GEOLOGY & GEOLOGIC HAZARD UPDATE & PRELIMINARY GEOTECHNICAL STUDY

BOLINAS MARINE FIELD STATION FACILITY

Geosphere Project No. 91-55182-PW
23 September 2020

PREPARED FOR:

MARIN COMMUNITY COLLEGE DISTRICT
835 College Avenue
Kentfield, CA 94904

PREPARED BY:

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ATLAS

Geosphere Consultants, Inc.
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

— AN ATLAS COMPANY —

September 23, 2020

Marin Community College District
835 College Avenue
Kentfield, California 94904

Attention: Mr. Isidro Farias, Director of Capital Projects

Subject: **Geology & Geologic Hazard Update and Preliminary Geotechnical Study**
College of Marin – Bolinas Marine Field Station Facility
72 Wharf Road, Bolinas, California 94924
Geosphere Project No. 91-55182-PW

Dear Mr. Farias:

Geosphere Consultants, Inc. has completed a Geology & Geologic Hazard Update and Preliminary Geotechnical Engineering Study for the proposed redevelopment of the District property located at 72 Wharf Road in Bolinas, California for a new Marine Field Station facility, including a new office/classroom building. This report has been prepared based on our discussions with Marin Community College District (District) personnel as well as the project architect. Transmitted herewith are the results of our findings, conclusions, and preliminary recommendations regarding site geologic hazard constraints, as needed geologic mitigation strategies for development of the property, geotechnical design considerations affecting the proposed development, and discussion of and preliminary recommendations for building foundation support. In general, the proposed site improvements are considered geotechnically feasible provided the geotechnical and geologic considerations described in this report are addressed in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact Mr. Dare at (925) 314-7180, or by e-mail at cdare@geosphereinc.net. We greatly appreciate the opportunity to be of service to the District and to be involved in this project.

Sincerely,
GEOSPHERE CONSULTANTS, INC.

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73 Wharf Road
Bolinas, California

Client: **Marin Community College District**
Kentfield, California 94904

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this study was to evaluate the surface and subsurface conditions at the site and develop conclusions and recommendations regarding the feasibility of development at the project site from a geotechnical and geologic standpoint. This study provides our findings and conclusions regarding geologic hazards and geotechnical design and construction considerations affecting the development of the site as intended for a new Marine Field Station learning facility. Our work included development of recommendations regarding geologic hazard mitigation necessary for long-term use of the site, as well as feasible foundation types, site grading, and natural and excavation slope stability. This study was performed in accordance with the scope of work outlined in our proposal to the Marin Community College District (District), and as authorized by the District on June 17, 2020.

The scope of this study included the review of available geotechnical literature for the site, geologic interpretation and field reconnaissance mapping onto 2018 LiDAR base, geologic research, the drilling of three (3) exploratory borings within the project site, laboratory testing of selected samples retrieved from the borings, engineering and geologic analysis of the accumulated data, and preparation of this report. The conclusions and preliminary design recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering and geologic judgment and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The project site is located along the shoreline of Bolinas Lagoon in the town of Bolinas, California, as shown on Plate 1, *Vicinity Map*. The site, with an address of 72 Wharf Road, was formerly used as the College of Marin Marine Biology Laboratory, and is currently dormant. The facility is generally comprised of structurally substandard structures including a single-story, wood-framed laboratory building on the west, a small storage

building, a reinforced concrete, water storage tank behind the laboratory, a two-story wood-framed, main house (Bolinas Marine House) at the center of the property, and a small mechanical building at the southeast corner of the house. Segmented remnants of dilapidated timber and concrete retaining walls can be found along the back of the property (Plate 2, *Site Plan*; Plate 3, *Photo Gallery*).

The house and laboratory were constructed in 1914, and were both remodeled in 1964. The house has a structurally supported, raised-wood floor over a crawl space. Foundations for both the house and laboratory building are unknown, but are assumed to consist of continuous wall footings with interior column spread footings (Degenkolb, 2005).

Paved driveways are present on both sides of the main house, and three concrete paved parking spaces are present along the edge of Wharf Road in front of the laboratory building. The facility also includes a timber pile-supported dock on the east side of Wharf Road along the edge of Bolinas Lagoon. The dock was structurally upgraded in 1994. The east side of Wharf Road across the main house and laboratory appears to be supported by an intact and upright, approximately 6-foot high steel sheet-pile seawall (Plate 3, Photo 2).

The site is on the steep eastern margin of a marine terrace (Plate 1). The developed northern part of the property to Wharf Road is relatively flat with grade elevations ranging from El. +7 (NGVD Datum) to approximate El. +12 feet at the base of a very steep, generally densely vegetated cut slope at the southern (rear) margin, as indicated on Plate 4, *Proposed Development.* As illustrated on Plate 5, *Aerial Topography and Drainage,* the slope that ascends approximately 30 feet above the building pad followed by a steep to moderately steep to gentle native inclination to the top of the terrace. There are other adjoining and neighboring older residential and commercial properties on Wharf Road, including 52 Wharf Road (Bolinas Gallery) whose property extends upslope directly behind the District's property. The average geographic coordinates of the site used for engineering analysis are 37.910 degrees north latitude and 122.6838 degrees west longitude.

1.3 Proposed Development

Based on the Planning Package (dated December 11, 2019) provided by the Architect, we understand the proposed project will consist of demolition and removal of all existing structures on the site to make way for construction of a new 2,416 square foot, single story marine biology laboratory facility designed to modern structural standards, as shown on Plate 4. The new building will include a laboratory classroom, office, storage,

and utility space, and restrooms to support college science programs. The proposed improvements also include a new paved parking area in front of the new building, retaining walls, utilities, and surface drainage measures.

1.4 Validity and Use of Report

This report is a preliminary study intended to help evaluate the critical geologic and geotechnical aspects of the site, which will affect the viability and potential cost of developing the site with the intended development scheme. This level of study provides guidance on geologic and geotechnical issues, which may affect the type and amount of site grading required, mitigation of site geologic hazards potentially affecting the development, and alternative appropriate foundation design types for the planned development. Final project design may require additional future study and engineering based on the District's evaluation of the conclusions and preliminary recommendations presented in this report, and a design-level geotechnical engineering study will need to be performed in order to fully develop design level recommendations for civil and structural design of the project.

This report should be considered valid for five years after publication. If the actual development differs considerably from that described above, conclusions and recommendations regarding the geotechnical feasibility of development could differ and may need to be re-evaluated by Geosphere.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), California Geological Survey (CGS), and other government agencies, and other online sources, as listed in the References section. A previous study conducted for the District prepared by Fugro West, Inc. (Fugro), titled, *Preliminary Focused Geologic Hazard Assessment, Marine Biology Laboratory, Bolinas, California*, dated October 6, 2005, was also reviewed, and this report was used as a baseline for updating our current geologic hazard assessment presented herein.

2.2 Geologic Reconnaissance and Mapping

Site-specific engineering geologic field mapping was limited due to vegetation cover and precipitous terrain. Interpretation of LiDAR bare earth imagery and topography was relied on to accomplish the engineering geologic and geologic hazard mapping presented in Section 4, *Geology and Geologic Hazard Evaluation*.

2.3 Geotechnical Field Exploration

In order to characterize the geotechnical subsurface conditions across the property, a field exploration program was conducted, which consisted of drilling three exploratory borings on July 14, 2020 under the supervision of a Geosphere Project Engineer and at the locations depicted on Plate 4 and Plate 6, *Engineering Geologic Map, Cross Sections A-A' & B-B', respectively*.

The borings were drilled to total depths of 15 to 25 feet using a track-mounted, Deeprock DR8K drill rig operated by Clear Heart Drilling, Inc. of Santa Rosa, equipped with seven-inch diameter, hollow-stem augers. The Geosphere Engineer visually classified the materials as the borings were advanced based on the Unified Soil Classification System. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners, and disturbed samples recovered using a two-inch outside diameter Standard Penetration Test (SPT) sampler. The samplers were driven by means of a 140-pound automatic hammer with an approximate 30-inch fall. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. For reporting purposes, all of the blow counts recorded using Modified California (MC) split spoon samplers in the field were subsequently converted to equivalent SPT blow counts using appropriate modification factors

suggested by Burmister (1948); i.e., multiplied by a factor of 0.65 assuming a liner sample with an inner diameter of 2.5 inches. Therefore, the boring logs provided in this report all show *equivalent SPT blow counts* for the MC sampler in lieu of blow counts recorded in the field. Following the completion of drilling, the boreholes were backfilled with drill cuttings and sealed at the surface using a cement grout.

The boring logs with descriptions of the various materials encountered in each boring, a key to the boring symbols, and select laboratory test results are included in Appendix A. Ground surface elevations indicated on the soil boring logs were estimated to the nearest foot based on the grading plan prepared for the project by CSWSt[2].

2.4 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are either presented on the boring logs, and/or are included in Appendix B. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2216 and ASTM 2937) – In-situ dry density and/or moisture tests were conducted on several samples to measure the in-place dry density and/or moisture content of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soils. Test results are shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) - Atterberg Limits tests were performed on select samples of cohesive soils encountered at the site. Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, and help to evaluate the expansive characteristics of the soil and determine the USCS soil classification. Test results are presented in Appendix B, and on the boring logs.

Particle Size Analysis (Wet and Dry Sieve; ASTM D422, D1140, and CT202) - Sieve analysis tests were conducted on select samples to measure the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented in Appendix B.

Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850m) – An Unconsolidated-Undrained triaxial strength test was conducted on a sample of cohesive soil material to measure the undrained shear strength of the tested material, which is useful in evaluating the foundation support characteristics of the soil. The sample was loaded under increasing axial load until near failure, defined as either peak deviator stress, or deviator stress at

15% strain, whichever occurs first. The deviator stress at failure is divided by two to obtain the undrained shear strength. The test result is presented in Appendix B.

Consolidation Test (ASTM D2435) – A consolidation test was performed on a relatively undisturbed sample of subsurface soil interpreted from field evaluation to be weak and potentially compressible. Consolidation test results are used in the analyses of potential future site settlement occurring due to the squeezing of pore water out of saturated, compressible materials in response to added loading from sources such as from new fill or structure (e.g., building foundation) loads. The results of the consolidation test is included in Appendix B.

Soil Corrosivity, Redox (ASTM G200), pH (ASTM G51), Resistivity (ASTM G57), Chloride (ASTM D4327), and Sulfate (ASTM D4327) - Soil corrosivity testing is performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the California Building Code (CBC) and International Building Code (IBC). The soil corrosivity results summary is presented in Appendix B, and the results are discussed in Section 3.3.

3.0 SUBSURFACE CONDITIONS

3.1 Subsurface Soil Conditions

During our subsurface exploration program, we investigated the site subsurface soils and evaluated soil conditions to the maximum depth of about 25 feet performed for this study. Based on our subsurface exploration and collected data, the subsurface materials generally consisted of a relatively loose to very loose, clay, silt-sand mixture directly overlying either clayey or sandy soils or, siltstone bedrock.

The surficial soils in the borings were found to consist primarily of a dark brown to gray to black, very loose to loose, sand-silt mixture (USCS classification as silty sand) varying in thickness from approximately 8 feet at Boring B-1, to about 4 feet in Boring B-3, comprised of slope wash from upslope sources with a possible overlying thin layer of surficial fill. The uppermost 3 feet of the soil profile, based on blow counts, appeared to be very loose in consistency. Below these near-surface materials, a thin 2- to 3-foot thick layer of apparent estuarine deposits, consisting of stiff silty clay with organics was encountered in Boring B-1. An apparent mixture of slope debris and artificial fill composed of loose silty sand with organics was encountered in Boring B-3.

Moderately weathered to very severely weathered siltstone bedrock was encountered below the surficial soils at approximate depths of 11 and 6 feet in Borings B-1 and B-3, respectively, and below the slope wash soils at a depth of 5 feet in Boring B-2, which was advanced near the toe of the ascending slope at the rear of the property. Interpretive cross section across the width and along the length of the property are included in Plate 6.

Atterberg Limits tests performed on two samples of the near-surface soils within the uppermost 6 feet resulted in measured Liquid Limits (LL) of 26 and 22, and corresponding respective Plasticity Indexes (PI) of 3 and 0, indicative of low plasticity and very low expansion potential (i.e., non-expansive material).

Additional details of materials encountered in the exploratory borings, including laboratory test results, are included in the boring logs in Appendix A, and laboratory test summaries are presented in Appendix B.

3.2 Groundwater

Groundwater was encountered in all of our borings at approximate depths ranging from 2.5 to 14 feet at the completion of drilling, corresponding to approximate Elevations of +4.5 to -5.0 feet. The measured shallow groundwater depth may be representative of a temporary perched groundwater condition. The site is adjacent to Bolinas Lagoon, the edge of which is located on the east side of adjacent Wharf Road, and as such, groundwater

elevations are likely to be affected by tidal fluctuations within the lagoon associated with the Pacific Ocean. Per the NOAA website, Mean Higher-High Water (MHHW) for Bolinas Lagoon is at El. +2.04, based on Mean Sea Level Datum.

We note that the borings may not have been left open for a sufficient amount of time to establish equilibrium groundwater conditions, and in addition may be influenced by tidal fluctuations. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage. We did not perform a detailed investigation of local groundwater conditions, which was beyond the scope of this study.

3.3 Corrosion Testing

One sample of the near surface fill soils was tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized on the following table.

Table 1: Summary of Corrosion Test Results

Soil Description	Boring No.	Sample Depth (feet)	Soluble Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Black Sandy Clay	B-1	1 - 3	402	73	205	3,554	Positive	5.6

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria summarized in Table 2:

Table 2: Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus pozzolan	0.45	4,500

The water-soluble sulfate content was measured to be 402 mg/kg or 0.0402% by dry weight in the soil sample, suggesting the site soil could have a negligible impact on buried concrete structures at the site. However, we note that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion content is above 0.06% by dry weight. The chloride content was measured to be 73 mg/kg or 0.0073% by dry weight in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation-reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105, and shown on Table 3.

Table 3: Soil Test Evaluation Criteria (AWWA C-105)

Soil Characteristics	Points		Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.			**Redox Potential, mV**	
<700	10		>+100	0
700-1,000	8		+50 to +100	3.5
1,000-1,200	5		0 to 50	4
1,200-1,500	2		Negative	5
1,500-2,000	1		**Sulfides**	
>2,000	0		Positive	3.5
PH			Trace	2
0-2	5		Negative	0
2-4	3		**Moisture**	
4-6.5	0		Poor drainage, continuously wet	2
6.5-7.5	0		Fair drainage, generally moist	1
7.5-8.5	0		Good drainage, generally dry	0
>8.5	5			

Assuming poor site drainage, the tested soil sample had a total score of 5.5 points, indicating a non-corrosive rating for the tested sample. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe, and use of cathodic corrosion protection is often recommended.

