DARWIN MYERS ASSOCIATES
ENVIRONMENTAL RESEARCH ■ ENGINEERING GEOLOGY

April 28, 2026

Dianna Lecca, Project Planner
Contra Costa County
Department of Conservation & Development
Community Development Division
30 Muir Road
Martinez, California 94553

Subject: **Geologic Peer Review / CDVR24--01044 / 2nd Letter**
1518 Barth Avenue / APN 419-192-015
Bacilia Macias Architect (appli.) / E. Landeros. (owner)
San Pablo Area, Contra Costa County
DMA Project 3011.26

Dear Dianna,

On January 20, 2025 we issued a comment letter on the captioned project. Since that letter was issued the project geotechnical engineer has, in effect, resigned or been fired by their client, and no replacement has been named. Furthermore, John Campbell + Associates did not review or comment on grading as well as neglecting to comment on the proposed drainage. Preliminary recommendations were provided for the project but it is not stated that the project geotechnical engineer performed a site reconnaissance, and the scope of the Campbell + Associates investigation did not include subsurface exploration or laboratory testing of samples or slope stability analyses. Furthermore, Campbell + Associates neglected to reference a previous geotechnical report for the same site. That report was prepared by Cal Engineering & Geology (CEG). [1] That firm no longer exists. They were bought out by Haley Aldrich (HA), a civil engineering firm. In effect the acquisition of CEG allowed the scope of HA's staff to include geotechnical engineers, engineering geologist and field staff to implement their geo-recommendations. To my knowledge, HA tends to focus on major public works projects; not construction of a single-family residence. We reference the CEG report only to indicate that it presents the results of a broader scoped investigation that did include subsurface exploration, laboratory testing of samples and engineering analysis of the data gathered. Referencing the subsurface data could have strengthened the Campbell + Associates evaluation of the site or perhaps would have resulted in more conservative *Preliminary Recommendations*.

One further comment. County and neighborhood concerns are focused on slope stability, which implies knowledge of the bedrock geology and engineering properties of the over-lying Quaternary deposits. In our opinion it is critical that the applicant retain a geotechnical engineer that is experienced in hillside development, and ideally the geotechnical engineer having experience in the West County area would be an asset. There is something that we have not mentioned, which pertains to the geotechnical standard of care. Geotechnical engineers are expected to consider the effect of development on adjacent parcels. The proposed project should not reduce the stability of the adjacent developed lots and ideally the project should strive to improve the outlook for long-term stability. Similarly, runoff from a project site should not be allowed to worsen or create drainage problems for neighboring lots that are down-gradient.

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[1] Cal Engineering & Geology, 2008, *Geotechnical Investigation, Proposed New Residence, 1518 Barth Avenue, Richmond, CA.* CEG Job #070780, (report dated May 13, 2008).

1308 PINE STREET ■ MARTINEZ, CA 94553 ■ 925/370-9330

DMA Findings

We were provided the opportunity to provide final comments on the Geologic/ Geotechnical Conditions of Approval. Those comments are presented below.

GEO-1 There shall be no a) clearing of the site, b) grading, c) tree removal, or d) instillation of utilities prior to the issuance of construction permits.

GEO-2 All grading, excavation and filling shall be conducted during the dry season (April 15 through October 15) only, and all areas of exposed soils shall be revegetated to minimize erosion and subsequent sedimentation. After October 15, only erosion control work shall be allowed. Any modification to the above schedule shall be subject to review by the BID Grading Inspector, and the review / approval of the Zoning Administrator.

GEO-3 At least 45 days prior to issuance of Construction Permits, the project proponent shall submit a design level geotechnical recommendations that are based on adequate subsurface and laboratory test data. The expectations of the County for the scope of the investigation include the following:

A. The project geotechnical engineer shall provide design-level recommendations intended to guide preparation of grading, drainage and foundation plans. Construction drawings shall be subject to final review to ensure that geotechnical recommendations have been successfully incorporated into construction drawings. The geotechnical engineer shall excavate and log borings or test pits at/near the four corners of the area proposed for grading to establish the depth to bedrock and characterize the engineering properties of the bedrock and Quaternary deposits. The report shall include geologic cross-sections showing the geotechnical engineer's interpretation of subsurface sites conditions; the logs shall not be diagrammatic or generalized; they shall show the details of the earth materials penetrated. Representative samples shall be retrieved for laboratory testing. The logs should show the weathering profile, and comment on the effect of weathering on engineering properties of the units penetrated.
B. Samples collected during the subsurface exploration plan shall be subject to laboratory testing (we suggest that consideration be given to the following tests: moisture/density, compressive strength, shear strength, expansion potential, gradation testing of Quaternary deposits, and corrosion potential testing.),
C. Provide an original geologic map of the site that represents the geotechnical engineer's and/or engineering geologist's interpretation of site conditions (i.e., bedrock lithology, presence of any significant features (shear zones, bedding, deeply weathered zones, as well as native soils / artificial fill / Quaternary deposits).
D. The geotechnical update report shall provide mitigation measures for any significant impacts that are confirmed to be present of the site. That analysis shall give consideration to the effect of grading on the stability of adjacent parcels, and the effect of site runoff on the adjacent down-gradient parcels (e.g., recommended gradient of proposed engineered slopes shall give consideration to their effect on the adjacent parcels and improvement on those adjacent lands. This may warrant use of retaining walls instead of steeping natural slopes).
E. Provide detailed recommendations for geotechnical monitoring and testing during the construction period, which shall commence during clearing and stripping and extend through the grading period, and implementation of foundation recommendations, retaining wall construction and installation of drainage recommendations.
F. Roof gutter waters and waters falling on other impervious surfaces shall be conveyed to adequate discharge points and/or outfall into existing storm drainage facilities. This may warrant use of a

private on-site stormwater detention facility, that would become the maintenance responsibility of the owner.

GEO-4 The geotechnical report shall be subject to review by the County's peer review geologist, and review/approval of the Zoning Administrator. Improvement, grading and building plans shall carry out the recommendations of the approved report.

GEO-5 The geotechnical report required by GEO-3 routinely includes recommended geotechnical observation and testing services during construction. These services are essential to the success of the project. They allow the geotechnical engineer to *(i)* ensure geotechnical recommendations for the project are properly interpreted and implemented by contractors, *(ii)* allow the geotechnical engineer to view exposed conditions during construction to ensure that field conditions match those that were the basis of the design recommendations in the approved report, and *(iii)* provide the opportunity for field modifications of geotechnical recommendations (with BID approval), based on exposed conditions. The monitoring shall commence during clearing, and extend through grading, installation of recommended drainage facilities, and foundation related work, including retaining wall construction. A ***hard hold*** shall be placed on the "final" building inspection, pending submittal of a final report from the project geotechnical engineer that documents their observation and testing services, including the testing of any required backfill (e.g., backfilling of utility trenches). The monitoring report shall also include the geotechnical engineer's opinion on the compliance of the as graded, as-built project with all recommendations in the design level report.

GEO-6 The project proponent shall record a deed disclosure that is intended to *(i)* identify the project geotechnical engineers and reference all reports and letters issued by the geotechnical engineers (i.e., provide full bibliographic citation to these documents), *(ii)* provide information that explains on how an interested party could access these documents, *(iii)* state that no changes to site grading or drainage can be allowed without prior review and approval of the Department of Conservation and Development. Note that DCD's review/ approval may require justification from the property owner's geotechnical engineer, and *(iv)* explain that the property owner assumes monitoring and maintenance responsibility for all drainage improvements on the parcel. <u>A draft of the proposed Deed Disclosure language must be reviewed and approved by the Community Development Division (CDD) prior to recordation; after the Deed Disclosure is recorded, the project proponent must provide CDD with a copy of the recorded document to serve as evidence that the requirements of GEO-6 were satisfied.</u>

Purpose and Limitations

The purpose of our review was to provide a professional opinion on the adequacy of the geotechnical reconnaissance report submitted with the application to construction of a single-family residence on the captioned property. Specifically, we provide technical advice to assist the Community Development Division with discretionary permit decisions. Our services for this project have been limited to review of the reports cited in our two peer review letters. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the engineering geology profession. We trust this letter provides the evaluation and comments that you requested. Please call if you have any questions.

