GEOTECHNICAL STUDIES AND FAULT HAZARD INVESTIGATION POLICE DEPARTMENT BUILDING SITE ARLINGTON AVENUE, APN: 572-040-011 KENSINGTON, CALIFORNIA

Prepared for:
David Aranda, Kensington Police Protection and Community Services District
10940 San Pablo Avenue
El Cerrito, California

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GEOTECHNICAL CONSULTANTS

September 23, 2024

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September 23, 2024
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David Aranda
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10940 San Pablo Avenue
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RE: Geotechnical Studies and Fault Hazard Investigation
Police Department Building Site
Arlington Avenue, APN: 572-040-011
Kensington, California

Dear Mr. Aranda:

At your request, we have performed geotechnical studies and a fault hazard investigation for a possible new Kensington Police Protection & Community Services District (KPPCSD) facility. The site being considered is a large vacant property located along the uphill (east) side of Arlington Avenue, south of the Kensington Library, and west or downslope of the main portion of Kensington Park as shown on the attached Vicinity Map, Figure 1. The property has the following coordinates: Latitude 37.9127, Longitude -122.2809.

It should be noted that we previously performed an initial geotechnical/geological study of the site based on literature review and a site reconnaissance. The results of the previous study were summarized in our report sent to you dated November 15, 2023. Generally, we concluded that the primary geologic and geotechnical hazards to potentially impact the project were landsliding and fault rupture.

This current investigation has been performed jointly by Alan Kropp & Associates (AKA) and Earth Focus Geological Services (EFGS). The emphasis of EFGS was on the geologic characterization of the site along with the fault and geologic hazard assessments. AKA was primarily focused on the geotechnical elements and geotechnical design recommendations.

1.00 PROPOSED CONSTRUCTION

No conceptual plans for the site have been developed for the project, but the facility will likely include a main building (with perhaps 3,000 square feet of floor space over one or two stories), parking for 15 vehicles, and an access driveway from Arlington Avenue. Given the sloping terrain within the site, significant excavations into the southwest-sloping hillside and grading will likely be necessary.

Based on conversations with your architect (Mallory Cusenbery), two conceptual development locations on the site are currently being considered; both of these locations are within the northern half of the site. The actual layout of structures, parking areas, and access routes, along with the associated grading, has not been established. In our current study, we did not evaluate the southern half of the site nor did we study the upper east portion of the property nearest the main portion of Kensington Park above the 655-foot contour as shown on the Site Plan, Figure 2; if future plans involve development in these areas, additional geotechnical/geologic studies will be needed. We should note that, depending on our review of the actual project design once plans become available, supplemental subsurface exploration may be necessary in areas and to depths not previously investigated.

2.00 GEOLOGIC AND GEOTECHNICAL EMPHASES

2.01 Geologic Emphasis

The site is located within the Alquist-Priolo Earthquake Fault Zone (APEFZ) established by the State of California for the seismically active Hayward fault as shown on Figure 3 (California Geological Survey, 1982). Therefore, a fault investigation conforming to the standards of the California Geologic Survey had to be performed to clear the proposed building area of the site. In addition, some concerns regarding slope stability at the site have been raised. The site is also included within a State of California Seismic Hazard Zone for Earthquake-Induced Landslides as shown on Figure 4 (California Geological Survey, 2024a). Our assessment of the site presented here follows general guidelines established by California Geologic Survey Special Publication 117A (2008). Therefore, fault evaluation and landslide assessment have been performed by Certified Engineering Geologist, Patrick Drumm, for EFGS.

2.02 Geotechnical Emphasis

In addition to the geologic elements, we also performed geotechnical exploration of the site to guide the new building, parking areas, and driveway development. Since this is a sloping site, our investigation included an assessment of the hillside stability consequences of grading including any excavations into the hillside that will require retaining wall construction. Thus, this study provides geotechnical design input for lateral force input into the structural design of retaining walls, site grading, and typical recommendations for building foundations and floor slab subgrade, exterior flatwork, and the parking lot subgrade.

3.00 PURPOSE OF WORK

The purposes of our services were to evaluate critical geologic and geotechnical issues that might exist at the site. These included:

- Assess whether or not an active fault passes through the site, which would therefore dictate the need for a building setback zone on the property;

- Study potential concerns related to possible slope stability on the site and develop any appropriate remedial work needed; and

- Evaluate the geotechnical conditions in the areas for the proposed building, parking areas, and access routes in order to develop geotechnical recommendations for the building and site improvements (excluding pavement design for parking lots).

4.00 SCOPE OF WORK

As described in our proposal dated May 21, 2024, our scope of work generally included:

- Review the data compiled during our initial site assessment;
- Review selected published geologic maps and reports;
- Review available consultant reports for the immediate area;
- Perform a reconnaissance of the site and vicinity to observe current site conditions and evidence of fault locations and other geologic conditions;
- Drill four exploratory borings at the site;
- Perform laboratory tests on recovered soil samples from borings;
- Excavate an exploratory fault trench;
- Perform detailed geologic logging of the trench by our Certified Engineering Geologist;
- Interact with the Contra Costa County peer reviewing geologist;
- Consultations as needed with KPPCSD and the project team; and
- Preparation of a detailed geotechnical and geologic report with supporting figures.

5.00 SITE INVESTIGATION

5.01 Existing Geologic and Geotechnical Data Review

A variety of published sources were reviewed to evaluate geologic and geotechnical data relevant to the site. These sources included geologic and geotechnical literature, reports, and maps published by various public agencies, along with unpublished fault and geotechnical data from private consultants, including a previous nearby fault evaluation report by us. Maps that were reviewed included topographic, geologic, and preliminary photo-interpretive landslide maps prepared by the U.S. Geological Survey, as well as geologic, landslide, and fault maps prepared by the California Geological Survey. A list of the published and unpublished sources used in our investigation is presented at the end of this report.

5.01.1 Regional Geology

The site is located in the Berkeley Hills, east of San Francisco Bay, in the northern portion of the Coast Ranges geomorphic province of California. Northwest-southeast-trending mountain ranges and valleys that generally parallel the major geologic structures, such as the San Andreas and the nearby Hayward-Rodgers Creek faults, characterize the Coast Ranges geomorphic province. The oldest widespread rocks in the region are highly deformed sedimentary, igneous, and metamorphic rocks of the Franciscan Complex of

Cretaceous to Late Jurassic age (200 million to 65 million years before present). These rocks are in fault contact with both similar-age sedimentary rocks of the Knoxville Formation and also much younger sedimentary rock of the Orinda Formation of Late Miocene age (approximately 13 million years before present) as shown on Figure 5. These rocks have been extensively deformed by repeated episodes of folding and faulting (Dibblee, 1980 and 2005; Ellen and Wentworth, 1995; Graymer and others, 1996; and Radbruch and Case, 1967).

Within the region, many of the valley areas have been partially filled with unconsolidated sedimentary deposits of Quaternary age (the last 1.8 million years). These deposits, which include alluvium and colluvium, underlie the valley bottoms and generally consist of interbedded clay, silt, sand, and gravel. Individual landslides and larger landslide complexes have been mapped along the slope areas throughout the Berkeley Hills (Herd, 1978; and Nilsen, 1975). The natural landscape of the site area has been modified by residential development and the construction of streets 100 years ago or more, creating relatively steep cut slopes and scattered pockets of artificial fill materials. Some of the developed areas have been superimposed on pre-existing geologic hazards that were not recognized at the time.

5.01.2 Regional Seismicity

5.01.2.1 *Faults*

Seismic activity within the northern Coast Ranges is generally associated with active faults belonging to the San Andreas system of faults, including major active structures both east and west of the site, as shown on the Regional Active Fault Map of Figure 6. According to published maps, the project site is within close proximity to mapped traces of the northwest-southeast-trending Hayward-Rodgers Creek fault zone (Lienkaemper, 1992; Radbruch, 1967; and Radbruch-Hall, 1974). The site is included within an Alquist-Priolo Earthquake Fault Zone (APEFZ) for the Hayward-Rodgers Creek fault, as established by the State of California for seismically active faults and shown on Figure 3 (California Geological Survey, 1982; and Hart and Bryant, 1997).

The principal active faults in the region are the Hayward-Rodgers Creek fault, with the closest trace mapped approximately 300 feet southwest of the site (California Geological Survey, 1982); the San Andreas fault at approximately 19.2 miles to the west; and the Calaveras fault at approximately 13 miles to the southeast. Other major active faults in the region include the San Gregorio fault at approximately 21.5 miles to the west, the Greenville fault at approximately 18.5 miles east, the Concord-Green Valley fault at approximately 14 miles northeast, and the West Napa fault at approximately 18.5 miles northeast (Jennings and Bryant, 2010). Table 1 summarizes the fault parameters of known active faults closest to the site:

Table 1: Faults and Fault Parameters

Fault	Distance and Direction from Site[1]	Maximum Moment Magnitude
Hayward-Rodgers Creek	300 feet southwest	7.6
Calaveras (north of Calaveras Reservoir)	13.0 miles southeast	6.8
Concord-Green Valley	13.0 miles northeast	6.9
San Andreas (1906 rupture)	19.2 miles west	7.9
San Gregorio	21.0 miles west	7.3
Greenville	19.8 miles east	6.9
West Napa	17.4 miles northeast	6.5

[1] Measured from Lienkaemper (1992), Wagner et al. (1990), and Jennings and Bryant (2010).

The term “active fault,” as used herein, refers to a fault that has experienced movement during Holocene time or within the last 11,000 years (Hart and Bryant, 1997). The official State of California Alquist-Priolo Earthquake Fault Zone at this location depicted the active Hayward fault as consisting of a single inferred trace located downslope to the southwest of the site. Multiple traces within the fault zone have been mapped to the northwest and southeast of the site as shown on Figure 3. The single fault trace is approximately 300 feet to the southwest of the site (California Geological Survey, 1982).

Comparatively, a fault map prepared by the U.S. Geological Survey for recently active traces along the Hayward-Rodgers Creek fault shows one to two fault traces located downslope to the southwest of the site as shown on Figure 7. The closest fault trace is approximately 400 feet to the west (Lienkaemper, 1992). This map combines site-specific fault exploration data performed by private consultants for nearby properties with geologic interpretations of geomorphic characteristics consistent with active fault-related features. Not all of the possible fault traces associated with the Hayward-Rodgers Creek fault zone are included on this map.

The Hayward fault segment of the Hayward-Rodgers Creek fault zone is a northwest-trending zone about 70 miles long, which extends from southeastern San Jose, through multiple East Bay communities, and into San Pablo Bay. Beneath San Pablo Bay, the fault is believed to step to the right (east), continuing north as the Rodgers Creek segment into the Santa Rosa area. To the south, near San Jose, the Hayward-Rodgers Creek fault merges with the Calaveras fault (Jennings and Bryant, 2010). The Hayward-Rodgers Creek fault last ruptured along the southern segment near Castro Valley in a major earthquake in 1868 will be discussed later in our report. The average recurrence interval of 161 (± 65) years; it is considered to present a high rupture hazard in the Bay Area in the near future (Lienkaemper and others, 2012).

Well-documented surface fault creep has occurred along the Hayward-Rodgers Creek fault at average rates ranging from about 0.2 to 0.4 inches per year (Lienkaemper and others, 1991). More recently, there has been recognition of variability in creep rates, both spatially along the fault trace and temporally. Lienkaemper and others (2012) describe several discrete fault segments that have experienced increased or decreased creep rates since the 1989 Loma Prieta Earthquake, including one apparent locked segment that may indicate it to be the next segment to rupture.

Studies by the U.S. Geological Survey’s Working Group on California Earthquake Probabilities (Aagaard and others, 2016) have estimated a 72-percent probability that at least one magnitude-6.7-or-greater earthquake will occur in the San Francisco Bay Region before the year 2043. They estimated that the highest probability for a magnitude-6.7-or-greater earthquake would be on the Hayward-Rodgers Creek fault, at 33 percent. Additionally, there is a 22 percent probability for a magnitude-6.7-or-greater earthquake to occur on the Northern San Andreas fault, located approximately 19.2 miles to the west, and 16 percent probability for a magnitude-6.7-or-greater earthquake to occur on the Concord fault, located approximately 13 miles to the northeast, during that same period.

5.01.2.2 *Previous Nearby Fault Investigation Studies*

In 2008, we prepared a fault evaluation study for the nearby Kensington Elementary School as part of our scope of work. We reviewed available unpublished reports, maps, and exploratory logs prepared by private consultants for previous fault investigations (Alquist-Priolo) within the State of California Earthquake Fault Zone for sites near the Kensington Elementary School (see Figure 1). Our report for the Kensington Elementary School site, dated March 8, 2008, included a review of eleven stereo-pairs of historic aerial photographs dating from 1946 through 2002. Most of these previous fault evaluation studies reviewed targeted the mapped traces of the Hayward fault zone downslope and west of Arlington Avenue. None of these previous fault investigations encountered any significant evidence for active faulting within the various sites studied. Our review of historic aerial photographs did not show any obvious evidence of active fault features near the Kensington Elementary School site or near the current project site.

The closest of these previous fault evaluation studies to the site was conducted in 1988 by Durham, Durham, and Mannion for the Kensington Youth Hut. In addition to reviewing several past geologic investigations for nearby sites and reviewing historic aerial photographs, the researchers logged four backhoe pits excavated in an alignment along the south side of Kensington Road perpendicular to the orientation of the Hayward fault zone. The study was supplemented by 12 auger bore holes surrounding the Youth Hut. The researchers concluded that no active fault crossed the Kensington Youth Hut site. Instead, a non-fault contact was identified between poorly consolidated sedimentary rocks and greenstone that had previously been mapped as a fault.

Additionally, the researchers for the Kensington Youth Hut proposed that a curved northwest-trending fault was passing through our current project site separating greenstone on the south from serpentinite on the north. This fault was suggested based on a review of aerial photographs that apparently indicated tonal variations in the ground surface thought to be the result of two different lithologies. They also mapped another more continuous fault along Windsor Avenue upslope to the east of the current project site (Durham and others, 1988).

5.01.3 Historic Seismicity

The San Francisco Bay Region has experienced several large earthquakes during historical time. A summary of the more significant earthquakes in the region is given below.

1. <u>The Hayward Earthquake of October 21, 1868</u>

   On October 21, 1868, an earthquake of about M 6.8 occurred on the southern segment of the Hayward-Rodgers Creek fault, causing significant damage throughout the region. Surface ground rupture occurred over a length of approximately 30 miles. The northern limit of ground rupture was in the vicinity of Mills College. The epicenter of the 1868 earthquake was located in the Castro Valley area (Goter, 1988; and Stover and Coffman, 1993).

2. <u>The 1858 and 1911 Earthquakes</u>

   Two other earthquakes greater than M 6 are thought to have occurred on the Hayward-Rodgers Creek fault (Steinbrugge and others, 1986). These occurred in 1858 (M 6.1) and 1911 (M 6.6). Both of these earthquakes were centered in or near the southern portion of the Hayward fault (Stover and Coffman, 1993).

3. <u>The San Francisco Earthquake of April 18, 1906</u>

   The largest historical earthquake in the region was the great San Francisco earthquake of April 18, 1906, which occurred on the San Andreas fault near San Francisco. This earthquake caused strong-to-violent ground shaking throughout much of west-central California and caused widespread damage. It is estimated to have been of M 8.3 (Stover and Coffman, 1996).

4. The Loma Prieta Earthquake of October 17, 1989

On October 17, 1989, the M 7.1 Loma Prieta earthquake occurred near the San Andreas fault in the Santa Cruz Mountains. The earthquake resulted in 63 deaths and approximately $6 billion dollars in damage over a wide area (McNutt and Sydnor, 1990; and Stover and Coffman, 1993). Moderate ground shaking was felt in the Kensington area.

5. Napa Earthquake of August 24, 2014

The Napa earthquake (M 6.0) occurred on a previously unmapped fault near the City of Napa. Although the earthquake was felt widely throughout the region, only one fatality occurred. Damage in the city of Napa was estimated at $360,000,000. The earthquake resulted in a ground rupture that extended south as far as the Napa Airport. Ground shaking in the Berkeley area was moderate (Brocher and others, 2015).

5.01.4 Geologic Literature Review

5.01.4.1 *Nearby Faults*

In addition to the Hayward-Rodgers Creek fault zone, there are other minor faults mapped throughout the Berkeley Hills near the project site. The most continuous of these other faults include the northwest-southeast-trending Wildcat fault mapped along Wildcat Canyon located approximately 2,200 feet to the northeast (Dibblee, 1980 and 2005). None of these other faults, including the Wildcat fault, have been recognized as being seismically active (Hart and Bryant, 1997).

5.01.4.2 *Bedrock*

The subject site is on the west flank of the Berkeley Hills, which are underlain by a variety of bedrock types that range from Jurassic to Quaternary in age. The Hayward-Rodgers Creek fault zone has severely complicated the identification and geologic mapping of bedrock units within the site area. Bedrock mapped in the site vicinity generally consists of igneous and metamorphic rocks assigned to the Franciscan Complex (see Figure 5). The Franciscan Complex is a widespread collection of rocks that were formed within a subduction zone and later accreted to the west coast of California during Late Jurassic to Early Cretaceous time. The Franciscan Complex rock types exposed in bold outcrops along this portion of the East Bay Hills have generally been mapped as relatively hard greenstone, graywacke sandstone, and chert. The softer bedrock within the Franciscan Complex that is not well exposed and is generally considered to be a mélange typically consisting of hard rock blocks within a sheared fine-grained matrix (Bishop and others, 1973; Dibblee, 1980 and 2005; Ellen and Wentworth, 1995; and Graymer and others, 1994).

Depending on the published geologic maps reviewed, the site and adjacent properties are directly underlain by a fault bounded sliver of serpentinite bedrock or a mélange of serpentinite that include other rock types within a sheared matrix. Relatively hard keratophyre, a related intrusive igneous rock, is mapped to the southeast. Marine sedimentary rocks of similar age assigned to the Knoxville Formation consisting of shale and sandstone are mapped to the south. Younger nonmarine sedimentary rocks of the Orinda Formation generally consisting of mudstone, siltstone, sandstone, and conglomerate are mapped upslope to the east (Bishop and others, 1973; Dibblee, 1980 and 2005; Ellen and Wentworth, 1995; Graymer and others, 1996; and Radbruch and Case, 1967).

