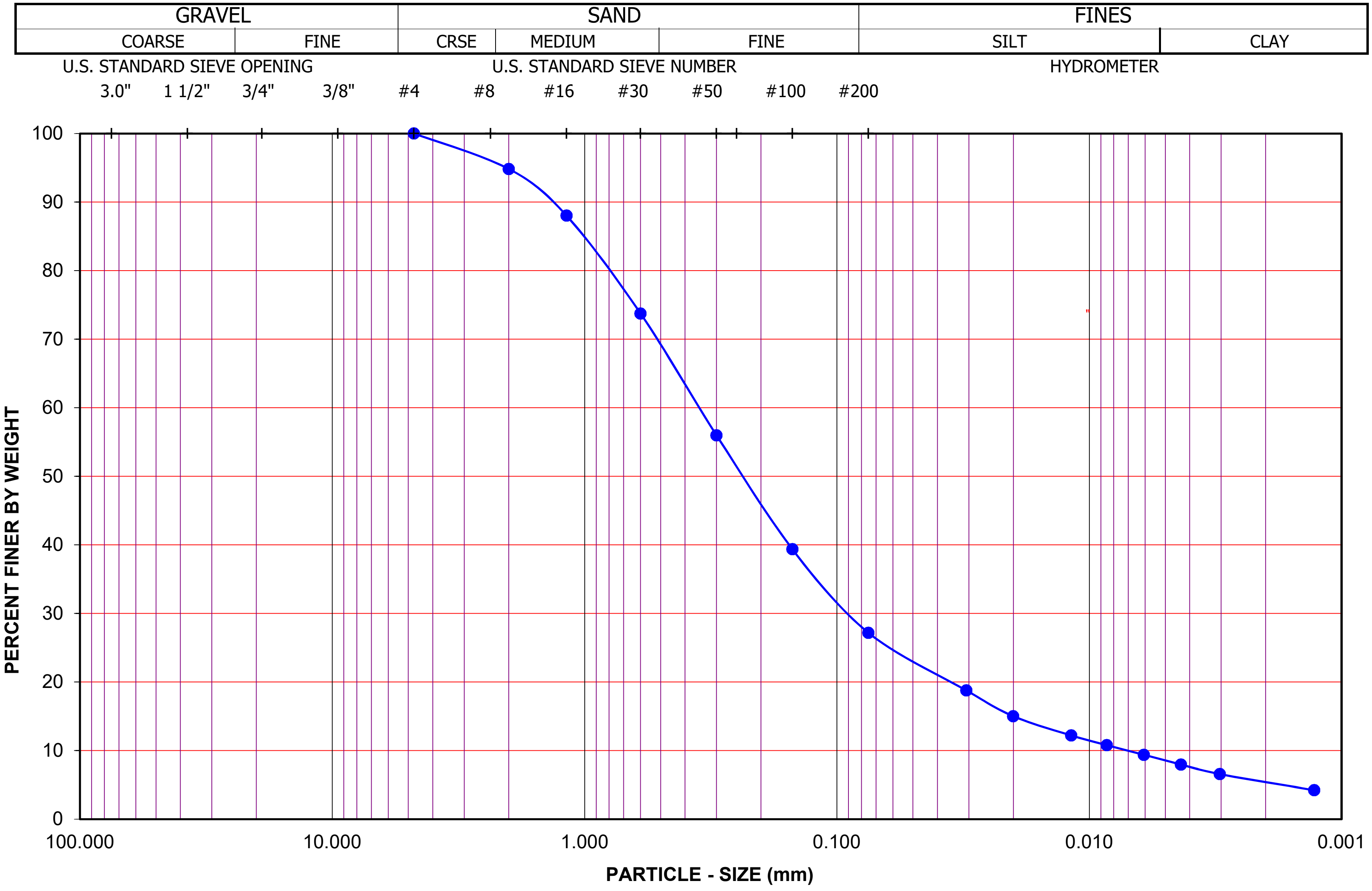
Hydrometer

Wt. of Air-Dry Soil (g) 101.12

Wt. of Dry Soil (g) 100.20

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
12-May-21	7:30	0		9.0			
	7:32	2	24.2	9.0	29.0	18.8	0.0307
	7:35	5	24.2	9.0	25.0	15.0	0.0200
	7:45	15	24.1	9.0	22.0	12.2	0.0118
	8:00	30	23.9	9.0	20.5	10.8	0.0085
	8:30	60	23.8	9.0	19.0	9.4	0.0061
	9:30	120	23.5	9.0	17.5	8.0	0.0043
	11:40	250	23.9	9.0	16.0	6.6	0.0030
13-May-21	7:30	1440	23.2	9.0	13.5	4.2	0.0013



Project Name: Warmington Vista - Hannalei

Project No.: 13102.001

Boring No.: LB-2

Sample No.: B-2

Depth (feet): 12-14

Soil Type : SC

Soil Identification: Olive clayey sand (SC)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

GR:SA:FI : (%) 0 : 73 : 27

May-21

Leighton

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: Warmington Vista - Hannalei
Tested By: GB/GEB
Date: 05/10/21
Project No.: 13102.001
Checked By: A. Santos
Date: 05/24/21
Boring No.: LB-3
Sample No.: B-2
Depth (feet): 6-9
Soil Identification: Olive sandy lean clay s(CL)

% Gravel	**0**	**Soil Type**
% Sand	**50**	**s(CL)**
% Fines	**50**	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	121.45	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	120.48	108.37
Wt.of Air-Dry Soil + Cont. (g)	546.74	Wt. of Container No.____ (g)	1.00	61.19	82.53
Wt. of Container	137.65	Moisture Content (%)	0.00	1.64	
Dry Wt. of Soil (g)	409.09	Wt. of Dry Soil (g)			25.84

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	0.00	100.0
No. 4	0.00	100.0
No. 10	0.92	99.8
Pan		

