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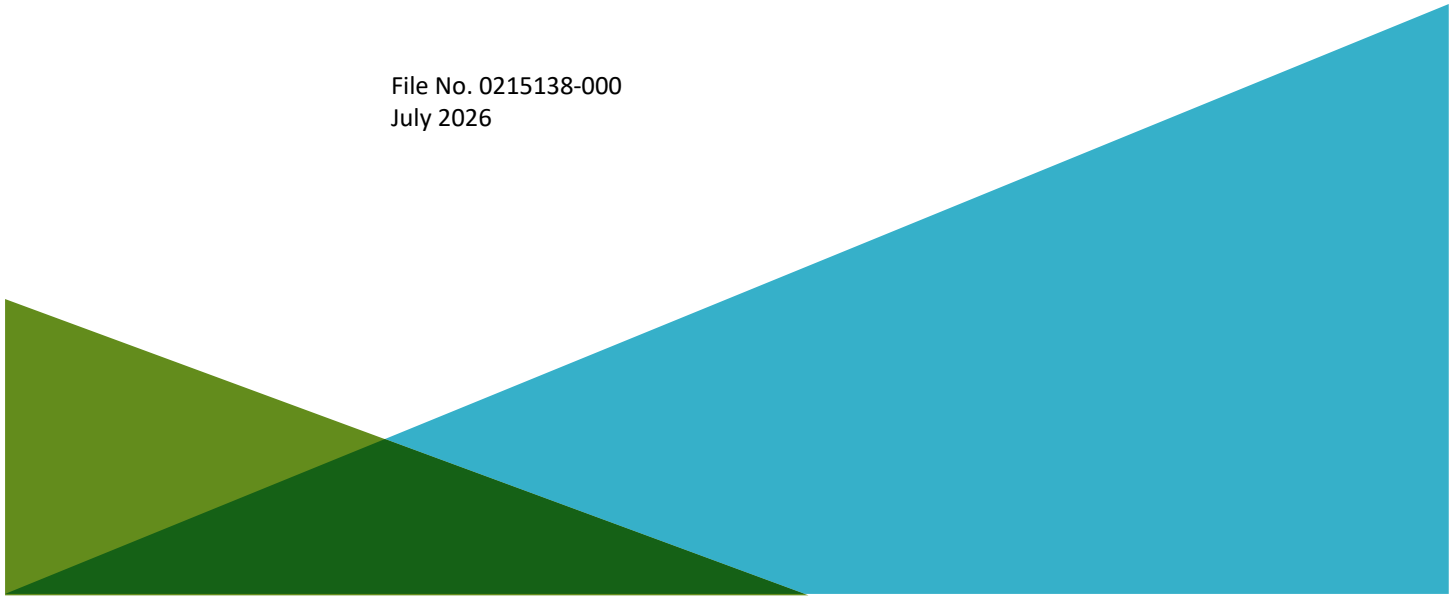
REPORT ON

PRELIMINARY GEOTECHNICAL AND GEOHAZARD EVALUATION BOLINAS HIGHLANDS ALONG ALTURA AVENUE, CRESENTE AVENUE, AND CLIFF AVENUE BOLINAS, CALIFORNIA

by
Haley & Aldrich, Inc.
Walnut Creek, California

for
Bolinas Highlands
Bolinas Highlands, California

File No. 0215138-000
July 2026



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July 27, 2026
File No. 0215138-000

Bolinas Highlands
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Bolinas, California 94965

Attention: Hugh Davis, P.E., G.E.
c/o Geoff Evans and Dana Taylor

Subject: Preliminary Geotechnical and Geohazard Evaluation
Bolinas Highlands along Altura Avenue, Cresente Avenue, and Cliff Avenue
Bolinas Highlands, California

Ladies and Gentlemen:

Enclosed is Haley & Aldrich's (Haley & Aldrich's) preliminary geohazard evaluation within the Bolinas Highlands boundary and geotechnical report for the conceptual alternatives for the repair of the slope failure along Cliff Avenue in Bolinas, California. We understand the Bolinas Highlands (BH) has requested our services under the Marin County Public Works Permanent Road Divisions (PRD). We further understand that it is desired to provide stabilization measures within the failed section of Cliff Avenue, restoring access for residents and emergency vehicles prior to the onset of the winter storm season. Additionally, two additional areas of concern along Altura Avenue where significant pavement cracking was observed were evaluated as part of the preliminary geohazard evaluation and subsurface exploration program.

Haley & Aldrich performed a LiDAR scan of the project area using a drone, prepared an updated topographic base map, and completed reconnaissance level mapping of the project area. This mapping was used to evaluate the geologic hazards within the BH boundary and develop findings of the conditions of the hillside above and below the roadway, identifying potential geohazards that may further impact the road and surrounding improvements. Haley & Aldrich also performed a geotechnical investigation to explore subsurface conditions at the site consisting of five soil borings, geotechnical laboratory testing and engineering analysis. This included three borings on Cliff Avenue and two borings on Altura Avenue.

This report contains discussions of our findings regarding the identified geohazards, general road and drainage conditions, boring logs and laboratory test results, geotechnical recommendations, and conceptual alternatives for the repair along Cliff Avenue.

Additionally, this report includes figures including a site plan showing our boring locations, site topography, and key site features. Figures also include site geology and subsurface conditions, and a

cross section showing the subsurface conditions encountered within the vicinity of the Cliff Avenue slope failure. Boring logs and test results from the subsurface exploration and lab testing have been included within the report appendices.

We appreciate the opportunity to provide our services to you with this project. If you have any questions, please contact David Burger at 925.433.5012.

Sincerely yours,
HALEY & ALDRICH, INC.

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Program Manager, Engineering Geologist

Rocio Briseno, P.G., C.E.G
Technical Specialist, Geologist

Mark Myers, P.E., G.E.
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Enclosures

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REPORT ON

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BOLINAS HIGHLANDS ALONG ALTURA AVENUE, CRESENTE AVENUE, AND CLIFF AVENUE
BOLINAS, CALIFORNIA

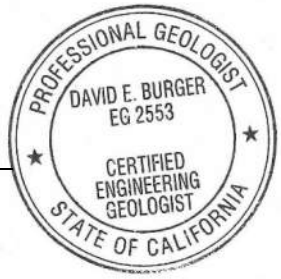
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1. Introduction

1.1 GENERAL

Haley & Aldrich, Inc. prepared this preliminary geotechnical and geohazard assessment report for Bolinas Highlands to evaluate reported slope movement, road distress, and potential geohazards in Bolinas, California. The assessment focused on portions of Altura Avenue, Cresente Avenue, and Cliff Avenue, and on preliminary geohazard conditions within the Bolinas Highlands boundary. This report also contains the evaluation of conceptual alternates and preliminary geotechnical recommendations and considerations to stabilize the slope failure impacting Cliff Avenue to restore vehicular access.

1.2 PROJECT DESCRIPTION

The project area is within the Bolinas Highlands neighborhood, a hillside residential area above the coastal community of Bolinas and inland of Bolinas Beach (Figure 1).

Bolinas Highlands requested a geotechnical evaluation of the slope movement and road distress along Altura Avenue, Cresente Avenue, and Cliff Avenue (see Figures 2A through 2E). The reported areas of concern include slope instabilities along the road at the three locations and broader potential geohazards within the Bolinas Highlands boundary.

This report presents Haley & Aldrich's preliminary findings and geotechnical design and construction recommendations for potential repair and stabilization measures. If requested, Haley & Aldrich can prepare a supplemental proposal for additional services related to final design of the preferred repair alternatives and road stabilization measures.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of Haley & Aldrich's work is to characterize geologic and geotechnical conditions, identify potential factors contributing to roadway distress and slope movement, and develop preliminary recommendations for road stabilization and repair.

The scope of work included:

- Review of available data and documents provided by Bolinas Highlands and Marin County, and review of relevant published geologic maps, soil maps, historical groundwater data, and available information in Haley & Aldrich's files.
- Development of a topographic base map using unmanned aerial system LiDAR.
- Completion of reconnaissance-level geologic mapping using the LiDAR-based topographic map.
- Completion of a subsurface exploration program consisting of five exploratory soil borings.
- Laboratory testing of selected soil samples to evaluate index and engineering properties.
- Development of conceptual alternatives to provide temporary and permanent stabilization of the slope failure impacting Cliff Avenue.
- Preparation of this Preliminary Geotechnical and Geohazard Report.

Haley & Aldrich's work has been limited to the geologic and geotechnical evaluation described in the proposal. Development of final roadway stabilization design, roadway improvements, drainage improvements, monitoring programs, plans, specifications, engineer's estimates, and construction-phase services are beyond the current scope of services, but may be provided under a supplemental

proposal if requested. Evaluation or identification of hazardous materials at the site is also beyond the current scope of work.

2. Summary of Pertinent Geological and Geotechnical Information

We were provided an AutoCAD survey file for the failure that occurred on the downslope edge of Cliff Avenue and impacted the property below the road located at 65 Brighton Avenue. The Survey data title "Cliff Road Slide" prepared by the Marin County Department of Public Works and dated 18 February 2026 provided topographic contours between 52 Cliff Avenue to Brighton Avenue. This data was reviewed and further incorporated into the development of our updated topographic base map and used during the development of our subsurface exploration program.

Additionally, we reviewed the following documents:

- Accessor parcel maps;
- Published geologic and soil data including geology maps, site hazard maps, and seismic hazard maps; and
- Historical aerial imagery.

3. Geological Conditions

3.1 REGIONAL SETTING

The project site is within the Coast Ranges geomorphic province of California. The Coast Ranges in Marin County consist of a complex assemblage of sedimentary, volcanic, metamorphic, and crystalline rocks that have been folded, faulted, sheared, and uplifted by regional tectonic forces.

The Bolinas area lies near the active transform boundary between the Pacific Plate and the North American Plate, which is expressed locally by the San Andreas Fault zone. Regional geologic conditions are therefore strongly influenced by faulting, uplift, coastal erosion, landsliding, and variable bedrock weathering. Slopes in the Bolinas area may include bedrock-controlled hillsides, colluvium deposits, landslide deposits, marine terrace deposits, and localized artificial fill associated with roadway and residential development. Because the project site is in a coastal hillside setting, geologic conditions can vary over short distances.

3.2 BEDROCK GEOLOGY

The geologic setting is shown on the Regional Geology Map, Figure 3. The distribution of geologic materials in the site vicinity has much to do with the site's location on the west side of Bolinas Lagoon, the nearby Pacific coastline and Bolinas Beach, the San Andreas Fault zone along Bolinas Lagoon, and the steep coastal hillsides that rise north of the low-lying portions of Bolinas.

The general vicinity of the project site has been mapped by several geologists (Clark and Brabb, 1977; Gluskoter, 1969; Wagner et al., 1991; Gramer et al., 2006). Maps by these geologists generally agree that the project area is underlain by the Pliocene to Pleistocene age bedrock materials of the Merced Formation (Figure 3). Gluskoter describes the Merced Formation as composed of fine grained, friable sandstones and siltstones, but also contains minor amounts of fossiliferous sandstone and pebble conglomerate.

3.3 SURFICIAL SOILS

The surficial soils in the vicinity of the project site have been mapped by the USDA National Resource Conservation Service as belonging to the Tomales-Steinbeck loam (NRCS, 2026) (Figure 4). These moderately well-drained soils are formed from residuum weathered from sandstone. Runoff is high. The plasticity index (PI) generally ranges from non-plastic to moderate plasticity (35 percent), depending on soil component and depth.

3.4 REGIONAL SEISMICITY

The project site is located in Bolinas, California, along the Marin County coast within one of California's more seismically active regions. Regional seismicity is primarily controlled by the San Andreas Fault system, a right-lateral strike-slip fault system that forms the major tectonic boundary between the Pacific Plate to the west and the North American Plate to the east. Near Bolinas, the San Andreas Fault trends northwest-southeast along the coastal margin and has strongly influenced the regional structural grain of the Coast Ranges.

The transform boundary between these two plates has resulted in a broad zone of multiple subparallel faults within the North American Plate, along which right-lateral strike-slip faulting predominates. The San Andreas Fault accommodates less than half the average relative plate motion in this broad transform boundary.

Since the project site is located in the seismically active area, it will likely experience strong ground shaking from a large (Moment Magnitude [Mw] 6.7) or greater earthquake along one or more of the nearby active faults during the design lifetime of the project. Table 3-1 shows the approximate distances between the project site and various major surface fault traces within 50 kilometers of Bolinas. The active faults in the vicinity of Bolinas are shown on the Fault Activity Map, Figure 5.

Table 3-1: Distances to Selected Major Active Fault Surface Traces

Fault Name	Distance and Direction from Site to Surface Fault Traces
San Andreas	0.6 km northeast
Rogers Creek	35.7 km northeast
West Napa	47.6 km northeast
Hayward	29.9 km northeast
Serra	30.3 km southeast

3.5 GEOHAZARD MAPPING

3.5.1 Active Faulting

According to CGS (2018), a Holocene-active fault is defined as a fault that has had surface displacement within Holocene time (the last 11,700 years), and a pre-Holocene fault is defined as a fault whose recency of past movement is older than 11,700 years. The Alquist-Priolo Earthquake Fault Zoning Act only addresses the hazard of surface fault rupture for Holocene-active faults, although pre-Holocene-active faults may also have the potential for future surface fault rupture (CGS, 2018). An Earthquake Fault Zone is a regulatory zone surrounding the surface traces of active faults in California. The Alquist-Priolo Earthquake Fault Zoning Act's main purpose is to prevent the construction of buildings used for human occupancy on the surface trace of active faults. Before a new project is permitted, cities and counties require a geologic investigation to demonstrate that proposed buildings will not be constructed on active faults.

The project site is located within an Earthquake Fault Zone for active faults as designated by the State Geologist (CDMG,1974).

According to the United States Geological Survey's (USGS) Quaternary Fault and Fold Database, the active fault is identified as the San Andreas fault zone and it has a reported slip rate of greater than 5.0mm per year (USGS, 2026).

3.5.2 Landslides

The California Division of Mines and Geology, now the California Geological Survey, prepared a geologic map of the Bolinas Peninsula study area that includes mapped landslide deposits and related slope-instability features (Figure 6) (CGS, 1977). The map identifies a landslide feature on the slope above Brighton Avenue that encompasses the hairpin turn on Cliff Avenue in the vicinity of the recent landslide impacting the roadway on Cliff Avenue. The mapping also identifies two landslide features along Altura Avenue and another along the majority of the sea cliff face along the southern, eastern, and northern sides of the Bolinas Highlands area. The landslide deposits are described as predominantly unconsolidated and unsorted soil and rock debris, or colluvium, that have moved downslope. The map explanation also indicates that the study area is underlain by the Merced Formation, which is noted to be very susceptible to landsliding on moderate to steep slopes or in open cuts.

The California Division of Mines and Geology slope-stability map for the Bolinas Peninsula classifies the Hillside zones of the project area as Zones 3 and 4, while the flatter area near the top of the Bolinas Highlands is classified as Zone 2. (CGS, 1977). Zone 3 includes areas where slope steepness approaches the stability limits of the underlying geologic materials. These areas are considered less stable than Zones 1 and 2, but generally more stable than Zone 4. Zone 4 is the least stable category and includes most landslide deposits in upslope areas, whether active or inactive, and slopes with substantial evidence of downslope creep. The map indicates that Zone 4 areas should be considered naturally unstable and subject to potential failure even without human activity. The map notes that these stability categories are interpretive and apply generally to broad areas. Local conditions may vary within each mapped zone.

3.5.3 Liquefaction

Soil liquefaction is a phenomenon in which saturated, cohesionless soils (generally sands) lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and silts. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type and relative density; 3) overburden pressure, and 4) depth to groundwater.

According to a liquefaction susceptibility study by Witter and others (2006), the project site is located in an area of very low liquefaction susceptibility.

