Geotechnical Evaluation

Entrada Way Culvert Slip Out

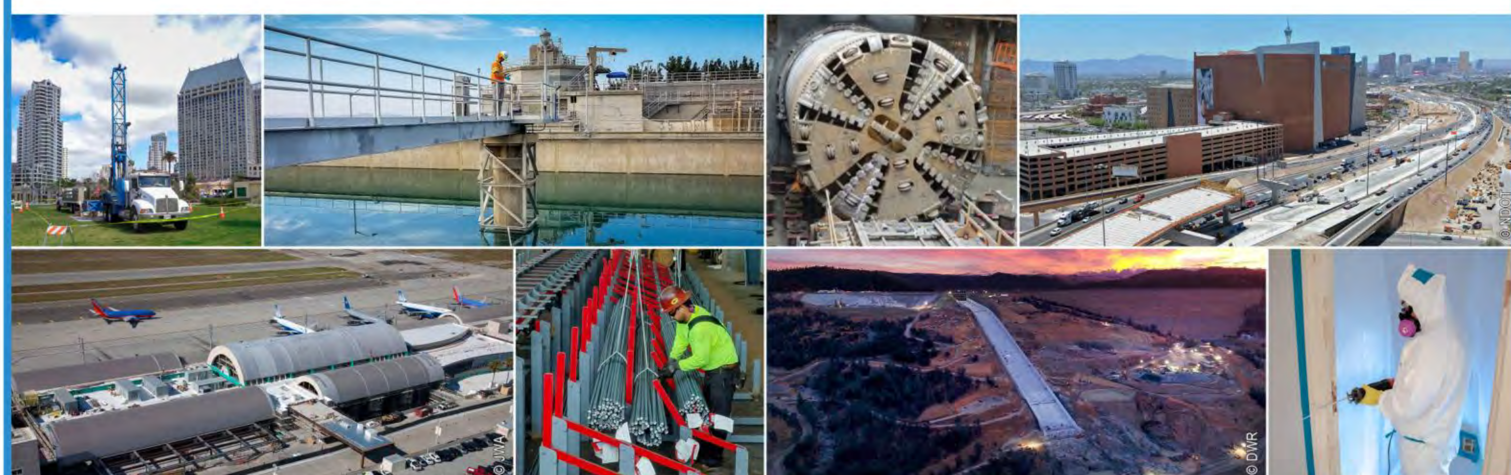
Entrada Way

La Honda, California

Lotus Water

660 Mission Street, 2nd Floor | San Francisco, California 94105

October 14, 2025 | Project No. 405000001



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Ninyo & Moore

A SOCOTEC COMPANY

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1 INTRODUCTION

In accordance with your request and authorization, Ninyo & Moore has performed a geotechnical evaluation of the Entrada Way bridge site located in La Honda, California (Figure 1). The scope of our evaluation was conducted in accordance with our Proposal No. 08SJO02-02749, dated August 7, 2024. The purpose of our study was to evaluate subsurface conditions at the Entrada Way bridge and provide the design team with lateral loading design parameters and support on slope stability for rock armor wall and other potential stabilization options. This report presents the findings and conclusions from our geotechnical evaluation, and our recommendations for the geotechnical aspects of design and construction of this project.

In 1972, a Foundation Investigation of the Entrada Way Bridge was performed by Leighton-Yen & Associates, Inc. Additionally, in 2009, a Soil and Foundation Report (Garbe) and an Engineering Geologic Reconnaissance (Hoexter) were performed pre-construction for 8800 La Honda Road, located in the northeast corner of the La Honda Road/Entrada Way intersection. Data from these previous evaluations was reviewed and used to the extent possible in the preparation of this report.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Reviewing readily available geologic and seismic literature pertinent to the project area including geologic maps and previous geotechnical reports, landslide maps, regional fault maps, seismic hazard maps, and other information provided by the client.
- Performing a site reconnaissance to observe the general site conditions and mark the boring locations.
- Coordinating with Underground Service Alert to locate the underground utilities in the vicinity of the proposed exploratory borings.
- Subcontracting with a private utility locator to further check the boring locations for underground utility conflicts.
- Obtaining the proper permit(s) from the County of San Mateo.
- Preparing a traffic control plan and subcontracting with a traffic control crew.
- Performing a subsurface exploration consisting of drilling, sampling and logging two exploratory borings to depths up to 25¾ feet below the existing ground surface (bgs).
- Performing laboratory tests on selected soil samples to evaluate soil moisture and density, #200 wash, soil gradation, Atterberg limits, and shear strength.
- Compiling and analyzing the field and laboratory data and the findings from our geologic review to evaluate the site conditions.

- Preparing this geotechnical report presenting the findings and conclusions from our evaluation including our geotechnical recommendations for mitigation and restoration of the slope failure.

3 SITE DESCRIPTION AND BACKGROUND

The Entrada Way bridge is located between Ventura Avenue and State Route 84 (La Honda Road) in La Honda, California (Figure 2). The bridge is relatively flat, with elevations ranging from about 398 feet above NAVD 88 mean sea level (MSL) in the east to about 401 feet MSL in the west. The site is located within the USGS La Honda 7.5-Minute Quadrangle.

Based on our discussions with you, we understand a big damaging event occurred in 2017 that resulted in erosion and loss of some of the roadway embankment. The observed erosion is located on the northeast abutment, at approximate coordinates 37.3191° north latitude, 122.2736° west longitude. Temporary repairs were performed. However, it is our understanding that the creek is migrating into the left bank of the existing culvert. This project aims at providing a resilient and long-lasting solution for the roadway embankment. We understand that stabilization alternatives may include soldier pile walls and lagging and may include wooden (timber) piles.

4 FIELD EXPLORATION AND LABORATORY TESTING

4.1 Field Exploration

Our field exploration included a site reconnaissance and subsurface exploration of the project site. The subsurface exploration, conducted on December 13, 2024, consisted of drilling, logging and sampling of two hollow-stem auger geotechnical borings. Prior to commencing the subsurface exploration, we coordinated with Underground Service Alert (USA) and retained a private utility locator to locate and mark existing utilities in the vicinity of our exploration locations. The approximate locations of our subsurface explorations are presented on Figure 2. We had planned to drill one boring on the opposite side of the road but overhead powerlines were present and we could not drill there.

4.1.1 Geotechnical Borings

Ninyo & Moore engaged California Geotech Services, LLC, headquartered in Livermore, California, to drill two geotechnical borings with a Mobile B-24 truck-mounted drill rig, equipped with 4-inch-diameter solid-flight augers. The exploratory borings were advanced to depths up to 25¾ feet bgs. Bulk samples for the upper 5 feet of the subsurface were obtained from the auger cuttings. Disturbed and relatively undisturbed soil samples were collected at selected depths from within the boreholes. A brief explanation of the field procedures for collecting the samples and the boring logs are presented in Appendix A.

A representative from Ninyo & Moore logged the samples in the field before transporting them to our geotechnical laboratory for testing. These field logs were then used to develop the boring logs included with this report (Appendix A). The borings were backfilled with neat cement grout and patched with fast-setting concrete mix at the completion of drilling and sampling. Excess soil cuttings generated during drilling were dispersed in unpaved areas.

4.2 Laboratory Testing

Geotechnical laboratory testing of soil samples recovered from the borings included in-place moisture content and density, Atterberg Limits, percent of particles finer than the No. 200 sieve, soil gradation, and shear strength. The results of the in-place moisture content and dry density tests are shown at the corresponding sample depths on the boring logs in Appendix A. The results of the remaining laboratory tests performed are presented in Appendix B.

5 GEOLOGIC AND SUBSURFACE CONDITIONS

5.1 Regional Geologic Setting

The site is located in the Santa Cruz Mountains, which are part of the Coast Ranges geomorphic province of California (Figure 1). The Coast Ranges have experienced a complex geological history that has resulted in a series of northwest-trending mountain ranges and valleys. The present physiography and geology of the Coast Ranges are a result of deposition and deformation along the boundary between the North American plate and the Pacific plate, a tectonic setting experiencing both translational and compressional stresses. Movement along this boundary is largely concentrated along well-known fault zones, including the San Andreas, Calaveras, and Hayward faults, as well as associated lesser-order faults.

Bedrock in the Coast Ranges typically consists of igneous, metamorphic, and sedimentary rock ranging in age from the Jurassic to the Pleistocene.

5.2 Site Geology

Regional mapping by Brabb et, al. (1998) indicates the site is underlain by the Tehana Member (Tpt) of the Purisima Formation. The Tehana Member (Pliocene and upper Miocene) consists of very fine- to medium-grained lithic sandstone and siltstone, with some silty mudstone. A map of the regional geology is presented as Figure 3.

5.3 Subsurface Conditions

The following sections provide a generalized description of the geologic units encountered during our subsurface evaluation at the project site. More detailed descriptions are presented on the boring logs in Appendix A. Our interpretation of the subsurface conditions is depicted on the geologic cross-section presented as Figure 6.

5.3.1 Pavement Section

We encountered paved surfaces at both boring locations. The pavement section at both locations consisted of 3-inches of asphalt concrete (AC) over 9-inches of aggregate base (AB) rock.

5.3.2 Artificial Fill

Artificial fill was encountered in both locations. The fill extended from the bottom of the pavement section to approximately 7½ feet bgs in each boring. In general, the fill consisted of very sandy lean clay with gravel.

5.3.3 Alluvium

We encountered alluvium beneath the fill in each boring. In Boring B-1, the alluvium was approximately 14 feet thick and consisted of stiff lean clay overlying stiff fat clay, with medium dense sand at the bottom of the unit. In Boring B-2, the alluvium was approximately 7½ feet thick, and consisted of very stiff fat clay with clasts of decomposed sandstone.

5.3.4 Residual Soil

We encountered residual soil beneath the alluvium at a depth of about 21½ feet in boring B-1 and about 14 feet in boring B-2. The encountered residual soil consisted of dark yellowish-brown to greenish-gray, wet, medium dense clayey sand with fine grained sandstone clasts.

5.3.5 Bedrock (Sandstone)

Bedrock was encountered below the residual soil to the terminal depth of both borings. In general, the bedrock consisted of moderately hard, fine-grained sandstone with variable amounts of weathering.

5.4 Groundwater

Groundwater was encountered in Boring B-1 at a depth of approximately 18 feet bgs. Groundwater may rise to a level higher than observed due to the relatively low seepage rate in clay and limited time for observation. Fluctuations in groundwater levels will occur due to variations in precipitation, ground surface topography, creek water surface flow and elevation, subsurface stratification, irrigation, groundwater pumping, and other factors that may not have been evident at the time of our field evaluation. In addition, seeps may be encountered at elevations above the groundwater levels encountered due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

6 GEOLOGIC HAZARDS AND CONSIDERATIONS

This study considered a number of geologic and seismic hazards relevant to the site including seismic hazards, landsliding, slope stability, and regional ground subsidence. These hazards are discussed in the following subsections.