5.01.4.3 *Landslides*

There have been a variety of landslide maps compiled for the site area, each with different objectives. Several individual landslides and landslide complexes are mapped along the west flank of the Berkeley Hills near the site (Dibblee, 1980 and 2005). A published landslide map by the U.S. Geological Survey suggests that the west flank of the hills is affected by a questionable large landslide complex that begins at the top of the ridge near Highland Boulevard and extends downslope where the regional slope flattens. The project site is within the upper portion of this questionable large landslide complex (see Figure 9, Nilsen, 1975). Comparatively, another published landslide map by the U.S. Geological Survey shows that the site is within an enormous ancient landslide that covers the entire west flank of the hills and for at least 1 mile to the northwest and more than 1 mile to the southeast of the site (Herd, 1978).

According to recent published information by the California Geological Survey, no landslides have been mapped within or adjacent to the site (Plate 1.2, California Geological Survey, 2024b). An older map also published by the California Geological Survey (1973), then known as the California Division of Mines and Geology, shows that the site is not within a landslide, but adjacent to a large landslide to the northwest as shown on Figure 8. The slopes within and adjacent to the site between Arlington Avenue and Kensington Park are included within a State of California Seismic Hazard Zone for Earthquake-Induced Landslides as depicted on Figure 4 (California Geological Survey, 2024a).

A more detailed landslide map for the site vicinity has been prepared by us, based on an extensive working knowledge of the site area (Alan Kropp & Associates, 1995). We have identified “active” and “potentially active” landslide masses and shown these in red and yellow, respectively. Our map suggests that the project site is near the upper portion of a potentially active landslide that continues farther downslope to the west. The lateral margin of this potentially active landslide is to the northwest of the site.

More recently, landslide research has focused on the identification of actively moving landslides along the west flank of the Berkeley Hills, based on high-resolution interferometric synthetic aperture radar (InSAR) from satellites (Hilley and others, 2004). Movement is measured in very small increments of millimeters per year from data collected in the early 1990s through the early 2000s. The results are plotted in a range of colors from blue to red with red identifying areas of where movement is pronounced. The map shows definitive, although very small, movements within the site vicinity south of Arlington Avenue. No such movements are suggested within or adjacent to the site.

5.02 <u>Geologic Site Reconnaissance and Mapping</u>

We visited the project site several times prior to the boring and trench exploration phase of the project to observe the site conditions, identify any obvious slope instability and active fault features, and geologically map bedrock exposures. The proposed site is below and to the west of the main portion of Kensington Park. The park generally consists of an extensive level area with tennis courts, play areas, picnic tables, restroom facilities, and a paved cut-de-sac access off of Windsor Avenue. It appears that the level park area was created by excavating into the natural slope. We observed that portions of the upper southeast-facing slope below the picnic areas had also been graded in the past to form a level bench area that follows the contours of the natural slope. The lower shoulder of this bench is underlain by artificial fill as shown on Figure 2. The slope area of the site to the north within the trees is littered with boulders that appeared to have been pushed over the slope during of the park area (see Figure 2).

Below the park facilities and at the north end of the site, we observed scattered boulders of various sizes amongst the trees. The boulders were relatively hard and thought to be possibly keratophyre (see Figure 2). Even though some of these boulders were partially buried in the ground, it is unlikely that the boulders represent the underlying bedrock. Since some of the boulders were above the ground surface and located behind tree trunks suggests that the boulders were generated by the grading and development of the park and most likely were pushed over the side of the slope.

Serpentinite bedrock is exposed in the near-vertical road cut along the upslope side of Arlington Avenue. We geologically mapped the road cut and two isolated exposures of serpentinite bedrock along the slope areas within and near the site. The bedrock was observed to be relatively hard, closely fractured, and exhibited a prominent foliation striking northwest and dipping to the northeast from 50 to 75 degrees. The locations of bedrock exposures are shown on Figure 2. Serpentinite bedrock was also exposed in a cut slope behind the restroom facilities within Kensington Park upslope to the east of the site.

During our geologic mapping, we observed an accumulation of rock fragments along the base of the road cut adjacent to Arlington Avenue from erosion of the cut slope. We did not observe any obvious signs of active landsliding within the site, such as ground cracks/scarps, depressions, or ponding water. However, the terrain at the extreme south end of the property labeled “Not A Part” on Figure 2 exhibited hummocky topography consistent with landslide terrain. This portion of the property does not affect the proposed site. Further, we did not identify any obvious features consistent with active faulting within or adjacent to the site, such as offset curbs or breaks in the topography.

Across Arlington Avenue to the west of the site, we walked along Arlington Court that extends to the southwest. The active trace of the Hayward fault zone has been mapped at this location. We observed a prominent dip in Arlington Court west of the intersection that coincides with a linear depression occupied by residences. Curbs and gutters along the dip segment of Arlington Court appeared to have been replaced sometime in the past and do not show any obvious evidence of offset from fault creep.

5.03 Site Conditions

The site consists of a southwest-facing slope between Arlington Avenue and the main portion of Kensington Park. The slope is generally grass covered. There are trees that outline the top of the slope along the edge of the park facilities. Relatively dense tree cover conceals the slope area at the north end of the site. The top of the road cut above Arlington Avenue is lined with trees. The most prominent geomorphic feature is a southwest-draining ravine within the northern portion of the site as shown on Figure 2.

The total relief across the site from Arlington Avenue to edge of the level park area is approximately 70 feet. The average slope gradient within the site is approximately 3:1 (horizontal to vertical). The slope flattens to approximately 4:1 near Arlington Avenue. There are locally steeper and flatter slopes throughout the site. The road cut along Arlington adjacent to the south portion of the site averages approximately 1.5:1 with some near vertical portions held into place with tree roots. A short wood retaining wall up to 2 feet high supports the lower portion of the cut slope to the south of the site.

5.04 Exploratory Trench Excavation

Prior to any excavations within the site, we notified Underground Service Alert (U.S.A.). Between July 12 and 14, 2024, we geologically logged a continuous exploratory trench excavated within the project site between the upper graded bench and Arlington Avenue at the approximate location shown on Figure 2. The trench was oriented approximately perpendicular to the mapped trace of the Hayward fault zone. The trench was excavated using a Caterpillar mini-excavator with a 36-inch bucket. The excavation was approximately 130 feet long and up to 7 feet deep. The trench excavation was shored for safety, the walls cleaned by hand, and a string line datum with station numbers was used to log the trench.

We logged the south wall of the trench and produced a graphic log of the exposure at a scale of 1 inch = 2 feet (see Plate 1). After logging was complete, we invited Mr. Darwin Myers, the Contra Costa County peer reviewing geologist, to observe and comment on the trench exposures. Mr. Myers visited the project site on July 15, 2024. He will also provide a review and written comments on this final report.

The trench excavation exposed a small wedge of artificial fill along the upper portion of the slope associated with the graded bench and an isolated fill area near the lower central portion. A sequence of native surficial soils was capping bedrock throughout the entire length of the trench. The native soils were 1 to 2 feet thick along the upper portion of the slope and they thickened to over 6 feet deep in the lower portion of the trench (see Figure A-3).

The bedrock exposed within the trench was consistently serpentinite with variations in color, weathering, strength characteristics, and degree of fracturing. The top of the bedrock surface dove below the bottom of the trench excavation at Station 108. The prominent foliations measured within the bedrock was striking northwest and northeast and dipping to the southwest and southeast from 43 to 76 degrees. The prominent joint planes were measured as striking northwest and northeast and dipping at moderate to steep inclination in all directions (see Figure 2).

No obvious signs of active faulting were identified within the soils or bedrock. However, two old shear zones within the bedrock were identified between Stations 12 and 20, and Stations 28 and 38. Seepage and ponding water was encountered in the trench associated with the shear zone at Station 28. A drafted version of our graphic field log from the trench is reproduced as Figure A-3. Below is a brief summary of our observations and interpretations of the trench exposure with horizontal station references.

5.04.1 Soil and Bedrock Descriptions from the Trench Excavation

Artificial Fill (af): The artificial fill materials encountered from Stations 0 to 10 and from Stations 78 to 85 were up to 2 feet thick. The fill generally consisted of lean clay with sand and rock fragments. The fill was slightly moist and generally in a firm condition with some loose gravel zones. The fill was placed directly on the native soils.

Native Soils (AB1, AB2, AB3, AB4, and AB5): We observed a continuous layered sequence of surficial soils generally consisting of fat clay with variable amounts of fine sand, fine gravel, and some small rock fragments. The thickness of the soils ranged from 1 to 2 feet in the upper portion of the slope and thickened to more than 6 feet along the lower portion of the slope closest to Arlington Avenue. The soil horizons were sloping down toward the west and subparallel to the ground slope. Individual soil layers were distinguished from one another based on color, amount of gravel/rock fragments, relative moisture content, consistency, and the presence of clay films. Discontinuous shear planes were observed in the lower portions of the soil sequence where the soils were moist and soft to firm approximately 3.5 to 4 feet below the ground surface as shown on Plate 1. Desiccation cracks were observed in the upper 2 feet closest to the ground surface where the soils were slightly moist and stiff.

Serpentinite Bedrock (Units 1 through 8): Below the surficial soils, we encountered a variety of serpentinite bedrock that differed in color, degree of weathering, field hardness, and fracture frequency. Most of the bedrock logged in the trench was slightly weathered and relatively hard. The fracturing created cobble and boulder size rock fragments. Some boulders excavated from the trench were up to 18 inches diameter. Fractures surfaces were typically iron stained. Some of the fractures within Unit 8 contained fibrous mineralization. This mineralization may be natural occurring asbestos (NOA).

Old Shear Zones (Unit 3): We observed two relatively soft bands of serpentinite separating relatively hard serpentinite bedrock between Stations 12 and 22, and between Stations 28 and 38. The soft serpentinite was internally sheared and contained some polished hard serpentinite cobbles and a single large, hard boulder. We interpret these soft serpentinite units to be old shear zones. Attitudes measured on the shear zones were northeast-striking and southeast-dipping from 18 to 38 degrees. We did not observe any obvious offsets or displacements in the overlying soils that would suggest that the shear zones were seismically active or part of the Hayward fault zone.

5.04.2 Trench Backfill

After completion of the geologic logging, the trench was loosely backfilled with the excavated materials, watered with a hose connected to a portable tank, and track-walked. An erosion control mat was placed along the backfilled ground surface. We should point out that the trench backfill would not be compacted to engineering standards, and some settlement of the backfill should be anticipated. In the event that engineered compacted backfill is considered necessary, we can provide an estimate for the cost of compaction and testing. It should be noted that the cost of compacting the backfill at the end of the trenching program would be high. In general, it is typically more cost effective to postpone compaction until project construction begins, when suitable equipment will be on-site.

5.05 Exploratory Borings

On July 15, 2024, we explored the subsurface conditions surrounding the trench at the project site by drilling four exploratory test borings at the approximate locations shown on the attached Site Plan, Figure 2. Portable drilling equipment outfitted with 4.5-inch solid stem augers and a tripod for sampling was used to advance the borings to depths ranging from 3.0 feet to 8.5 feet. The portable drilling equipment could not penetrate the hard bedrock to any significant depth. After drilling was completed, all of the boreholes were backfilled with cement grout.

During drilling, our field representative monitored the advancement of the drilling and made notes of any changes in drilling conditions that were observed or commented on by the driller. Samples of soil and rock were obtained using 2-inch O.D. Standard Penetration Test (SPT) and 3-inch O.D. Modified California samplers. The samplers used during drilling were driven using a 140-pound trip hammer. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the attached boring log.

The materials encountered in the borings generally consisted of lean to fat clay with variable amounts of sand and gravel. Approximately 3 feet of native soils resting on bedrock were logged in the upper and middle slope areas in Borings B-1 and B-2. Thicker native soils were encountered in the lower slope nearest Arlington Avenue in Borings B-3 and B-4. The bedrock encountered in the last foot of Boring 4 (a depth of 6 to 7 feet) may have been displaced by an old landslide.

Detailed descriptions of the materials encountered in the borings are found on the boring logs presented in the attached Appendix A. A Key to Exploratory Boring Logs, Figure A-1, and Physical Properties Criteria for Rock Descriptions, Figure A-2, are also presented in Appendix A. The attached logs and related information depict subsurface conditions only at the specific locations shown on the Site Plan and on the particular date designated on the logs. The logs may have been modified from the original logs recorded during drilling as a result of further study of the collected samples, laboratory tests, or other efforts. Also, the passage of time may result in changes in the subsurface conditions, due to environmental changes. The locations of the borings were approximately determined by hand tape measurement from existing field landmarks, and the ground surface elevations at each boring location were approximately determined by interpolation of topographic map contours. The locations and elevations should be considered accurate only to the degree implied by the method used.

5.06 Groundwater

Groundwater was not encountered in the exploratory borings at the time of drilling. The borings were grouted immediately after drilling, in accordance with Contra Costa County drilling permit requirements. It should be noted that groundwater measurements in the borings may have been made prior to allowing a sufficient period of time for the equilibrium groundwater conditions to become established. In addition, fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors not evident at the time the measurements were made. Due to the sloping nature of the terrain, it is our opinion that seepage could occur in excavations and behind retaining walls, particularly after prolonged rains during a relatively heavy rainy season.

5.07 Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. The following geotechnical laboratory tests were performed on selected soil samples in general accordance with the listed ASTM standard:

- Water content per ASTM Test Designation D-2216;
- Dry density per ASTM Test Designation D-2937;
- Atterberg Limits per ASTM Test Designation D-4318; and
- Percent passing No. 200 sieve per ASTM Test Designation D-1140.

The water content and dry density tests were performed to evaluate the variations in soil moisture and the soil's in place density, respectively. The Atterberg Limits tests were performed to evaluate the soil's expansive potential. The results of the percentage passing the No. 200 sieve tests were used to aid in the classification of the soils. The results of these tests are presented on the boring logs at the appropriate sample depths.

An Atterberg Limits analysis was conducted on a sample of the sandy clay at a depth of approximately 3½ feet in Boring 3. This analysis indicated a material with a Plasticity Index (PI) of 52 and a Liquid Limit (LL) of 72. This is indicative of a material with a very high expansion potential and a corresponding very high potential for shrink/swell behavior with changes in moisture content. There are also some clay soils on the site with a much lower expansion potential, but our design will be conservative in nature and focus on the most problematic clays present.

5.08 Interpretation of Subsurface Conditions

No evidence of active faulting was identified within the limits of the fault trench excavated through the portion of the property slated for development. The exploratory borings logged to the north of the trench excavation confirm the consistency of the bedrock lithology underlying the site. Our interpretation of the subsurface geologic conditions is depicted in a geologic cross section that passes through the site from the top of slope to Arlington Avenue utilizing our collected surface data as shown on Figure 10.

The project site is underlain by serpentinite bedrock as shown on published geologic maps. The bedrock consists of both relatively soft shear zones and friable units to hard fractured boulder size blocks. The structural fabric or foliation of the rocks is generally south-dipping (orientated parallel to the site slope) and southeast-dipping (orientated into the site slope). The foliation is generally favorable for gross slope stability.

The surficial soils up to 3 feet thick cap bedrock across the upper slopes of the site. The soils thicken in the lower portion of the site to depths of 6 to 8 feet as explored. The lower slope area closest to Arlington Avenue underlain by an accumulation of native soils more than 3 feet thick is outlined on Figure 2 and labeled colluvium. The native soils are expansive as is evident by the desiccation cracks observed along the ground surface and likely prone to downslope creep.

The upper portion of a possible old landslide may exist near Arlington Avenue. Evidence for the presence of this possible landslide is a drop in the top of the bedrock surface logged in the trench near Station 108 that coincided with a thickening of the native soils or colluvium. Boring B-4 drilled downslope of the west end of the fault trench confirmed the top of bedrock is abruptly lower than that explored upslope to the east. If present, this old landslide would continue offsite under Arlington Avenue to the southwest as shown on Figure 10.

6.00 EVALUATIONS AND CONCLUSIONS

6.01 General Site Suitability

Based on our research and subsurface investigation that was limited to the north portion of the property as previously discussed, it is our opinion that the project site is suitable for the proposed new Kensington Police Station from a geologic/geotechnical standpoint. However, all of the conclusions and recommendations presented in this report should be incorporated into the design and construction of the project to minimize possible geotechnical problems. The primary geologic hazards identified from our evaluation include long-term downslope creep of surficial soils, possible landsliding, potentially expansive surficial soils, bedrock with variable strength properties, and possible naturally occurring asbestos minerals within the bedrock.

Due to the moderate sloping nature of the site, significant grading will be required to develop the project. Stepped retaining walls are anticipated to support permanent excavations into the hillside. Excavated soil and rock materials will have to be hauled off of the site. Surface and subsurface drainage systems will have to be incorporated into the design and construction of the project.

A discussion of our evaluation of the potential site geologic hazards and primary geotechnical considerations for site development is presented below.

6.02 Potential Geologic and Seismic Hazards

6.02.1 Potential for Fault Rupture

Fault rupture is a direct seismic hazard where the ground surface ruptures or breaks along fault traces during earthquakes, and vertical and/or lateral displacements occur. During large earthquakes, such as the 1906 San Francisco earthquake, ground displacements of more than 10 feet have been documented. Because the Hayward-Rodgers Creek fault is predominately a strike-slip fault, the most likely ground displacement would be mainly lateral movements of the ground.

The nearest active trace of the Hayward-Rodgers Creek fault is mapped approximately 300 feet downslope to the west of the project site (see Figure 3). Due to the close proximity, the project site and surrounding areas are within a State of California Earthquake Fault Zone for seismically active faults as shown on Figures 3 and 4. All of the active traces for the Hayward-Rodgers Creek fault zone are mapped downslope to the west of the site along the west side of Arlington Avenue (State Geologist, 1982; and Lienkaemper, 1992).

Our research of previous fault evaluation studies and review of aerial photographs conducted for the nearby Kensington Elementary School in 2008 did not identify any active fault features passing through the current project site. During our many geologic reconnaissance visits to the site and adjacent properties, we did not observe any obvious signs of active faulting, such as offset curbs or distressed cultural features. The most prominent geomorphic feature consistent with active faulting near the site is the dip in Arlington Court along the west side of Arlington Avenue as previously mentioned.