These results are preliminary, and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Evaluation of corrosion potential can be revisited prior to final detailed design of the project, particularly for site utilities, if needed. A California-registered professional corrosion engineer should evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals, to assess the need for, and design if necessary, long-term corrosion control measures, in the event such analysis is desired or required by the designers or regulatory agencies for project design.

4.0 GEOLOGY & GEOLOGIC HAZARDS

4.1 Site Geology

The site is at the base of a dissected marine terrace underlain by Plio-Pleistocene marine Merced Formation translated to Bolinas by right-lateral strike slip movement from a faulted relatively shallow depositional basin in the vicinity of Lake Merced. In Bolinas, the terraced Merced Formation occurs as a relatively narrow northwest-trending block within the San Gregorio and San Andreas Fault Zones on the southwest landward flank of Bolinas Lagoon northwestward up Pine Gulch Creek (Plate 7, *Regional Geologic Map*). Merced formation in Bolinas appears to be generally massive, with the few mapped bedding attitudes exhibiting gentle to moderately steep east to southeast dip (Plate 8, *Areal Geologic Map*).

The site is located on the southeast end of Bolinas Plateau marine terrace locally segmented into four parts from incision by relatively short, north and northeast trending structural drainages (Plate 9, *Geomorphic Map*). The steep perimeter flanks of the terrace segments have been pervasively dissected by parallel, seasonal channels draining the terrace surfaces and have produced debris flows and block landslides that impose a significant hazard to development on the terrace margins (Plate 9; also Plate 10, *Landslide Map*).

Interpretation of LiDAR bare earth imagery revealed the flank of Terrace Segment 1 (T_1) adjoining the site has sustained significant surficial debris flow, block, and rock/debris fall-avalanche failures (Plate 9; also Plate 11, *Surficial Landslide Map and Key to Photo Gallery*). Reconnaissance observations revealed evidence of debris flow and rock/debris fall/avalanche deposits at the base of the cut slope along the south margin of the site; however, evidence of impact to structures was not detected. Debris flow runouts depicted on Plate 11 are believed to be remnants of older events as we understand there have been no reports of notable recent slope failures.

A local exposure of the massive Merced Formation mantled by a thin veneer of erodible sandy silt with clay colluvium was mapped at the base of the cut slope in the southeast corner of the property. It was composed of very weathered to severely weathered, closely fractured sandy siltstone with a steep, dilated pervasive, adverse southwest dipping joint set and a low-angle northwest dipping wavy shear surface interpreted as an old landslide failure surface. Merced Formation in a scar of an apparent recent debris slide event was observed in the cut slope on the neighboring property from a distance and captured in a drone image (Plate 3, Photo 4). We suspect the debris flow runout on the east side of the property is in part derived from that source area. Evidence of global slope instability affecting the site was not detected.

The site lies within an area which can become quickly flooded in the event of tsunami, as indicated on Plate 12, *Tsunami Inundation Map*.

4.2 Seismic Induced Hazards

4.1.1 Ground Shaking

The magnitude 7.9, 1906 Earthquake, centered in the Golden Gate offshore of San Francisco approximately 30 miles to the south, caused severe seismic shaking widespread damage to structures and infrastructure in the site area from liquefaction along the margin of Bolinas Lagoon and landslides from the steep terrace flanks (Lawson and others, 1908; Figure 1). Following are reported accounts of earthquake shaking Impacts in Bolinas from the earthquake shaking (Lawson and others, 1908):

"Along the main street of Bolinas stand most of the houses. Of these about two-thirds were heaved, slid, tipt, and shattered into uninhabitable condition. Along the bay shore were 7 buildings. Of these 6 went over or down. At the Flagg Staff (sic) Inn the tipping of the house has thrown it so far east into the bay that one may sit along the upper floor and fish in 4 feet of water along the upper edge of the same room" (Figure 1).

"At the village of Bolinas, the soil has slopped down easterly toward the lagoon and on the east side of the road, which runs north and south, the building are entirely demolished, while those further up the hills on the west side are not so badly damaged."

…"the mouth of Bolinas bay…the high cliff –about 150 ft.—at the end of the peninsula have crumbled and fallen down carrying small trees with them."

Figure 1. Effects of 1906 earthquake along Wharf Road in site area. (Photos courtesy of A, Bolinas Museum and B, Marin County Free Library)

Since the 1906 event, there have been three moderate earthquakes one on each of the different segments of the Bay Area San Andreas Fault: 1957, magnitude 5.3 Daly City Earthquake centered on the San Francisco Peninsula Segment offshore of Daly City; 1989, magnitude 6.9 Loma Prieta Earthquake centered on the Santa Cruz Mountain Segment in Aptos; 1999, magnitude 5 Bolinas Earthquake centered on North Coast Segment beneath Bolinas Lagoon. Each of the earthquakes is believed to have caused moderate to strong ground shaking in the Bolinas but without notable damage to structures.

4.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean, poorly-graded sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. Typically, liquefaction potential increases with increased duration and magnitude of cyclic loading. However, because of the higher intergranular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 to 50 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

The soils encountered in our subsurface investigation indicated loose to very loose, granular silty sand occurring at depths of up to 5 to 8 feet in the borings. Therefore, a liquefaction analysis of these soils was conducted.

The initiation of liquefaction settlement results from seismic shaking, the magnitude of settlement increasing with increasing site ground accelerations. The 2019 CBC specifies that that Peak Ground Acceleration (PGA_M) as defined in the CBC be used for liquefaction and other seismic analyses. This resulted in a PGA_M used in our analysis of 1.26 g. We also used a Modal Magnitude of 7.6 applicable to the San Andreas Fault. A groundwater depth of 2.5 feet was assumed for analysis. A Factor-of-Safety (FS) of 1.0 was assumed to initiate dynamic settlement.

We utilized the software LiquefyPro, Version 5 (CivilTech Software, 2011) to perform our liquefaction analysis for our borings. The following Table 4 presents a summary of our analysis results.

Table 4: Summary of Liquefaction Settlement Analysis Results

Boring No.	Calculated Liquefaction Settlement (inches)
B-1	1.2
B-2	1.0
B-3	0.95

Lateral spreading is lateral ground movement, with some vertical component, resulting from liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances, generally when the liquefied layer is in relatively close proximity to an open, free slope face such as the bank of a creek channel. Lateral spreading can cause surficial ground tension cracking (i.e., lurch cracking) and settlement. Lateral spreading within the town of Bolinas adjacent to the lagoon resulting from the 1906 San Francisco earthquake was documented by Lawson and others (1908); also see Section 4.1.1.

The site is adjacent to Bolinas Lagoon on the east side of Wharf Road, with an otherwise free face supported by a vertical sheet pile wall along the eastern edge of Wharf Road opposite the District property. The uppermost approximately 8 feet of the site soil profile is considered to be liquefiable, and as such is susceptible to lateral movement toward the lagoon. The lateral continuity of the liquefiable near-surface soils were not conclusively determined, but assuming the near-surface soils are relatively similar across Wharf Road, we judge there to be a significant potential for some lateral spreading movement to occur at the site under existing site conditions, but lateral movement may be restrained at least in part by the presence of the sheet pile bulkhead wall. A more rigorous evaluation of the lateral spreading potential of the site beyond the scope of this initial study can be conducted if needed, depending on building foundation selection and associated site grading for the project.

4.1.3 Dynamic Densification (Settlement)

Dynamic densification or settlement is a process in which unsaturated, relatively clean sands and silts are densified by the vibratory motion of a strong seismic event. The soils encountered during our exploration above the estimated historic high groundwater table generally contained a relatively high fines content (i.e., exceeding 40 percent). Therefore, it is our opinion that the site is not susceptible to, and should not be significantly affected by dynamic settlement.

4.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones for designated Earthquake Fault Zones surrounding faults or fault segments judged sufficiently active, well defined and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997) for the San Andreas Fault (Plate 12, *Earthquake Fault Zones Map*). Permanent ground displacement on the segment of causative trace of the San Andreas Fault was confined to Bolinas Lagoon approximately 1,000 feet northeast of the site and was associated with 297 miles of documented fault ground surface rupture between Cape and San Juan Bautista.

On the basis of empirical evidence from past major earthquakes, it is generally accepted that future fault ground surface rupture will occur on the Historic trace of San Andreas Fault that ruptured in the 1906 earthquake offshore of the site (Plate 9). The Quaternary faults near the site (Plate 9) are reportedly well constrained and without reported evidence of Holocene activity (U.S. Geological Survey, 2002).

A lineament analysis based upon interpretation of bare earth LiDAR imagery revealed an absence of evidence of pervasive linear geomorphic or surface tonal features sympathetic in orientation or magnitude of rifting consistent with strike-slip faulting in the active fault zone extending upstream of the mouth of the Pine Gulch Creek northwest of Bolinas Lagoon. Numerous short, discontinuous linear drainages detected on the flanks of the terrace segments and less across the terrace surface are judged to represent joint-controlled differential erosion truncated by the northeast trending incisions dissecting the marine Terrace Segments 1-4, Plate 9.

Given the above, we judge the risk low for occurrence of fault surface rupture on the site during a future major earthquake. We expect the future ground rupture to be confined to trace approximately 1,000 feet offshore where rupture was observed following the 1906 earthquake.

5.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

The following preliminary conclusions and recommendations pertaining to the proposed redevelopment of this site are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

5.1 Conclusions

The site is considered to be geotechnically and geologically feasible for potential future development as being considered. The predominant geotechnical and geological issues that potentially affect development of this site are summarized below. Design-level studies for the proposed improvements should address these issues on a location-specific basis, as applicable.

Seismic Ground Shaking – Due to the location of the site adjacent to the main trace of the San Andreas Fault, the site should be designed with the expectation that the site could be subjected to severe ground shaking during the life of the development. As a minimum, new building design should consider the effects of seismic activity in accordance with the latest edition of the California Building Code (CBC).

Fault Ground Rupture – The project site is located within a State of California Earthquake Fault Zone for the San Andreas Fault. Ground rupture and permanent displacement has been documented along the main (1906) trace of the San Andreas Fault, located at its closest point to the project site, in Bolinas Lagoon approximately 1,000 feet northeast of the site. As discussed in Section 4.1.4, no evidence of pervasive linear geomorphic or tonal features, nor identified active or potentially fault traces were judged to project though the project site, and based on historic empirical evidence, we expect future fault surface displacement to most likely occur along the historic active fault trace. Therefore, we judge the potential for future fault rupture through the site to be low.

Weak, Compressible Surficial Soils – The surficial soils underlying the property were found in our borings to be very loose in the uppermost 3 feet of the soil profile, loose below a depth of 3 feet, and relatively compressible. As such, the near-surface soils are considered relatively weak from a structural support standpoint, and generally unsuitable in their present condition for foundation support, assuming acceptable foundation settlements, including static and seismic, for all but relatively lightly loaded, flexible structures where some seismic settlement would be acceptable. In addition, the thickness of the weak surficial soils likely vary across the new building

footprint, with a thickness likely less than 5 feet at the southeast corner of the footprint to near 10 feet at the northwest corner of the footprint, leading to potential differential settlement across the structure.

These near-surface soils may need to be reworked under the building foundations, or the foundations deepened to bear on competent native supporting material or bedrock, which is estimated to be present at a depth of about 5 to 6 feet beneath much of the new building, but likely deeper toward the northwest corner of the proposed building. Alternative foundation systems appropriate for the new laboratory/classroom building, together with remedial earthwork grading, as appropriate, are discussed in Section 5.2.1.

Seismically-Induced Settlement Considerations – The very loose to loose, granular fill soils as well as the more granular native soils underlying the project site are considered to be susceptible to liquefaction settlement and possible lateral spreading when subjected to very strong seismic shaking where saturated and under the water table. These soils were encountered in the borings to maximum depths of about 5 to 8 feet. As the encountered soils had a fines content measured to be at least 39 percent, they are not considered susceptible to dynamic (dry) settlements. As discussed in Section 4.1.2, calculated liquefaction settlements under the design earthquake using ground shaking criteria specified in the 2019 CBC ranged between 1 and 1¼ inches. Corresponding differential settlements across a structure would be considered to be about one-half to two-thirds the corresponding total settlement across the footprint of any structure.

These potential liquefaction settlements need to be considered in the design of any new building foundations. Alternatives for mitigating seismic settlements include:

- removal and recompaction of the liquefaction-susceptible soils below the building pad, and use of shallow footing foundations or a structural mat foundation;
- deepening the footing foundations to bear directly on the underlying competent siltstone bedrock; or
- use of deep foundations such as Cast-in-Drilled-Hole (CIDH) foundations (drilled piers) or auger-cast piles deriving support from the underlying bedrock.

See Section 5.2.1 for a further discussion of foundation considerations.

Liquefaction of the shallow near-surface soils during a very strong or design earthquake could also result in local lateral spreading of liquefied soil toward Bolinas Lagoon bordering the east side of Wharf Road. We note, however,

that the presence of the pile-supported dock as well as the sheet-pile bulkhead wall on the east side of the road will likely provide some resistance to lateral soil movement.

A more detailed evaluation of lateral spreading potential cannot be made based on our existing data, but we note that partial to complete removal and replacement or reworking of the liquefaction-susceptible soils, if selected, would significantly decrease the potential for lateral spreading in any case. Should a more rigorous evaluation of lateral spreading potential be desired, additional exploration closer to the lagoon, likely on Wharf Road, would be required to establish lateral continuity of the liquefaction-susceptible layer, and can be performed if feasible as part of the design-level geotechnical study.

Stability of Adjacent Ascending Slope – It is our opinion the steep slope on the north side of the property is highly susceptible to slope failure. The cut slope is oversteepened relative to the lithologic strength and pervasive adverse discontinuities (i.e. jointing) that historically has resulted in relatively shallow landsliding in the form of block glides and rock fall. Concentrated runoff from residential development at the top of the native hillside contributes to the potential for long runout of debris flow confined in the three swales that intersect the top of the cut slope on the southern margin of the site. Elsewhere, there is a perceived high risk for episodic debris slides and rock fall/avalanche from the face of the cut slope during rainfall. Given the accounts from the 1906 earthquake, the weathered and thoroughly jointed Merced Formation has high propensity for seismically-induced landsliding, which we judge would be more significant than under static conditions.

It is our opinion comprehensive landslide mitigation is warranted. Given the limited space, we anticipate steel mesh anchored to the ascending slope behind the new building with soil-nails could offer sufficient resistance to slough of surficial soil and weathered rock and rock fall while offering structural reinforcement to resist potentially deeper seismically-induced slope failure. We note that the portion of upper slope overlooking the District's property is owned by others (i.e., is part of the 52 Wharf Road property) and therefore stabilization of this portion of slope may be more problematic.

In addition, consideration should be given to design and construction of the rear (southern) wall on the new building to resist impact and temporarily contain up to 50 cubic yards of soil and rock debris from a rock fall or debris slide/flow (e.g., reinforced concrete stem walls in lower half of southern building profile).