6.1 Seismic Hazards

The seismic hazards considered in this study include the potential for ground surface rupture due to faulting, seismic ground shaking, and seismic slope stability. These potential hazards are discussed in the following subsections.

6.1.1 Historical Seismicity

Figure 4 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000. Table 1, below, summarizes historical seismic events in the region based on a search of the USGS Earthquake Catalog for events between 1800 and today, with a magnitude of 6.5 or more, and located within a 100-kilometer (62-mile) radius of the site.

Table 1 – Summary of Historical Seismicity[1]

Event ID	Date	Magnitude	Approximate Distance from Site	
			Km	Miles
The 1989 Loma Prieta Earthquake	10-18-1989	6.9	47.0	29.2
The 1906 San Francisco Earthquake	04-18-1906	7.9	53.8	33.4
The 1868 Hayward Fault Earthquake	10-21-1868	6.8	45.0	28.0
South of San Jose	10-08-1865	6.5	35.6	22.1
Near San Juan Bautista	01-18-1840	6.5	86.2	53.6
The 1838 San Andreas Fault Earthquake	06-25-1838	7.4	11.1	6.9

Note:
United States Geological Survey (USGS), Earthquake Hazards Program, *https://earthquake.usgs.gov/earthquakes/search/*

Records compiled by Schmitt et al. (2017), indicate that no ground effects related to historical seismic activity (e.g., liquefaction, sand boils, lateral spreading, ground cracking, landsliding) have been reported for the site vicinity.

6.1.2 Ground Surface Fault Rupture

The site is not located within an Alquist-Priolo (AP) Earthquake Fault Zone established by the State Geologist (CDMG, 1983). These zones delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (CGS, 2018), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,700 years. The regional fault map presented as Figure 4 indicates that several faults are located near the site. The closest fault rupture hazard zone to the site is associated with the San Andreas fault, which is within approximately 4¾ miles of the site to the west.

Based on our review of the referenced geologic maps, known active faults are not mapped on the subject site and the site is not located within a fault-rupture hazard zone established by the California Geological Survey. Based on this fault rupture is not considered to be a design consideration. However, some localized ground cracking or lurching may occur during seismic events.

6.1.3 Strong Ground Motion

Based on historic activity, the potential for future seismic ground motion at the site is considered significant. Seismic design criteria to address ground shaking are provided in Section 8.1. The peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE_G) was calculated in accordance with the American Society of Civil Engineers (ASCE) 7-16 Standard and the 2022 California Building Code (CBC). The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.989g using the seismic design tool developed by the Structural Engineers Association of California in conjunction with the Office of Statewide Health Planning and Development (SEAOC/OSHPD, 2023). The PGA_M is based on a mapped MCE_G peak ground acceleration of 0.899g for the site and a site coefficient (F_{PGA}) of 1.1 for Site Class D. Site Class D was selected based on our subsurface exploration and review of CGS Map Sheet 48 (CGS, 2016), which provided a shear wave velocity for the site of approximately 1,150 feet per second (fps) over a depth of 30 meters (Vs30).

6.1.4 Seismic Slope Stability

The proposed project is located within an area that has not yet been evaluated by the CGS for earthquake-induced landslides.

6.2 Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soil containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this volume change can damage structures and flatwork. Laboratory testing was performed on near-surface soil samples to evaluate the Atterberg Limits of the soil. The tests were performed in accordance with the American Society of Testing and Materials (ASTM) Standard D4318 (Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils). The results of our laboratory test indicate that samples tested had a plasticity index (PI) varying from 21 to 44. This result is indicative of a moderate to high expansion characteristic. To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, foundations should be designed for expansive soils. We anticipate that suitable foundation embedment depths and subgrade preparation can be used to mitigate the expansive soil conditions.

6.3 Flooding

6.3.1 Riverine

Based on the flood insurance rate map (FIRMs) prepared by the Federal Emergency Management Agency (FEMA, 2012), La Honda Creek is a regulatory floodway.

6.3.2 Dam Inundation

Properties located downstream of dams can be inundated with flood waters if the dams were to fail. Dam owners are required to prepare inundation maps showing the limits of flooding caused by dam failure. There are no dams or large reservoirs located upstream of the site that could fail and inundate the site (DSOD, 2024). As such, inundation due to dam failure is not a design consideration for this project.

6.3.3 Tsunami and Seiche Inundation

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. The Tsunami Hazard Area Map for San Mateo County (State of California, 2021) maps the site outside a tsunami hazard area.

Seiches are waves generated in a large enclosed body of water. Based on the inland location of the site and considering that there are no large enclosed bodies of water nearby, the potential for damage due to tsunamis or seiches is not a design consideration.

6.4 Unsuitable Materials

Fill materials that were not placed and compacted under the observation of a geotechnical engineer, or fill materials lacking documentation of such observation, are considered to be undocumented fill and unsuitable as a bearing material below foundations due to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. Undocumented fill was encountered in our borings at the site. The fill extended to depths of up to approximately 7½ feet bgs.

6.5 Static Settlement

The subsurface conditions encountered in our borings generally consisted of stiff to very stiff sandy lean and fat clays; medium dense clayey sand; and moderately hard bedrock. Highly compressible soils were not encountered during our field investigation.

The sustained loads for the proposed project are expected to be relatively light. If other structures are planned, we anticipate that the total static settlement of structures due to sustained loads will be less than 1 inch provided the recommendations presented in this report are followed.

7 CONCLUSIONS

Based on our review of the referenced background data, our site field reconnaissance, and subsurface evaluation, it is our opinion that there are no known geotechnical constraints that would preclude construction of the project provided that the recommendations presented in this report are incorporated into the design and construction of the proposed improvements. Key findings from our geotechnical evaluation and subsurface exploration include the following:

- The subsurface exploration for this study encountered fill, alluvium, residual soil, and bedrock. The encountered fill and alluvium consisted primarily of cohesive soils (lean and fat clays).
- Groundwater was encountered at a depth of approximately 18 feet below existing grade at location B-1. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 5.4.
- The site will experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault.
- Ground surface rupture due to faulting is not a design consideration based on the location of the project.

- Based on deep foundation support for the retaining wall, we estimate settlement up to approximately ½ inch with a differential static settlement of up to ¼ inch.
- Excavations that remain unsupported, are exposed to water, encounter seepage, or expose granular soil may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

8 RECOMMENDATIONS

The following sections present our geotechnical recommendations for design and construction of a soldier pile and lagging wall, and earthwork. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, relevant grading ordinances, and appropriate construction practices. The geotechnical engineer should observe and evaluate excavations for earthwork and foundation constructions. Evaluations performed by the geotechnical engineer during the course of construction may result in new recommendations, which could supersede the recommendations provided in this section.

We recommend that the construction drawings be submitted to Ninyo & Moore for review to evaluate conformance to the geotechnical recommendations provided in this report. We further recommend that a pre-construction conference be held in order to discuss the recommendations presented in this report. The County of San Mateo and/or their representatives, other governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

8.1 Seismic Design Criteria

Table 2 presents the Risk-Targeted, Maximum Considered Earthquake (MCE_R) spectral response accelerations consistent with the 2022 California Building Code (CBC) and corresponding site-adjusted and design level spectral response accelerations based on the USGS seismic design maps (SEAOC/OSHPD, 2023). Seismic Site Class D was selected based on review of CGS Map Sheet 48 (CGS, 2016). The seismic design criteria provided in the table may be used for structures with a fundamental period of ½ second or less such that the exception to Site Class D in Section 20.3.1-1 of ASCE Standard 7-16 is applicable.

Table 2 – California Building Code Seismic Design Criteria

Seismic Design Parameter Evaluated for 37.5711ºNorth latitude, 121.9697ºWest longitude	Value
Site Class	D – Stiff Soil
Site Coefficient, F_a	1.0
Site Coefficient, F_v	--
Mapped Spectral Acceleration at 0.2-second Period, S_s	2.208 g

Table 2 – California Building Code Seismic Design Criteria	
Seismic Design Parameter Evaluated for 37.5711ºNorth latitude, 121.9697ºWest longitude	**Value**
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.729 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	2.208 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	--
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1. 472 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	--
Seismic Design Category for Risk Category II	D

8.2 Retaining Wall

Recommendations are presented in the following sections for a soldier-pile-and-lagging wall. Note that other wall types with deep foundation support may also be viable. Ninyo & Moore can review alternative wall types and provide additional recommendations, as needed.

8.2.1 Soldier-Pile-and-Lagging Wall

A soldier-pile-and-lagging wall should consist of wide-flange steel beams set into a drilled hole and backfilled with concrete, with wood, steel, or pre-cast concrete lagging to support the retained soil between the soldier piles. We understand that wooden (timber) piles may also be considered for use at this site. Wood lagging, if selected, should be appropriately treated to reduce the potential for deterioration and potential damage from fire. We recommend that the wall be designed with free-board for collection of slope debris, and periodic maintenance for removal of accumulated debris will be required. The area immediately upslope of the wall should be easily accessible for such maintenance. The planned location and construction of the wall should take into consideration existing utilities, property/easement boundaries, and roadway shoulder requirements.

For vertical downward loading, the soldier piles should be designed for an allowable side friction of 50*D (where D is the depth) pounds per square foot (psf) per foot of embedment into dense to very dense granular soils. For piers drilled into bedrock, the friction will depend upon the design and construction method(s) and we can provide allowable friction and end bearing values later in discussions with the structural engineer. For preliminary planning and design, for embedment into the bedrock, we recommend using 1,000 psf allowable skin friction and 25 ksf allowable end bearing. The allowable side friction can be increased by one-third for all loads, including wind or seismic. Due to the primarily cohesive nature of the soils encountered in our borings above bedrock, we recommend pre-drilling holes slightly larger in diameter than the pile diameter and then backfill with grout to ensure a tight fit and minimize disturbance.

Soldier piles should extend a minimum of 2 times the proposed pile diameter into dense to very dense granular soils, or extend a minimum of 1.5 times the pile diameter into moderately hard bedrock. A layer of dense granular soil, approximately 1½ feet thick, was encountered in boring B-1 at a depth of approximately 20 feet. Bedrock was encountered at a depth of approximately 25 feet in boring B-1 and at approximately 15½ in boring B-2. The required depth of embedment to resist lateral loads should be evaluated by a structural engineer. Ninyo & Moore should provide observations during construction to confirm the depth of embedment.

Soldier-pile-and-lagging walls that retain site soil may be designed for the lateral earth pressures in Table 3. A soil unit weight of 110 pcf and friction angle of 30 degrees may be used for design in drained conditions.

