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**UPDATED GEOLOGIC AND GEOTECHNICAL INVESTIGATION**
**88 VISION ROAD (APN 112-141-03/04)**
**INVERNESS, CALIFORNIA**

March 4, 2021

Job No. 2626.002

Prepared For:
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CERTIFICATION

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## 1.0 INTRODUCTION

This report presents the results of our Phase 1 Geologic and Geotechnical Investigation for the planned new residence at an undeveloped site (APN 112-141-03/04) on Vision Road in Inverness, California. The site is located in the "second valley" of Inverness, just east of the existing residence at #92 Vision Road. A Site Location Map is shown on Figure 1.We previously performed a Geologic and Geotechnical Investigation on behalf of a previous owner in consideration of a similar scope of work, as summarized in our report dated May 22, 2019.

Our current work has been performed in accordance with our Agreement for Professional Services dated February 22, 2021. The purpose of our services is to review existing site conditions and the current proposed scope of work, review the results of our previous Investigation at the site, and to provide supplemental/updated geotechnical recommendations and design criteria as warranted. The scope of our services is outlined in our proposal letter dated February 21, 2021 and include the following:

- Performance of a site reconnaissance to observe existing conditions;
- Review of our previous Geologic and Geotechnical Investigation;
- Development of updated seismic design criteria in accordance with the 2019 California Building Code, along with supplemental/updated geotechnical recommendations specific to the currently-proposed project; and
- Preparation of this Updated Geotechnical Investigation report.

Issuance of this report completes the scope of our Phase 1 services. Future phases of work are anticipated to include Geotechnical Consultation and Plan Review (Phase 2) and Geotechnical Observation/Testing during construction (Phase 3).

## 2.0 PROJECT DESCRIPTION

The project site consists of an irregularly-shaped, approximately 1.0-acre property comprised of 2 separate assessor's parcels along the north side of Vision Road, just east of the exiting residence at #92 Vision. Based on review of current plans, proposed improvements include a new two-story, single-family residence sited in the central part of the property. Access will be provided by a new circular driveway extending up from the Vision Road frontage. Ancillary improvements will include a new septic system in the western part of the property, new exterior decks and flatwork areas, an improved and/or re-aligned driveway, underground utilities, site drainage, and other typical residential items. A Site Plan showing the approximate extents of the planned improvements is presented on Figure 2.

Preliminary plans indicate the new residence will be supported on a cast-in-place pier and grade beam foundation system, and may utilize retaining walls up to 6- or 8-feet high to accommodate a "daylight" lower floor space in combination with a sloping crawl space. As such, moderate grading is expected for the project, primarily consisting of excavations up to 8- or 10-feet deep.

## 3.0 SITE CONDITIONS

### 3.1 Regional Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

The oldest rocks in the region are the sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age (190- to 65-million years old) Franciscan Complex. Within Marin County, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age locally overlie the basement rocks of the Franciscan Complex. Tectonic deformation and erosion during late Tertiary and Quaternary time (the last several million years) formed the prominent coastal ridges and intervening valleys typical of the Coast Ranges province. The youngest geologic units in the region are Quaternary age (last 1.8 million years) sedimentary deposits, including alluvial deposits which partially fill most of the valleys and colluvial deposits which typically blanket the lower portions of surrounding slopes.

Regional geologic mapping (California Division of Mines and Geology, 1977) indicates that the majority of the site is underlain by granitic rock of Cretaceous age (about 65-million years old). A contact juxtaposing granite to the northwest against Holocene-age (less than 11,000-years old) alluvial deposits to the southeast is shown nearly coincident with the edge of Vision Road along the southeastern property line. Also shown is an active (secondary) trace of the San Andreas Fault which produced documented surface rupture during the 1906 earthquake. The fault trace trends roughly north-northeast and passes about 400-feet west of the site. The main trace of the San Andreas Fault is located about 1,000-feet east of the site, beneath Tomales Bay. A Regional Geologic Map is shown on Figure 3.

### 3.2 Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

### 3.2.1 Regional Active Faults

The California Division of Mines and Geology (1998) has mapped various active and inactive faults in the region. These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. For the purpose of this and all projects regulated under the Alquist-Priolo Act, an "active" fault is defined as one that has ruptured within Holocene time (the last 11,000 years). The nearest known active fault to the site are the San Andreas and San Gregorio Faults. As noted above, the nearest mapped active trace of the San Andreas Fault is located approximately 0.12 kilometers (400 feet) west of the site, while the main trace of the San Andreas Fault is located beneath Tomales Bay, about 0.3 kilometers (1,000-feet) east of the site.

### 3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that at least 6 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers (62 miles) of the site between 1900 and 2021. These earthquakes are summarized in Table 1.

**Table 1 – Significant Historic Earthquake Activity**

| Epicenter (Latitude, Longitude) | Historic Richter Magnitude | Year | Approximate Distance (km) |
|---|---|---|---|
| 38.06°N, -122.40°W | 7.7 | 1906 | 42(2) |
| 38.22°N, -122.31°W | 6.0 | 2014 | 51 |
| 37.85°N, -121.82°W | 5.8 | 1980 | 97 |
| 37.56°N, -122.72°W | 5.7 | 1957 | 62 |
| 38.38°N, -122.41°W | 5.0 | 2000 | 51 |
| 38.82°N, -122.84°W | 5.0 | 2014 | 80 |

(1) Reference: USGS Circular Area Earthquake Search Catalogue, accessed September 4, 2018.
(2) Surface rupture occurred within about 0.12 kilometers (400 feet) of the site, as noted on Figure 3.

### 3.2.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (2003, 2008; Field et al, 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

The 2003 study (UCERF) specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. The 2008 study (UCERF2) applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models. The most recent 2015 study (UCERF3) further expanded the database of faults considered and allowed for consideration of multi-fault ruptures, among other improvements.

Conclusions from the most recent UCERF3 indicate the highest probability of an earthquake with a magnitude greater than 6.7 on any of the active faults in the San Francisco Bay region by 2045 is assigned to the Rodgers Creek Fault, located approximately 33 kilometers northeast of the site, at 33%. The San Andreas Fault, located within 400-feet of the site, is assigned a probability of 22%. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

### 3.3 Surface Conditions

We performed a site reconnaissance on February 25, 2021 to observe and document existing conditions at the site. The project site is composed of 2 adjacent Assessor's parcels which together form an irregularly-shaped area of about 1.0-acre. The site is bounded to the southeast by Vision Road, and elsewhere by existing semi-rural residential development typical of West Marin. The site generally slopes steeply to the south-southeast at average inclinations of about 3:1 (H:V), with surface elevations ranging from about +25-feet along the Vision Road frontage to over +100-feet in the northeast, upland part of the property[1]. Aside from an existing graded driveway and small wooden retaining wall, the site is essentially undeveloped.

During our reconnaissance, we did not observe any rock outcrops within the site, and surface soils were noted to consist of loose to medium-dense sand and silty sand (decomposed granite). Limited exposures of completely-weathered granite were noted in steep cut slopes along the north side of Vision Road to the east and west of the site, but are largely obscured by heavy vegetation. We also noted the presence of several deeply-incised drainage channels, flowing roughly to the southeast, which emanate onto the valley floor at the approximate location of the planned residence. We noted channel banks up to about 6-feet high and inclined near-vertical, and observed slightly hummocky topography at the base of the slope indicative of possible historic debris flows.

### 3.4 Subsurface Exploration

Subsurface exploration for the project included performance of 5 soil borings, drilled at the approximate locations shown on Figure 2 on April 20, 2018. Our borings were excavated to maximum explored depths between about 7.5- and 18-feet below the ground surface by use of a portable hydraulic-powered drill rig equipped with 4-inch solid-stem augers. Borings were logged by our Engineering Geologist and samples were collected at select intervals for further examination. Brief descriptions of the terms and methodology used in classifying earth materials are provided on the Soil and Rock Classification Charts, Figures A-1 and A-2, respectively, and the Boring Logs are shown on Figures A-3 through A-7.

[1] Surface elevations taken from Marin County GIS (www.marinmap.org) and are based on NAVD88.

In addition to soil borings, we also excavated and logged an exploratory fault trench at the site in September of 2018. Our trench extended to maximum depths of about 5-feet below the ground surface for a distance of 102-feet along the Vision Road frontage as shown on Figure 2. The results of our fault trench exploration are discussed in greater detail in Section 4.2 of this report, and our Fault Trench Logs are shown in Appendix B.

### 3.5 Subsurface Conditions and Groundwater

Our subsurface exploration generally confirms the mapped geologic conditions at the site (California Division of Mines and Geology, 1977). Based on our subsurface exploration, the site is generally underlain by a thin mantle of silty to sandy residual soil (decomposed granite) over shallow granitic rock. Borings 1 and 2, drilled on the slope at the approximate location of the potential eastern footprint for the planned residence, each encountered about 5-feet of residual soil/decomposed granite composed of loose sand with silt. Each boring encountered highly to completely weathered granite bedrock at a depth of about 5-feet, which generally graded harder and stronger. Borings 1 and 2 were terminated at depths of 8.5 and 8.75-feet, respectively.

Borings 3 through 5 were drilled along the Vision Road frontage in the south-central and southeast part of the property. These borings each encountered a surface layer of clayey sand alluvium/colluvium which was underlain by weathered granite bedrock at depths of 5- and 7-feet in Borings 3 and 4, respectively. Boring 5, drilled in the southeast corner of the property, encountered about 12-feet of alluvial soils over decomposed granite. No weathered bedrock was encountered in the upper 17-feet of the subsurface in Boring 5.

Groundwater was encountered in Borings 3, 4 and 5 at about 3 feet below ground surface. Minor groundwater seepage was encountered near the soil/bedrock contact in the fault trench excavation. Because the borings and fault trench were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Based on the site's location at the base of steeply-sloping areas underlain by relatively pervious weathered granite, we anticipate groundwater may exist within 3-feet of the ground surface throughout the year, and could be higher during the winter months, particularly in the lower-lying parts of the site.

## 4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards, discusses their potential impacts on the planned improvements, and identifies proposed remedial options. The primary geologic hazards which could affect the proposed development are strong seismic ground shaking, erosion, and slope instability. Other geologic hazards are judged relatively insignificant with regard to the proposed project. Each geologic hazard considered is discussed in further detail in the following paragraphs.

### 4.1 Fault Surface Rupture

The provisions of the Alquist-Priolo Earthquake Fault Zoning Act ("A-P Act", 1972) and the current edition of the California Building Code (2016) dictate that structures intended for human occupancy (more than 2,000 hours per year) may not be developed within 50-feet of an active fault trace unless a smaller setback can be justified by appropriate geologic evidence. For the purpose of the A-P Act, an "active" fault is defined as one that has ruptured within Holocene time (the last 11,000 years). Typically, direct observation via logging trench excavations is the most effective means through which to determine whether or not, A) a fault trace is present, B) if any identified fault trace should be considered "active", and C) whether the proposed project is therefore subject to restriction under the auspices of the A-P Act.