We did not uncover any evidence of active faulting passing through the project site within the limits of our trench excavation. Our exploratory borings drilled north of the trench confirmed that the same serpentinite bedrock underlies the project site and precludes the presence of a possible fault shown in this area from the Kensington Youth Hut fault study by Durham and others (1988). It is therefore our opinion that the risk of fault rupture within the north portion of the site downslope and west of the 655-foot contour is low during the life of the proposed structure. We have not cleared the remainer of the property upslope and east of the 655-foot contour within the State of California Earthquake Fault Zone of active faulting.

6.02.2 Downslope Creep of Surficial Soils

Due to the highly expansive nature of some of the native soils and the presence of pockets of artificial fill along the slope, the site is prone to long-term downslope creep. These surficial soils include topsoil and colluvium that thicken toward the downslope portion of the property. These surficial soils encountered in our exploratory borings and trench excavation should not be relied upon for lateral foundation support of new structures if the foundations are located on sloping ground.

6.02.3 Landslide Hazard Evaluation

Some of the published geologic maps reviewed for this evaluation suggest that the site and surrounding areas are within a large landslide complex that begins near the top of the ridge to the east and continues to the base of the regional slope well to the west. Other published maps show that the project site is not within a landside, but located to the south of a landslide that continues downslope to the northwest. Further, the InSAR satellite data collected in the early 1990s through the early 2000s indicates no ground movements within or near the site during this time period.

We have identified a possible landslide in the lower portion of the slope near Arlington Avenue that, if present, would continue offsite under Arlington Avenue to the southwest. Evidence for the presence of this possible landslide is a drop in the top of the bedrock surface logged in the west end of the trench excavation that coincides with a thickening of the colluvial soils. It is our opinion that the risk of earthquake-induced landsliding to impact the proposed project is considered to be low provided our recommendations are incorporated into the design and construction of the proposed residence.

6.02.4 Earthquake Shaking

As noted earlier, the subject site is located in the highly seismic San Francisco Bay Area, and there is a strong probability that a moderate to severe earthquake will occur during the life of the structure. The proposed structure at the site will very likely experience strong ground shaking during a major earthquake in the life of the structure. The California Building Code has adopted provisions for incorporation of strong ground shaking into the design of all structures. Our recommendations for geotechnical parameters to be used in the structural seismic design of the addition and site improvements are presented in Section 7.08, "California Building Code Seismic Design Parameters."

6.02.5 Liquefaction

Liquefaction is a condition where soils undergo a sudden loss of strength during earthquake shaking. Soils prone to liquefaction include loose silts, sands, and gravels that are saturated. The project site is not within or adjacent to a State of California Seismic Hazard Zone for Liquefaction as shown on Figure 4. The site is underlain by clay soils underlain over shallow bedrock materials, and therefore, we believe that the risk of liquefaction is very low.

6.02.6 Naturally Occurring Asbestos (NOA)

The serpentinite bedrock contains a variety of serpentine minerals. The fibrous serpentine mineral, chrysotile, is the most common form of naturally occurring asbestos (NOA). Exposure to asbestos fibers have potential human health consequences (see CGS Note 14, 2002 in Appendix B).

Fibrous mineralization thought to be chrysotile was observed along some of the fracture surfaces in Unit 8 logged in the trench excavation. Determining whether or not the on-site bedrock contains asbestos minerals is beyond the scope of our work. Laboratory testing would be necessary to confirm the presence of NOA within the soils and bedrock underlying the project site. It should be noted that the area where there appeared to be chrysotile was very limited, but some special measures to control the consequences of this material during grading may become necessary.

6.03 Geotechnical Considerations

The primary considerations for geotechnical design at the site are:

- The shrink/swell behavior of the surficial soils and selection of an appropriate foundation type;
- Creep of soils on sloping ground;
- The presence of existing fill soils at the site;
- The presence of the existing ravine;
- Site drainage;
- Excavations into bedrock; and
- Future slope performance.

Each of these conditions is discussed individually below.

6.03.1 Foundation Selection

The Plasticity Index (PI) of a sample of the most expansive clay soils at the site was 52%, with a Liquid Limit of 72%. These values indicate these clays soils are very highly plasticity and with a corresponding very high potential for shrink/swell (expansive) behavior with changes in moisture content. After site grading occurs, portions of the proposed building pad will likely include areas of building support on in-place clays, clay fill from site excavations, and serpentinite bedrock. If conventional shallow footing foundations were constructed directly on clay materials, these foundations may undergo significant movements in the life of the structure while footings on bedrock man have minimal movement. In order to account for the building pad subgrade variability, we recommend the proposed building be supported on a relatively rigid mat slab foundation, underlain by a minimum of 36 inches of compacted non-expansive soil material.

We should also note that due to the highly expansive clay soils, we are concerned about these soils moving and causing damage to exterior flatwork. Therefore, we are recommending the exterior flatwork be underlain by a minimum of 18 inches of compacted non-expansive material.

6.03.2 Soil Creep on Sloping Ground

As noted above, some of the surficial soils at the site are highly expansive and prone to volume changes (shrinkage and swelling) with seasonal fluctuations in soil moisture. When these soils are located on moderate slopes, they have a tendency to creep downhill. Such movements can induce high loads on elements intended for lateral restraint (such as retaining walls, stairs, or posts), and damage the elements if not properly designed to accommodate the lateral forces.

6.03.3 Existing Fill Soils

Two areas of fill materials were encountered at the site (see Figure 2). We have not received any documentation to indicate these materials were placed under engineering observation. The fill materials appeared to be soft, and no ground preparation was noted below the fill materials in our trench logging. Therefore, we believe these materials could be subject to future settlement or lateral movements below the proposed building or on other portions of the site, and we recommend the fill materials be entirely removed by excavation. It is possible the excavated materials may be reused in new engineered fill at the site.

6.03.4 Existing Ravine

As seen on the Site Plan, there is a significant drainage ravine that extends through the project area. This ravine appears to have developed as a result of the discharge of runoff water from the park area uphill. The project civil engineer will need to develop a design to properly capture and discharge the water in the Arlington Avenue storm drain inlet. During site grading, all loose and moist soils from within the ravine will need to be removed and then the materials needed for the final development scheme properly placed back as engineered fill.

6.03.5 Site Drainage

To minimize infiltration of surface runoff at the site which results in weakening, along with shrink/swell behavior of the surficial soils, drainage should be carefully controlled in the entire development area. All water collected on impervious surfaces should be captured and conveyed to an appropriate discharge area, in accordance with proper bio-infiltration BMPs and standards such as C-3 provisions. No water should be allowed to flow from the any impervious area onto an adjacent slope.

6.03.6 Excavations into Bedrock

Excavations for our exploratory trench within the bedrock were difficult to remove using a Caterpillar mini-excavator with a 36-inch bucket. The fracturing within the serpentinite bedrock facilitated excavation and generated some large boulders. The sidewalls of the trench were relatively uneven due to the plucking of large rock fragments. It is our opinion that conventional equipment will be adequate to excavate the bedrock but relatively large blocks or boulders of hard bedrock should be anticipated as part of the excavation. In addition, loose boulders may be present on the site as a result of past grading operations uphill. No boulders should be placed within the engineered fill without establishing special locations in advance and designating possible disposal areas on the grading plan.

6.03.7 Future Slope Performance

It should be noted that because of the geologic setting and the sloping terrain, there is always the possibility for shallow sliding and erosion on the site. Such movements, if they do occur, may not seriously affect the performance of the facilities, but significant costs may be incurred to repair the area. The planting of erosion resistant vegetation and the installation and maintenance of the drainage recommendations on sloping areas as contained in this report will reduce the likelihood of such shallow instabilities.

We recommend that the drainage and erosion control recommendations we present in this geotechnical investigation report be carefully followed. In addition, we recommend these facilities be maintained once they are installed.

7.00 RECOMMENDATIONS

7.01 No Fault Setbacks

No evidence of active faulting was identified within the limits of our trench excavation and therefore, no fault setbacks for future construction are established for the north portion of the property defined between the 655-foot contour and Arlington Avenue as shown on Figure 2.

7.02 Site Clearing and Stripping

The portions of the site to be developed should first be cleared and stripped of vegetation and designated trees (including the root balls). These materials should be removed from the site or stockpiled for selective use in landscape areas. Where such materials are used in landscape areas, they should not be placed within 5 feet of improvements, should not have a thickness greater than 2 feet, and should be appropriately moisture conditioned and compacted.

Depressions created from the removal of trees, or other clearing and stripping operations, that are not fully removed by project grading should be backfilled with compacted engineered fill placed in accordance with the recommendations contained in the following sections of this report. It is recommended that we be present at the time of tree stump removal, and other clearing operations, so that we may confirm that all improper materials are adequately removed and that new fill is being placed over a properly prepared subgrade. In addition, test trench backfill areas not fully removed by project grading should also be removed and properly compacted under our observation.

7.03 Fill Composition and Placement

On-site soils and excavated bedrock materials can generally be used for engineered fill, except where the material has an organic content of greater than 3 percent by volume. All engineered fill placed at the site should typically not contain rocks or lumps greater than 6 inches in greatest dimension, with no more than 15 percent larger than 2.5 inches (except as discussed below). Imported fill material used at the site should have a Plasticity Index (PI) of 15 or less, including the placement of non-engineered fill below the building mat foundation. Fill materials should be placed on a firm, unyielding subgrade and should generally be placed in lifts not exceeding 8 inches in uncompacted thickness. All fill slopes (as well as cut slopes) should be constructed with finish slopes no steeper than 2½:1 (horizontal to vertical).

Excavated rocky materials up to 18 inches may be selectively placed in designated disposal areas shown on the grading plan if approved by our representative in the field. The oversized materials should be well mixed with other materials less than 3 inches in size, and not placed in direct contact with other oversized materials to reduce the likelihood of voids developing between the individual pieces of rock. No oversized material should be placed within 15 feet of a slope face.

Where fill is placed on sloping ground with a gradient of greater of 5:1, a typical keyway and bench system should be excavated into the hill to support the fill, as shown on Figure 11, Fill Placement on Slopes.

As noted previously, some special provisions may be necessary where serpentinite bedrock is excavated and it contains naturally occurring asbestos minerals within the bedrock. These provisions will be applicable to both the situation where the materials are placed in the engineered fill materials as well as when the materials are off-hauled from the site.

7.04 Fill Compaction

The materials used for engineered fill should generally be compacted to at least 90% relative compaction (ASTM Test Designation D-1557, latest edition), and the upper 6 inches of subgrade soils below pavement areas and building areas compacted to at least 95% relative compaction. The surface of all new fill slopes should be either back-rolled or compacted beyond the limits of the slope and back cut in order to achieve satisfactory compaction.

7.05 Fill Subdrainage

As shown on Figure 11, subdrainage should be installed at the base of grading keyways on slopes and on selected benches as part of the grading operations. It is possible some fill placement will extend across buildings or other improvement areas, and subdrains in such areas should be located to minimize potential interference with the planned improvements. In order to accomplish this layout, careful survey control will be required during subdrain installation.

Typically, we recommend the construction of subdrains with clean drain rock (½- or ¾-inch) wrapped in a geotextile drainage fabric (Mirafi 140N or approved equivalent). Alternatively, However, unwrapped drainage gravel meeting the specifications of Cal Trans Class 2 permeable material can be utilized as an alternative to geotextile wrapped drain gravel.

A 4-inch minimum diameter, rigid, perforated plastic pipe (SDR35, Schedule 40, or equivalent) should be provided at the base of each subdrain in order to collect and convey water toward a suitable outlet point. The pipe should drain to a low point and then be connected to a solid pipe in order to transmit any collected water to an appropriate discharge location. For discharge lines, we recommend the pipe consist of rigid, non-perforated plastic pipe (SDR35, Schedule 40, or equivalent).

The “high” end and all 90-degree bends of the subdrain pipe should be connected to a riser, which extends to the surface and acts as a cleanout. The number of cleanouts can be reduced by installing “sweep” 90-degree bends or pairs of 45-degree bends in succession, instead of using “tight” 90-degree bends. “Sweep” 90-degree bends are similar to those used in sanitary sewer pipe connections.

It will be critical that subdrain lines are not compromised by future site development and that the subdrains are maintained in the future. Therefore, it is strongly recommended that the location of all subsurface drain lines and any outflow lines be accurately documented in the field by a qualified land surveyor and that an as-built map be prepared.

7.06 Erosion Control and Maintenance

At the maximum inclinations indicated above, the cut and fill slopes will probably be subjected to some minor erosion and/or sloughing, thus requiring periodic maintenance of the slopes. We recommend any new cut or fill slopes be planted with erosion resistant vegetation and an erosion control netting system installed in accordance with applicable BMPs. A landscape architect experienced in erosion control planting should be consulted prior to selection of vegetation.

7.07 Structure Foundation

The proposed structure should be supported on a reinforced concrete mat foundation placed on 36 inches of non-expansive fill. The mat should be at least 18 inches in thickness. The mat should be designed assuming an allowable (factored) bearing capacity of 1,000 pounds per square foot (psf) for dead plus live loads (factor of safety ≈ 2). This allowable bearing pressure is a net value; therefore, the weight of the mat can be neglected for design purposes. The mat should be integrally connected to all portions of the structure so that the entire foundation system moves as a unit. The mat should be reinforced with top and bottom steel in both directions to allow the foundation to span local irregularities that may result from potential differential settlement. As a minimum, we recommend that the mat be reinforced with sufficient top and bottom steel to span as a simple beam an unsupported distance of at least 10 feet. For mat modeling purposes, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) can be assumed.

Lateral loads on the structure may be resisted by passive pressures acting against the sides of the mat. We recommend an allowable passive pressure equal to an equivalent fluid weighing 300 psf per foot of depth (factor of safety ≈ 2), starting at 6 inches below finished grade. Alternatively, an allowable friction coefficient of 0.30 (factor of safety ≈ 2) can be used between the bottom of the mat and the subgrade soils. If the perimeter of the mat is poured neat against the soils, the passive pressure and friction coefficient may be used in combination.

The building slab should be underlain by 6 inches of free draining gravel to serve as a capillary barrier between the subgrade material and the slab. A vapor retardant membrane (15 mil StegoWrap or approved equivalent) should be placed over the gravel. The membrane should be lapped and tapped in accordance with the manufacturer's recommendations. The gravel and membrane can be used in lieu of an equivalent thickness inches of non-expansive import fill recommended under slabs.

We also recommend that the specifications for slab-on-grade floors require that moisture emission tests be performed on the slab prior to the installation of the flooring. No flooring should be installed until safe moisture emission levels are recorded for the type of flooring to be used.

7.08 California Building Code Seismic Design Parameters

This Section provides seismic design parameters based on ASCE 7-16. The reported seismic design parameters as presented in Table 2, below, were developed utilizing the ASCE 7 on-line hazard report tool and are based on the site coordinates, site class (geology), and the assumed risk category of the building.

Table 2: Ground Motion Parameters Based on ASCE 7-16
(Site Coordinates: Latitude 37.843565°, Longitude -122.154266°)

Parameter	Value	ASCE 7-16 Reference
Risk Category (Assumed)	IV	Table 1.5-1
Site Class	C	Table 20.3-1
S_s	2.59g	Figure 22-1
S_1	0.99g	Figure 22-2
S_{MS}	2.69g	Equation 11.4-1
S_{M1}	1.36g	Equation 11.4-2
S_{DS}	1.79g	Equation 11.4-3
S_{D1}	0.9g	Equation 11.4-4
PGA_M	0.95g	Equation 11.8-1
T_L	8 Seconds	Figure 22-14

7.09 Exterior Slabs-on-Grade

We recommend exterior slabs-on-grade be supported on a minimum of 18 inches of imported, compacted, non-expansive fill. In order to minimize volume change of the subgrade soils below the non-expansive material, the native subgrade soil should be scarified to a depth of 6 inches, moisture conditioned to 3% to 5% above optimum water content and compacted to the requirements for structural fill. Prior to the construction of the slabs, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support.

Exterior slabs should be structurally independent from building foundations. Score cuts or construction joints should be provided at a maximum spacing of 10 feet in both directions. The slabs should be appropriately reinforced according to structural requirements; concentrated loads may require additional reinforcing. Minor movement of concrete slabs-on-grade with resulting cracking should be expected. Therefore, partition walls or doorway trim boards should not be supported directly on the concrete slab and steps to the structure from the slab area should be created with a void between the steps and the structure foundations. The recommendations presented above, if properly implemented, should help minimize the magnitude of this cracking.

7.10 Retaining Walls

Retaining walls should be designed to resist both ultimate (non-factored) lateral earth pressures and any additional lateral loads caused by surcharge loads on the adjoining ground surface. We recommend walls be designed to resist the equivalent fluid pressures indicated in the table below. The appropriate design values should be chosen based on the condition of the wall (restrained or unrestrained) and the angle of the slope behind the wall. Unrestrained wall pressures should only be considered applicable where it would be structurally and architecturally acceptable for the wall to laterally deflect 2 percent of the wall height. All building retaining walls should be designed for the restrained condition.

Table 3: Retaining Wall Lateral Pressures

Condition	Slope Inclination	
	4:1[1] or flatter	2:1
Unrestrained	50 pcf[2]	60 pcf
Restrained	65 pcf	80 pcf

[1] Inclination behind wall, horizontal to vertical.
[2] "pcf" signifies "pounds per cubic foot" equivalent fluid pressure.

- A linear interpolation should be used to determine design values for retaining walls where the slope behind the wall is between 4:1 and 2:1. Slopes steeper than 2:1 are not anticipated at the site.

- For surcharge loads, increase the ultimate (non-factored) design pressures behind the wall by an additional uniform pressure equivalent to one-half (for restrained condition) or one-third (for unrestrained condition) of the maximum anticipated surcharge load applied to the surface behind the wall. The downslope driveway wall should be designed for an appropriate vehicle surcharge load behind the wall.

For walls in excess of 6 feet in retained height, the walls should be designed for an additional seismic pressure increment modeled as a uniform lateral pressure applied over the height of the wall of 12H psf, where H is the wall height in feet.