Table 3 – Lateral Earth Pressures for Soldier-Pile-and-Lagging Walls

Backfill or Ground Slope (H:V)	Active + Surcharge Lateral Earth Pressure[1] (psf)	At-Rest + Surcharge Lateral Earth Pressure [2] (psf)	Seismic + Active Lateral Earth Pressure[3] (psf)	Allowable Passive Lateral Earth Pressure[4] (psf)
2:1	66·(H+D) + 80	69·(H+D) + 120	24·(H+D) + 66·(H+D)	165·D
Level ground	36 (H+D) + 80	55 (H+D) + 120	24·(H+D) + 36·(H+D)	165·D

Notes:

1 Equivalent fluid active lateral earth pressure plus uniform lateral earth pressure due to 240 psf surcharge. Acts on lagging and drilled hole diameter below lagging where soldier pile drilled hole filled with concrete.
"H" is height of lagging. "D" is soldier pile depth below lagging.

2 Equivalent fluid at-rest lateral earth pressure plus uniform lateral earth pressure due to 240 psf surcharge. Acts on lagging and drilled hole diameter below lagging where soldier pile drilled hole filled with concrete.

3 Equivalent fluid seismic lateral earth pressure plus equivalent fluid active lateral earth pressure. Acts on lagging and drilled hole diameter below lagging where soldier pile drilled hole filled with concrete.

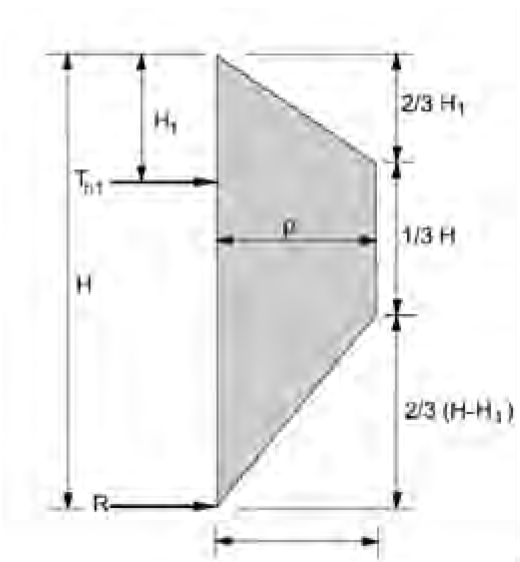
4 Equivalent fluid passive lateral earth pressure. Acts on soldier pile below lagging considering an effective pile width equivalent to 300 percent of concrete-filled hole diameter but not more than center-to-center pile spacing. Includes Safety Factor of 2.0.

Cantilever soldier-pile-and-lagging walls may be designed for active or at-rest lateral earth pressures. Wall deflection equivalent to about ¼ percent of wall height may be needed to reduce at-rest earth pressures to the active condition. Recommended active and at-rest lateral earth pressures are provided in Table 3 for sloping and level backfill conditions.

The lateral earth pressure due to a backfill surcharge of 240 psf is provided in Table 3 for active and at-rest conditions.

Cantilever soldier-pile-and-lagging walls with a lagging height more than 6 feet should consider seismic lateral earth pressure in conjunction with the active lateral earth pressure for seismic conditions. Recommended equivalent fluid seismic and active lateral earth pressures are provided in the Table 3. The values provided in the table include consideration for the cohesion of the native soil and presume that the vertical seismic coefficient (kv) is zero and the horizontal seismic coefficient (kh) is 50 percent of PGA_M but not more than the yield coefficient (ky) for global wall stability.

For a soldier pile wall with ground anchors, for a single anchor row, a trapezoidal distribution as shown below should be used with an anchor load based on 1,050 psf earth pressure for at-rest conditions and 700 psf for active conditions. The load is applied to the lagging and to the supporting piles if the piles are embedded such that there is a difference in height of soil on one side of the piles vs. the other, i.e., the piles are essentially a part of the retaining wall.



Where there is a short slope length of 6 to 8 feet, at a 2H to 2.25H to 1V behind the wall, the earth pressure should be increased by 25 percent.

Seismic pressure of 230 psf should be applied to the wall as a uniform pressure with depth.

Passive earth pressure support/resistance should begin at a point 2 feet below the anticipated future bottom of the creek channel.

For ground anchor capacity, our geotechnical investigation encountered refusal at a layer of sandstone beneath the site. For preliminary planning and design purposes, we recommend an unfactored bond stress of 50 psi, minimum bonded length of 25 ft and unbonded length of 10 ft.

8.2.2 Excavations

The geotechnical engineer should observe the drilling and construction of the soldier piles and installation of the lagging. Deeper drilling will encounter bedrock which may result in difficult drilling conditions. Drilled holes for soldier pile installation may need to be stabilized by use of temporary casing or drilling slurry. Standing water should be removed from the drilled hole before placement of concrete to backfill the drilled hole around the pile. Alternatively, a tremie pipe could be used to deliver the concrete to the bottom of the drilled hole below standing water. Casing should be removed from the drilled hole as the concrete is placed and the concrete should be placed in a manner that reduces the potential for segregation of the components or impacting the side of the excavation. Drilled holes above the planned bottom of lagging should be backfilled with CLSM to stabilize the hole while the excavation proceeds and lagging is installed. To reduce the potential for settlement of the retained soil, voids behind the lagging or the soldier piles should be backfilled with compacted fill during installation of the lagging.

8.2.3 Drainage

Drainage should be provided behind the lagging to mitigate the buildup of hydrostatic pressure from seeping groundwater. The drainage may consist of geocomposite strip drains (Miradrain 6000XL or similar) about 12 to 16 inches wide pinned to the face of the slope behind the lagging, fabric side to soil as the excavation proceeds downward in advance of lagging installation. The geocomposite drains should be arranged in vertical strips at a horizontal spacing equivalent to half the center-to-center soldier pile spacing with about 12 inches of overlap over lower drains to promote vertical flow. Weep holes should be provided to collect water from near the bottom of the vertical geocomposite drains and divert it to a swale or onto ground that slopes away from the wall at no less than 5 percent or no less than 2 percent if paved. As an alternative to weep holes, the strip drains may be extended under the lagging into a subdrain trench constructed in front of the wall. The subdrain trench should be backfilled with well-graded permeable aggregate conforming with the criteria for Class 2 permeable material in Section 68 of the California Standard Specifications (Caltrans, 2022) and capped with pavement, flatwork, or one foot of native soil. Perforated pipe (Schedule 40 PVC) should be placed within the subdrain trench and sloped at 1 percent or more towards a solid collector pipe (Schedule 40 PVC). Approximately 4 cubic feet of well-graded permeable aggregate per linear foot should be placed around the perforated pipe. The collector pipe should be sloped at 2 percent or more to discharge at a suitable outlet away from the wall. Cleanouts should be provided to facilitate periodic maintenance. The outfall from the collector pipe should be equipped with an energy dissipater and rodent screens.

Additional drainage improvements should include: (1) clearing of the surface drainage inlets; (2) evaluating the integrity of existing surface drainage collection/discharge system and updating as needed; and, (3) installing an energy dissipater and rodent screens at the outfall location for the collector pipe. Additional measures should be considered to protect drainage inlets from blockage due to soil or debris.

8.3 Earthwork

Earthwork associated with the proposed project is expected to include site clearing, excavations, subgrade preparation, and fill placement, and finish grading to establish site drainage. Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented below. The geotechnical consultant should observe and evaluate excavations for earthwork and foundation construction. Evaluations performed by the geotechnical consultant during the course of construction may result in new recommendations, which could supersede the recommendations provided in this section.

8.3.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the recommendations presented in the report. The owner and/or their representative, the engineer, Ninyo & Moore and the contractor should be in attendance to discuss project schedule and requirements for earthwork and foundations.

8.3.2 Site Preparation

Site preparation should begin with the demolition of the designated existing improvements and removal of vegetation, utility lines, surface obstructions (e.g., pavements, aggregate base, curb/gutter, foundations), rubble and debris, and other deleterious materials from areas to be graded. Vegetation should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be removed from the site for disposal in an appropriate landfill. Soil containing roots or other organic matter may be stockpiled for later use as landscaping fill, as authorized by the owner's representative. Active utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout. Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

8.3.3 Excavation Stabilization

Excavations should be stabilized by benching or laying slopes back in accordance with the Occupational Safety and Health Administration (OSHA) Excavation Rules and Regulations.

Alternatively, an internally-braced shoring system or trench shield conforming to the OSHA Excavation Rules and Regulations may be used to stabilize excavation sidewalls. Recommended OSHA material type classifications for site soil are provided in Table 4 along the corresponding allowable temporary slope inclinations and lateral earth pressures for design or selection of excavation shoring. The recommendations listed in this table are based upon the limited subsurface data provided by our subsurface exploration and reflect the influence of the environmental conditions that existed at the time of the exploration. The recommendations also presume that the excavations will not extend below a plane extending down and away from the foundation bearing surfaces of existing adjacent structures at an angle of 2:1 (horizontal to vertical). Dewatering should be performed as-needed to depress groundwater levels below the bottom of excavations.

Table 4 – OSHA Material Classifications and Allowable Slopes

Material	OSHA Classification	Allowable Temporary Slope	Lateral Earth Pressure on Shoring (psf)
Fill & Alluvium (above groundwater)	Type B	1h:1v (45°)	45×D + 72

Note:
'D' is depth of excavation in feet

Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse. The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The contractor should take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design.

Excavation subgrade, if exposed to water, may become unstable and subject to pumping under heavy equipment loads. The contractor should be prepared to stabilize subgrades after exposure to water. In general, unstable subgrade conditions may be mitigated by scarification and aeration to dry the soil to the optimum moisture content or treating the soil with quicklime. Alternatively, unstable subgrade may be removed and replaced with engineered fill. Construction of a bridging layer consisting of crushed rock or granular fill with geotextile may be needed to support the engineered fill layer so that the specified compaction can be achieved. Appropriate mitigation measures will be influenced by the conditions encountered. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed.

8.3.4 Subgrade Observation

Following completion of site clearing and stripping and prior to placement of fill, the exposed subgrade should be observed by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of Ninyo & Moore in accordance with the recommendations provided in this section or supplemental recommendations by the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil, and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from excavation bottoms and below bearing surfaces to a depth at which suitable subgrade is exposed, as evaluated in the field by Ninyo & Moore.

8.3.5 Fill Material Recommendations

In general, fill should not consist of pea gravel and should be free of rocks or lumps in excess of 3-inches in median dimension, hazardous materials above appropriate screening levels, trash, debris, and vegetation or other deleterious material. The on-site soil is generally not suitable in the current state, from a geotechnical perspective, for reuse as structural fill. The on-site soil will need to be excavated and blended with low expansion potential soil, processed to meet the preceding criteria and the criteria for import fill discussed below, and moisture conditioned to achieve a moisture content approximately 2 percentage points above the optimum. Moisture conditioning by adding water or by mixing and aerating to reduce the moisture content should be anticipated. Site soil, if saturated due to exposure to groundwater or rainy conditions may need an extended drying time to achieve the optimum moisture content.

