GEOTECHNICAL STUDY

PREPARED FOR
David Montalbo

PROJECT
NEW HOME CONSTRUCTION

LOCATION
Newell in Walnut Creek
APN # 238-050-007

REPORT DATE
June 18, 2019

PREPARED BY
Bear Engineering Group, Inc.
3530 Kevin Place
Concord, CA
bearengineeringgroup@yahoo.com
ph: 925-550-7232

PROJECT NO.

45-2019-01

Mr. David Montalbo
1731 First Avenue
Walnut Creek, CA 94597

Subject: **Geotechnical Study (New Home Construction)**
Newell in Walnut Creek
APN # 238-050-007

Dear Mr. Montalbo;

We are pleased to present this Geotechnical Study for the proposed improvements located at the subject site. This report describes the services performed and presents our conclusions and recommended geotechnical design criteria for construction.

In our opinion, the site is suitable for the proposed construction, provided the recommendations in this report are integrated into the design and implemented to during construction. We reserve the right to make ancillary recommendations at any time during construction, based on circumstances that may arise during construction.

It has been a pleasure to be of service to you on this project. Should you have any questions concerning the discoveries, recommendations or conclusions of the attached report, please contact this office at your earliest convenience.

Very truly yours,

Bear Engineering Group, Inc.

REGISTERED PROFESSIONAL ENGINEER
MARK L. SCHROEDER
CE 75485
Exp. 6-30-20
CIVIL
STATE OF CALIFORNIA

Mark L. Schroeder, P.E.M.S.G.E.
Principal Engineer

TABLE OF CONTENTS

LIST OF FIGURES

INTRODUCTION

Section 1.1 - Project Description and Location

The subject site is located between Olympic Boulevard and Boulevard Way intersection as shown as ***Figure 1.***

The slope is heavily vegetated with wild grasses, low shrubs and mature trees. The topography of the project area consists of valleys and their surrounding hills and ridges. The hillside consists of nonmarine sandstone with surface feature indicating moderate stability. The site is about 2 ½ acres with 80 percent of the property considered hillside with the remaining portion of the parcel. The remaining 20 percent judged to be flat. To increase the flat portion is planned to be increased by cutting will at the toe of the slope. The two-story residence is will consists of typical wood framing.

Section 1.2 - Purpose

The purpose of this study was to evaluate the soil and geologic characteristics relevant to the improvements of the subject parcel. General foundation engineering design and geotechnical recommendations are provided based on the physical characteristics of the subsurface materials and the geotechnical limitations created by the site's surface features.

Section 1.3 - Scope

The scope of our services for the proposed planned improvement construction, as set forth in our February 25, 2019 agreement included the following tasks as listed below. This phase of the study did not include assessments for toxic substances or soil or groundwater contamination:

- Site reconnaissance to review existing site features and proposed boring locations.
- A subsurface exploration program involving two borings to a maximum depth of 25.0 feet below existing grade unless bedrock refusal was encountered.
- Soil Sampling for classification using ASTM D 2487 procedure.
- Laboratory testing of selected soil samples to evaluate in-situ moisture/density and Unconfined Compression Strength (ASTM 2166) of the subsoil.
- Reviewing of proposed residence layout to provided value engineering
- Review of United States Geological Survey (USGS) Earthquake Hazards Program (2007), to select nearest fault source that could potentially impact the site.
- Provide the near-surface Hazard Response Spectra and Design parameter seismic design criteria and per the International Residential Code (IRC)
- Engineering analyses to develop geotechnical recommendations for design and construction of the project.
- Preparation of this engineering report.

SECTION 2.0 SITE SETTING

Section 2.1- Regional Geology

The subject site is situated within the eastern portion of the San Francisco Bay Area, in the Coast Range Geologic Province of California. The East Bay Hills lie within the region of coastal California referred to by geologists as the Coast Ranges geomorphic province which extends from Southern Oregon to Southern California. The Coast Range landscape is characterized throughout its length by a series of rugged, subparallel, northwest-trending mountain ranges, most of which are structurally influenced by the San Andreas rift zone stimulating mountain ranges and intervening valleys. In general, the area is underlain by Tertiary marine and non-marine sedimentary rocks developed approximately 2 million to 62 million years ago.

Section 2.2- Local Geologic Setting

The project site lies northwest of the San Ramon Valley, which is a basin surrounded by the East Bay Hills that were formed from younger rocks uplifted between the Hayward and Calaveras fault zones. This subject site is underlain by is underlain by Miocene Series formations. Dibblee (1980) describes the unit below site as being on the cusp of alluvium surficial deposits and nonmarine fossiliferous sandstone from the Miocene period. R.W. Graymer indicates the site is underlain by the Briones Formation as provided in ***Figure 2*** from the same period.

Section 2.3 - Seismic Setting

The subject property, like all properties in the San Francisco Bay Area, is situated in a very seismically active region. Major fault zones, are fractures in the upper crust, formed when very large blocks of the Earth's lithosphere slide along, over, or under other blocks (the lithosphere is the rigid outer part of the Earth composed of the crust and the upper mantle). The collective motion of the blocks generate shearing forces in the mantle that when released result in rupture producing fault displacements. The sudden release of elastic strain energy that accompanies fault rupture is what causes the ground to shake. Faults are not consider as a single crack in the Earth's crust but actually reflect a very complex systems composed of many faults not all of which are moving at the same time or in the same way.

Table 1 provides estimated magnitude earthquakes from known active quaternary faults in the Bay Are with descriptions of the faults provided in subsequent paragraphs. ***Figure 3*** illustrates the fault systems relative to the subject site.

The probability is considered to be moderate to high for a major earthquake to occur in the Bay Area within the economic lifetime of the proposed structure. Several active and potentially active faults occur in the region. Geologic references indicate that no fault trace designated active or potentially active passes through the subject property (Graymer, Jones and Brabb, 1994). Table 1 below illustrates the fault systems capable of producing ground shaking at the subject site and there distances to the subject site.
The long-term occurrence of earthquakes modeling was founded on geologic and geophysical observations and constrained by plate tectonics. The Working Group on California Earthquake Probabilities has conducted regional modeling for the known Bay Area Fault Systems. Using this model, the Working Group has developed a map with the probability estimates over a 30-year time period. The map is presented as ***Figure 4***.

> *The Alquist-Priolo Earthquake Fault Zoning Act was passed by the California Legislature in 1972 to mitigate the hazard of surface faulting to structures. Its intent is to increase safety and minimize the loss of life during and immediately following earthquakes by facilitating seismic retrofits to strengthen buildings against ground shaking. The Act addresses only surface fault rupture; it is not*

directed toward other earthquake hazards. No faults have been mapped crossing the site, and the site is not within an Alquist-Priolo Special Studies Zone (California Geological Survey [CGS], 2007).

TABLE 1
BAY AREA ACTIVE FAULTS

Faults	Magnitude ELLSWORTH	Distance from Site (miles)	Fault Classification
Calaveras	7.3	6.5 SE	Active
Concord-Green Valley Connected	6.5	5.29 NE	Active
Hayward	7.3	9.2 W	Active
Greenville Connected	6.5	12.3 SE	Active
Mount Diablo Thrust	6.7	6.4 SE	Active
San Andres	8.0	29.9 W	Active
Rodgers Creek	6.7	27.2 NW	Active

- An "active" fault is defined by the State of California as a fault that has had surface displacement within Holocene time (approximately the last 10,000 years). A "potentially active" fault has shown evidence of displacement during Quaternary time (approximately the last 2 million years). The fault classifications are derived from the Fault Activity Map of California and Adjacent Areas (Jennings, 1994).

- Moment magnitude (Mw) is related to the physical size of a fault rupture and movement across a fault. Moment magnitude provides a physically meaningful measure of the size of a faulting event (CDFG, 1997). The Maximum Moment Magnitude Earthquake, derived from the joint CDMG/USGS Probabilistic Seismic Hazard Assessment for the State of California (USGS, 1996).

Concord-Green Valley: The Concord Fault is a Holocene active dextral strike-slip fault characterized by aseismic creep (rate 3.0 mm/yr. to 3.5 mm/yr.; Galehouse, 2000). Three sections area associated with this fault. Section 1 traverses the town of Concord and borders the western side of Lime Ridge. The northern end of the fault is assumed to connect with the Green Valley fault. The southern extent is relatively unknown but is thought to be to be connected to Mt. Diablo Thrust Dibblee (1980, c). Extending from Lime Ridge to the southern extent of the fault, the Concord Fault is delineated by a southwest-facing escarpment along the west side of Lime Ridge. Schwartz, 2008, suggests the activity on the fault to be during the Holocene age. The 2003 Working Group for California Earthquake Probability assigned a 4% probability that the Concord-Green Valley Fault system would produce a magnitude 6.5 or greater earthquake in the next 30 years.

Calaveras Fault: Historically active major dextral strike-slip fault that is part of the larger San Andreas Fault system. The fault zone extends for about 90 miles from the San Ramon area southeast to about 19 miles south of Hollister. The fault is divided into 4 sections from north to south they are the Northern Calaveras, Central Calaveras, Southern Calaveras, and Paicines sections. North of Calaveras has a slip rate of 5-6 mm/yr. (Kelson and others, 1996).

Greenville Clayton Section: Historically active dextral strike-slip faults located in the Diablo Range. The fault zone extends from northwest of Livermore Valley along the Marsh Creek and Clayton faults towards Clayton Valley. Wright and others (1982) reported that fault-related topographic features are poorly developed and differ significantly from the Marsh Creek-Greenville segment. Colburn (1961) reported that the Clayton section is generally characterized by subdued saddles and subdued hill fronts. Unruh and Sawyer (1995, 1998) suggested that slip from the Greenville fault is transferred to the Concord fault along the Mt. Diablo fold and thrust belt and that only minimal slip continues to the Clayton fault.

