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Bill Olechnowicz, Project Architect
MWA Architects
471 Ninth Street
Oakland, California 94607

Preliminary Geotechnical & Geologic Report
Skylonda Fire Station No. 58
17290 Skyline Boulevard
San Mateo County, California

Dear Mr. Olechnowicz:

Transmitted herewith is the report of our preliminary geotechnical and geologic evaluation of the Skylonda Fire Station site on Skyline Road, San Mateo County. This report includes the results of our literature research and site reconnaissance by both our Certified Engineering Geologist and Registered Geotechnical Engineer. The conclusions, opinions, and recommendations presented in this report are based information obtained from these tasks and have not benefited from a site-specific investigations or laboratory testing.

We thank you for the opportunity to perform these services. Please do not hesitate to contact us, should you have any questions or comments.

Very truly yours,
BAGG Engineers

REGISTERED PROFESSIONAL ENGINEER
JASON VAN ZWOL
No. 854
Exp 6/30/14
GEOTECHNICAL
STATE OF CALIFORNIA

Jason Van Zwol
Principal Engineer

REPORT

PRELIMINARY GEOLOGIC & GEOTECHNICAL EVALUATION
SKYLONDA FIRE STATION No. 58
17290 SKYLINE BOULEVARD
SAN MATEO COUNTY, CALIFORNIA

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REPORT

PRELIMINARY GEOLOGIC & GEOTECHNICAL EVALUATION
SKYLONDA FIRE STATION No. 58
17290 SKYLINE BOULEVARD
SAN MATEO COUNTY, CALIFORNIA

1.0 INTRODUCTION

This report summarizes the findings of our preliminary geologic and geotechnical evaluation of the Skylonda Fire Station No. 58, located on the southwest side of Skyline Boulevard, about 700 feet north of its intersection with La Honda Road in San Mateo County. The attached Plate 1, Vicinity Map, shows the general location of the site, and Plate 2, Site Plan, shows the layout of the existing site. This report was prepared in accordance with the scope of services outlined in our Proposal Number 13-436, dated October 8, 2013.

2.0 SITE DESCRIPTION

The subject property is occupied by a metal apparatus building measuring roughly 40 by 120 feet near Skyline Blvd, with a relatively large and generally flat paved area in front of the building. This flat pad area was apparently created on the order of 50 to 60 years ago and contains a steep cut bank along the northern side and a fill slope along the south and southwestern sides. Access is from Linwood Way near the northwest end of the site, with a second driveway that passes the office and barracks building to the east and enters Skyline near Alice's Restaurant at La Honda Road.

Available plans indicate there are five leach lines located in front of the existing apparatus building. Two old lines are reportedly about 10 feet deep and located parallel to and roughly 10 and 26 feet from the building. Three newer lines, about 7 to 8 years old, are spaced at 10 feet and are about 4½ feet deep.

The site contains several scattered fir and redwood trees around the perimeter and near the office building. The cut slope immediately behind the building is roughly 12 feet in height and is nearly vertical. The fill slope on the southwest side of the site is roughly 10 feet in height at a gradient of roughly 2H:1V (horizontal to vertical). The native slopes in the immediate area are more gentle on the order of 4 to 6H:1V.

3.0 PROJECT DESCRIPTION

The exact nature of the project is not known at this time. Alternatives being considered include: construction of a new barracks building; construction of a complete new facility, including apparatus building, dispatch, and barracks; or construct a new facility at an unidentified new site. The location of new structures on the subject site could also be located within the general area of the existing structures, or could occupy a significant portion of the paved area in front (southwest) of the existing apparatus building.

Buildings would most likely be two-story structures, except the apparatus building would be a high-bay structure. Consideration has also been given to providing a lower floor, or basement with access to Linwood and/or Lakewood Way below the existing fill slope at the edge of the paved area.

4.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our services was to provide preliminary geologic and geotechnical evaluation of the site, with general criteria for the design and construction of the proposed buildings. As indicated, this has been accomplished by performing a research and review of geologic and geotechnical literature pertinent to the site area, performing a geologic reconnaissance of the site and immediate vicinity, and performing engineering analyses as needed to develop preliminary conclusions, opinions, and recommendations regarding:

- Geologic setting of the site
- Geologic hazards affecting the site
- General criteria site grading and earthwork
- Expected requirements for foundation types and design criteria
- Lateral earth pressures for retaining wall design
- Support for slabs-on-grade and pavements

Toward this end, the scope of our services consisted of the following specific tasks:

1. Conduct a review of the available geologic literature, including maps, published reports, Special Studies Zone maps, and geo-hazard maps pertinent to general site area.

2. Conduct an engineering geologic as well as a geotechnical site reconnaissance to map any potential geologic hazards that may affect the building site and immediate vicinity, as well as geotechnical constraints impacting the future site development.

3. Prepare a consultation report summarizing the results of our geologic reviews and reconnaissance, as well as our preliminary recommendations for site grading, building foundations, and drainage requirements for alternatives being considered.

5.0 GEOLOGY AND SEISMICITY

5.1 Regional Geology

The site and the San Francisco Bay Area lie within the Coast Ranges geomorphic province, a series of discontinuous northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the San Francisco Bay Area is illustrated in studies by Schlocker (1970), Wagner et al. (1991), Chin et al. (1993), and Wentworth et al. (1995) among others.