Sieve after Hydrometer & Wet Sieve

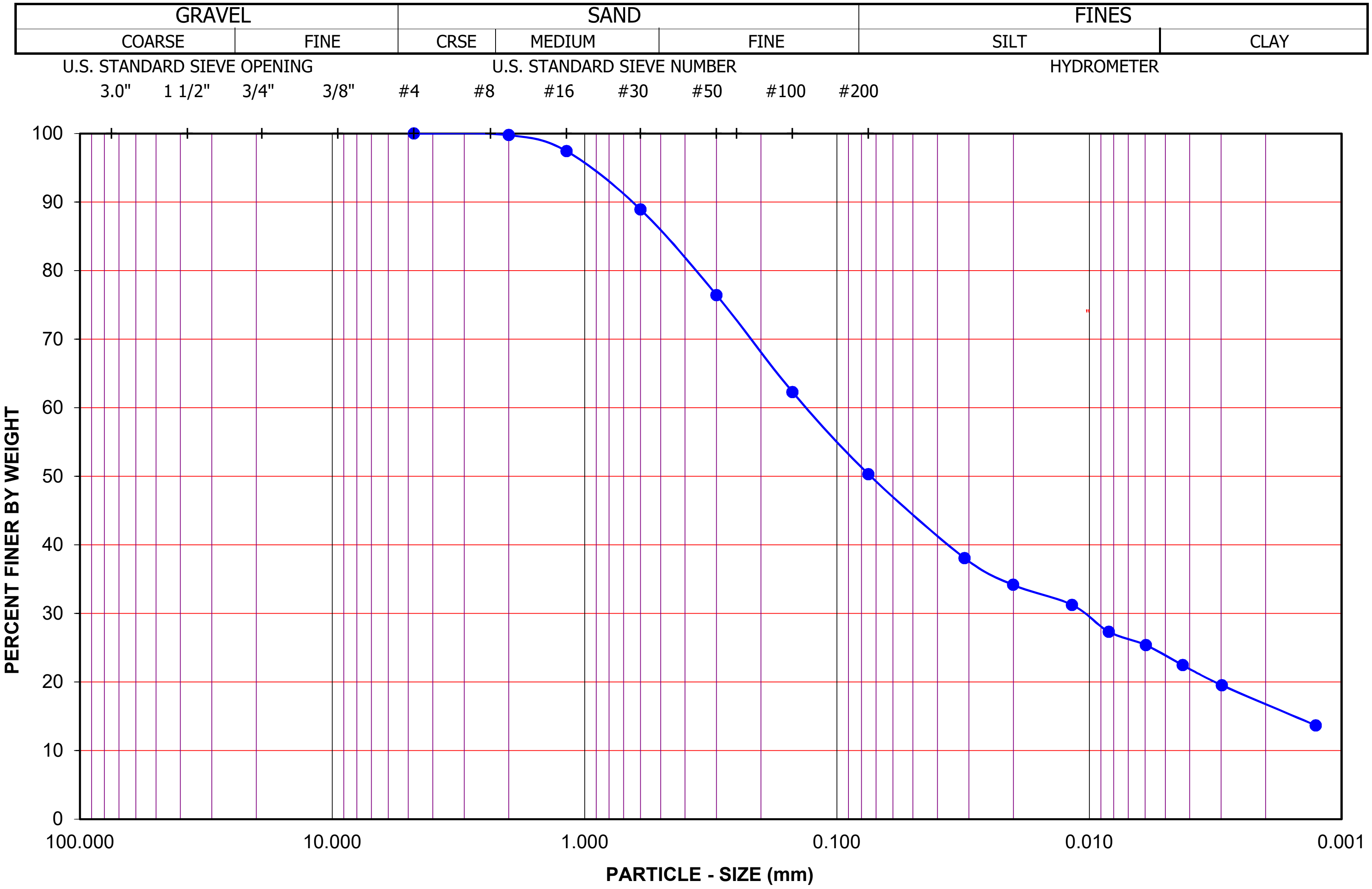
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	99.8
No. 16	1.19	97.7	97.4
No. 30	5.51	89.1	88.9
No. 50	11.85	76.6	76.4
No. 100	19.03	62.4	62.3
No. 200	25.11	50.4	50.3
Pan			

Hydrometer

Wt. of Air-Dry Soil (g) 51.50
Wt. of Dry Soil (g) 50.67

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
12-May-21	7:22	0		9.0			
	7:24	2	23.8	9.0	28.5	38.1	0.0312
	7:27	5	23.8	9.0	26.5	34.2	0.0201
	7:37	15	23.9	9.0	25.0	31.3	0.0117
	7:52	30	23.8	9.0	23.0	27.3	0.0084
	8:22	60	23.6	9.0	22.0	25.4	0.0060
	9:22	120	23.5	9.0	20.5	22.5	0.0043
	11:32	250	23.9	9.0	19.0	19.5	0.0030
13-May-21	7:22	1440	23.2	9.0	16.0	13.7	0.0013



Project Name: Warmington Vista - Hannalei

Project No.: 13102.001

Boring No.: LB-3

Sample No.: B-2

Depth (feet): 6-9

Soil Type : s(CL)

Soil Identification: Olive sandy lean clay s(CL)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 422

GR:SA:FI : (%) 0 : 50 : 50

May-21

Sieve & Hydro LB-3, B-2 @ 6-9

APPENDIX D

Field Percolation Test Results

Boring Percolation Test Data Sheet

Project Number:	13102.001b	**Test Hole Number:**	LP-1
Project Name:	Warmington Santa Fe Vista	**Date Excavated:**	4/26/2021
Earth Description:	Older Alluvium	**Date Tested:**	4/27/2021
Liquid Description:	Tap water	**Depth of boring (ft):**	10
Tested By:	KMD	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	4/27/21 7:15 AM	**Length of slotted casing (ft):**	5
Start Time for Standard:	4/27/21 8:15 AM	**Depth to Initial Water Depth (ft):**	6
Standard Time Interval		**Porosity of Annulus Material, *n* :**	0.35
Between Readings, mins:	30	**Bentonite Plug at Bottom:**	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	7:15	30	5.48	54.2	0.2	0.01
	7:45		5.50	54.0		
P2	7:45	30	5.50	54.0	0.5	0.01
	8:15		5.54	53.5		
1	8:15	30	5.54	53.5	0.0	0.00
	8:45		5.54	53.5		
2	8:45	30	5.54	53.5	0.0	0.00
	9:15		5.54	53.5		
3	9:15	30	5.54	53.5	0.0	0.00
	9:45		5.54	53.5		
4	9:45	30	5.54	53.5	0.0	0.00
	10:15		5.54	53.5		
5	10:15	30	5.54	53.5	0.0	0.00
	10:45		5.54	53.5		
6	10:45	30	5.54	53.5	0.1	0.00
	11:15		5.55	53.4		
7	11:15	30	5.55	53.4	0.2	0.01
	11:45		5.57	53.2		
8	11:45	30	5.57	53.2	0.2	0.01
	12:15		5.59	52.9		
9	12:15	30	5.59	52.9	0.2	0.01
	12:45		5.61	52.7		
10	12:45	30	5.61	52.7	0.2	0.01
	13:15		5.63	52.4		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 ReadingsLast Readings) = 0.01 in./hr.

Boring Percolation Test Data Sheet

Project Number:	13102.001b	**Test Hole Number:**	LP-2
Project Name:	Warmington Santa Fe Vista	**Date Excavated:**	4/26/2021
Earth Description:	Older Alluvium	**Date Tested:**	4/27/2021
Liquid Description:	Tap water	**Depth of boring (ft):**	6
Tested By:	KMD	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	4/27/21 7:35 AM	**Length of slotted casing (ft):**	5
Start Time for Standard:	4/27/21 9:12 AM	**Depth to Initial Water Depth (ft):**	1
Standard Time Interval Between Readings, mins:	30	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	7:35	30	1.00	60.0	1.7	0.04
	8:05		1.14	58.3		
P2	8:05	30	0.91	61.1	2.3	0.06
	8:35		1.10	58.8		
1	9:12	30	2.72	39.4	0.1	0.00
	9:42		2.73	39.2		
2	9:42	30	2.73	39.2	0.0	0.00
	10:12		2.73	39.2		
3	10:12	30	2.73	39.2	0.0	0.00
	10:42		2.73	39.2		
4	10:42	30	2.73	39.2	0.0	0.00
	11:12		2.73	39.2		
5	11:12	30	2.73	39.2	0.1	0.00
	11:42		2.74	39.1		
6	11:42	30	2.74	39.1	0.2	0.01
	12:12		2.76	38.9		
7	12:12	30	2.76	38.9	0.2	0.01
	12:42		2.78	38.6		
8	12:42	30	2.78	38.6	0.2	0.01
	13:12		2.80	38.4		
9	13:12	30	2.80	38.4	0.1	0.00
	13:42		2.81	38.3		
10	13:42	30	2.81	38.3	0.1	0.00
	14:12		2.82	38.2		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 ReadingsLast Readings) = 0.01 in./hr.