In addition to restricting development near active faults, the A-P Act requires the California Geological Survey to publish maps delineating Earthquake Fault Zones ("APEFZs") within which Fault Trench Investigations are required prior to review of development applications. Based on review of the applicable Alquist-Priolo map for the Inverness Quadrangle (California Division of Mines and Geology, 1974), the project site lies entirely within the APEFZ associated with an active secondary trace of the San Andreas Fault, as shown on Figure 5. While not mapped at a scale accurate enough to be applied on a site-specific basis, the A-P map indicates that the surface trace of the secondary fault passes about 400-feet west of the site along a trend of approximately N20°W, and that surface rupture was documented and photographed in the field following the 1906 event.

In order to evaluate the existence of active faults proximal to the proposed building envelope, we excavated a 102-foot-long trench to a maximum depth of about 5-feet at the location shown on Figure 2. The trench was located to provide evaluation of potential faults within a previously-proposed building envelope and extending 25-feet beyond each end – note that the current building enveloped has been shifted about 30-feetfarther west. The trench was excavated and the sidewalls cleaned with hand tools on September 15, 2018, and the trench walls were examined on September 16, 2018. Also, on September 16 surface topography was surveyed using a hand level, stations were marked at 5-foot lateral intervals, and the north trench wall was logged at a scale of 1 inch = 5-feet. Our Exploratory Fault Trench Log is shown along with a Stratigraphic Column and description of soil units on Figures B-1 and B-2. The trench was backfilled with compacted native soils on September 17, 2018.

Soils exposed in our fault trench generally consisted of silty to clayey sand alluvium/colluvium underlain by completely weathered (decomposed) granite. In the western portion of the trench, modern silty sand colluvium rests directly on the weathered granite. East of Station 0+22, silty

and clayey sand alluvium, deposited by southeast-flowing channels which emanate from the slopes east of the planned building area overlie the colluvium and underlying bedrock. Since the trench was excavated essentially orthogonal to the channel alignment, the trench wall provides a cross-sectional view of the channel. During our trenching, we did not observe any evidence of faulting, such as offset soil strata, sheared material, slickens, or other features indicative of previous fault activity.

During follow-up reconnaissance visits in 2018 and 2020 we observed an exposure of very hard brecciated granite, consisting of angular, cobble-size fragments of granite within a dark brown, highly sheared and recrystallized matrix, on the north side of Vision Road about 400-feet east of the site. This exposure had been obscured during our previous reconnaissance by heavy vegetation and is located almost exactly where the 1906 rupture trace has been previously mapped by others as shown on Figure 3.

Therefore, on the basis of our fault trench and follow-up reconnaissance observations, we conclude that the 1906 rupture trace is likely mapped accurately by others as shown on Figure 3, and that no active faults exist within the majority of the current building envelope. As noted above, the proposed structural footprint has been shifted about 30-feet to the west, such that the exploratory trench stops about 5-feet east of the western edge of the building. However, based on our trench observations (specifically a lack of noted deformation, chemical alteration, or other features indicative of nearby/immediately proximal faulting), site and vicinity reconnaissance observations, and experience with previous active fault Investigations in the greater Bay Area, we judge the likelihood that active faults exist within the building envelope is very low, and the likelihood of active faults lying within 25-feet of the west end of the proposed building is generally low. Therefore, the risk of damage to the planned residence due to surface fault rupture is judged to be low.

*Evaluation:* *No significant impact.*
*Recommendations:* *No remedial measures required.*

### 4.2 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site specific geologic conditions. Given the proximity of the site to the San Andreas Fault, high levels of ground shaking should be anticipated. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations that provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table 2.

**Table 2 – Estimated Peak Ground Accelerations for Principal Active Faults**

| Fault | Moment Magnitude for Characteristic Earthquake[1] | Closest Estimated Distance (km)[1] | Median Peak Ground Acceleration (g)[1] |
|---|---|---|---|
| San Andreas | 8.0 | 0.55[2] | 0.58 |
| San Gregorio | 7.4 | 28 | 0.14 |
| Rodgers Creek | 7.3 | 33 | 0.12 |
| Hayward | 7.3 | 42 | 0.10 |
| Maacama | 7.4 | 50 | 0.08 |

1) Caltrans ARS Online v2.3.09, accessed on September 6, 2018.
2) Note distance generated reflects approximate distance to main trace of fault. As previously discussed, surface rupture occurred within 400-feet of the site within 1906; however, this secondary trace is not expected to have significant seismogenic potential as regards generation of strong seismic ground waves.

The calculated bedrock accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements, such as light fixtures, shelves, cornices, etc., to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to its proximity and historic rate of activity, the San Andreas Fault presents the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

*Evaluation:* *Less than significant with remedial measures.*
*Recommendations:* *Minimum measures include design of new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.*

### 4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. This phenomenon can occur in saturated, loose, granular deposits subjected to seismic shaking. Liquefaction can result in flow failure, lateral spreading, settlement, and other related effects. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with relatively high fines content (i.e., containing 35 to 50 percent clay and silt particles that pass the #200 sieve) provided the fines exhibit a plasticity index less than 7.

The results of our subsurface exploration indicate that the planned building envelope is underlain by medium-dense silty sand residual soils which are not likely to liquefy. Some of the sandy alluvium encountered in the lower-lying portions of the site east of the planned structure are prone to local liquefaction and sand boils; however, no significant improvements are currently planned in these areas. Therefore, we judge the risk of damage to the proposed residence due to liquefaction is generally low.

*Evaluation:* *No significant impact.*
*Recommendations:* *No remedial measures required.*

### 4.4 Seismically-Induced Ground Settlement

Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Loose, granular soils were not observed within the planned building envelope, and the risk of damage due to seismically-induced settlement is judged to be low.

*Evaluation:* *No significant impact.*
*Recommendations:* *No remedial measures required.*

### 4.5 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, uneven floors, and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures.

Based on our exploration, bedrock is shallow and near-surface soils are generally of low plasticity. Therefore, the risk of expansive soil affecting the proposed improvements is low.

*Evaluation:* *No significant impact.*
*Recommendations:* *No remedial measures required.*

### 4.6 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays or loose soils. Based on our exploration, bedrock is relatively shallow throughout the site and settlement risk is generally low.

*Evaluation:* *No significant impact.*
*Recommendations:* *No remedial measures required.*

### 4.7 Slope Instability/Landslides

The project site is located near the base of a steeply-sloping ridgeline with average inclinations of about 3:1 (Horizontal:Vertical). Locally, slopes in the southeastern part of the site approach 1.5:1, while steep banks along the drainage channels east of the building site are locally near-vertical.

During our site reconnaissance, we noted topography in the southeastern part of the site is suggestive of previous slope instability, including steep, rounded and overgrown scarps up to about 5-feet high and apparent rotated slump blocks. Since no slide debris was observed in our fault trench exposure at the base of the site, it is likely that the debris from previous episodes of instability has been washed away. We also noted local small slumps along the over-steepened channel banks above the building site.

While no significant evidence of recent or imminent instability was observed, such as tension cracks, fresh scarps, or debris piles, we judge that there is a moderate risk of damage due to slope instability, particularly small debris-flows emanating from the deep drainage gully above the eastern edge of the building area. This risk will be increased at times of soil saturation, such as following heavy rains, and during a seismic event.

*Evaluation:* *Less than significant with remedial measures.*

*Recommendations:* *New structures should be founded on a deep foundation system composed of drilled piers and interconnected grade beams to reduce the risk of damage due to undermining in the event of future instability. Additionally, remedial measures should be provided for the risk of debris-flows emanating from the steep channel above the building site. Suitable remedial options could include avoidance (re-siting the structure away from the mouth of the channel) or constructing a new debris barrier near the mouth of the channel. More detailed discussion regarding optional measures for slope-instability risks is provided in Section 5 of this report.*

### 4.8 Erosion

Sandy soils on moderately steep slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity.

Steep terrain and relatively loose surface soils make the site prone to erosion. Construction of the proposed improvements may result in changes to existing surface drainage patterns which, if not properly addressed during design and construction, could lead to concentrated surface water flows and increased erosion. Considering the steeper terrain that surrounds the project site, and the disturbance to existing vegetation and drainage patterns that may result from the proposed improvements, we judge the risk of damage to improvements due to erosion is moderate to high.

*Evaluation:* *Less than significant with remedial measures.*

*Recommendations:* *For new improvements at the site, careful attention should be paid to finished grades and the project Civil Engineer should design the site drainage system to collect surface water into a storm drain system and*

*discharge water at appropriate locations. Re-establishment of vegetation on disturbed areas will minimize erosion. Erosion control measures during and after construction should be in accordance with a prepared Storm Water Pollution Prevention Plan and should conform to the most recent version of the California Stormwater Quality Association Best Management Practice Handbook (CSQA, 2003) or similar standards.*

### 4.9 Tsunami and Seiche

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. Based on our review of tsunami inundation maps (California Emergency Management Agency, 2009), the site is not mapped within a tsunami inundation zone. Therefore, tsunami inundation is not considered a significant hazard at the site.

*Evaluation:* *No significant impact.*
*Recommendations:* *No remedial measures required.*

### 4.10 Flooding

The proposed improvements are located at elevations ranging from about 20 to 110 feet above sea level and are not mapped within a FEMA 100-year flood zone (Federal Emergency Management Agency, 2009). Therefore, large scale flooding is not considered a significant hazard at the project site. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.

*Evaluation:* *No significant impact.*
*Recommendations:* *The project Civil Engineer or Architect should evaluate the risk localized flooding and provide appropriate storm drain design.*

## 5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration, we judge that construction of the proposed residence and associated improvements is feasible from a geotechnical standpoint. Primary geotechnical considerations for the project will include providing adequate foundation support and seismic design for the new structure, as well as providing adequate remedial measures for potential landslides and debris flows which could impact the residence. Additional discussion and recommendations addressing these and other considerations are presented in the following sections.

### 5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2019) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity of several nearby faults, we recommend the CBC coefficients and site values shown in Table 3 be used to calculate the design base shear of the new construction.

**Table 3 – 2019 California Building Code Seismic Design Criteria**

| Parameter | Design Value |
|---|---|
| Site Class | $B_{(estimated)}$ |
| Site Latitude | 38.102°N |
| Site Longitude | -122.862°W |
| Spectral Response (short), $S_S$ | 2.446 g |
| Spectral Response (1-sec), $S_1$ | 1.026 g |
| Site Coefficient, $F_a$ | 1.0 |
| Site Coefficient, $F_V$ | 1.0 |

Reference: SEAOC/OSHPD Seismic Design Maps (web-based seismic response calculator tool), https://seismicmaps.org/, accessed February 24, 2021.