In addition to the preceding criteria, imported fill should be close graded with 35 percent or more by dry weight passing the No. 4 sieve and either: an expansion index of 50 or less, a plasticity index of 12 or less, or less than 10 percent by dry weight passing the No. 200 sieve. Imported fill should also meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million [ppm] or less, a soluble sulfate content of approximately 0.15 percent [1,500 ppm] or less, and a pH value of 5.5 or more). Material considered for use as fill should be evaluated before it is imported to the site. The contractor should be responsible for the uniformity of import material brought to the site.

The native soil is suitable for other uses including the modeling of the engineered log structures (ELS). We understand that this modeling uses one standard set of parameter values (from a Table titled “Bank Soil Properties Lookup Table”, provided by the project team) for a soil type, unless modified by site conditions. The closest soil type to the site soils is “Clay, firm”. For this site the following two recommended values are slightly different than the Table values, all other values should be taken from the Table:

- Average Dry Unit Weight: 86 pcf
- Void Ratio: 0.9

8.3.6 Fill Placement and Compaction

The exposed subgrade in areas to receive fill should be scarified and moisture conditioned as needed to achieve a moisture content about 2 percentage points above the optimum, before compaction, by mechanical means, to 90 percent or more of the reference density as evaluated by ASTM D1557 on a dry density basis. Fill should then be placed and compacted in lifts by hand tampers or mechanical means to 90 percent of the ASTM D1557 reference density. The fill should be moisture conditioned to approximately 2 percentage points above the optimum before it is compacted. The allowable lift thickness is influenced by the type of compaction equipment utilized but generally should not exceed 8 inches in loose thickness. Compacted fill should be maintained in a moist condition by sprinkling water or covering with plastic. Compacted fill that has dried out and loosened or developed desiccation cracking should be scarified, moisture-conditioned, and recompacted as per the requirements above before additional fill is placed.

8.3.7 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with the recommendations in Section 8.3.3. Loose or soft soil exposed at the bottom of the trench should be removed or compacted to achieve a firm condition. Conduits and pipelines should be supported on granular bedding material that extends from the springline of the pipe to 6 inches below the pipe. Granular bedding and pipe zone material should consist of aggregates ordinarily used for highway base and subbase with 100 percent by dry weight passing the 1-inch sieve, 90 to 100 percent passing the No. 4 sieve, and no more than 5 percent passing the No. 200 sieve. Controlled low strength material (CLSM) may be used as an alternative to granular bedding and pipe zone material. CLSM should consist of a mixture of water, Portland cement, fly ash, and sound aggregate that flows without segregation of aggregates and produces an unconfined compressive strength of 50 to 150 pounds per square inch (psi) with a compressive strength of 50 psi developed about 1 hour after placement.

Prior to placement of bedding material, excavation subgrade should be compacted to a firm condition as needed. Debris, organic matter, or other unsuitable material exposed at the bottom of the excavation should be removed and replaced with additional bedding. Where a firm subgrade condition cannot be achieved by compaction before placement of bedding, the unstable subgrade should be overexcavated to a depth of 6 inches and backfilled with 6 inches of ¾-inch crushed rock that is compacted into the subgrade followed by 6 inches of granular bedding material that is compacted to a firm condition. Granular bedding material placed on firm subgrade or bedding material should be shoveled under pipe haunches, as needed, and compacted to 90 percent of the ASTM D1557 reference density on a dry density basis by manual tampers or mechanical compactors.

Additional granular bedding that is moisture conditioned to approximately 2 percentage points above the optimum should be placed as pipe zone material and compacted in lifts to 90 percent of the ASTM D1557 reference density by manual tampers or mechanical compactors to 12 inches over the pipe or conduit. Trench backfill that conforms with the recommendations for general fill in Section 8.3.5 and is moisture conditioned to approximately 2 percentage points above the optimum should be placed above the pipe zone fill and compacted in lifts to 90 percent of the ASTM D1557 reference density by manual tampers or mechanical compactors. Native material excavated for the trench can be reused as trench backfill, including for a new storm drain pipe that will exit a manhole and run under a new vegetated creek bank and through the planned ELS (engineered log structure) before out letting into the creek, provided that the material is moisture conditioned to at least 3 percentage points above the optimum and processed, as needed, to remove vegetation, debris, deleterious material, and rocks or clods that cannot pass the 2-inch sieve. Trench backfill under pavement in the public right of way should consist of aggregate base that conforms with Section 26 of the California Standard Specifications (Caltrans, 2022) and is compacted to 95 percent of the ASTM D1557 reference density on a dry density basis after the material is moisture conditioned to approximately 2 percentage points above the optimum. The allowable lift thickness for pipe zone and trench backfill is influenced by the type of compaction equipment utilized but generally should not exceed 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill. Densification of pipe zone and trench backfill by flooding or jetting should not be permitted. CLSM may be used as pipe zone and trench backfill. Lift thickness for CLSM should not exceed 3 feet. Measures to mitigate flotation of the pipe or conduit should be taken, as needed, where CLSM is used as bedding or pipe zone fill.

8.3.8 Rainy Weather Considerations

Construction should be performed during the period between approximately April 15 and October 15 to avoid the rainy season. We recommend that cuts at the toe of slope adjacent to the northern edge of the road be performed in the summary to reduce the risk of instability. In the event that grading is performed during the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agency having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the project drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed.

8.4 Geotechnical Engineer of Record Services

The recommendations provided in this report are based on the findings from this investigation and preliminary assumptions regarding the proposed design. Ninyo & Moore should review the plans that are developed by the design-build team, evaluate if additional subsurface information is needed, prepare a report to present the findings from supplemental exploration, and provide recommendations to supplement or modify, as appropriate, the geotechnical recommendations in this report.

During construction, Ninyo & Moore should evaluate the exposed subsurface conditions for consistency with the conditions encountered in the discrete borings performed for this study, and to check that the work conforms with the geotechnical recommendations. Specifically, the geotechnical engineer should be retained to:

- Observe preparation and compaction of pavement subgrade.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill and aggregate base.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe drilling and construction of soldier piles, and installation of lagging.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

9 LIMITATIONS

The field evaluation and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through subsurface exploration. Subsurface evaluation will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site can change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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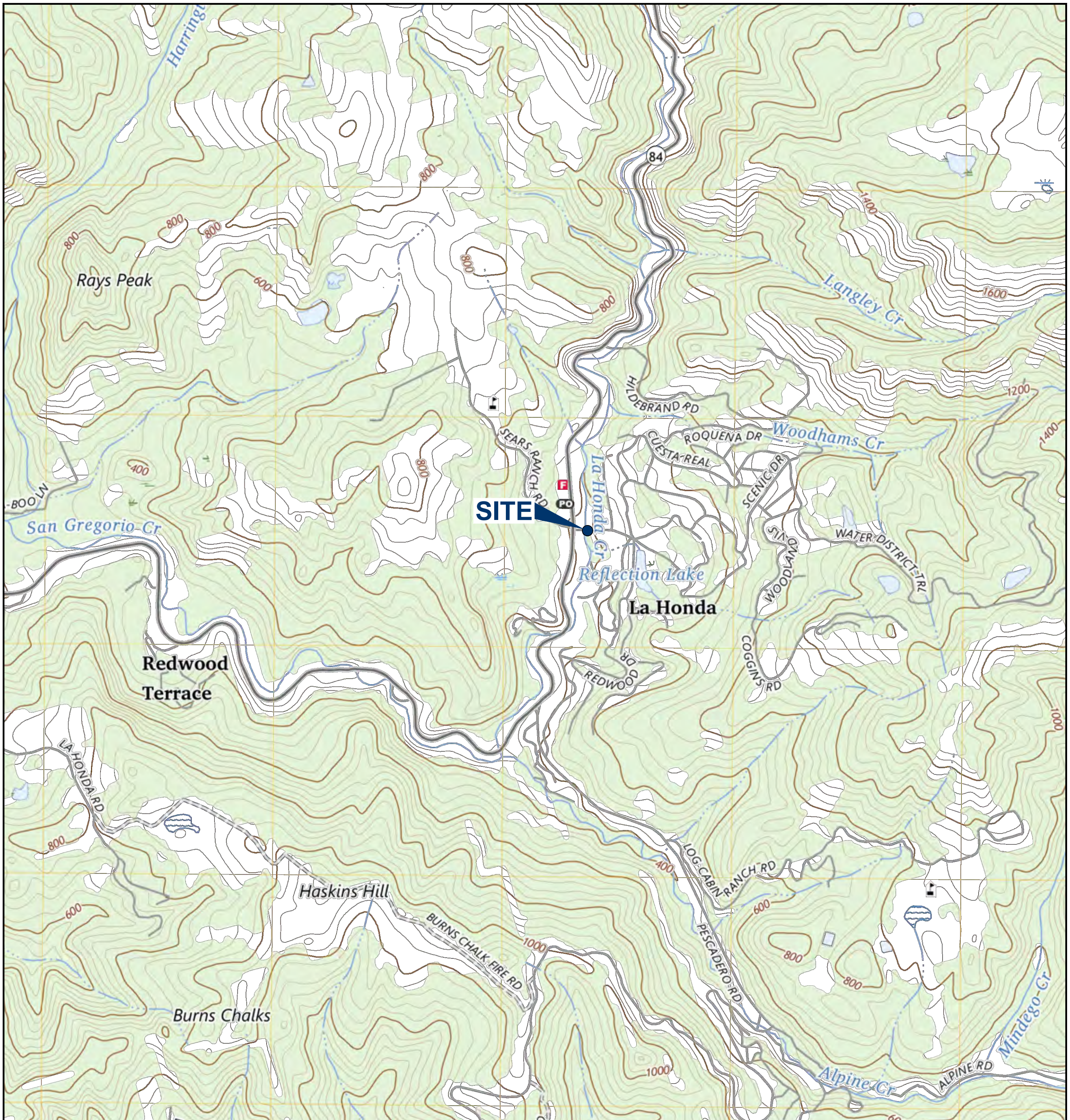
FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 2025