APPENDIX E

Seismic Analysis

Vista Hannalei

Latitude, Longitude: 33.180161, -117.217197

Date	5/26/2021, 11:37:08 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	0.912	MCE_R ground motion. (for 0.2 second period)
S_1	0.336	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.094	Site-modified spectral acceleration value
S_{M1}	0.504	Site-modified spectral acceleration value
S_{DS}	0.73	Numeric seismic design value at 0.2 second SA
S_{D1}	0.336	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.394	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.473	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.912	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.992	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.336	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.365	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.919	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.922	Mapped value of the risk coefficient at a period of 1 s

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APPENDIX F

General Earthwork and Grading Specifications

Leighton

APPENDIX F

LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

TABLE OF CONTENTS

1.0 GENERAL

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

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

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate 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 rectified.

2.0 PREPARATION OF AREAS TO BE FILLED

2.1 Clearing and Grubbing

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

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

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

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be 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.

2.3 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 evaluated by the Geotechnical Consultant during grading.

2.4 Benching

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

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 FILL MATERIAL

3.1 General

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

3.2 Oversize

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

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 SUBDRAIN INSTALLATION

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

6.0 EXCAVATION

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

7.0 TRENCH BACKFILLS

7.1 Safety

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

7.2 Bedding and Backfill

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

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

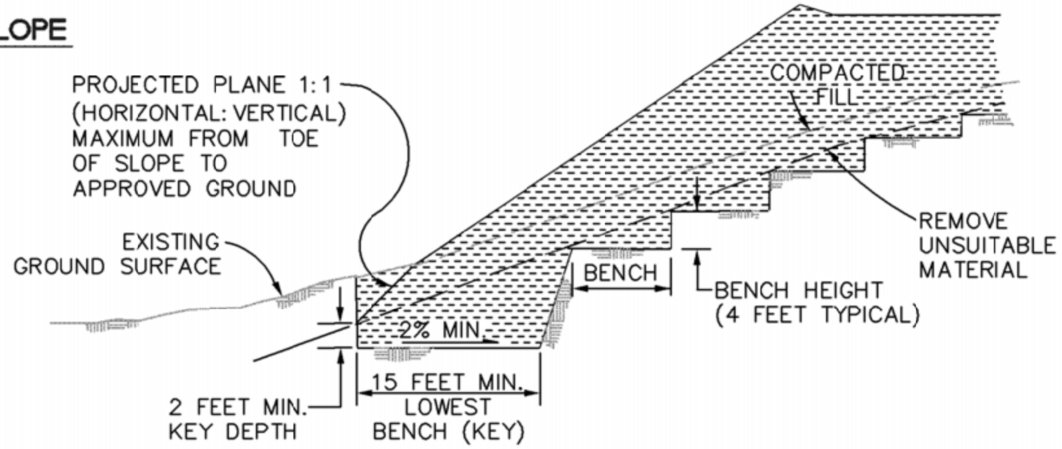
7.3 Lift Thickness

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.