## 5.2 Site Grading

Moderate site grading, including cuts and fills up to a few feet high, is anticipated for the project. Site grading should be performed in accordance with the recommendations and criteria outlined in the following sections.

### 5.2.1 Site Preparation

Clear pavements, old foundations, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site.

Where fills or structural improvements are planned, the subgrade surface should be scarified to a depth of 8 inches, moisture conditioned to slightly above the optimum moisture content, and compacted to at least 90 percent relative compaction. Areas exposing bedrock at subgrade need not be scarified and recompacted. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. Subgrade preparation should extend a minimum of 5-feet beyond the planned building envelope in all directions. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we will provide supplemental recommendations to address the specific condition.

### 5.2.2 Excavations

Site excavations for new foundations, utilities, and other improvements will generally encounter a few feet of silty sand (decomposed granite) over highly weathered granitic rock. Based on our subsurface exploration, we judge that excavations within the upper 10 feet of the ground surface can likely be performed with “traditional” equipment, such as medium-size dozers and excavators. However, deeper excavations may encounter rock which cannot be efficiently excavated with typical equipment and requires specialized techniques or equipment to excavate (e.g., jackhammers or hoe-rams). Therefore, we recommend inclusion of a line item and clear definition for “hard rock excavation” in the project bid documents. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria as appropriate.

### 5.2.3 Fill Materials, Placement and Compaction

As a minimum, fill materials should be non-expansive and free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and have a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of 4 inches. Some of the onsite soils may be suitable for use as fill provided they meet the criteria specified above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to near the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of 8 inches-thick or less and uniformly compacted to at least 90 percent relative compaction. In pavement areas, the upper 12 inches of fill should be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

#### 5.2.4 Permanent and Temporary Cut and Fill Slopes

Permanent cut slopes in onsite soils and weathered bedrock should be inclined no steeper than 2:1. Permanent cuts steeper than 2:1 will need to be retained with properly designed and drained retaining walls. New fill slopes steeper than 10:1 should be keyed, benched, and fully drained as shown conceptually on Figure 6, and should not exceed 2:1. Permanent fills steeper than 2:1 are generally not recommended at the site.

Onsite soils should be considered "Type C" materials per Cal/OSHA categorization. As such, temporary cuts should be inclined no steeper than 1.5:1. Steeper cuts in more competent bedrock may be possible, but geologic inspection during construction will be required in that event.

### 5.3 Foundation Design

To reduce the risk of damage to new improvements due to potential slope instability and erosion, we recommend the residence be supported on a drilled pier foundation system. The drilled piers should extend a minimum of 5 feet into competent weathered bedrock.

The piers should be designed using an allowable skin friction of 2,000 pounds per square foot and a minimum diameter of 18 inches. Lateral resistance for drilled piers on level terrain should be calculated using a passive resistance of 400 pounds per cubic foot (equivalent fluid pressure) applied over two pier diameters. The upper 3 feet of embedment should be ignored in calculating the lateral and vertical capacity of drilled piers. Individual piers should be interconnected with reinforced concrete grade beams.

Drilled piers constructed on sloping ground should be designed to account for a creep pressure of 60 pounds per cubic foot applied to a depth of 5 feet below ground surface. Additionally, the lateral resistance for drilled piers on slopes inclined at 2:1 should be calculated using a reduced passive resistance of 250 pounds per cubic foot. The passive resistance for intermediate slopes (i.e., inclined flatter than 2:1) should be calculated using linear interpolation between the values for level terrain and 2:1 slope inclination.

### 5.4 Retaining Walls

Retaining walls may be necessary to support planned cuts or fills as required by the grading plans. New walls should be supported on drilled pier foundations designed per Section 5.3 of this report. Retaining walls that can deflect a small amount at the top, such as site or landscape walls, can be designed using the unrestrained criteria shown in Table 4. Walls that are structurally connected at the top and not allowed to deflect (such as basement walls or tieback walls) are considered restrained. Restrained conditions are commonly designed using a uniform earth pressure distribution rather than an equivalent fluid pressure. Lateral support can be obtained

from either passive soil resistance (i.e., keyways) or frictional sliding resistance of footings or from tiebacks. In addition to the soil loads, the retaining walls should be designed to resist temporary seismic loads as well as any anticipated traffic surcharge loads (such as for walls supporting driveway areas, if planned).

**Table 4 – Active Earth Pressure for Retaining Wall Design**

| Backfill Inclination[1] | Unrestrained[2,3] | Restrained[3,4] |
|---|---|---|
| Level | 45 pcf | 30 x H psf |
| 3:1 | 50 pcf | 35 x H psf |
| 2:1 | 60 pcf | 40 x H psf |

Notes:
(1) Interpolate earth pressures for intermediate slopes
(2) Equivalent fluid pressure.
(3) Wall design should account for a seismic surcharge of 15 x H (in psf) in addition to active pressure
(4) Rectangular distribution, H is wall height in feet.

Wall drainage is required for all retaining walls taller than 3 feet. Either Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material can be used for wall drainage. The drainage should be collected in a 4-inch, SDR 35, perforated PVC drain line at the base of the wall. The permeable material should extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. Alternatively, drainage panels, such as Mirafi 100N, may be utilized. A schematic retaining wall drainage detail is shown on Figure 7.

### 5.5 Slope Instability Remedial Measures

As previously discussed, we judge there are a few options by which risks associated with slope instability related to debris flows from upslope areas may be reduced. We note that the current building pad has already been shifted about 30-feet to the west versus previous iterations, and that the majority of the structure is sited on the topographic “nose” west of the gully, where the risk of instability is judged to be appreciably lower. However, the planned entry area at the east end of the house remains within the mouth of the gully, and we judge some measure of protection from potential debris impact should be provided. The least expensive method may be to re-site the building slightly farther west or redesign the entryway to otherwise avoid the gully area. However, we understand this may conflict with architectural plans and/or the planned septic system.

Another suitable remedial measure could include constructing a debris deflection wall or debris barrier (Geobrugg® or similar) above the east end of the house, near the mouth of the gully, which is designed to absorb the force of debris impact and protect the residence from damage. A deflection wall would likely be of soldier-pile and timber lagging type construction, be on the order of 3- to 4-feet high and be aligned with the goal of deflecting semi-fluid debris east of the structure, over the entry stairs and driveway where the risks of significant structural damage are much lower. The debris barrier is an engineered system consisting of steel vertical supports with breakaway bases and a chain-link mesh “curtain” supported by the vertical posts and horizontal wire ropes supports at the top and bottom. The wire ropes incorporate a brake-ring system allowing for rapid

deceleration of the slide mass and are anchored using drilled and grouted rock anchors. The debris barrier is designed to deform elastically during slide impact and sustain repairable damage. If a debris flow were to occur, the debris barrier may be re-used by removing captured slide debris and repairing broken upright supports and any other damaged components.

A debris barrier of this type is typically designed in-house by the manufacturer and the exact design will depend on the chosen placement. The barriers typically utilize shallow foundations and rock anchors for foundation support. The debris barrier should be a minimum of 6-feet-high to provide adequate protection. Typical anchor loads for debris barriers of this height are on the order of about 50 kips. The anchors should be designed using an allowable skin friction of 2,000 pounds per square foot and a minimum drilled diameter of six inches.

Note that none of the aforementioned remedial measures will reduce the actual risk of slope instability. The deflection wall will reduce the risk of debris impact and resulting damage to the structure, but would allow debris to impact the entry stairs and driveway area which could require clean-up and repair. A debris barrier would more effectively "trap" debris on the slope above it and as such would further reduce the risk and scope of site improvement damage, but would still need to be cleared of accumulated debris and/or repaired as needed in response to significant events and as part of regular property maintenance.

### 5.6 Concrete Slabs-On-Grade

Reinforced concrete slab-on-grade floors are judged to be appropriate for the site conditions. The concrete slabs-on-grade may be poured monolithically or separated with a cold joint at the Structural Engineer's discretion. We recommend that interior concrete slabs have a minimum thickness of 5 inches and be reinforced with steel reinforcing bars (not mesh). Slabs should be placed on a moist subgrade to reduce potential for future expansive behavior. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a 4-inch layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker, should be placed over the free draining gravel. The vapor barrier should meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio since eliminating the sand can cause cracking or "curling" of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least 4 inches of Class 2 Aggregate Base (Caltrans, 2015) compacted to at least 95 percent relative compaction.

### 5.7 Exterior Flatwork

Exterior concrete walkway slabs not subjected to vehicle loads should be a minimum of 4-inches-thick and underlain with 4 inches or more of Class 2 Aggregate Base. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper 8 inches of subgrade on which aggregate base is placed should be prepared as previously discussed under Section 5.2.

Where improved performance is desired (i.e., reduced risks of cracking or small settlements), exterior slabs can be thickened to 5 inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than 6 feet apart in both directions and that the reinforcing bars, if used, extend through the control joints. Some movement should be expected due to seasonal shrink/swell of soils.

### 5.8 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the residence. Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent).

Roof gutter downspouts may discharge onto the pavements but should not discharge onto landscaped areas immediately adjacent to the home. Provide area drains for landscape planters adjacent to buildings and parking areas, and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

Given the slopes that exist throughout the project site, we recommend including foundation drains around the upslope side of all building foundations which are not provided with a retaining wall and associated backdrain. A schematic foundation drain detail is included as Figure 8.

### 5.9 New Utilities

Excavations for utilities will be in weathered granitic rock and may encounter groundwater at shallow depths if wintertime or early spring work is performed. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations. Pursuant to OSHA classifications, on-site soils should be considered Type C.

Bedding materials for utility pipes should be poorly graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils, moisture conditioned and placed in thin lifts and compacted to at least 90 percent. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

## 6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for site development and foundation design when they are nearing completion to confirm that the intent of our recommendations has been incorporated and to provide supplemental recommendations as needed. During construction, we must inspect geotechnical items relating to site preparation and grading, retaining walls and foundation construction. We should observe foundation excavations, subgrade preparation and compaction, proper moisture conditioning of soils, fill placement and compaction, retaining wall drainage and backfilling and other geotechnical-related work items.