N

SCALE (FEET)
0 2,000 4,000

405000001.dwg 01/03/2025 AEK

FIGURE 1

SITE LOCATION

ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY
LA HONDA, CALIFORNIA

405000001 I 10/25

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: GOOGLE EARTH, 2025

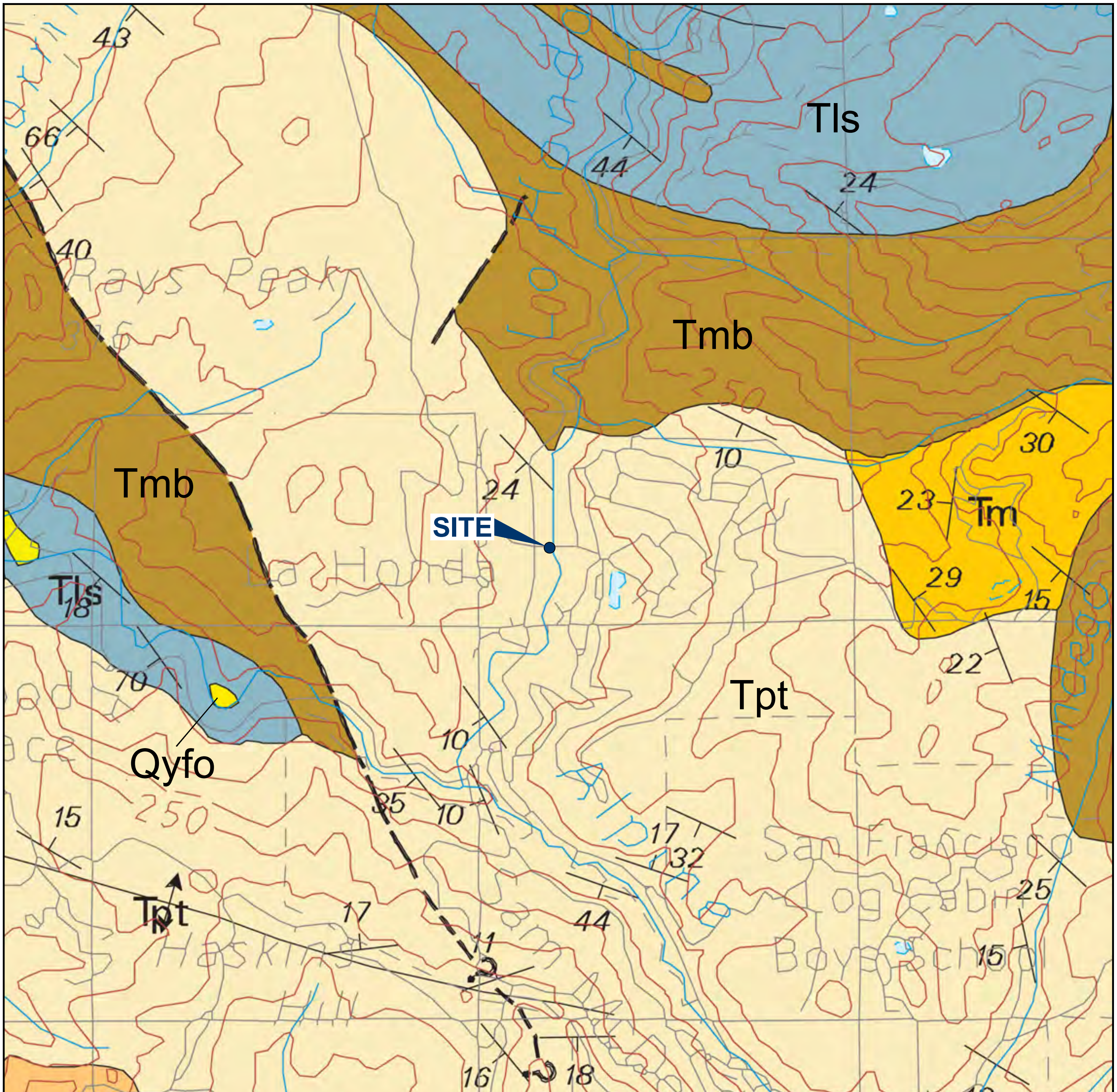
FIGURE 2

EXPLORATION LOCATIONS

ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY
LA HONDA, CALIFORNIA
405000001 I 10/25

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

405000001.dwg 01/06/2025 AEK



LEGEND

Qyfo	YOUNGER (OUTER) ALLUVIAL FAN DEPOSITS (HOLOCENE)	
Tpt	TAHANA MEMBER (PLIOCENE/MIOCENE)	
Tm	MONTEREY FORMATION (MIOCENE)	
Tls	LAMBERT SHALE & SAN LORENZO FORMATION, UNDIVIDED (MIOCENE)	
Tmb	MINDEGO BASALT & RELATED VOLCANIC ROCKS (MIOCENE)	

THRUST FAULT

FAULT

GEOLOGIC CONTACT

STRIKE AND DIP OF BEDDING

N

SCALE (FEET)

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: BRABB, ET AL, 1998

405000001.dwg 01/03/2025 AEK

FIGURE 3

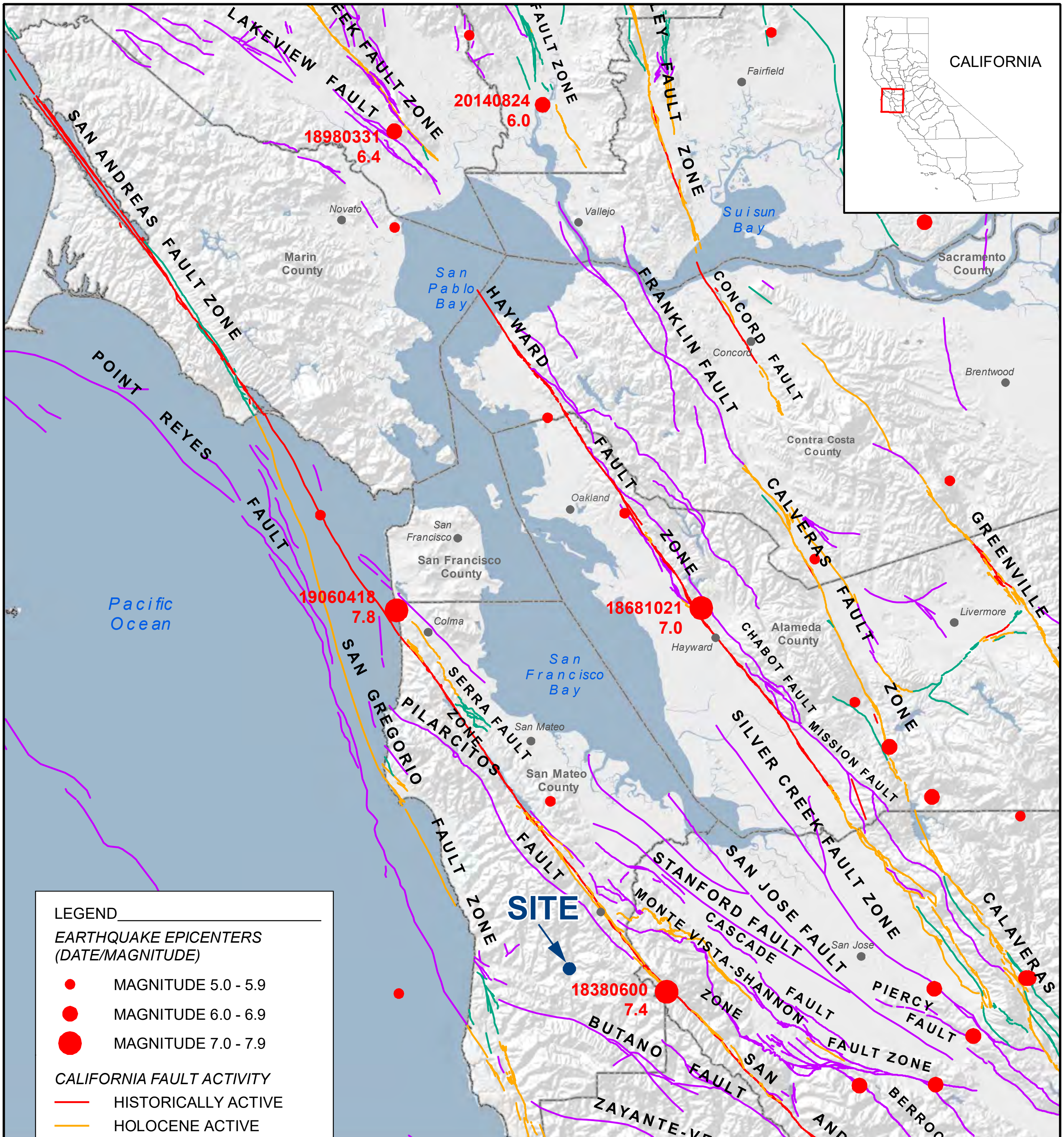
Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY
LA HONDA, CALIFORNIA

405000001 | 10/25



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, 2010, FAULT ACTIVITY MAP OF CALIFORNIA;
CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49

N

SCALE (MILES)
0 10 20

405000001_FL.pdf 01/03/2025 AEK

FIGURE 4

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY
LA HONDA, CALIFORNIA
405000001 | 10/25

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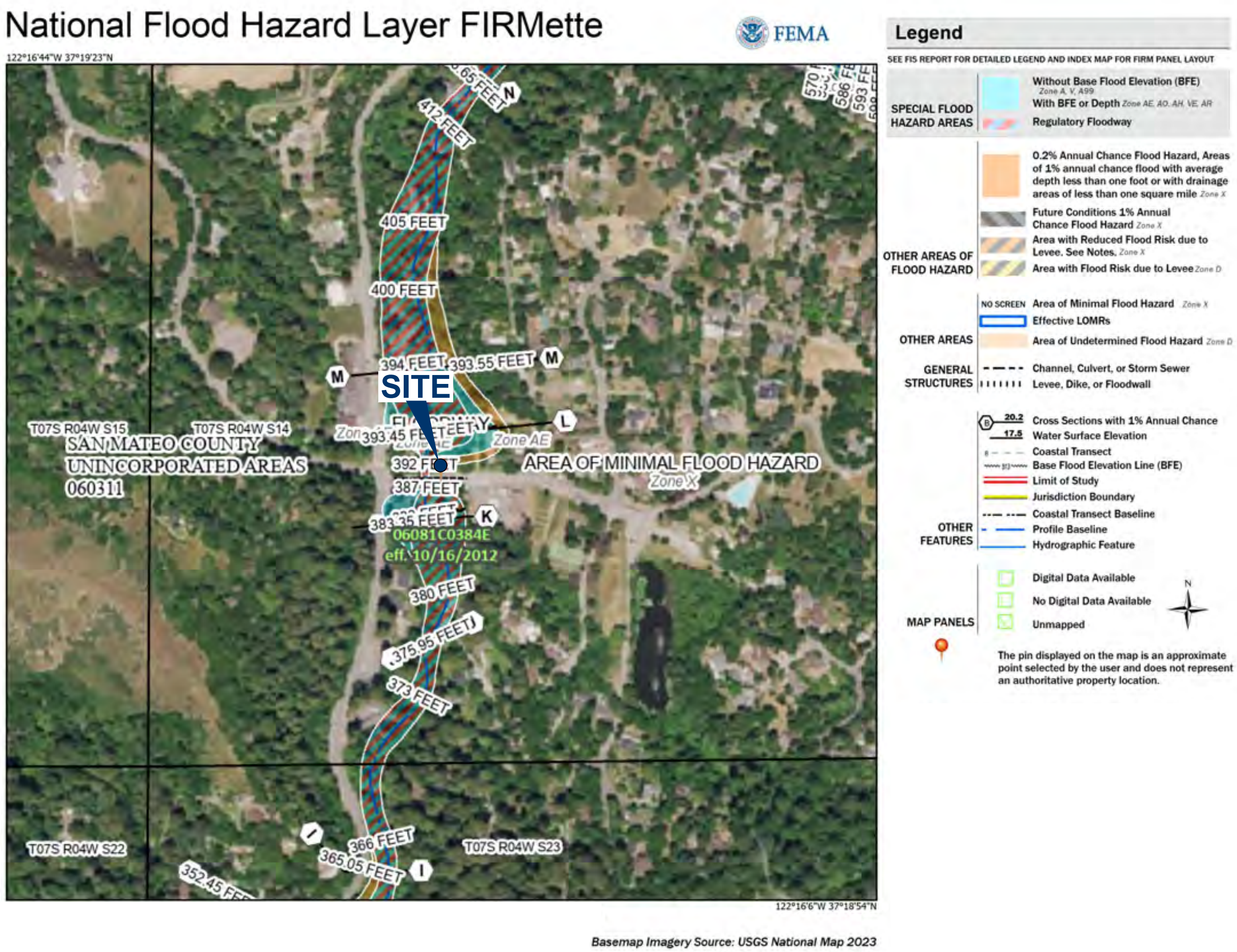
Geotechnical & Environmental Sciences Consultants