Hayward Fault: This fault is located in the eastern San Francisco Bay region and generally trends along and bounds the western side of the East Bay Hills (Aydin, 1982). The fault zone has three sections (Working Group on Northern California Earthquake Probabilities, 1996. The segment boundary between the Northern and Southern Hayward faults was long considered to be delineated by the location of the northern boundary of rupture associated with the Mw~7 1868 earthquake and the southern boundary of rupture associated with the 1836 (Working Group on California Earthquake Probabilities, 1988). The Hayward fault is characterized by fault creep along the Northern and Southern sections. A preferred average creep rate of 4.6 mm/yr. was reported by Lienkaemper and Galehouse (1997).

Mt. Diablo Thrust Fault: The Mount Diablo Thrust Fault is approximately 15 miles long, and dips at an angle of 38 degrees to the northeast. The Mount Diablo Thrust Fault is capable of generating an earthquake of magnitude MW=6.7. The predicted rupture surface begins 5 miles below the surface, and there is thus no surface expression of the fault, and a low likelihood of surface rupture in the event of a large earthquake on the fault. No large historic earthquakes are known to have occurred on the Mount Diablo Thrust Fault. The recurrence interval for large earthquakes along the fault is predicted to be about 400 years.

The peak of Mt. Diablo is the topographic culmination of the northwest-trending Mt. Diablo anticline, a southwest-vergent fold located in a restraining step between the dextral Greenville and Concord faults. Unruh and Sawyer (1997) proposed that Mt. Diablo anticline is a fault-propagation fold developed above a blind, northeast-dipping thrust fault. Based on variations in the geometry of the fold along trend, it is possible that the Mt. Diablo thrust fault is divided into at least two structural segments that are offset in a right-stepping sense. The two segments are informally referred to herein as the “northwest segment” and “southeast segment”. The structural boundary between the two segments is interpreted to be near the town of Alamo, and is spatially associated with a northeast-trending alignment of earthquakes informally called the “Alamo swarm” (Oppenheimer and Macgregor-Scott, 1992).

San Andreas Fault: San Andreas Fault zone is the principal element of the San Andreas Fault system, a network of faults with predominantly dextral strike-slip displacement that collectively accommodates the majority of relative N-S motion between the North American and Pacific plates. The San Andreas Fault zone is considered to be the Holocene and historically active dextral strike-slip fault that extends along most of coastal California. The fault zone first gained international scientific attention immediately following the great 1906 San Francisco earthquake.

Lafayette-Reliez Valley Faults: The northern Calaveras fault is transferred to the interior of the northern East Bay hills by a complex system of poorly integrated strike-slip faults and shear zones that are connected by restraining step-overs. At the northern end of the Calaveras fault, the majority of dextral slip steps west across the northeast-vergent Las Trampas anticline onto the dextral Reliez Valley and Lafayette faults. Slip is transferred onto these structures from the Lafayette-Reliez Valley faults through a series of short restraining step-overs in the Briones hills region. Associated crustal shortening is responsible for creating the locally high topography of the Briones hills. The Briones lineament is associated with the "Briones swarm", a cluster of small earthquakes that form a NNW-trending alignment, and which exhibit dextral slip on NNW-striking nodal planes. Lafayette has a very high earthquake risk, with a total of 3,144 earthquakes since 1931. The USGS database shows that there is a 98.87% chance of a major earthquake within 50km of Lafayette within the next 50 years. The largest earthquake within 30 miles of Lafayette, CA was a 6.0 Magnitude in 2017. The Reliez, Southampton and Franklin Faults are for the most part poorly characterized strike-slip faults but may contribute to the approximately 4 to 7 mm/yr. of distributed dextral slip between the northern Calaveras and Concord faults.

Franklin Fault: Is believed to be a part of the West Napa fault zone which produced a 6.2 magnitude earthquake in August 2014. This would suggest the Franklin fault (FF) in combination with the (WNFZ) the fault system is at least 75 km. Previously published potential-field data indicate that the WNFZ extends northward to the Maacama fault (MF), and previous geologic mapping indicates that the FF extends southward to the Calaveras fault (CF); which would increase the fault zones length by 110 km.

Rodgers Creek Fault: The northern continuation of the Hayward fault, the Rodgers Creek fault extends from San Pablo Bay through Santa Rosa. Like the Hayward fault, the Rodgers Creek is one of the most active in the Bay Area. Estimated slip rates are 9 mm/yr. Hayward-Rodgers Creek fault zone is ~190 km, extending from Alum Rock in the south to just north of Healdsburg. The ability of an earthquake on the Hayward fault to continue onto its northern extension along the Rodgers Creek fault (or vice versa) greatly depends on the geometrical relationship between these faults beneath San Pablo Bay.

In general, the rupture displacement in an earthquake is typically about 1/20,000 of the rupture length. The rupture velocity is about 3 km/s, so the rupture duration in seconds is given by fault length in kilometers divided by 3.

Section 2.4 Surface Fault Rupture

Based on the information provided in Hart and Bryant (1997), the site is not within an Alquist-Priolo Earthquake Fault Zone. Based on our review of the Graymer et. al. geologic maps for the area, no known active, or potentially active faults cross or project toward the site. Additionally, no evidence of active faulting was visible on the site during our site reconnaissance, therefore, it is our opinion that the potential for fault-related surface rupture at the site is low.

SECTION 3.0 - SITE EXPLORATION AND LABORATORY TESTING

Section 3.1 - Field Exploration

Field exploration of the site was conducted on June 6, 2019, consisting of two (2) exploratory boreholes to a maximum depth of approximately 17-feet below existing grade. The Boring locations are presented as ***Figure 8*** Borings were drilled using a truck-mounted drilling unit with a 4-inch solid stem auger. Samples were obtained by driving a 2-inch Modified California Sampler at 18-inch intervals into underlying soil using a 140-pound hammer free falling 30-inches. The number of blows required to drive the sampler was recorded in 6-inch penetration intervals. The last 12 inches of penetration is provided on the Log of Borings as penetration resistance per foot. Blow counts provided have been corrected for energy efficiency. The boring was backfilled with Portland cement by the tremmie method. Description and identification of the samples were conducted in the field using ASTM D2488 and D2487 methods provided as the Log of Borings ***Figure 9 and 10***.

Section 3.2 - Laboratory Testing

Laboratory testing was conducted on selected soil samples to obtain data on density, moisture content (ASTM D2167), and soil description and identification (ASTM 2488). Laboratory test results are presented on the Log of Test Borings.

SECTION 4.0 - SURFACE AND SUBSURFACE CONDITIONS

Section 4.1 Subsurface Conditions

Boring 1*: Encountered yellow brown very hard dense sandstone from the surface to 3 feet at which point the drilling rig was unable to penetrate.*

Boring 2: *Was founded in the flat area about 10 feet north of toe of the existing hillside. Surface soil from 2 to 8 feet was classified as yellow to medium brown, moderately dense sandy loam Alluvium deposits. Drilling became more difficult at about 12 feet with sampling at 15 feet indicating yellow brown moderately dense slightly cemented sandstone. This material continued to the refusal depth of 17 feet.*

Section 4.2 - Groundwater

No groundwater was found in either boring. However, groundwater conditions can deviate from those conditions encountered at the boring locations. Should this be revealed during construction, Bear Engineering Group should be notified immediately for possible revisions to the recommendations that follow.

Section 4.3 - Liquefaction

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility enough to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean (relatively free of clay), relatively young and fine-grained sand deposits. The condition therefore requires the coexistence of susceptible soils, strong ground motions, and shallow groundwater. Within alluvial deposits, sand units conducive to liquefaction may occur as broad layers; confined lens shaped bodies or elongated sinuous channel deposits. The thickness of such units can vary significantly as well. In areas where strong seismic shaking has occurred, clean and loose gravel deposits have been known to liquefy as well. The primary factors affecting liquefaction potential of a soil deposit are:

1. Level and duration of seismic ground motions
2. Soil type and consistency
3. Liquefaction of sediment requires that the sediment be water saturated

All sections of the San Francisco Bay region have the potential to be shaken hard enough for sediment to liquefy. Geologic and liquefaction susceptibility mapping (***Figure 5***) indicates the site has a low probability to liquefy. Field exploration and laboratory results confirm this finding.

Section 4.4 - Settlement

All foundations settle to some extent as the earth materials around and beneath them adjust to loads of the building. Where foundation settlement occurs at roughly the same rate throughout all portions of a building, it is termed uniform settlement. Settlement that occurs at differing rates between different portions of a building is termed differential settlement.

The discontinuity of the soil behind the rock wall and native soil lends its self to differential movement as the fill thickness of the level pad is assumed to be reduced where the pad meets the existing slope. Satellite images suggest the undocumented fill behind the wall was placed after 1993. Considering there is no conclusive evidence of densification of the pad materials it is difficult to predict potential settlement. The slopes in the area are prone to movement which could affect vertical displacement.

Section 4.5 – Landslide Evaluation

Two sets of forces compete to shape the hillsides in steep terrain: the load imposed by gravity, which tends to pull the hillside materials downslope, versus the resistance of these materials to moving. Gravitational loading, acting in the downslope direction, is proportional to the weight of the soil. The resistance of slope materials to sliding or other deformation is expressed by shear strength. For most hillslope materials, shear strength is derived largely from the frictional forces between the grains of soil or rock acting normal (perpendicular) to the slope.

A landslide is a downslope movement of rock or soil, or both, occurring on the surface of rupture—either curved (rotational slide) or planar (translational slide) rupture—in which much of the material often moves as a coherent or semi-coherent mass with little internal deformation. Varying classifications of landslides are associated with specific mechanics of slope failure and the properties and characteristics of failure types. Types of movement are listed below.