4. Field Investigations

RECONNAISSANCE-LEVEL GEOLOGIC MAPPING

4.1 BASEMAP

Haley & Aldrich prepared an updated topographic base map using our Unmanned Aerial System (UAS), AKA drone, based LiDAR. We completed the UAS flight of the project area to obtain geo-referenced high-resolution photographs and LiDAR data points to establish topographic contours of the site on 1 May 2026. The UAS-derived LiDAR data was set to control points established at various locations within the project area. The UAS flight was designed to be repeatable if future flights are requested by Marin County. Data collected from subsequent flights can then be used to determine potential vertical and horizontal change conditions of the surface features.

The LiDAR data was tied to survey controls set and completed by a licensed surveyor provided by Marin County. This dataset could then be used during design phases of the project.

The relevant information from our data was subsequently overlaid onto the LiDAR-based topographic base map.

The high-resolution imagery and contours are shown in Figures 2A through 2E.

4.2 GEOLOGIC MAPPING AND RECONNAISSANCE

A geologic site reconnaissance was completed on June 1, 2026 to document surface features and potential geologic hazards at the project site. Geologic hazards or related features were delineated on the UAS LiDAR- derived topographic basemap. The site conditions were also photographed at the time of the field reconnaissance. The identified geomorphic features include but are not limited to, shallow and deep-seated landslide features, colluvium and artificial deposits, areas of creep and/or pavement cracking, significant and/or intense soil erosion, and relevant established site features.

In general, the slope instabilities are concentrated on the slopes above and below the roads. The sea cliff has a significant number of failures that are primarily localized in the surficial soil detaching from the underlying bedrock materials. The hillside areas are covered with heavily vegetated zones that obscure the limits of the observed features; thus the limits of features identified from the reconnaissance level geologic mapping are approximate and inferred at localized areas.

Over steepened slopes directly adject to the road were identified. These areas include near vertical to approximately 1:1 Horizontal:Vertical (H:V) inclinations. These over steepened areas were primarily identified along the upslope side of the road where the road alignment was cut into the hillside.

Site features identified include existing walls that support the downslope edge of the road at localized areas above existing areas of fill creep, slope instabilities, or where the road traverses across colluvial ravines. Furthermore, relevant site features identified on the mapping include drainage outlets, areas where the existing pavement is cracked or has been previously repaired, and areas of past slope repairs where large boulders and rip rap have been placed.

A landslide feature was identified on Cliff Avenue above 65 Brighton Avenue. The landslide headscarp encroached into the pavement area approximately 3 feet. The landslide debris, encompassing the shoulder and downslope edge of the pavement areas, traveled downslope depositing debris into the property below, impacting accessory structures and associated back yard improvements. The failure has reduced the width of the road to approximately 7 feet limiting the accessibility of vehicular traffic

beyond the failure area. The failure was approximately 30 feet wide and primarily estimated to be between 8 to 10 feet deep. This portion of the road was closed to vehicular traffic and sandbags had been placed to divert surface water flows away from the failure.

An additional failure located east of 66 Altura Avenue was identified encroaching into the pavement area. The headscarp of this failure impacted the outer edge of the pavement shoulder area and exposed a headscarp of approximately 18 inches tall. The edge of the failure was lined with sandbags to minimize surface water flows over the edge of road.

The results of our geologic mapping and reconnaissance are shown in Figures 2A through 2E. A geologic cross section of through the Cliff Avenue failure is shown on Figure 7.

4.3 SUBSURFACE EXPLORATIONS

4.3.1 Exploratory Borings

Haley & Aldrich explored the subsurface conditions at the Site on June 1 and 2, 2026 by drilling five soil borings to depths ranging from approximately 23.0 to 31.5 feet below ground surface (bgs), using a track-mounted drill rig equipped with 8-inch-diameter solid-stem augers. Three borings were drilled in the vicinity of the landslide on Cliff Avenue and two borings were drilled along Altura Avenue. The borings were drilled by ConeTec of Richmond, California, under the direction of our field geologist. The boring locations are shown in Figures 2A and 2C.

Upon completion, the borings were backfilled with neat cement grout in accordance with County of Marin Environmental Health Services permit standards.

4.3.2 Logging and Sampling

The materials encountered in the borings were logged in the field by a Haley & Aldrich geologist. The soil was visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM D2487 and D2488.

During the drilling operations, soil samples were obtained using the following sampling methods:

- California Modified (CM) Sampler; 3.0-inch outer diameter (O.D.), 2.5-inch inner diameter (I.D.) (ASTM D1586); or
- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0-inch O.D., 1.375-inch I.D. (ASTM D1586).

The CM and SPT samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound cathead hammer, dropping 30 inches. The number of blows required to drive the SPT or CM samplers through each 6-inch interval was recorded for each sample and are included on the boring logs in Appendix A. The blow counts included on the boring logs represent the field values and are uncorrected.

Soil samples retrieved from the borings were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were then taken to geotechnical laboratories for testing and storage.

4.4 GEOTECHNICAL LABORATORY TESTING

Laboratory testing was performed to obtain information concerning the qualitative and quantitative physical properties of the samples recovered during the subsurface exploration program. Tests were performed at Cooper Testing Laboratories in Palo Alto, California, in general conformance with applicable ASTM standards. The following tests were performed:

- Moisture Content and Dry Unit Weight (ASTM D2216);
- Atterberg Limits (ASTM D4318);.

Two near-surface soil samples were also submitted and tested for corrosion properties, including pH, resistivity, sulfate content, and chloride content.

The laboratory testing results are presented on the boring logs and contained in Appendix A.

5. Subsurface Soil and Groundwater Conditions

5.1 SOIL CONDITIONS

5.1.1 Cliff Avenue Borings

The subsurface conditions encountered in our borings are generally consistent with regional geologic mapping.

Borings B-01 and B-02, drilled near the landslide headscarp, encountered approximately 7.5 to 8.0 ft of artificial fill; While boring B-03, drilled upslope of the landslide, encountered approximately 3 ft of artificial fill. The fill generally consisted of loose, poorly graded sand to poorly graded sand with silt.

Denser, in-place poorly graded sand to poorly graded sand with silt was encountered below the artificial fill and interpreted as colluvium. The density of this material generally increased with depth. Sandstone was encountered at approximately 17.0 ft below ground surface in borings B-01 and B-02, and at approximately 7.0 ft below ground surface in boring B-03.

5.1.2 Altura Avenue Borings

Borings B-04 and B-05 were drilled along Altura Avenue. Boring B-04 encountered approximately 6.5 ft of artificial fill consisting of silty sand. Colluvium consisting of poorly graded sand and silty sand was encountered below the fill. Sandstone was encountered at approximately 20 ft below ground surface.

Boring B-05 encountered approximately 2.5 ft of fill consisting of silty sand. Colluvium consisting of sandy silt, silt, and silty lean clay was encountered below the fill. Bedrock consisting of pebble conglomerate and sandstone was encountered at approximately 20.5 ft below ground surface.

5.2 GROUNDWATER CONDITIONS

Groundwater was encountered within two of the borings on June 1 and 2, 2026. Boring B-01 and B-02 encountered groundwater at approximately 7 and 8 feet below grade, respectively. Given the shallow depth of the water encountered, the steep hillside conditions, and conditions encountered below, we interpreted this groundwater be perched with the fill materials above the colluvium contact zone. Note that groundwater fluctuates over time due to rainfall, irrigation, construction activities, drainage conditions, and other factors.

6. Discussion

6.1 GENERAL SUMMARY

Based on the findings of our geotechnical investigation, the hillside portion of the Bolinas Project area contains a significant number of slope instabilities. These instabilities have exhibited evidence of past and recent impacts to the road and downslope improvements. The most recent impact from the slope instabilities to the road has occurred on Cliff Avenue above 65 Brighton Avenue, and within Altura Avenue at three locations; 1) a slope failure east of 66 Altura Avenue, and 2) pavement cracking at two locations north of 28 and 30 Cresente Avenue. Based on the findings of our mapping and investigations, the Cliff Avenue failure has resulted in the loss of the outer 3 feet of road and restricting vehicular access. The failure along Altura Avenue near 66 Altura has resulted in the loss of the outer 6 to 8 inches of pavement, while the sites on Altura Avenue north of 28 and 30 Cresente Avenue are exhibiting arcuate cracking that extends into the approximate centerline of the road.

The stabilization of the Cliff Avenue failure is feasible provided the recommendations presented in this report are implemented during design and construction. The primary geotechnical considerations for the planned improvement include the following:

- Slope stability;
- Drainage;
- existing road fill materials and colluvial soils;
- corrosivity of in situ soils.

6.2 SLOPE STABILITY AND ROAD EVALUATION

The project area within the Bolinas Highlands is situated within an approximate 130 tall sandstone knob with slope inclinations ranging between approximately 2:1 H:V on the west facing slope to approximately 1:1 H:V on the east facing slope. The upper elevation of the project area is relatively flat with gradients gradually deceasing from north to south. The steepest gradients are located along the poriferae of the project area between the base of the slope and the plateau at the upper elevations. The project area has the majority of the residences situated on the flatter upslope areas and along the base of the hillside. Several residences are also situated on the steep hillside area along Cliff Avenue.

The hillsides are heavily vegetated and covered with shallow surficial soils overlying sandstone bedrock. The soils are primarily sandy with fine-grained materials observed in the ravine areas.

The conditions of the road vary across the project area. In general, the roads located in the upslope elevations within the flatter gradients are in good to fair condition. The conditions of the roads on the slopes are variable. Areas where the roads cross ravine areas or are directly adjacent to identified instabilities are in poorer condition than other areas. Evidence of past repairs and stabilization measures in these areas were identified. These include asphalt patches, earth retention structures, and drainage modifications or controls.

The outer edge of the road where the road crosses ravine areas or required fill placement to achieve the desired grades is likely founded on undocumented fill. The fill is interpreted as a “push-fill” material likely placed during original road grading. We interpret that the fill materials were likely generated during the original construction of the roadway from the excavation of the hillside upslope of the road alignment. These materials were then subsequently placed on the downslope side of the road. Based on

the low blow count values during our subsurface exploration, appropriate compaction effort or keying of the fill materials was likely not completed at the time of placement.

Drainage improvements within the roadway were limited. A subsurface drainage, or "french drain" was identified at several locations along the upslope side of the road on portions of Cliff and Altura Avenue. Additionally, a few drop inlet structures were identified at several locations. Drainage from the residences were observed to discharge directly onto the slopes or at the road elevations. Several drainage pipes were observed with "T-type" energy dissipaters where the slope intersect the road. Evidence of surface flows discharging over the downslope edge of the road was observed. These areas exhibited increased cracking, shallow failures along the downslope edge, and areas of increased erosion. Past measures including the placement of asphalt curbs and sandbag were observed to limit the surface water sheet flows from discharging over the edge of the road and slopes below.

Based on the dynamics of the site, lateral soil creep is a key consideration to account for to ensure the long-term stability of the road and shallow founded struts on the hillside areas. This process entails the slow, downslope movement of surficial soils, specifically those located with ravine areas or subject to concentrated drainage discharge or flows. This gradual, lateral movement occurs under sustained shear stresses and can be driven by a variety of factors.

Seasonal groundwater fluctuations and stormwater events may bring upon seepage and increases in pore water pressures. Higher pore-water pressures within the surficial soils reduce effective stress and shear strength, particularly in loose soils. These conditions can facilitate lateral movement, specifically loose undocumented fills that are present along the outer road alignment and underlying colluvial soils.

Additionally, over steepened slopes caused by grading activities during the development of road, and/or erosion and localized undercutting/scour can continue to steepen slopes and remove toe support. This steepening process in conjunction with the continual shear stresses from the self-weight of the soils in places the site in a continual state of potential creep.

6.3 CLIFF AVENUE FAILURE

The failure along Cliff Avenue resulted in the loss of the outer 3 feet of the pavement and adjacent shoulder area. The failure has been interpreted to be the result of the saturation and displacement of the fill supporting the downslope edge of the road. The low blow count results of the fill material obtained during our subsurface exploration suggest that the fill was placed as a "push-fill" material, as described in the section above. We judge that the fill materials likely received minimal compaction effort and were not keyed into the underlying colluvial and/or competent materials during the original placement and development of the road. Given our understanding of the site conditions, it is likely that subsurface drainage measures are also not present or inadequate.

The surface drainage controls of the road in this area appeared to be inadequate. The configuration and high gradients of the road combined with the direct stormwater discharge from the residences above onto the slope in this area resulted in concentrated flows into the area of the failure.

Evidence of past instabilities are also present in the vicinity of this failure. A past repair was completed at the base of the slope directly above the failure. Additionally, geologic mapping of the area suggested a larger deep seated landside was present. These data suggest the surficial soils were already somewhat unstable and/or had a high potential for movement.

We judge the failure can be repaired to restore the functionality and accessibility of the road. The repair should be founded in competent materials and proper subsurface drainage measures installed. Localized surface drainage improvements should also be completed. This will require controlling impacts

from upslope sheet flows, reducing the potential for direct discharge onto the slope below the repair, and providing discharge into an appropriate facility or location downslope of the repair.

For the design of any supporting foundation system in this site, specifically drilled piers, the creeping surficial soils discussed above will exert continuous lateral earth pressure against the piers. If not sufficiently embedded into competent materials and reinforced to handle the induced bending moments, the structural integrity of the foundation elements could be compromised. In addition, ongoing soil movement below the repair could remove passive earth resistance from the piers over time, thereby reducing the effective embedment depth of the foundations. The impact of creeping soils should be a design consideration for the repair.

A geologic cross section incorporating our interpretation the subsurface conditions from the geologic mapping and subsurface exploration program is shown in Figure 7.

6.4 ALTURA AVENUE - FAILURE NEAR 66 ALTURA AVENUE

The failure that occurred east of 66 Altura Avenue was limited to the shoulder and outer edge of the road pavement. This feature exhibited a scarp of approximately 18 inches tall with bedrock interpreted to be approximately two to three feet below the pavement grade. Given the gradient of the road, we have judged that this failure was likely a result of concentrated surface water flows discharging directly onto the slope below. Given the steep inclination of the slope face and shallow soils coving the bedrock, we interpret that the surficial soils detached from the underlying bedrock due to saturation and erosion. This failure will likely continue to retrogress into the pavement area if left in the current condition.

We recommend that the outer face of the road be supported by a retaining wall to provide long term support of the edge of pavement. The retaining wall should be designed to resist surcharge loads produced by emergency, municipal, and heavy commercial vehicles. Consideration should be made to constructing an earth retention system as soon as feasible.

If the retaining wall cannot be constructed before the onset of the winter storm season, consideration should be made to winterizing the area. This may be accomplished by covering the slide area and slope below with plastic sheeting to at least 20 feet below the road. Additional sandbags should be placed at the edge of the pavement to reduce the potential for water to discharge directly on the slope below. The sandbags should be extended at least 15 feet beyond either side of the failure. A flexible pipe may be placed in the low point of the sand bag alignment to capture the surface waters that are retained behind the sand bags. The pipe should be extended at least 20 feet downslope and a ‘t” type energy dissipator placed on the downslope end of the pipe.

6.5 ALTURA AVENUE - EXPLORATION AREA

Two locations along Altura Avenue have experienced significant cracking. To determine if these areas were experiencing landsliding or determine the potential for future instabilities, a subsurface exploration was performed at each location.

The reconnaissance level geologic mapping revealed that arcuate cracking was present at each location to the approximately centerline of the road. Additionally, each location was centralized within a ravine areas where slope instabilities were identified above and below the road.

The results of the subsurface exploration revealed that the upper 2.5 to 6.5 feet of soil consisted of loose to very loose fill, consisting of sandy and silty material. Below this fill, we encountered colluvium. The colluvium underlying the B-04 location was primarily medium dense silty sand, while the colluvium at the B-5 location was very soft silt materials.

The area near the B-04 location was previously addressed with the installation of retaining wall along the downslope side of the road to west of the boring and recent asphalt patching at the boring location. An asphalt curb was also placed along the downslope side of the road to limit the amount of surface water flow discharging directly onto the slope below. The cracking at the B-04 location was observed and interpreted to suggest recent and ongoing movement. A curb was not observed at the B-05 location. Cracking was present and appeared to have been recently resealed.