N

SCALE (FEET)

0 600 1,200

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: FEMA, 2025

FIGURE 5

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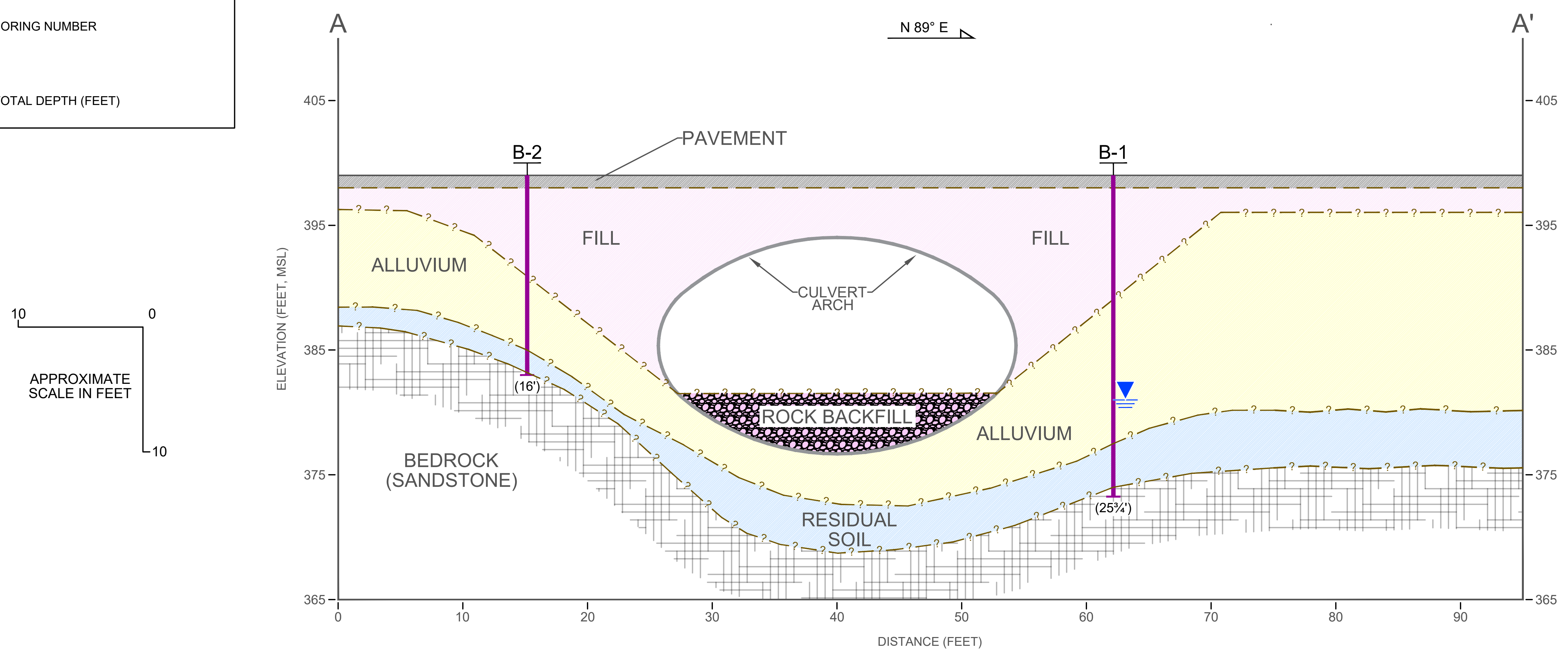
FEMA FLOOD MAP

ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY
LA HONDA, CALIFORNIA

405000001 | 10/25

LEGEND

(pink)	ARTIFICAL FILL
(yellow)	ALLUVIUM
(blue)	RESIDUAL SOIL
(hatch)	BEDROCK
— ? —	GEOLOGIC CONTACT QUERIED WHERE UNCERTAIN
MSL	MEAN SEA LEVEL
▼	GROUNDWATER (MEASURED AT TIME OF EXPLORATION)
B-1	BORING NUMBER
(25¾')	TOTAL DEPTH (FEET)



405000001-XSEC.DWG

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.
REFERENCES: LEIGHTON-YEN & ASSOCIATES, INC. (1972), GOOGLE EARTH (2025), CANTWELL JR., S.H. (1973)

FIGURE 6

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GEOLOGIC CROSS SECTION A-A'
LA HONDA CREEK SLIPOUT
ENTRADA WAY BRIDGE
LA HONDA, CALIFORNIA

405000001 | 10/25

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer (as indicated) falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following methods.

Modified Split-Barrel Drive Sampler
Relatively undisturbed soil samples were obtained in the field using a modified split-barrel drive sampler. The sampler, with an external diameter of 3.0 inches, was lined with 6-inch-long, thin brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

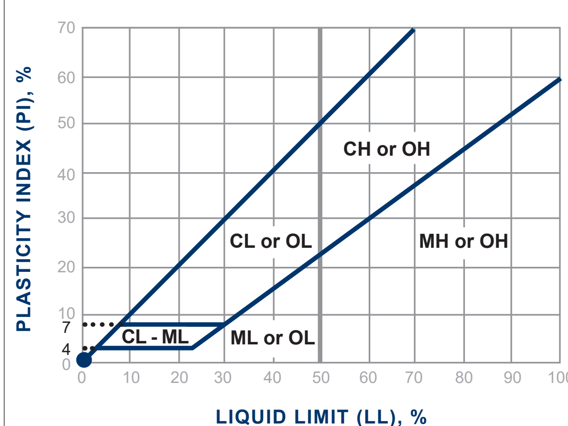
Soil Classification Chart Per ASTM D 2488

Primary Divisions		Secondary Divisions	Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

DATE DRILLED 12/13/2024 BORING NO. B-1

GROUND ELEVATION 400 ± MSL SHEET 1 OF 2

METHOD OF DRILLING 4" SSA, B-24 TRUCK MOUNTED RIG (CALGEO)

DRIVE WEIGHT 140 lbs (Cathead) DROP 30"

SAMPLED BY CEH LOGGED BY CEH REVIEWED BY RPM/ARD

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						ASPHALT CONCRETE: Approximately 3 inches thick.
					CL	AGGREGATE BASE: Approximately 9 inches thick.
						FILL: Pale olive, moist, very stiff, sandy lean CLAY; angular gravel; red iron oxide staining. (PP=4.0 tsf)
	Driven	28	36.2	83.4		
5	Driven	27	33.5	85.3		
					CL	ALLUVIUM: Dark brown to olive, moist, very stiff, sandy lean CLAY. (PP=4.0 tsf)
	Driven	17				
10	Driven	15				
					CH	Dark olive, moist, stiff, sandy fat CLAY.
15	Driven	24				
					SC	Dark olive, moist, medium dense, clayey SAND with gravel; medium grain size sand.
			▽			Wet.
20						

FIGURE A- 1

ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY, LA HONDA, CALIFORNIA

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 12/13/2024 BORING NO. B-1 GROUND ELEVATION 400 ± MSL SHEET 2 OF 2 METHOD OF DRILLING 4" SSA, B-24 TRUCK MOUNTED RIG (CALGEO) DRIVE WEIGHT 140 lbs (Cathead) DROP 30" SAMPLED BY CEH LOGGED BY CEH REVIEWED BY RPM/ARD **DESCRIPTION/INTERPRETATION**
20		■	65				SC	ALLUVIUM (Continued): Dark olive, wet, dense, clayey SAND with gravel.
		▼	25				SC	RESIDUAL SOIL: Dark yellowish-brown to greenish-gray, wet, medium dense, clayey SAND; fine grained sandstone clasts.
25		▼	50/3"	18.5			ROCK	ROCK (TAHANA MEMBER): Light gray, fresh, moderately hard, SANDSTONE; fine grained.
30								Total Depth = 25.75 feet (auger refusal). Backfilled with neat cement on 12/13/2024, shortly after completion of drilling. Notes: Groundwater was encountered at 18 feet below grade during drilling. It may rise to a higher level in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently acccurate for preparing construction bids and design documents (Google, 2024).
35								
40								

DATE DRILLED	12/13/2024	BORING NO.	B-2
GROUND ELEVATION	398 ± MSL	SHEET	1 OF 2
METHOD OF DRILLING	4" SSA, B-24 TRUCK MOUNTED RIG (CALGEO)		
DRIVE WEIGHT	140 lbs (Cathead)	DROP	30"
SAMPLED BY	CEH	LOGGED BY	CEH
REVIEWED BY	RPM/ARD		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
								AGGREGATE BASE: Approximately 9 inches thick.
							CH	FILL:
		■	27	33.4	81.9			Olive to dark brown, moist, very stiff, sandy fat CLAY; reddish-yellow iron oxide staining; trace angular gravel. (PP>4.5 tsf)
5		■	18	33.8	80.2			Wood debris.
							CH	ALLUVIUM: Dark brown, moist, very stiff, fat CLAY; decomposed sandstone clasts. (PP>4.5 tsf)
		■	22	28.6	87.0			
10		■	32	14.7	101.7			0.1 to 0.2 inches gray sandstone clasts.
							SC	RESIDUAL SOIL: Dark yellowish-brown to greenish-gray, wet, medium dense, clayey SAND; fine grained sandstone clasts.
15		▲	50/6"					
							ROCK	ROCK (TAHANA MEMBER): Gray, fresh, moderately hard, SANDSTONE; fine grained.
								Total Depth = 16 feet (auger refusal). Backfilled with neat cement on 12/13/2024, shortly after completion of drilling.
								Notes:
								Groundwater was not encountered during drilling. It may rise to a higher level in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
20								

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ENTRADA WAY CULVERT SLIP OUT
ENTRADA WAY, LA HONDA, CALIFORNIA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 12/13/2024 BORING NO. B-2 GROUND ELEVATION 398 ± MSL SHEET 2 OF 2 METHOD OF DRILLING 4" SSA, B-24 TRUCK MOUNTED RIG (CALGEO) DRIVE WEIGHT 140 lbs (Cathead) DROP 30" SAMPLED BY CEH LOGGED BY CEH REVIEWED BY RPM/ARD **DESCRIPTION/INTERPRETATION**
20								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently acccurate for preparing construction bids and design documents (Google, 2024).
25								
30								
35								
40								

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

200 Wash Analysis
An evaluation of the percentage of particles finer than the No. 200 sieve in selected samples was performed in general accordance with ASTM D 1140. The test results are presented on Figure B-1.