1. **ROCKFALL**: Falls are abrupt, downward movements of rock or earth, or both, that detach from steep slopes or cliffs. Triggering mechanism is typically undercutting of slope by natural processes.

2. **TOPPLE**: A topple is recognized as the forward rotation out of a slope of a mass of soil or rock around a point or axis below the center of gravity of the displaced mass. Triggering mechanism sometimes driven by gravity exerted by material located upslope from the displaced mass and sometimes by water or ice occurring in cracks within the mass; also, vibration, undercutting, differential weathering, excavation, or stream erosion.

3. **ROTATIONAL LANDSLIDE**: A landslide on which the surface of rupture is curved upward (spoon-shaped) and the slide movement is more or less rotational about an axis that is parallel to the contour of the slope. Triggering mechanism, Intense and/or sustained rainfall.

4. **TRANSLATIONAL LANDSLIDE**: The mass in a translational landslide moves out, or down and outward, along a relatively planar surface with little rotational movement or backward tilting. Triggering mechanism, primarily intense rainfall, rise in ground water within the slide due to rainfall.

5. **LATERAL SPREADS**: Lateral spreads usually occur on very gentle slopes or essentially flat terrain, especially where a stronger upper layer of rock or soil undergoes extension and moves above an underlying softer, weaker layer.

6. **FLOWS:** A spatially continuous movement in which the surfaces of shear are short-lived, closely spaced, and usually not preserved.

Landslide Susceptibility Maps ***Figure 6*** characterizes the slopes south of the planned residence as rugged and very steep terrain generally greater the 26 degrees placing the site in category 4 suggesting landslides are probable. This map is based on the presumption that steeper slopes are more probable to movement. The landslide hazard map showing definite landslide or possible impending movement does not identify mapped slide in the general area of the subject site as shown as ***Figure 7***.

Section 4.6 - Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may cause unacceptable settlement or heave of surface structures, concrete slabs supported-on-grade, or pavements.

Section 4.7 - Findings

Based on published data and confirmed by exploratory field results well cemented sandstone is located near the surface above the planned residence on the slope and at about 12-feet in the flat area near the toe of slope. The general area of the flat zone is considered to be alluvium soil based on geologic maps suggesting bedrock may be slightly deeper than 12 feet moving away from the slope. Surface soil did not exhibit significant expansion and contraction capabilities although some minor vertical movement could be expected. Dynamic resistance, lack of groundwater and laboratory data indicates the site is not likely to liquefy although this is predicated on the magnitude, proximity and increase in ground water levels in the winter. The Lafayette-Reliez Valley Faults to the north are not estimated to deliver significant shaking at the site as the fault length is considered short. The Calaveras and Concord Faults are the two fault systems capable of generating significant ground movement at the site. The Working group of California suggests there is a 63 chance of a 6.7 magnitude earthquake occurring from one of the fault systems listed in Table 1 by 2032. The site is not considered to be not be within an Alquist-Priolo Special Studies Zone and no faults have been mapped crossing the site, suggesting ground rupture conditions to be low. The slope above the residence shows indications of shallow slumps, colluvium soil moving over the shallow bedrock. Slope instability maps classify the slopes as being prone to movement although published Landslide Hazard Maps did not log landslide activity in the general vicinity of the subject site. Based on field observations and exploratory results the slope behind the house does have the ability to flow as surface run-off is typically necked to low surface features which accumulate significant amounts of water and slope inclination will generate moderate to high surface water velocities. Soil in the flat are is considered to be homogeneous but depending on water transport placement the thickness of the material may vary moving away from the toe of slope.

SECTION 5.0 – CONCLUSION

It is our opinion, based on an analysis of the data and information obtained from the site exploration, laboratory testing, and geotechnical evaluation, published data and our experience and knowledge of the soil conditions in the area, the site is geotechnically suitable for the proposed development provided the recommendations contained herein are incorporated into the project designs and adhered to during construction.

The principle adverse geotechnical factors affecting the development of the site are the following;

1. Slope inclinations producing moderately high surface run-off water velocities resulting in erosion and potential earthflow conditions.

2. Differential movement is possible affecting surface structures.

3. Strong ground shaking from a seismic event. The close proximity to the Calaveras or Concord Fault has the potential to induce intense ground shaking as well as the other fault systems described. This may induce seismically related differential movement.

4. Moderate to low potential for expansion and contraction of near surface soil is anticipated.

5. Liquefaction is unlikely based on Filed and laboratory results.

6. The site is not within any current Alquist-Priolo Earthquake Fault Zone. There are no known faults or shear zones extended through the site that would endanger the development due to ground rupture.

SECTION 6.0 – RECOMMENDATIONS

Section 6.1 – Geotechnical Hazards

Risk of geotechnical hazards will always exist due to uncertainties of geologic conditions and the unpredictability of seismic activity in the Bay Area. However, in our opinion, based on available data, there are no indications of geotechnical hazards that would preclude use of the site for the proposed development. The proposed structures should be designed to meet the current International Residential Code (IRC) requirements to limit potential damage from ground shaking and ground failure.

Section 6.2 - Seismic Criteria

The 2012/2015 International Building Code was reviewed to provide the seismic criteria for structural design of the foundation and building. Site Coordinates Latitude, Longitude: 37.8852135, -122.0891651 Risk Category II, Site Soil Classification Site Class D – "- Stiff Soil"

TABLE 2
SIESMIC DESIGN CRITIERA

Site Class	**D**
Short Period Spectral Acceleration, Ss'	**1.771 g**
1 sec Period Spectral Acceleration, S1	**0.6 g**
Site Coefficient Fa	**1.00**
Site Coefficient Fu	**1.50**
Max Short Period Spectral Response Accelerations SMs **SMs=Fa x Ss**	**1.771 g**
Max 1 sec Period Spectral Response Accelerations SM1 **SM1= SM1 = Fv x S1**	**0.9 g**
Dampened Design Spectral Response-Short Period **SDs=2/3 x SMs**	**1.18 g**
Dampened Design Spectral Response-1 sec Period **SD1=2/3 x SM1**	**0.6 g**

- **MCEG peak ground acceleration PGA 0.665g**

Severe ground shaking can involve forces that damage structures not designed to withstand them. The estimated peak ground acceleration (10 percent probability of being exceeded in 50 years) in the project area is moderate for California (and high from a national perspective)—in the range of 35–65 percent of the acceleration of gravity (g) (Petersen et al. 1999). Project elements will be designed to withstand such forces.

Section 6.3 - Grading

Final grading plans were not available during preparation of this report. We have provided two grading recommendations provided in the subsequent paragraphs. We expect grading to be limited to preparing the building pad and cutting into the slope to align the planned residence so as to limit tree removal position the residence to blend into the natural topography as much as possible. The following steps will be necessary to accomplish these objectives.

Clearing, Stripping, Grubbing, and Debris Removal

Trees, roots, vegetation, and organic surficial soil shall be removed from structural areas unless specified otherwise by the Geotechnical Engineer or the Engineer's Representative. The depth of organic soil to be removed will be recommended by the Geotechnical Engineer or the Engineer's Representative but, in general, will probably vary from about 4 to 6 inches.

Strippings are defined as surface vegetation and organic surficial soil. Strippings may not be used in fill unless specifically authorized and observed by the Geotechnical Engineer or the Engineer's Representative. Stripping material may be stockpiled for landscaping use, with the approval of the landscape architect. The final clearing, stripping, and grubbing shall be approved by the Geotechnical Engineer before further grading is started.

Concrete pavement, building rubble, concrete foundations, and any other debris noted at or below the existing ground surface should be removed as part of the site preparation for the proposed construction area.

Erosion Control:

The extent and duration of ground disturbing activities during and immediately following periods of rain shall be limited, to avoid the potential for erosion which may be accelerated by rainfall on exposed slopes. Erosion and sediment control plans shall be designed by the Civil Engineer.

To reduce the potential for erosion, all permanent cut-and-fill slopes on-site should be seeded or planted with lightweight, deep-rooting, drought-resistant vegetation. A landscaping expert should be consulted for ground cover recommendations. Excessive landscape irrigation or leakage from irrigation lines can cause localized slope failures. Therefore, irrigation systems for slope vegetation should be designed and maintained to minimize leakage onto graded slopes. Vegetation on natural slopes should remain natural and not be landscaped or irrigated in the same manner as graded slopes.

To minimize erosion during construction the slope shall be protected with erosion control blankets. The mats are designed to increase soil stabilization, decrease the effects of erosion, and allow vegetation to effectively take root. They may be rolled up during construction. Erosion Control specifications are provide in Table 3.

TABLE 3
COIR MAT TECHNICAL SPECIFICATIONS SEMI-PERMANENT CONTROL (4 - 6 YEARS)

Product	Open Area	Weight	Sizes	Uses
Coir Mat 70	50%	20.6 oz./yds^2 700 g/m^3	13.1 ft. x 83 ft. 13.1 ft. x 165 ft.	Slopes: 2:1 or 1:1 Flows: up to 12 fps

The semi-permanent coir mats typically provide erosion control for approximately 4 to 6 years, depending on your area conditions. Coir mats are made with open weaves to allow for reseeding and vegetation both before and after installation. Offering a higher strength design, erosion control mats can accommodate areas with steep slopes and increased water flow.

Building Pad Preparation

We applicable crawl space soil shall be compacted to a depth of 18-inches with materials being re-established to develop a uniform graded area. Soil shall be compacted to a minimum relative compaction of 95 percent at 3 percent over optimal laboratory moisture levels in accordance with ASTM D 1557. Utility trenches outside the limits of the foundation may be compacted to 90 percent relative compaction with the same moisture contents as stated above.