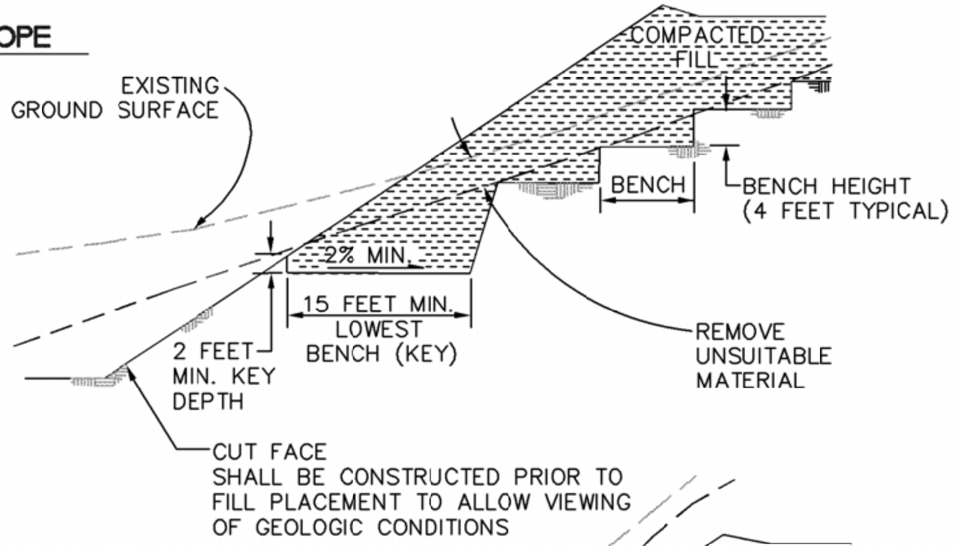
7.4 Observation and Testing

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

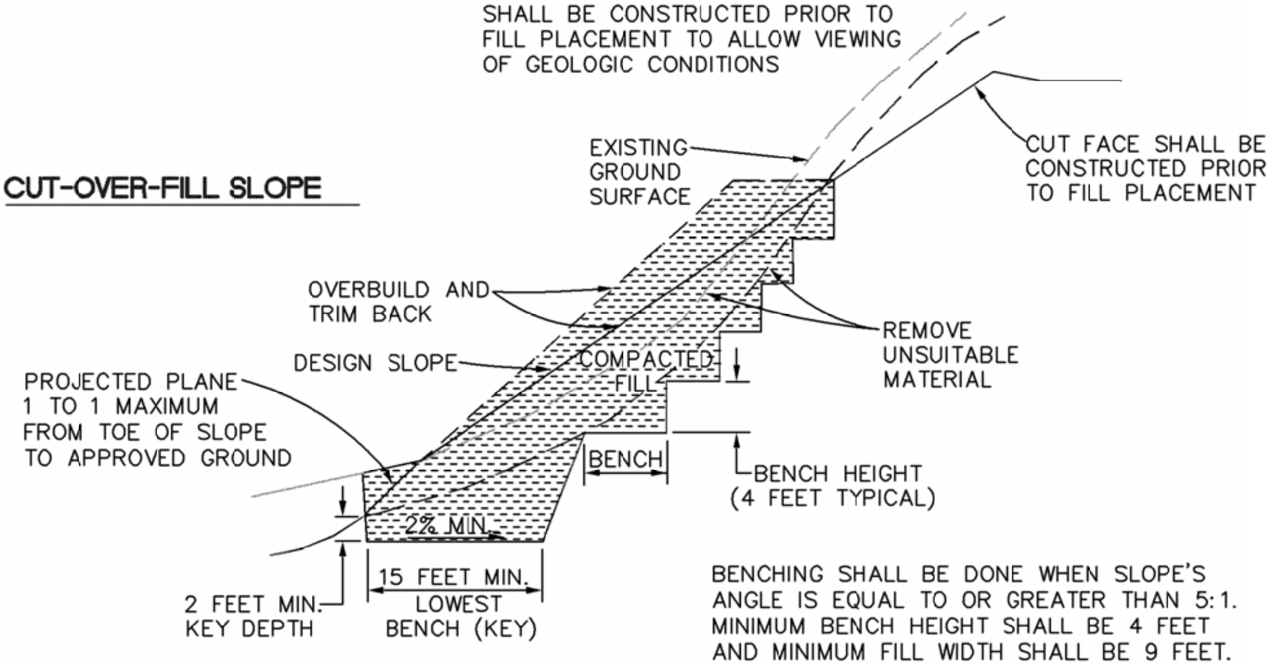
FILL SLOPE



FILL-OVER-CUT SLOPE



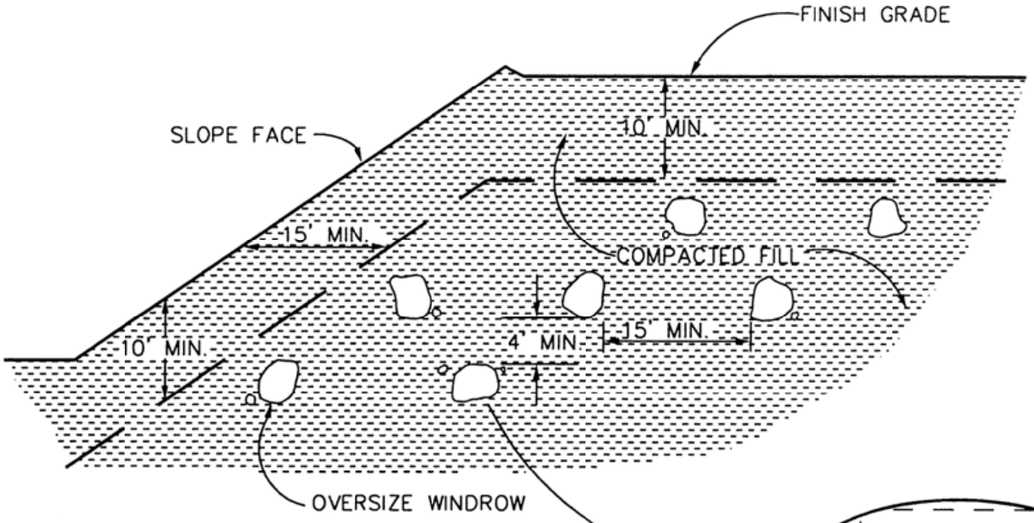
CUT-OVER-FILL SLOPE

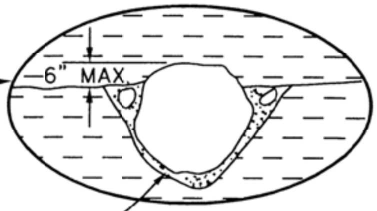


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

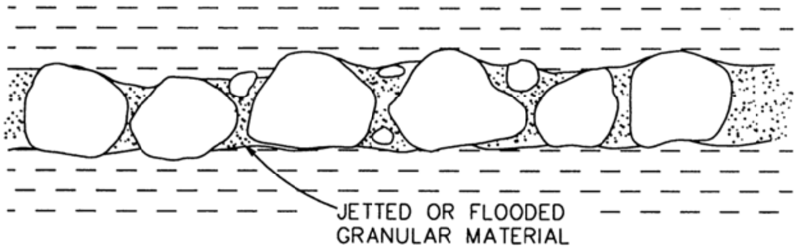




GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

DETAIL

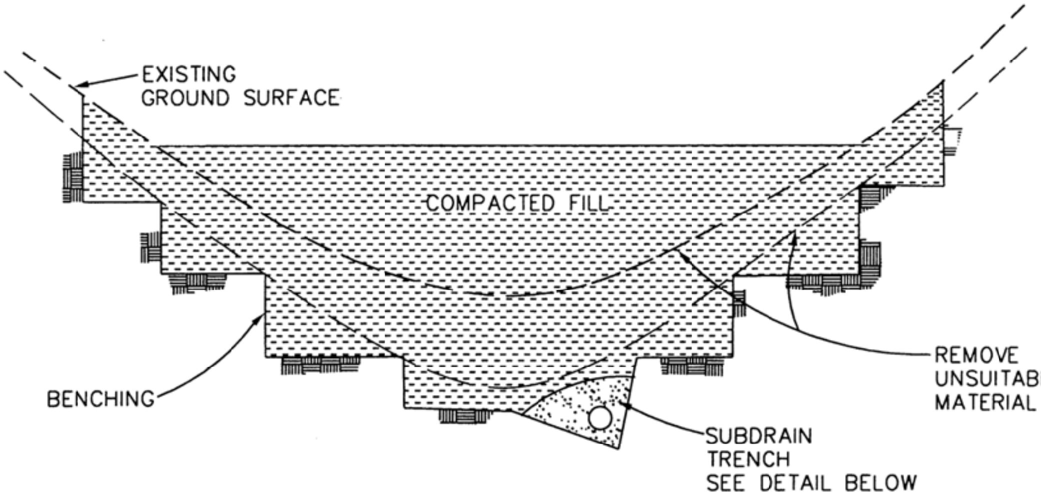
- * OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- * EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- * BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- * DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- * WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