## 7.0 LIMITATIONS

We believe this report has been prepared in accordance with generally accepted geotechnical engineering practices in Marin County at the time the report was prepared. This report has been prepared for the exclusive use of the project Owner and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area. The exploratory test pits and description of soils encountered reflect conditions only at the location of the test pit at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage

## 8.0 LIST OF REFERENCES

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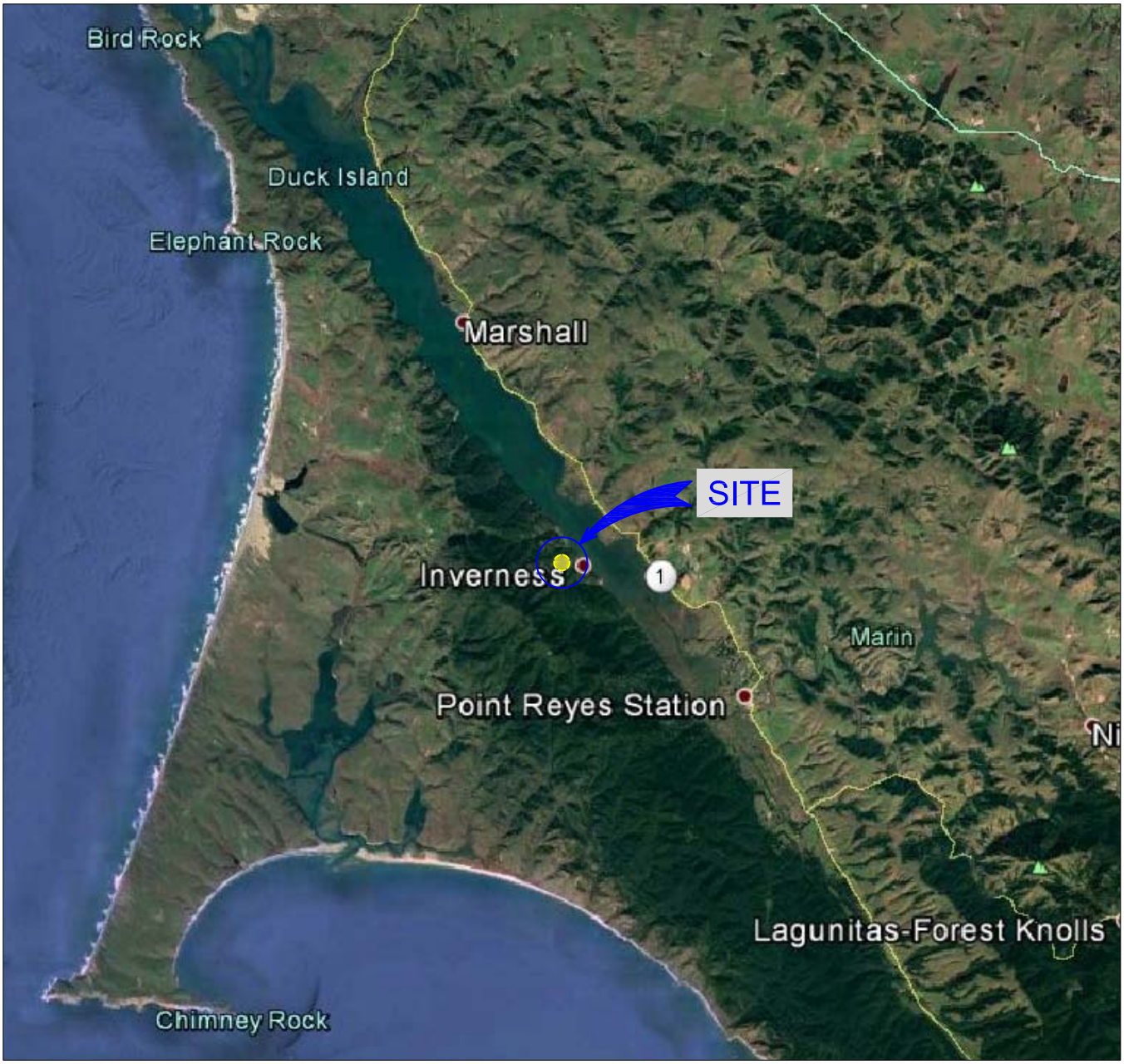


SITE COORDINATES
LAT. 38.1022°
LON. -122.8612°

# SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2018

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FILENAME: 2626.001 STD FIGS.dwg

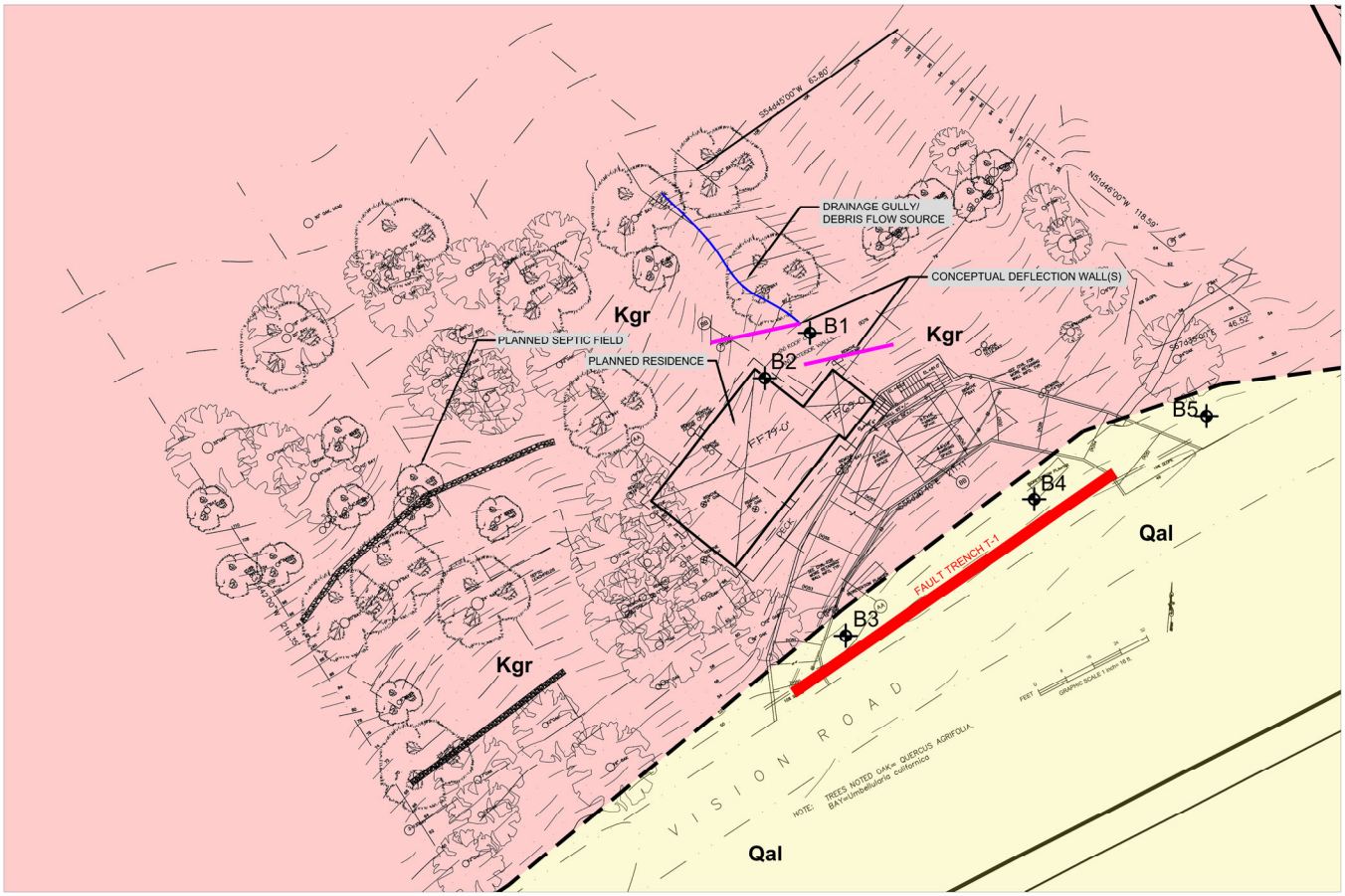
SITE LOCATION MAP

Vision Road
APN 112-141-03/04
Inverness, California

Project No. 2626.001 Date: 8/31/2018

Drawn ENE
Checked

1
FIGURE




# SITE PLAN AND GEOLOGIC MAP

SCALE

0 25 50 100 FEET

NORTH

**Qal** ALLUVIUM (QUATERNARY)
Unconsolidated silty and clayey sand deposited in active stream channels.

**Kgr** GRANITE (CRETACEOUS)
Granite and granodiorite, highly to completely weathered, friable.

GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE

EXPLORATORY FAULT TRENCH BY MPEG, 2018

TEST BORING BY MPEG, 2018.

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SITE PLAN AND GEOLOGIC MAP

Vision Road
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Lagunitas, California

Project No. 2626.002 Date: 8/31/2018

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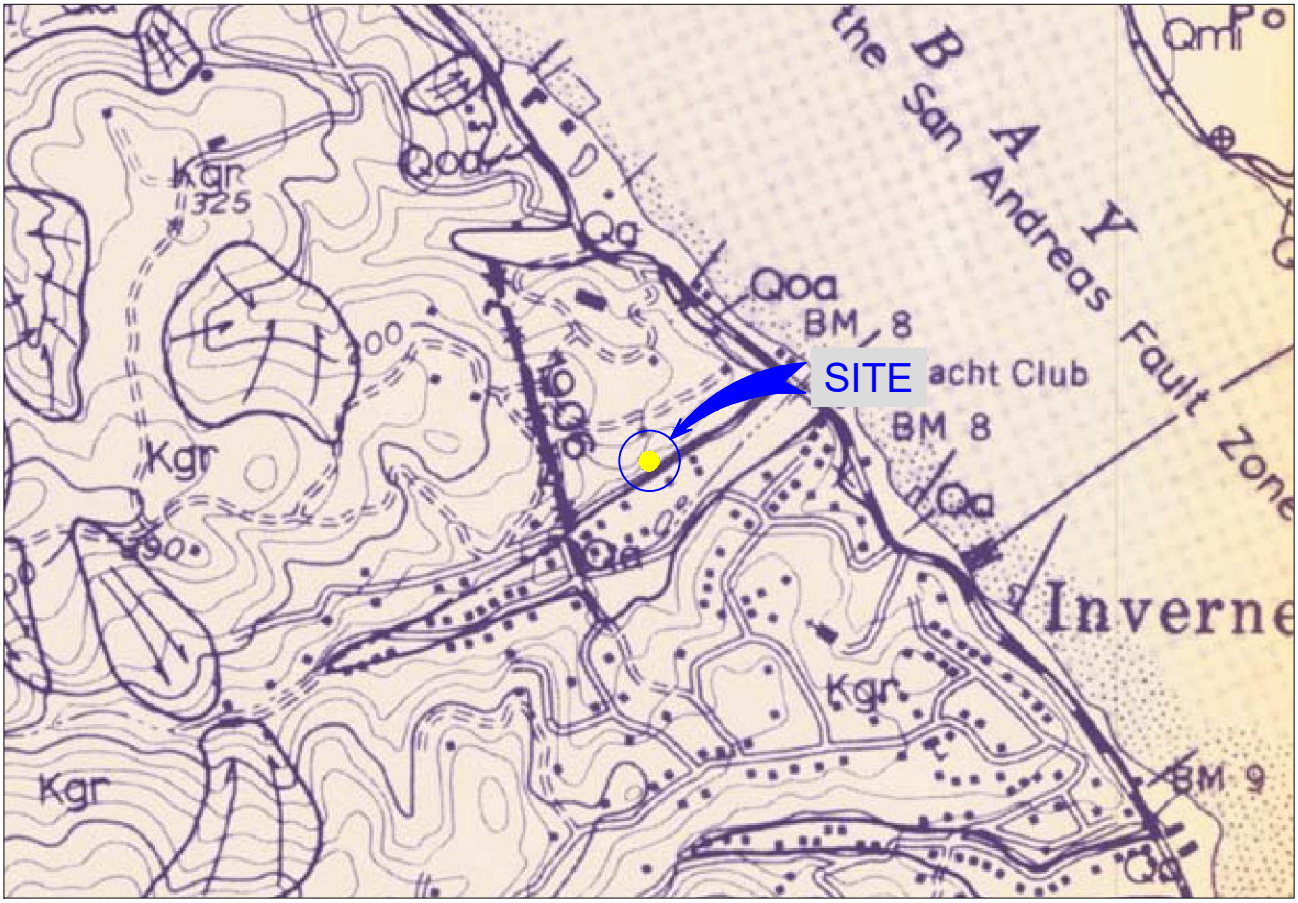
Checked