Section 6.4 - Foundation

We understand that the proposed improvements will be of typical wood-framed construction. Anticipated foundations loads are expected to be relatively light. We recommend that the new construction be placed upon a pier and grade beam support system. If another system is desired, this office should be called for supplemental recommendations. Such recommendations would be presented as an addendum to this report. The following foundation recommendations are based on the anticipated soil conditions underlying the project site and building pad preparations as described above.

TABLE 4
PIER AND GRADE-BEAM FOUNDATION DESIGN CRITERIA

Diameter	Minimum 16 in.
Spacing	Minimum 4 pier diameters, center to center. Maximum spacing to be decided by the Project Structural Engineer.
Embedment*	Minimum of 5 feet into acceptable material as determined by our geotechnical engineer or his representative during drilling. We estimate pier depths to vary from 12 to 15-feet depending on location.
Friction Value	Allowable friction value of 500 psf, which may be increased by 1/3 for wind and seismic loads
Passive Value	300 pcf to be taken over 1-1/2 times the pier diameter commencing 3 feet below lowest adjacent grade.

- ***Depth of pier embedment is measured from the bottom of the grade beam and may be changed based on field conditions observed. We should be present during drilling to render any pier depth alteration as necessary.***
- ***Neglect the upper 2-feet in the design calculations for piers.***
- ***Any wall that is incorporated into the foundation of a building or restrained at the top should be designed with a 60 psf uniform lateral surcharge load in addition to the lateral earth pressures given above.***
- ***All basement walls shall be protected with impervious barrier We recommend Mirafi G100N. Drainage panels shall be installed in accordance with manufacture specifications. All walls shall have Caltrans Class 2 filter rock minimum 12 inches thick with minimum 4-inch diameter pipe located at base of wall discharging to suitable location that will not impact future performance of the foundation.***

Estimations of overall settlement are considered to be 1-inch from a seismic event with potential differential settlement of 1/2 inch over a 20 feet distance.

We recommend that all pier holes be cleaned of slough and loose material prior to placing steel reinforcement and concrete. Failure to clean the pier holes adequately may result in significant adverse differential settlement of the foundation or hydro-swell of the pier. We recommend that all piers use a tremmied system for concrete placement in order to assure maximum friction is obtained and to expel any subsurface water encountered during pier drilling. The tremmie hose must maintain a constant head during concrete placement. If groundwater is encountered pier holes shall be pumped free of water and re-drilled as needed to clear loose soil from the base of the pier.

Grade beams shall extend a minimum of 18-inches below lowest adjacent grade to provide greater resistance to water infiltration into the crawl space from exterior storm or irrigation water. The piers and grade beams shall contain steel reinforcement over their entire length with reinforcement as directed by the Project Structural Engineer in accordance with applicable UBC or American Concrete Institute standards. In no case, however, should the grade beams contain less than two No. 5 (grade 60) reinforcing bar (or equivalent steel area) in both the top and bottom of each beam, and the piers should have no less than four No. 5 (grade 60) reinforcing bars. The reinforcement steel for the pier should be tied into the top steel grade beam for continuity and integrity.

Section 6.5 - Concrete Slabs-on-Grade, Floors

We recommend that the minimum slab-on-grade floor structurally independent of the foundation system with a minimum thickness of 6 inches. We recommend minimum reinforcement of No. 4 reinforcing bars spaced at 18 inches on center, or with an alternate reinforcement system as required by the project structural engineer. In general, the reinforcement should be supported by concrete Dobies to attain its greatest efficiency in minimizing the cracking of the slabs. Crack control joints should be located as directed by the structural engineer.

Concrete slab-on-grade floors should be underlain by a minimum 4-inch-thick layer of Caltrans Class 2 Base rock fill compacted to a minimum of 90 percent relative compaction. If potential moisture vapor transmission through the slab is objectionable, we recommend that an impermeable membrane of STEGO WRAP VAPOR BARRIER (Min 15-MIL) thickness be placed above the base rock overlain by 2 inches of clean sand to assist in proper curing of the slab unless object able by the structural engineer. The membrane should be placed in accordance with the manufacturer's specifications. Any punctures or damage to the membrane that may occur must be repaired in accordance with the manufacturer's specifications. Some moisture transmission should be expected where a membrane vapor barrier is not utilized.

Recommendations presented in the American Concrete Institute should be complied with for all concrete placement and curing operations. Improper curing techniques and/or excessive slump (water-cement ratio) could cause excessive shrinkage, cracking, or curling. Adding more water may make concrete more workable but it also means the drying time can increase to unreasonable levels as the rate of evaporation is dependent on a number of variables.

Section 6.6 - Miscellaneous Flatwork

All exterior concrete flat work shall be structurally independent of the foundation to provide freedom of movement to allow for soil volume changes. All walkways shall be a minimum thickness of four inches and be underlain by a 4-inch thick cushion of "sand or crushed rock". Reinforcement of the walkways shall consist of a minimum No. 3 reinforcement bars placed in a grid pattern at 16 inches on center. Subsoil material shall be moisture conditioned and compacted to a relative compaction of 90 percent at 3-5 percent over optimum moisture values. Ponding of storm or irrigation water adjacent to any structure is prohibited. Walkways shall be designed to slope to area drains or a minimum grade of 2 percent away from structures discharging to a suitable controlled location.

The owners must be advised that some vertical displacement of exterior flatwork should be anticipated. Proper site drainage, maintenance and controlling landscape irrigation is recommended to reduce the amount of vertical displacement that may occur.

Section 6.7 - Retaining Walls

All retaining walls shall be designed for fully-drained conditions. The proposed design should be reviewed by our firm to confirm that the retaining wall configuration is compatible with the assumed parameters. The Table 5 below presents our design criteria recommendations for any retaining wall and applies to walls up to 6 feet in height. Design pressures are expressed as equivalent fluid pressures. Walls greater than 6-feet will be considered on a case by case basis. The conditions provided below are for typical cast-in-place concrete walls. All walls shall rest upon piers in accordance with Section 6.4. It may be necessary to install a diversionary wall on the slope above the residence in the event a earthflow condition develops. This wall is estimated to be 3-5 feet tall with a minimum of 2-feet of freeboard. The grades behind the wall shall be designed to deflect the viscous flow of fine-grained materials away and to an appropriate location which will enable clean-up. The diversionary wall shall be fully drained and founded on piers.

TABLE 5
RETAINING WALL DESIGN CRITERIA

RETAINING WALL DESIGN CRITERIA		
Gradient of Backfill	Equivalent Fluid Weight (pcf)	Friction* Factor
Level	40	.35
3:1 to Level	50	.35
Steeper than 3:1 (Maximum 2:1)	65	.35

- *Where retaining walls form the base of the structure, deflection calculations or allowance for wall movement should be included in the final planning for the structure. Dynamic uniform Loading .55 H^2 reversed tri-angular distribution.*
- *The values provided above are based on implementing Sec 6.3*

All retaining walls should be free draining. We recommend installing a minimum 4-inch diameter perforated SDR 35 pipe upon 2-inch of Caltrans Class 2 permeable filter rock at the base of the wall. The trench and pipe should be sloped a minimum of 1 percent grade discharging to a suitable outlet location as directed by the Civil Engineer. A minimum of one cleanout shall be placed at the end of any wall. The cleanout shall be installed with a minimum sweeping 45 degree bend. A 12-inch wide minimum section of Cal Trans permeable filter rock should be installed behind the wall extending a minimum of 1 foot below top of wall capped with compacted on-site material to finish surface. Mirror drain shall be used behind structural walls.

An earthen swale or concrete v-ditch is recommended for the diversionary wall.

Section 6.8 - Utility Trenches

All trenches should be backfilled with native materials compacted uniformly in accordance with Section 6.3 of this report. If local building codes require the usage of sand as the trench backfill, all utility trenches entering the building must be plagued with CDF (120 psi minimum (Sand Slurry).

Jetting of trench backfill is not recommended as it may result in an unsatisfactory degree of compaction. All disturbed areas within 5 feet of the foundation from trench excavation, including electric lines, must be reprocessed as engineered fill.

Section 6.9 - Drainage

A five percent gradient should be maintained for landscaped areas immediately adjacent to the structure (within 5-feet). In general, water should not be allowed to collect near the surface of the foundation or floor slab areas of the structures during or after construction. Down-spout locations directed to solid tight-line connections discharging to a suitable location away from the foundation. We recommend sufficient area drain inlets be placed in flat work areas to reduce the potential ponding and the effects of expansion and contraction of the near surface soil in the event of surface water migration. If vegetation must be planted adjacent to a structure, plants that require very little moisture should be considered. Sprinkler heads should not be placed where they would saturate the foundation soil; a drip irrigation system is preferred unless drop inlets or area drains are utilized to control drainage around the foundation.

Future landscaping, construction of walkways, planters and walls, etc. must never modify site drainage unless additional measures to enhance drainage (e.g., area drains, additional grading) are designed and constructed in accordance with an addendum to this report.

Section 6.10 - Excavations

The contractor is solely responsible for protecting excavations by shoring, sloping, benching or other means as required to maintain stability of both the excavation sides and bottom. Bear Engineering Group does not assume any responsibility for construction site safety or the activities of the contractor. There shall be no vertical cut close steeper than 2.75H:1V under any circumstances.

If excavations are made during the rainy season (normally from November through April), particular care shall be taken to protect slopes against erosion. Measures to help mitigate erosion, such as the installation of berms, plastic sheeting, or other devices, may be warranted.

Section 6.11 - Plan Review

The city or county may require a Plan Review prior to approving the submitted design drawings and construction documents. Please note this is not free.