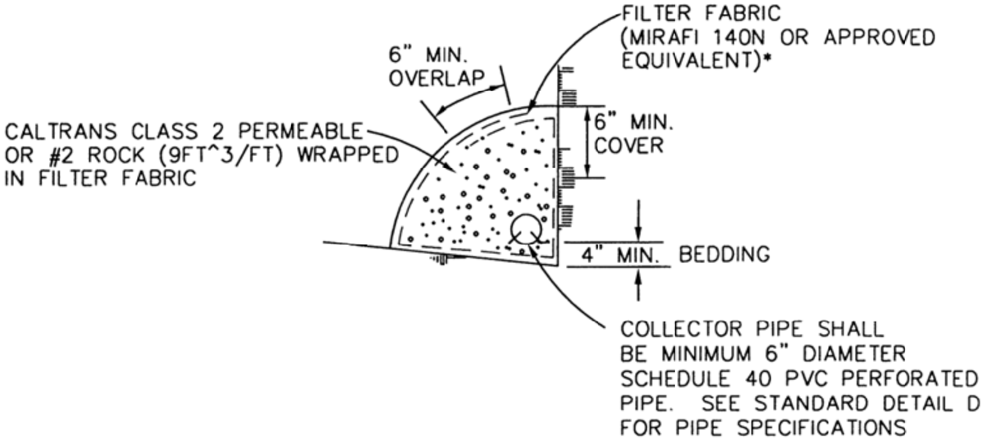


TYPICAL PROFILE ALONG WINDROW

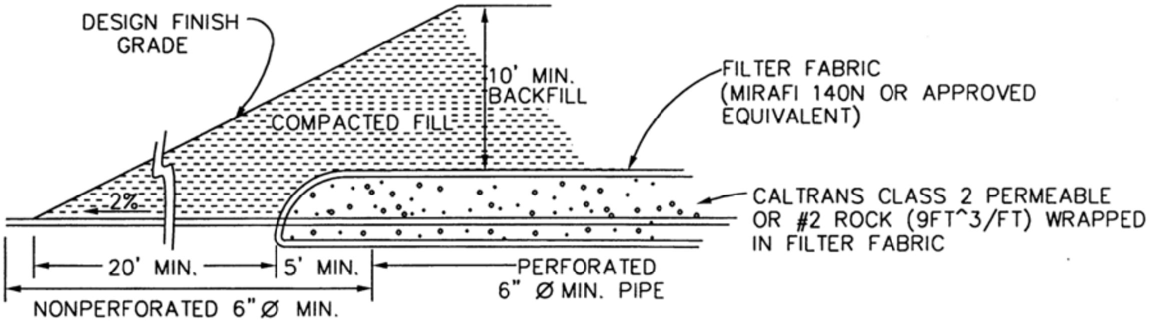
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





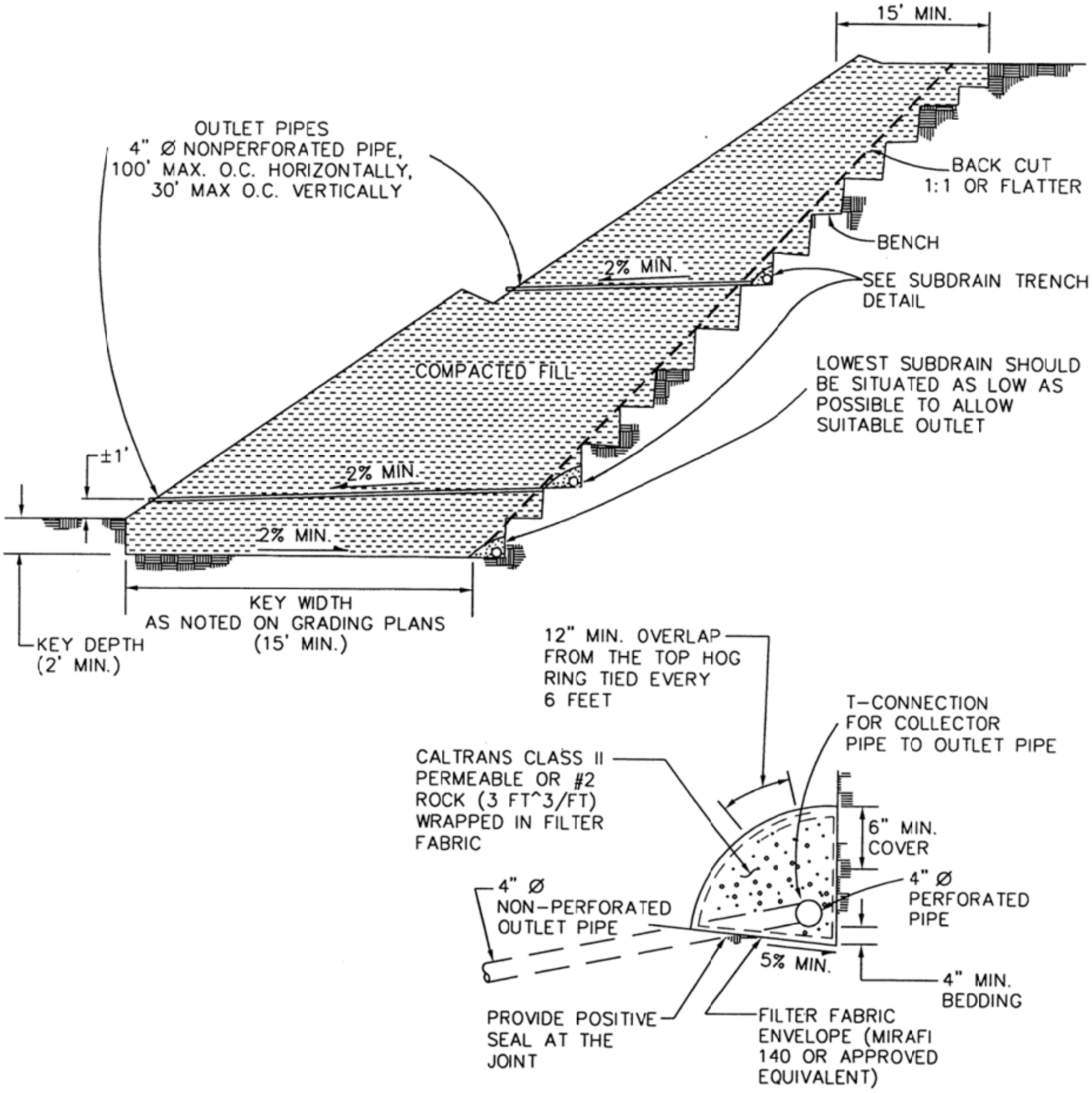
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C



SUBDRAIN TRENCH DETAIL

SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

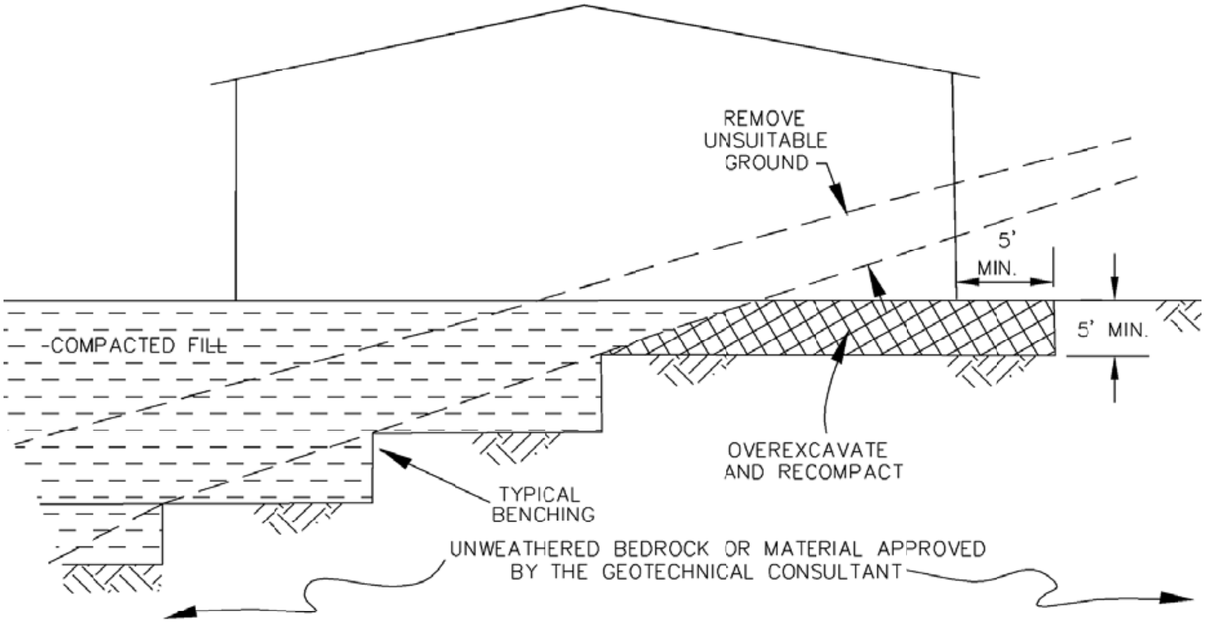
SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