2
FIGURE




# REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

**LEGEND**

| | |
|---|---|
| Qa | Alluvium - (Holocene)<br>Poorly consolidated, poorly sorted clay, silt, sand, and gravel usually fill stream and valley floors. |
| Kgr | Granodorite and Granite of Inverness Ridge - (Upper Cretaceous)<br>Granodiorite and granite are exposed along Inverness Ridge, where dikes and masses of aplite and alaskite are locally abundant. |

Contact - Dashed where approximately located or gradational

Fault - Dashed where inferred, dotted where concealed. Half arrows indicate relative horizontal movement

Reference: Wagner, D.L. and Smith, T.C. (1977), "Geology of the Tomales Bay Study Area", United States Geological Survey Open-File Report 77-15, Plate 2, Map Scale 1:12,000.

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REGIONAL GEOLOGIC MAP

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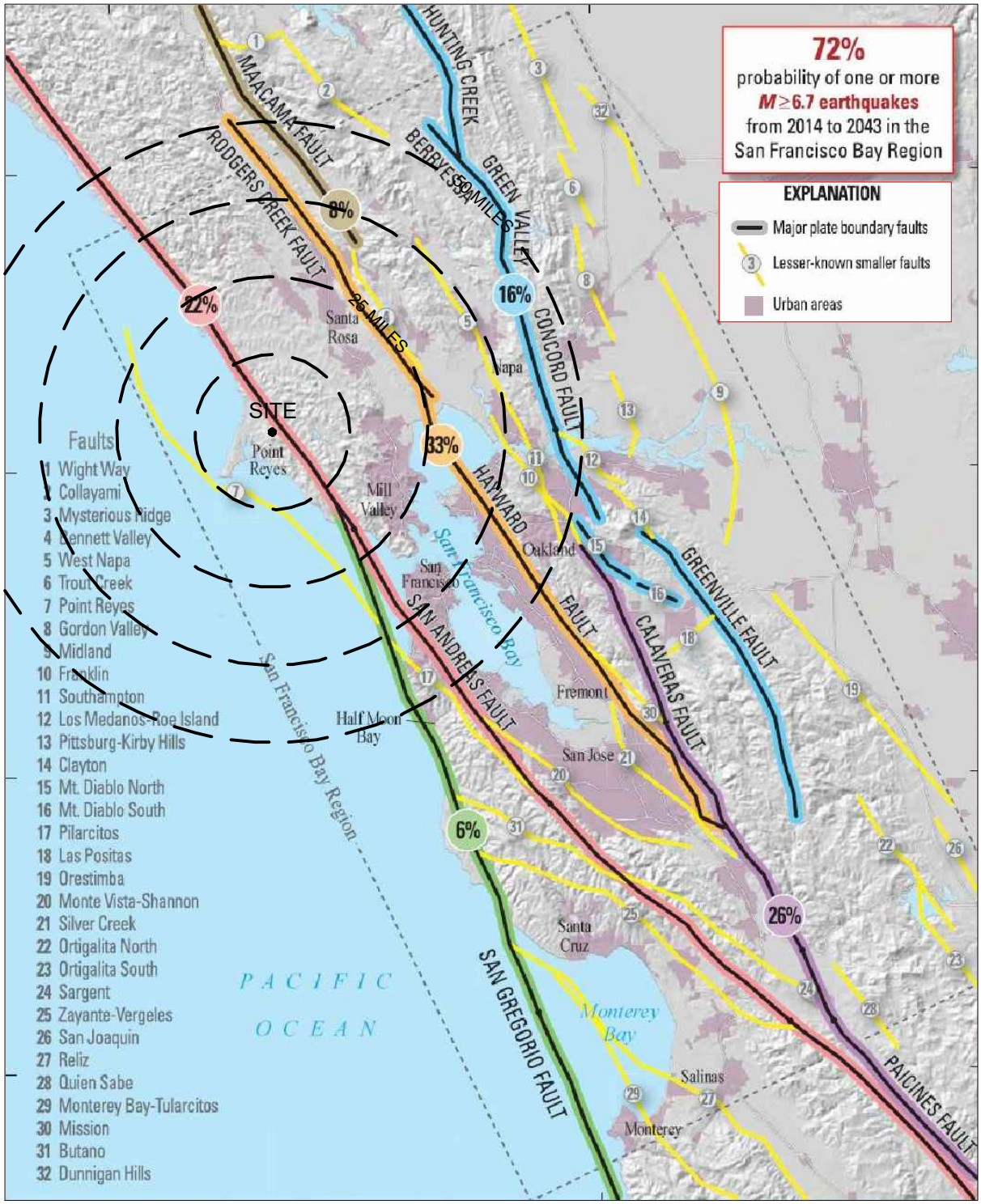
Project No. 2626.002 Date: 3/4/2021

Drawn ENE
Checked

3
FIGURE




SITE COORDINATES
LAT. 38.1022°
LON. -122.8612°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

| | | | | |
|---|---|---|---|---|
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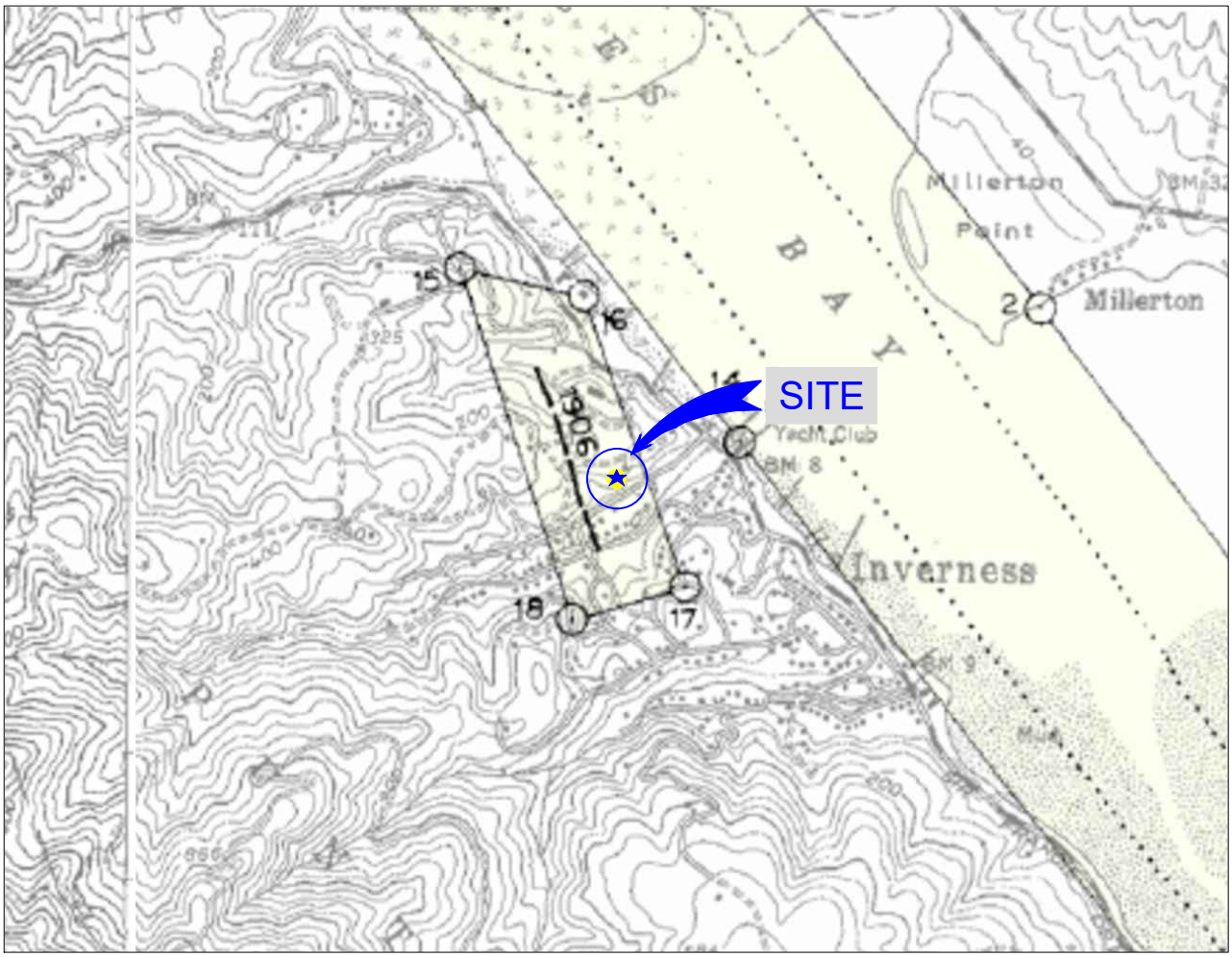
| A CALIFORNIA CORPORATION, © 2018, ALL RIGHTS RESERVED<br>FILENAME: 2626.001 STD FIGS.dwg | | Vision Road<br>APN 112-141-03/04<br>Lagunitas, California<br>Project No. 2626.002 Date: 3/4/2021 | Drawn ENE<br>Checked | 4<br>FIGURE |




# ALQUIST PRIOLO EARTHQUAKE FAULT ZONES

(NO SCALE)

NORTH

**MAP EXPLANATION**

**Potentially Active Faults**

Faults considered to have been active during Quaternary time; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Aerial photo lineaments (not field checked), based on youthful geomorphic and other features believed to be the results of Quaternary faulting.

**Special Studies Zone Boundaries**

These are delineated as straight-line segments that connect consecutively numbered turning points so as to define one or more special studies zone segments.

Seaward projection of zone boundary.