Section 6.12 - Construction Observations

Our representative must be present during grading to observe the work performed and to perform whatever testing is necessary to properly evaluate the quality of the materials and their relative compaction. Foundation observation by our representative is recommended so that the foundation excavations are excavated to the design depth for proper bearing into the underlying materials. The depth of the foundations is dependent upon site grading and other unforeseen local anomalies, and thus the actual depths may vary. Please note if the City of Berkeley inspector allows for the placement of concrete for piers without Bear Engineering Groups inspections our insurance will not cover the project and the city shall assume full responsibility for the foundations. WE MUST BE NOTIFIED DURING PIER DRILLING NO EXCEPTIONS for the purpose of logging piers.

At the completion of site grading operations and foundation excavations, we will submit a report that summarizes the work observed and the results of all tests performed by our firm during the construction phase of the project, along with any supplemental recommendations that may be warranted. To allow proper scheduling so that our personnel are present at the job site when needed, we should be furnished no less than 2 working days advance notice of when work requiring our presence will be accomplished.

Section 6.13 - Site Safety

All excavations and site work must comply with applicable local, state, and federal safety regulations. Construction site safety is the responsibility of the contractor, who shall be solely responsible for the means, methods, and sequencing of construction operations. Our services and recommendations for site safety are available upon request and are advisory only and supplemental to current regulatory standards. Bear Engineering Group, Inc. assumes no responsibility for construction site safety or the contractor's activities during any phase of the construction project.

Section 6.14 - Miscellaneous

Our exploration did not reveal the presence of buried items such as leaching fields, septic tanks, storage tanks, etc. at the location of the borings. If such items are encountered during grading or demolition, our firm should be notified immediately to provide recommendations for proper disposal procedures.

SECTION 7.0 - LIMITATIONS

This report has been prepared for the exclusive use of Mr. Montalbo and his consultants for specific application to the proposed development. If changes occur in the nature, design location, or configuration of the proposed development, the conclusions and recommendations contained here shall not be considered valid. Changes must be reviewed by our firm.

The analysis, opinions, conclusions, and recommendations submitted in this report are based in part on the referenced materials, site visit and evaluation, and subsurface exploration. The nature and extent of variation among exploratory borings may not become evident until construction. If variations appear, it will be necessary to re-evaluate or revise recommendations made in this report.

The recommendations in this report are contingent on conducting an adequate testing and monitoring program during construction of the proposed development. Unless the construction monitoring and testing program is provided by or coordinated with our firm, Bear Engineering Group will not be held responsible for compliance with design recommendations presented in this report and other supplemental reports submitted as part of this report. Our services have been provided in accordance with generally accepted geotechnical engineering practices. No warranties are made, express or implied, as to the professional opinions or advice provided. Recommendations contained in this report are valid for a period of 1 year; after 1 year they must be reviewed by this firm to determine whether or not they still apply.

SECTION 8.0 - REFERENCES

Graymer, R.W., Jones, D.L., and Brabb, E.E., 1994, Preliminary geologic map emphasizing bedrock formations in Contra Costa County, California: A digital database: U.S. Geological Survey Open-File Report 94-622.

E.J. Helley and R.W. Graymer, 1997, Quaternary Geology of Contra Costa County, California and Surrounding Areas, Department of the Interior U.S. Geological Survey, Open File Report 97-98 Sheet 1 of 2

Nilsen, T.H., Taylor, F.A., and Brabb, E.E., 1976, Recent landslides in Alameda County, California (1940-71), U.S. Geological Survey Bulletin 1398, 21 p., scale 1:62,500

Working Group on California Earthquake Probabilities, 1999, Earthquake Probabilities in the San Francisco Bay Region: 2002 to 2031—A Summary of Findings, U.S. Department of the Interior U.S. Geological Survey, Open-File Report 99-517

Division of Mines and Geology, 1995, Landslide Hazards in the Martinez-Orinda-Walnut Creek Area Contra Costa County, California, Open File Report 95-12

Wayne D Haydron, 1995, Landslide Susceptibility Map Landslide Hazard in the Martinez-Orinda-Walnut Creek area, Contra Costa County, Landslide Hazard Identification Map No. 32. Division of Mines and Geology, Open File Rpt. 95-12, Plate 32A

Wayne D Haydron, 1995, Landslide Susceptibility Map Landslide Hazard in the Martinez-Orinda-Walnut Creek area, Contra Costa County, Landslide Hazard Identification Map No. 32. Division of Mines and Geology, Open File Rpt. 95-12, Plate 32B

Keith L. Knudsen, Janet M. Sowers, Robert C. Witter, Carl M. Wentworth, and Edward J. Helley, Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California: a Digital Database, U.S. Geological Survey Open-File Report 00-444, Online version 1.0

Alan Kropp & Associates, Inc., 1995, Landslide Map Berkeley Hills

Hart, E.W. and Bryant, W.A. 1997, Fault-Rupture Hazard Zones in California: California Division of Mines and Geology, Special Publication 42, 1999 revised edition and Supplement 3.

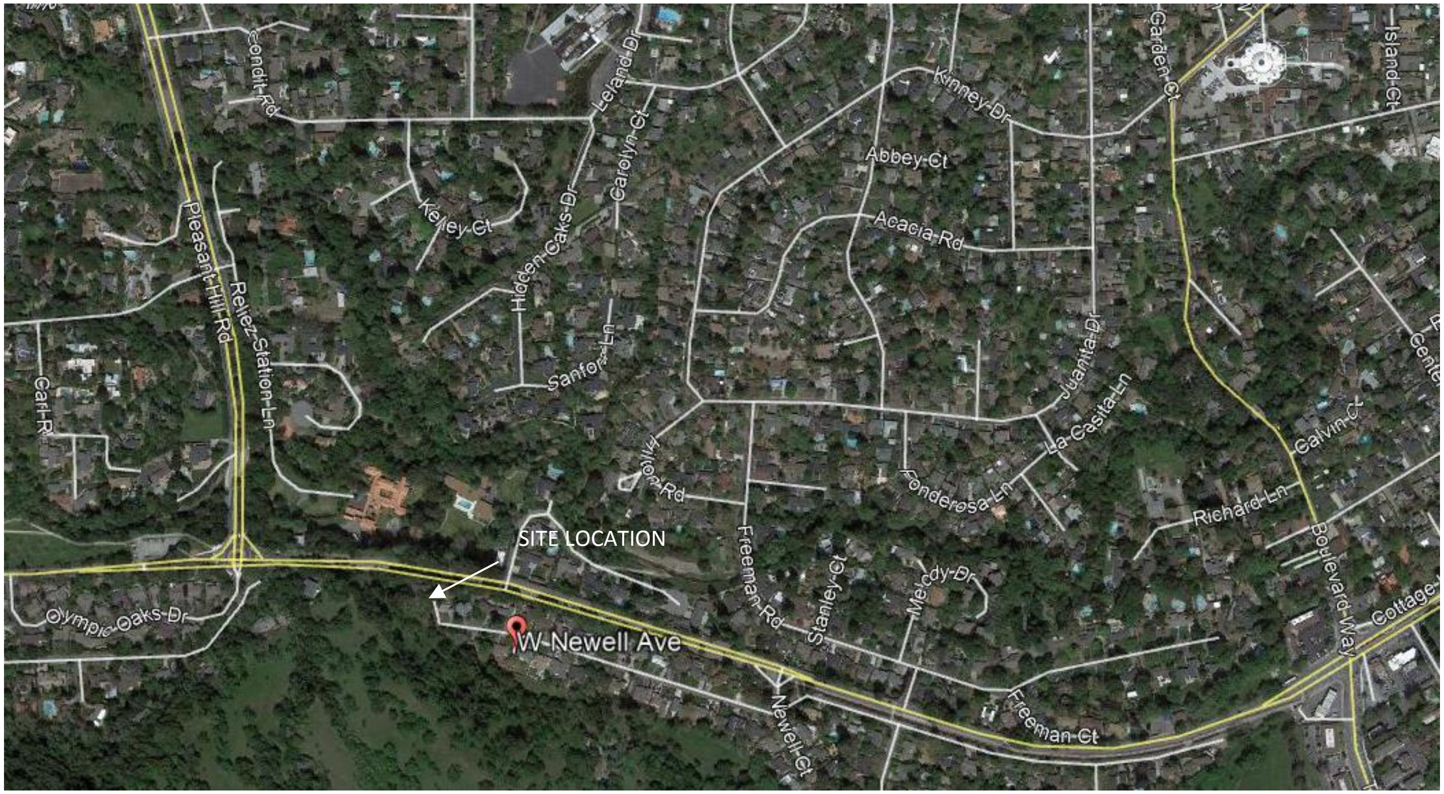
T.H. Nilsen, 1973, Preliminary Photointerpretation Map of Landslide and other Surficial Deposits of the Concord 15-Minute Quadrangle and the West Oakland Richmond and Part of the San Quentin 7 ½ Minute Quadrangle, Contra Costa and Alameda Counties, California, U.S. Department of the Interior U.S. Geological Survey, Misc. Field Studies Map MF-493 Sheet 1 of 2

Contra Costa County General Plan, 2010, Safety Element, Chapter 10

Charles W. Jennings and William A. Bryant, 2010, Fault Activity Map of California, California Department of Conservation, California Geological Survey.