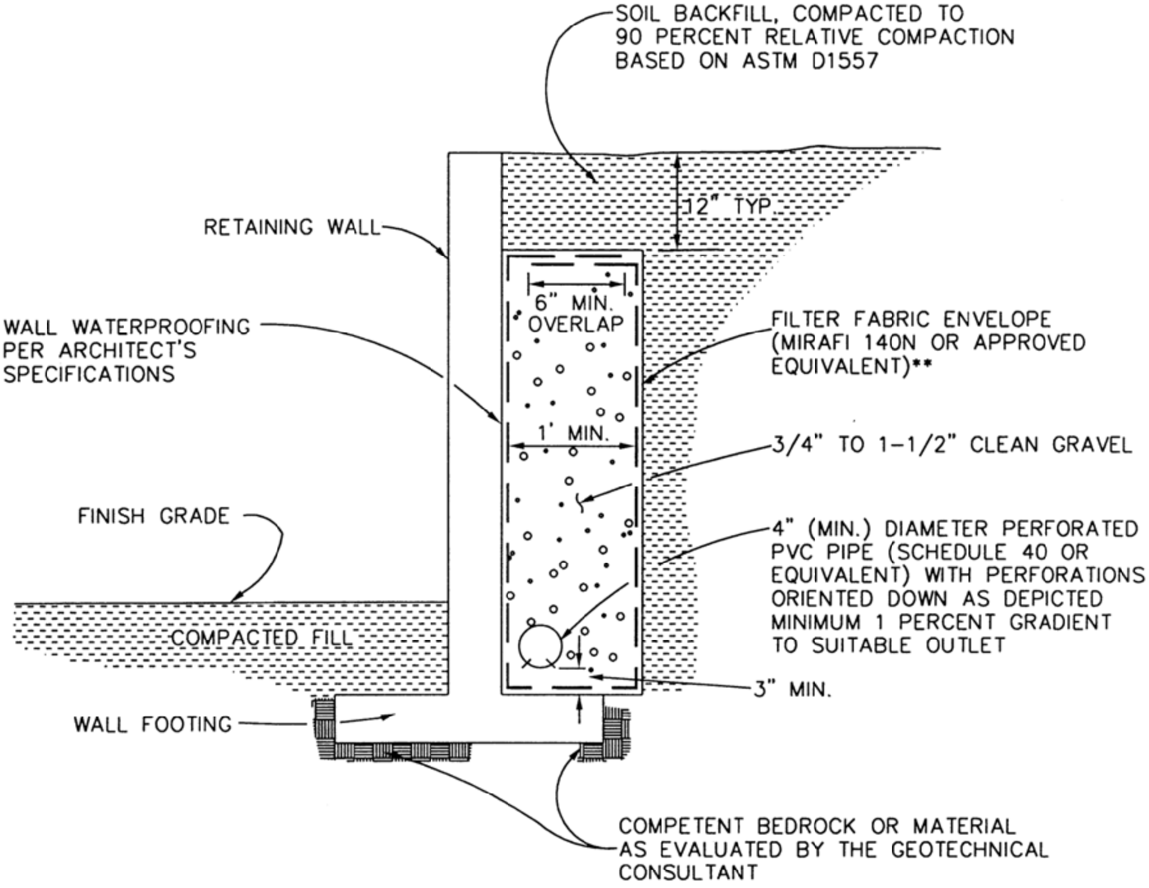
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D

CUT-FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

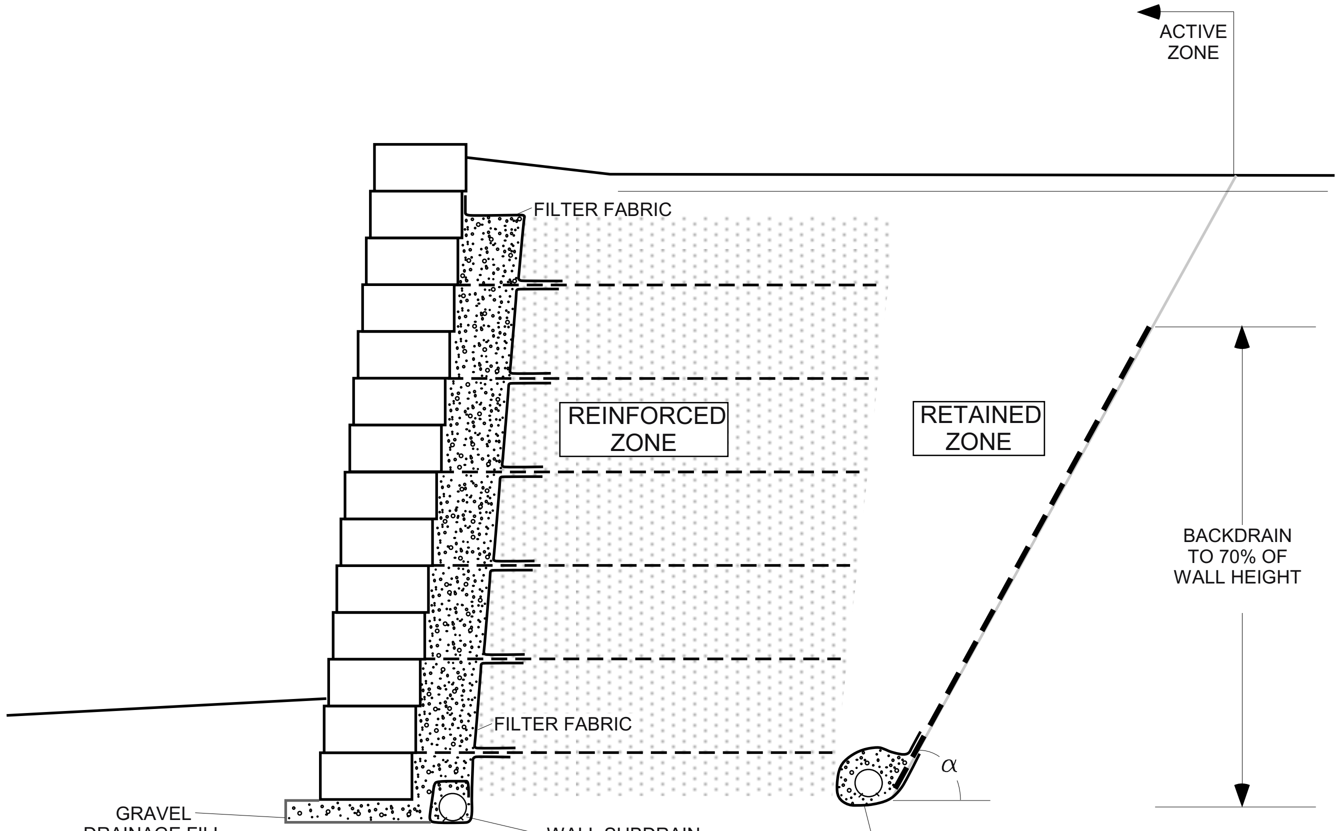
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F