The above pressures assume that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration. Adequate drainage may be provided by a subdrain system (see Typical Retaining Wall Subdrain Detail, Figure 12) consisting of a 4-inch, rigid, perforated pipe, bedded in ¾-inch, clean, open-graded rock. As shown on Figure 12, the recommended location of the subdrain pipe is behind the heel of the footing. Although we have observed the subdrain pipe is often placed on top of the heel of the footing, it has been our experience that this may lead to moisture seeping through the wall resulting in dampness and staining on the opposite wall face despite the application of waterproofing. However, if such seepage or dampness is acceptable (in front of landscape walls, for example), then the subdrain pipe may be placed on top of the heel of the footing. To prevent ponding of water on top of the heel of the footing, we recommend that the top of the heel be sloped to drain away from the wall with a minimum positive gradient of 5 percent. The perforated drainpipe should be sloped to drain with a minimum positive gradient of 2 percent. The entire rock/pipe unit should be wrapped in an approved, non-woven, polyester geotextile such as Mirafi 140N or 140NL, or a 4-ounce equivalent. The rock and fabric placed behind the wall should be at least one foot in width and should extend to within one foot of finished grade. The upper one foot of backfill (6 inches for walls less than 5 feet in height) should consist of on-site, compacted, relatively impervious soils (an impermeable plug). We should note flexible, perforated pipe (flexline), 2000-Pound Crush, Leachfield, and ASTM F810 pipe are not acceptable for use in the subdrain because of the likelihood of damage to the pipe during installation and the difficulty of future cleaning with mechanical equipment without damaging the pipe. We recommend the use of Schedule 40 PVC, SDR 35 PVC or ABS drainpipe, or equivalent for the drain system. The subdrain pipe should be connected to a system of closed pipes (non-perforated) that lead to suitable discharge facilities. At the location where the perforated subdrain pipe connects with the solid discharge drainpipe, drainrock backfill should be discontinued. A "clay plug" should be constructed out of relatively impervious soils to direct collected water into the perforated pipe and minimize the potential for water collecting around the solid drainpipe and saturating the adjacent soils. We recommend waterproofing be applied to any proposed retaining walls where applicable. The specification of the type of waterproofing and the observation of its installation should be performed by the architect and/or structural engineer.

In addition to the drainage details noted above, the "high" end and all 90-degree bends of the subdrain pipe should be connected to a riser which extends to the surface and acts as a cleanout. The number of cleanouts can be reduced by installing "sweep" 90-degree bends or pairs of 45-degree bends in succession instead of using "tight" 90-degree bends. "Sweep" 90-degree bends are similar to those used in sanitary sewer pipe connections.

Lined surface ditches with a minimum width of 12 inches should be provided behind any walls that will have an exposed sloping surface steeper than 4:1 behind them. These ditches, which will collect runoff water from the slopes, should be sloped to drain (minimum 2 percent positive gradient) to suitable discharge facilities. If the lined surface ditches consist of reinforced concrete, expansion joints should be provided every 10 feet. The top of the walls should extend at least one foot above the ditch (6 inches for walls less than 5 feet in height). All structural backfill placed behind retaining walls should be compacted in accordance with the requirements provided in Section 7.04, “Fill Compaction.” Special care (such as the use of lightweight equipment) should be taken during wall backfill compaction operations to minimize overstressing of the wall.

Retaining walls that are within 10 feet from the crest of down sloping areas should be supported on pier foundations. Friction piers should have a minimum diameter of 16 inches, and there should be a minimum center-to-center spacing of at least three pier diameters between adjacent piers. The piers should generally extend to a depth adequate to provide at least 8 feet of embedment into bedrock. Since bedrock was encountered at a depth of approximately 2 to 5 feet in the borings, the piers should generally extend to a minimum depth of about 10 to 13 feet below the existing ground surface. An allowable (factored) skin friction values of 650 psf can be used in the bedrock. To minimize damage resulting from the potential surficial soil creep movements, we recommend the piers be designed to resist an ultimate (non-factored), uniform lateral pressure of 300 psf acting against the projected diameter of the pier to a depth of 3 feet below the ground surface. For piers located in areas where there is a level surface for a horizontal distance of at least 10 feet around the proposed pier location and to the nearest adjacent face of the slope, creep loads can be omitted.

Lateral loads on the piers may be resisted by passive pressures acting against the sides of the piers. We recommend an allowable passive pressure equal to an equivalent fluid weighing 400 psf per foot of depth to a maximum value of 4,000 psf (factor of safety ≈ 2). This value can be assumed to be acting against two times the diameter of the individual pier shafts starting at a depth of 3 feet.

The bottom of pier excavations should be reasonably free of loose cuttings and soil fall in prior to installing reinforcing steel and placing concrete. It is our recommendation that the contractor be made aware of the subsurface conditions outlined in this report and he obtain construction equipment appropriately sized to perform the recommended work. In particular, the piers must extend a minimum of 10 to 13 feet below the ground surface, which is likely about 8 feet into bedrock. Equipment capable of performing this recommendation should be employed. Any accumulated water in pier excavations should be removed prior to placing reinforcing steel and concrete, or the concrete should be tremied from the bottom of the hole.

The proper handling of spoils excavated during the pier drilling is very important. If these materials are left in a loose condition on a slope, they will have a tendency to creep down hill and/or erode during periods of heavy rainfall. Therefore, we recommend these materials be removed from the site, placed and compacted as engineered fill, or placed as wall backfill where settlement would not cause a problem.

Observations during pier drilling operations should be performed by a representative of our firm to confirm that anticipated conditions are being encountered. If drilling refusal is encountered, we can coordinate a review of the conditions and drilling equipment adequacy, as well as conduct discussions with the project structural engineer.

8.00 FUTURE GEOTECHNICAL SERVICES

As no specific development plans have been provided so far, there may be the need for additional geotechnical studies after the plans are developed to provide recommendations for specific items not covered by this report. This will be particularly important when the site grading plan has been established as it is likely there will need to be major grading operations performed at the site, along with numerous retaining walls, to provide a level building pad and parking areas. We can provide the scope and cost of such future work after reviewing the plans.

9.00 REPORT LIMITATIONS AND CLOSURE

This report has been prepared for your exclusive use and that of your consultants for specific application to the proposed project, in accordance with generally accepted geological and geotechnical engineering practices. No other warranty, either expressed or implied, is made. In the event the nature, design, or location of the proposed facility differs significantly from what has been noted above, or if any future additions are proposed, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property, due to natural processes or the works of people. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after three years without being reviewed by this office.

We are pleased to have been of service to you on this project and look forward to working with you during the plan review and construction phases of the work.

If you have any questions concerning this report, please call us.

Very truly yours,

Alan Kropp, G.E.
Principal Engineer
Alan Kropp & Associates

Patrick Drumm, P.G., C.E.G., C.H.G.
Certified Engineering Geologist
Earth Focus Geological Services, Inc.

AK/PD/jc

Copies: Addressee (PDF) - DAranda@kppcsd.org

Attachments: Figure 1, Vicinity Map
Figure 2, Site Plan
Figure 3, Alquist-Priolo Earthquake Fault Zone Map
Figure 4, CGS Zones of Required Investigation
Figure 5, Regional Geologic Map
Figure 6, Regional Active Fault Map
Figure 7, Recently Active Traces of the Hayward Fault
Figure 8, California Geological Survey Landslide Map
Figure 9, U.S. Geological Survey Landslide Map
Figure 10, Geologic Cross-Section
Figure 11, Fill Placement on Slopes
Figure 12, Typical Retaining Wall Subdrain Detail

Appendix A, Boring and Test Trench Logs
Appendix B, CGS Note 14
Appendix C, Homeowner's Guide to Maintenance

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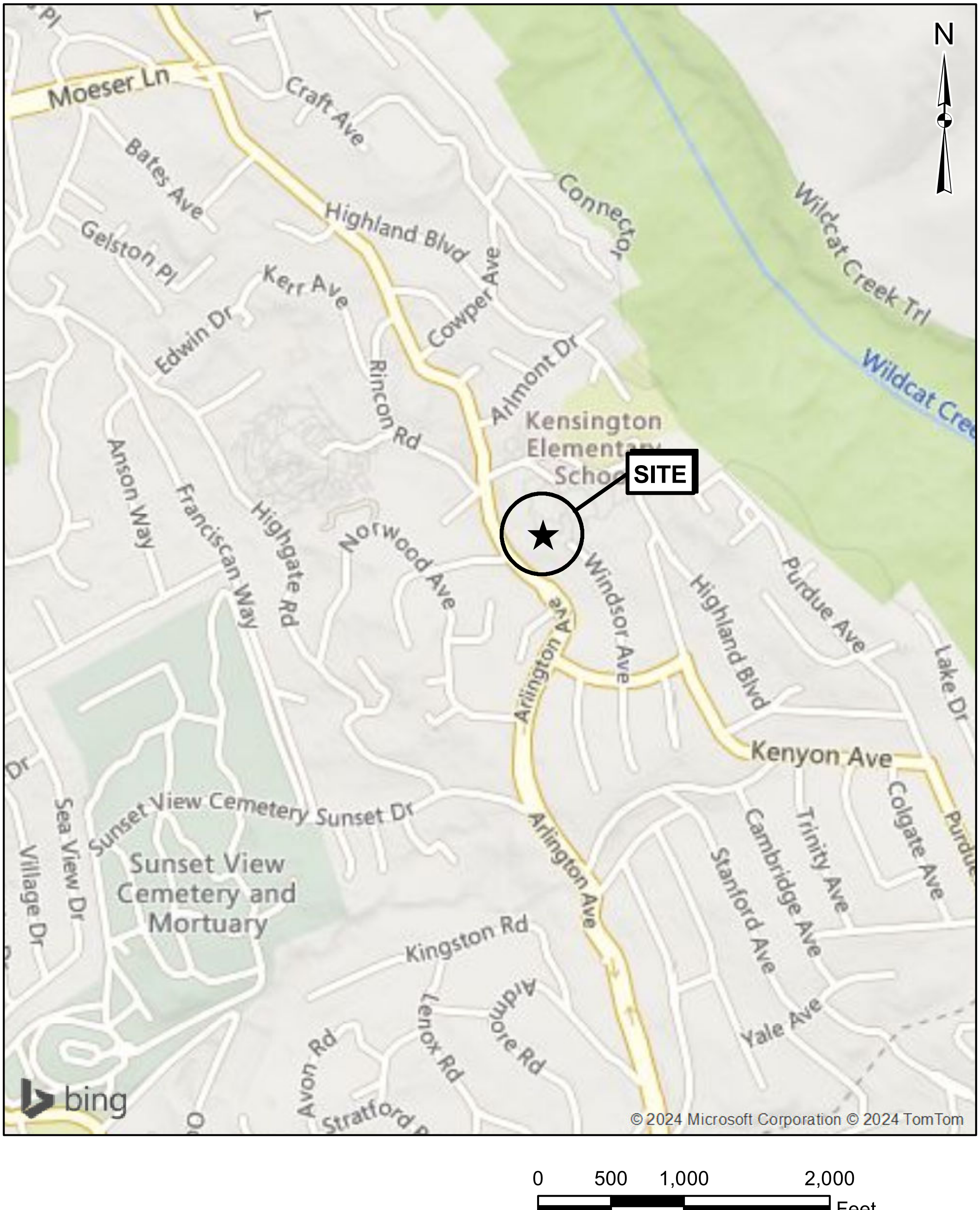

Original figure produced in color.

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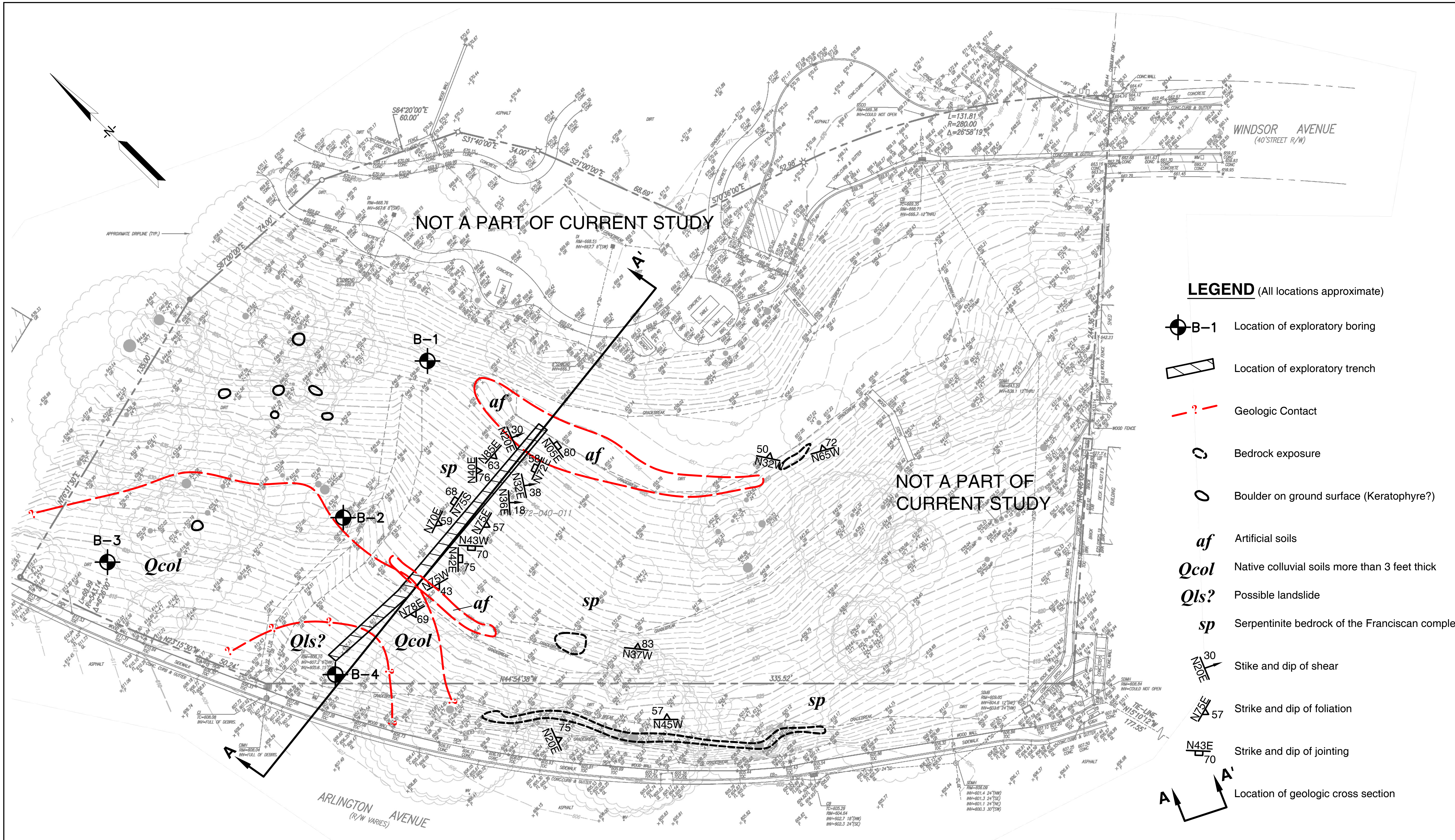
VICINITY MAP		
POLICE DEPARTMENT BUILDING SITE Kensington, California		
PROJECT NO.	DATE	FIGURE 1
3194-1A	**September 2024**	



Original figure produced in color.

Source: "Boundary & Topographic Survey, APN: 572-040-011" drawn by Kister, Savio & Rei, Inc., dated 4/9/2024.

20 0 40 80
APPROXIMATE SCALE FEET

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SITE PLAN		
POLICE DEPARTMENT BUILDING SITE Kensington, California		
PROJECT NO. **3194-1A**	DATE **September 2024**	FIGURE **2**

Base: *Alquist Priolo Earthquake Fault Zone Map, Richmond Quadrangle, California, 1982.*

N

LEGEND

MAP EXPLANATION

Potentially Active Faults

1906 C

Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; 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.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

Original figure produced in color.