Based on the mapping and results of our subsurface investigation, we judge that the fill and colluvial soils are being subjected to long term downslope soil creep. The surficial soils are likely creeping along the bedrock contact with rates of creep dependent on saturation of the surficial soils. It is recommended to reduce the potential for surface water being introduced into the surficial soils and being discharged onto the slope directly below areas of distress. Installation of an asphalt or concrete lined swale along the upslope side of the road should be considered. Where a swale cannot be introduced due to limitations from the width of the road, surface waters originating from the eastern side of the ravine should be directed into a drop inlet, allowed to flow through a flexible pipeline and discharged to an appropriate location downslope and west of the ravines. If this is not achievable, consideration should be made to installing a retaining wall along the downslope edge of the road, installing additional subsurface drainage measures including horizontal drains, aka hydraugers, or removal of the upper 5 feet of soil and replacement with a geogrid reinforced light weight engineered fill and subsurface drainage.

Based on our findings and the data available, drainage and/or road stabilization measures should be considered a high priority and completed as soon as feasible to limit the risk of a slope failure impacting performance and accessibility of the road.

6.6 CORROSION POTENTIAL

Corrosion testing was performed on two soil samples collected from the site. One of the samples was taken within the surficial soils and the other taken within the bedrock zone. A testing suite in general accordance with State of California Department of Transportation (Caltrans) methods were performed with results presented below.

The laboratory results for corrosion testing are summarized in Table 2 and provided in Appendix A.

Table 6-1. Corrosion Testing Results

Boring No.	Depth (feet bgs)	Minimum Resistivity (ohm-cm) (CT 643)[a]	Chloride (mg/kg [ppm]) (CT 417)[a]	Sulfate (mg/kg [ppm]) (CT 417)[a]	pH (CT 643)[a]
B-01	20.5	786	15	225	7.1
B-02	11.5	6323	9	19	6.8

Notes:
ft bgs = feet below ground surface
mg/kg = milligrams per kilogram
ohm-cm = ohm-centimeter
ppm = parts per million
[a] *Testing conducted in accordance with Caltrans methods.*

Caltrans Corrosion Guidelines (May 2021) identifies a site to be corrosive for structural elements if one or more of the following conditions exist:

- Chloride concentration is 500 parts per million (ppm) or greater.

- Sulfate concentration is 1500 ppm or greater.
- pH is 5.5 or less.

A minimum resistivity value for soil and/or water less than 1,500 ohm-centimeters (ohm-cm) indicates the presence of high quantities of soluble salts and a higher propensity for corrosion but is not included as a parameter to define a corrosive area per Caltrans criteria. The minimum resistivity of the tested bedrock from Boring B-01 was below the 1,500 ohm-cm threshold and therefore indicates a potential for corrosion, while the tested sample within the surficial soils from Boring B-02 fell above this threshold value and does meet the criteria for corrosion. Based on the results of the testing, the samples tested had values for Chloride, Sulfate, pH that did not meet the Caltrans criteria for a corrosive site.

According to American Concrete Institute (ACI) 318 Section 4.3, Table 4.3.1 (ACI, 2019):

- Sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible (no restrictions on concrete type).
- Water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to concrete.

Based on these results, the samples tested had values for sulfate and chloride that did not meet ACI criteria and are considered non-corrosive to concrete.

These corrosion testing results should be considered preliminary and are an indicator of potential corrosivity of the onsite soils. Other soils found on site may be more, less, or of similar corrosive nature. Our scope of services did not include corrosion mitigation. Thus, consideration should be made to retain services with a qualified corrosion engineering consultant to provide appropriate mitigation measures for any structural elements that may be affected by the *in situ* soils.

It should also be noted that ACI does Chapter 19 does have requirements that increase concrete cover based on exposure categories. The proximity of the site to the ocean is a consideration for concrete cover for rebar and should be considered for exposed metal.

7. Cliff Avenue Conceptual Alternative Stabilization Measures

7.1 GENERAL

We understand that it is desired to stabilize the failure and restore accessibility beyond the failed section of roadway along Cliff Avenue. Per our discussions with the County, temporary stabilization measures may need to be constructed to allow for access beyond the slide and restore the functionality of the roadway before the onset of the 2026 wet weather season. Therefore, we have developed a stabilization alternatives that can be implemented in 2026 to reopen the roadway before the winter. It is possible that some could be sequenced to allow temporary reopening and then completed in 2027.

7.2 CONCETUAL DESIGN ALTERNATIVES

Three conceptual design alternatives for repairing the sites are:

A) Cantilever Retaining Wall with Light Weight Backfill
B) Tieback Retaining Wall
C) Mechanically Stabilized Earth Retaining Wall supported on a concrete pier and grade beam foundation.

Note each of these alternatives considered vehicle surcharge loads during the development of the conceptual design elements.

We also considered an embankment repair. The depth to competent material and the site geometry, make the repair extremely difficult and one which should not be attempted late in the year. As previously noted, we selected three alternatives which could be completed before the winter or at a minimum could be sequenced to provide benefit to reopen the roadway.

7.3 ALTERNATIVE A – SOLDIER PILE RETAINING WALL WITH LIGHTWEIGHT BACKFILL

The soldier pile alternative for a cantilever design will require lightweight backfill due to the steep terrain and depth of undocumented fill. The wall shall be designed to retain the height between the bedrock contact and the street elevation. The final design may retain some soil and some lightweight backfill if the excavation is limited to the depth of the undocumented fill.

For Alternative A, there are several wall facing options which include pressure treated wood lagging, concrete lagging, concrete panels, poured in place panels, and shotcrete.

7.3.1 Design and Assumptions – Soldier Pile Retaining Wall

Geotechnical design criteria and assumptions recommended for analysis of this soldier pile wall alternative:

- Active earth pressure varies based on the lightweight material but is likely 10 to 30 pcf/ft;
- Active earth pressure for onsite soils is 45 pcf/ft;
- Seismic earth pressure varies based on the material used but will likely be on the order of 75% to 100% of the static earth pressure;
- Passive earth pressure of 250 pcf/ft over two pile diameters, neglecting depth until a horizontal distance exists from the pile to the slope face of 10 feet;
- The piles should be designed to retain the entire depth from bedrock to the ground surface;

- Minimum pile diameter of 18 inches;
- Maximum pile spacing of 8 feet center to center;
- Typical pile spacing 2 to 3 pile diameters;
- Minimum pile depth of 8 feet into bedrock, however, the final geometry will determine the pile length;
- Project geotechnical engineer should observe excavations of the piles to verify that the embedment requirements for the piles have been met.
- The method of calculations shall be similar to those documented in the Caltrans Trenching and Shoring Manual, latest edition.

The following structural criteria will likely be used to design the retaining wall.

- Compressive strength of concrete of 2500 psi (ACI 214);
- Steel Beam Strength, 50 ksi60 (ASTM A706).

For this alternative, pile would be excavated, steel I-beams and concrete placed. Typically, an excavatable low strength concrete would be placed above the bedrock contact to allow chipping to expose the beam for the placement of lagging and backfill materials. The sequence of construction will depend on the design.

7.4 ALTERNATIVE B – SOLDIER PILE RETAINING WALL WITH TIEBACKS

Adding tiebacks to soldier pile walls dramatically reduces the bending moment over a cantilever soldier pile alternative allowing steel beams. The wall shall be designed to retain the height between the bedrock contact and the street elevation.

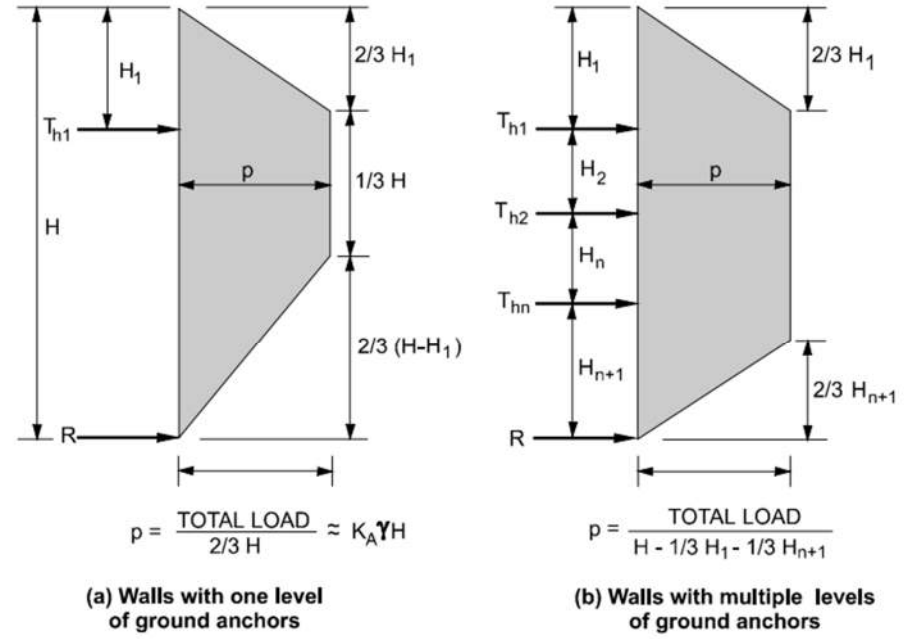
For Alternative B, there are several wall facing options which include pressure treated wood lagging, concrete lagging, concrete panels, poured in place panels, and shotcrete.

7.4.1 Design and Assumptions – Soldier Pile Retaining Wall

Geotechnical design criteria and assumptions recommended for analysis of this soldier pile wall alternative the following:

- Active earth pressure for onsite soils is 45 pcf/ft;
- Seismic earth pressure for onsite material of 32 pcf/ft;
- Passive earth pressure of 250 pcf/ft over two pile diameters, neglecting depth until a horizontal distance exists from the pile to the slope face of 10 feet;
- The piles should be designed to retain the entire depth from bedrock to the ground surface;
- Minimum pile diameter of 18 inches;
- Maximum pile spacing of 8 feet center to center;
- Typical pile spacing 2 to 3 pile diameters;
- Minimum pile depth of 8 feet into bedrock, however, the final geometry will determine the pile length;
- Project geotechnical engineer should observe excavations of the piles to verify that the embedment requirements for the piles have been met.

- The design shall be consistent with FHWA Geotechnical Circular No. 4, Ground Anchors and Anchored Systems (1999). The design will need to evaluate both a cantilever height above the elevation of the tieback and a tieback design to see what governs the varies design elements;
- The tieback design shall use the apparent earth pressure distribution shown on pg. 51, Figure 24 of the FHWA document where Kaɣ is the active efp recommended herein.



H_1 = Distance from ground surface to uppermost ground anchor

H_{n+1} = Distance from base of excavation to lowermost ground anchor

T_{hi} = Horizontal load in ground anchor i

R = Reaction force to be resisted by subgrade (i.e., below base of excavation)

p = Maximum ordinate of diagram

TOTAL LOAD = $0.65\ K_A \gamma H^2$

Figure 24. Recommended apparent earth pressure diagram for sands.

- For the Seismic case, the seismic and static earth pressures can be added;
- The tieback design shall assume double corrosion protection for a permanent installation;
- Strand anchors are preferred over bar anchors.

The following structural criteria will likely be used to design the retaining wall.

- Compressive strength of concrete of 2500 psi (ACI 214);
- Steel Beam Strength, 50 ksi60 (ASTM A706).

- Tiebacks are typically 150 ksi bar or 270 ksi strands

For this alternative, piles would be excavated, steel I-beams and concrete placed. Typically, an excavatable low strength concrete would be placed above the bedrock contact to allow chipping to expose the beam for the placement of lagging and backfill materials. The sequence of construction will depend on the design, but typically, the wall is excavated and lagged to the tieback elevation where the tieback is then installed. Lagging can then extend below the tieback elevation.

7.5 ALTERNATIVE C – PILE SUPPORTED SEGMENTAL BLOCK RETAINING WALL

This design alternative includes the construction of a new mechanically stabilized earth wall (MSEW) that uses segmental block wall (SRW) units and geogrid reinforcement supported on a pier supported grade beam along the downslope edge of the road alignment. The design will require drilling cast-in-place piers along the proposed alignment and connecting the pier with a grade beam. Segmental block units and geogrid will then be constructed above the grade beam to an elevation slightly above the existing road grade. The existing fill materials and colluvial soils behind the wall will be excavated to the top of the grade beam elevations. The excavations across the road will extend to 70 to 100 percent of the wall height to allow for geogrid to be placed at designated block course elevations. The excavation will likely extend across the road and result in the removal and replacement of undocumented fill with engineered fill thereby improving the subgrade while restoring the roadway. The pavement section will be placed over the wall backfill to restore pavement grades. Limited drainage improvements will be constructed upslope and downslope of the repair to capture or redirect flows. Drainage improvements will include the construction of asphalt and/or concrete curbs, swales, inlets and buried stormwater pipes. Temporary construction in 2026 would not be required since this alternative is intended to be a permanent repair. A Caltrans metal beam guard rail could be incorporated into the repair. If it was desired for the MBGR to look weathered, corten steel could be specified.

7.5.1 Design and Assumptions – Segmental Block Retaining Wall

- The SRW wall will only require minimum embedment since it will be founded on a concrete grade beam. The design assumes a minimum embedment into the existing grade along the downslope edge of the road alignment of 8 inches to avoid exposing the pile and grade beam foundations;
- The grade beam shall step as needed in 8, 16, or 24 inch increments to match the height of the segmental block units;
- The maximum segmental block wall height will depend upon the design layout. It is likely to be in range of 8 to 14 feet tall;
- Seismic analysis should be performed for internal stability using the highest PGA for the Mononabe okabe method;
- Global stability should be performed for static and seismic based on the PGA from Section 8.3 to check compound and global failure modes which are failures through the reinforced zone of the retaining wall and outside the reinforced zone;
- Design calculations assume a level backfill and vehicle surcharge;
- Segmental block units have a depth of 18-24 inches deep and a face which is 18 inches wide and 8 inches tall are recommended;
- The retaining wall will include a permanent drainage system consisting of Caltrans Class 2 permeable material. The drainage zone should be 12 inches wide and extend to a depth of 1 foot below the

finished grade at the top of the wall. The upper 12 inches will be capped at the surface with compacted baserock and asphalt pavement. The design assumes no buildup of hydrostatic pressure behind the retaining wall.

- The rock used for filling blocks shall be an angular crushed rock (to be similar to the material used when the connection capacity testing is performed for the geogrid connection capacity to the block);
- A concrete curb will be placed at the top of the wall to prevent any surface/sheet flows from flowing over the face the wall onto the property below. Water will be directed into an inlet drainage ditch that will be directed to an appropriate discharge location.
- The foundations and retaining zone of the wall will be observed by the project geotechnical engineer, prior to wall construction;
- All fill materials shall be compacted to a relative compaction of no less than 90% in accordance with ASTM D-1557.
- A subdrain at the back of the reinforced zone is also recommended.

7.5.2 Design and Assumptions – Pier and Grade Beam Foundation

Geotechnical design criteria and assumptions recommended and used for analysis of the pier supported grade beam foundation include the following:

- Active earth pressure of 45 pcf/ft;
- Seismic earth pressure of 32 pcf/ft;
- Passive earth pressure of 250 pcf/ft over two pile diameters, neglecting depth until a horizontal distance exists from the pile to the slope face of 10 feet;
- The piles should be designed to retain the tributary area of earth pressure located below the MSEW portion of the wall;
- Minimum pile diameter of 18 inches;
- Maximum pile spacing of 8 feet center to center;
- Minimum pile depth of 8 feet into bedrock, however, the final geometry will determine the pile length;
- Minimum grade beam width shall be the greater of 6 inches more than the pile diameter and at least the width of the segmental block unit;
- Project geotechnical engineer should observe excavations of the piles to verify that the embedment requirements for the piles have been met.

The following structural criteria will likely be used to design the retaining wall.

- Compressive strength of concrete of 2500 psi (ACI 214);
- All reinforcing steel will be grade 60 (ASTM A706).