GRAVEL
DRAINAGE FILL
MIN 6" BELOW WALL
MIN 12" BEHIND UNITS AND 24" FROM FACE
<u>WHERE WALL HEIGHT > 10'</u>
SAME AS ABOVE, BUT BELOW UPPER 10'
INCREASE GRAVEL TO
MIN 24" BEHIND UNIT AND 36" FROM FACE
<u>WHERE WALL HEIGHT > 20'</u>
SAME AS ABOVE, BUT BELOW UPPER 20'
INCREASE GRAVEL TO
MIN 36" BEHIND UNIT AND 48" FROM FACE

REAR SUBDRAIN:
4" (MIN) DIAMETER PERFORATED PVC PIPE
(SCHEDULE 40 OR EQUIVALENT) WITH
PERFORATIONS DOWN. SURROUNDED BY
1 CU. FT/FT OF 3/4" GRAVEL WRAPPED IN
FILTER FABRIC (MIRAFI 140N OR EQUIVALENT)

OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER,
BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20 AND LIQUID LIMIT < 40
FOR WALL HEIGHT 10 FEET OR TALLER, PLASTICITY INDEX < 6
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
FOR WALL HEIGHT > 20 FEET, REDUCE ALLOWABLE RANGE % PASSING NO. 200 SIEVE TO 0-15

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH AND UNIT WEIGHT REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45+\Phi/2$, WHERE Φ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL G

REVISED 11/16

APPENDIX G

Infiltration Rate Form I-8

Leighton

Categorization of Infiltration Feasibility Condition	FORM I-8

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Based on our field percolation testing, the in-situ infiltration rates of the soils within the limits of proposed residential development are generally less than 0.5 inches per hour (Leighton, 2021). The field infiltration rate is 0.01 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

The geotechnical hazards would not be increased provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the proposed limits of Hydromodification Basins at the subject site. The field infiltration rate is 0.01 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

<table>
<tr><th colspan="4">FORM I-8 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that the risk of groundwater contamination would not be increased provided there are no known contaminated soil or groundwater sites within 250 feet of the proposed Hydromodification Basins at the subject site. The field infiltration rate is 0.01 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that potential water balance issues would not be affected provided there are no unlined site drainages/creeks/streams within 250 feet of the proposed Hydromodification Basins at the subject site. The field infiltration rate is 0.01 inches per hour..

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2</td><td>Go to Part 2</td></tr>
</table>

FORM I-8 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	**Yes**	**No**
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Based on our field percolation testing, the in-situ infiltration rates of the soils within the limits of proposed the site are less than 0.5 inches per hour (Leighton, 2021), but not greater than 0.01 inches per hour. The field infiltration rate is 0.01 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	**Yes**	**No**
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

For a partial infiltration condition (greater than 0.01 inches per hour), the risk of geotechnical hazards will not be increased by partial infiltration provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the vicinity of proposed Hydromodification Basins at the subject site. Mitigation includes subsurface vertical barriers and subdrains to limit perched ground water mounding conditions.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

FORM I-8 Page 4 of 4

<table>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
For a partial infiltration condition (greater than 0.01 inches per hour), the risk of groundwater contamination will not be increased by partial infiltration provided there are no known contaminated soil or groundwater sites within 250 feet of the proposed Hydromodification Basins at the subject site.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
For a partial infiltration condition (greater than 0.01 inches per hour), violation of downstream water rights is not anticipated based on the site location and that there are no unlined site drainages/creeks/streams within 250 feet of the proposed Hydromodification Basins at the subject site.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.
If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration feasibility</td></tr>
</table>

APPENDIX H

GBA, Important Information About This Geotechnical-Engineering Report

Important Information about This

Geoenvironmental Report

Geoenvironmental studies are commissioned to gain information about environmental conditions on and beneath the surface of a site. The more comprehensive the study, the more reliable the assessment is likely to be. But remember: Any such assessment is to a greater or lesser extent based on professional opinions about conditions that cannot be seen or tested. Accordingly, no matter how many data are developed, risks created by unanticipated conditions will always remain. *Have realistic expectations.* Work with your geoenvironmental consultant to manage known and unknown risks. Part of that process should already have been accomplished, through the risk allocation provisions you and your geoenvironmental professional discussed and included in your contract's general terms and conditions. This document is intended to explain some of the concepts that may be included in your agreement, and to pass along information and suggestions to help you manage your risk.

Beware of Change; Keep Your Geoenvironmental Professional Advised

The design of a geoenvironmental study considers a variety of factors that are subject to change. Changes can undermine the applicability of a report's findings, conclusions, and recommendations. *Advise your geoenvironmental professional about any changes you become aware of.* Geoenvironmental professionals cannot accept responsibility or liability for problems that occur because a report fails to consider conditions that did not exist when the study was designed. Ask your geoenvironmental professional about the types of changes you should be particularly alert to. Some of the most common include:

- modification of the proposed development or ownership group,
- sale or other property transfer,
- replacement of or additions to the financing entity,
- amendment of existing regulations or introduction of new ones, or
- changes in the use or condition of adjacent property.

Should you become aware of any change, *do not rely on a geoenvironmental report.* Advise your geoenvironmental professional immediately; follow the professional's advice.

Recognize the Impact of Time

A geoenvironmental professional's findings, recommendations, and conclusions cannot remain valid indefinitely. The more time that passes, the more likely it is that important latent changes will occur. *Do not rely on a geoenvironmental report if too much time has elapsed since it was completed.* Ask your environmental professional to define "too much time." In the case of Phase I Environmental Site Assessments (ESAs), for example, more than 180 days after submission is generally considered "too much."

Prepare To Deal with Unanticipated Conditions

The findings, recommendations, and conclusions of a Phase I ESA report typically are based on a review of historical information, interviews, a site "walkover," and other forms of noninvasive research. When site subsurface conditions are not sampled in any way, the risk of unanticipated conditions is higher than it would otherwise be.

While borings, installation of monitoring wells, and similar invasive test methods can help reduce the risk of unanticipated conditions, *do not overvalue the effectiveness of testing.* Testing provides information about actual conditions only at the precise locations where samples are taken, and only when they are taken. Your geoenvironmental professional has applied that specific information to develop a general opinion about environmental conditions. *Actual conditions in areas not sampled may differ (sometimes sharply) from those predicted in a report.* For example, a site may contain an unregistered underground storage tank that shows no surface trace of its existence. *Even conditions in areas that were tested can change*, sometimes suddenly, due to any number of events, not the least of which include occurrences at adjacent sites. Recognize, too, that *even some conditions in tested areas may go undiscovered,* because the tests or analytical methods used were designed to detect only those conditions assumed to exist.