William A. Bryant, Ellen F. Sander, and Christopher J. Wills, September 2007, Digital Database for the Concord and Green Valley Faults, U.S. Geological Survey

FIGURES

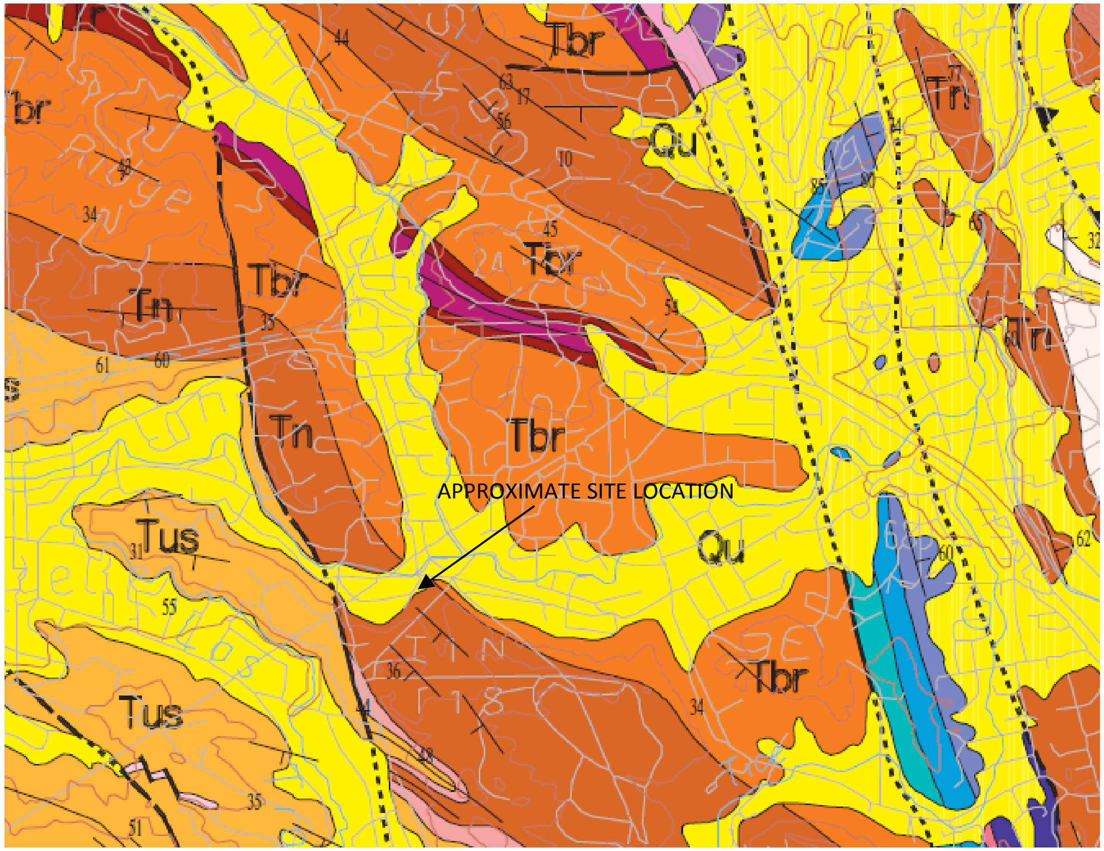


Bear Engineering Group, Inc.

TITLE **Vicinity Map**
W. Newell
Walnut Creek, CA APN # 238-050-007

SIZE | FIGURE NO. 1 | REV

SCALE Not to Scale | SHEET



Qu — Undivided Quaternary deposits

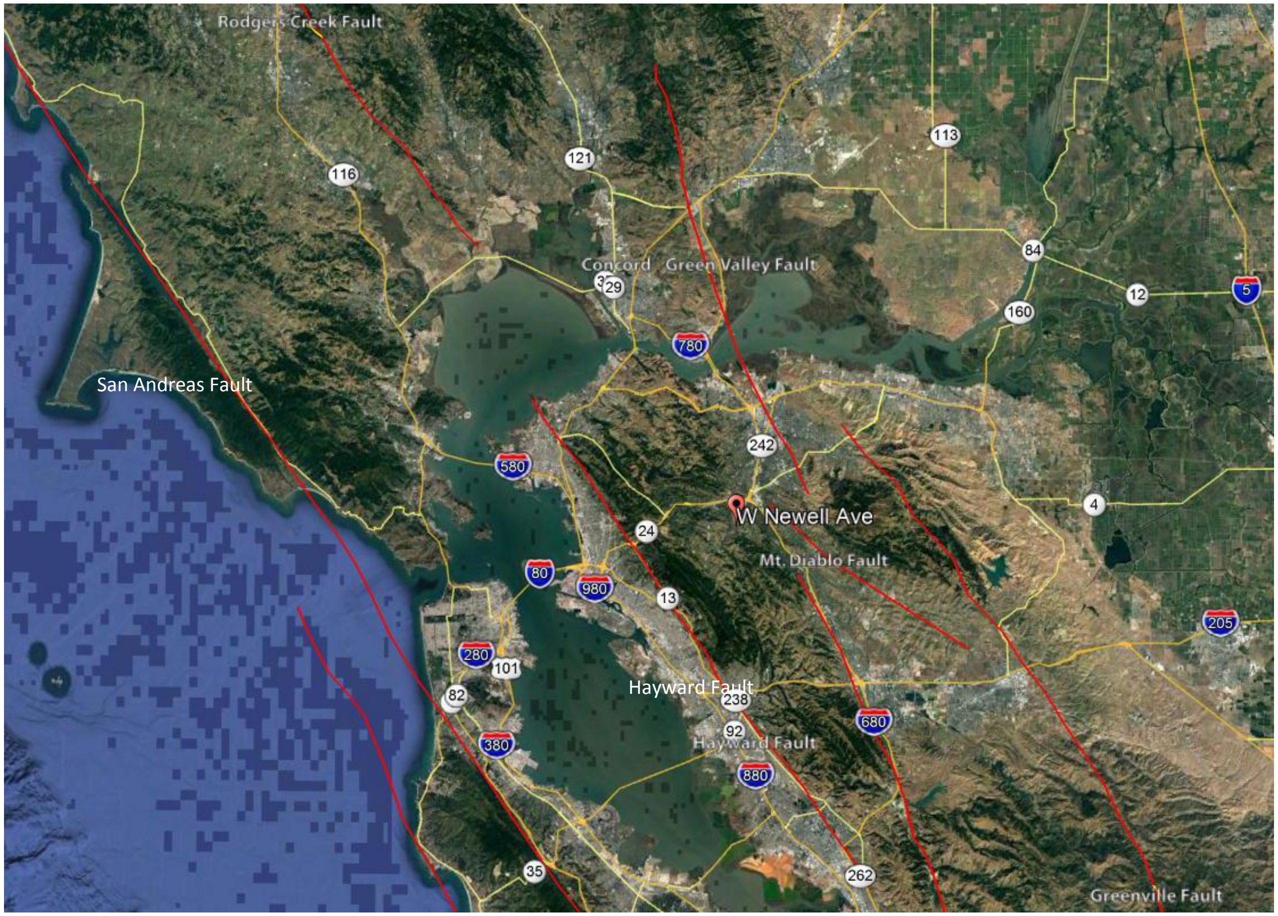
Tbr — Briones Formation,

U.S. Geological Survey
Miscellaneous Field Studies
MF–2342

Bear Engineering Group, Inc.

TITLE **Preliminary Geologic Map**
W. Newell
Walnut Creek, CA APN # 238-050-007

SIZE	FIGURE NO. 2	REV
SCALE Not to Scale		SHEET



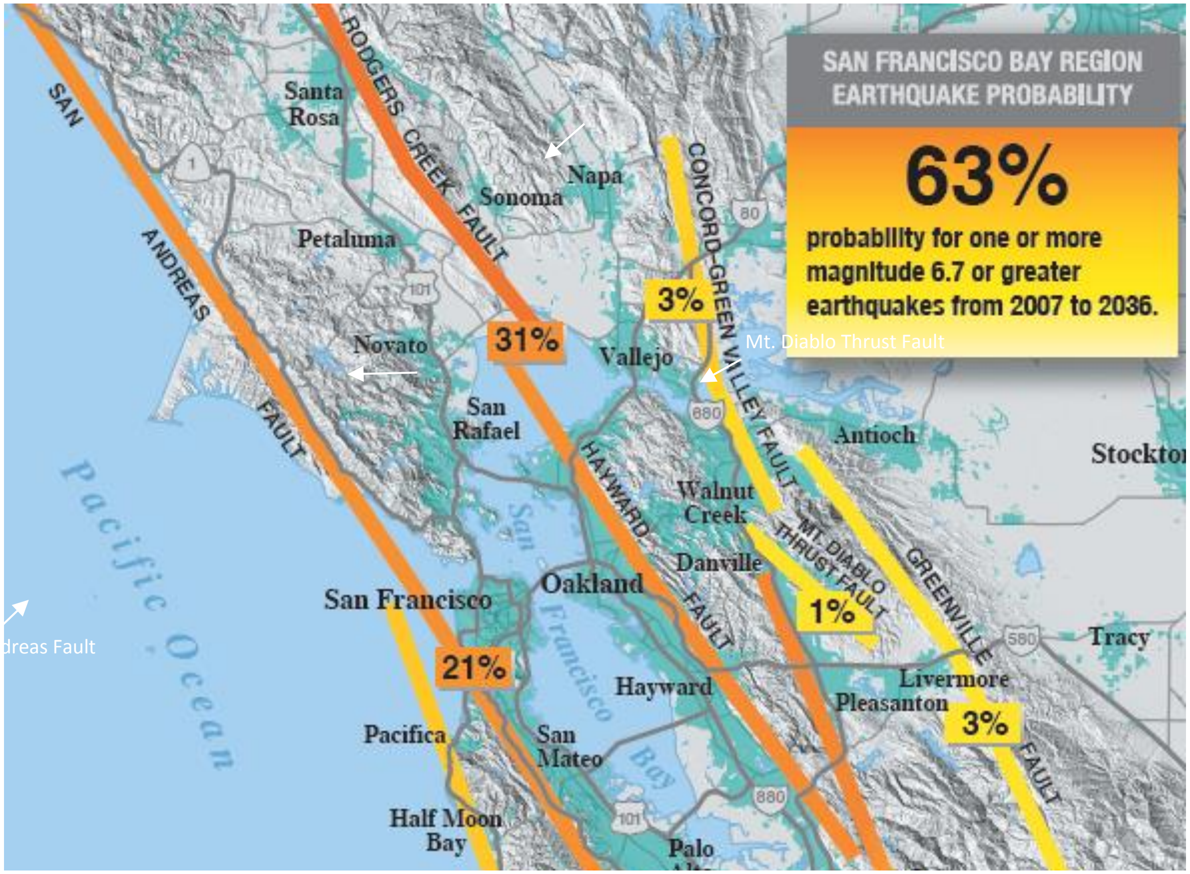
The closest significant fault is the Concord-Green Valley fault is located at about 5.2 km to the east

The San Andreas and Other Bay Area Faults, USGS
CALIFORNIA FAULT PARAMETERS - San Francisco Bay Region". California Geological Survey. 2003

Bear Engineering Group, Inc.