REFERENCE: CGS Special Studies Zones, Inverness and Drakes Bay Quadrangles, 1974

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ALQUIST-PRIOLO EARTHQUAKE FAULT ZONE MAP

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Inverness, California

Project No. 2626.002 Date: 3/4/2021

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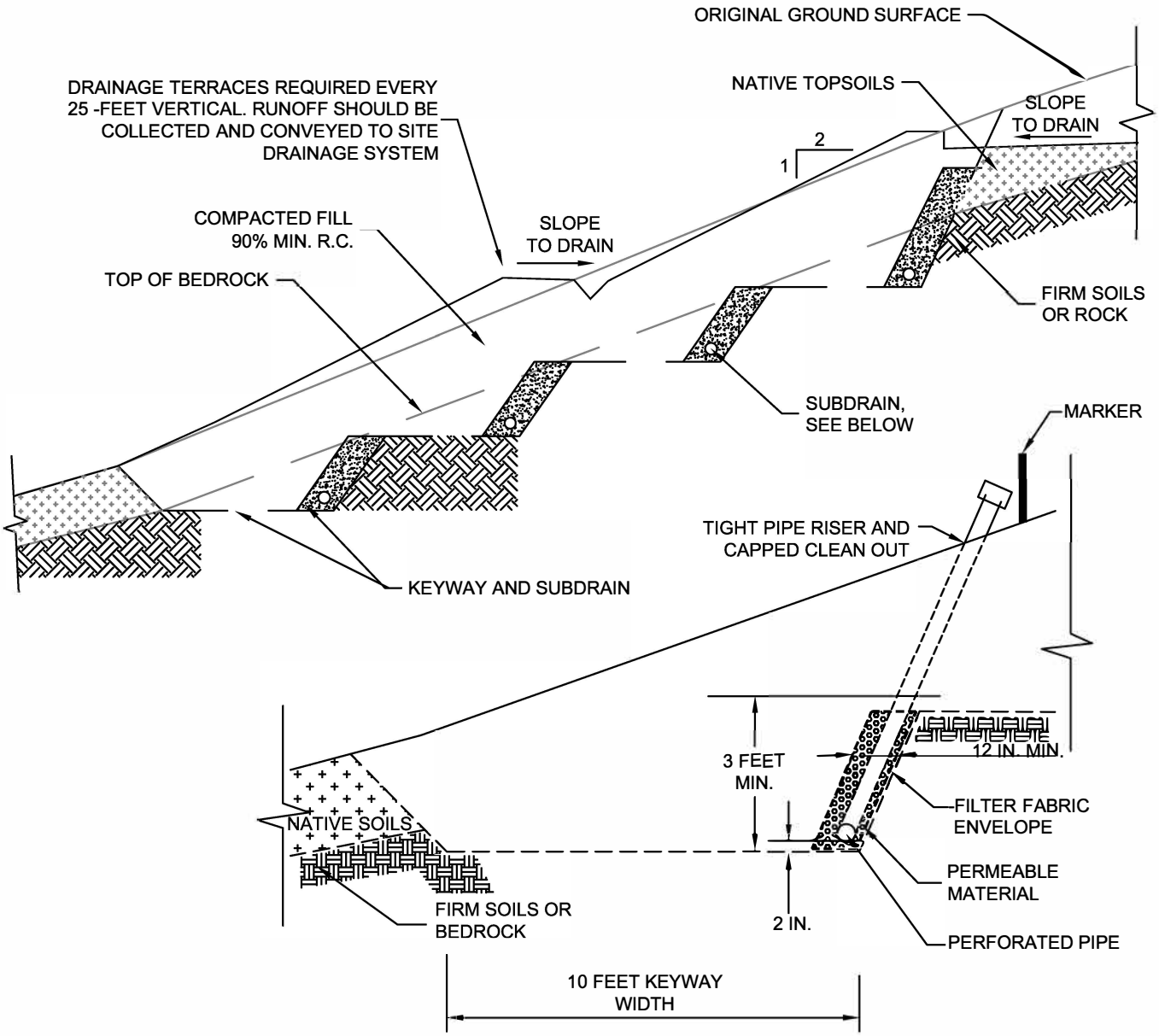
5
FIGURE




NOTES:

1. Subdrain drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material.

2. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet, with tight pipe to gravity discharge.

3. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

4. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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FILENAME: 2017 Hillside Fill Construction.dwg

TYPICAL HILLSIDE FILL CONSTRUCTION

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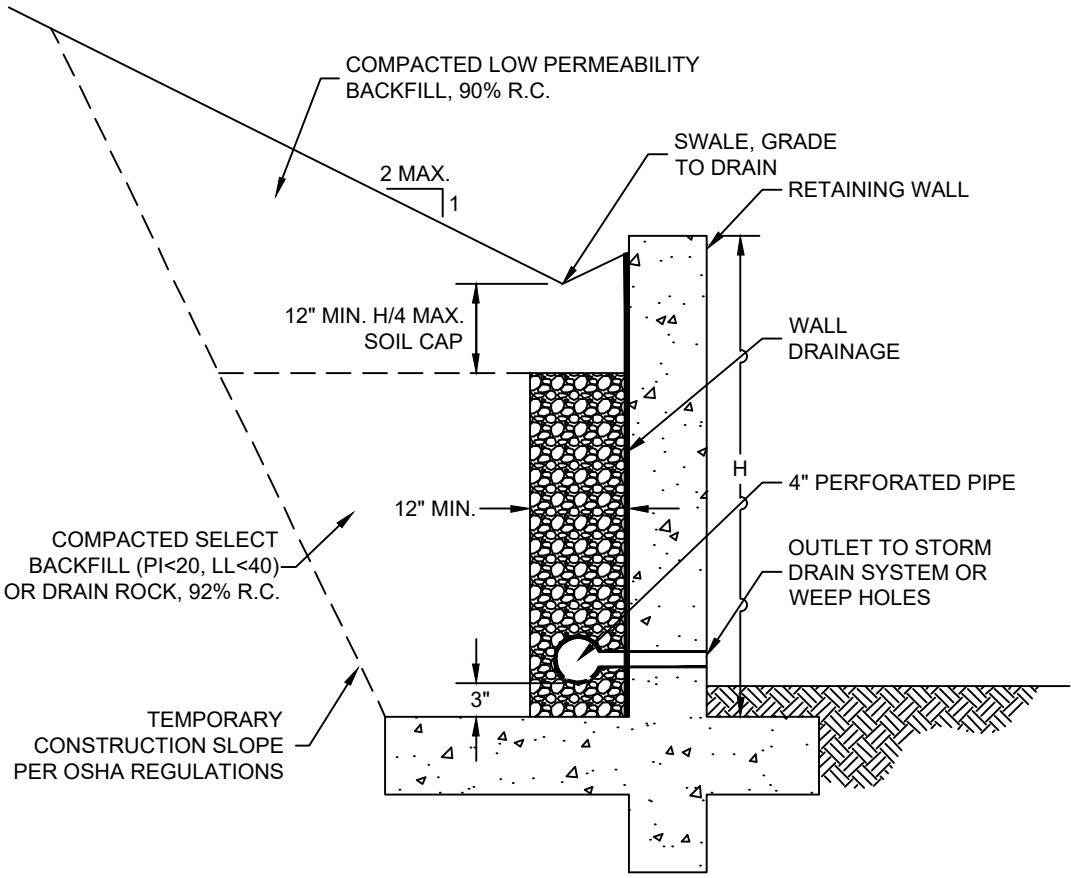
6
FIGURE




NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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TYPICAL RETAINING WALL BACKDRAIN DETAIL

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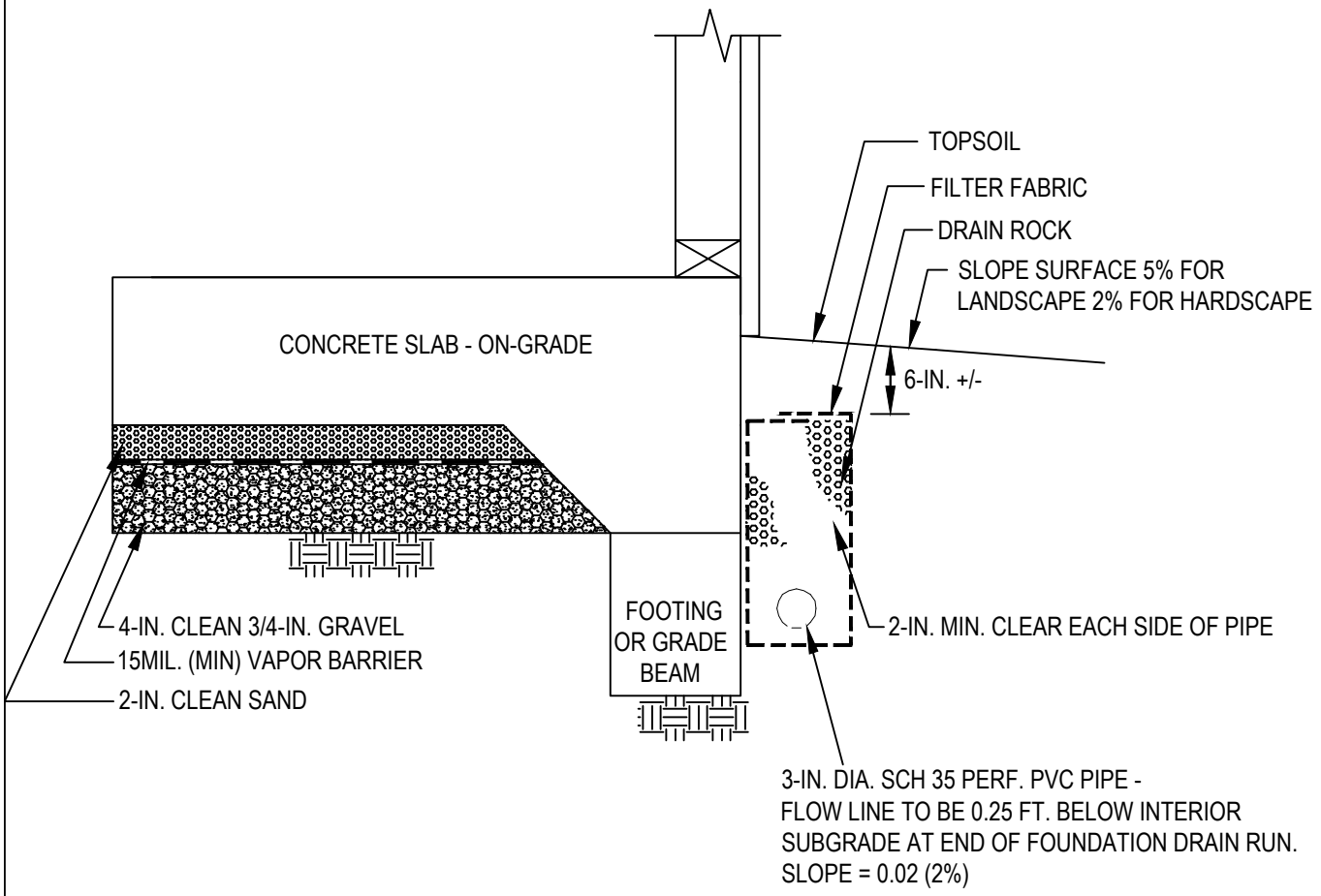
7
FIGURE




NOTES:

(1.) DO NOT CONNECT DOWNSPOUT LEADER TO FOUNDATION DRAIN

(2.) DISCHARGE THROUGH 4-IN DIAM., SCH 35, NON-PERFORATED PIPE, SLOPE 0.02 (2%) UNLESS OTHERWISE SPECIFIED

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TYPICAL FOUNDATION DRAIN DETAIL

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Project No. 2626.002 Date: 3/4/2021

Drawn MFJ
Checked

8
FIGURE

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# APPENDIX A
# SUBSURFACE EXPLORATION AND LABORATORY TESTING

## A. SUBSURFACE EXPLORATION

We explored subsurface conditions with five exploratory borings drilled with track-mounted equipment on April 20, 2018 at the approximate locations shown on the Site Plan, Figure 2. The exploration was conducted under the technical supervision of our Field Geologist who examined and logged the soil materials encountered and obtained samples. The subsurface conditions encountered in the test borings are summarized and presented on the boring logs, Figures A-3 through A-7.