Shallow Groundwater – Groundwater was encountered during our field exploration at depths varying between about 2.5 and 14 feet below the existing grade, as discussed in Section 3.2. Groundwater at the site will likely vary due to tidal influence, and as a result, the near-surface soils may be consistently near saturated, to saturated.

New construction will likely require either foundations to be founded at greater depths, or remedial earthwork involving over-excavation of soils in the building pad to take place. Such operations may be impacted by the presence of perched or otherwise shallow groundwater. Excavations deeper than 3 feet below existing subgrade may encounter perched or standing groundwater. At a minimum, exposed subgrade soils if remedial excavation earthwork is performed will likely be saturated and unstable under construction equipment loads, requiring subgrade stabilization and possible dewatering in order to compact the exposed bottom and to construct the overlying engineered fill layer. Deepened footing or deep foundations (e.g., CIDH piers) may encounter caving issues, or require dewatering during construction operations.

Tsunami Hazard Zone – The project site, as is all of the waterfront area of Bolinas, is mapped as within a zone of potential inundation as a result of a major earthquake occurring within or near the Pacific Ocean, where a tsunami could be generated to an extent sufficient to impact the greater San Francisco Bay region, including Bolinas. A more detailed or probabilistic assessment of tsunami potential sufficient to impact the local area is beyond the scope of this study.

5.2 Preliminary Geotechnical Recommendations

A discussion of and preliminary recommendations concerning geotechnical and foundation issues at the site are provided as follows:

5.2.1 Foundations

The proposed laboratory/classroom building may be supported by a variety of foundation types, each with particular advantages. Potential new foundations consist of shallow spread footing foundations, deeper pier foundations, or a structural mat slab foundation. Engineering analyses of our preliminary data suggest that building loads may induce static settlements ranging from about 1 to 2 inches for shallow foundations, depending on the magnitude of structural loading, due to compression and/or consolidation of the underlying, weak near-surface and organic lagoon soils. In addition, liquefaction settlements of the existing soil profile about an inch or more may also occur due to a strong design seismic event that may occur on the San Andreas Fault located directly

adjacent to the site. Differential settlements may be significant, exceeding 1 inch across the structure, due to the varying soil thickness below the site due to the variation in depth to bedrock across the site.

Footing Foundations – The proposed single-story wood-frame building may be supported by conventional spread and continuous footing foundations. However, to limit total and differential settlement to under 1 inch, the footings should be underlain by reworked and/or imported engineered fill to the bottom depth of the loose fill materials. The bottom of the loose materials is estimated below the building pad to range from 5 feet or less along the rear, southern perimeter of the building, to 9 to 10 feet below the northwestern corner of the building. Due to the small footprint of the building (24 to 28 feet wide by 89 feet long), overexcavation of the entire pad may be found to be more economical, and would also mitigate liquefaction and lateral spreading potential below the building, as well as allow the use of a slab-on-grade floor. Alternatively, footings may be extended deeper, and supported directly on bedrock, although in this case, the building floor would need to be structurally supported by the footing foundations. In addition, where the building subgrade soils are not reworked as engineered fill, lateral resistance of the existing soils will be reduced under both static and seismic conditions, and stiffening elements such as use of grade beams or interior continuous footings may be required to provide additional lateral resistance.

With the indicated soil improvements or footing embedment into bedrock, typical allowable dead plus live loads for footings may be in the range of 3,000 to 3,600 pounds per square foot (psf). Footings supported by engineered fill would have a minimum required embedment of 18 inches below pad subgrade.

Deep Foundations – Deeper foundations such as CIDH or Auger Cast Pile (ACP) foundations can also be used at the site to support the building loads. CIDH foundations would derive support by skin friction between the outside perimeter of the foundations and surrounding bedrock underlying the weak and liquefaction-susceptible soils. However, lateral resistance of such foundations may be limited within the upper portion of the piers/piles due to the weak nature of the near-surface fill soils. In addition, construction of CIDH foundations may require the use of casing or drill slurry to prevent caving of the drill holes should standing groundwater be encountered in the granular fill and native soils overlying the bedrock. ACP foundations would not be affected by the presence of groundwater, but due to the small size of the building, would probably not be cost effective due to mobilization and demobilization costs.

Pier/pile foundations may be expected to be in the range of 10 to 15 feet in length. Use of deep foundations would likely require that the main floor of the building be structurally supported by the pier foundation. Interconnecting grade beams would also provide some additional lateral resistance to wind or seismic forces.

Structural Mat Foundation – As a third foundation alternative, a structural mat foundation system may be used, where the mat slab would also serve as a slab-on-grade floor. The structural mat slab foundation should bear on a relatively uniform subgrade, in this case, consisting of a four-foot thick, engineered fill layer consisting of excavated and reworked, or imported select fill, compacted to engineered fill standards. We note that such an excavation to the indicated depth may expose oversaturated soils or encounter a local shallow groundwater table. In that case, the excavated surface may be unstable to construction equipment and may require dewatering and/or subgrade stabilization to allow construction equipment to operate on the subgrade and construct the supporting engineered fill layer. We note that the depth of over-excavation using the structural mat option may be somewhat less than the over-excavation required using footing foundations, but nonetheless should significantly reduce the potential for liquefaction settlement and lateral spreading. Long-term settlement of a mat foundation given the expected relatively light structural loads for the building is expected to be less than 1 inch, with additional seismic settlement likely less than ½ inch.

5.2.2 Site Grading

Site Preparation, Fill Materials and Grading - In general, the site should be cleared of the existing onsite buildings and other structures, existing vegetation, organic topsoil, debris, existing loose or soft surficial fill, and other deleterious materials within the proposed development area. Removed fill soil may be evaluated by the Geotechnical Engineer for possible reuse and placement as engineered fill, but we anticipate some soils containing organic materials will be encountered, which would not be suitable for reuse as engineered fill material. The grading contractor should be aware of buried objects and underground utilities at the site, which are to be removed or abandoned appropriately. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with properly compacted engineered fill or other material approved by the Geotechnical Engineer. The Geotechnical Engineer should observe all excavation and backfill operations during site grading.

Imported fill, if required, should be non-expansive in nature, with a Plasticity Index of 12 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should also be free of

environmental contaminants, organic materials and debris, and should not contain rocks or lumps greater than six inches in maximum size, and at least 95 percent smaller than three inches in maximum size. All import fill materials should be approved by the Geotechnical Engineer prior to use on site.

Newly placed engineered fill soil should be properly moisture conditioned; placed in uncompacted, loose lifts 8 inches in maximum thickness; and compacted to a minimum 90 percent relative compaction based on the ASTM D1557 (Modified Proctor Method) compaction standard. Note that the fine, silty nature of the near-surface soils would make proper moisture conditioning for compaction more difficult than usual.

The completed building pad subgrade, as well as exposed excavation subgrade prior to placement of engineered fill or backfill must be stable under equipment loads prior to building construction or engineered fill placement, respectively. Because the existing onsite soils, particularly at depth, are expected to be near saturated to completely saturated with significant silt content, the exposed excavation subgrade is expected to be unstable, and would require stabilization using geotextile, geogrid, or chemical stabilization (e.g., cement treatment), depending on extent of instability encountered during construction, as well as relative cost considerations. Also, note that due to the proximity to sea level, groundwater seepage could be encountered in deeper excavations, with some form of dewatering possibly required to keep standing water out of the excavation.

Excavation Slopes – The onsite near-surface subsurface soils encountered during our field investigation consisted primarily of very loose to loose granular soils within the uppermost 6 to 8 feet of the soil profile. Therefore, we anticipate that temporary cut excavation slopes that do not exceed this depth should be OSHA Type “C”, consisting of maximum slope inclinations of 1½:1 (horizontal to vertical). Temporary slopes in bedrock may be near-vertical or as advised during construction by the engineering geologist.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly, the findings and preliminary recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geosphere after a period of five (5) years has elapsed from the date of issuance of this report.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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Publications may have been used as general reference and not specifically cited in the report text.

PLATES

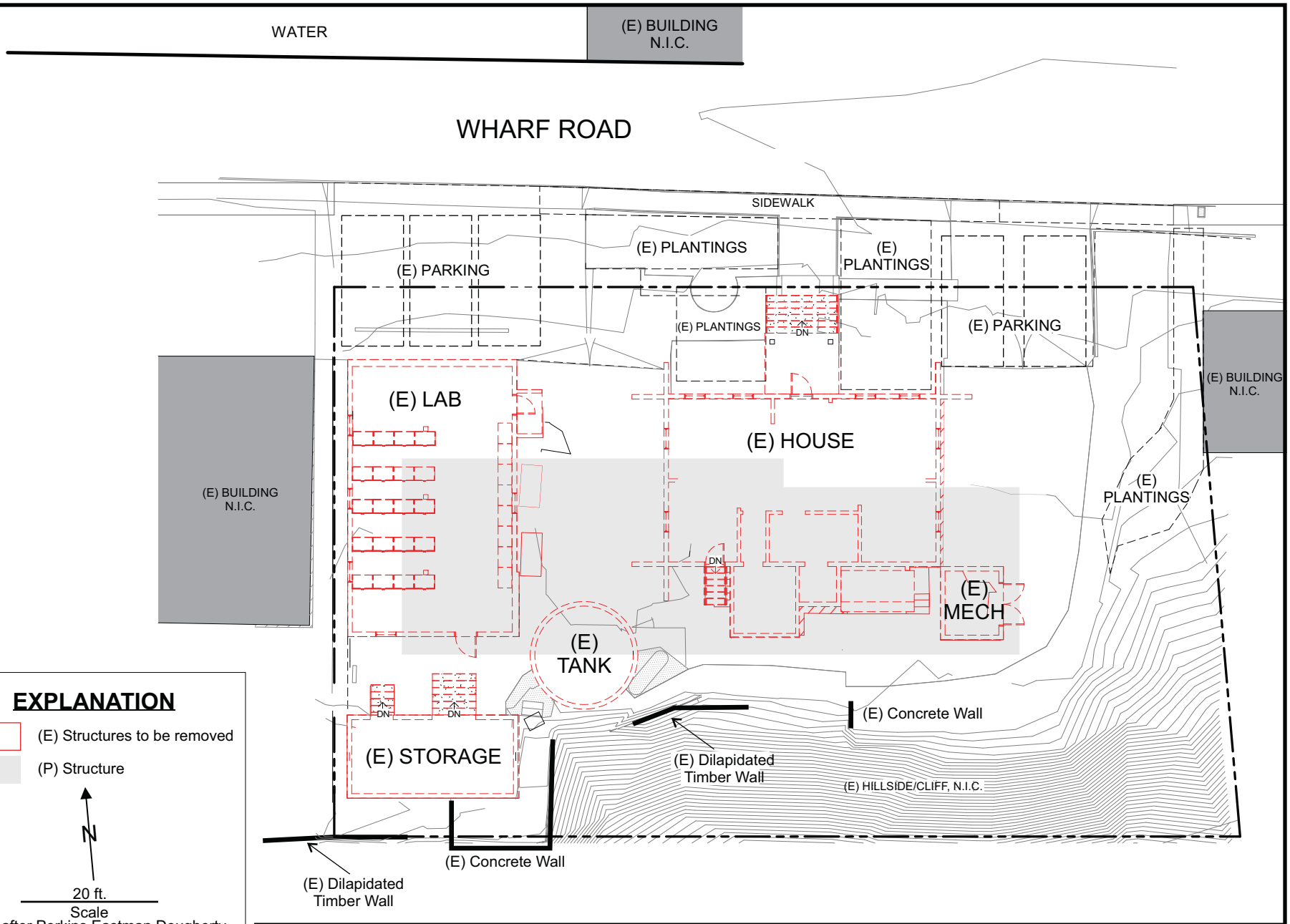
N

1000 ft.
Scale

Contour Interval = 40'

U.S.G.S. (1954)

Geosphere Consultants, Inc.	**Job No.:** 91-55185-PW **Approved:** JEB **Date:** 08.01.2020	**VICINITY MAP** Bolinas Marine Field Station | College of Marin 72 Wharf Road Bolinas, California	**Plate** 1



EXPLANATION

(E) Structures to be removed

(P) Structure

N

20 ft.
Scale

Source: 1) after Perkins Eastman Dougherty Sheet AD101, Dated 12.11.2019
2) annotations from 08.21.2020 site reconnaissance by GCI

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Approved: JEB
Date: 08.13.2020

SITE PLAN

Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California

Plate
2

Photo 1. North aerial drone view across residential development at west end of Cresente Ave. contributing concentrated runoff to slope above project site (arrow).

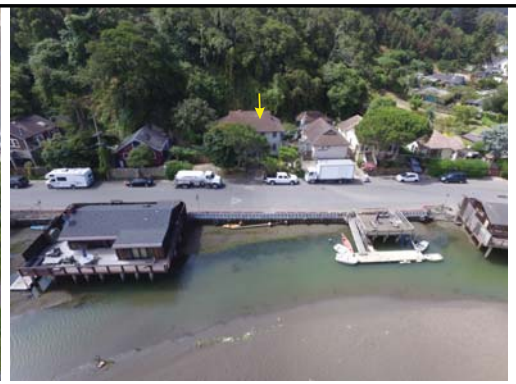

Photo 2. Southerly aerial drone view of project site from across channel on margin of Bolinas Lagoon (arrow). Note approximate 6-foot high steel sheet pile/whaler seawall support to edge of Wharf Road.

Photo 3. Westerly aerial drone view across project site. Top of cut slope approximately defined by white dashed line. Arrows are in principal swales that direct runoff and debris flows to the site.

Photo 4. Southwest aerial drone view of exposure of Merced Formation in cut slope on adjoining property.

Photo 5. Westerly aerial drone view of existing structures on project site. Arrow points to exposure of Merced Fm. comprising a landslide block detached from cut slope at the eastern swale re-entrant depicted in Photo 3 & locale of Photos 6 and 7.

Photo 6. Westerly view of pervasive rodent burrowing in erodible Clayey Silty SAND colluvium exposed in the flank of the debris slide scar at the swale re-entrant at the southeast corner of the project site.

Photo 7. Westerly view of steep, adverse joints in the Merced Fm. landslide block at the base of the cutslope depicted in Photo 5.

Note: See Plate 11 for photo line of site.