7.6 ASSUMPTIONS

The alternatives presented have assumed that access to the site will be primally along Cliff Avenue and may require permission into the property located at 65 Brighton Avenue. We have not included a cost estimate for the contractor(s) to protect the improvements within the downslope parcel or maintain site controls. This cost will vary based on the means and methods for access presented by the contactor(s). It should be assumed that an estimated $10,000 to $20,000 will be required for access, maintaining site controls, and repair of damaged and or protection of improvements.

Additionally, we have assumed that all work will be completed from the road. No heavy equipment is anticipated within the 65 Brighton Avenue property, and protection measures will need to be installed below the work areas. We have assumed limited disturbance in 65 Brighton Avenue property, primarily involving laborers and hand-operated equipment.

7.7 COMPARISON OF ALTERNATIVES

All three alternatives can be used effectively at the site. Here are some key differences:

- The steel beams for the soldier pile alternatives can be long, and we would need to check on availability of beams for the project. Generally, beams can be found that can be delivered to the site within about 2 weeks;
- For tiebacks, there is sequencing that requires testing. This sequence adds cure time for concrete and grout. The installation and sequencing of testing and curing likely adds about 3 to 4 weeks to the installation;
- Lightweight fill limits would need to be evaluated since landslides are noted up slope. If there was a concern of reducing support to upslope failures, permanent stabilization piles would need to be added upslope.
- Lightweight backfill will likely result in needing to off haul on site material that is replaced by lightweight backfill.
- Some lightweight backfill alternatives may have longer lead time than others. During the design, the availability should be a consideration in selection of lightweight backfill.
- For the soldier pile alternatives, the most commonly used facing is pressure treated wood lagging. A concrete facing would increase the cost.
- For the MSEW alternative, the final depth of the wall could require temporary stabilization piles on the upslope side of the road. The temporary stabilization, if needed, can be removed or left in place.
- For the MSEW design, the grade beam can designed for 2500 psi concrete allowing a higher strength mix to achieve the strength in less than 7 days;
- The MSEW alternative uses readily available blocks, concrete, and rebar.
- The MSEW alternative does not include steel exposed to salt air from the Pacific Ocean.

7.8 RECOMMENDED ALTERNATIVE

Overall, the lightweight fill cantilever design is the easiest, but has significant drawbacks related to off hauling material and bring in lightweight fill. It also likely requires a second row of piles to support the hillside above the roadway. All of these dramatically increase the cost of the alternative.

Tieback walls are also common these days, but generally constructed by specialized firms. If a shotcrete facing is used for the tieback wall, corrosion resistance would be dramatically improved. This will increase the cost of the wall.

It is our opinion that the MSEW wall supported on a pier and grade beam foundation is an excellent alternative that does not require a specialty contractor. It is also very resistant to corrosion. The wall facing is noncorrosive and the pier and grade beams are both buried and have a minimum of 3 inches clear distance between steel and the face of concrete. Additionally, since we recommend the loose fill be placed as engineered fill, the only additional construction effort is placing block and geogrid during this effort.

For these reasons, it is our opinion that Alternative C is the best and cheapest alternative to construct.

8. Design and Construction Considerations and Recommendations

Considerations and recommendations for the geotechnical aspects of the proposed roadways stabilization alternatives are presented in the subsequent sections of this report. Our recommendations are based upon the findings of this investigation. These recommendations may need to be modified if any changes with respect to the proposed work occur.

8.1 PIERS / PILE FOUNDATIONS

The construction process for all 3 alternatives entails drilling an open shaft to the design depths. Once the shaft has been completed, a prefabricated steel reinforcement cage or steel I-beam is installed, and the concrete is subsequently placed.

The piers should penetrate through the zones to be retained (fill and colluvium) and be embedded into the underlying competent bedrock material (anchor zone). The embedment within this anchor zone should be long enough to develop sufficient lateral resistance to counteract the retaining wall loading. The depth and diameter of the piles will be depend on the alternative selected.

8.1.1 Construction Considerations

8.1.1.1 Installation Feasibility

The subsurface explorations were able to extend to over 25 feet below grade using standard drilling equipment without encountering refusal and penetrating through the existing fill, colluvium and bedrock materials. Large cobbles or extensive root systems were not encountered. Based on these field observations, conventional drilling equipment should be able to advance through the in situ soil and bedrock materials down to the recommended design depths for the proposed cast-in-place piers.

8.1.1.2 Excavation Protection

The loose and soft fill and colluvial materials make the site highly susceptible to caving during drilling of pier holes or open excavations steeper than 1:1 H:V (Horizontal:Vertical). To limit the potential for these issues during construction of the cast-in-place piers, measures such as the use of temporary steel casing to stabilize the shaft and facilitate the installation of the pier may need to be implemented. The casing should penetrate through the unstable upper materials or as otherwise determined at the time of drilling by an engineer or geologist overseeing the drilling operations.

8.1.1.3 Groundwater

Groundwater is not anticipated during drilling operations. However, limited perched groundwater may be encountered near the base of the undocumented artificial fill. If groundwater is encountered, the water will likely be displaced or pumped from the shaft during construction of the piers. The Contractor should be prepared to collect, store, filter, test, and dispose of the excess water in accordance with the project permits and per the public agencies having jurisdiction.

8.1.1.4 Excavation Spoils

We anticipated that the excavated materials will be deemed suitable for use as engineered fill. Depending on the alternative selected, spoils generated from excavations of the piers may need be removed from the site. Spoils should be disposed of in accordance with all applicable state and local provisions.

8.2 EARTHWORK

8.2.1 Site Preparation

The project area where the proposed pier foundations and areas to receive engineered fill should be cleared of all surface vegetation, debris, topsoil, and other deleterious materials. Based on the site conditions, the existing pavement section including existing asphalt and base materials will need to be removed. All underground utilities located within the limits of the project site should either be protected in-place or removed during construction.

To prepare the site to support the heavy equipment and other associated construction materials, a suitable working area will be needed. Grading should be performed to establish a level working pad where needed.

Any depressions or voids resulting from the site preparation work should be cleaned and backfilled with compacted engineered fill.

8.2.2 Excavations

Excavations for this project will include general grading to prepare the site for pier construction and or excavation of existing pavement, undocumented artificial fill, colluvium and bedrock materials within the area of the planned repair. Excavations may include trenching for installation of new utility or drainage lines. Excavations for drainage lines may extend up to 5 feet below the existing grades and along the existing road alignment or as required through private parcel. The onsite soils within the vicinity of Cliff Avenue failure consisted primarily of sandy and silty fill and colluvial deposits. Excavations in these materials should be able to be made with conventional equipment such as excavators.

Given the relatively loose, sandy and silty materials within the upper 8 to 10 feet below the existing road grade, temporary vertical cuts should not exceed 3 feet and should not be surcharged. Excavations greater than 3 feet in depth may be sloped at a maximum gradient of 1.5:1 (horizontal to vertical) to a maximum height of 10 feet. The slope gradient should be uniform from the top to the toe with no vertical components. Any excavations extending deeper than 10 feet, subject to surcharging, or which will undermine an adjacent private property line or public right-of-way shall utilize shoring.

For any excavations that will be conducted adjacent to existing foundations, these should be done above a reference plane with a gradient of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations. This is to prevent the removal of lateral support to the existing foundations.

All potential surcharge loads (e.g., construction equipment/materials, soil stockpiles) should be set back from the top of any vertical or sloped excavation a distance equivalent to the height of the cut unless the excavation is shored. Surfaces exposed in sloped excavations should be kept moist; however, not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made

to protect the face of the slopes from erosion during periods of rainfall. The soils exposed within the cuts should be observed during excavation by the geotechnical engineer or his/her representative.

8.2.3 Subgrade Preparation

The subgrade for areas to receive engineered fill should be scarified to a minimum depth of 8 inches, moisture conditioned and compacted until a firm condition is achieved. This firm subgrade should be able to be proof rolled using a fully loaded water truck or equipment that will be used during construction and show no signs of yielding.

If areas of wet or soft soils are encountered during subgrade preparation, the following measures can be applied:

- Excavate down until a firm subgrade has been reached and backfill with engineered fill.
- Over-excavate the unsuitable soils to a depth of 12 inches and replace with compacted aggregate fill such crushed stone or clean well-graded gravel material reinforced with a woven geotextile.

8.2.4 Materials for Engineered Fill

The in-situ soils may be used as engineered fill for backfilling of excavations, open-cut trenches, or structural support for ancillary improvements provided it has an organic content not exceeding 3 percent by weight, debris, or deleterious materials. No rocks or lumps larger than 3 inches in the greatest dimension shall be allowed.

In general, engineered fill materials, derived from onsite materials are anticipated to be acceptable unless otherwise specified by the geotechnical engineer.

All imported fill should be reviewed and approved by the geotechnical engineer prior to importation to the site. A minimum of five days will be required to evaluate and test the suitability of all proposed imported materials.

The import materials shall be non-expansive and have a Plasticity Index less than 12 percent and a Liquid Limit of 30 percent or less with a minimum friction angle of 32 degrees. The import material shall not contain rocks or lumps larger than 4 inches in greatest dimension and should not contain more than 15 percent of the material larger than 3 inches. These materials shall be free of organic debris and contaminated materials.

Imported fill materials should be placed and compacted to a minimum of 90 percent relative compaction at a moisture content of at least 2 percent over optimum as determined by the ASTM D-1557 (latest revision) test procedure. Materials placed within the upper 12 inches below the pavement section should be compacted to a minimum of 95 percent relative compaction at a moisture content of at least 2 percent over optimum.

8.2.5 Engineered Fill Placement and Compaction

Engineered fill should be placed on competent subgrade, spread evenly in uniform lifts not exceeding 8 inches, moisture conditioned to a minimum of 3 percent above optimum moisture, thoroughly mixed, and adequately compacted. The minimum degree of compaction of engineered fills used for grading should achieve a minimum relative compaction of 90 percent per ASTM D1557. Compaction of coarse aggregates used to bridge soft subgrades (i.e., crushed stone, clean gravel) should be compacted to a

minimum of 95 percent relative compaction with a moisture content of +/- 2 percent the optimum moisture value. The upper 12 inches of engineered fill below the pavement section should be compacted to a minimum of 95 percent relative compaction and moisture conditioned to minimum of 2 percent above optimum.

Compaction requirements for trench materials are specified in the subsequent section. Engineered fill materials that do not meet the specified relative compaction should be ripped, moisture conditioned and re-worked until the required relative compaction and moisture content are attained.

8.2.6 Trench Excavations and Materials

All trenches shall conform to current Cal/OSHA requirements. If the silty or clayey materials at the trench subgrade are found to be unstable (i.e., soft or mucky due to excessive moisture), then the subgrade shall be stabilized. If such conditions are encountered, the subgrade should be over-excavated 12 inches and replaced with angular crushed rock wrapped in nonwoven geotextile fabric. The fabric will prevent the underlying fines from migrating up into the crushed rock.

For the pipe bedding, it is recommended that at least 6 inches of crushed stone or equivalent be placed along the trench bottom. The use of an alternative to clean sand should be considered as a bedding material, since the sand may act as a conduit for groundwater which could accelerate erosion or discharge at undesirable locations. Measures such as the use of compacted clay trench plugs could be considered to prevent water migration into the trenches.

The pipe zone, which extends from the top of the bedding up to 12 inches above the top of pipe, should consist of free-draining sand (at least 90 percent passing a No. 4 sieve and less than 5 percent passing a No. 200 sieve), or other specified material, compacted to a minimum of 90 percent relative compaction. Directly atop of the piping, the backfill may be mechanically tamped to avoid damaging the conduit.

The final backfill is the portion extending from top of the pipe zone up to the finished grade. The in-situ soils may be reused as backfill provided they are free of organic matter and debris. This backfill should be compacted to at least 90 percent relative compaction (ASTM D-1557 latest edition). The trench should be capped with at least 12 inches of compacted, onsite soil similar to that of the adjoining subgrade. The upper 12 inches of trench backfill in areas to be paved should be compacted to a minimum of 95 percent relative compaction (ASTM D-1557). Water jetting or flooding to attain compaction of backfill should not be permitted. If the in-situ soils are excessively moist, then they will need to be sufficiently dried before placement.

8.2.7 Wet Weather Construction

We do not recommend that earthwork be performed during wet weather seasons. If site grading or construction is to be performed during rainy periods, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms could cause unstable excavations, delays to construction, and damage to previously completed work by saturating compacted fills and subgrades or flooding excavations.

Earthwork during rainy months will require extra effort and caution from the contractors. The contractor should be responsible for protecting their work to avoid damage by stormwater runoff or associated impacts from weather events. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by weather events.

8.3 SEISMIC GROUND MOTION PARAMETERS

The proposed slope repair will need to account for earthquake loading primarily in the form of significant ground accelerations during the design life of the planned repair. As presented earlier in the report, the site-specific analyses did not identify any liquefiable soils. Based on the characteristics of the in situ materials, the site is considered to be Site Class CD.

With this determination and in accordance with the California Building Code (CBC), which currently references the American Society of Civil Engineers (ASCE) 7-22, the mapped seismic parameters were obtained as shown in Table 6 below.

Table 8-1. 2025 CBC Mapped Seismic Parameters[1]

Parameter	Value
Risk Category	II
Site Class (Estimated Vs30=360 m/s)	CD
0.2 Second Spectral Response Acceleration, S_s	2.77 g
1.0 Second Spectral Response Acceleration, S_1	1.09 g
Site-Modified Spectral Response Acceleration for Short Periods, S_{MS}	2.93 g
Site-Modified Spectral Response Acceleration for 1-Sec Periods, S_{M1}	2.94 g
Designed Spectral Response Acceleration for Short Periods, S_{DS}	1.95 g
Designed Spectral Response Acceleration for 1-Sec Periods, S_{D1}	1.96 g
PGA Adjusted for Site Class Effects (PGA_M)	1.03 g

8.4 SURFACE DRAINAGE

Surface drainage was a significant contributor to the failure that is being repaired. Site grades need to be designed to direct flow to the inboard edge of the road. Additionally, it is likely grades may need to be adjusted beyond the repair to improve drainage, since the small segment of road represented by the repair is only a small part of a larger drainage. Surface drainage, specifically of stormwater runoff, should be directed to suitable an appropriate outlet.

8.5 EROSION AND SEDIMENT CONTROL

Erosion and sediment control should be implemented to minimize the discharge of sediment outside the proposed areas of disturbance and work activities. The contractor should be responsible for implementing and maintaining these measures throughout construction. All erosion control measures should comply with applicable local, state, and federal environmental provisions and project-specific permit conditions.

To mitigate surface erosion and sediment transport from the general construction area, the following measures are recommended:

- **Perimeter Controls:** Install 100 percent biodegradable fiber rolls (i.e., wattles) or compost filter socks along the perimeter of the work limits, particularly along the edge of payment or hardscape areas, to intercept and filter sheet flow from the site.

[1] Used Estimated Vs30=360 m/s (Site Class CD), values represent comparison of Vs30/1.3 and Vs30*1.3

- **Stockpile Management:** Excavated soils and construction materials should be stockpiled away from the edge of slope as far as practical. Stockpiles should be covered with secured plastic sheeting when not in active use.
- **Site Access:** A stabilized construction entrance (e.g., clean, crushed stone or tracking control mats) should be considered to prevent the transfer of mud and sediment onto adjacent paved roadways.

If construction activities require equipment directly on the slope below the road, temporary isolation measures will be required to protect the downslope improvements. Recommended methods include the use of post supported plywood, wire-backed silt fencing and sandbags.

Sandbag cofferdams should also be considered at regular intervals along contour on the pavement areas to reduce sheet flow velocities and control surface waters entering or exiting the site.

8.6 SITE RESTORATION

Portions of the slope and surrounding areas of the roadway disturbed during the construction activities should be stabilized by way of revegetation. slope surfaces should be tracked or raked to a depth of 1 to 2 inches in preparation for seeding. If the soil is highly compacted or lacks organic matter, a light application of weed-free compost may be incorporated. Once the surface is prepared, a native seed mix, specifically formulated for the local conditions, should be seeded directly onto the soil surface.