“Undisturbed” samples were obtained using a 3-inch diameter, split-barrel Modified California Sampler with 2.5 by 6-inch tube liners or a 2.5-inch diameter, split-barrel Standard Penetration Test (SPT) Sampler with no liners. The samplers were driven by a 140-pound hammer at a 30-inch drop. The approximate number of blows required to drive the samplers 18 inches is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory. All borings were backfilled upon completion with native spoils.

| MAJOR DIVISIONS | | SYMBOL | DESCRIPTION |
|---|---|---|---|
| COARSE GRAINED SOILS over 50% sand and gravel | CLEAN GRAVEL | GW | Well-graded gravels or gravel-sand mixtures, little or no fines |
| | | GP | Poorly-graded gravels or gravel-sand mixtures, little or no fines |
| | GRAVEL with fines | GM | Silty gravels, gravel-sand-silt mixtures |
| | | GC | Clayey gravels, gravel-sand-clay mixtures |
| | CLEAN SAND | SW | Well-graded sands or gravelly sands, little or no fines |
| | | SP | Poorly-graded sands or gravelly sands, little or no fines |
| | SAND with fines | SM | Silty sands, sand-silt mixtures |
| | | SC | Clayey sands, sand-clay mixtures |
| FINE GRAINED SOILS over 50% silt and clay | SILT AND CLAY liquid limit <50% | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity |
| | | CL | Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays |
| | | OL | Organic silts and organic silt-clays of low plasticity |
| | SILT AND CLAY liquid limit >50% | MH | Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts |
| | | CH | Inorganic clays of high plasticity, fat clays |
| | | OH | Organic clays of medium to high plasticity |
| HIGHLY ORGANIC SOILS | | PT | Peat, muck, and other highly organic soils |
| ROCK | | | Undifferentiated as to type or composition |

## KEY TO BORING AND TEST PIT SYMBOLS

### CLASSIFICATION TESTS

| | |
|---|---|
| PI | PLASTICITY INDEX |
| LL | LIQUID LIMIT |
| SA | SIEVE ANALYSIS |
| HYD | HYDROMETER ANALYSIS |
| P200 | PERCENT PASSING NO. 200 SIEVE |
| P4 | PERCENT PASSING NO. 4 SIEVE |

### SAMPLER TYPE

- MODIFIED CALIFORNIA
- STANDARD PENETRATION TEST
- THIN-WALLED / FIXED PISTON
- HAND SAMPLER
- ROCK CORE
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

### STRENGTH TESTS

| | |
|---|---|
| TV | FIELD TORVANE (UNDRAINED SHEAR) |
| UC | LABORATORY UNCONFINED COMPRESSION |
| TXCU | CONSOLIDATED UNDRAINED TRIAXIAL |
| TXUU | UNCONSOLIDATED UNDRAINED TRIAXIAL |
| | UC, CU, UU = 1/2 Deviator Stress |

### SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

| | |
|---|---|
| 25 | sampler driven 12 inches with 25 blows after initial 6-inch drive |
| 85/7" | sampler driven 7 inches with 85 blows after initial 6-inch drive |
| 50/3" | sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive |

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SOIL CLASSIFICATION CHART

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Project No. 2626.001 Date: 9/7/18

Drawn MFJ
Checked

A-1
FIGURE

## FRACTURING AND BEDDING

| Fracture Classification | Spacing | Bedding Classification |
|---|---|---|
| Crushed | less than 3/4 inch | Laminated |
| Intensely fractured | 3/4 to 2-1/2 inches | Very thinly bedded |
| Closely fractured | 2-1/2 to 8 inches | Thinly bedded |
| Moderately fractured | 8 to 24 inches | Medium bedded |
| Widely fractured | 2 to 6 feet | Thickly bedded |
| Very widely fractured | greater than 6 feet | Very thickly bedded |

## HARDNESS

| | |
|---|---|
| Low | Carved or gouged with a knife |
| Moderate | Easily scratched with a knife, friable |
| Hard | Difficult to scratch, knife scratch leaves dust trace |
| Very hard | Rock scratches metal |

## STRENGTH

| | |
|---|---|
| Friable | Crumbles by rubbing with fingers |
| Weak | Crumbles under light hammer blows |
| Moderate | Indentations <1/8 inch with moderate blow with pick end of rock hammer |
| Strong | Withstands few heavy hammer blows, yields large fragments |
| Very strong | Withstands many heavy hammer blows, yields dust, small fragments |

## WEATHERING

| | |
|---|---|
| Complete | Minerals decomposed to soil, but fabric and structure preserved |
| High | Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates |
| Moderate | Fracture surfaces coated with weathering minerals, moderate or localized discoloration |
| Slight | A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation |
| Fresh | Rock unaffected by weathering, no change with depth, rings under hammer impact |

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

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ROCK CLASSIFICATION CHART

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A-2
FIGURE

# BORING 1

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 4/20/18
ELEVATION: 55 - feet*
*REFERENCE: Google Earth, 2018

| DEPTH meters | DEPTH feet | SAMPLE | SYMBOL (4) | | BLOWS / FOOT (1) | DRY UNIT WEIGHT pcf (2) | MOISTURE CONTENT (%) | SHEAR STRENGTH psf (3) | OTHER TEST DATA | OTHER TEST DATA |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | | | SAND WITH SILT (SP-SM)<br>Dark brown, wet, medium dense, fine to medium-grained, ~10% low plasticity silt, abundant organics<br>[COLLUVIUM/RESIDUAL SOIL] | | | | | | |
| | 1 | | | | | | | | | |
| | 2 | | | | | | | | | |
| | 3 | | | | | | | | | |
| 1 | | | | | | | | | | |
| | 4 | | | | 26 | | | | | |
| | 5 | | | | 50/6" | | | | | |
| | | | | GRANITE<br>Light brown, completely weathered, friable<br>[BEDROCK] | | | | | | |
| | 6 | | | | | | | | | |
| 2 | | | | | | | | | | |
| | 7 | | | | | | | | | |
| | 8 | | | | | | | | | |
| | | | | | 88/9" | | | | | |
| | 9 | | | | | | | | | |
| | | | | Boring terminated at 9.25-feet.<br>No groundwater encountered during exploration. | | | | | | |
| 3 | 10 | | | | | | | | | |

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m$^3$ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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APN 112-141-03/04
Inverness, California

Project No. 2626.001 Date: 9/718

Drawn MFJ
Checked

A-3
FIGURE

# BORING 2

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 4/20/18
ELEVATION: 55 - feet*
*REFERENCE: Google Earth, 2018

| DEPTH meters | DEPTH feet | SAMPLE | SYMBOL (4) | | BLOWS / FOOT (1) | DRY UNIT WEIGHT pcf (2) | MOISTURE CONTENT (%) | SHEAR STRENGTH psf (3) | OTHER TEST DATA | OTHER TEST DATA |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | | | SAND WITH SILT (SP-SM)<br>Dark brown, wet, medium dense, fine to medium-grained, ~10% low plasticity silt, abundant organics<br>[COLLUVIUM/RESIDUAL SOIL] | | | | | | |
| | 1 | | | | | | | | | |
| | 2 | | | | | | | | | |
| | 3 | | | | | | | | | |
| 1 | | | | | 22 | | | | | |
| | 4 | | | | | | | | | |
| | 5 | | | GRANITE<br>Light brown, completely weathered, friable<br>[BEDROCK] | 50/6" | | | | | |
| | 6 | | | | | | | | | |
| 2 | | | | | | | | | | |
| | 7 | | | | | | | | | |
| | 8 | | | | 55/6" | | | | | |
| | 9 | | | Boring terminated at 9.0-feet.<br>No groundwater encountered during exploration. | | | | | | |
| 3 | 10 | | | | | | | | | |

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m$^3$ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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Drawn MFJ
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A-4
FIGURE

# BORING 3

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 4/20/18
ELEVATION: 25 - feet*
*REFERENCE: Google Earth, 2018

| DEPTH meters / feet | SAMPLE | SYMBOL (4) | | BLOWS / FOOT (1) | DRY UNIT WEIGHT pcf (2) | MOISTURE CONTENT (%) | SHEAR STRENGTH psf (3) | OTHER TEST DATA | OTHER TEST DATA |
|---|---|---|---|---|---|---|---|---|---|
| 0 / 0 | | | CLAYEY SAND (SC)<br>Dark brown, wet, medium dense, fine to medium-grained, ~20% low plasticity clay [ALLUVIUM] | | | | | | |
| 1 | | | | | | | | | |
| 2 | | | | | | | | | |
| 3 / 1 | | | | | | | | | |
| 4 | | | SILTY SAND (SM)<br>Light brown, wet, fine to medium-grained, ~15% low plasticity silt [RESIDUAL SOIL] | 22 | | | | | |
| 5 | | | | | | | | | |
| 6 / 2 | | | GRANITE<br>Light brown, completely weathered, friable [BEDROCK] | | | | | | |
| 7 | | | | 33 | | | | | |
| 8 | | | Boring terminated at 7.5-feet.<br>Groundwater encountered at 3.0-feet during exploration. | | | | | | |
| 9 | | | | | | | | | |
| 10 / 3 | | | | | | | | | |

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT $kN/m^3$ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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Drawn MFJ
Checked