0 750 1,500 3,000 Feet

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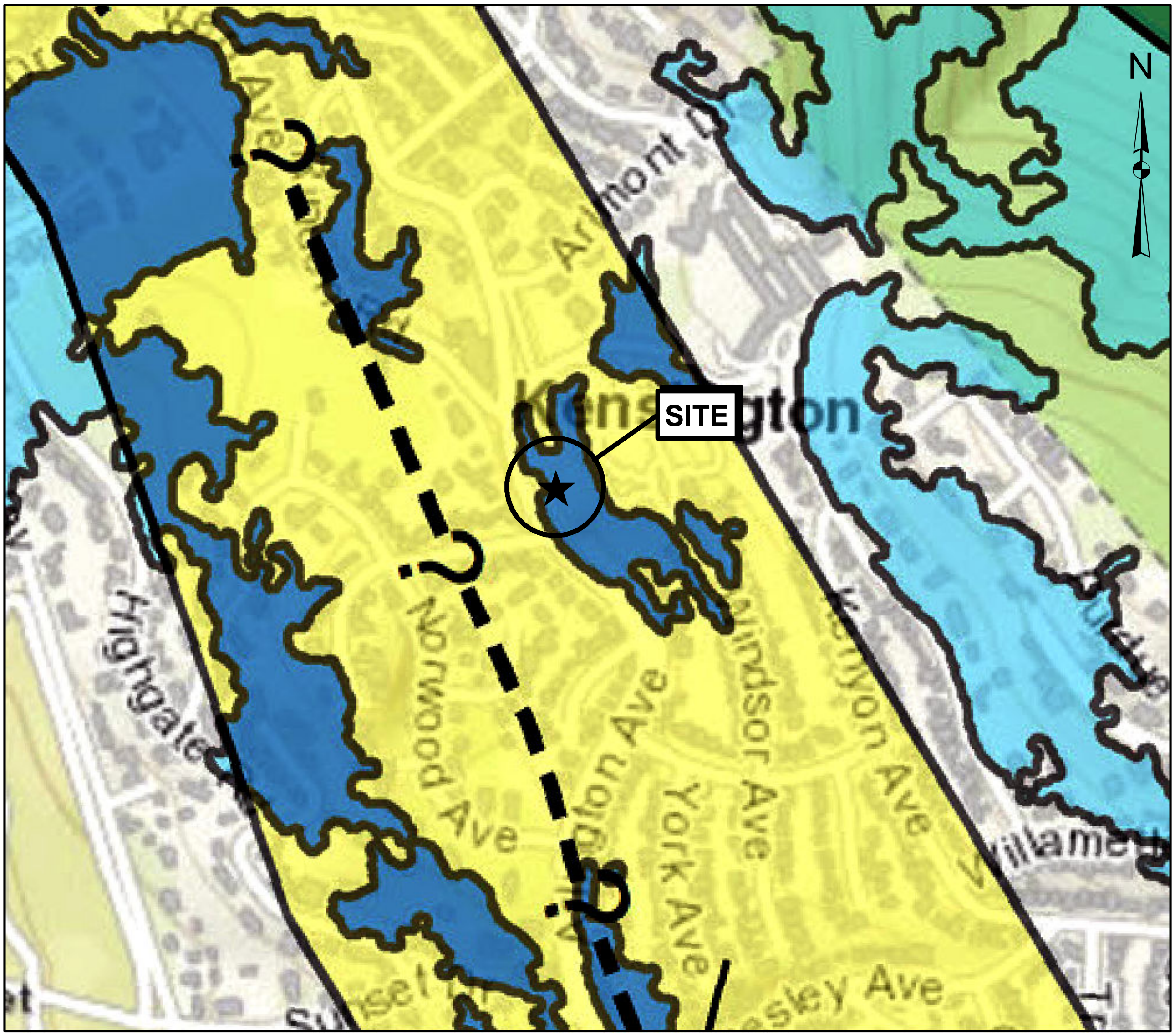
ALQUIST-PRIOLO EARTHQUAKE FAULT ZONE MAP		
POLICE DEPARTMENT BUILDING SITE Kensington, California		
PROJECT NO.	DATE	FIGURE 3
3194-1A	September 2024	



LEGEND

Source: California Geological Survey, https://maps.conservation.ca.gov/cgs/EQZApp/app/

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
1906 C ?
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black where Accurately Located; Long Dash in Black where Approximately Located; Short Dash in Black where Inferred; Dotted Line in Black where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

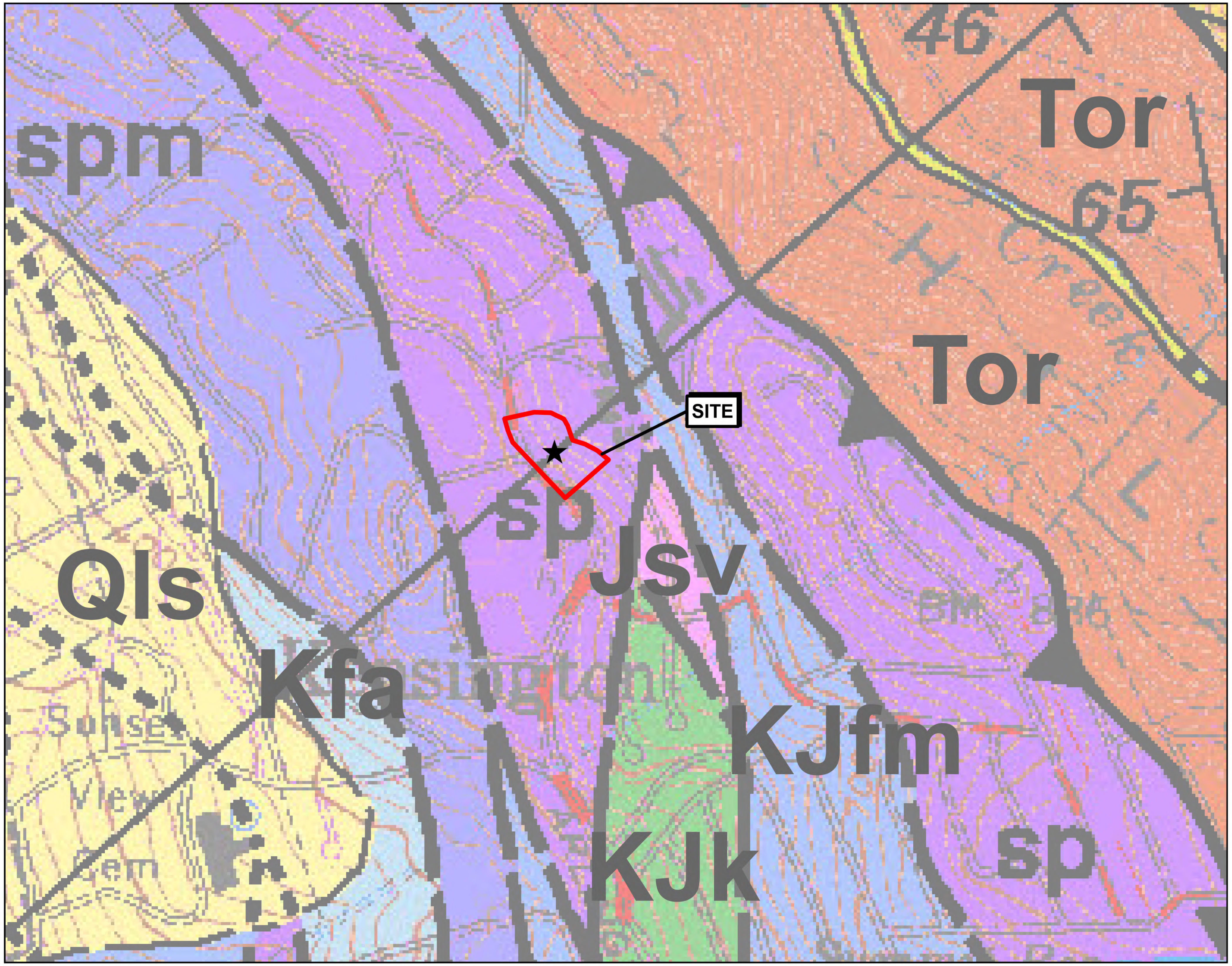
Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

Note. Mitigation methods differ for each zone --
AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

0 500 1,000
Feet

Original figure produced in color.

ALAN KROPP & ASSOCIATES *Geotechnical Consultants*	**CGS ZONES OF REQUIRED INVESTIGATION**		
	POLICE DEPARTMENT BUILDING SITE Kensington, California		
	PROJECT NO.	DATE	FIGURE **4**
	3194-1A	**September 2024**	



Base: Graymer, 2000.

N

LEGEND

Qls	Landslide deposits (Holocene and/or Pleistocene)
Tor	Orinda Formation (late Miocene)
KJk	Knoxville Formation (Early Cretaceous and Late Jurassic)
sp	Serpentinite
Jsv	Keratophyre and quartz keratophyre (Late Jurassic) (Late Cretaceous to Late Jurassic)
KJfm	Franciscan complex, mélange (Cretaceous Late Jurassic), includes mapped locally: Graywacke and meta-graywacke blocks
Kfa	Sandstone of the Alcatraz terrane of Blake and others (1984)(Cretaceous)
spm	Serpentinite matrix mélange

0 250 500 1,000 Feet

Original figure produced in color.

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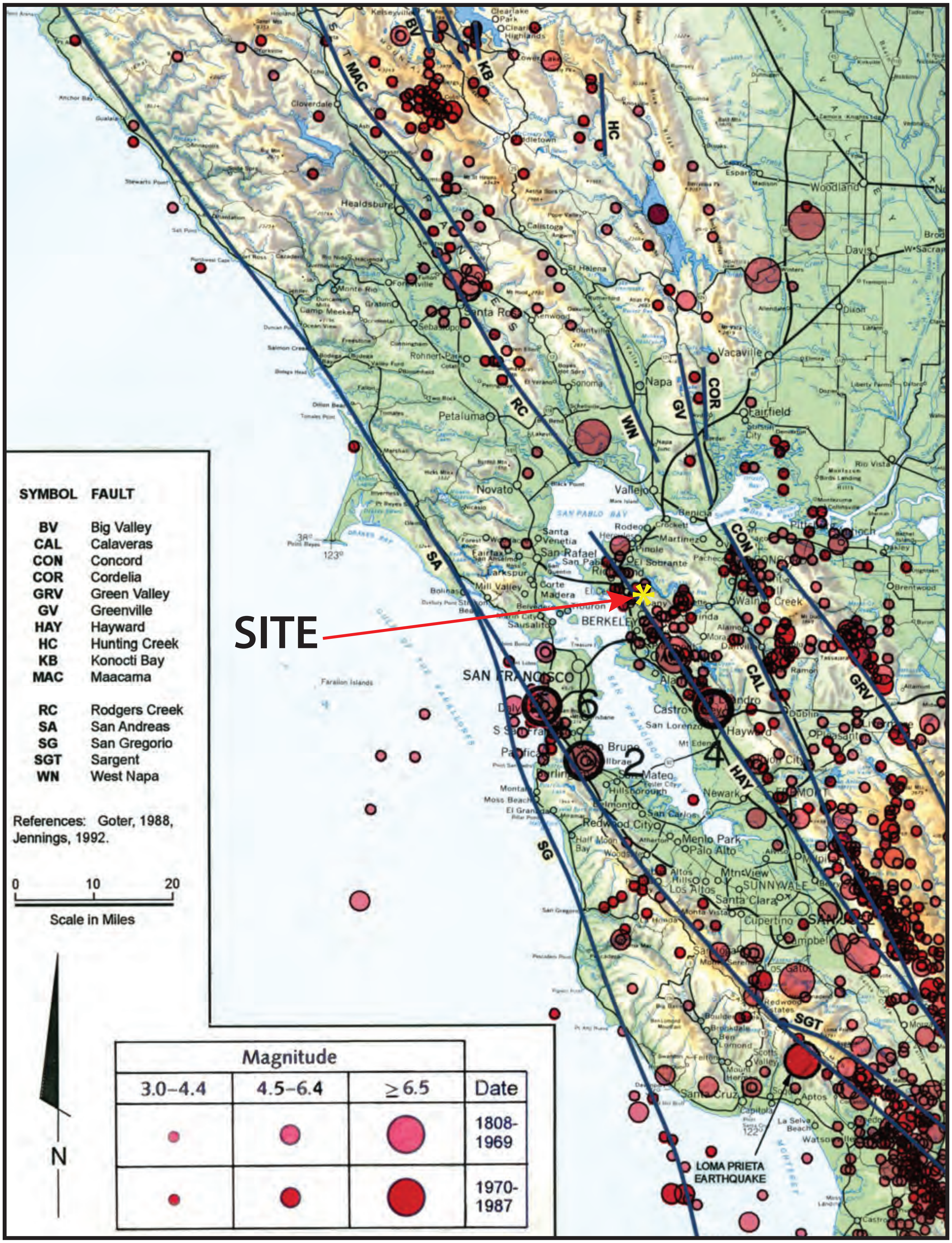
REGIONAL GEOLOGIC MAP

POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	FIGURE 5
3194-1A	**September 2024**	



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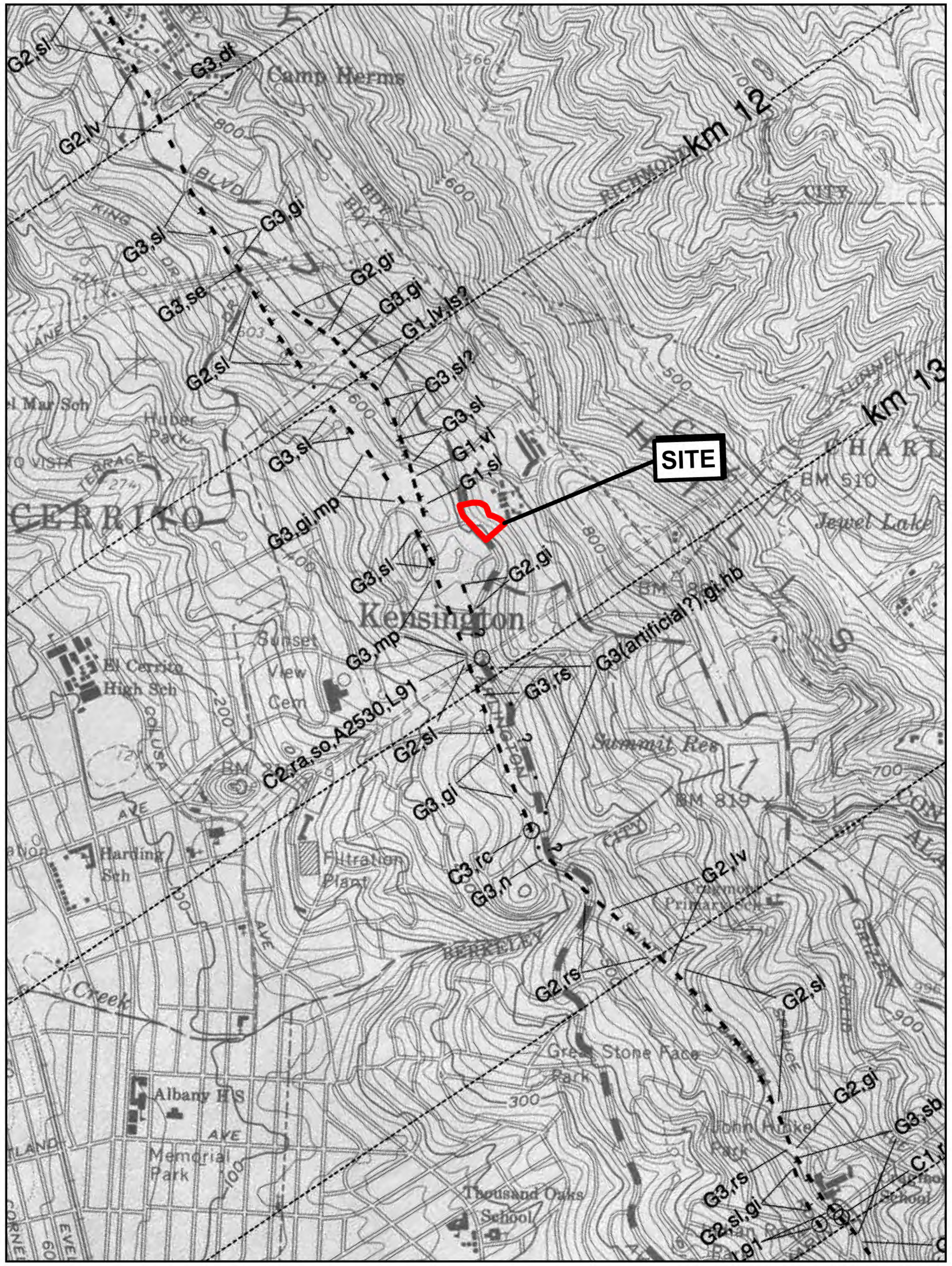
REGIONAL ACTIVE FAULT MAP

POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	FIGURE 6
3194-1A	**September 2024**	



Base: Lienkaemper, 1991, Map of Recently Active Traces of the Hayward Fault, Alameda, and Contra Costa Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-2196.

Original figure produced in color.

N

LEGEND

EXPLANATION

Fault – Dashed to show approximate maximum uncertainty in lateral position of fault traces. Each dash represents 30 m on map. Traces mainly derive from geomorphic data, but creep and trench data used where available to constrain the position of most intense deformation more precisely. Particular observations used to locate faults are annotated on the map in abbreviated form (see Abbreviations). Hachures indicate the lower side in apparent vertical separations of the ground surface from geomorphic evidence. Dotted traces interpolate fault locations where evidence is concealed or destroyed; queries indicate speculative connections where data is absent or suspect. Sawteeth on upper plate of fault that has significant thrust or reverse component

- ≤± 20 m uncertainty (20-m space on map between 30-m dashes)
- ≤± 40 m uncertainty (40-m space on map between 30-m dashes)
- ≤± 60 m uncertainty (60-m space on map between 30-m dashes)

- **Creep locality**
- **Trench location** – Actual length
- **Short trench** – Not to scale
- **Fault location in trench** – No geomorphic trace recognized
- **Small tectonic depression** (df or dr) – Not to scale
- **Tectonic depression** (df or dr) **or pressure ridge** (pr) – Actual size, hand drafted
- **Spring or seep** (sp) – Located on active fault trace
- **Monoclinal flexure**
- A **Trilateration monument** – See Prescott and Lisowski (1983)

ABBREVIATIONS

C - CREEP EVIDENCE

1 - strongly pronounced fault creep
2 - distinct and certain creep evidence
3 - inconclusive evidence for creep
? - additional uncertainty in tectonic origin
aa - alinement array
cb - concentration of cracks in above grade structure
cc - concentration of cracks in concrete slab
cp - concentration of pavement cracks
cs - curb separating from sidewalk or pavement
ec - en echelon left-stepping cracks in pavement
jo - opening of joints or cracks in concrete
pp - multiple patches in pavement
pu - compressional pop-up or buckle in concrete
ra - right-laterally offset aqueduct, water pipe, or tunnel
rb - distortion or racking of above-grade structure (including separating additions and stairways)
rc - right-laterally offset curb or form line
rf - right-laterally offset fence line
rp - right-laterally offset painted line
rr - right-laterally offset railroad tracks or guardrail
rs - right-laterally offset sidewalk
rt - right-laterally offset line of trees
rw - right-laterally offset wall
so - surveyed offset feature

G - GEOMORPHIC FEATURES

1 - strongly pronounced feature
2 - distinct feature
3 - weakly pronounced feature
? - additional uncertainty in tectonic origin
af - alignment of multiple features as listed
as - arcuate scarp
bt - downthrown surface tilts back toward fault
df - depression form by some aspect of fault deformation, undifferentiated
dr - sag, depression formed in right stepover of fault trace
gi - linear break (or gradual inflection) in slope
hb - linear hillside bench
hv - linear hillside valley
ls - fault scarp height enlarged by landsliding
lv - linear valley or trough
mp - Youngest traces disturbed by human activities. Mapped trace bisects disturbed zone. Dash gap equals half width of disturbed zone.
n - notch
pr - pressure ridge in left stepover
rr - right-laterally offset ridge line
rs - right-laterally offset stream or gully
sb - broad linear scarp (implies multiple traces)
sc - scissor point, sense of vertical separation reverses
se - subsoil exposed
sl - linear scarp, undifferentiated
sn - narrow linear scarp (implies dominant trace)
sp - spring
ss - swale in saddle
vl - line of vegetation

T - TRENCH EXPOSURES
(and other geologic evidence)

H1 - Holocene age of offset determined by radiocarbon (^{14}C) dating
H2 - Modern soil or alluvial unit distinctly offset, or contains features conclusive of shearing, such as gouge, rotated pebbles, transported materials in shear zone, and filled fissures over distinct Pleistocene faults
H? - Inconclusive signs of Holocene offset, such as steps in base of soil or apparent shears in clay-rich materials. Without corroboration such evidence neither proves nor disproves either existence or age of faulting
H - Active trace reported in trench, trench logs not in public file
HP - Distinct faulting in unconsolidated alluvium of possibly Holocene or more likely latest Pleistocene age
F? - Feature shown as fault in log resembles nontectonic feature such as bedrock-alluvial contact, buried terrace riser, or landslide plane
NF - No fault observed
P - Distinct evidence of significant faulting in Pliocene or Pleistocene sediments
RC - Roadcut log
WB - Ground water barrier
U - Age of faulting unobtainable because surficial deposits removed

REFERENCE CODES (see also Abbreviated Map References and text for full references.)