The site is located along the northern portion of the Santa Cruz Mountains along the top of a ridgeline that extends northwestward along the west side of the San Andreas fault, paralleling it, in San Mateo County. Geologic and geomorphic structures within the San Francisco Bay Area are dominated by the San Andreas fault, a right-lateral strike-slip fault that extends from the Gulf of California in Mexico to the Coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates. To the west of the San Andreas fault is the Pacific Plate and to the east, the North American Plate. In the San Francisco Bay Area, movement along this plate boundary is concentrated on the San Andreas fault and to a lesser magnitude, along a number of other faults that include the Hayward and Calaveras faults among others.

Basement rocks west of the San Andreas fault zone are generally granitic, while to the east they consist of a mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205-65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.8 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have been extensively folded and faulted as a result of late Tertiary and Quaternary regional compressional forces. The inland valleys, as well as the structural depression within which the San Francisco Bay is located, are filled with unconsolidated to semi-consolidated continental deposits of Quaternary age (about the last 1.8 million years). Continental surficial deposits (alluvium, colluvium, and landslide deposits) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel while the Bay deposits typically consist of very soft organic-rich silt and clay (Bay mud) or sand.

5.2 Site Geology

The site area has been mapped by the California Division of Mines and Geology (1961), Brabb and Pampeyan (1972 and 1983), Wentworth et al. (1985), Pampeyan (1994), and Brabb et al. (1998). Brabb et al. (1998) show the site area to be within the Skylonda structural block and they map the bedrock occupying the site area as Lambert shale (Oligocene to lower Miocene), a whitish siliceous shale bedrock that is considered to be a member of the Monterey formation.

Our consulting Certified Engineering Geologist (CEG) performed a reconnaissance of the fire station and surrounding areas on November 17, 2013. The site is situated along Skyline Boulevard a long the top of a ridgeline. The main apparatus building is a steel shell building that is situated along the north side of the site where a relatively level and board pad paved pad has been created by cutting into the hillside side immediately west of Skyline boulevard. The cut measures up to 12 feet in height and exposes colluvial soils comprised a sandy/silty matrix supporting whitish siliceous shale fragments along the north end of the main building. Immediately behind the central portion of the building where the cut slope is highest, in-place siliceous shale bedrock is exposed. The shale appeared laminated, friable, weak, gritty, closely and highly fractured, and bedded striking about 40 degrees west of north and dipping about 12 to 15 degrees northeastward (into the hill).

The eastern, inboard half of the relatively level and broad paved pad area appeared to be made by cutting into the hill while the outer western margin appeared to have been created by placing the cut materials as fill. A fill wedge measuring about 10 feet in height with an approximate gradient of up to about 2H:1V marks the northern portion of the western side of the pad area. Beyond the fill wedge to the west, the original slope measured less than 10 feet in height with an approximate gradient of about 6H:1V and extended down to Blakewood Drive.

5.3 Faulting

The general area, as is the entire San Francisco Bay Area, is considered to be an active seismic region due to the presence of several active earthquake faults. Four, northwest-trending major earthquake faults that comprise the San Andreas fault system extend through the Bay Area. They include the San Andreas fault located about 2 km to the east-northeast, the Monte Vista-Shannon fault located about 4¾ km to the southeast, the Hayward fault located about 32 km to the northeast, and the Calaveras fault located about 40 km to the east. The inactive Pilarcitos fault is mapped about 0.8 km to the northeast of the site, and the San Gregorio fault is located roughly 13 km to the west southwest.

The following table lists the nearest major faults in the area, their distance to the site, and their expected maximum magnitude earthquake.

Table 1
Significant Earthquake Scenarios

Fault	Approximate Distance from Site (kilometers)[1]	Direction from Site	Potential Moment Magnitude (M_W)[2]
Pilarcitos	0.8	NE	n/a
San Andreas (Entire)	2	ENE	7.9-8.0
San Andreas (Peninsula)	2	ENE	7.1-7.2
Monte Vista – Shannon	4¾	SE	6.3-6.5
San Gregorio	13	WSW	7.4-7.5
Hayward – Rogers Creek	32	NE	7.2-7.3
Calaveras	40	ENE	6.8-7.0

[1]USGS Fault files w/ Google Earth
[2]Working Group on California Earthquake Probabilities, 2008.

5.4 Liquefaction Potential

Soil liquefaction is a condition where saturated granular soils near the ground surface undergo a substantial loss of strength due to increased pore water pressure resulting from cyclic stress applications induced by earthquakes or other vibrations. In the process, the soil acquires mobility sufficient to permit both vertical and horizontal movements, if not confined. Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained, sands, and loose silts with very low cohesion. The fill soils in the western portion of the site, which were likely obtained from cuts to the east are expected to contain significant clayey fines and are considerably above the expected water table.

Youd and Perkins (1987), Knudsen et al. (1997 and 2000), and Witter et al. (2006) show the site area to be underlain by bedrock where the potential for liquefaction is considered nil. The site area is underlain by bedrock and fill soils of unknown quality. However, we understand the fill soils have been in place for over 50 years (giving them time to consolidate somewhat) and the surface pavement, which was placed about 7 years ago, is in good shape, suggesting the fill is firm and relatively dense. In addition, groundwater is anticipated to be relatively deep. Furthermore, there is no history of liquefaction or historic ground failures associated with earthquakes at the site.

However, some earthquake-induced slope failures were reported in areas to the west of the site near Lower Crystal Springs Reservoir (Youd and Hoose, 1978). Based on this information, it is our opinion that the liquefaction potential is considered to be low to nil.