A-5
FIGURE

# BORING 4

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 4/20/18
ELEVATION: 25 - feet*
*REFERENCE: Google Earth, 2018

| DEPTH meters | DEPTH feet | SAMPLE | SYMBOL (4) | | BLOWS / FOOT (1) | DRY UNIT WEIGHT pcf (2) | MOISTURE CONTENT (%) | SHEAR STRENGTH psf (3) | OTHER TEST DATA | OTHER TEST DATA |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | | | CLAYEY SAND (SC)<br>Dark brown, wet, loose, fine to medium-grained, ~20% low plasticity clay<br>[ALLUVIUM] | | | | | | |
| | 1 | | | | | | | | | |
| | 2 | | | | | | | | | |
| 1 | 3 | | | | | | | | | |
| | 4 | | | Grades medium brown at 4.0-feet. | | | | | | |
| | 5 | | | Grades light brown, with ~20% clay at 5.0-feet. | | | | | | |
| | 6 | | | | | | | | | |
| 2 | | | | Stiffer drilling noted at 7.0-feet. | | | | | | |
| | 7 | | | | | | | | | |
| | 8 | | | GRANITE<br>Light brown, completely weathered, friable<br>[BEDROCK] | | | | | | |
| | 9 | | | | | | | | | |
| 3 | 10 | | | Boring terminated at 10.5-feet.<br>Groundwater encountered at 3.0-feet during exploration. | 48 | | | | | |

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT $kN/m^3$ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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Project No. 2626.001 Date: 9/718

Drawn MFJ
Checked

A-6
FIGURE

# BORING 5

EQUIPMENT: Portable Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 4/20/18
ELEVATION: 25 - feet*
*REFERENCE: Google Earth, 2018

| DEPTH meters / feet | SAMPLE | SYMBOL (4) | | BLOWS / FOOT (1) | DRY UNIT WEIGHT pcf (2) | MOISTURE CONTENT (%) | SHEAR STRENGTH psf (3) | OTHER TEST DATA | OTHER TEST DATA |
|---|---|---|---|---|---|---|---|---|---|
| 0 / 0 | | | CLAYEY SAND (SC)<br>Dark brown, wet, loose, fine to medium-grained, ~20% low plasticity clay<br>[ALLUVIUM] | | | | | | |
| 2 | | | Grades medium dense, medium brown at 4.0-feet.<br>Grades light brown, with ~20% clay at 5.0-feet. | | | | | | |
| 4 | | | | | | | | | |
| 6 / 2 | | | | | | | | | |
| 8 | | | | | | | | | |
| 10 / 3 | | | | | | | | | |
| 12 | | | Soft drilling noted at 12.0-feet | | | | | | |
| 14 / 4 | | | | 17 | | | | | |
| 16 / 5 | | | | | | | | | |
| 18 | | | Boring terminated at 18.0-feet.<br>Groundwater encountered at 3.0-feet during exploration. | | | | | | |
| 20 / 6 | | | | | | | | | |

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT $kN/m^3$ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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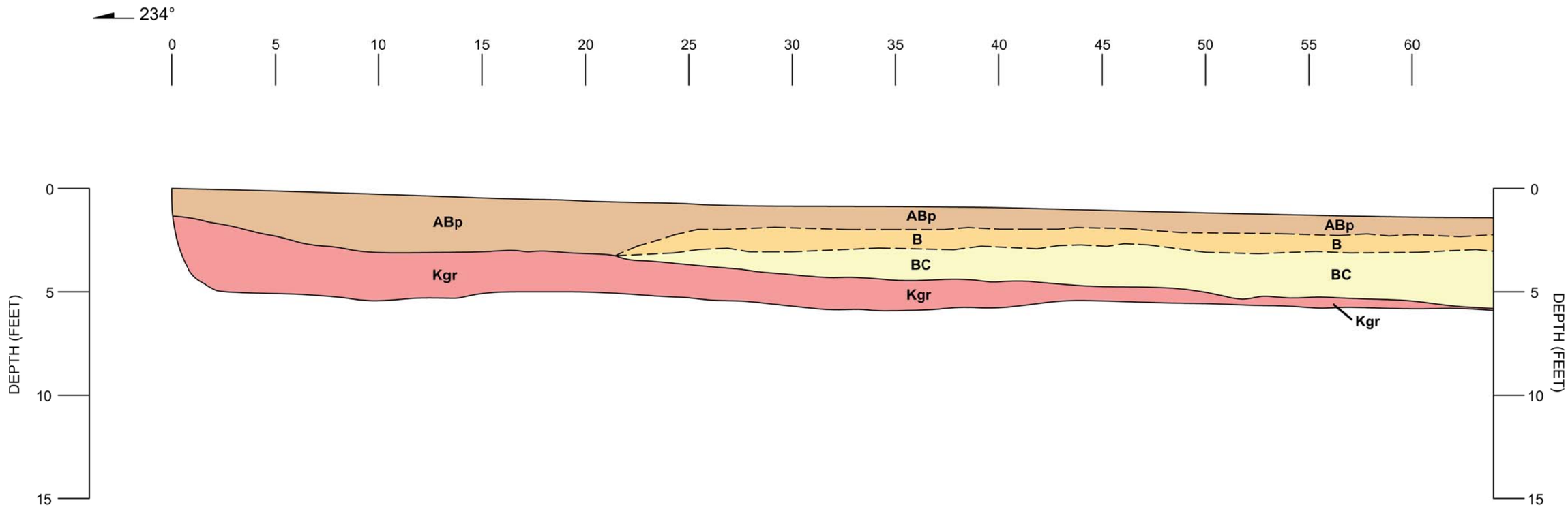
A-7
FIGURE

# APPENDIX B
# EXPLORATORY FAULT TRENCH LOGS




## STRATIGRAPHIC COLUMN AND KEY TO LOG SYMBOLS

**ABp** — SILTY SAND - (Late Holocene)
Brown (7.5YR 4/2) to Dark Grayish Brown, loose to medium dense, dry to moist, faint platy structure, abundant roots, locally contains man-made debris, garbage [MIXED FILL/COLLUVIUM]

**B** — SAND WITH SILT (Late Holocene)
Light gray (7.5YR 7/1), medium dense, faint granular to platy structure, abundant random tubular pores, some rootlets, note upper contact marked by stone line of occasional brown weathered granite cobbles [MIXED COLLUVIUM/ALLUVIUM]

**BC** — CLAYEY SAND (Late Pleistocene to Late Holocene)
Dark gray (2.5Y 4/1) with local brown and orange mottling, medium dense to dense, wet, rare fine granular granite gravels, faint horizontal lamination [ALLUVIUM]

**Kgr** — GRANITE (Cretaceous)
White to light gray, mottled brown and orange, completely weathered, friable. Note minor perched groundwater at upper contact. [SALINIAN BLOCK BEDROCK]

——— GEOLOGIC CONTACT, SHARP
------ GEOLOGIC CONTACT, GRADATIONAL

SCALE
0 2.5 5 10 FEET

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FAULT TRENCH T-1 - LOG OF NORTH WALL

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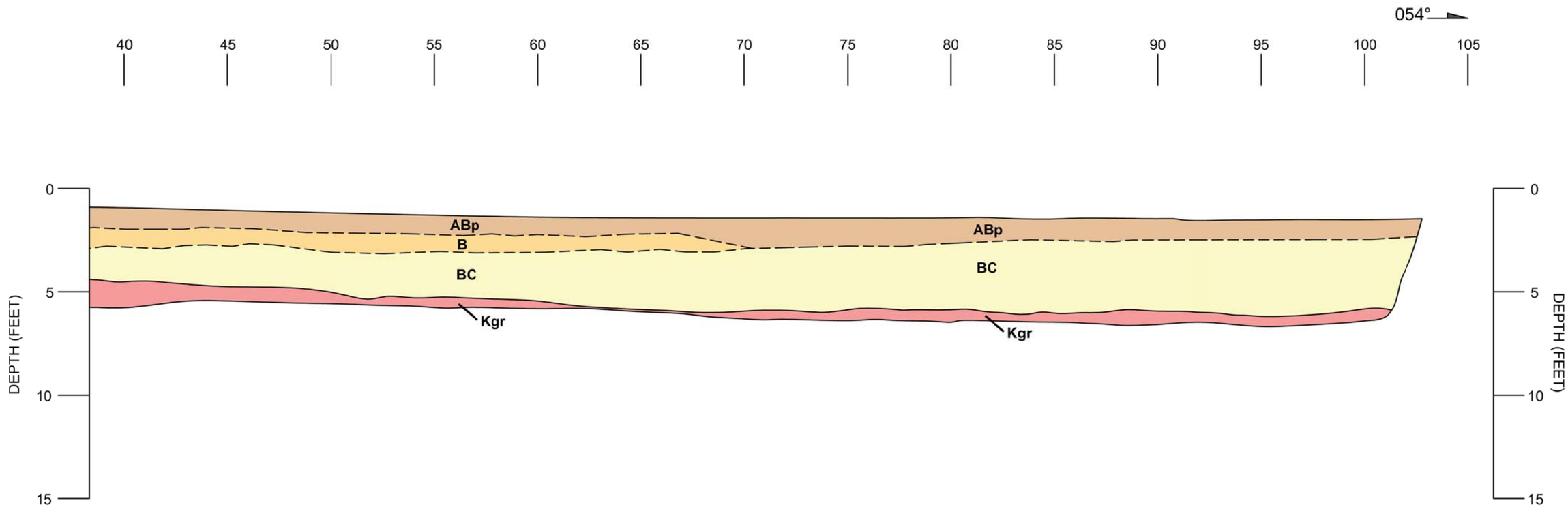
Inverness, California

Project No. 2626.001 Date: 9/7/18

Drawn MFJ
Checked

B-1
FIGURE




## STRATIGRAPHIC COLUMN AND KEY TO LOG SYMBOLS

**ABp** — SILTY SAND - (Late Holocene)
Brown (7.5YR 4/2) to Dark Grayish Brown, loose to medium dense, dry to moist, faint platy structure, abundant roots, locally contains man-made debris, garbage [MIXED FILL/COLLUVIUM]

**B** — SAND WITH SILT (Late Holocene)
Light gray (7.5YR 7/1), medium dense, faint granular to platy structure, abundant random tubular pores, some rootlets, note upper contact marked by stone line of occasional brown weathered granite cobbles [MIXED COLLUVIUM/ALLUVIUM]

**BC** — CLAYEY SAND (Late Pleistocene to Late Holocene)
Dark gray (2.5Y 4/1) with local brown and orange mottling, medium dense to dense, wet, rare fine granular granite gravels, faint horizontal lamination [ALLUVIUM]

**Kgr** — GRANITE (Cretaceous)
White to light gray, mottled brown and orange, completely weathered, friable. Note minor perched groundwater at upper contact. [SALINIAN BLOCK BEDROCK]

——— GEOLOGIC CONTACT, SHARP
------ GEOLOGIC CONTACT, GRADATIONAL

SCALE
0 2.5 5 10 FEET

| | | |
|---|---|---|
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| | | Drawn MFJ, Checked |
| | | B-2 FIGURE |