A2456 - Trench log or creep evidence in Alquist-Priolo report AP-2456, filed at California Division of Mines and Geology (CDMG), San Francisco.
C200 - Trench log or creep evidence in non-Alquist-Priolo consultant's report filed at CDMG.
G70 - Non-Alquist-Priolo unpublished report referenced in abbreviated references as G70.

0 750 1,500 3,000 Feet

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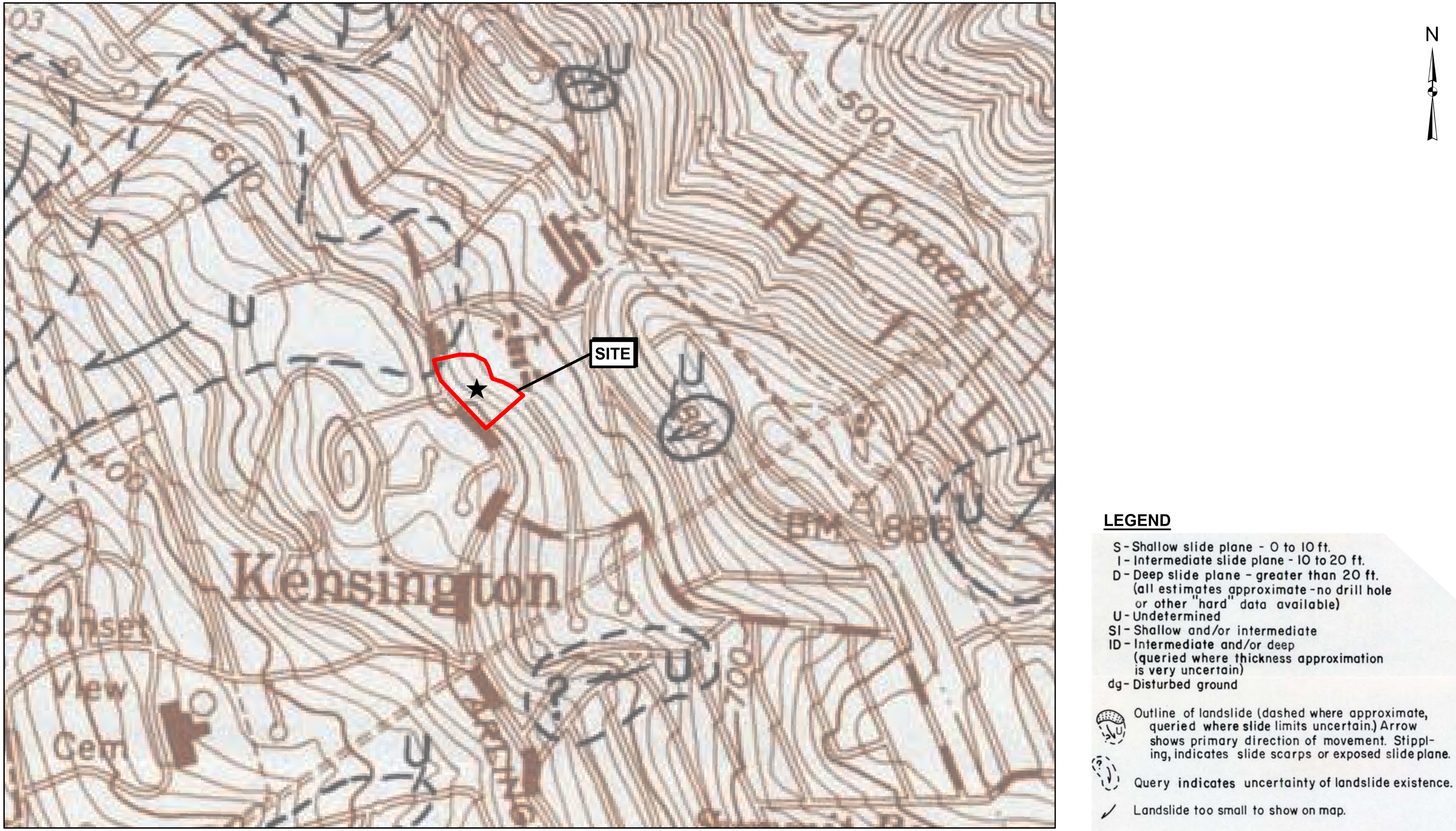
RECENTLY ACTIVE TRACES OF THE HAYWARD FAULT		
POLICE DEPARTMENT BUILDING SITE Kensington, California		
PROJECT NO.	DATE	FIGURE 7
3194-1A	September 2024	



Base: CDMG Tri Cities Preliminary Report 19, Plate 6, 1973.

Original figure produced in color.

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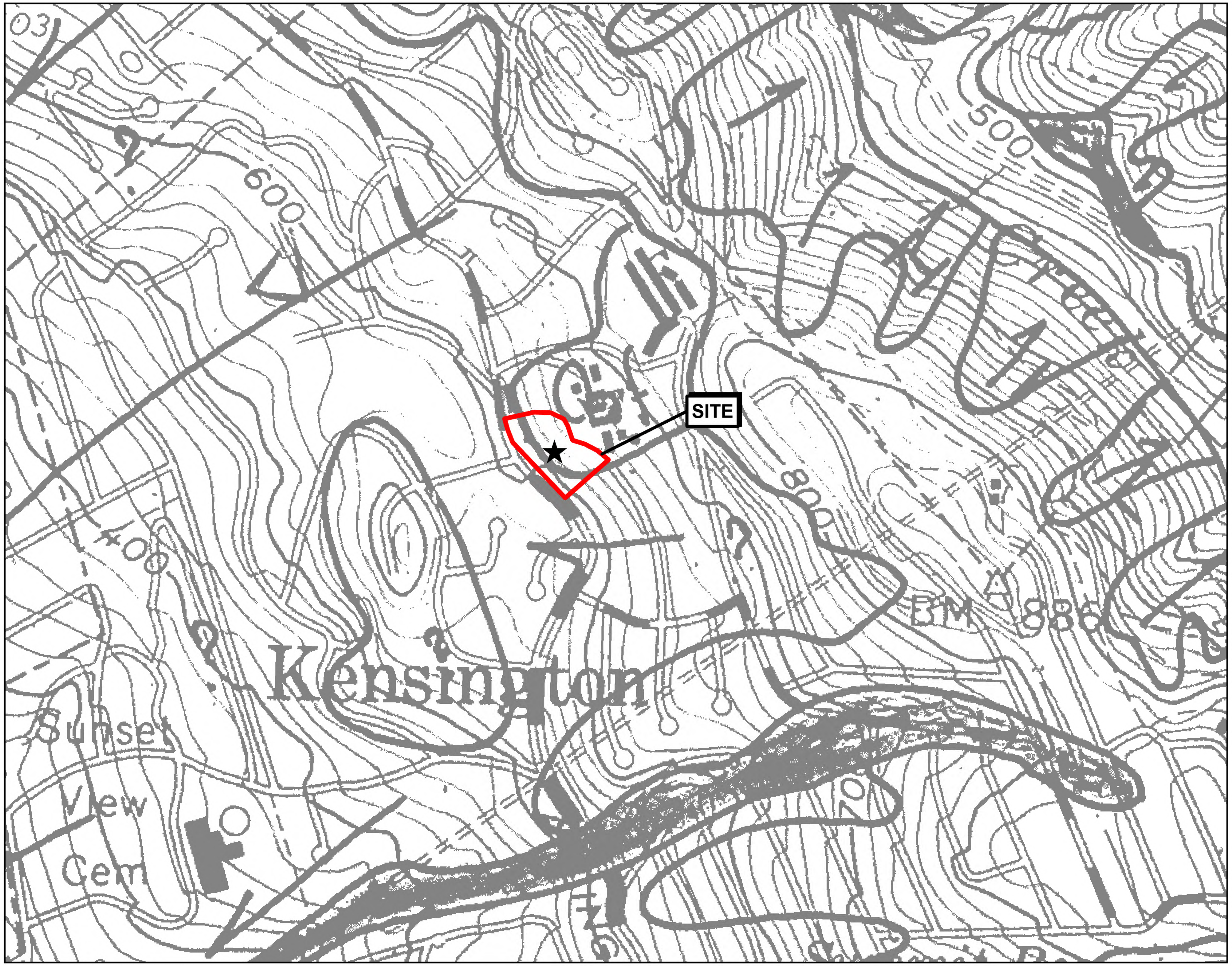
CALIFORNIA GEOLOGICAL SURVEY LANDSLIDE MAP		
POLICE DEPARTMENT BUILDING SITE Kensington, California		
PROJECT NO.	DATE	FIGURE 8
3194-1A	September 2024	



Base: Nilsen, USGS OFR 75-277-415

Original figure produced in color.

0 250 500 1,000 Feet

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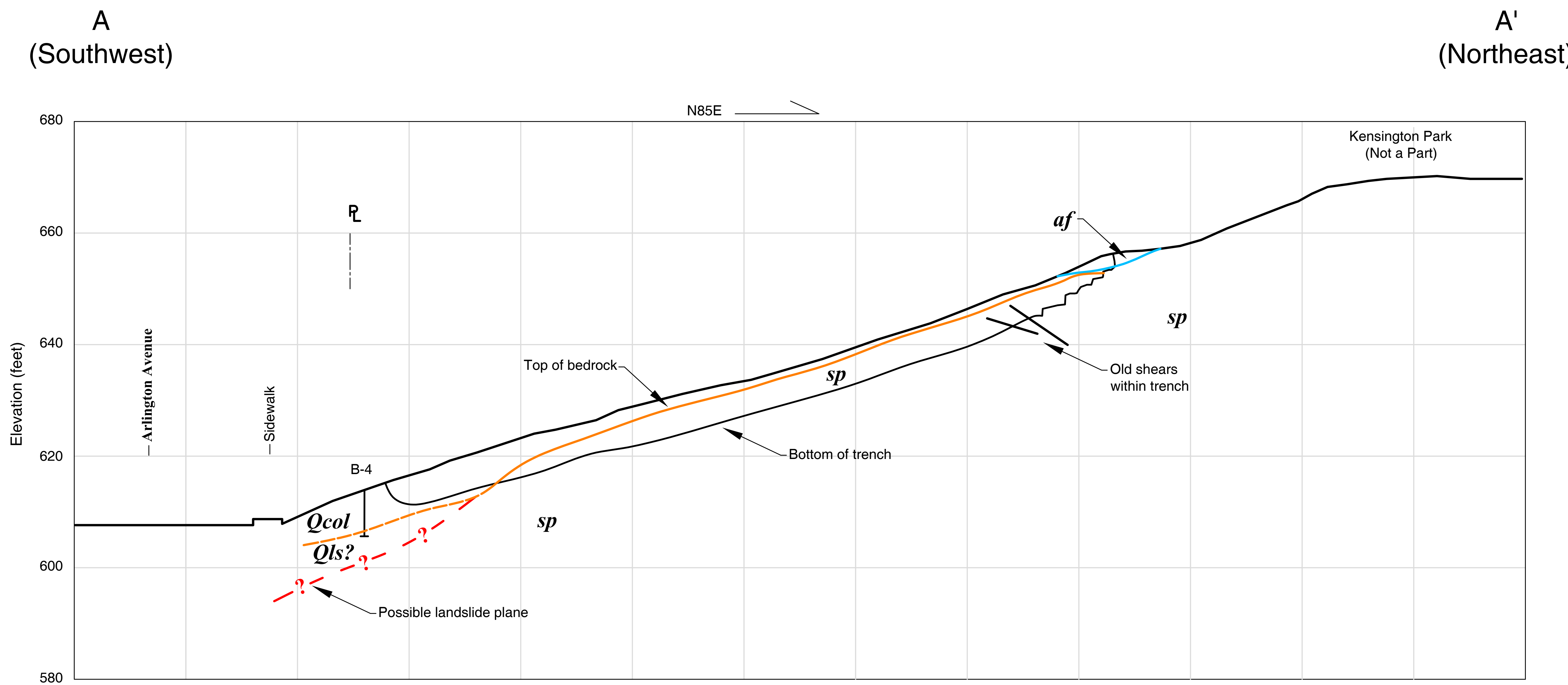
U.S. GEOLOGICAL SURVEY LANDSLIDE MAP		
POLICE DEPARTMENT BUILDING SITE Kensington, California		
PROJECT NO.	DATE	FIGURE 9
3194-1A	September 2024	



LEGEND

af	Artificial soils
Qcol	Native colluvial soils
Qls?	Possible landslide
sp	Serpentinite bedrock of the Franciscan complex

NOTES:
1. See Figure 2 for location of cross section.
2. Cross section represents idealized conditions based on limited subsurface data. Significant variations may be encountered during construction.
3. Original figure produced in color.

10 0 20 40
APPROXIMATE SCALE FEET

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& ASSOCIATES
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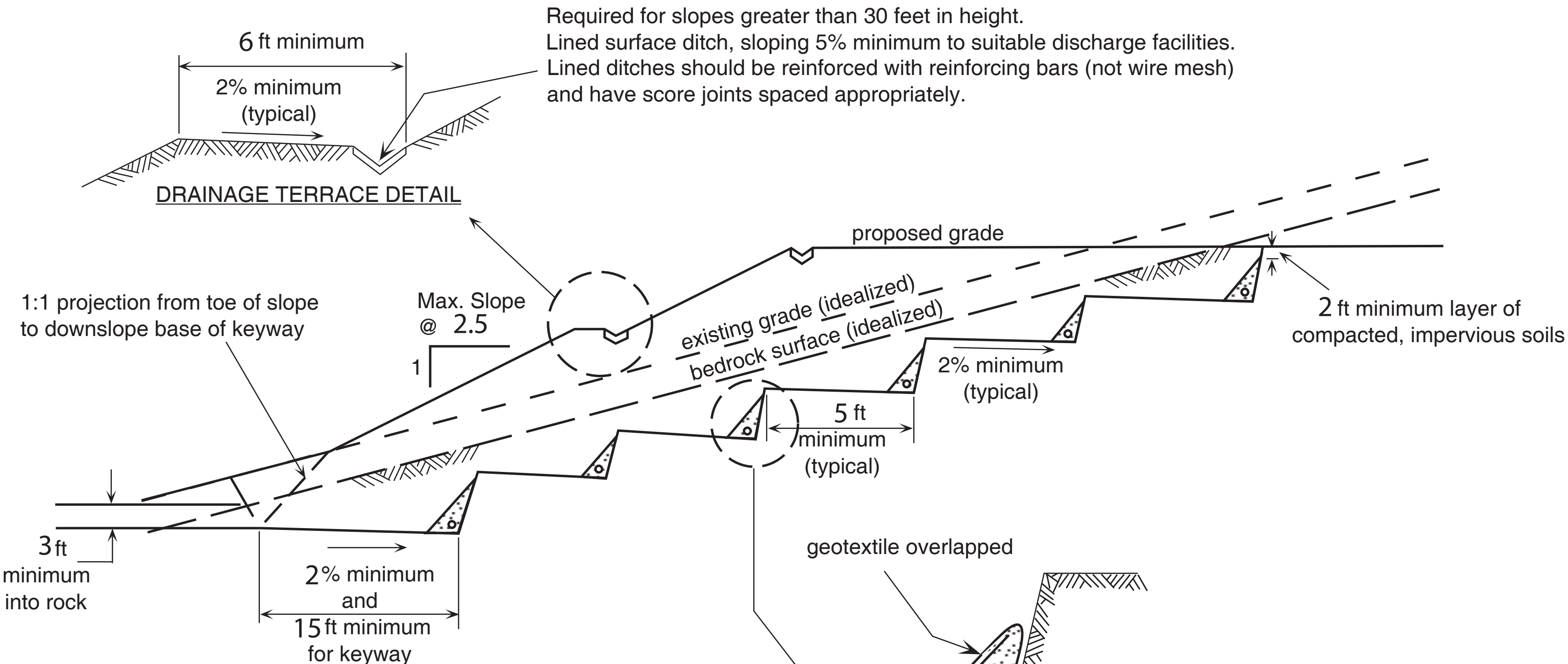
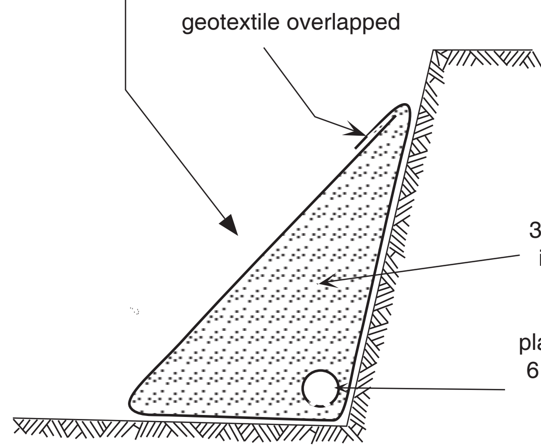
Consultants

GEOLOGIC CROSS-SECTION A-A'
POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	FIGURE
3194-1A	**September 2024**	**10**





SUBDRAIN DETAIL (TYPICAL)

NOTES:

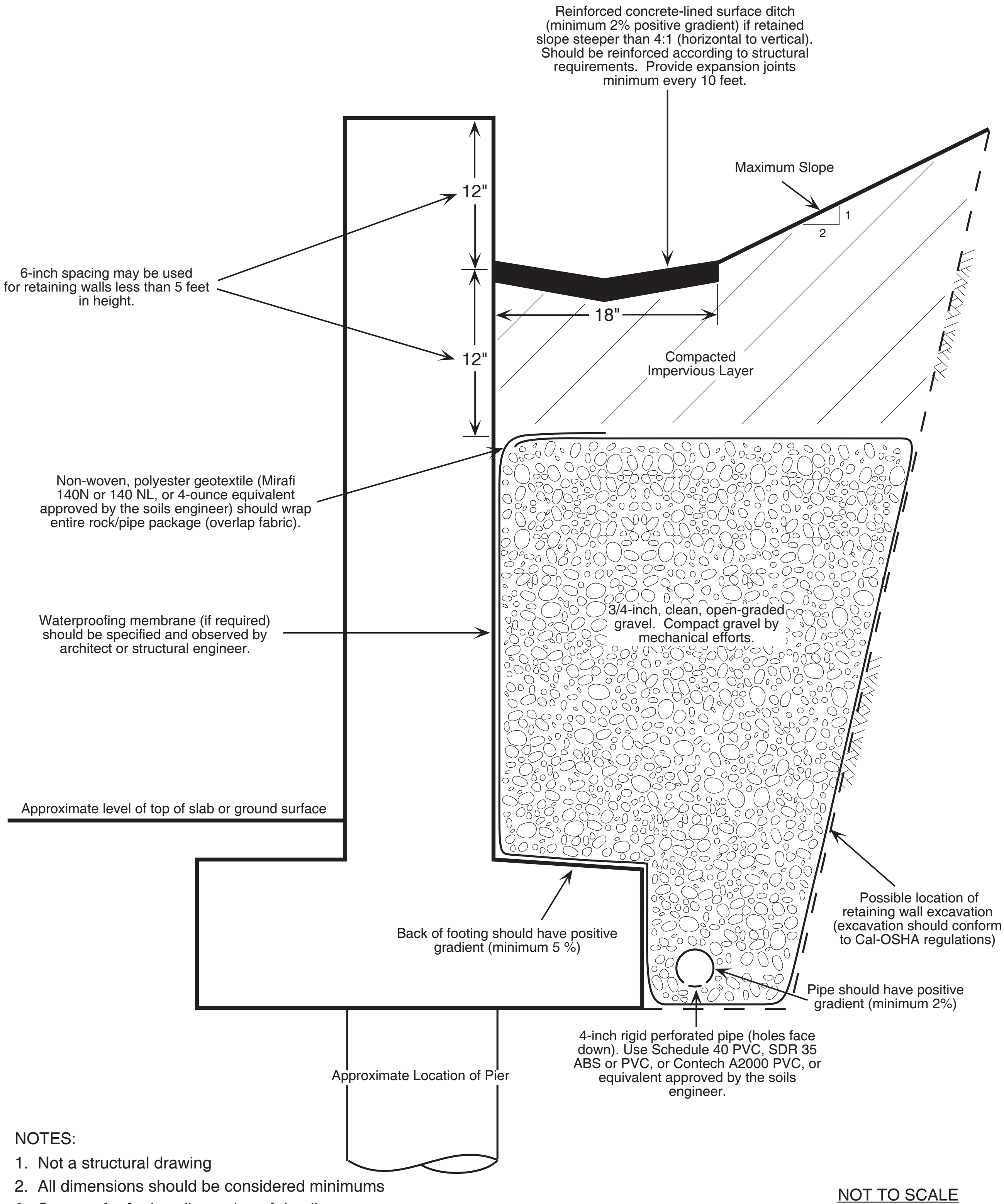
1. This is an idealized cross-section through a typical fill slope, representing idealized subsurface conditions. It is not to scale and has not been developed from site specific information. Significant variations may be encountered during construction.

2. Dimensions of keyways and benches are subject to approval in the field by the geotechnical engineer's representative.

3. Subdrains may not be needed on each bench; number and location of subdrain (s) should be determined in the field by the geotechnical engineer's representative at the time of construction.

NOT TO SCALE

ALAN KROPP & ASSOCIATES Geotechnical Consultants	FILL PLACEMENT ON SLOPES		
	POLICE DEPARTMENT BUILDING SITE Kensington, California		
	PROJECT NO.	DATE	FIGURE 11
	3194-1A	September 2024	



NOTES:

1. Not a structural drawing
2. All dimensions should be considered minimums
3. See text for further discussion of detail

NOT TO SCALE

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TYPICAL RETAINING WALL SUBDRAIN DETAIL

POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	FIGURE **12**
3194-1A	**September 2024**	

APPENDIX A

BORING AND TEST TRENCH LOGS

SOIL CLASSIFICATION CHART

PRIMARY DIVISIONS			SECONDARY DIVISIONS: CRITERIA *	GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	Cu ≥ 4 AND 1 ≤ Cc ≤ 3 *A*	GW	Well-graded gravel
			Cu < 4 AND/OR 1 > Cc > 3	GP	Poorly-graded gravel
		GRAVELS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	GM	Silty gravel
			FINES CLASSIFY AS CL OR CH	GC	Clayey gravel
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	Cu ≥ 6 AND 1 ≤ Cc ≤ 3	SW	Well-graded sand
			Cu < 6 AND/OR 1 > Cc > 3	SP	Poorly-graded sand
		SANDS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	SM	Silty sand
			FINES CLASSIFY AS CL OR CH	SC	Clayey sand
FINE-GRAINED SOILS 50% OR MORE PASSES THE NO.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%	INORGANIC	PI > 7 AND PLOTS ON OR ABOVE "A" LINE	CL	Lean clay
			PI < 4 OR PLOTS BELOW "A" LINE	ML	Silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OL	Organic Clay & Organic Silt
	SILTS AND CLAYS LIQUID LIMIT 50% OR MORE	INORGANIC	PI PLOTS ON OR ABOVE "A" LINE	CH	Fat clay
			PI PLOTS BELOW "A" LINE	MH	Elastic silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OH	Organic Clay & Organic Silt
	HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR	PT	Peat

REFERENCE: Unified Soil Classification System (ASTM D2487-17ε1)

* Criteria may be done on visual basis, not necessarily based on lab testing

A – $C_u = D_{60}/D_{100}$ & $C_c = (D_{30})^2 / (D_{10} \times D_{60})$

GRAIN SIZES

U. S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

ABBREVIATIONS

INDEX TESTS

LL - Liquid Limit (%) (ASTM D4318-17ε1)

PI - Plasticity Index (%) (ASTM D4318-17ε1)

-200 - Passing No. 200 Sieve (%) (ASTM D1140-17)

STRENGTH TESTS

PP - Field Pocket Penetrometer test of unconfined compressive strength (tsf)

TV - Field Torvane test of shear strength (psf)

UC - Laboratory unconfined compressive strength (psf) (ASTM D2166/2166M-16)

TXUU - Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D2850-23)

MISCELLANEOUS

ATOD - At time of drilling

psf/tsf - pounds per square foot / tons per square foot

psi - pounds per square inch (indicates relative force required to advance Shelby tube sampler)

SYMBOLS

- Standard Penetration Test Split Spoon (2-inch O.D.)
- Modified California Sampler (3-inch O.D.)
- Thin-walled Sampler Tube (either Pitcher or Shelby) (3-inch O.D.)
- Rock Core
- Bag Sample
- Groundwater Level during drilling
- Groundwater Level after drilling

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KEY TO EXPLORATORY BORING LOGS

POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	FIGURE A-1
3194-1A	**September 2024**	

CONSOLIDATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

BEDDING OF SEDIMENTARY ROCK

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 feet	Very thick-bedded
Blocky	2.0 to 4.0 feet	Thick-bedded
Slabby	0.2 to 2.0 feet	Thin-bedded
Flaggy	0.05 to 0.2 feet	Very thin-bedded
Shaly or platy	0.01 to 0.05 feet	Laminated
Papery	Less than 0.01 feet	Thinly laminated

FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0 feet
Occasionally fractured	1.0 to 4.0 feet
Moderately fractured	0.5 to 1.0 feet
Closely fractured	0.1 to 0.5 feet
Intensely fractured	0.05 to 0.1 feet
Crushed	Less than 0.05 feet

HARDNESS

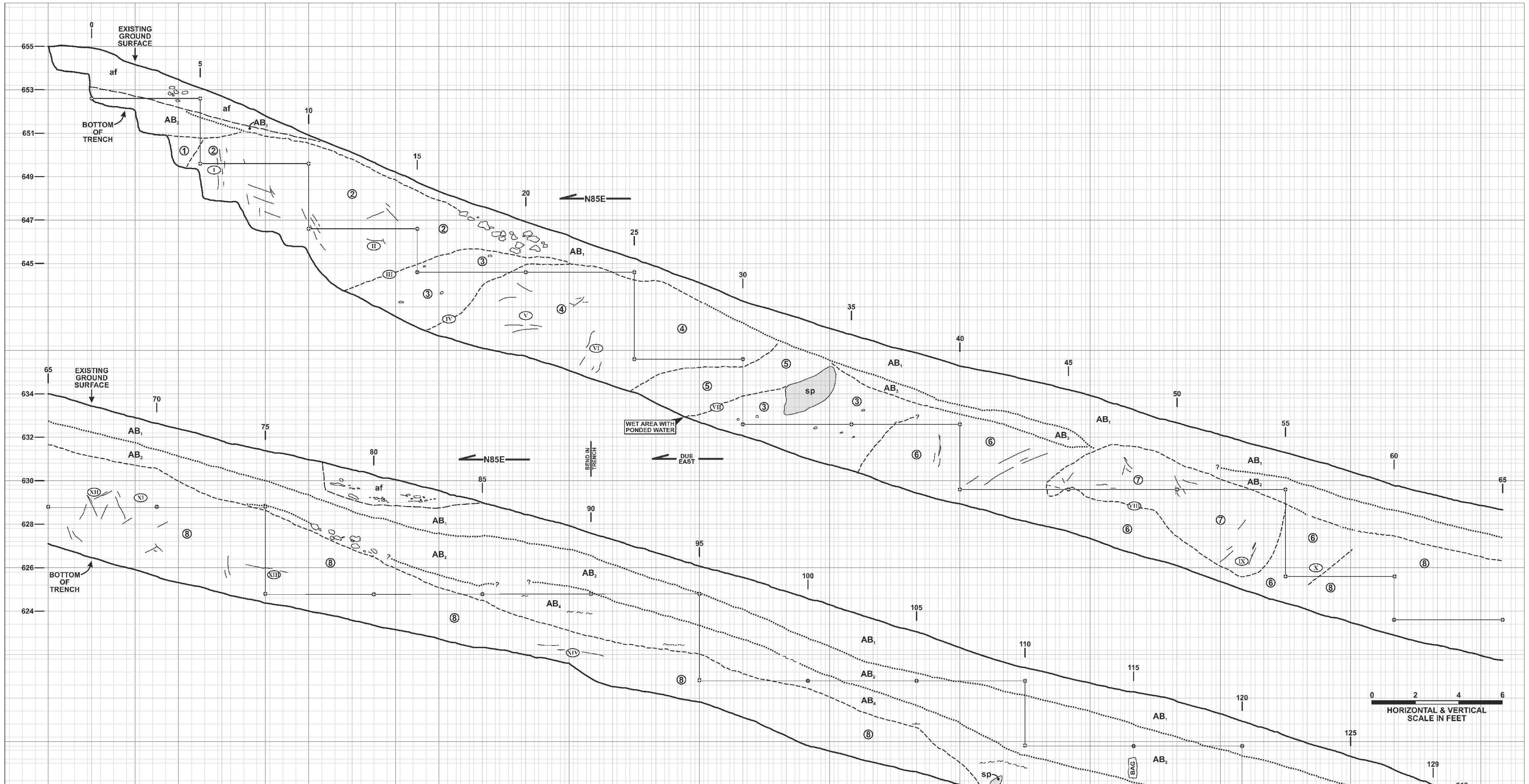
1. **Soft** - Reserved for plastic material alone.
2. **Low Hardness** - Can be gouged deeply or carved easily by a knife blade.
3. **Moderately Hard** - Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - Can be scratched by a knife blade with difficulty; scratch produces little powder and is often faintly visible.
5. **Very Hard** - Cannot be scratched by a knife blade; leaves a metallic streak

STRENGTH

1. **Plastic** - Very low strength.
2. **Friable** - Crumbles easily by rubbing with fingers.
3. **Weak** - An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately Strong** - Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** -Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very Strong** -Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

WEATHERING - the physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. **Deep** - Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
M. **Moderate** - Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
L. **Little** - No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
F. **Fresh** - Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.



EXPLANATION

FRANCISCAN COMPLEX BEDROCK

① Serpentinite, mottled greenish gray and yellowish brown, friable, small rock fragments with completely weathered matrix

② Serpentinite, greenish gray (5 GY 6/1), hard, closely fractured into cobble-sized fragments, slightly weathered with iron staining along fracture surfaces

③ Serpentinite, pale olive (10 YR 6/2), soft, completely weathered, internally sheared throughout, occasional hard polished clasts

④ Serpentinite, olive gray (5 Y 4/0), hard, closely fractured into cobble- and boulder-sized fragments, slightly weathered with iron staining along fractures

⑤ Serpentinite, mottled orange brown and black, friable, small rock fragments within completely weathered matrix

⑥ Serpentinite, mottled orange brown and gray green, friable, shattered harder rock fragments of all sizes, layered or banded appearance

⑦ Serpentinite, mottled black, hard, closely fractured into cobble- and boulder-sized fragments, slightly weathered with iron staining along fractures

⑧ Serpentinite, mottled black and dark green, hard, occasionally to closely fractured, cobble- to bouder-sized fragments up to 18" in diameter; fibrous NOA along some joint surfaces (chrysotile?)

Logged by Patrick Drumm, CEG No. 1916, 07/14/2024

SOILS

af: Lean clay with sand and rock fragments (CL); mottled gray, slightly moist, firm; some loose gravel zones, layered appearance *[artificial fill]*

AB_1: Lean to fat clay with sand and fine gravel (CL/CH); dark gray (10 YR 4/1), slightly moist, stiff; desiccation cracks, roots *[native soil]*

AB_2: Fat clay with sand (CH); very dark gray (10 YR 3/1), moist, firm; abundant random polished surfaces *[native soil]*

AB_3: Fat clay with sand (CH); mottled gray green and gray brown, moist, soft to firm *[native soil]*

AB_4: Fat clay with sand and fine gravel (CH); very dark gray (10 YR 3/1), moist, soft to firm; scattered discontinuous shear planes *[native soil]*

AB_5: Fat clay with sand and rock fragments (CH); dark brown (10 YR 4/3), moist firm; abundant clay films *[native soil]*

SYMBOLS

- Top and bottom of exploratory trench
- String line datum
- Approximate geologic contact
- Gradational geologic contact
- Layering or other planar fabric
- Weak blocky soil structure
- Internal shear planes within soils
- sp: Serpentinite boulder
- BAG: Soil sample collected for laboratory testing

ATTITUDES

- I Jointing: N05E / 80SE
- II Jointing: N72E / 58NW
- III Shear: N20E / 30SE (measured across trench)
- IV Shear: N32E / 38SE (measured across trench)
- V Foliation: N85E / 63SE
- VI Foliation: N40E / 76SE
- VII Shear: N36E / 18SE (measured across trench)
- VIII Foliation: N75E / 57SE
- IX Jointing: N75E / 68NW
- X Foliation: N70E / 59SE
- XI Jointing: N43W / 70SW
- XII Jointing: N42E / 75SE
- XIII Foliation: N75W / 43SW
- XIV Foliation: N78E / 69SE

3 0 6 12
APPROXIMATE SCALE FEET

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TRENCH LOG

POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	FIGURE
3194-1A	September 2024	A-3

DRILL RIG: Minute Man	SURFACE ELEVATION: +/- 547 feet (see note 4)	LOGGED BY: TP
DEPTH TO GROUNDWATER: (see note 1)	BORING DIAMETER: 3 inches	DATE DRILLED: 7/15/24

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean, Sandy - with fine to medium grained sand, some angular gravel, trace silt, dry	Dark Brown	Very Stiff	CL/CH	1		[28]	21	91	
				2		[41]	22	83	
SERPENTINITE - trace fine grained sand, trace silt, trace rock fragments, moist	Mottled Greenish Brown and Dark Brown	Hard	BEDROCK	3					

Bottom of boring at 3.2 feet.

NOTES:

1. No groundwater was encountered at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.

4. Elevations were estimated from topographic map by Kister, Savio & Rei, dated 4/9/24.

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EXPLORATORY BORING LOG

POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	SHEET	BORING NO.
3194-1A	**September 2024**	**1 of 1**	**B-1**

DRILL RIG:	SURFACE ELEVATION: +/- 542 feet (see note 4)	LOGGED BY: TP
DEPTH TO GROUNDWATER: (see note 1)	BORING DIAMETER: 3.5 inches	DATE DRILLED: 7/15/24

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean to Fat - some fine to medium grained sand, trace silt, trace angular gravel, dry to moist	Brown	Stiff to Very Stiff	CL/CH	1		[11]	19	82	
				2			14	107	
-gravel layer				3		[30]			
SERPENTINITE - trace fine grained sand, trace silt, trace rock fragments, moist	Greenish Brown	Hard	BEDROCK	4		50/6"	12		

Bottom of boring at 4.5 feet.

NOTES:

1. No groundwater was encountered at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. Elevations were estimated from topographic map by Kister, Savio & Rei, dated 4/9/24.

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EXPLORATORY BORING LOG
POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	SHEET	BORING NO. B-2
3194-1A	**September 2024**	**1 of 1**	

DRILL RIG: Truck Mounted, Solid Flight Auger	**SURFACE ELEVATION:** +/- 522 feet (see note 4)	**LOGGED BY:** TP
DEPTH TO GROUNDWATER: (see note 1)	**BORING DIAMETER:** 4.5 inches	**DATE DRILLED:** 7/15/24

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean to Fat - trace fine grained sand, trace silt, trace angular gravel, dry to moist	Brown	Stiff	CL/CH	1					
				2					
	Brown and Olive Brown	Very Stiff		3		[20]	20	97	LL = 72 PI = 52 -200 = 80%
CLAY, Fat - some rock fragments, trace fine to medium grained sand, trace silt, dry to moist	Reddish Brown	Hard	CH	4					
				5		11	19		
				6		[50/6"]	21	97	
				7					
-grinding on rock boulder at 8 feet				8		50/1"	17		

Bottom of boring at 8.5 feet.