5.5 Other Geologic Hazards

5.5.1 Potential for Fault-Related Ground Surface Rupture

The Skylonda fire station site is not situated within an Alquist-Priolo Earthquake Fault Zone established by the CGS around faults that are considered as active (CGS, 2000). The closest fault to the site is the inactive Pilarcitos fault, which is mapped less than half a mile to the northeast of the site as noted above and the closest active and zoned fault capable of producing ground surface rupture to the site is the San Andreas fault, which is located about 1.5 miles to the east northeast. Based on this information, it is our opinion that the potential for fault-related ground surface rupture at the school campus is low.

5.6.2 Potential for Lateral Spreading

There are no creek channels crossing the fire station site or bordering it. The site area is generally underlain by bedrock and the potential for liquefaction is considered low. In addition, the groundwater level is anticipated to be relatively deep. Based on this information, the potential for lateral spreading to occur within the site limits is considered minimal.

5.6.3 Potential for Slope Instability

The site area is situated along a ridge top with relatively gentle localized slope areas. No slope failures or signs or slope instabilities were observed along the sloping areas by our consulting CEG during his reconnaissance of the site area. The area beyond Skyline Boulevard to the east is relatively level and lacking a driving force, which would impact the stability of the localized sloping areas. Therefore, the potential for slope instabilities to occur and impact the proposed development is considered very low.

5.6.4 Potential for Flooding

The site is situated at an approximate elevation of 1,500-foot above mean sea level and there are no dams located in the area at a higher topographic position than the site. Accordingly, the potential for flooding at the site is considered low

5.6.5 Potential for Tsunami and Seiches

Tsunamis are seismic sea waves that are typically an open ocean phenomena caused by underwater landslides, volcanic eruptions, or seismic evens. They primarily impact low-lying coastal areas. Being near the top of the Santa Cruz Mountains, tsunamis are not believed possible.

Seiches are earthquake-generated waves or oscillations (sloshing) of the water surface in restricted bodies of water. The closest body of water is the Skylonda Reservoir located roughly 100 feet from the property and on the order of 25 feet lower in elevation. Thus, we judge the potential for seische-related flooding to occur at the site to be very low.

5.6.6 Town of Woodside Geologic Hazard Zones

Cotton, Shires and Associates, Inc. (May, 2012) prepared a map titled *Geologic Hazard Zones* which shows fault hazard zones (Zone FS), slope instability zones (Zone S), and expansive bedrock zones (Zone E). The site is situated outside the limits of all the above-noted geologic hazard zones. The site is shown to be located in standard constraints (Zone A).

6.0 ANTICIPATED GEOTECHNICAL CONDITIONS

As indicated the site is underlain by fill soils in the western portion of the potential building site, and by the Lambert shale formation, with an overlying blanket of residual and/or colluvial soils. The anticipated engineering characteristics of these materials are described below

6.1 Fill Soils

Based on available information, we understand the site was originally graded sometime in the 1940s or 50s. This makes the fill embankment at least 50 years old, which means it has most likely come to an equilibrium under current conditions. The paved surface, both in front of the apparatus building and near the adjacent fill slope, are in relatively good condition, which suggest that at least the top portion of the fill has been somewhat compacted and is able to support the fire trucks without rutting.

Based on the cut bank exposed behind the existing apparatus building, we would expect the fill soils to consist of a gravelly clay. We would also anticipate the fill soils would be able to support parked fire trucks on an appropriately reinforced concrete slab. However, buildings would most likely have to be supported at depth on the native bedrock materials.

6.2 Native Soils

As indicated, the native soils consist of a blanket of residual and/or colluvial soils overlying a siliceous shale bedrock. Soils blanketing the Lambert shale are usually not expansive, and are expected to provide relatively good foundation support. The surficial soils are expected to be variable in thickness but are typically on the order of 5 feet thick.

The lower bedrock should provide very good foundation support.

6.3 Groundwater

The depth to groundwater is not known, but is expected to be at considerable depth in this area; however, zones of seepage frequently can and do develop at the base of soils and on top of firmer bedrock materials.

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 General

Based on the available information and our site reconnaissance, it is our opinion that the proposed project is feasible from a geotechnical engineering viewpoint. However, all conclusions and recommendations presented in this report must be verified or modified based on a site-specific subsurface investigation consisting of several borings and subsequent laboratory testing of collected soil samples. When more detailed development plans are available, they should be submitted to our office so the field exploration can be properly designed to address the proposed development.

Based on available historic information and our surface observations, we anticipate the existing fill soils will be able to support fire trucks parked in a new apparatus building with an appropriately reinforced concrete floor slab. Subject to the results of a site investigation, we recommend that any new building on the site should be supported on firm native soils or bedrock materials at depth. Depending on the building location and the proposed grading, this could be accomplished with either spread footings or drilled, reinforced concrete piers.

The site will experience very strong ground shaking from future earthquakes during the anticipated lifetime of the project. The intensity of the ground shaking will depend on the magnitude of the earthquake, distance to the epicenter, and the response characteristics of the on-site soils. While it is not possible to totally preclude damage to structures during major earthquakes, strict adherence to good engineering design and construction practices will help reduce the risk of damage.

7.2 CBC Seismic Design Parameters

Based on the geology of the site and vicinity, it is our opinion that the site will be classified as a "soft rock/very dense soil" with blow counts greater than 50 and a Class "C" profile. Wills, et al (2000) also has classified the Lambert Shale and Monterey formation as Class "C".