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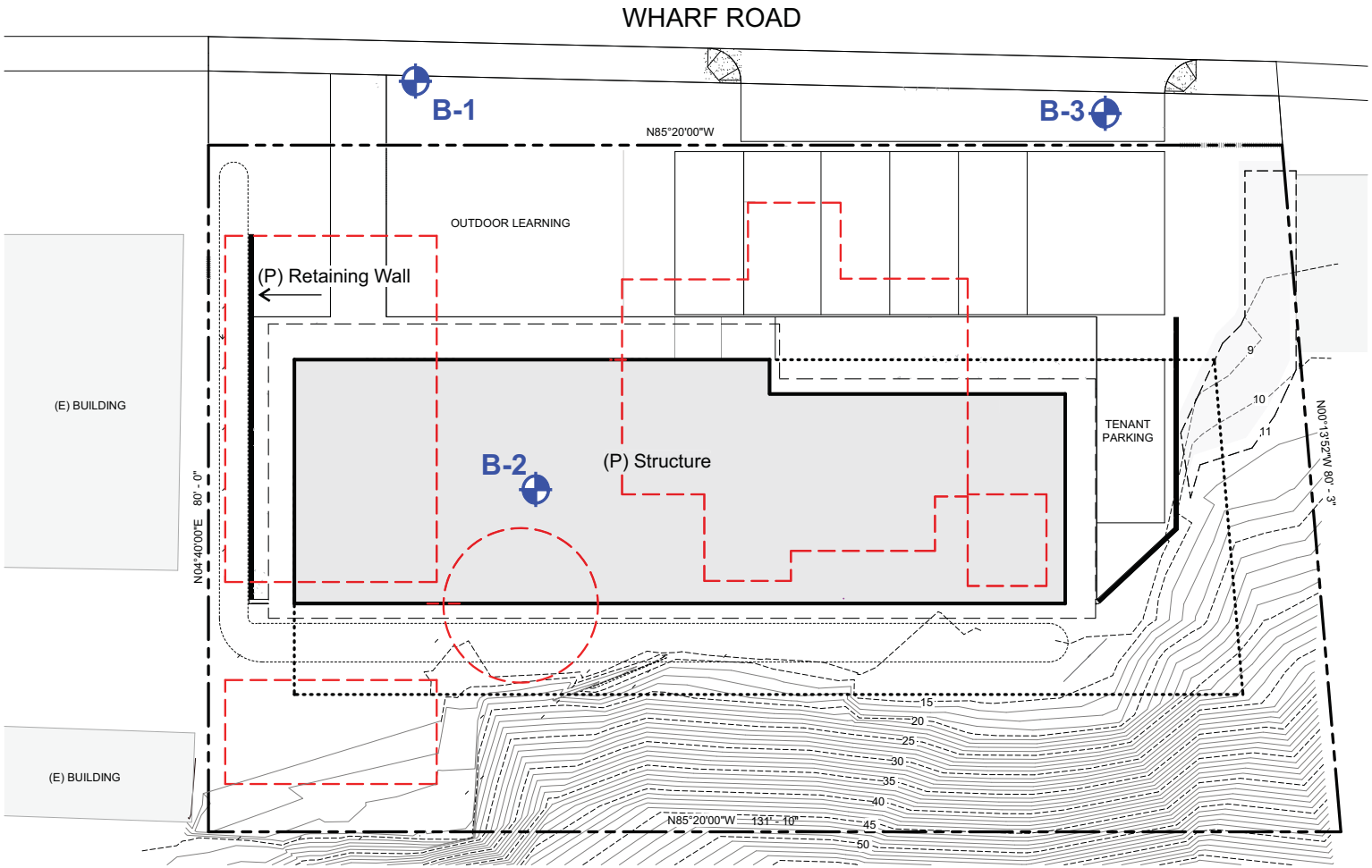

EXPLANATION

B-1 Approx. location of Boring 1

(E) Structures to be removed

(P) New Structure

N

20 ft.
Scale

Contour Interval 1ft.

Source: Perkins Eastman Dougherty
Site Plan - Zoning Requirement
Sheet G003, Dated 12.11.2019

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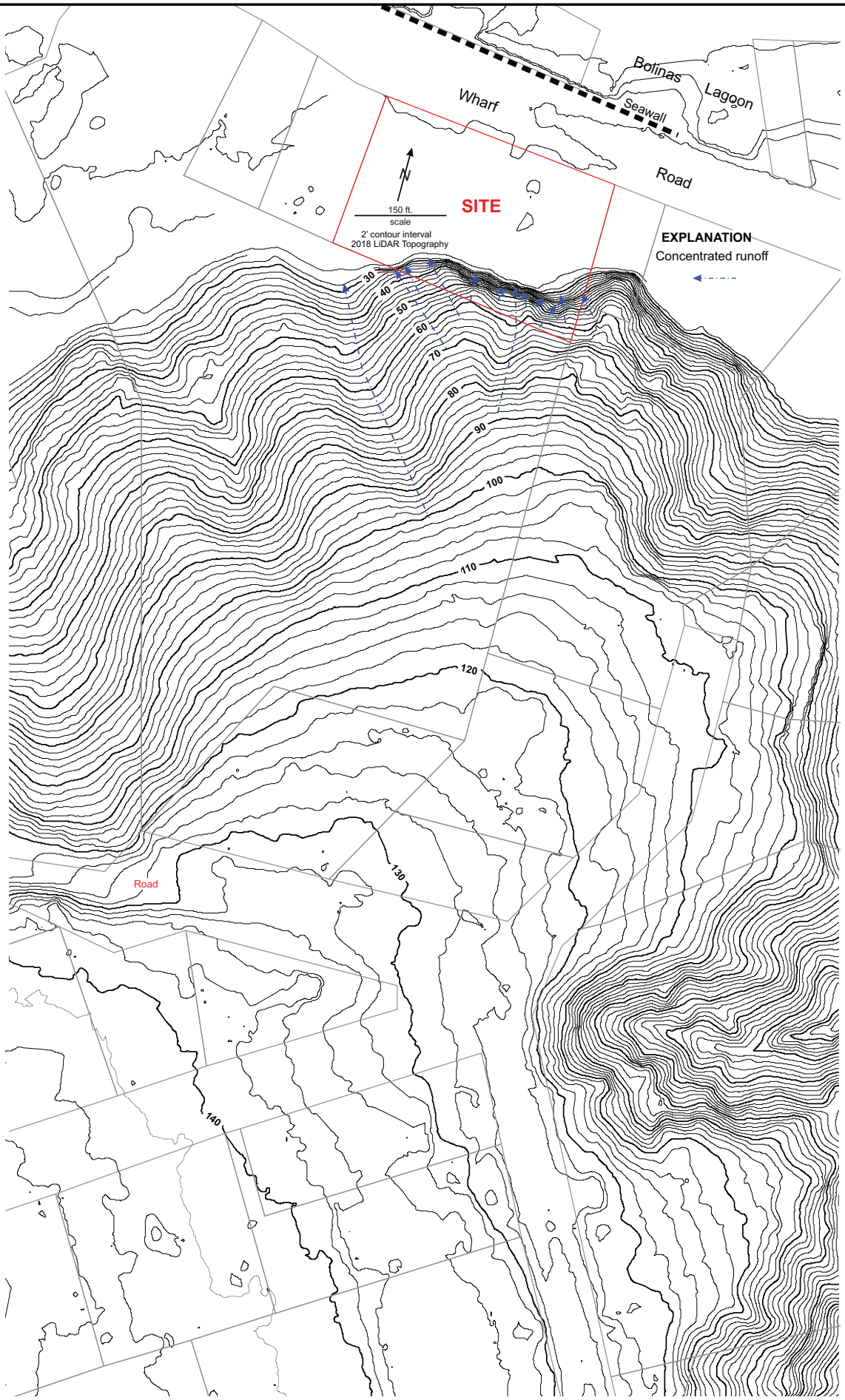
Approved: JEB
Date: 08.13.2020

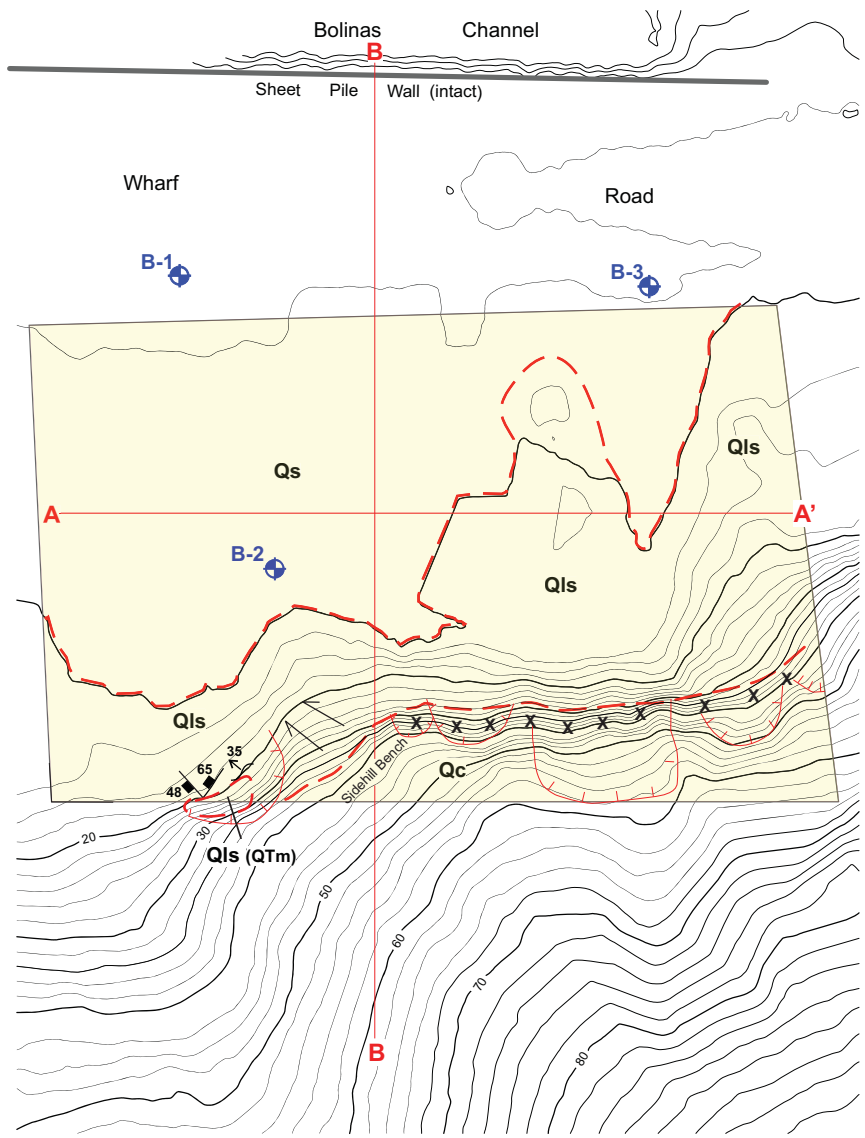
PROPOSED DEVELOPMENT

Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California

Plate
4



Geosphere Consultants, Inc	**Job No.:** 91-55182-PW **Approved:** JEB **Date:** 09.13.2020	**AREAL TOPOGRAPHY & DRAINAGE** Bolinas Marine Field Station | College of Marin 72 Wharf Road Bolinas, California	**Plate** 5



EXPLANATION

Units

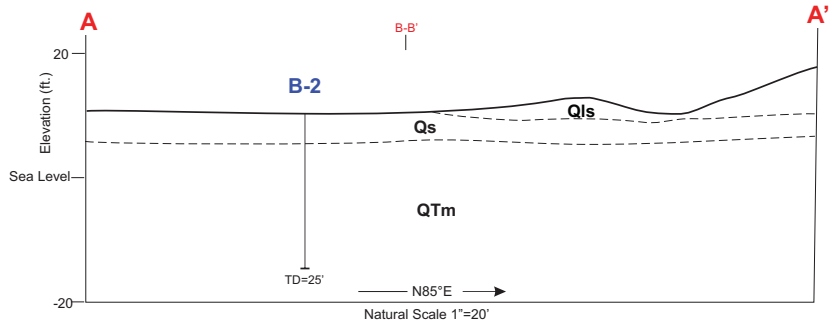
- **Qc** Colluvium
- **Qs** Undivided surficial deposits (slope wash and fill)
- **Qes** Estuarine deposit
- **QTm** Merced Formation