Gradation Analysis
Gradation analysis tests were performed on selected soil samples in accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-2 and B-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative samples to evaluate the liquid limit, plastic limit, and plasticity index in accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-4.

Direct Shear Test
Direct shear test was performed on an undisturbed (and remolded) sample in accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-5.

Unconsolidated-Undrained Triaxial Compression Test
Triaxial compression tests were performed on selected relatively undisturbed samples in accordance with ASTM D 2850. The test specimens were exposed to a confining stress without drainage for consolidation and then sheared at the as-received moisture content under undrained conditions. The test results are shown on Figure B-6.

SAMPLE LOCATION	SAMPLE DEPTH (feet)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	15.5-16.0	Clayey SAND with gravel	85	47	SC

PERFORMED IN ACCORDANCE WITH 1140

FIGURE B-1

NO. 200 SIEVE ANALYSIS TEST RESULTS

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ENTRADA WAY, LA HONDA, CALIFORNIA

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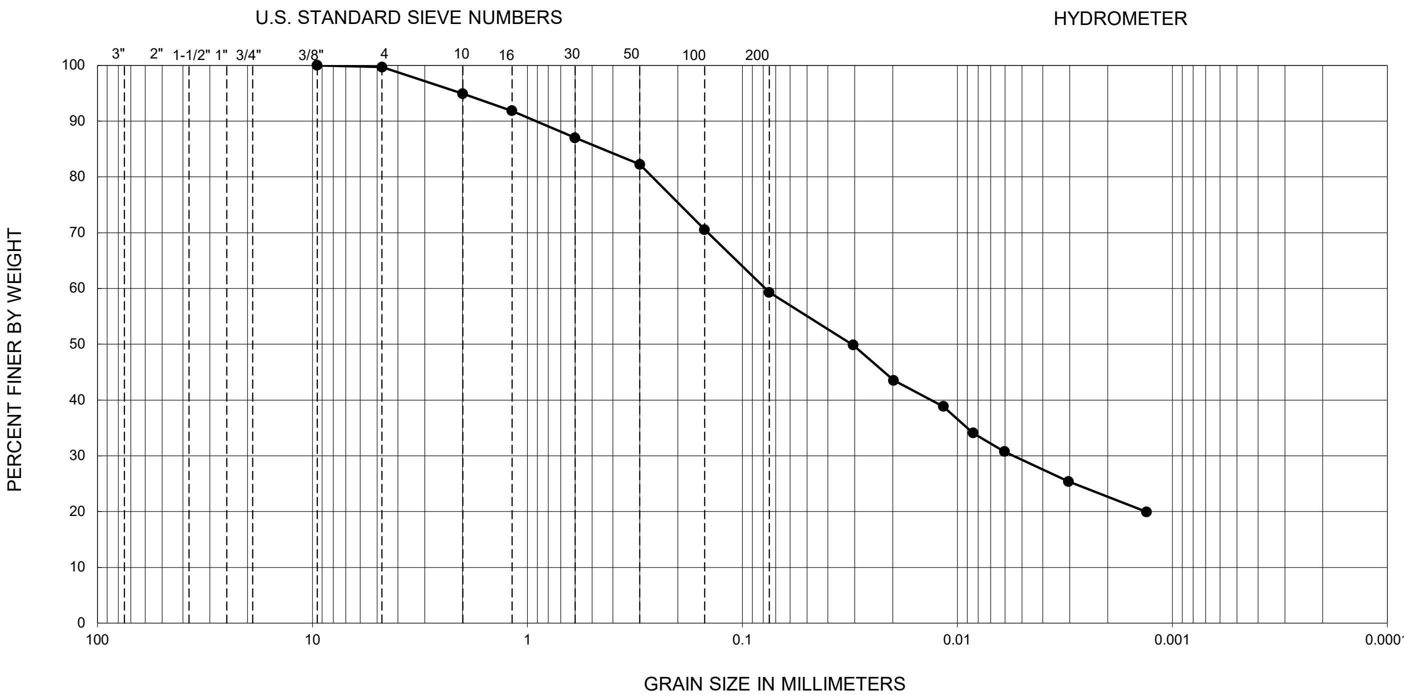
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



SYMBOL	SAMPLE LOCATION	DEPTH (feet)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	D_{10}	D_{30}	D_{60}	C_u	C_c	PASSING NO. 200 (percent)	USCS
●	B-1	9.0-9.5	--	--	--	--	0.01	0.08	--	--	50	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 0.0

Group Name: Sandy Lean CLAY

% Gravel 0

% Sand 50

% Fines 50

FIGURE B-2

GRADATION TEST RESULTS

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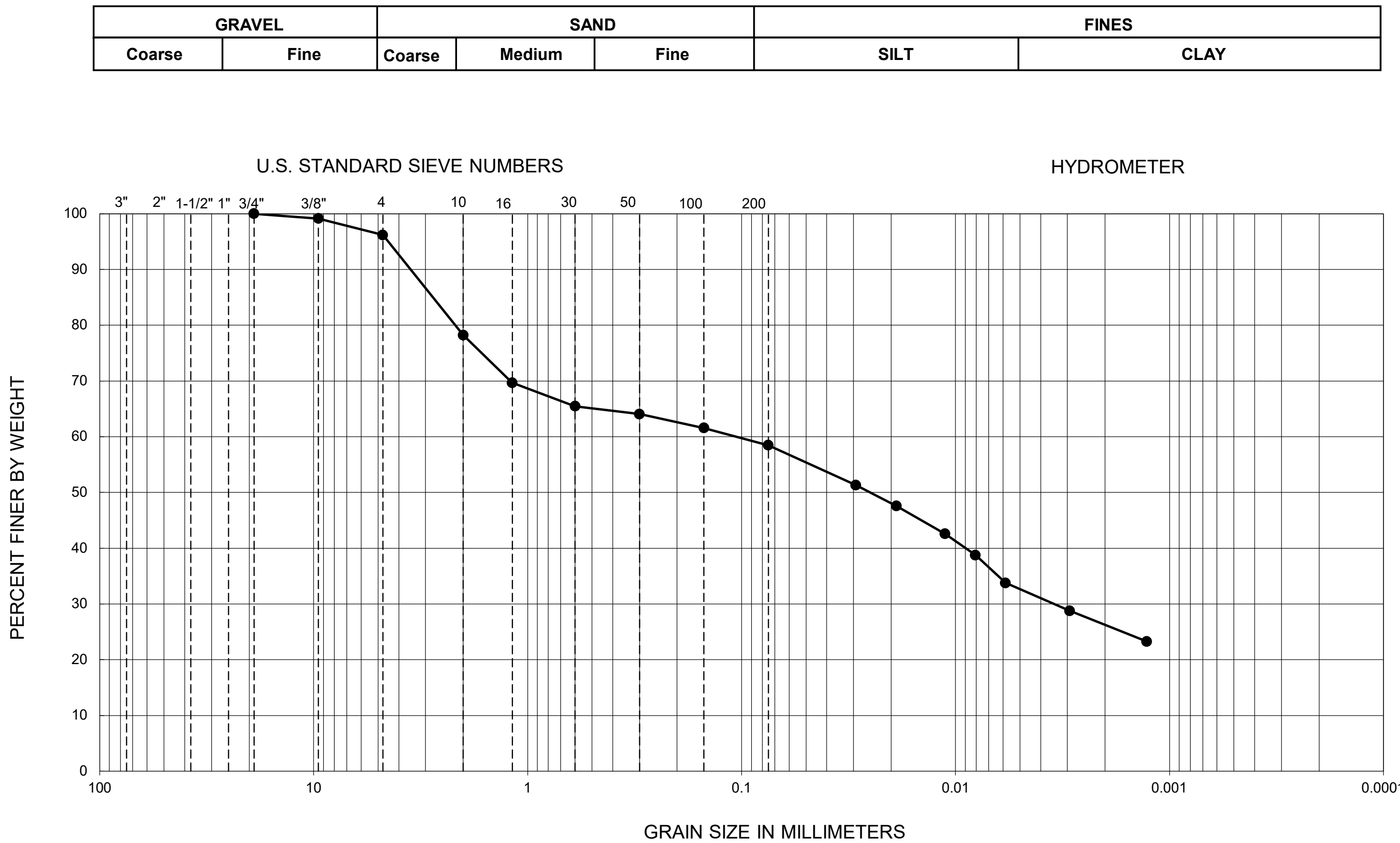

SYMBOL	SAMPLE LOCATION	DEPTH (feet)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	D_{10}	D_{30}	D_{60}	C_u	C_c	PASSING NO. 200 (percent)	USCS
●	B-2	5.5-6.0	--	--	--	--	0.00	0.11	--	--	51	CH

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 0.0

Group Name: Sandy Fat CLAY

% Gravel 4

% Sand 45

% Fines 51

FIGURE B-3

GRADATION TEST RESULTS

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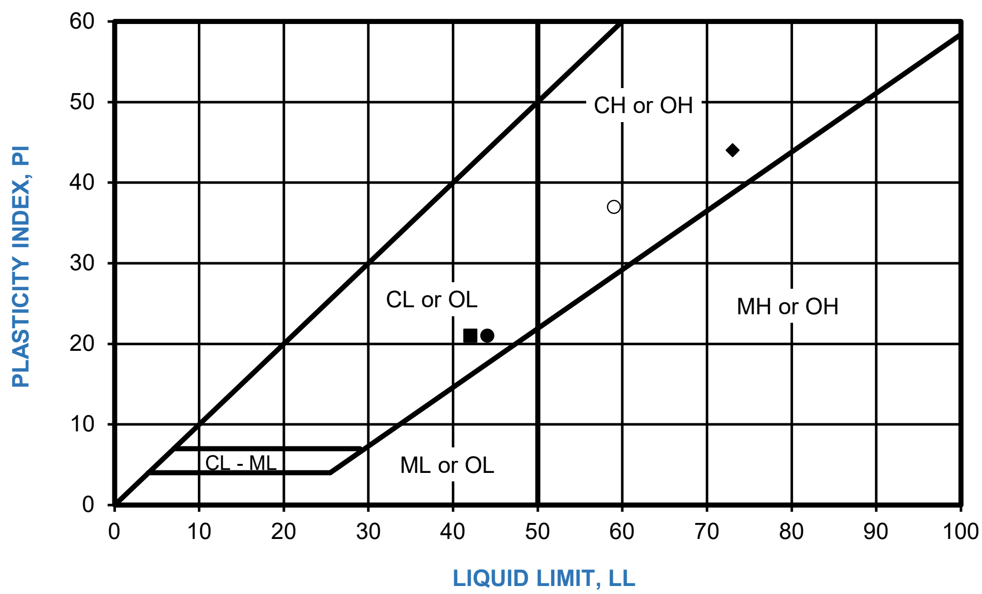
SYMBOL	LOCATION	DEPTH (feet)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	9.0-9.5	44	23	21	CL	CL
■	B-1	25.0-25.8	42	21	21	CL	SANDSTONE
◆	B-2	5.5-6.0	73	29	44	CH	CH
○	B-2	10.5-11.0	59	22	37	CH	CH