Using the site coordinates of 37.3877 degrees North Latitude and 122.2669 degrees West Longitude, and the USGS website for U.S. Seismic Design Maps (http://geohazards.usgs.gov/designmaps/us/application.php), earthquake ground motion parameters were computed in accordance with 2013 California Building Code are as listed in the following table. If the site is to be designed according to the 2010 Building Code, let us know and we will provide revised parameter values.

Table 2
Parameters for Seismic Design

2013 CBC Site Parameter	Value
Site Latitude	37.3877°N
Site Longitude	122.2669° W
Site Class – ASCE 7-10, Table 20.3-1	Soft Rock – Class C
Mapped Spectral Acceleration for Short Periods S_s – Figure 1613.3.1(1	2.480g
Mapped Spectral Acceleration for 1-second Period S_1 – Figure 1613.3.1(2)	1.095g
Site Coefficient F_a – Table 1613A.3.3(1)	1.0
Site Coefficient F_v – Table 1613A.3.3(2)	1.3

7.3 Site Grading

Site grading on this site is not expected to be significant, unless a basement or lower level is added to the new structure located within the existing fill area west and southwest of the existing apparatus building. This would require excavations and backfill behind retaining walls. If this is done, the excavated soils will have to be either hauled off-site, or placed as engineered fill somewhere else on the site. These items are discussed below.

In general, the term compact and its derivatives mean that all on-site soils and/or imported fill soils should be moisture conditioned to slightly over optimum moisture content, and compacted to 95 percent within the top 12 inches of pavement subgrades and anywhere below foundations in accordance with ASTM Test Method D1557, and to at least 90 percent in other areas. The term also implies that fill materials should be placed in layers not exceeding 8 inches in loose thickness, and each lift should be thoroughly moisture conditioned and compacted before succeeding lifts are placed.

Excavation can be accomplished with conventional equipment and is not expected to encounter groundwater. Excavations should be sloped or shored in accordance with CalOSHA requirements. We anticipate the upper fills must be classified as a Type "A" soils, while the native soils will likely be classified as a Type "B" soil.

All aspects of site grading including clearing/stripping, demolition, building pad preparation, placement of fills or backfills and preparation of subgrades should be performed under the observation of BAGG's field representatives. It must be the Contractor's responsibility to select equipment and procedures that will accomplish the grading as described above. The Contractor must also organize his work in such a manner that one of our field representatives can observe and test the grading operations.

7.4 Foundations

Based on our preliminary soils information, it is our opinion that the anticipated buildings should be supported on foundations established in firm native soils or bedrock. Depending on the location and on the amount of grading performed at the site, this can be accomplished with either conventional spread footings, or drilled piers.

Where buildings will straddle a transition from cut to fill, the majority of the building will have to be supported on drilled piers. Only in those areas where it can be confirmed by the Geotechnical Engineer's observations in the field that the grade beams expose firm, competent bedrock (not surficial soils), can the piers be eliminated. Pending the site investigation, it should be anticipated that the suitable bedrock is blanketed by at least 5 feet of residual and/or colluvial soils.

Alternatives being considered also indicate there is a possibility the new structure may span over the existing leach lines. Because the leach lines are likely backfilled with loose rock, continuous footings should be designed to span a distance of at least 4 feet across the leach lines. Isolated footings should not be located on top of the leach lines, or within three feet of the edge of the trench. Drilled piers in the vicinity of the leach lines should derive support from soils/bedrock below the bottom of the trenches, or below a plane rising at 1:1 from the bottom of the trench.

7.4.1 Conventional Shallow Footings

Shallow footings should established at a minimum depth of 18-inches below the lowest adjacent final grade and penetrate at least 12 inches into firm native soils where fill is present. We anticipate such footings can be designed with bearing pressures of about 2,000 psf for dead loads and 3,000 psf for total design loads. The total design pressures may be increased by one-third for short-term loads such as wind or seismic loads.

The bottom of the foundation excavations should be firm, clean, and free of any loose or yielding soils. BAGG should be contacted to inspect the footings prior to placement of steel and concrete. The foundation excavation should not be allowed to dry out or crack. Any dried, cracked soils, as determined by the Geotechnical Engineer, should be removed to expose firm, moist soil and replaced with properly moisture conditioned and compacted fill soils, or lean concrete.

7.4.2 Drilled Piers

Where conventional footings reaching firm native soils would be unfeasible, building loads should be supported on drilled, reinforced, and cast-in-place concrete piers. Within the native soils skin friction support is expected on the order of 400 to 500 psf for total loads. Skin friction within the upper fill soils should be ignored for supporting vertical loads.

Pier drilling will have to be performed with the full-time observation of the Geotechnical Engineer to verify that each pier penetrates into suitable native soil and/or bedrock. All pier holes should be relatively clean and free of loose soils before reinforcing steel or concrete is placed in the hole. Although unlikely, if water or seepage is encountered in the pier hole, it should be pumped from the hole before concrete is poured, or the concrete should be placed with a tremie pipe to displace the water from the hole.

7.5 Retaining Walls

Retaining walls should be designed to resist lateral earth pressures from adjoining natural materials and backfills. We anticipate free standing walls supporting native materials or compacted fill soils can be designed to resist active lateral pressures taken as an equivalent fluid pressure of 45 pounds per cubic foot (pcf) for level backfill, while restrained walls will be designed to resist “at-rest” soil pressures based on an equivalent fluid weight of 65 pcf. These pressures will have to be increased by about 2 pcf for every 5 degrees increase in backfill slope. Seismic loading on the below-grade retaining walls may be taken as a rectangular pressure distribution equal to 10H, where H is the height of the wall. In addition, surcharge pressures should be added to the lateral load on the walls at the rate of 30 percent of the applied vertical load for cantilevered walls and at the rate of 50 percent for fully restrained walls.