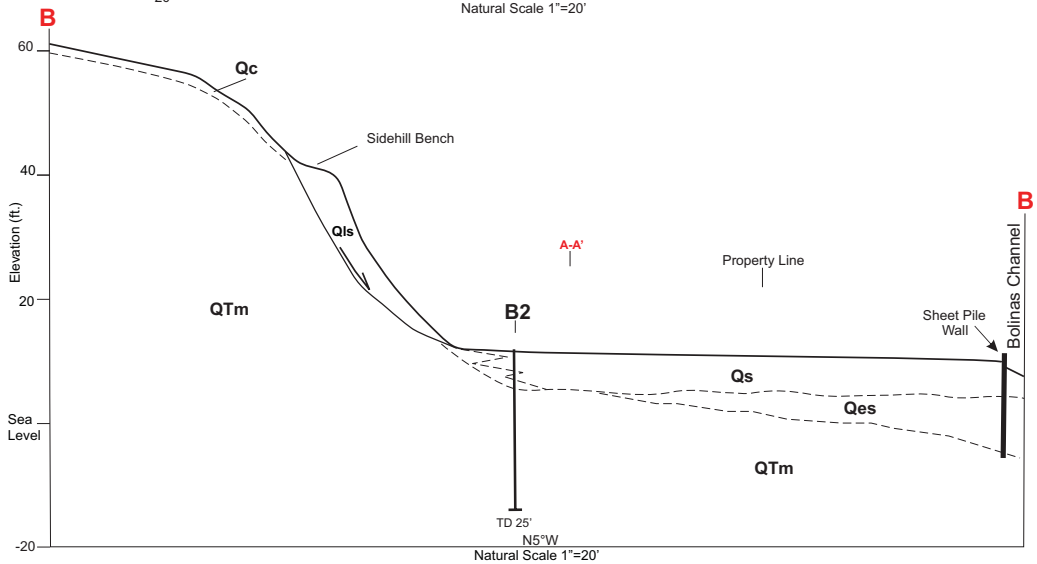
N

20 ft.
Scale
Contour Interval 2'
Base: 2018 LiDAR Topography

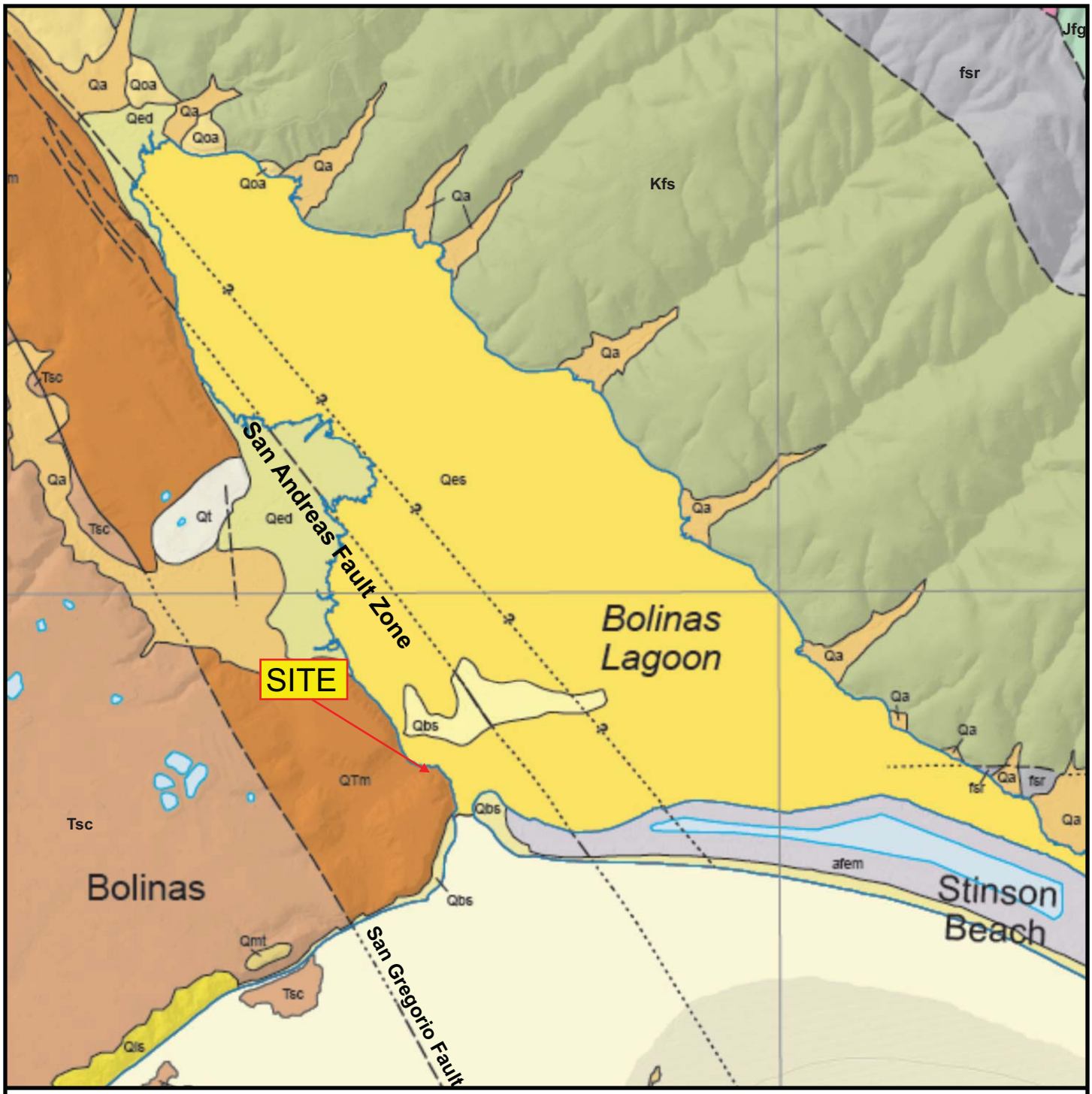
Symbols

- Geologic contact, inferred
- 65 Joint attitude
- 35 Bedrock landslide rupture surface attitude
- Surficial landslide source area scarp
- X X Rockfall/soil avalanche source area
- B-3 Approximate Boring 3 location
- A A' Line of Cross Section A-A'





Geosphere Consultants, Inc.	**Job No.:** 91-55182-PW **Approved:** JEB **Date:** 08.13.2020	**ENGINEERING GEOLOGIC MAP, CROSS SECTIONS A-A' & B-B'** Bolinas Marine Field Station | College of Marin 72 Wharf Road Bolinas, California	**Plate** 6



EXPLANATION

afem	Artificial Fill over estuarine mud (late Holocene)
Qbs	Beach Sand deposits (late Holocene)
Qes	Estuarine deposits (Holocene)
Qed	Estuarine-delta deposits (Holocene)
Qa	Alluvial deposits, undivided (Holocene)
Qoa	Older alluvium, undivided (Holocene)
Qmt	Marine Terrace deposits (Holocene)
Qt	Stream Terrace deposits (Holocene)
QTm	Merced Formation (Pleistocene & Pliocene)
Tsc	Santa Cruz Mudstone (late Miocene)
Kfs	Sandstone & Shale in San Bruno Mountain terrane
fsr	Central Belt Mélange (Cretaceous)
Jfg	Greenstone in Nicasio Reservoir terrane (Jurassic)

Contact. Solid where certain; Dashed where approximate.

Fault. Long dashed where approximate; Short dashed where inferred; Dotted where concealed; Queried where uncertain.

N

2000 ft.
Scale

Johnson and others (2015)

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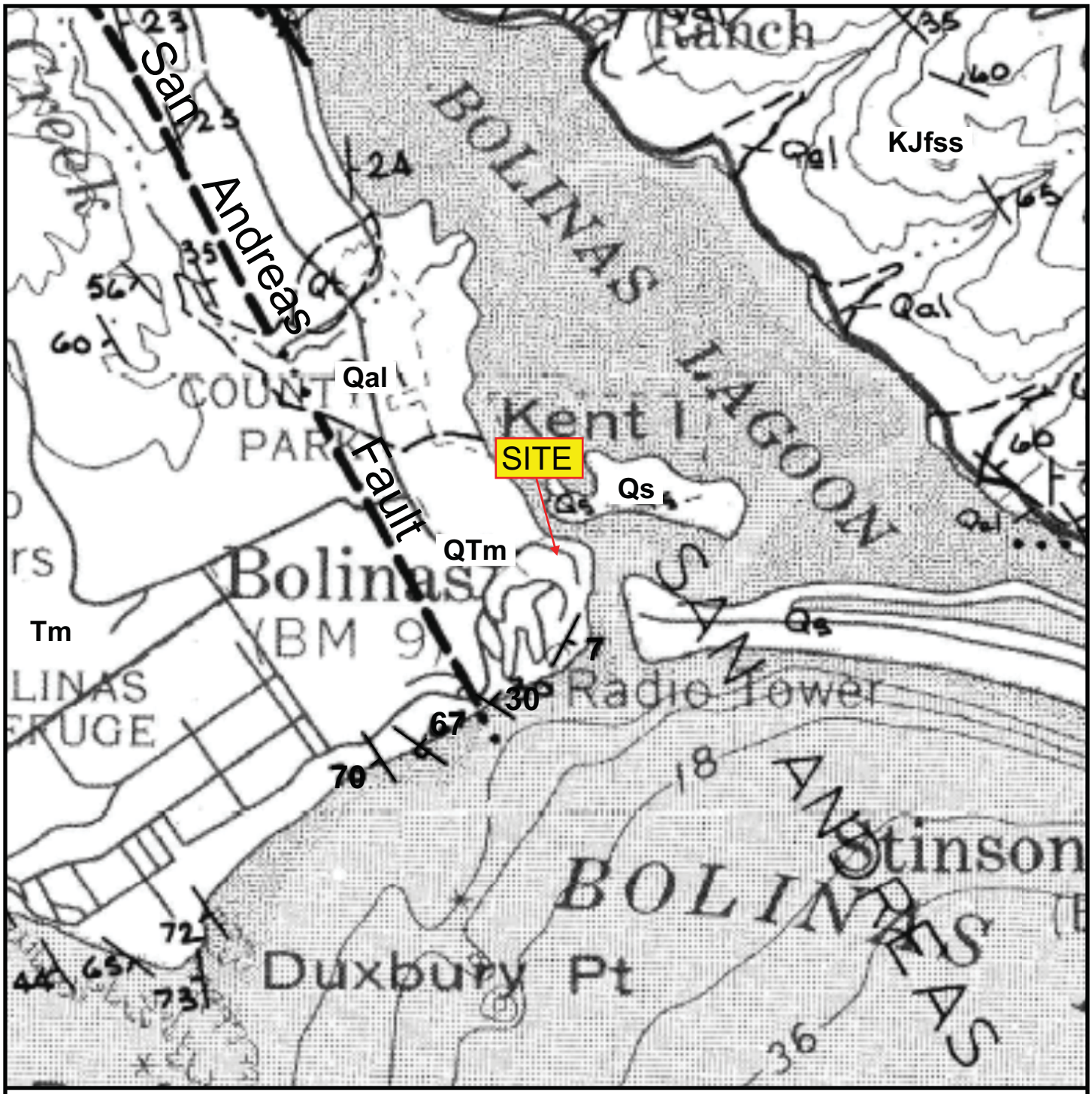
Approved: JEB
Date: 08.01.2020

REGIONAL GEOLOGIC MAP

Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California

Plate
7



EXPLANATION

Qal	Alluvium
Qs	Beach Sand
QTm	Merced Formation
Tm	Monterey Formation
KJfss	Sandstone & Interbedded Shale

N

2000 ft.
Scale

Blake and others (1974)

Contact. Dashed where approximate; Queried where uncertain. Dotted where concealed.

Fault. Dashed where approximate; Queried where uncertain. Dotted where concealed.

30 Beds inclined

67 Beds overturned

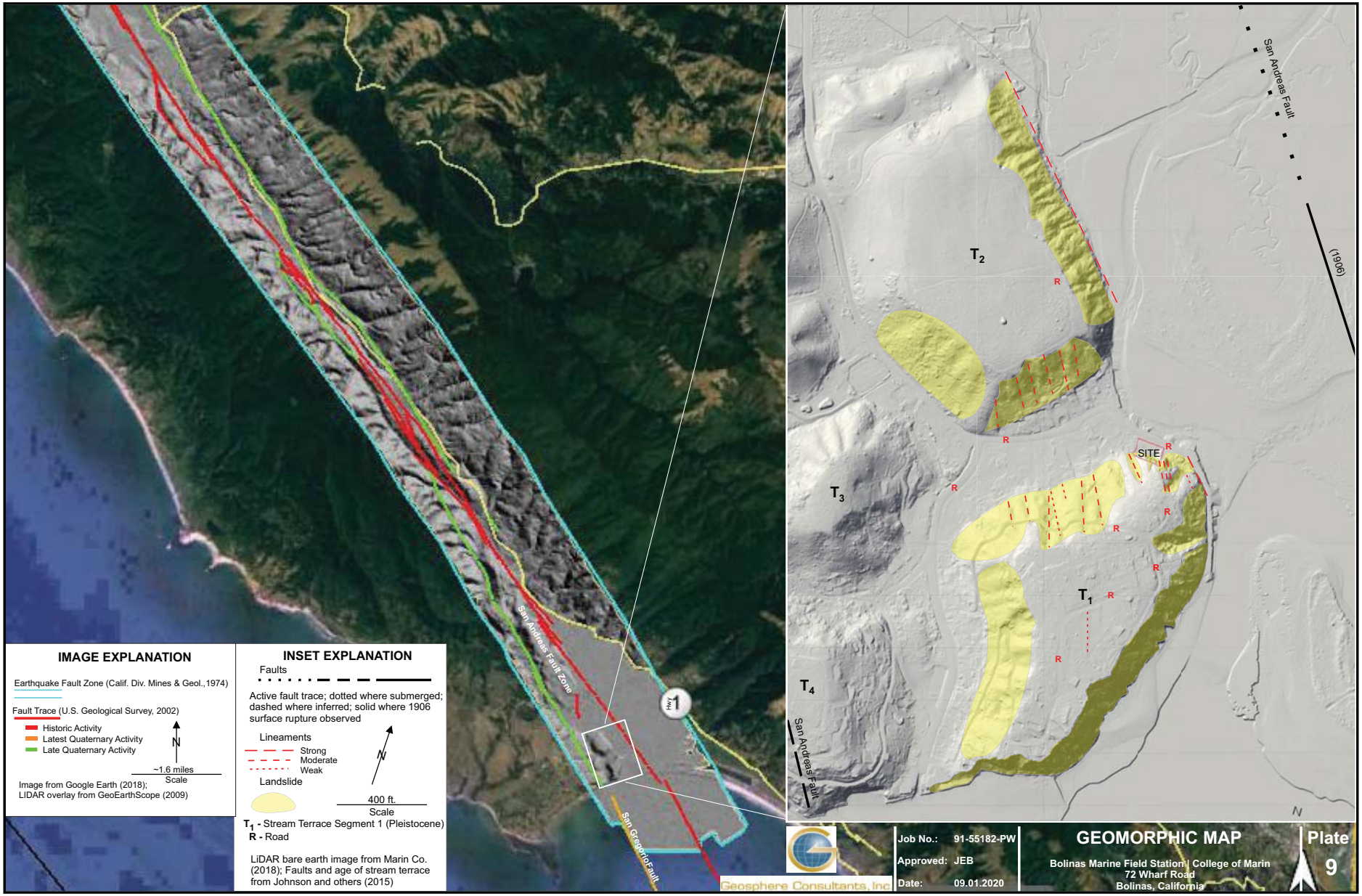
Geosphere Consultants, Inc.

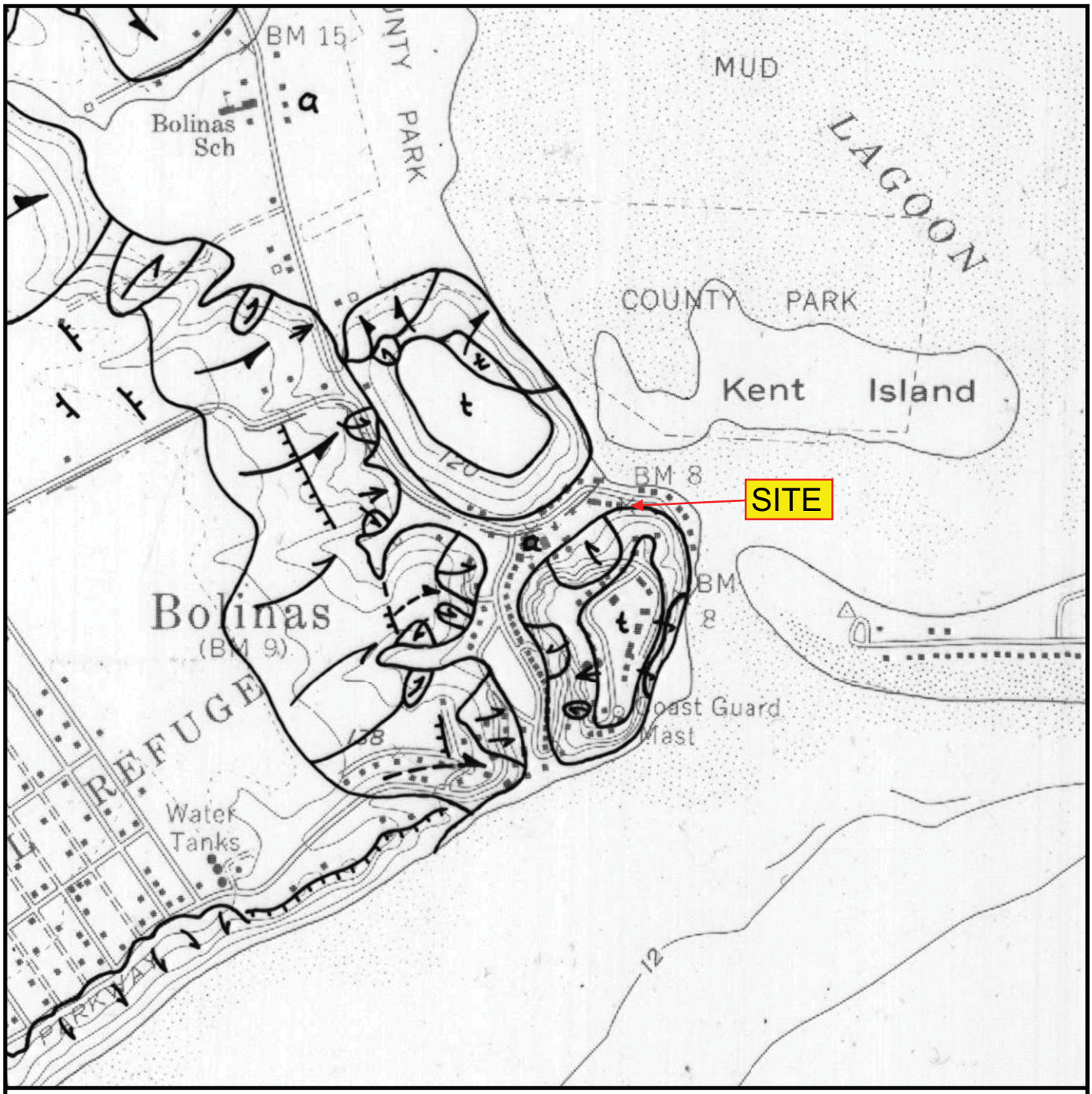
Job No.:	91-55185-PW
Approved:	JEB
Date:	08.01.2020