TITLE **Bay Area Fault Map**
W. Newell
Walnut Creek, CA APN # 238-050-007

SIZE	FIGURE NO. 3	REV
SCALE Not to Scale		SHEET



Legend:

% Probability of magnitude 6.7 or greater quakes before 2036 on the indicated fault

Increasing probability along fault segments

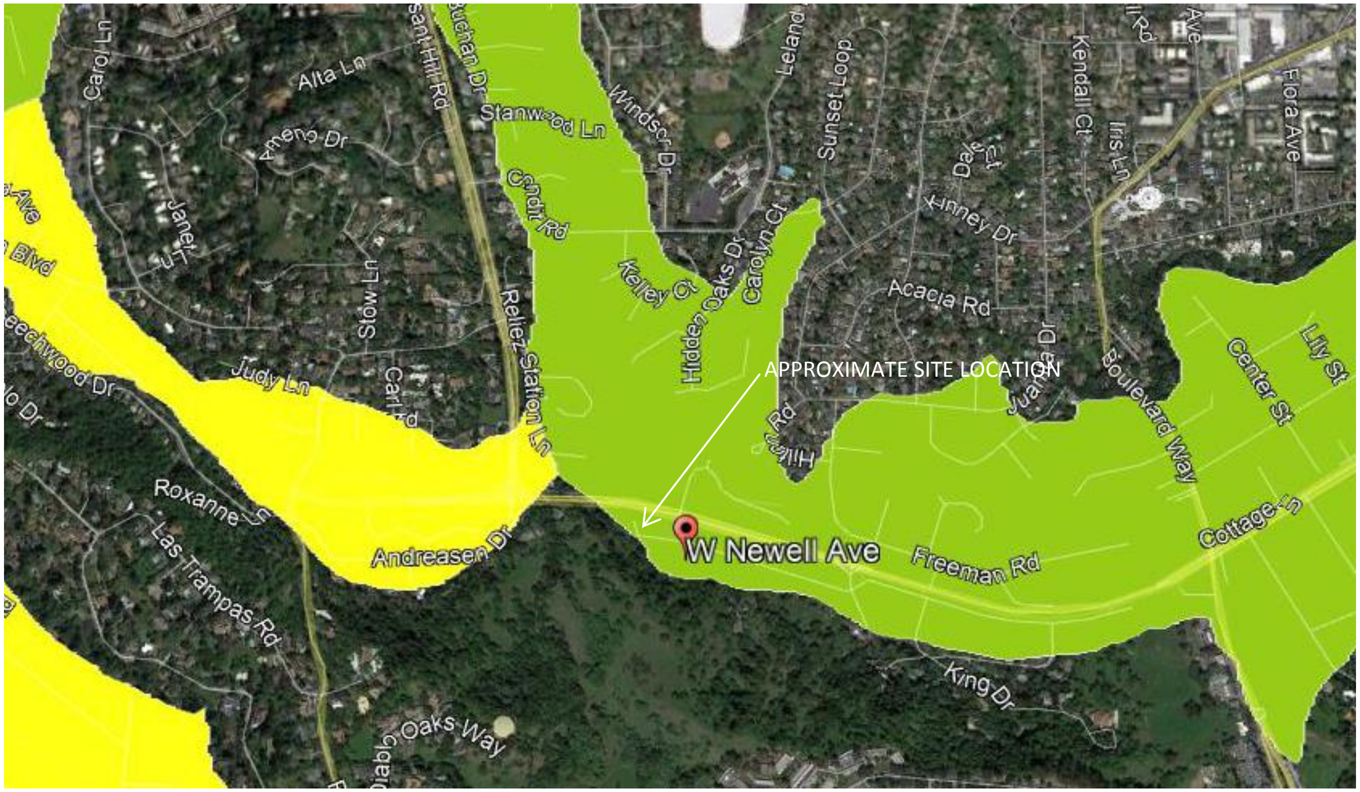
Expanding urban areas

Working Groups on California Earthquake Probabilities 2008

Bear Engineering Group, Inc.

TITLE **Probability Map**
W. Newell
Walnut Creek, CA APN # 238-050-007

SIZE	FIGURE NO. 4	REV
SCALE Not to Scale		SHEET



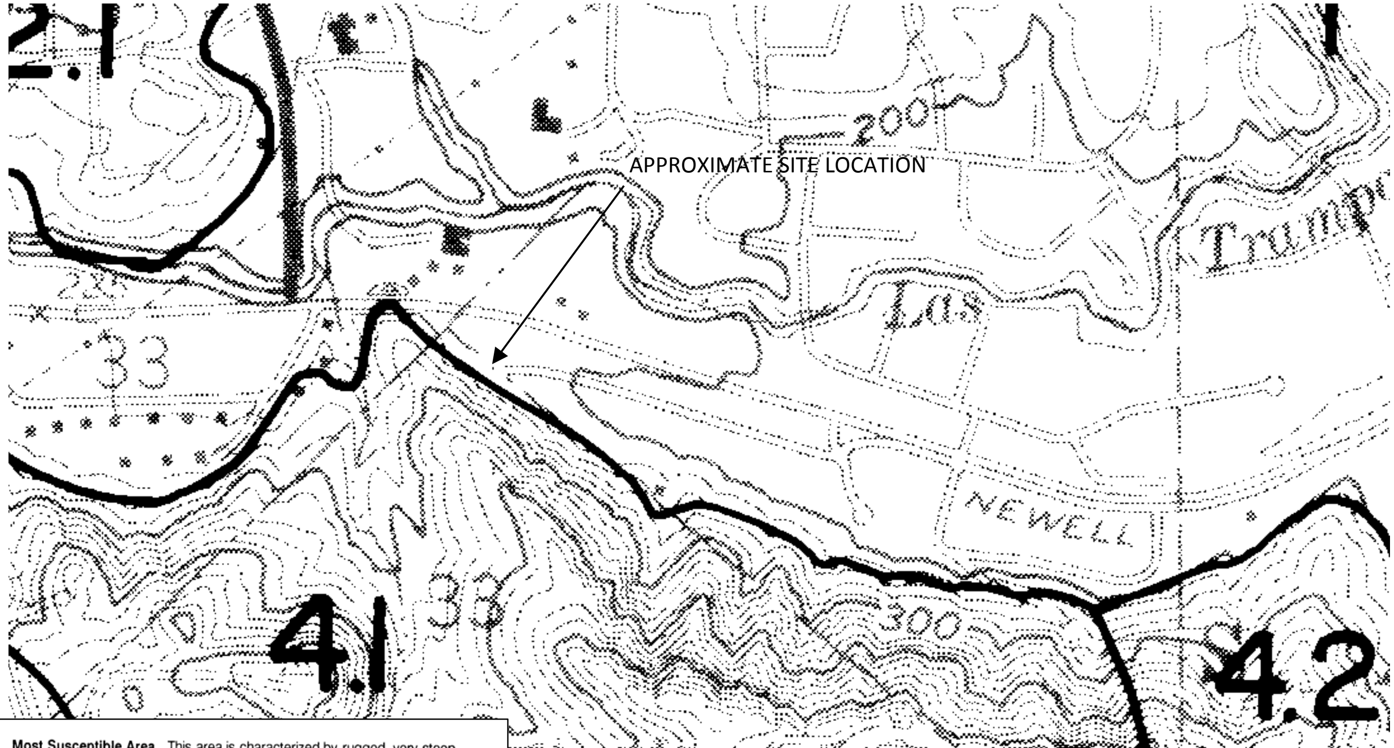
Low Susceptibility

SOURCE: U.S. Geological Survey Open-File Report 00-444

Bear Engineering Group, Inc.