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE B-4

ATTERBERG LIMITS TEST RESULTS

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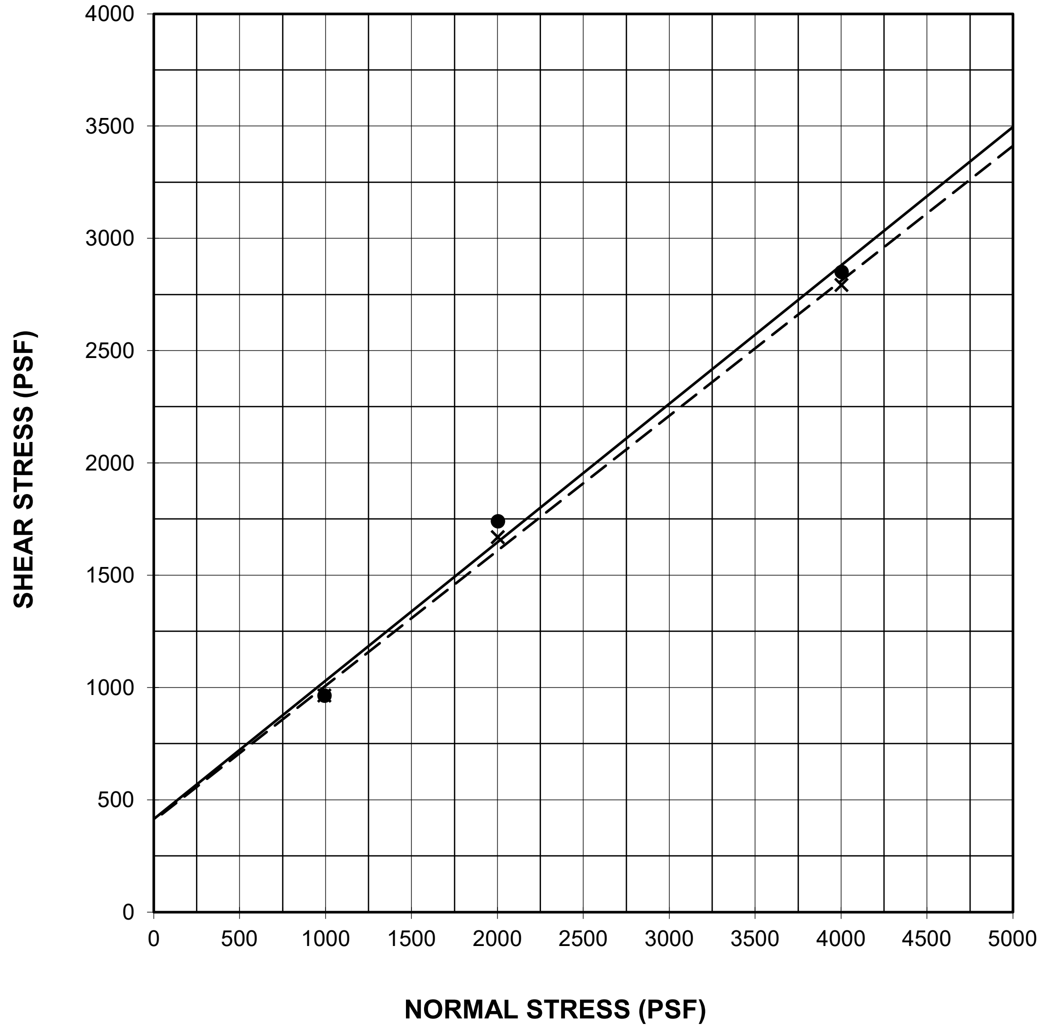

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Clayey SAND	—●—	B-1	15.5-16.0	Peak	415	32	SC
Clayey SAND	– –X– –	B-1	15.5-16.0	Ultimate	407	31	SC

PERFORMED IN ACCORDANCE WITH ASTM D 3080

FIGURE B-5

DIRECT SHEAR TEST RESULTS

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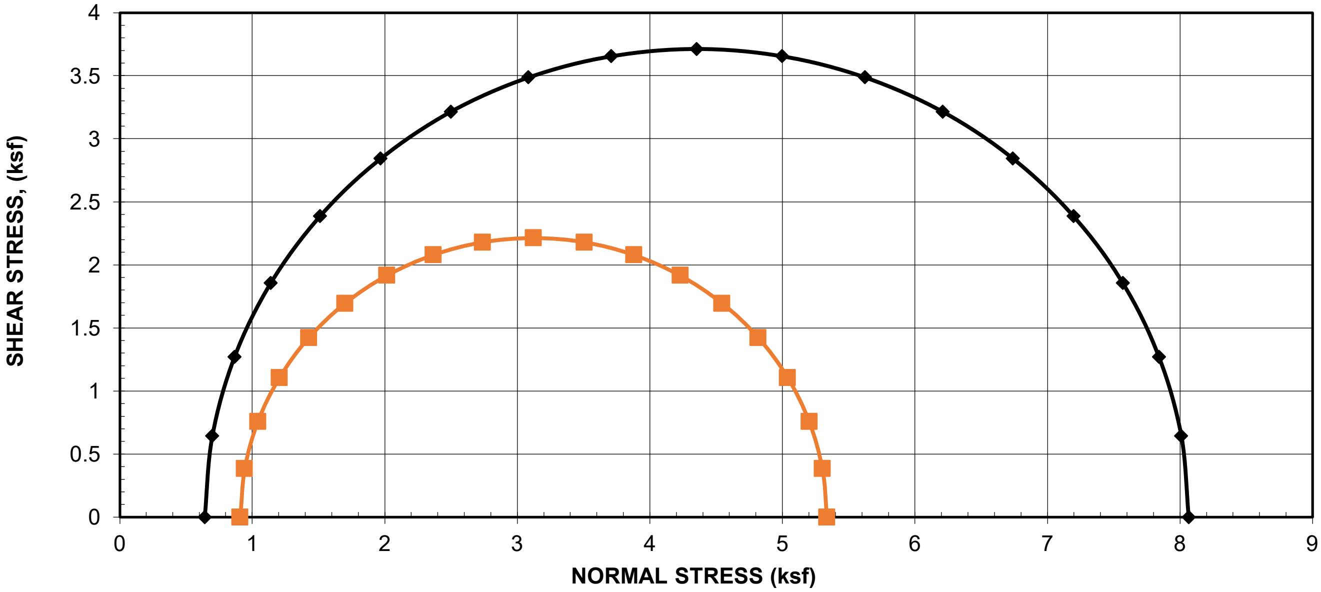

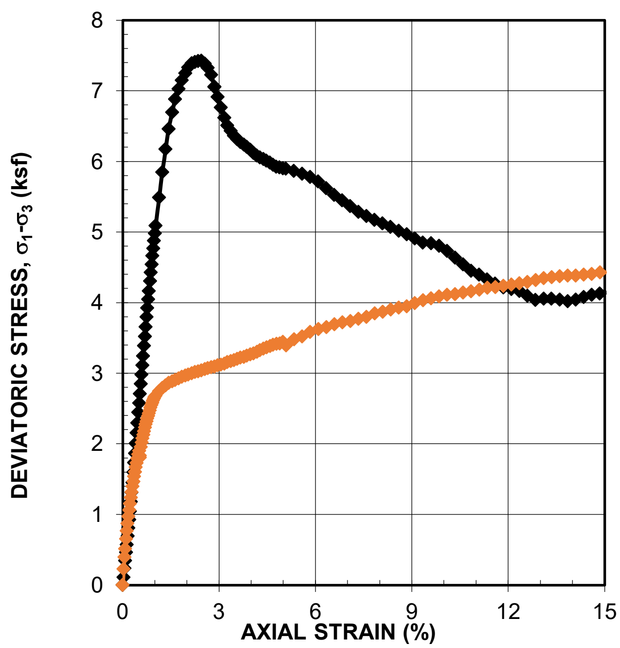

SYMBOL		◆ (black)	◆ (orange)		
SAMPLE LENGTH, (in)		5.70	5.47		
SAMPLE DIAMETER, (in)		2.41	2.38		
SPECIFIC GRAVITY, ( )		2.65	2.65		
INITIAL	MOISTURE, (%)	33.5	28.6		
	DRY DENSITY, (pcf)	85.3	87.0		
	VOID RATIO, ( )	0.939	0.901		
	SATURATION, (%)	94.6	84.1		
CELL PRESSURE, (ksf)		0.6	0.9		
BACK PRESSURE, (ksf)		0.0	0.0		
STRAIN RATE, (%/minute)		1.0	1.0		
AT FAILURE	ELAPSED TIME, tf (min)	2.4	14.8		
	AXIAL STRAIN, εf (%)	2.4	14.9		
	DEVIATOR STRESS (ksf)	7.42	4.43		
	MAJOR STRESS, σ1f (ksf)	8.07	5.34		
	MINOR STRESS, σ3f (ksf)	0.64	0.91		
MEMBRANE CORRECTION USED		FALSE	FALSE		

	DESCRIPTION (USCS SOIL TYPE)	SAMPLE LOCATION	SAMPLE DEPTH (feet)	COMPRESSIVE STRENGTH (ksf)	UU SHEAR STRENGTH s_u, (ksf)	REMARKS
◆	Lean CLAY (CL)	B-1	6.0-6.5	7.42	3.71	
◆	Fat CLAY (CH)	B-2	8.5-9.0	4.43	2.21	

PERFORMED IN ACCORDANCE WITH ASTM D 2850 ON INTACT SPECIMENS

MOISTURE CONTENT & DENSITY EVALUATED BY ASTM D 2216 & ASTM D 7263, SPECIFIC GRAVITY ASSUMED

FIGURE B-6

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION RESULTS

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