Immediately following seeding, rolled erosion control products (RECPs) should be installed. All RECPs should consist entirely of 100 percent biodegradable materials such as coir woven blankets or jute netting. To mitigate long-term environmental impacts, RECPs containing synthetic, plastic sheeting are not recommended.

The RECPs should maintain continuous contact with the seeded soil. Gaps between the RECPs and soil should be avoided as they could allow erosion to occur underneath the blanket. The RECPs should be anchored to the soil surface with biodegradable wooden stakes in lieu of metal staples at a frequency specified by the manufacturer.

As a supplemental measure, the use of bioengineering techniques such as live staking may be considered to provide long-term, deep-seated root reinforcement of the creek banks. If bioengineering is utilized, a qualified landscape architect should be consulted to develop a site-specific planting and maintenance plan.

It is recommended that the revegetation progress be monitored for at least one full growing season. Any areas with detected erosion along the slopes (i.e., gullies), displaced RECP, or insufficient vegetative growth (i.e., less than 70 percent coverage) should be repaired and reseeded immediately.

8.7 TECHNICAL REVIEW AND CONSTRUCTION OBSERVATIONS

Prior to construction, the geotechnical engineer should review the project plans and specifications for the preferred alternative (if prepared by others) for conformance with the intent of the recommendations presented in this report. Our office should be notified at least 48 hours before any required field observations by the contractor. Our presence should not be construed as an acceptance of responsibility for performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans and specifications.

9. Construction Costs

Haley & Aldrich developed a conceptual cost estimate for Alternative C based on a similar repair completed following intense storms in January 2023.

The conceptual repair is approximately $506,100 and includes:

- Seven 30-inch diameter piles that extend 20 feet deep ($140,000);
- 40 feet of grade beam that is 24 inches thick and 36 inches wide. For the estimate we used a thickness of 36 inches to account for stepping the grade beam ($75,000);
- 400 square feet of segmental block wall facing and wall drain, ($84,000);
- Excavation and backfill ($96,300)
- 40 feet of metal beam guard rail and terminal system ($20,000);
- A 50 foot long, 12 foot wide, pavement section ($44,900); and
- Mobilization (10%, $45,900).

The costs indicated above are approximate and do not include an estimated cost for design and construction related fees. Upon request and the selection of a preferred alternative, Haley & Aldrich can prepare a supplemental proposal and scope to prepare plans, specifications, and a detailed engineers estimate.

Note, the cost estimate above also does not include a contingency. We recommend that a minimum 20 percent contingency fund be allocated to account for any unanticipated conditions, contractor change orders, or other items that may be requested or required during the design and construction phases of the project. For this project the additional contingency is approximately $50,600.

As indicated above, the estimated costs for maintaining site controls and repairing damaged improvements for the property below the repair have not been in our engineering estimates. Permitting fees may also vary depending on the selected alternative and have not been included within this memorandum.

Additionally, H&A costs for construction observation and testing services have not been included. These services can be provided on a time and expenses basis and will vary depending on the level of effort required for the selected alternative. Additionally, our effort varies based on agency. In some cases, we have a limited role supplementing agency staff, while in other cases, the agencies ask us to handle all aspects of the project thus fulfilling the resident engineer/construction manager role.

The cost will depend on the alternative, contractor schedule and efficiency, and requested construction oversight and testing services scope. It is common for the services to vary between 5 and 40 percent of the construction cost based on the complexity and level of involvement. Small complex sites result in higher cost than a large simple project. A scope for these services can be provided upon request following the completion of the preferred alternative design, review of a project schedule from a selected contractor, and clarity of level of involvement requested.

10. Limitations

The conclusions and recommendations presented in this report are based on information provided to our office by the client regarding the proposed project, subsurface conditions encountered from our explorations, results of laboratory testing, and professional judgment. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

Site conditions described in the text of this report are those existing at the time of our field investigation and are not necessarily representative of the site conditions at other times or locations. The nature and extent of subsurface variations between borings may not become evident until construction. If variations are encountered during construction, Haley & Aldrich should be notified promptly so that conditions can be reviewed and recommendations reconsidered as appropriate.

The evaluation or identification of the presence of hazardous materials at the site was not requested and is beyond the scope of this project. This report should not be construed as an environmental audit or study. If there are any questions regarding this report, or if we may be of further service, please contact our office.

11. References

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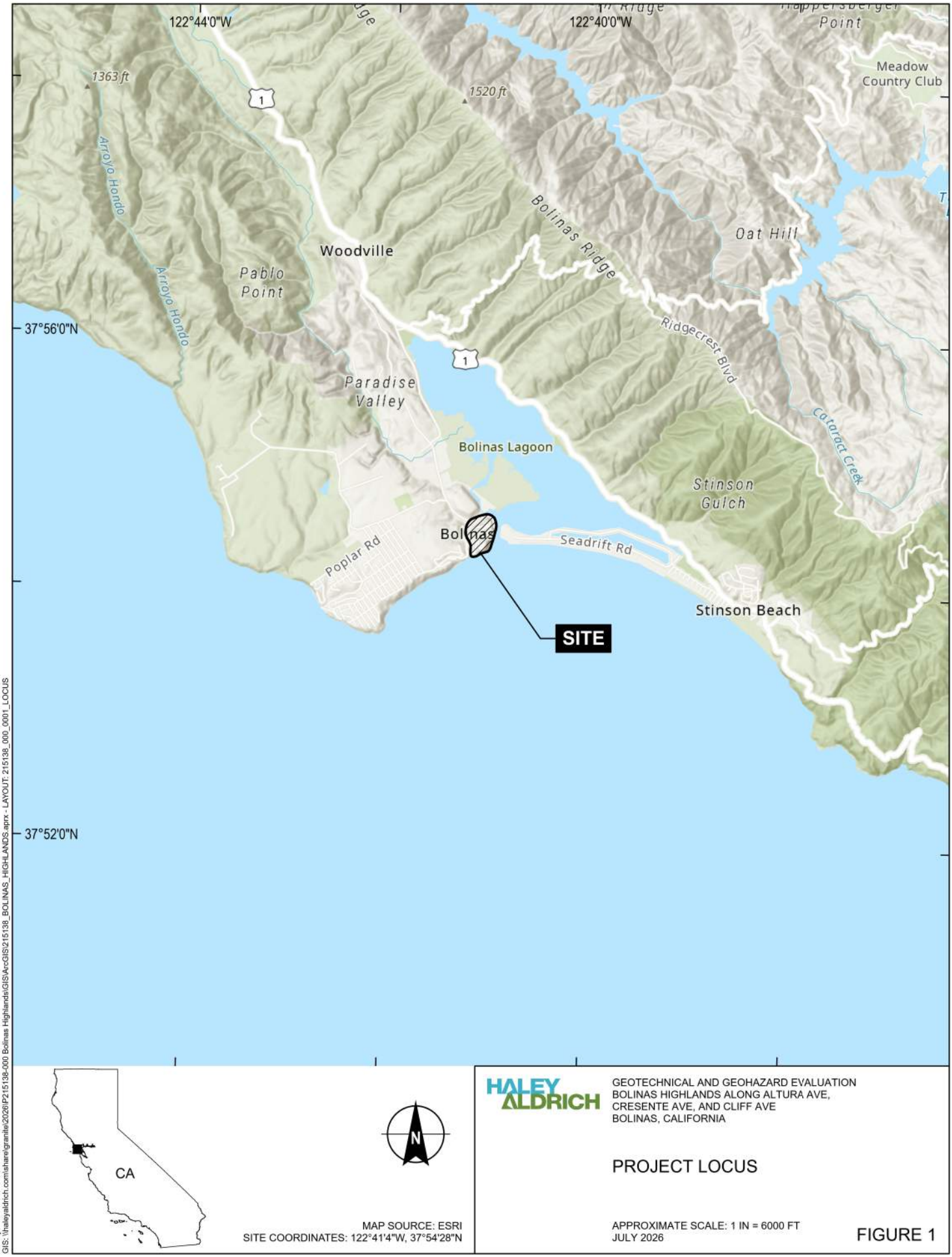
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FIGURES



MAP SOURCE: ESRI
SITE COORDINATES: 122°41'4"W, 37°54'28"N

HALEY ALDRICH

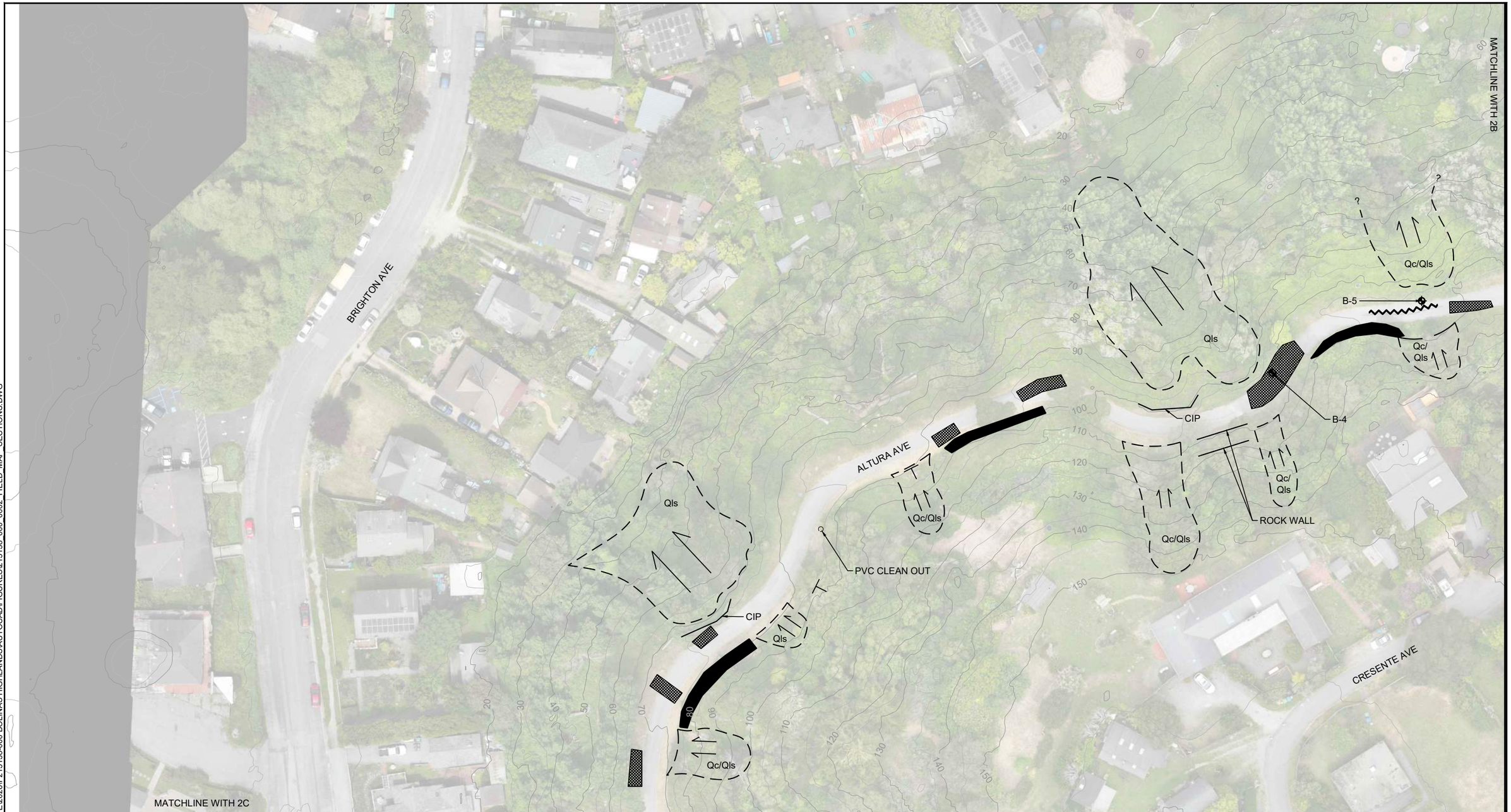
GEOTECHNICAL AND GEOHAZARD EVALUATION
BOLINAS HIGHLANDS ALONG ALTURA AVE,
CRESENTE AVE, AND CLIFF AVE
BOLINAS, CALIFORNIA

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 6000 FT
JULY 2026

FIGURE 1

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LEGEND

- APPROX LIMITS OF SLOPE INSTABILITY -ARROWS SHOW DIRECTION OF MOVEMENT -QUERIED WHERE INFERRED
- ASPHALT PATCH
- OVERSTEEPENED SLOPE
- CRACKS
- 3" DIAMETER PLASTIC PIPE OUTLET
- EXPLORATORY BORING (HALEY & ALDRICH, 2026)
- Qls LANDSLIDE DEPOSIT
- Qc COLLUVIUM
- Qaf ARTIFICIAL FILL
- CIP CAST-IN-PLACE CONCRETE RETAINING WALL

NOTES

1. TOPO AND ORTHO FROM H&A LiDAR SCANNING, COLLECTED MAY 1, 2026.
2. FIELD RECONNAISANCE MAPPING COMPLETED BY D. BURGER JUNE 1, 2026.

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SCALE IN FEET

HALEY ALDRICH

BOLINAS HIGHLANDS
BOLNAS, CA

SITE AND GEOLOGIC FEATURES MAP

SCALE: AS SHOWN
JULY 2026

FIGURE 2A

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LEGEND

- APPROX LIMITS OF SLOPE INSTABILITY -ARROWS SHOW DIRECTION OF MOVEMENT -QUERIED WHERE INFERRED
- ASPHALT PATCH
- OVERSTEEPENED SLOPE
- CRACKS
- 3" DIAMETER PLASTIC PIPE OUTLET
- EXPLORATORY BORING (HALEY & ALDRICH, 2026)
- Qls — LANDSLIDE DEPOSIT
- Qc — COLLUVIUM
- Qaf — ARTIFICIAL FILL
- CIP — CAST-IN-PLACE CONCRETE RETAINING WALL

NOTES

1. TOPO AND ORTHO FROM H&A LiDAR SCANNING, COLLECTED MAY 1, 2026.
2. FIELD RECONNAISANCE MAPPING COMPLETED BY D. BURGER JUNE 1, 2026.
3. OBSERVED 18" SCARP, UNDERLAIN BY 2" ASPHALT CONCRETE, UNDERLAIN BY 8" AGGREGATE BASE, UNDERLAIN BY 10" GRAVEL NEAR 66 ALTURA AVE.

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SCALE IN FEET

HALEY ALDRICH

BOLINAS HIGHLANDS
BOLINAS, CA

SITE AND GEOLOGIC FEATURES MAP

SCALE: AS SHOWN
JULY 2026

FIGURE 2B

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MATCHLINE WITH 2A

STEEL BEAM AND LAGGING WALL

ALTURA AVE

PARK AVE

BRIGHTON AVE

CRESENTE AVE

CRESENTE AVE

MATCHLINE WITH 2D

Qc/Qls

Qc/Qls

Qc/Qls

Qc/Qls

Qls

Qls

Qls

RIP-RAP

B-1

B-2

B-3

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CLIFF AVE

Qc/Qls

Qc

Qc/Qls

CROSS SECTION, SEE FIGURE 7

MATCHLINE WITH 2E

LEGEND

APPROX LIMITS OF SLOPE INSTABILITY
-ARROWS SHOW DIRECTION OF MOVEMENT
-QUERIED WHERE INFERRED

ASPHALT PATCH

OVERSTEEPENED SLOPE

CRACKS

3" DIAMETER PLASTIC PIPE OUTLET

EXPLORATORY BORING (HALEY & ALDRICH, 2026)

Qls LANDSLIDE DEPOSIT

Qc COLLUVIUM

Qaf ARTIFICIAL FILL

CIP CAST-IN-PLACE CONCRETE RETAINING WALL

NOTES

1. TOPO AND ORTHO FROM H&A LiDAR SCANNING, COLLECTED MAY 1, 2026.
2. FIELD RECONNAISANCE MAPPING COMPLETED BY D. BURGER JUNE 1, 2026.