Retaining walls should be supported on foundations as described in the “Foundations” section of this report.

The above lateral pressures do not include any hydrostatic pressures resulting from groundwater, seepage water, or infiltration of natural rainfall and/or irrigation water behind the walls. Therefore all walls over 3 feet in height should have a drainage blanket provided behind the wall. The drainage blanket should consist of a pre-manufactured drainage panel, or a one-foot-thick blanket of Caltrans Class 2 Permeable rock, or free-draining gravel encapsulated by a suitable filter fabric. A 12-inch cap of relatively impermeable soil should be placed at the top of the drainage blanket to minimize infiltration of surface water. The cap material should be compacted to a minimum of 90 percent relative compaction. A 4-inch diameter perforated PVC pipe could be installed at the base of the drainage blanket or the drainage layer to facilitate removal of water collected behind the wall.

7.6 Lateral Design

The lateral loads acting on the spread footings may be resisted by a combination of passive soil resistance and friction between the bottom of the footings and firm soil. The allowable passive resistance within firm native soils is expected to be on the order of 350 pcf. Within the existing fill, this will likely be reduced to about 250 pcf. For isolated piers, these values can be assumed to act over 1½ times the pier diameter.

The friction coefficient between the bottom of poured-in-place footings (not pier-supported grade beams) and undisturbed native soil is estimated to be 0.30. Both base friction and lateral passive resistance may be used in combination without reduction.

7.7 Slabs-on-Grade and Exterior Flatwork

Concrete floor slabs or exterior flatwork should be constructed on well compacted and moisture conditioned soil subgrade. All slabs should be reinforced as per the project Structural Engineer's recommendations. The subgrade should be approved by the Geotechnical Engineer immediately before the slab is poured.

In areas where moisture on the slab surface would be undesirable, 4 inches of approved, clean, free draining angular gravel should be placed beneath the concrete slab. The base course is intended to serve as a capillary break; however, moisture may accumulate in the base course zone. Therefore, a vapor barrier with a thickness of at least 15 mil (such as StegoWrap® or an approved equivalent) should be placed on the gravel base if moisture protection is desired and a damp slab is not desirable.

7.8 Pavement Design

It appears the existing pavement is functioning relatively well; however, typical pavement design is for an expected 20-year life and we understand the existing pavement has been in place for only about 7 or 8 years. Nevertheless, we would anticipate that design of a new AC pavement based on R-value testing of the subgrade soils would not require pavement thicknesses significantly different from the existing.

If the new development places the apparatus building at the surface of the existing fill soils, we would anticipate it will be necessary to re-work the upper 18 inches to 2 feet of subgrade soil and use a heavy concrete pavement (6 or 7 inches) for parking the fire trucks. (The existing apparatus building is located within a cut area and supported on firm native materials.)

8.0 CLOSURE

This report has been prepared based on our understanding of the proposed construction as described herein, on research of published literature pertinent to the site and vicinity, and on a reconnaissance of the site by our Certified Engineering Geologist and Registered Geotechnical Engineer. A site-specific soil investigation has not been completed at the site. The recommendations presented in this report are therefore only preliminary in nature, and must be substantiated or modified as necessary by a site-specific investigation consisting of subsurface soil borings and laboratory testing of soil and/or bedrock samples collected from the borings. No warrantee of any kind is given with this report.

The following references and plates are attached and complete this report:

Plate 1	Vicinity Map
Plate 2	Site Plan
Plate 3	Regional Geologic Map
Plate 4	Regional Fault Map