AREAL GEOLOGIC MAP

Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California

Plate
8


San Andreas Fault Zone
San Gregorio Fault
Hwy 1
San Andreas Fault
(1906)
T2
T3
T1
T4
SITE
R
N
IMAGE EXPLANATION
Earthquake Fault Zone (Calif. Div. Mines & Geol.,1974)
Fault Trace (U.S. Geological Survey, 2002)
Historic Activity
Latest Quaternary Activity
Late Quaternary Activity
N
~1.6 miles
Scale
Image from Google Earth (2018);
LIDAR overlay from GeoEarthScope (2009)
INSET EXPLANATION
Faults
Active fault trace; dotted where submerged; dashed where inferred; solid where 1906 surface rupture observed
Lineaments
Strong
Moderate
Weak
Landslide
N
400 ft.
Scale
T1 - Stream Terrace Segment 1 (Pleistocene)
R - Road
LiDAR bare earth image from Marin Co. (2018); Faults and age of stream terrace from Johnson and others (2015)
Geosphere Consultants, Inc.
Job No.: 91-55182-PW
Approved: JEB
Date: 09.01.2020
GEOMORPHIC MAP
Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California
Plate
9



EXPLANATION

Landslide. Identification confident to probable; queried where uncertain; movement style variable, including uncertain or indeterminate styles.

Small Landslide Deposits. Arrows indicate direction of inferred slope movement; deposits >100 ft., < 500 ft in max. dimension; confident to probable; queried where uncertain

N

1000 ft.
Scale

Wentworth and others (1975)

Block Slide. Identification confident to probable; queried where uncertain; consists of those landslides inferred to have moved downslope as relatively intact blocks.

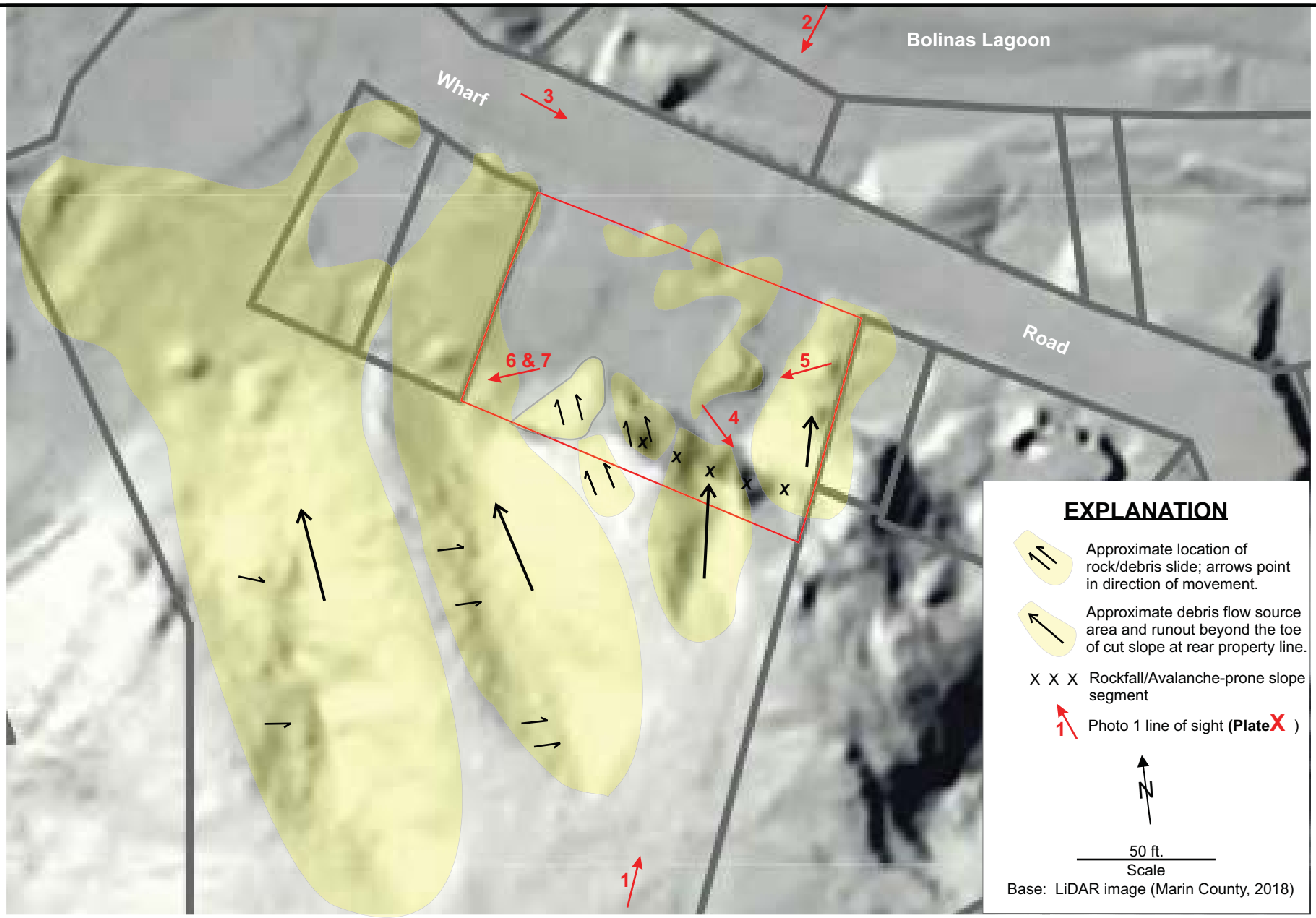
Sea Cliffs. Cliffs backing beaches or vacing open water, may produce falling rock and debris (line at top of cliff).

t

Terrace Deposits. Queried where identification uncertain; distinguished only locally.

Geosphere Consultants, Inc.

Job No.: 91-55185-PW **Approved:** JEB **Date:** 08.01.2020	**LANDSLIDE MAP** Bolinas Marine Field Station | College of Marin 72 Wharf Road Bolinas, California	**Plate** 10

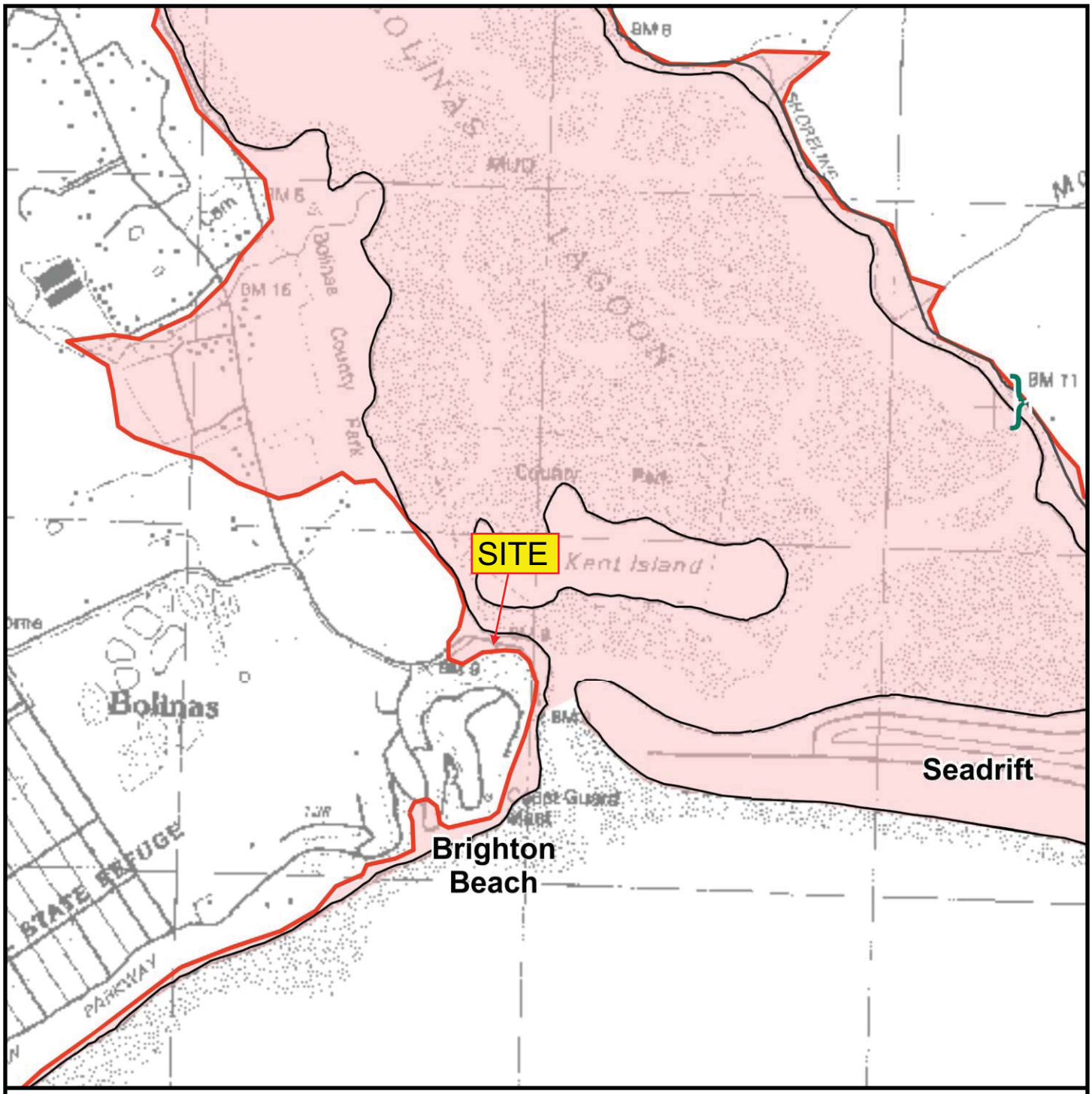


Geosphere Consultants, Inc.

Job No.: 91-55182-PW
Approved: JEB
Date: 08.13.2020

SURFICIAL LANDSLIDE MAP & KEY TO PHOTO GALLERY
Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California

Plate
11



EXPLANATION

Tsunami Inundation Line

Tsunami Inundation Area

Mean High Tide Line

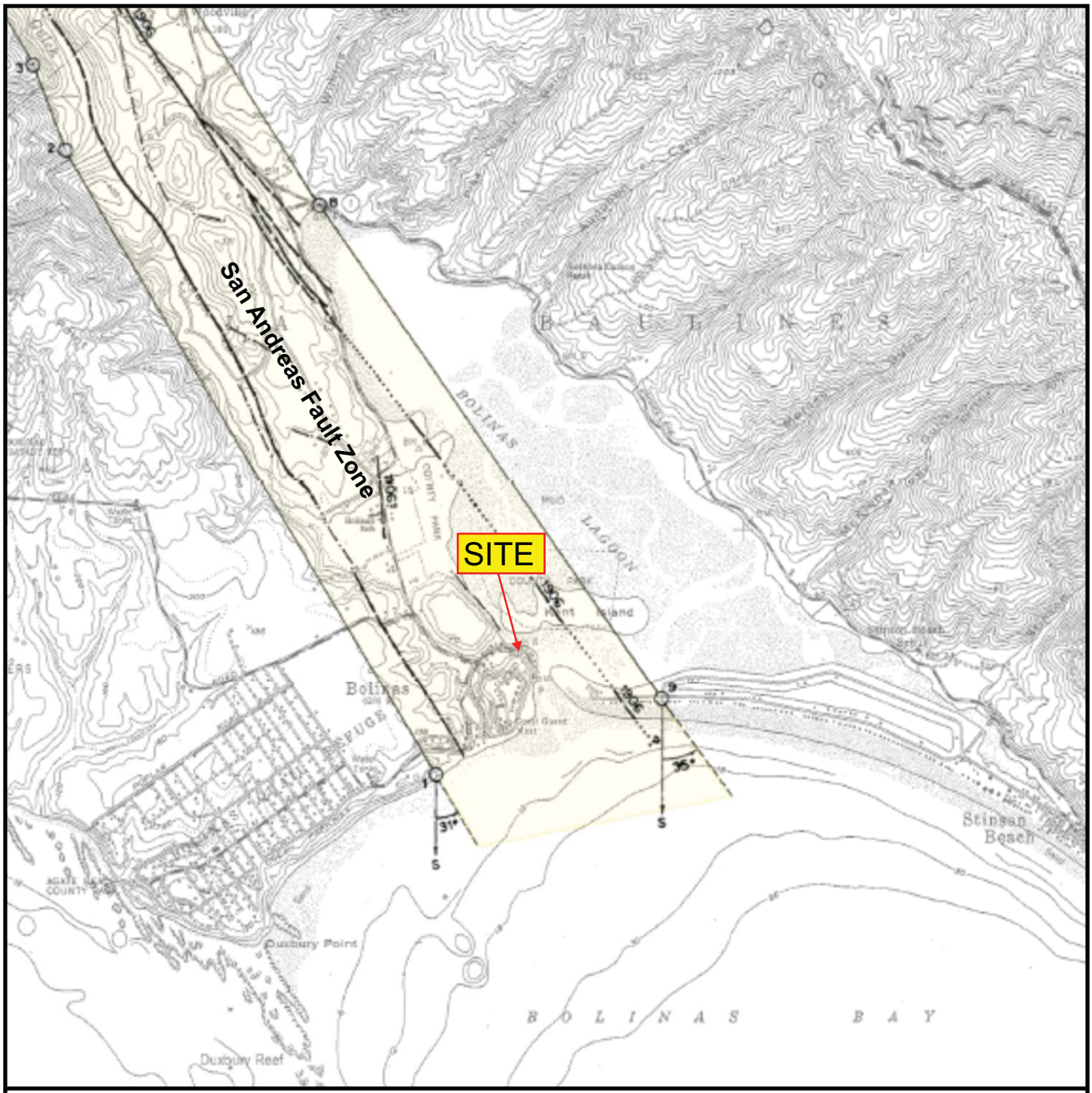
N

2000 ft.
Scale

CalEMA (2009)

Geosphere Consultants, Inc.