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SCALE IN FEET

HALEY ALDRICH

BOLINAS HIGHLANDS
BOLINAS, CA

SITE AND GEOLOGIC FEATURES MAP

SCALE: AS SHOWN
JULY 2026

FIGURE 2C

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LEGEND

- APPROX LIMITS OF SLOPE INSTABILITY -ARROWS SHOW DIRECTION OF MOVEMENT -QUERIED WHERE INFERRED
- ASPHALT PATCH
- OVERSTEEPENED SLOPE
- CRACKS
- 3" DIAMETER PLASTIC PIPE OUTLET
- EXPLORATORY BORING (HALEY & ALDRICH, 2026)
- Qls LANDSLIDE DEPOSIT
- Qc COLLUVIUM
- Qaf ARTIFICIAL FILL
- CIP CAST-IN-PLACE CONCRETE RETAINING WALL

NOTES

1. TOPO AND ORTHO FROM H&A LiDAR SCANNING, COLLECTED MAY 1, 2026.
2. FIELD RECONNAISANCE MAPPING COMPLETED BY D. BURGER JUNE 1, 2026.

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SCALE IN FEET

HALEY ALDRICH

BOLINAS HIGHLANDS
BOLINAS, CA

SITE AND GEOLOGIC FEATURES MAP

SCALE: AS SHOWN
JULY 2026

FIGURE 2D

\\HALEYALDRICH.COM\SHARE\GRANITE\2026\P215138-000 BOLINAS HIGHLANDS\AUTOCAD\FIGURES\215138 000 0002 FIELD MAP SECTIONS.DWG Sheet: 2D Printed: 7/23/2026 3:18 PM MACARREGUI

MATCHLINE WITH 2C

CLIFF AVE

PARK AVE

BRIGHTON AVE

CLIFF AVE

CLIFF AVE

CLIFF AVE

VALLINE LN

CRESENTE AVE

Qc/Qls

Qls

CIP

MANHOLES

STACKED BOULDERS

Qc/Qls

LEGEND

APPROX LIMITS OF SLOPE INSTABILITY
-ARROWS SHOW DIRECTION OF MOVEMENT
-QUERIED WHERE INFERRED

ASPHALT PATCH

OVERSTEEPENED SLOPE

CRACKS

3" DIAMETER PLASTIC PIPE OUTLET

EXPLORATORY BORING (HALEY & ALDRICH, 2026)

Qls LANDSLIDE DEPOSIT

Qc COLLUVIUM

Qaf ARTIFICIAL FILL

CIP CAST-IN-PLACE CONCRETE RETAINING WALL

NOTES

1. TOPO AND ORTHO FROM H&A LiDAR SCANNING, COLLECTED MAY 1, 2026.
2. FIELD RECONNAISANCE MAPPING COMPLETED BY D. BURGER JUNE 1, 2026.

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SCALE IN FEET

HALEY ALDRICH

BOLINAS HIGHLANDS
BOLINAS, CA

SITE AND GEOLOGIC FEATURES MAP

SCALE: AS SHOWN
JULY 2026

FIGURE 2E

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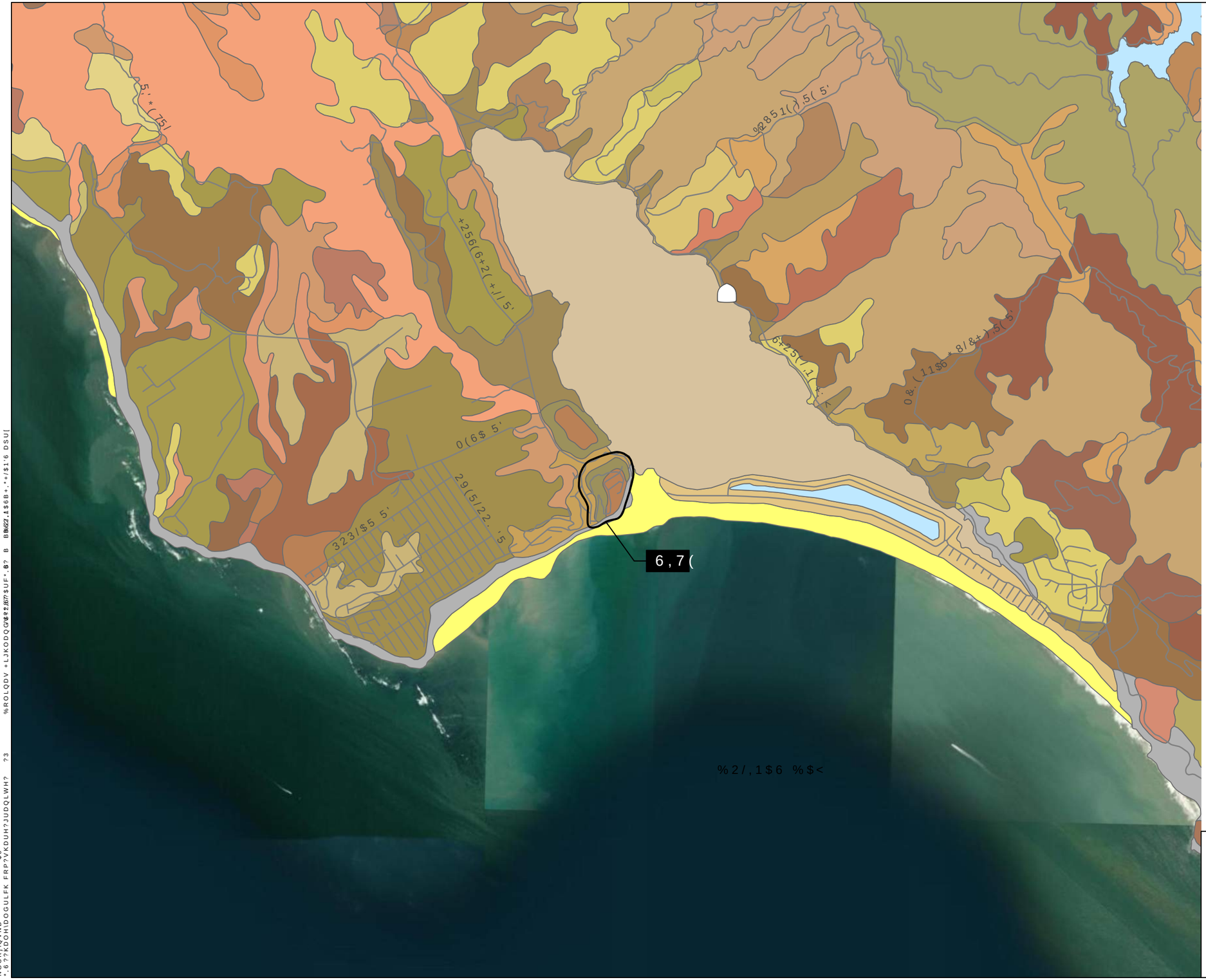
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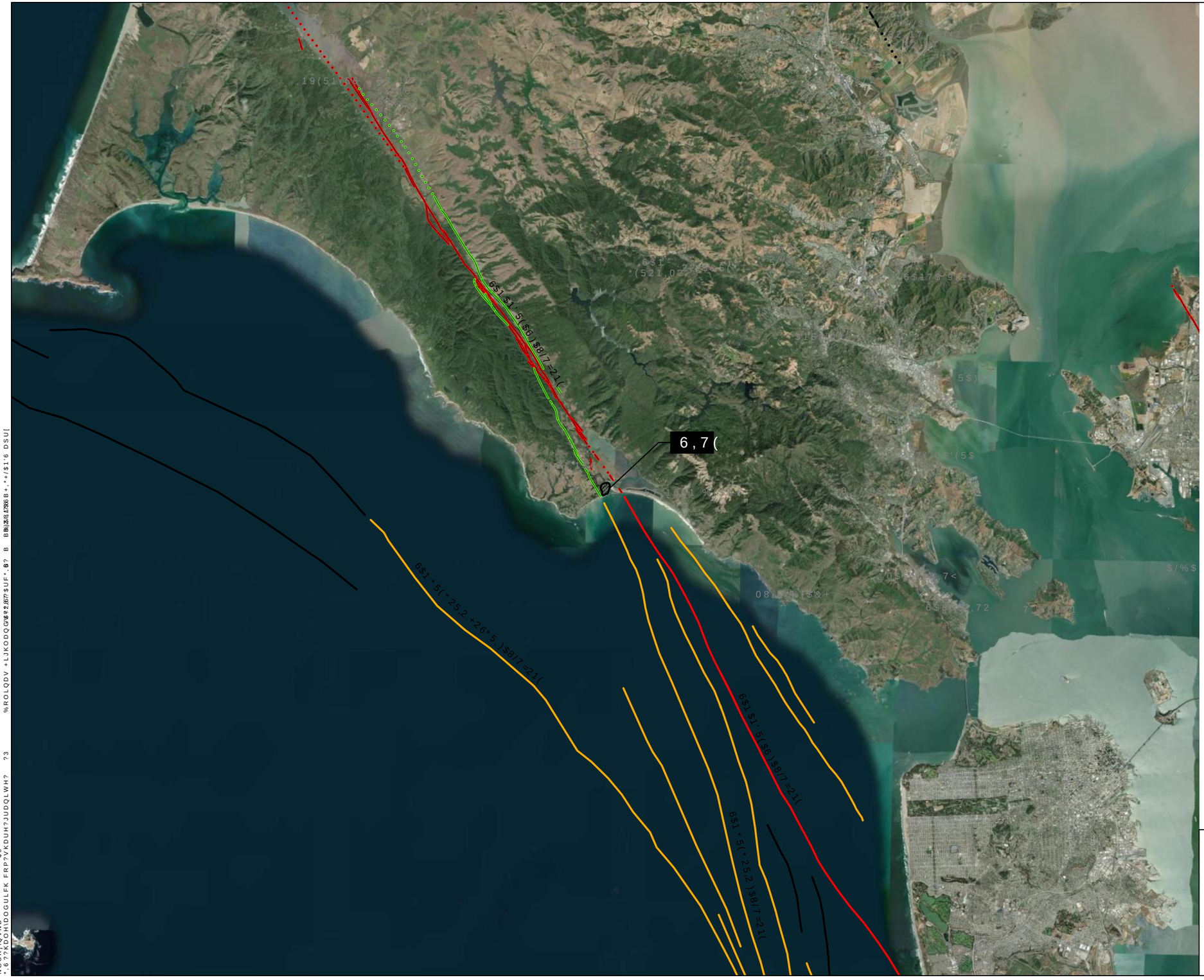
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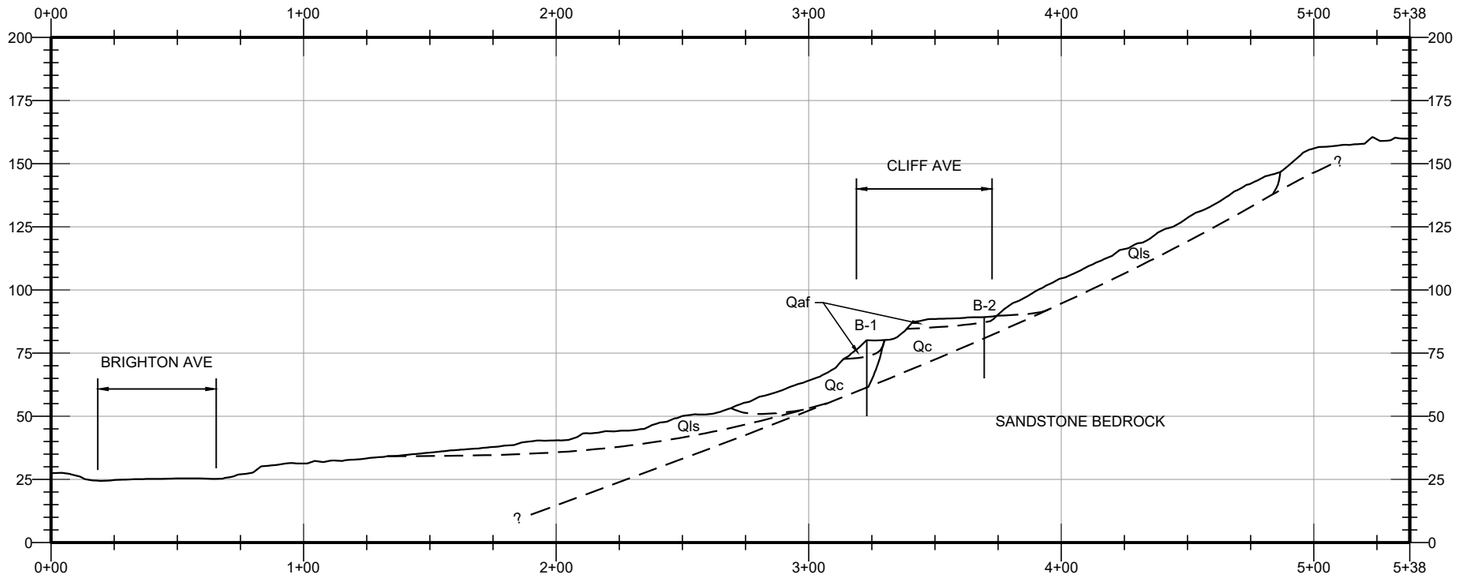
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LEGEND

Qls	LANDSLIDE DEPOSIT
Qc	COLLUVIUM
Qaf	ARTIFICIAL FILL

NOTES

1. EXPLORATORY BORINGS BY HALEY & ALDRICH COMPLETED ON JUNE 1, 2026 BY R. BRISEÑO.

N

0 60 120

SCALE IN FEET

HALEY ALDRICH

BOLINAS HIGHLANDS
BOLINAS, CA

GEOLOGIC CROSS SECTION

SCALE: AS SHOWN
JULY 2026

FIGURE 7

MACARREGUI Printed: 7/23/2026 3:32 PM Sheet: PROFILES
\\HALEYALDRICH.COM\SHARE\GRANITE\2026\P215138-000 BOLINAS HIGHLANDS\AUTOCAD\FIGURES\215138 000 0002 FIELD MAP SECTIONS.DWG

APPENDIX A
Boring Logs and Laboratory Results

KEY TO SYMBOLS

CLIENT Bolinas Highlands
PROJECT NAME Preliminary Geohazard Evaluation - Bolinas Highlands
PROJECT NUMBER 0215138-000
PROJECT LOCATION Bolinas Highlands, CA

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)


ASPHALT: Asphalt


FILL: Fill (made ground)


ML: USCS Silt


SANDSTONE: Sandstone


SM: USCS Silty Sand


SP: USCS Poorly-graded Sand


SP-SM: USCS Poorly-graded Sand with Silt

SAMPLER SYMBOLS


California Modified Sampler


Grab Sample


Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)
TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
▽ Water Level at Time Drilling, or as Shown
▼ Water Level at End of Drilling, or as Shown
▼ Water Level After 24 Hours, or as Shown

BORING NUMBER B-01

CLIENT Bolinas Highlands
PROJECT NAME Preliminary Geohazard Evaluation - Bolinas Highlands
PROJECT NUMBER 0215138-000
PROJECT LOCATION Bolinas Highlands, CA
DATE STARTED 6/1/2026 **COMPLETED** 6/1/2026
GROUND ELEVATION 77 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR ConeTec
COORDINATES: **LATITUDE** 37.90696 **LONGITUDE** -122.685365
DRILLING RIG/METHOD 8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING 7.0 ft / Elev 70.0 ft -
LOGGED BY R. Briseno **CHECKED BY** D. Burger
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0	5 in. ASPHALT									
	Poorly Graded SAND (SP) to Poorly Graded SAND with Silt (SP-SM): black, moist, loose, fine sand (ARTIFICIAL FILL)	GB								
2.5										
5.0		CM	2-2-4							
	Poorly Graded SAND (SP): black, moist, loose, fine sand angular gravel up to 1/2 in.									
	▽ Wet at 7 ft. (possible perched water)	SPT	5-7-11							
7.5	Poorly Graded SAND (SP): brown, moist, medium dense, fine sand (COLLUVIUM)									
10.0		CM	8-22-22		94	20				
12.5		SPT	6-12-25							
	Poorly Graded SAND (SP): gray, moist, medium dense, fine sand									
15.0										