TITLE	**Liquefaction Map** W. Newell Walnut Creek, CA APN # 238-050-007

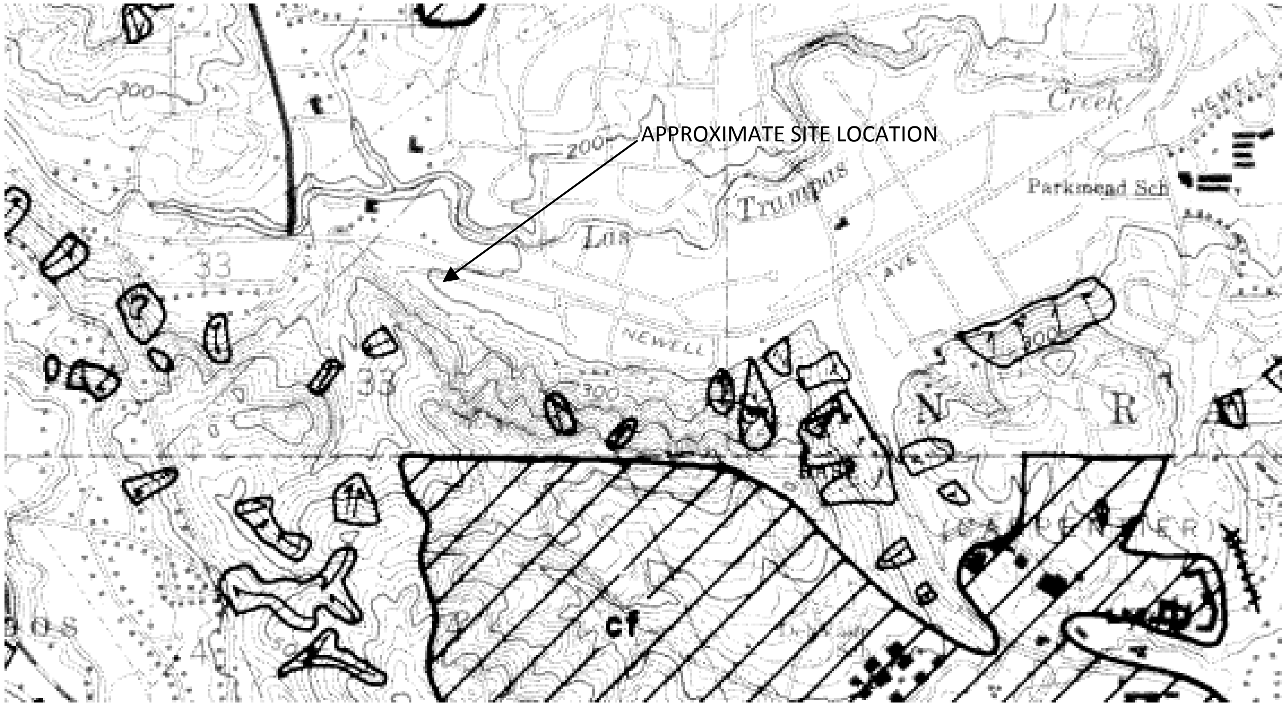
SIZE	FIGURE NO. 5	REV
SCALE Not to Scale		SHEET



AREA 4 **Most Susceptible Area.** This area is characterized by rugged, very steep terrain, with slopes that are generally greater than 50 percent (26 degrees) and locally 75 percent (37 degrees) or steeper. Most of the identified landslides are within Area 4. This category contains two subareas.

Landslide Susceptibility Map Landslide Hazard in the Martinez-Orinda-Walnut Creek area, Contra Costa County, Landslide Hazard Identification Map No. 32. Division of Mines and Geology, Open File Rpt. 95-12, Plate 32A

Bear Engineering Group, Inc.				
TITLE	**Landslide Susceptibility Map** W. Newell Walnut Creek, CA APN # 238-050-007			
SIZE	FIGURE NO. 6			REV
SCALE Not to Scale			SHEET	

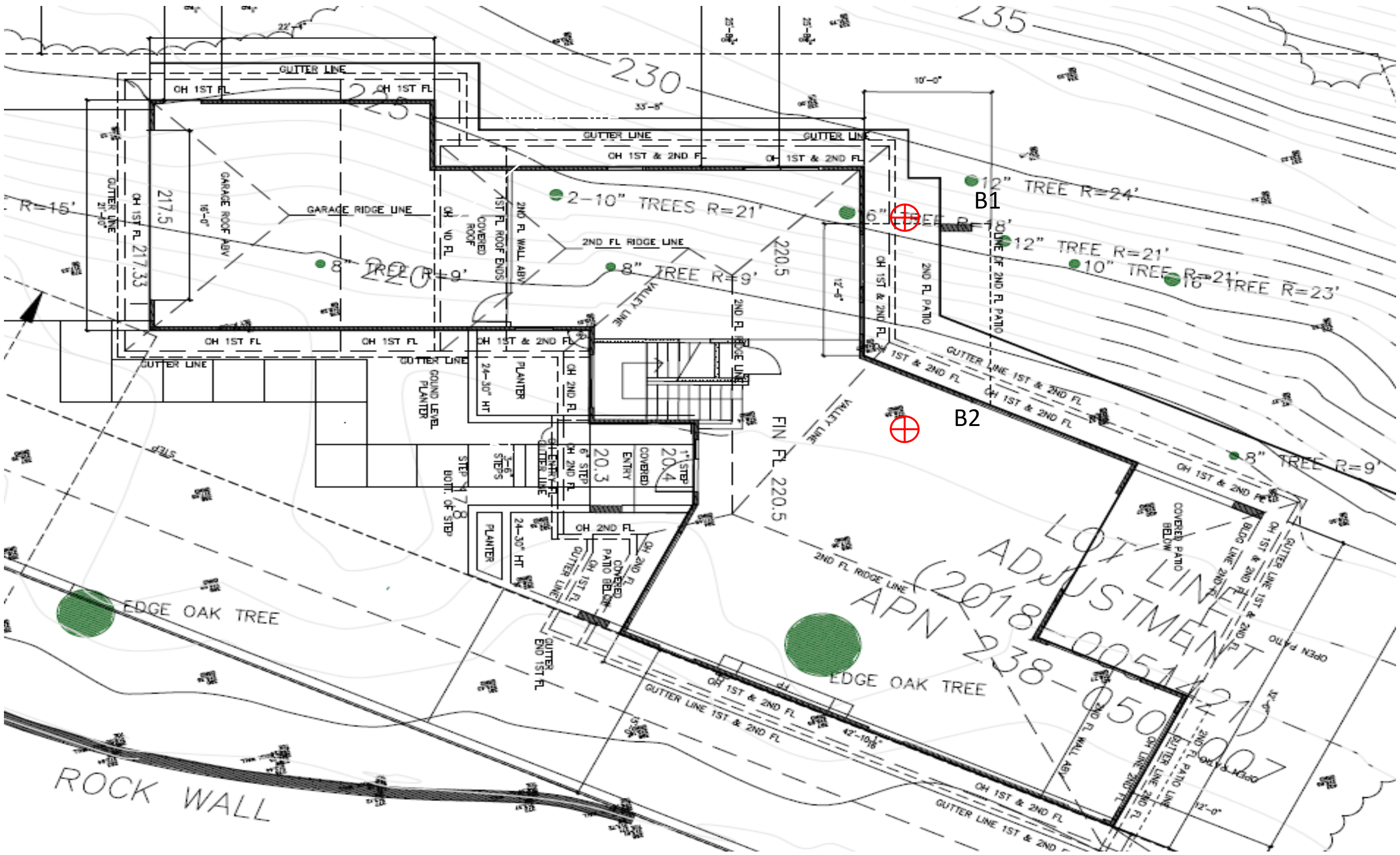


DEFINITE OR PROBABLE LANDSLIDE:

Landslide Hazard Map
Landslide Hazard in the Martinez-Orinda-Walnut Creek area, Contra Costa County, Landslide Hazard Identification Map No. , Open File Rpt. 95-12, Plate 32B

Bear Engineering Group, Inc.

TITLE	**Landslides Hazard Map** Landslide Susceptibility Map W. Newell
SIZE	FIGURE NO. 7
REV	
SCALE	Not to Scale
SHEET	



Bear Engineering Group, Inc.

TITLE	**Boring Location Map** Landslide Susceptibility Map W. Newell
SIZE	FIGURE NO. 8
REV	
SCALE	Not to Scale
SHEET	

BEAR ENGINEERING GROUP
Earth Science Consultants

Boring No. B1

Project No. 45-2019-01

Date: 6-7-19

Client: David Montalbo

Boring Depth: 4 ft.

Location: APN # 238-050-007

Ground Elev.

Driller: HIllside Drilling

Latititude: 37.8852135

Drilling Type: Solid Stem Auger

Longititude: -122.0891651

Compression Strength Penetrometer (tsf)

Shear Strength Torvane (tsf)

Sym. Denotes what test was taken falling between column denotes approx. strength in tsf.

Elev. (ft.)	Depth (ft.)	Sample No.	Blow Count	Lithology	Description	Dry Unit Wt. (pcf)	Moisture Content %	Plasticity Index	Unconfined Compression (psf)	% Passsing #200	1 2 3 4 5 6
		1	18		Silty clay medium brown, dry, moderately dense Colluvium	106.3	8.3				
					Sandstone red, tan, brown slightly weathered friable moderately hard moderately strong native						
		2	50/2"								
	5				Refusal 4.5 ft.						
	10										
	15										
	20										
	25										
	30										

BEAR ENGINEERING GROUP
Earth Science Consultants

Page 1 of 1

Boring No. B2

Project No. 45-2019-01

Date: 6-7-19

Client: David Montalbo

Boring Depth: 17 ft.

Location: APN # 238-050-007

Ground Elev.

Driller: HIllside Drilling

Latititude: 37.8852135

Drilling Type: Solid Stem Auger

Longititude: -122.0891651

Compression Strength Penetrometer (tsf)

Shear Strength Torvane (tsf)

Sym. Denotes what test was taken falling between column denotes approx. strength in tsf.

Elev. (ft.)	Depth (ft.)	Sample No.	Blow Count	Lithology	Description	Dry Unit Wt. (pcf)	Moisture Content %	Plasticity Index	Unconfined Compression (psf)	% Passsing #200	1 2 3 4 5 6
		1	16		Silty clay medium brown, dry, moderately dense Colluvium	101.71	11.3				
	5	2	33		Sandy loam, yellow brown, with trace gray silt Lenses, moist, medium dense, moderately Cemented, medium coarse alluvium	124.5	14.3				
	10	3	18		Sand-silty loam, yellow brown, moderately Cemented medium to fine coarse, dense alluvium						
		4	44		Increase torque at 13feet	125.6	9.6				
	15	4	50/4”		Sandstone red, tan, brown slightly weathered friable moderately hard moderately strong native						
					Refusal 17 ft.						
	20										
	25										
	30										