Job No.: 91-55185-PW **Approved:** JEB **Date:** 08.01.2020	**TSUNAMI INUNDATION MAP** Bolinas Marine Field Station | College of Marin 72 Wharf Road Bolinas, California	**Plate** 12



EXPLANATION

Contact. Dashed where approximate; Queried where uncertain. Dotted where concealed.

Fault. Dashed where approximate; Queried where uncertain. Dotted where concealed; 1906 surface rupture observed.

2 — 3 Straight-line segments defining one or more special studies zone. Numbers are turning points defining a zone segment.

N

2000 ft.
Scale

Contour Interval = 40'
California Division of Mines & Geology (1974)

Geosphere Consultants, Inc.

Job No.: 91-55185-PW
Approved: JEB
Date: 08.01.2020

EARTHQUAKE FAULT ZONES MAP

Bolinas Marine Field Station | College of Marin
72 Wharf Road
Bolinas, California

Plate
13

APPENDIX A

FIELD EXPLORATION

Key to Exploratory Boring Logs

Boring Logs

UNIFIED SOIL CLASSIFICATION (ASTM D-2487)						
Material Types	**Criteria for Assigning Soil Group Names**			**Group Symbol**	**Soil Group Names**	**Legend**
Coarse Grained Soils	Gravels	Clean Gravels	Cu≥4 and 1≤Cc≤3	GW	Well-Graded Gravel	
	>50% of	<5% Fines	Cu<4 and/or [Cc<1 or Cc>3]	GP	Poorly-Graded Gravel	
	Coarse Fraction	Gravels with Fines	Fines Classify as ML or MH	GM	Silty Gravel	
>50%	Passes on No. 4 Sieve	>12% Fines	Fines Classify as CL or CH	GC	Clayey Gravel	
Retained on	Sands	Clean Sands	Cu≥6 and 1≤Cc≤3	SW	Well-Graded Sand	
No. 200 Sieve	≥50% of	<5% Fines	Cu<6 and/or [Cc<1 or Cc>3]	SP	Poorly-Graded Sand	
	Coarse Fraction	Sands and Fines	Fines Classify as ML or MH	SM	Silty Sand	
	Passes on No. 4 Sieve	>12% Fines	Fines Classify as CL or CH	SC	Clayey Sand	
Fine Grained Soils	Silts and Clays	Inorganic	PI>7 and Plots≥"A" Line	CL	Lean Clay	
			PI<4 and Plots<"A" Line	ML	Silt	
	Liquid Limits<50	Organic	LL (Oven Dried)/LL(Not Dried <0.75)	OL	Organic Silt	
≥50% Passes	Silts and Clays	Inorganic	PI Plots≥"A" Line	CH	Fat Clay	
No. 200 Sieve			PI Plots<"A" Line	MH	Elastic Silt	
	Liquid Limits≥50	Organic	LL (Oven Dried)/LL(Not Dried <0.75)	OH	Organic Clay	
Highly Organic Soils		Primarily Organic Matter, Dark in Color and Organic Odor		PT	Peat	

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)				
SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

SOIL MOISTURE	
DESCRIPTOR	**DESCRIPTION**
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Satandard Proctor Optimum
Saturated	Free Water in Sample

PARTICLES SIZES		
COMPONENTS		**SIZE OR SIEVE NUMBER**
Boulders		Over 12 Inches
Cobbles		3 to 12 Inches
Gravels	-Coarse	3/4 to 3 Inches
	-Fine	Number 4 to 3/4 Inch
Sand	-Coarse	Number 10 to Number 4
	-Medium	Number 40 to Number 10
	-Fine	Number 200 to Number 40
Fines (Silt and Clay)		Below Number 200

Grab Bulk Sample

Standard Penetration Test

2.5 Inch Modified California

Shelby Tube

No Recovery

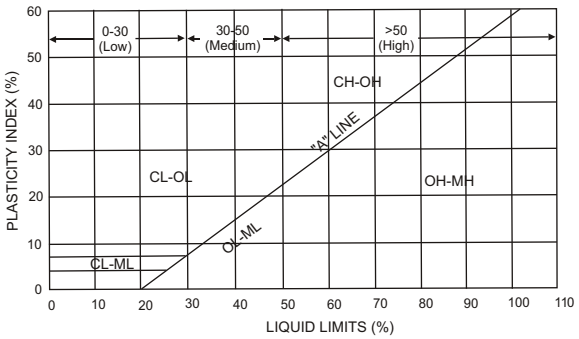
Initial Water Level Reading

Final Water Level Reading

Blow Count
The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5" indicates 50 blows recorded for 5 inches of penetration.

N-Value
Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 inch I.D) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test)

PLASTICITY CHART



CU -Consolidated Undrained triaxial test completed. Refer to laboratory results
DS - Results of Direct Shear test in terms of total cohesion (C, KSF) or effective cohesion and friction angles (C', KSF and degrees)
LL - Liquid Limit
PI - Plasticity Index
PP - Pocket Penetrometer test
TV - Torvane Shear Test results in terms of undrained shear strength (KSF)
UC - Unconfined Compression test results in terms of undrained shear strength (KSF)
#200 - Percent passing number 200 sieve
Cu - Coefficient of Uniformity
Cc - Coefficient of Concavity

General Notes

1. The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source that will be identified in the report or on the project site plan. The location and elevation of borings should be considered accurate only to the degree implied by the method used.

2. The stratification lines represent the approximate boundary between soil types. The transition may be gradual.

3. Water level readings in the drill holes were recorded at time and under conditions stated on the boring logs. This data has been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature and other factors at the time measurements were made.

4. The boring logs and attached data should only be used in accordance with the report.

ATLAS
Geosphere Consultants, Inc.
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources
— AN ATLAS COMPANY —

KEY TO EXPLORATORY BORING LOGS

BORING NUMBER B-1

ATLAS
Geosphere Consultants, Inc.
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources
— AN ATLAS COMPANY —

CLIENT Marin Community College District
PROJECT NAME College of Marin - Bolinas Marine Field Station
PROJECT NUMBER 91-55182-PW
PROJECT LOCATION 72 Wharf Road, Bolinas, CA 94924
DATE STARTED 7/13/20 **COMPLETED** 7/14/20
GROUND ELEVATION 7 ft **HOLE SIZE** 7"
DRILLING CONTRACTOR Clear Heart Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD Deeprock - DR8K
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CD
AT END OF DRILLING 2.50 ft. / Elev. 4.50 ft.
NOTES Elevations based on CSW/Stuber-Stroeh Engineering (2019).
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	ADJUSTED SPT BLOW COUNTS (N 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		**4" CONCRETE**										
		SILTY SAND (SM) Very loose, black, wet, low-medium plasticity, iron-oxide stain at the top (**Qs**).										
			MC 1-1		1-1-1 (2)	0.0	82	31	26	23	3	39
		no recovery	MC		2-3-7 (10)							
5		becomes medium dense, grey, moist, trace Gravel.	MC 1-2		4-5-5 (10)		93	26				24
		SANDY CLAY (CL)**:** Stiff, dark brown and grey, organic, highly plastic (**Qes**) TXUU @9': Su =1.23ksf	MC 1-3		2-3-4 (7)	0.50	96	27				
10												
		SANDY SILTSTONE (ROCK)**:** Soft, olive, severely to very severely weathered, massive (**QTm**).										
15			MC 1-4		12-16-18 (34)		96	25				
20		becomes grey brown and strong brown mottled	SPT 1-5		16-25-36 (61)							
25			SPT 1-6		14-26-40 (66)							

Bottom of borehole at 25.0 feet.

BORING NUMBER B-2

Geosphere Consultants, Inc.
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources
— AN ATLAS COMPANY —

CLIENT Marin Community College District
PROJECT NAME College of Marin - Bolinas Marine Field Station
PROJECT NUMBER 91-55182-PW
PROJECT LOCATION 72 Wharf Road, Bolinas, CA 94924
DATE STARTED 7/14/20 **COMPLETED** 7/14/20
GROUND ELEVATION 9 ft **HOLE SIZE** 7"
DRILLING CONTRACTOR Clear Heart Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD Deeprock - DR8K
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CD
AT END OF DRILLING 14.00 ft / Elev -5.00 ft
NOTES Elevations based on CSW/Stuber-Stroeh Engineering (2019).
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	ADJUSTED SPT BLOW COUNTS (N 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		**1" AC**										
		SILTY SAND WITH GRAVEL/FRAGMENTS (Qs)										
		SILTY SAND (SM)**:** Very loose, brown and dark brown, very moist, with clay mix, sample disturbed (**Qs**).	MC 2-1		1-0-1 (1)			24				49
		becomes loose, grey, very moist, fine sand.	MC 2-2		3-3-3 (6)	0.75	86	30				
5		**SILTSTONE** (ROCK): Soft, yellowish brown and grey with weak strong brown mottles, moist, moderately severe weathering, closely fractured, relic fabric (**QTm**)	MC 2-3		9-11-12 (23)		96	21				
10			SPT 2-4		8-11-16 (27)							
15		completely weathered, plastic, very moist	SPT 2-5		19-23-33 (56)							

Bottom of borehole at 15.0 feet.

BORING NUMBER B-3

ATLAS — Geosphere Consultants, Inc. — AN ATLAS COMPANY

CLIENT Marin Community College District
PROJECT NAME College of Marin - Bolinas Marine Field Station
PROJECT NUMBER 91-55182-PW
PROJECT LOCATION 72 Wharf Road, Bolinas, CA 94924
DATE STARTED 7/14/20 **COMPLETED** 7/14/20
GROUND ELEVATION 8 ft **HOLE SIZE** 7"
DRILLING CONTRACTOR Clear Heart Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD Deeprock - DR8K
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** CD
AT END OF DRILLING 12.00 ft / Elev -4.00 ft
NOTES Elevations based on CSW/Stuber-Stroeh Engineering (2019).
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	ADJUSTED SPT BLOW COUNTS (N 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		**3" AC**										
		CLAYEY SAND WITH GRAVEL (Qs)										
		CLAYEY SAND (SC) Loose, brown and dark brown mottled, moist.	MC 3-1		2-2-3 (5)	1.0	93	19				44
		SILTY SAND (SM): Loose, olive brown, moist to wet, organic. (**Qs**)	MC 3-2		2-1-3 (4)	1.3	91	27				
5			SH 3-3				102	23	22	23	NP	
		SILTSTONE (ROCK): Highly weathered, grey, firm, weak, moist, with orange stain (**QTm**).										
			SPT 3-4		11-13-15 (28)							
10												
			SPT 3-5		7-8-13 (21)							
15		becomes grey brown, no orange stain.	SPT 3-6		16-14-29 (43)							

Bottom of borehole at 15.0 feet.

APPENDIX B

LABORATORY TEST RESULTS

Atterberg Limits Results
Particle Size Distribution
Unconsolidated-Undrained Triaxial Test
Consolidation Test
Corrosivity Tests Summary

ATTERBERG LIMITS RESULTS

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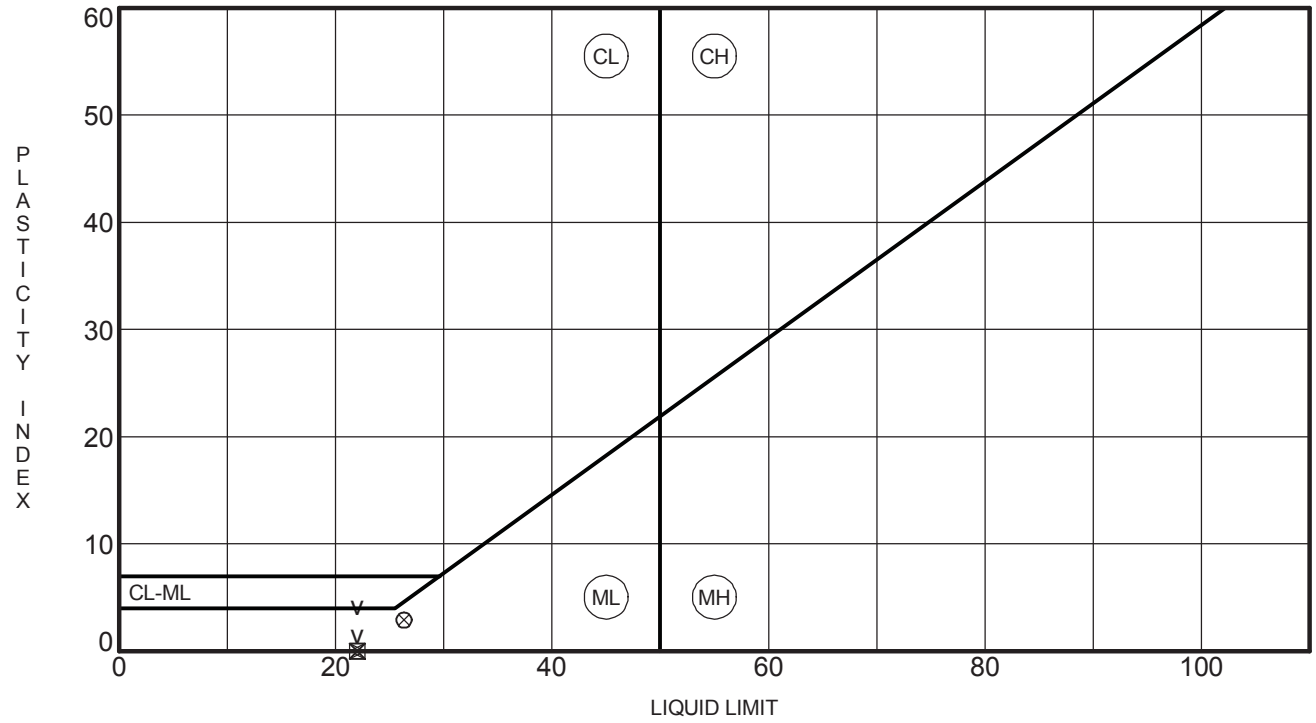
CLIENT Marin Community College District
PROJECT NAME College of Marin - Bolinas Marine Field Station
PROJECT NUMBER 91-55182-PW
PROJECT LOCATION 72 Whartf Road, Bolinas, CA 94924



	Specimen Identification		LL	PL	PI	Fines	Classification
⊗	**B-1**	**Depth: 2.0'**	**26**	**23**	**3**	**39**	**SILTY SAND (SM)**
⊠	**B-3**	**Depth: 5.0'**	**22**	**22**	**NP**		**SILTY SAND (SM)**