(Continued Next Page)

BORING NUMBER B-01

CLIENT Bolinas Highlands **PROJECT NAME** Preliminary Geohazard Evaluation - Bolinas Highlands

PROJECT NUMBER 0215138-000 **PROJECT LOCATION** Bolinas Highlands, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0		Poorly Graded SAND (SP): gray, wet, dense, fine sand	CM	12-17-32		104	23				
			SPT	6-19-19							
17.5		SANDSTONE: gray, moist, soft/friable, fine sand, iron stains, micaceous, thin planar bedding, intensely weathered, poorly cemented (BEDROCK)									
20.0			CM	11-20-34							
		SANDSTONE: dark gray, moist, soft/friable, fine sand, slightly weathered/fresh, moderately cemented									
22.5											
25.0		SANDSTONE: dark gray, moist, soft/friable, fine sand, slightly weathered/fresh, moderately cemented	CM	11-30-50		101	24				
27.5											
30.0		SANDSTONE: dark gray, moist, soft/friable, fine sand, slightly weathered/fresh, moderately cemented	CM	9-14-23		96	27				

Bottom of borehole at 31.5 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-02

CLIENT Bolinas Highlands
PROJECT NAME Preliminary Geohazard Evaluation - Bolinas Highlands
PROJECT NUMBER 0215138-000
PROJECT LOCATION Bolinas Highlands, CA
DATE STARTED 6/1/2026 **COMPLETED** 6/1/2026
GROUND ELEVATION 80 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR ConeTec
COORDINATES: **LATITUDE** 37.907037 **LONGITUDE** -122.685368
DRILLING RIG/METHOD 8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING 8.0 ft / Elev 72.0 ft
LOGGED BY R. Briseno **CHECKED BY** D. Burger
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0	12 in. ASPHALT									
	Poorly Graded SAND (SP): black, moist, loose, fine sand, trace subangular to subrounded gravel up to 1/4 in. (ARTIFICIAL FILL)	CM	6-7-6			15				
2.5		SPT	2-1-2							
5.0	Poorly Graded SAND (SP): black, moist, very loose, fine sand	CM	0-1-1		100	23				
7.5	▽ Wet at 8 ft. (possible perched water)	SPT	0-0-2							
	Poorly Graded SAND with Silt (SP-SM): brown and gray, moist, dense, fine sand (COLLUVIUM)									
10.0		CM	4-14-25		103 106	22 19				
12.5		SPT	11-16-20							
15.0										

(Continued Next Page)

BORING NUMBER B-02

CLIENT Bolinas Highlands **PROJECT NAME** Preliminary Geohazard Evaluation - Bolinas Highlands

PROJECT NUMBER 0215138-000 **PROJECT LOCATION** Bolinas Highlands, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0		Poorly Graded SAND with Silt (SP-SM): brown and gray, moist, dense, fine sand, iron stains	CM	11-16-30		99 100	19 25				
17.5			SPT	14-21-27							
		SANDSTONE: gray, moist, soft/friable, fine sand, iron stains, micaceous, thin planar bedding, moderately weathered, poorly cemented (BEDROCK)									
20.0			CM	9-15-17		101	24				
22.5		SANDSTONE: dark gray and brown, moist, soft/friable, fine sand, slightly weathered, moderately cemented, near vertical fracturing									
25.0		SANDSTONE: dark gray, moist, soft/friable, fine sand, slightly weathered, moderately cemented	CM	9-23-42		102	23				
27.5			SPT	8-15-21							

Bottom of borehole at 28.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-03

CLIENT Bolinas Highlands
PROJECT NAME Preliminary Geohazard Evaluation - Bolinas Highlands
PROJECT NUMBER 0215138-000
PROJECT LOCATION Bolinas Highlands, CA
DATE STARTED 6/1/2026 **COMPLETED** 6/1/2026
GROUND ELEVATION 89 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR ConeTec
COORDINATES: **LATITUDE** 37.90702 **LONGITUDE** -122.685198
DRILLING RIG/METHOD 8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING --- Not encountered
LOGGED BY R. Briseno **CHECKED BY** D. Burger
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		3 in. ASPHALT									
		Poorly Graded SAND (SP): brown and black, moist, loose grades to medium dense, fine sand (ARTIFICIAL FILL)	CM	7-6-7		101	20				
2.5		Wet from 2.5 to 3 ft.	SPT	3-13-17							
		Poorly Graded SAND (SP): brown, moist, medium dense, fine sand (COLLUVIUM)									
5.0			CM	13-26-40		104	16				
		Poorly Graded SAND (SP): brown, moist, very dense, fine sand									
7.5			SPT	13-24-32							
		SANDSTONE: gray, moist, soft/friable, fine sand, moderately weathered, poorly cemented (BEDROCK)									
10.0		SANDSTONE: brown, moist, soft/friable, fine sand, moderately weathered, poorly cemented	CM	15-26-39		91	14				
12.5			SPT	15-20-22							
15.0											

(Continued Next Page)

CLIENT Bolinas Highlands **PROJECT NAME** Preliminary Geohazard Evaluation - Bolinas Highlands

PROJECT NUMBER 0215138-000 **PROJECT LOCATION** Bolinas Highlands, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0		SANDSTONE: brown, moist, soft/friable, fine sand, moderately weathered, iron stains, poorly cemented	CM	13-23-49		102	12				
17.5		Color change to gray	SPT	21-33-42							
20.0		SANDSTONE: gray, moist, soft/friable, fine sand, moderately weathered, poorly cemented	CM	19-39-50		101	9				
22.5			SPT	20-28-39							

Bottom of borehole at 23.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-04

CLIENT Bolinas Highlands
PROJECT NAME Preliminary Geohazard Evaluation - Bolinas Highlands
PROJECT NUMBER 0215138-000
PROJECT LOCATION Bolinas Highlands, CA
DATE STARTED 6/2/2026 **COMPLETED** 6/2/2026
GROUND ELEVATION 108 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR ConeTec
COORDINATES: **LATITUDE** 37.908652 **LONGITUDE** -122.684798
DRILLING RIG/METHOD 8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING --- Not encountered
LOGGED BY R. Briseno **CHECKED BY** D. Burger
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		12 in. ASPHALT									
		Silty SAND (SM): moderate brown, moist, very loose, some rounded to subangular gravel up to 1/4 in., siltstone rock fragments (ARTIFICIAL FILL)	CM	5-4-4							
2.5			SPT	1-1-1							
5.0		Sandy SILT (ML): yellowish orange, moist, soft, fine sand, few fine rounded gravel, siltstone rock fragments	CM	3-4-8	1.25	119	12				
7.5		Silty SAND (SM): yellowish orange, moist, medium dense, fine sand (COLLUVIUM)	SPT	5-9-14							
10.0		Silty SAND (SM): yellowish orange, moist, medium dense, fine sand	CM	7-18-24		99	18				
12.5			SPT	10-14-18							
15.0											

(Continued Next Page)

BORING NUMBER B-04

CLIENT Bolinas Highlands **PROJECT NAME** Preliminary Geohazard Evaluation - Bolinas Highlands

PROJECT NUMBER 0215138-000 **PROJECT LOCATION** Bolinas Highlands, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0			CM	24-34-36		83	10				
17.5		Poorly Graded SAND (SP): yellowish orange, moist, dense, fine sand	SPT	13-17-26							
20.0			CM	20-38-50		101	10				
22.5		SANDSTONE: yellowish orange, moist, soft/friable, fine sand, very poorly cemented, very intensely weathered to decomposed, faint rock fabric (BEDROCK)	SPT	16-26-33							

Bottom of borehole at 23.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-05

CLIENT Bolinas Highlands
PROJECT NAME Preliminary Geohazard Evaluation - Bolinas Highlands
PROJECT NUMBER 0215138-000
PROJECT LOCATION Bolinas Highlands, CA
DATE STARTED 6/2/2026 **COMPLETED** 6/2/2026
GROUND ELEVATION 117 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR ConeTec
COORDINATES: **LATITUDE** 37.90878 **LONGITUDE** -122.684457
DRILLING RIG/METHOD 8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING --- Not encountered
LOGGED BY R. Briseno **CHECKED BY** D. Burger
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		12 in. ASPHALT									
		Silty SAND (SM): black and yellowish orange, moist, very loose, fine sand (ARTIFICIAL FILL)	CM	4-3-1		108	15				
2.5		Sandy SILT (ML): black, moist, very soft (COLLUVIUM)	SPT	1-0-0							
5.0		SILT (ML): black, moist, very soft, few fine sand	CM	0-1-2	0.5			24	22	2	
7.5			SPT	0-1-2			23				
10.0		SILT (ML): yellowish orange and gray, moist, firm, iron stains, soft zone at 11 ft.	CM	1-6-12	2.5 0	92	30				
12.5			SPT	3-5-6				25	28	NP	
15.0											

(Continued Next Page)

BORING NUMBER B-05

CLIENT Bolinas Highlands **PROJECT NAME** Preliminary Geohazard Evaluation - Bolinas Highlands

PROJECT NUMBER 0215138-000 **PROJECT LOCATION** Bolinas Highlands, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0											
		SILT (ML); yellowish orange and gray, moist, firm, iron stains	CM	2-7-15	4.5+	96	28				
17.5			SPT	5-7-10							
20.0		Gravelly SILT lens: greenish gray and brown, moist, firm, fine gravel									
		SANDSTONE: brown, moist, soft/friable, fine sand, very poorly cemented, very intensely weathered to decomposed (BEDROCK)	CM	8-23-36		104	11				
22.5											
25.0		SANDSTONE: brown, moist, soft/friable, fine sand, very poorly cemented, very intensely weathered to decomposed	CM	10-23-21							

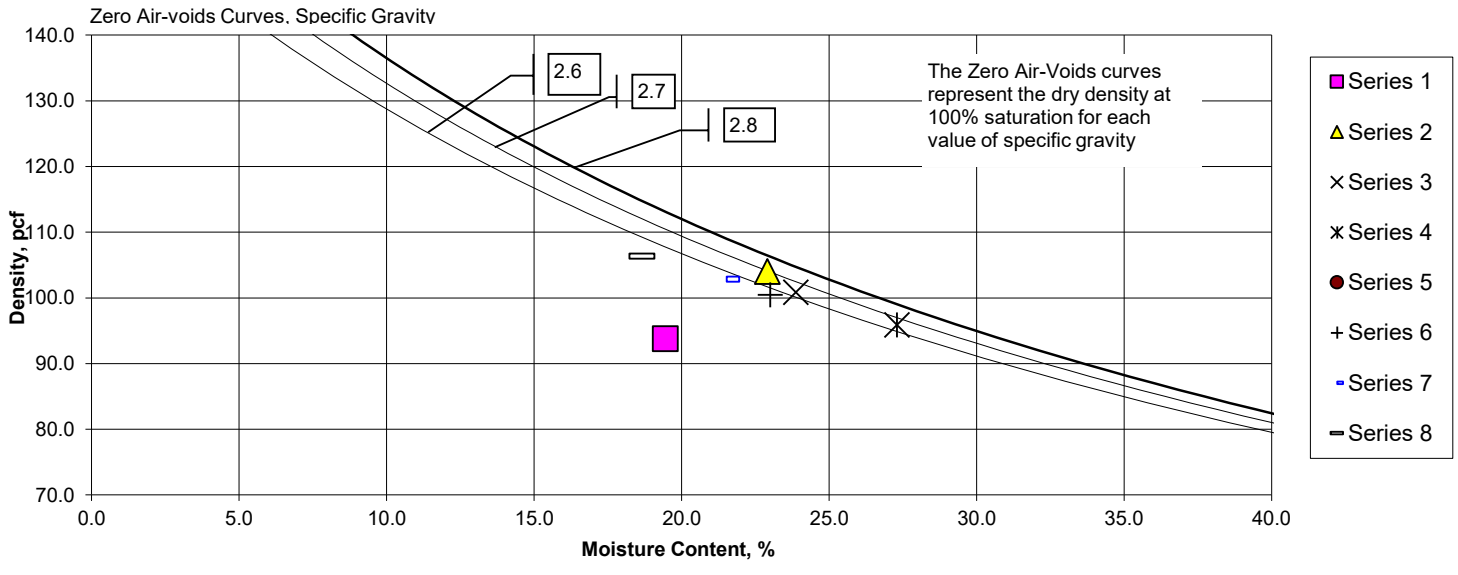
Bottom of borehole at 26.5 ft. Borehole backfilled with neat cement grout.

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	715-201a	**Project No.**	0215138-000	**By:**	RU
Client:	Haley & Aldrich	**Date:**	06/19/26		
Project Name:	Bolinas Highlands	**Remarks:**	B-02 @ 2.0' - sample disturbed; m/c only.		

Boring:	B-01	B-01	B-01	B-01	B-02	B-02	B-02	B-02
Sample:	1-5	1-8	1-13	1-15	2-2	2-5	2-7	2-8
Depth, ft:	11.0	16.0	26.0	31.0	2.0	6.0	10.5	11.0
Visual Description:	Brown Clayey SAND	Olive Brown Clayey SAND	Bluish Gray Sandy CLAY	Bluish Gray Sandy CLAY	Brown Sandy CLAY w/ Gravel	Brown Sandy SILT	Reddish Brown Sandy CLAY	Reddish Brown Sandy CLAY
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70		2.70	2.70	2.70
Moisture, %	19.5	22.9	23.9	27.3	14.6	23.0	21.5	18.7
Wet Unit wt, pcf	112.0	127.8	124.9	122.0		123.5	124.9	126.2
Dry Unit wt, pcf	93.7	104.0	100.9	95.9		100.4	102.8	106.4
Dry Bulk Dens.ρb, (g/cc)	1.50	1.67	1.62	1.54		1.61	1.65	1.70
Saturation, %	65.8	99.7	96.0	97.2		91.6	90.9	86.1
Total Porosity, %	44.4	38.3	40.2	43.1		40.4	39.0	36.9
Volumetric Water Cont,Θw,%	29.2	38.2	38.6	41.9		37.0	35.5	31.8
Volumetric Air Cont., Θa,%	15.2	0.1	1.6	1.2		3.4	3.5	5.1
Void Ratio	0.80	0.62	0.67	0.76		0.68	0.64	0.58
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

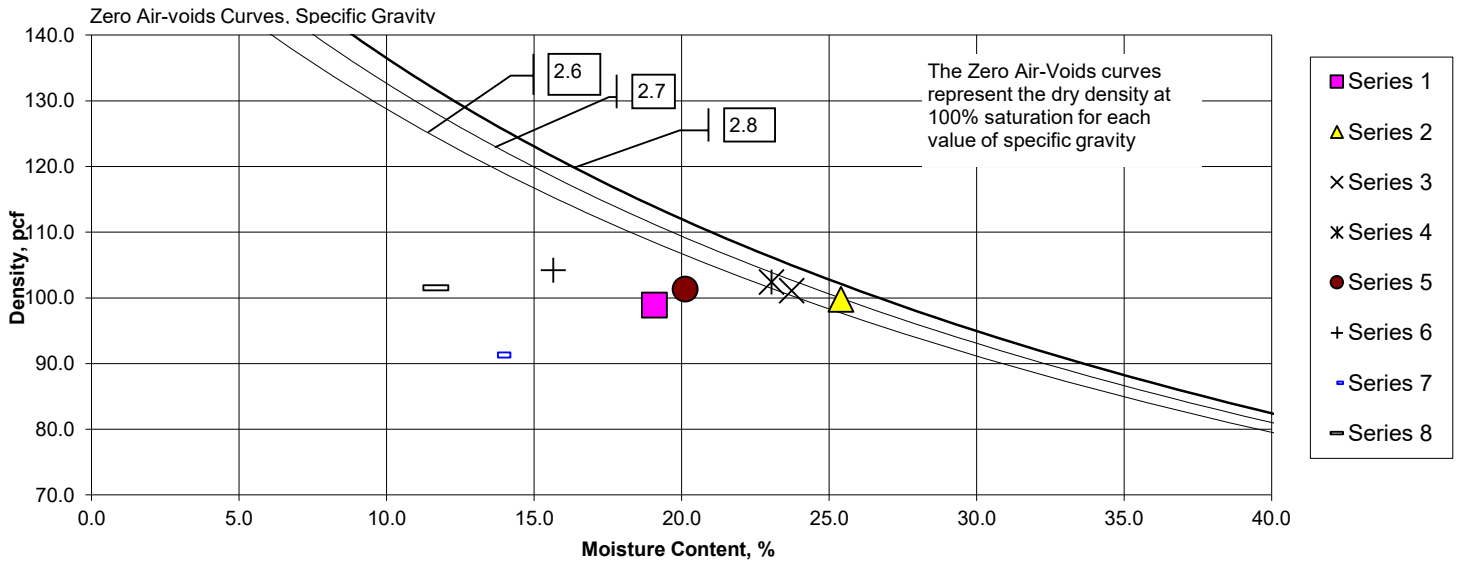


Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	715-201b	**Project No.**	0215138-000	**By:**	RU
Client:	Haley & Aldrich	**Date:**	06/19/26		
Project Name:	Bolinas Highlands	**Remarks:**			