NOTES:

1. No groundwater was encountered at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. Elevations were estimated from topographic map by Kister, Savio & Rei, dated 4/9/24.

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EXPLORATORY BORING LOG
POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	SHEET	BORING NO.
3194-1A	**September 2024**	**1 of 1**	**B-3**

DRILL RIG: Minute Man, Solid Flight Auger	**SURFACE ELEVATION:** +/- 511 feet (see note 4)	**LOGGED BY:** TP
DEPTH TO GROUNDWATER: (see note 1)	**BORING DIAMETER:** 3.5 inches	**DATE DRILLED:** 7/15/24

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
CLAY, Lean to Fat - trace fine grained sand, trace silt, trace angular gravel, dry to moist	Brown	Stiff	CL/CH	1		[11]	19	80	
				2					
				3					
				4					
CLAY, Fat - some rock fragments, trace of sand and silt, dry to moist	Brown	Very Stiff	CH	5					
SERPENTINITE - moderately weathered, moderately hard, trace clay/trace silt, trace fine grained sand, moist (possibly slide debris)	Black Mottled with Green and Brown	Hard	BEDROCK	6		[33]	36	74	
				7					

Bottom of boring at 7.0 feet.

NOTES:

1. No groundwater was encountered at the time of drilling and the boring was backfilled immediately after drilling. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. Elevations were estimated from topographic map by Kister, Savio & Rei, dated 4/9/24.

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EXPLORATORY BORING LOG
POLICE DEPARTMENT BUILDING SITE
Kensington, California

PROJECT NO.	DATE	SHEET	BORING NO.
3194-1A	**September 2024**	**1 of 1**	**B-4**

APPENDIX B

CGS NOTE 14

CALIFORNIA GEOLOGICAL SURVEY

SERPENTINE CALIFORNIA STATE ROCK

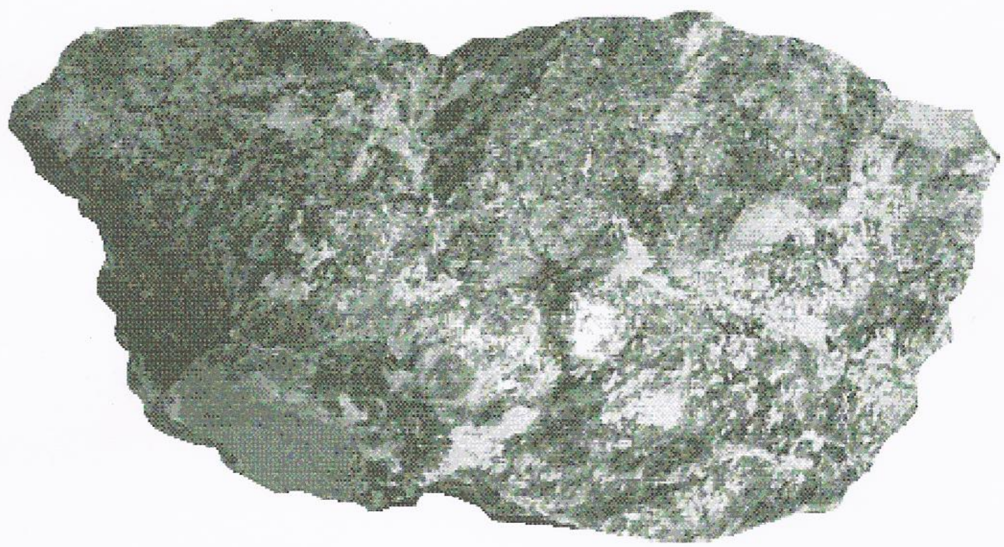
NOTE 14

Serpentine rock is apple-green to black and is often mottled with light and dark colored areas. It has a shiny or wax-like appearance and slightly soapy feel. Serpentine is usually fine-grained and compact but may be granular, platy or fibrous. It's found in central and northern California—in the Coast Ranges, Klamath Mountains and Sierra Nevada foothills.

Serpentine rock is primarily composed of one or more of the three magnesium silicate minerals: lizardite, chrysotile and antigorite. Chrysotile often occurs as fibrous veinlets in serpentine. Chrysotile in fibrous form is the most common type of asbestos. Asbestos is a group of silicate minerals that readily separates into thin, strong and flexible fibers that are heat resistant. Lizardite and antigorite don't form asbestos fibers and instead are plate-like.

Serpentine is metamorphic and/or magnesium-rich igneous rock, most commonly peridotite, from the earth's mantle. (The mantle is a thick layer of rock just below the earth's crust.) Peridotite underlying oceanic crustal rocks was metamorphosed to serpentine in subduction zones that existed at various times in California's past. (A subduction zone is where ocean crust rocks run into and slide underneath the edge of a continent.) Because serpentine has a much lower density than peridotite, it rose toward the surface along major regional thrust faults associated with the subduction zones.

Serpentine is the same rock type as serpentinite.



Serpentine often contains some asbestos. Exposure to asbestos fibers has potential human-health consequences. Therefore, the Air Resources Board restricts its use as unpaved road surface material.

For information on serpentine use in California, call the Air Resources Board at 800-363-7664 or your local Air Pollution Control District Office.

CALIFORNIA GEOLOGICAL SURVEY

CALIFORNIA CONSERVATION

Revised 5/2002

SERPENTINE FACTS

Composition: $Mg_6Si_4O_{10}(OH)_8$

Crystal system: Monoclinic (antigorite has a hexagonal polymorph, chrysotile two orthorhombic polymorphs).

Habit: Crystals unknown: the serpentine minerals usually occur in structureless masses, except when asbestiform.

Cleavage: None observable

Hardness: 4-6

Density: 2.5-2.6

Color: Usually green, also yellow, brown, reddish brown and gray.

Streak: White

Luster: Waxy or greasy in massive varieties, silky in fibrous material.

Occurrence: Serpentine is formed by the alteration of olivine and enstatite under conditions of low-and medium-grade metamorphism. It sometimes occurs as large rock masses. Massive serpentine is sometimes cut and polished as an ornamental stone.

Graphics by Dinah D. Maldonado, California Geological Survey.

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APPENDIX C

HOMEOWNER'S GUIDE TO MAINTENANCE

APPENDIX C

GUIDE TO THE MAINTENANCE OF HILLSIDE HOME SITES

During the wet winter season, homeowners, particularly those living in houses placed on fill (man-placed earth) or in the vicinity of excavated (cut) slopes, become concerned about the condition of their building site. In general, modern design and construction practice minimizes the probability of serious landsliding (slope failure). The grading codes of the local jurisdictions (cities and counties) in California concerning filled land, excavation, terracing, and slope construction are among the most stringent in the country and, if followed, are adequate to meet almost any natural occurrence. Therefore, the concern of the homeowner should be directed toward maintaining slopes, drainage provisions, and facilities so that they will perform as designed.

The following discussion, general recommendations, and simple precautions are presented to help the homeowner maintain their hillside building site.

The general public often regards the natural terrain as stable — "terra firma." This is, of course, an erroneous concept. Nature is always at work altering the landscape. Hills and mountains are worn down by mass wasting (erosion, sliding, creeping, etc.) and the valleys and lowlands collect these products. Thus the natural process is toward leveling the terrain. Periodically (over tens of millions of years), major land movements rebuild mountains and hills, and these processes begin again. In some areas these processes are very slow, and in others they are more rapid.

Development of hillsides for residential use is carried out, as far as possible, to enhance the natural stability of the site and to minimize the potential for instability resulting from the grading necessary to provide home sites, streets, yards, and other improvements. This has been done by the developer and designers on the basis of geologic and soil mechanics investigations. In order to be successful, the slope, drainage provisions, and facilities must be maintained by the homeowner.

Homeowners are accustomed to maintaining their homes. They expect to paint their houses periodically, replace wiring, clean out clogged plumbing, and repair roofs. Maintenance of the home site, particularly on hillsides, should be considered on the same basis, or even on a more serious basis because neglect can result in serious consequences. In most cases, lot and site maintenance can be taken care of along with landscaping, and can be carried out more economically than repair after neglect.

Most slope and hillside lot problems are associated with water. Uncontrolled water from a broken pipe, cesspool, or wet weather causes most damage. Wet weather is the largest cause of slope problems, particularly in California where rain is intermittent, but may be torrential. Therefore, drainage and erosion control are the most important aspects of home site stability; these provisions must not be altered without competent professional advice. Further, maintenance must be carried out to assure their continued operation.

As geotechnical engineers concerned with the problems of building sites in hillside developments, we offer the following list of recommended "Do's and Don'ts" as a guide to homeowners.

----- ----- -----

1. DO check roof drains, gutters and down spouts to be sure they are clear. Depending on your location, if you do not have roof gutters and down spouts, you may wish to install them because roofs, with their wide, flat area can shed tremendous quantities of water. Without gutters or other adequate drainage, water falling from the eaves collects against foundation and basement walls, which can be undesirable.

2. DO clear surface and terrace drainage ditches, and check them frequently during the rainy season. Use a shovel, if necessary. Ask your neighbors to do likewise.

3. DO be sure that all drainage ditches have outlet drains that are open. This should be tested during dry weather and can usually be done with a hose. If blockage is evident, you may have to clear the drain mechanically.

4. DO check all drains at top of slopes to be sure they are clear and that water will not overflow the slope itself, causing erosion.

5. DO keep subsurface drain openings (weep-holes) clear of debris and other material which could block them in a storm.

6. DO check for loose fill above and below your property if you live on a slope or terrace.

7. DO monitor hoses and sprinklers. During the rainy season, little, if any, irrigation is required. Over-saturation of the ground is unnecessary, increases watering costs, and can cause subsurface drainage.

8. DO watch for water backup of drains inside the house and toilets during the rainy season, as this may indicate drain or sewer blockage.

9. DO exercise ordinary precaution. Your house and building site were constructed to meet certain standards which should protect against any natural occurrence if you do your part in maintaining them.

----- ----- -----

1. DON'T block terrace drains and brow ditches on slopes or at the tops of cut or fill slopes. These are designed to carry away runoff to a place where it can be safely distributed. Generally, a little shovel work will remove any accumulation of dirt and other debris which may clog the drain. If several homes are located on the same terrace, it is a good idea to check with your neighbors. Water backed up on their property may eventually reach you. Water backed up in surface drains will tend to overflow and seep into the terraces, creating less stable slopes. Maintain the ground surface upslope of lined ditches to ensure that surface water is collected in the ditch and is not permitted to be trapped behind or under the lining.

2. DON'T permit water to collect or pond on your home site. Water gathering here will tend to either seep into the ground (loosening fill or natural ground), or will overflow into the slope and begin erosion. Once erosion is started, it is difficult to control and severe damage may result rather quickly.

3. DON'T connect roof drains, gutters, or down spouts to subsurface drains. Rather, arrange them so that water either flows off your property in a specially designed pipe or flows out into a paved driveway or street. The water then may be dissipated over a wide surface or, preferably, may be carried away in a paved gutter or storm drain. Subdrains are constructed to take care of ordinary subsurface water and cannot handle the overload from roofs during a heavy rain.

4. DON'T permit water to spill over slopes, even where this may seem to be a good way to prevent ponding. This tends to cause erosion and, in the case of fill slopes, can eat away carefully designed and constructed sites.

5. DON'T drop loose soil or debris over slopes. Loose soil soaks up water more readily than compacted fill. It is not compacted to the same strength as the slope itself and will tend to slide when laden with water; this may even affect the soil beneath the loose soil. The sliding may clog terrace drains below or may cause additional damage in weakening the slope. If you live below a slope, try to be sure that loose fill is not dumped above your property.

6. DON'T discharge water into subsurface blanket drains close to slopes. Trench drains are sometimes used to get rid of excess water when other means of disposing of water are not readily available. Overloading these drains saturates the ground and, if located close to slopes, may cause slope failure in their vicinity.

7. DON'T discharge surface water into septic tanks or leaching fields. Not only are septic tanks constructed for a different purpose, but they will tend, because of their construction, to naturally accumulate additional water from the ground during a heavy rain. Overloading them artificially during the rainy season is bad for the same reason as subsurface subdrains, and is doubly dangerous since their overflow can pose a serious health hazard. In many areas, the use of septic tanks should be discontinued as soon as sewers are made available.

8. DON'T over-irrigate slopes. Naturally, ground cover of ice plant and other vegetation will require some moisture during the hot summer months, but during the wet season, irrigation can cause ice plant and other heavy ground cover to pull loose. This not only destroys the cover, but also starts serious erosion. In some areas, ice plant and other heavy cover can cause surface sloughing when saturated due to the increase in weight and weakening of the near-surface soil. Planted slopes should be planned where possible to acquire sufficient moisture when it rains.

9. DON'T let water gather against foundations, retaining walls, and basement walls. These walls are built to withstand the ordinary moisture in the ground and are, where necessary, accompanied by subdrains to carry off the excess. If water is permitted to pond against them, it may seep through the wall, causing dampness and leakage inside the basement. Further, it may cause the foundation to swell up, or the water pressure could cause structural damage to walls.

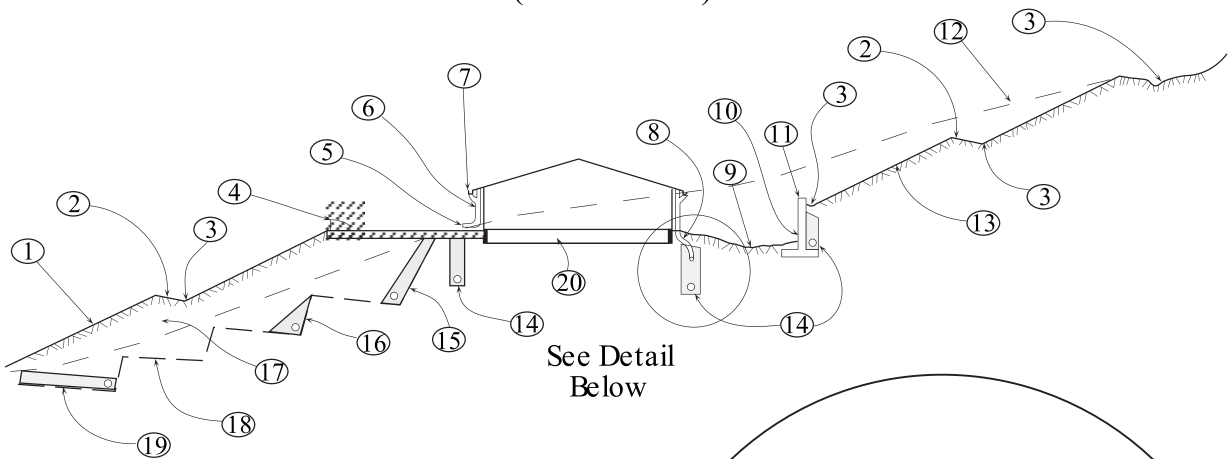
10. DON'T try to compact soil behind walls or in trenches by flooding with water. Not only is flooding the least efficient way of compacting fine-grained soil, but it could damage the wall foundation or saturate the subsoil.

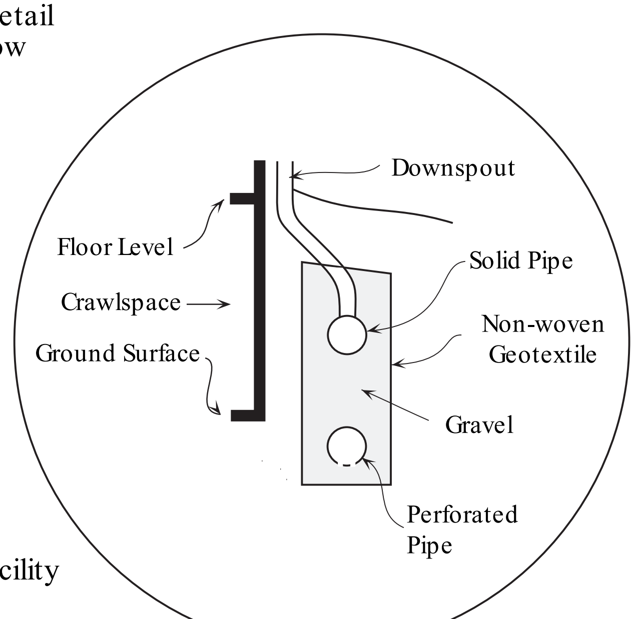
11. DON'T leave a hose and sprinkler running on or near a slope, particularly during the rainy season. This will enhance ground saturation which may cause damage.

12. DON'T block ditches which have been graded around your house or the lot pad. These shallow ditches have been put there for the purpose of quickly removing water toward the driveway, street or other positive outlet. By all means, do not let water become ponded above slopes by blocked ditches.

A typical slope section showing various grading and drainage requirements, as well as terms used for hillside developments, is attached.

TYPICAL SLOPE SECTION
(Not to Scale)





DETAIL
(Not to Scale)

CONDITIONS:

1. Fill Slope
2. Drainage Terrace
3. Lined Ditch
4. Curb to Prevent Slope Erosion
5. Drain Pipe Provided with Positive Outlet on Paved Surface
6. Downspout
7. Roof Gutter
8. Drain Pipe Connected to Solid Pipe and Discharged to an Approved Drainage Facility
9. Drainage Swale
10. Weep Holes Through Retaining Wall
11. Retaining Wall
12. Original Ground Surface
13. Cut Slope
14. Subdrain
15. Curtain Drain
16. Wedge Drain
17. Fill Compacted to Engineering Specifications and Benched into Competent Material
18. Bench
19. Blanket Drain
20. Crawlspace

Note: Acknowledgment is hereby made to the San Diego Chapter of the California Council of Civil Engineers and Land Surveyors

ALAN KROPP
& ASSOCIATES
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TYPICAL SLOPE DETAIL

FIGURE C-1