Manage your risks by retaining your geoenvironmental professional to work with you as the project proceeds. Establish a contingency fund or other means to enable your geoenvironmental professional to respond rapidly, in order to limit the impact of unforeseen conditions. And to help prevent any misunderstanding, identify those empowered to authorize changes and the administrative procedures that should be followed.

Do Not Permit Any Other Party To Rely on the Report

Geoenvironmental professionals design their studies and prepare their reports to meet the specific needs of the clients who retain them, in light of the risk management methods that the client and geoenvironmental professional agree to, and the statutory, regulatory, or other requirements that apply. The study designed for a developer may differ sharply from one designed for a lender, insurer, public agency...or even another developer. *Unless the report specifically states otherwise, it was developed for you and only you.* Do not unilaterally permit any other party to rely on it. The report and the study underlying it may not be adequate for another party's needs, and you could be held liable for shortcomings your geoenvironmental professional was powerless to prevent or anticipate. Inform your geoenvironmental professional when you know or expect that someone else—a third-party—will want to use or rely on the report. *Do not permit third-party use or reliance until you first confer with the geoenvironmental professional who prepared the report.* Additional testing, analysis, or study may be required and, in any event, appropriate terms and conditions should be agreed to so both you and your geoenvironmental professional are protected from third-party risks. *Any party who relies on a geoenvironmental report without the express written permission of the professional who prepared it and the client for whom it was prepared may be solely liable for any problems that arise.*

Avoid Misinterpretation of the Report

Design professionals and other parties may want to rely on the report in developing plans and specifications. They need to be advised, in writing, that their needs may not have been considered when the study's scope was developed, and, even if their needs were considered, they might misinterpret geoenvironmental findings, conclusions, and recommendations. *Commission your geoenvironmental professional to explain pertinent elements of the report to others who are permitted to rely on it, and to review any plans, specifications or other instruments of professional service that incorporate any of the report's findings, conclusions, or recommendations.* Your geoenvironmental professional has the best understanding of the issues involved, including the fundamental assumptions that underpinned the study's scope.

Give Contractors Access to the Report

Reduce the risk of delays, claims, and disputes by giving contractors access to the full report, *providing that it is accompanied by a letter of transmittal that can protect you* by making it unquestionably clear that: 1) the study was not conducted and the report was not prepared for purposes of bid development, and 2) the findings, conclusions, and recommendations included in the report are based on a variety of opinions, inferences, and assumptions and are subject to interpretation. Use the letter to also advise contractors to consult with your geoenvironmental professional to obtain clarifications, interpretations, and guidance (a fee may be required for this service), and that—in any event—they should conduct additional studies to obtain the specific type and extent of information each prefers for preparing a bid or cost estimate. Providing access to the full report, with the appropriate caveats, helps prevent formation of adversarial attitudes and claims of concealed or differing conditions. If a contractor elects to ignore the warnings and advice in the letter of transmittal, it would do so at its own risk. Your geoenvironmental professional should be able to help you prepare an effective letter.

Do Not Separate Documentation from the Report

Geoenvironmental reports often include supplemental documentation, such as maps and copies of regulatory files, permits, registrations, citations, and correspondence with regulatory agencies. If subsurface explorations were performed, the report may contain final boring logs and copies of laboratory data. If remediation activities occurred on site, the report may include: copies of daily field reports; waste manifests; and information about the disturbance of subsurface materials, the type and thickness of any fill placed on site, and fill placement practices, among other types of documentation. *Do not separate supplemental documentation from the report. Do not, and do not permit any other party to redraw or modify any of the supplemental documentation for incorporation into other professionals' instruments of service.*

Understand the Role of Standards

Unless they are incorporated into statutes or regulations, standard practices and standard guides developed by the American Society for Testing and Materials (ASTM) and other recognized standards-developing organizations (SDOs) are little more than aspirational methods agreed to by a consensus of a committee. The committees that develop standards may not comprise those best-qualified to establish methods and, no matter what, no standard method can possibly consider the infinite client- and project-specific variables that fly in the face of the theoretical "standard conditions" to which standard practices and standard guides apply. In fact, these variables can be so pronounced that geoenvironmental professionals who comply with every directive of an ASTM or other standard procedure could run afoul of local custom and practice, thus violating the standard of care. Accordingly, when geoenvironmental professionals indicate in their reports that they have performed a service "in general compliance" with one standard or another, it means they have applied professional judgement in creating and implementing a scope of service designed for the specific client and project involved, and which follows some of the general precepts laid out in the referenced standard. To the extent that a report indicates "general compliance" with a standard, you may wish to speak with your geoenvironmental professional to learn more about what was and was not done. *Do not assume a given standard was followed to the letter.* Research indicates that that seldom is the case.

Realize That Recommendations May Not Be Final

The technical recommendations included in a geoenvironmental report are based on assumptions about actual conditions, and so are preliminary or tentative. Final recommendations can be prepared only by observing actual conditions as they are exposed. For that reason, you should retain the geoenvironmental professional of record to observe construction and/or remediation activities on site, to permit rapid response to unanticipated conditions. *The geoenvironmental professional who prepared the report cannot assume responsibility or liability for the report's recommendations if that professional is not retained to observe relevant site operations.*

Understand That Geotechnical Issues Have Not Been Addressed

Unless geotechnical engineering was specifically included in the scope of professional service, a report is not likely to relate any findings, conclusions, or recommendations about the suitability of subsurface materials for construction purposes, especially when site remediation has been accomplished through the removal, replacement, encapsulation, or chemical treatment of on-site soils. The equipment, techniques, and testing used by geotechnical engineers differ markedly from those used by geoenvironmental professionals; their education, training, and experience are also significantly different. If you plan to build on the subject site, but have not yet had a geotechnical engineering study conducted, your geoenvironmental professional should be able to provide guidance about the next steps you should take. The same firm may provide the services you need.

Read Responsibility Provisions Closely

Geoenvironmental studies cannot be exact; they are based on professional judgement and opinion. Nonetheless, some clients, contractors, and others assume geoenvironmental reports are or certainly should be unerringly precise. Such assumptions have created unrealistic expectations that have led to wholly unwarranted claims and disputes. To help prevent such problems, geoenvironmental professionals have developed a number of report provisions and contract terms that explain who is responsible for what, and how risks are to be allocated. Some people mistake these for "exculpatory clauses," that is, provisions whose purpose is to transfer one party's rightful responsibilities and liabilities to someone else. Read the responsibility provisions included in a report and in the contract you and your geoenvironmental professional agreed to. *Responsibility provisions are not "boilerplate."* They are important.

Rely on Your Geoenvironmental Professional for Additional Assistance

Membership in the Geoprofessional Business Association exposes geoenvironmental professionals to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a geoenvironmental project. Confer with your GBA-member geoenvironmental professional for more information.

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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