PARTICLE SIZE DISTRIBUTION

ATLAS
Geosphere Consultants, Inc.
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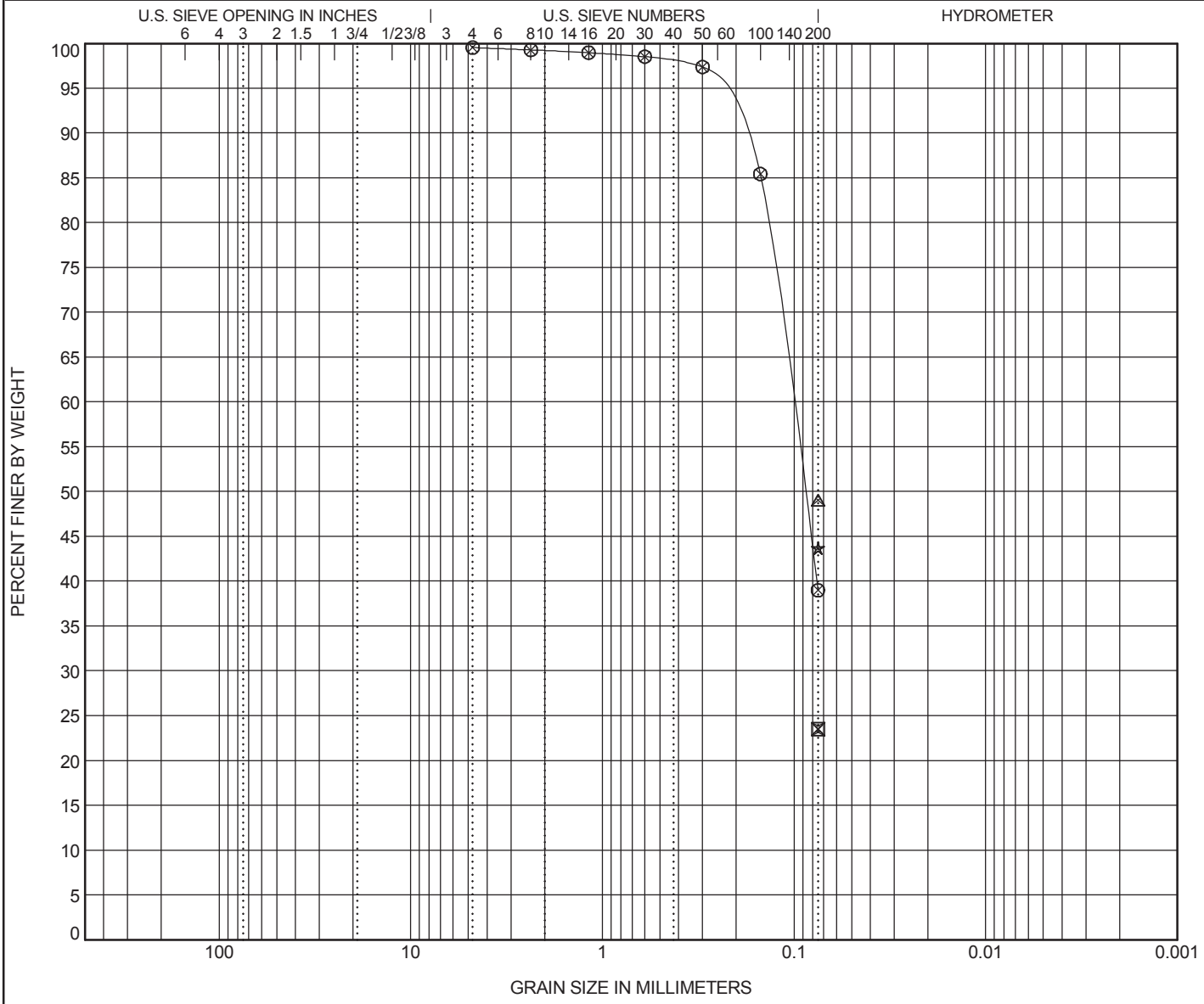
CLIENT Marin Community College District **PROJECT NAME** College of Marin - Bolinas Marine Field Station

PROJECT NUMBER 91-55182-PW **PROJECT LOCATION** 72 Whartf Road, Bolinas, CA 94924



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

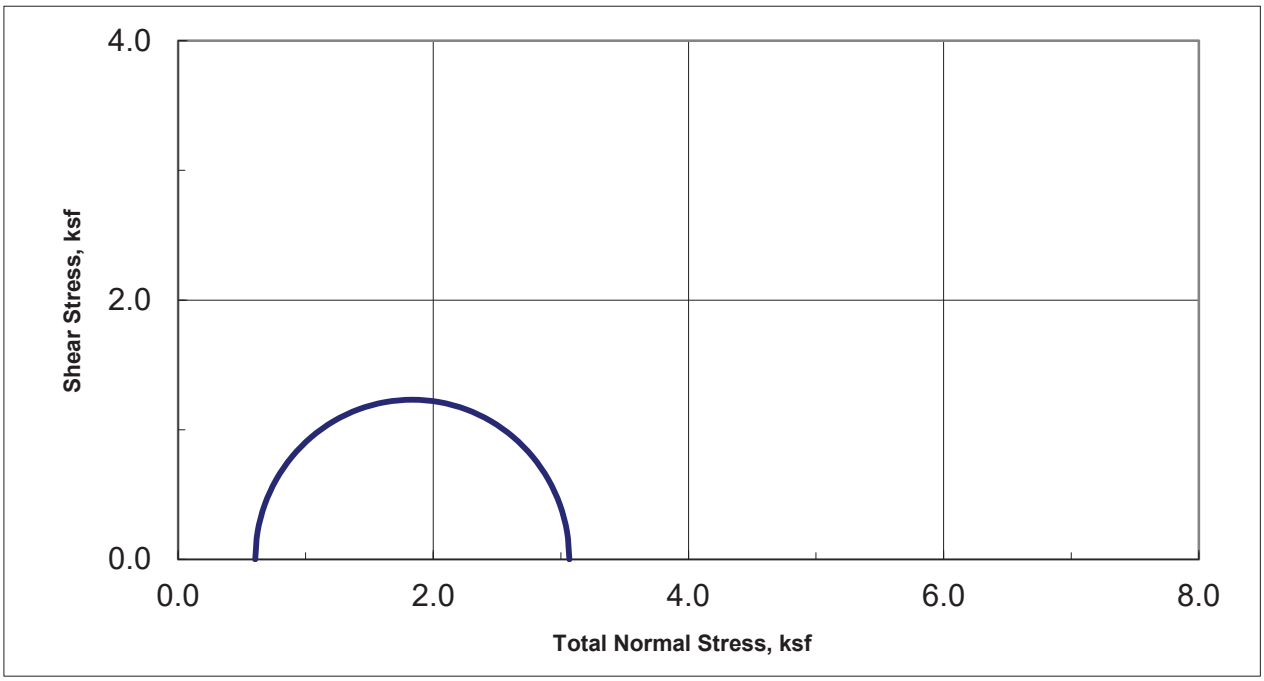
	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
⊗	**B-1**	**Depth: 2.0'**	**1.0SILTY SAND (SM)**	**26**	**23**	**3**		
⊠	**B-1**	**Depth: 6.0'**	**1.0SILTY SAND (SM)**					
△	**B-2**	**Depth: 2.5'**	**1.0SILTY SAND (SM)**					
★	**B-3**	**Depth: 2.0'**	**1.0SILTY SAND (SM)**					

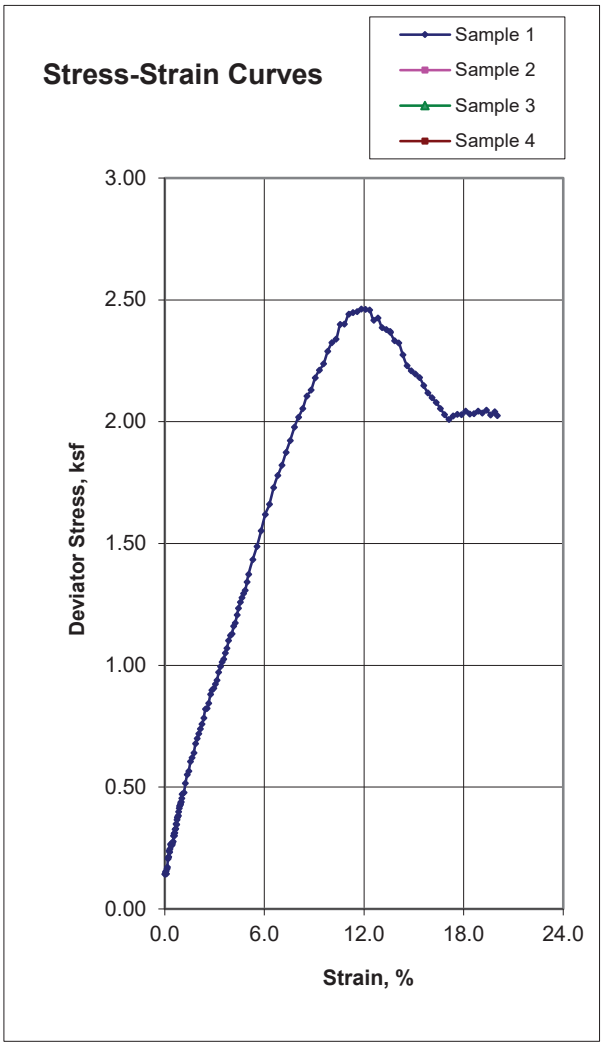
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
⊗	**B-1**	**Depth: 2.0'**	**4.75**	**0.103**				**60.6**	**39.0**	
⊠	**B-1**	**Depth: 6.0'**	**0.075**						**23.5**	
△	**B-2**	**Depth: 2.5'**	**0.075**						**49.0**	
★	**B-3**	**Depth: 2.0'**	**0.075**						**43.7**	

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	26.6			
Dry Den,pcf	96.2			
Void Ratio	0.753			
Saturation %	95.5			
Height in	4.99			
Diameter in	2.31			
Cell psi	4.2			
Strain %	11.83			
Deviator, ksf	2.463			
Rate %/min	1.00			
in/min	0.050			
Job No.:	724-232			
Client:	Geosphere Consultants			
Project:	91-55182-PW			
Boring:	B1-3			
Sample:				
Depth ft:	9			

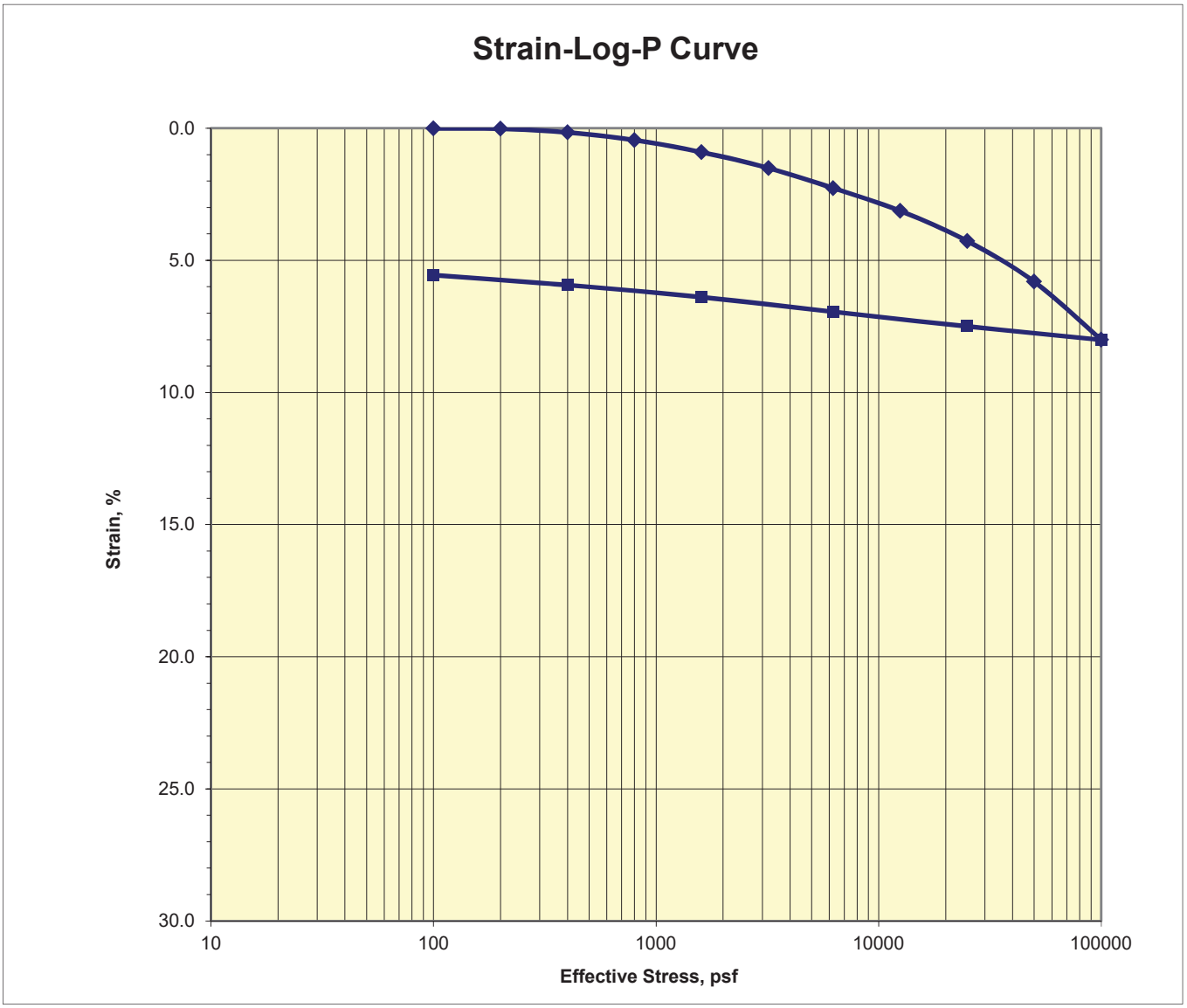
Visual Soil Description	
Sample #	
1	Dark Olive Gray Sandy CLAY
2	
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

Consolidation Test
ASTM D2435

Job No.:	724-232	**Boring:**	B3-5	**Run By:**	MD
Client:	Geosphere Consultants	**Sample:**		**Reduced:**	PJ
Project:	91-55182-PW	**Depth, ft.:**	5(Tip-16")	**Checked:**	PJ/DC
Soil Type:	Olive Brown Silty SAND			**Date:**	8/6/2020



Assumed Gs 2.7	Initial	Final
Moisture %:	23.2	21.2
Dry Density, pcf:	101.5	107.2
Void Ratio:	0.661	0.573
% Saturation:	94.6	100.0

Remarks:

Corrosivity Tests Summary

CTL # 724-231 **Date:** 7/21/2020 **Tested By:** PJ **Checked:** PJ
Client: Geosphere Consultants **Project:** Bolinas Field Lab Station **Proj. No:** 91-55182-PW
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP (Redox)		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%				Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
1	-	1-3	-	-	3,554	73	402	0.0402	5.6	205	23	Positive	35.2	Black Sandy CLAY