11.0 REFERENCES

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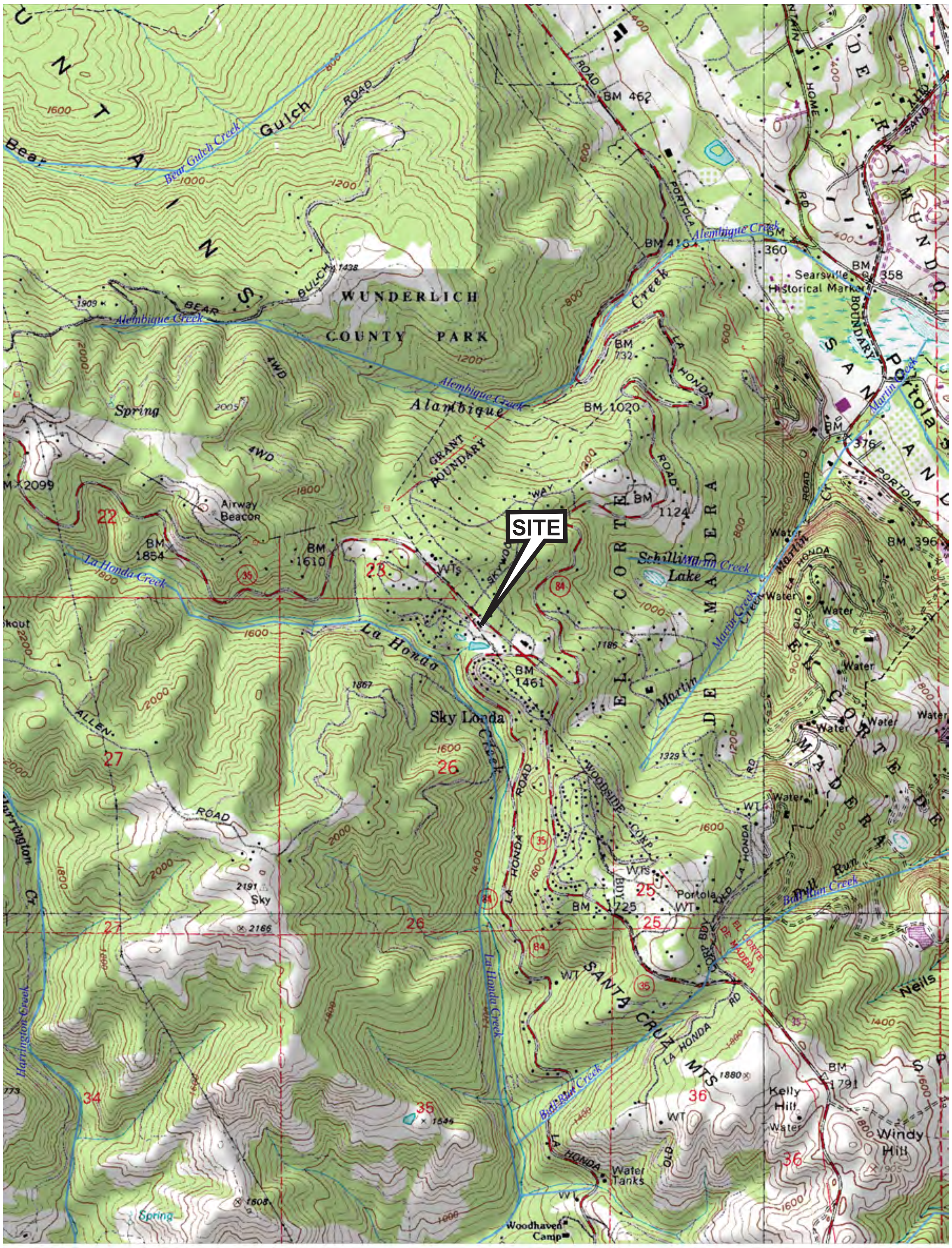

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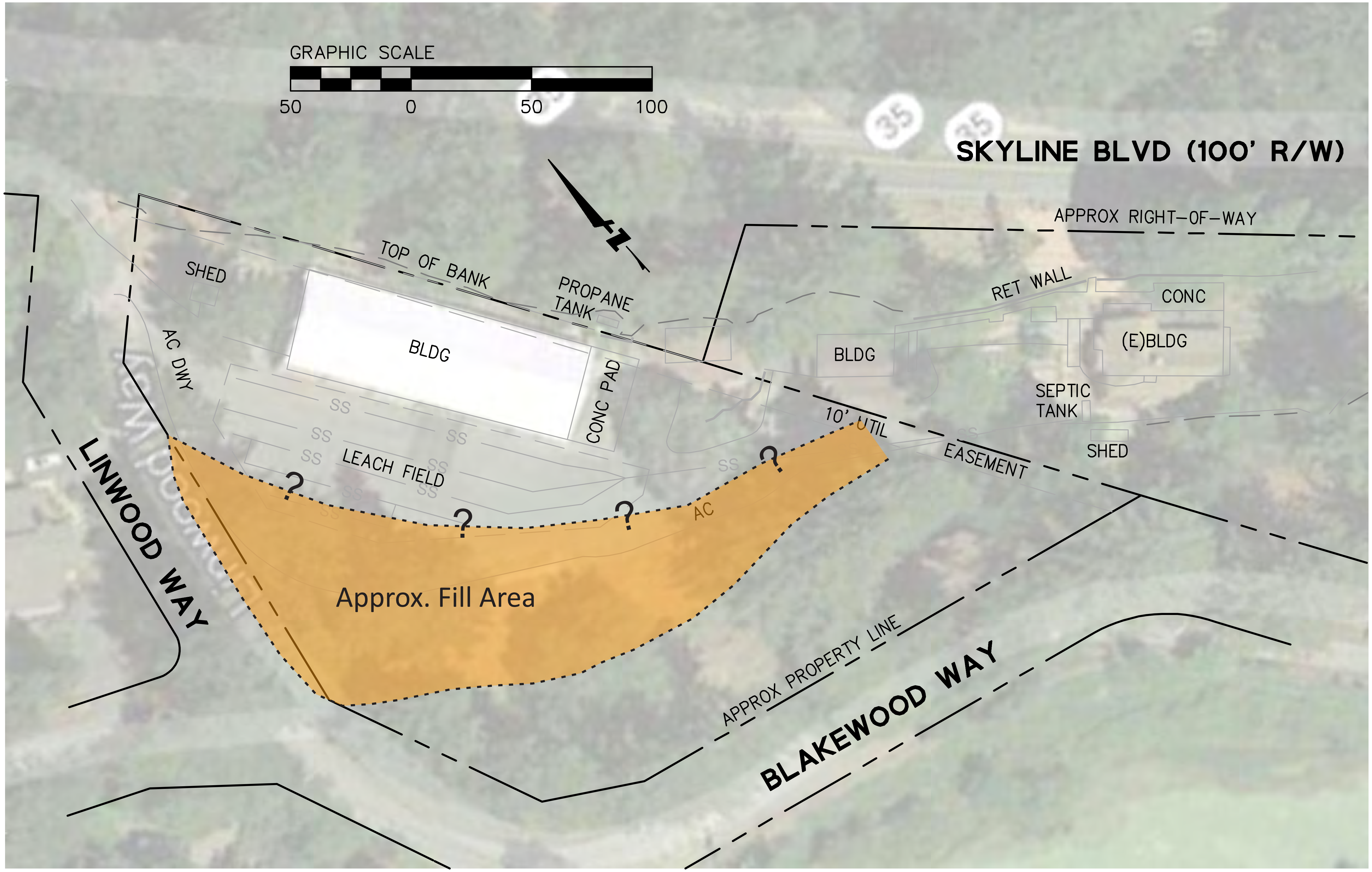
MN (13.8° E)