Boring:	B-02	B-02	B-02	B-02	B-03	B-03	B-03	B-03
Sample:	2-10	2-11	2-14	2-16	3-2	3-5	3-8	3-11
Depth, ft:	15.5	16.0	21.0	26.0	2.0	6.0	11.0	16.0
Visual Description:	Yellowish Brown Silty SAND	Yellowish Brown Silty SAND	Gray Silty SAND	Gray Silty SAND	Brown Silty SAND	Brown SAND w/ Clay	Yellowish Brown Silty SAND	Yellowish Brown Silty SAND
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Moisture, %	19.1	25.4	23.7	23.1	20.1	15.7	13.8	11.7
Wet Unit wt, pcf	117.8	125.1	125.1	126.0	121.7	120.5	103.9	113.3
Dry Unit wt, pcf	98.9	99.8	101.1	102.4	101.3	104.2	91.3	101.5
Dry Bulk Dens.ρb, (g/cc)	1.58	1.60	1.62	1.64	1.62	1.67	1.46	1.63
Saturation, %	73.1	99.5	96.0	96.4	81.9	68.4	44.0	47.7
Total Porosity, %	41.3	40.8	40.0	39.2	39.9	38.2	45.8	39.8
Volumetric Water Cont,Θw,%	30.2	40.6	38.4	37.8	32.7	26.1	20.1	19.0
Volumetric Air Cont., Θa,%	11.1	0.2	1.6	1.4	7.2	12.0	25.7	20.8
Void Ratio	0.70	0.69	0.67	0.65	0.66	0.62	0.85	0.66
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

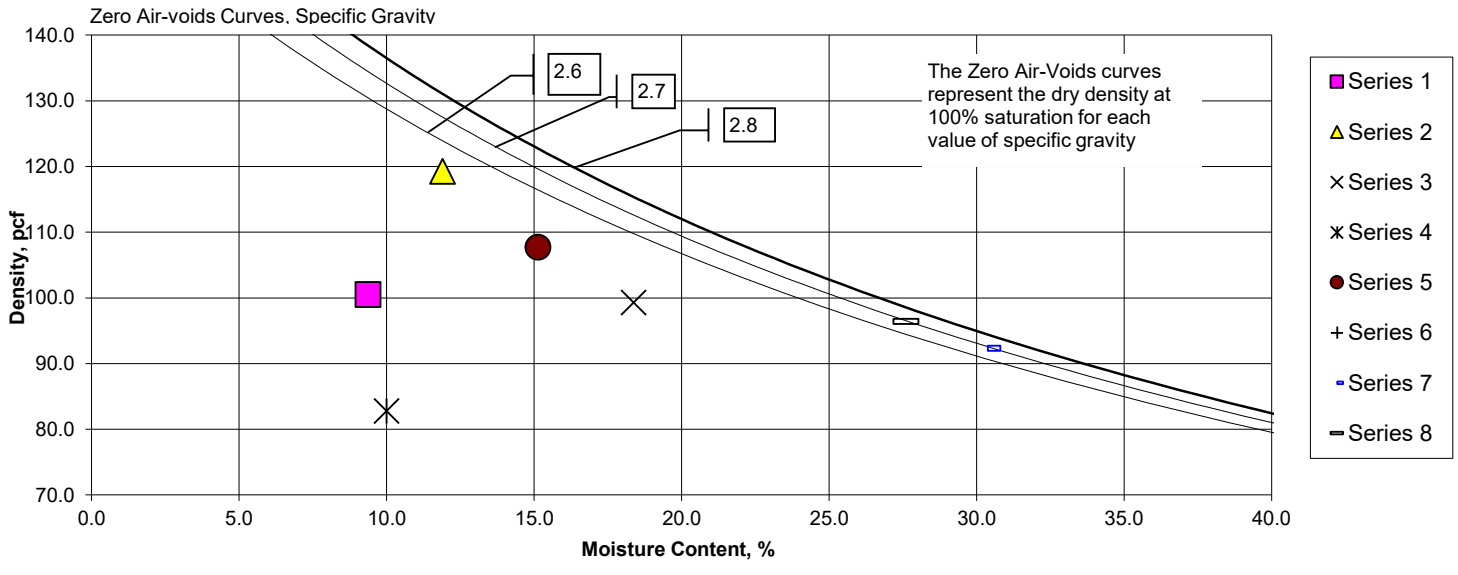


Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	715-201c	**Project No.**	0215138-000	**By:**	RU
Client:	Haley & Aldrich	**Date:**	06/22/26		
Project Name:	Bolinas Highlands	**Remarks:**			

Boring:	B-03	B-04	B-04	B-04	B-05	B-05	B-05	B-05
Sample:	3-14	4-3	4-5	4-6	5-2	5-6	5-8	5-11
Depth, ft:	21.0	6.0	10.5	16.0	2.0	6.5	11.0	16.0
Visual Description:	Olive Silty SAND	Brown Clayey SAND	Yellowish Brown Silty SAND	Yellowish Brown SAND	Olive Brown Silty SAND	Dark Grayish Bown Sandy SILT	Olive Brown SAND w/ Clay	Yellowish Brown Clayey SAND
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70		2.70	2.70
Moisture, %	9.4	11.9	18.4	10.0	15.1	22.8	30.4	27.6
Wet Unit wt, pcf	109.9	133.4	117.5	91.0	124.0		120.4	123.0
Dry Unit wt, pcf	100.5	119.2	99.3	82.7	107.7		92.3	96.4
Dry Bulk Dens.ρb, (g/cc)	1.61	1.91	1.59	1.33	1.73		1.48	1.54
Saturation, %	37.4	77.7	71.1	26.1	72.3		99.4	99.6
Total Porosity, %	40.4	29.3	41.1	50.9	36.1		45.2	42.8
Volumetric Water Cont,Θw,%	15.1	22.7	29.2	13.3	26.1		44.9	42.6
Volumetric Air Cont., Θa,%	25.3	6.5	11.9	37.6	10.0		0.3	0.2
Void Ratio	0.68	0.41	0.70	1.04	0.57		0.83	0.75
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

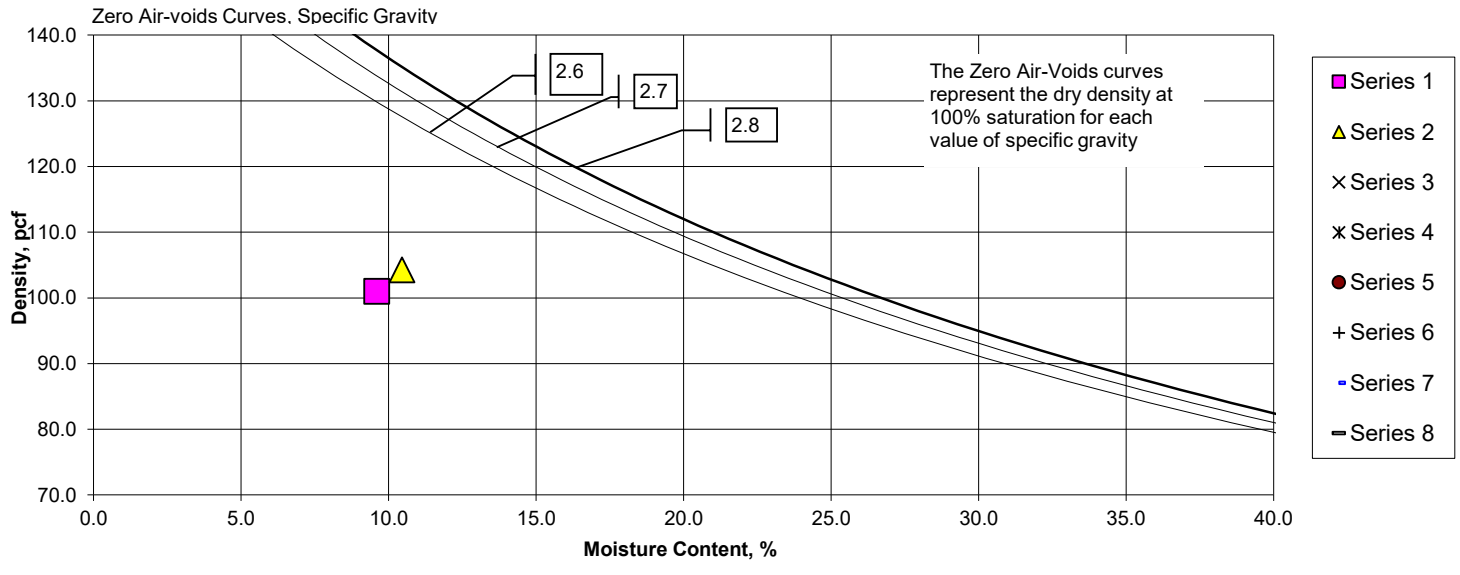


Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

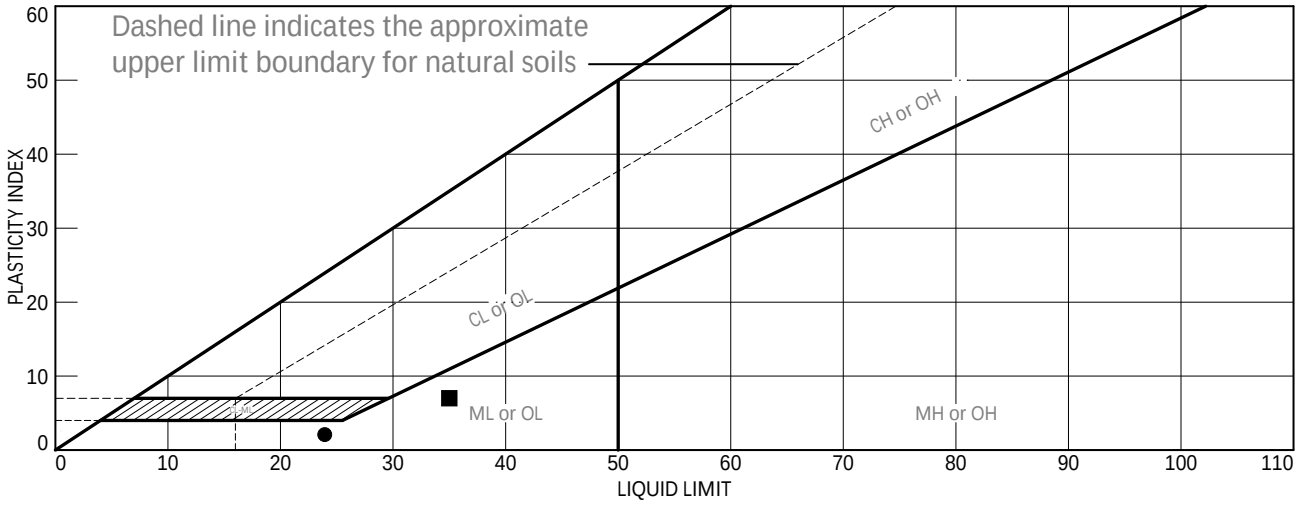
CTL Job No:	715-201d	**Project No.**	0215138-000	**By:**	RU
Client:	Haley & Aldrich	**Date:**	06/23/26		
Project Name:	Bolinas Highlands	**Remarks:**			

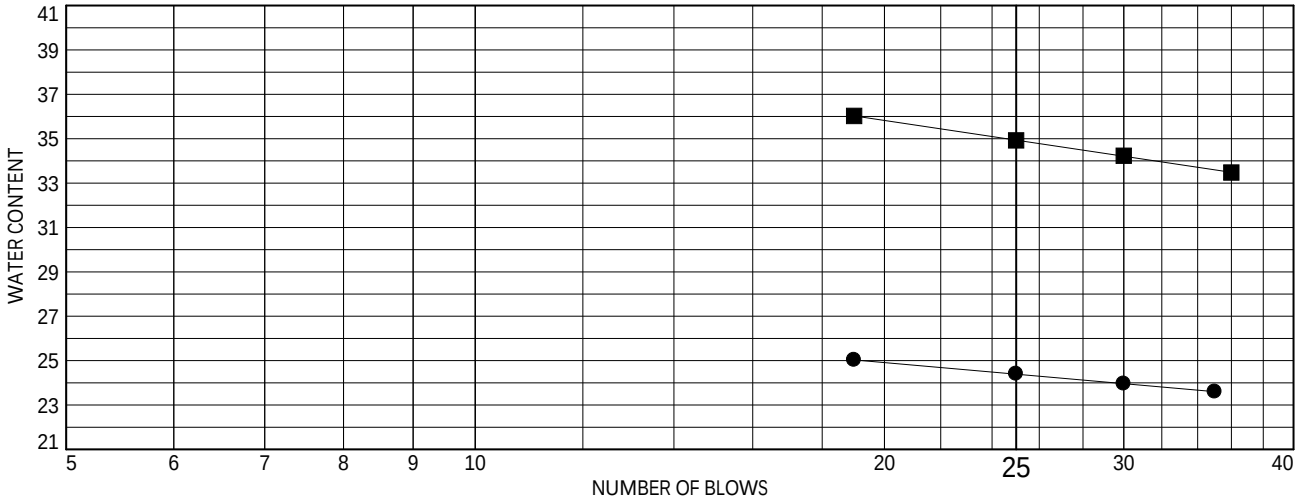
Boring:	B-04	B-05						
Sample:	4-12	5-14						
Depth, ft:	21.0	21.0						
Visual Description:	Light Olive Brown Clayey SAND	Brown Clayey SAND						
Actual G_s								
Assumed G_s	2.70	2.70						
Moisture, %	9.6	10.5						
Wet Unit wt, pcf	110.7	115.1						
Dry Unit wt, pcf	101.0	104.2						
Dry Bulk Dens.ρb, (g/cc)	1.62	1.67						
Saturation, %	38.7	45.8						
Total Porosity, %	40.1	38.2						
Volumetric Water Cont,Θw,%	15.5	17.5						
Volumetric Air Cont., Θa,%	24.6	20.7						
Void Ratio	0.67	0.62						
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.



LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Brown Silty SAND	24	22	2			
■	Yellowish Brown Sandy SILT	35	28	7			

Project No. 715-201 Client: Haley & Aldrich

Project: Bolinas Highlands - 0215138-000

●Source of Sample: B-05 Depth: 6.0' Sample Number: 5-5

■Source of Sample: B-05 Depth: 11.5' Sample Number: 5-9

Remarks:

●Sample was prepared using the wet prep method.

■Sample was prepared using the wet prep method.

COOPER TESTING LABORATORY

Figure

Corrosivity Test Summary

CTL # 715-201 **Date:** 6/25/2026 **Tested By:** PJ **Checked:** PJ

Client: Haley & Aldrich **Project:** Bolinas Highlands **Proj. No:** 0215138-000

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-01	1-10	20.5	-	786	-	15	225	0.0225	7.1	-	2.9	Bluish Gray Silty SAND
B-02	2-9	11.5	-	6323	-	9	19	0.0019	6.8	-	2.0	Yellowish Brown Silty SAND