0 200 400 600 800 1000 1200 m

Data Zoom 13-0

SKYLONDA FIRE STATION NO. 58 17290 SKYLINE BOULEVARD WOODSIDE, CALIFORNIA	**VICINITY MAP**		
	DATE: Nov. 2013	JOB NO: MWAAR-01-00	PLATE 1

BAGG ENGINEERS



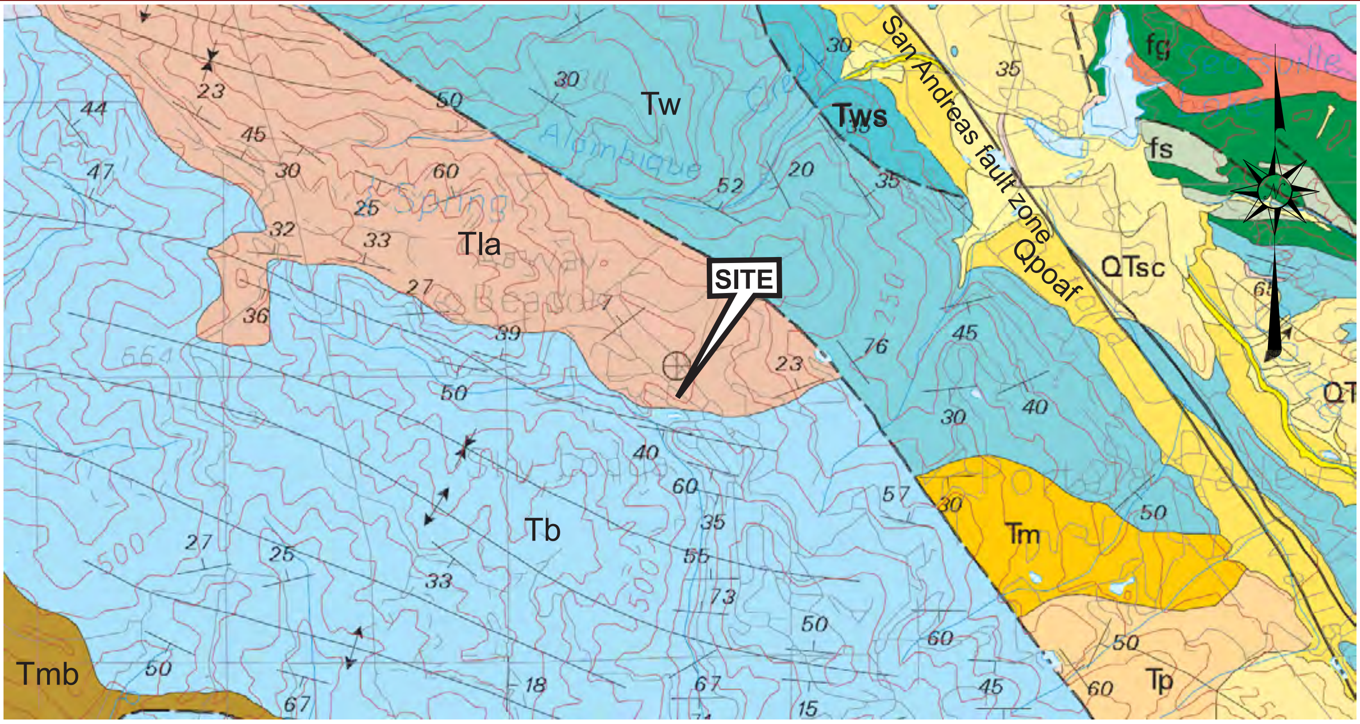
Base Map:
Existing Site Exhibit, titled
Skylonda Fire Station No. 58, Existing Site, San Mateo County, California
by BKF Engineers
dated 11/06/2013.

PRELIMINARY GEOLOGIC & GEOTECHNICAL EVALUATION
SKYLONDA FIRE STATION No. 58
17290 SKYLINE BOULEVARD
SAN MATEO COUNTY, CALIFORNIA

BAGG ENGINEERS

SITE PLAN

JOB NO:	SCALE:	DATE:	PLATE
MWAAR-01-00	shown	November 2013	2



LEGEND

Qpoaf **Older alluvial fan deposits (Pleistocene)**
Qtsc **Santa Clara Formation (lower Pleistocene and upper Pliocene)**
Tp **Purisima Formation (Pliocene and upper Miocene)**
Tm **Monterey Formation (middle Miocene)**
Tmb **Mindego Basalt and related volcanic rocks (Miocene and/or Oligocene)**
Tla **Lambert Shale (Oligocene and lower Miocene)** - Dark to pinkish-brown, moderately well cemented mudstone, siltstone, and claystone. Chert crops out in a few places in upper part of section, and sandstone bodies up to 30 m thick, glauconitic sandstone beds, and microcrystalline dolomite are present in places. Lambert Shale is generally more siliceous than San Lorenzo Formation and less siliceous than the Monterey Shale. It resembles Santa Cruz Mudstone and parts of Purisima Formation. Lambert Shale is about 1460 m thick.
Tb **Butano Sandstone (middle and lower Eocene)** - Light to buff, very fine- to very coarse-grained arkosic sandstone in thin to very thick beds interbedded with dark-gray to brown mudstone and shale. Conglomerate, containing boulders of granitic and metamorphic rocks and well-rounded cobbles and pebbles of quartzite and porphyry, is present locally in lower part of section. Amount of mudstone and shale varies from 10 to 40 percent of volume of formation. About 3000 m thick.
Tw **Whiskey Hill Formation (middle and lower Eocene)**
Tws **Shale in Whiskey Hill Formation (lower Eocene)**

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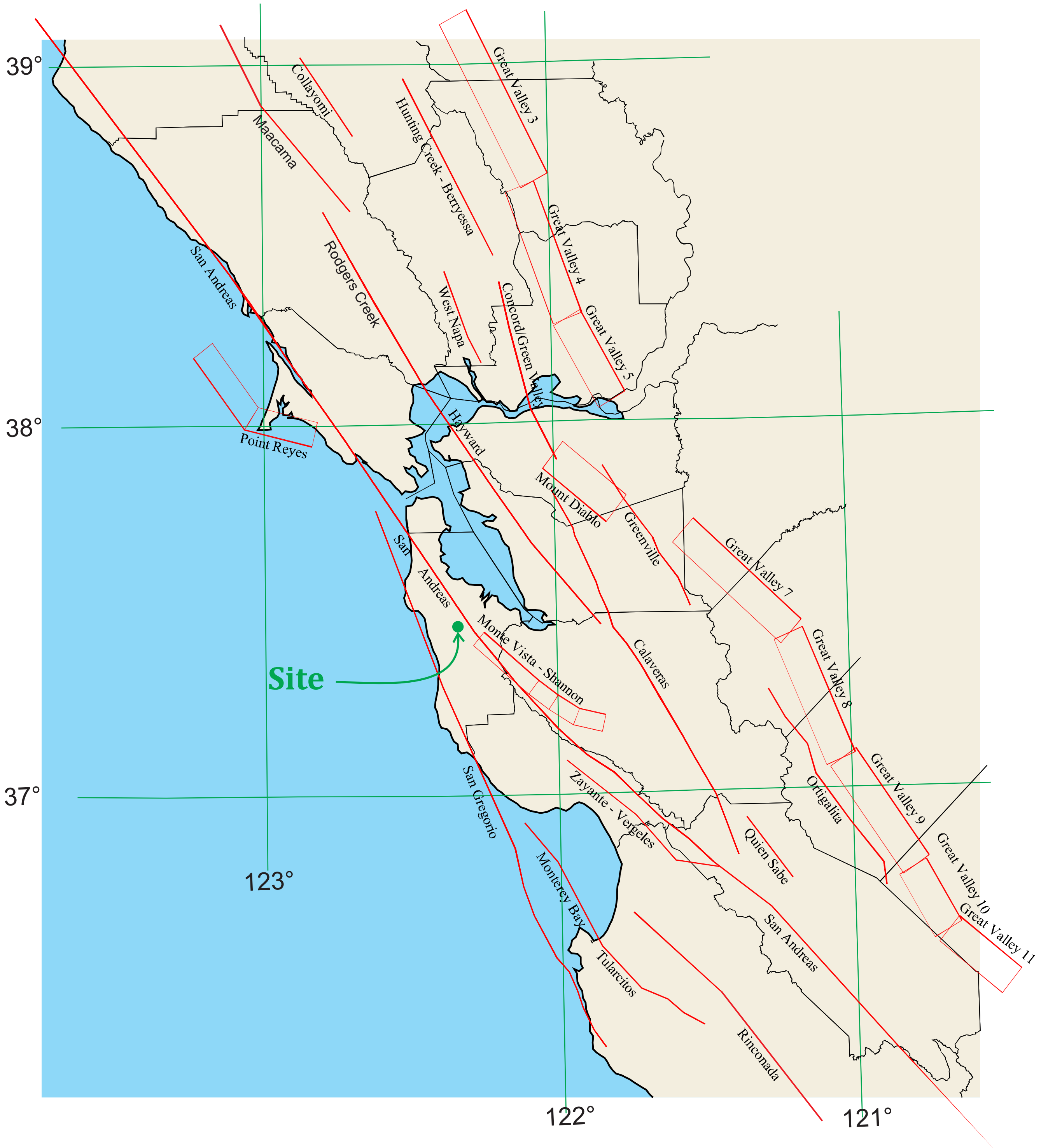
SKYLONDA FIRE STATION NO. 58
17290 SKYLINE BOULEVARD
WOODSIDE, CALIFORNIA

REGIONAL GEOLOGY MAP

DATE:	JOB NUMBER:	PLATE
Nov. 2013	WMAAR-01-00	3

BAGG ENGINEERS



Reference: Taken from the 2002 California Geological Survey Fault Model.

SKYLONDA FIRE STATION NO. 58 17290 SKYLINE BOULEVARD WOODSIDE, CALIFORNIA	REGIONAL FAULT MAP		
	DATE: Nov. 2013	JOB NUMBER: MWAAR-01-00	PLATE 4

BAGG ENGINEERS