Sincerely,
DARWIN MYERS ASSOCIATES

Darwin Myers

Darwin Myers, CEG 946
Principal

REGISTERED GEOLOGIST
W. DARWIN MYERS
No. 946
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

DARWIN MYERS ASSOCIATES
ENVIRONMENTAL RESEARCH ■ ENGINEERING GEOLOGY

January 20, 2025

Nai Saephan, Project Planner
Contra Costa County
Department of Conservation & Development
Community Development Division
30 Muir Road
Martinez, California 94553

Subject: **Geologic Peer Review / CDVR24--01044**
1518 Barth Avenue / APN 419-192-015
Bacilia Macias Architect (appli.) / E. Landeros. (owner)
San Pablo Area, Contra Costa County
DMA Project 3037.24

Dear Nai,

As the County Peer Review Geologist, we are responding to the *Agency Comment Request* received on December 9, 2024. The captioned project is located approximately ¼ miles northeast of the Alquist-Priolo *Earthquake Fault Zone* (EFZ) that encompasses recently active and potential active traces of the Hayward Fault, and it within the vicinity where substantial landslide deposits and tightly folded bedrock are delineated on published geologic maps. The purpose of our peer review letter is to provide the professional opinion of an engineering geologist on the adequacy of the published geologic, seismic and geotechnical data, in combination with the report issued by John Campbell + Associates (JCA)[1] for the limited purposes of deeming the application complete from a *Geology and Soils* perspective. Our comments are organized to a) provide our understanding of the project, followed by b) a discussion of the geologic and seismic setting of the site, and c) a brief overview of Health - Safety Element Policies that are pertinent to the project. With that background, we then provide peer review comments on the investigation of JCA, followed by our evaluation and recommendations.

Understanding of Project

The application is a request for a variance to allow the following variances from the standards of the prevailing R-6 Zoning District to allow the construction of a new 2-story, single-family residence on a legally established, vacant lot. The application includes a request for approval of multiple variances: front yard setback, side yard setback, a permit for removal of one code protected tree and work within the dripline of another tree that is to be retained. Additionally, a small lot design review is required for the construction of the proposed 2,238 sq. ft. residence. The architect has prepared a Site Plan, floor plans, proposed exterior elevations and aerial images of the proposed new residence.[2] Topographically the project site is a steep, northwest-facing slope, with 28 ft. of relief and a gradient of approximately 45 percent.

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[1] John Campbell + Associates,2024, *Geotechnical Investigation of Proposed Residence, 1518 Barth Ave., San Pablo, CA*, JCA Job # 2024.9.2035 (17 pgs., report dated October 30, 2024; date stamped September 18, 2024).

[2] Bacilia Macias Architecture, 2024, *New Residence, 1518 Barth Ave., San Pablo, CA 94806, APN 419-192-015* (7 Sheets, dated September 18, 2024).

1308 PINE STREET ■ MARTINEZ, CA 94553 ■ 925/370-9330

Geologic and Seismic Setting

1. Introduction

We reviewed geologic reports and maps issued by the California Geological Survey (CGS) and the United States Geological Survey (USGS), along with the Soil Survey of Contra Costa County. With this background we a) analyzed a stereo pair of historic vertical-angle aerial photographs,[3] b) reviewed Safety Element maps and policies and c) reviewed previous engineering geology reports that provide data on site conditions in the neighborhood, d) evaluated the data gathered, and e) prepared the peer review letter presented herein which presents our evaluation and recommendations.

2. Surface Fault Rupture

The northwest-trending Hayward fault bisects the neighborhood. It is considered active by both the CGS and USGS. The official Alquist-Priolo Earthquake Fault Zone (EFZ) Map indicates the EFZ zone, in the vicinity of the site, is approximately ¼-mile wide and trends about N30°W. Figure 1 is a Vicinity Map that shows the local road network, as well as the EFZ (shaded orange), permanent open space (green) and creeks and water bodies (shaded blue). The project site is represented by a red dot located within a bullseye). In summary, the CGS has delineated a broad EFZ because information on the precise location of the active trace(s) is sketchy. The project site is approximately 1,000 ft. northeast of the A-P Zone. Recently active and potentially active fault traces may be present anywhere in the EFZ.

It should be recognized that the neighborhood that includes the site was developed when the CGS originally delineated the A-P Zone in the 1970s. Landslides, along with the actives of man (which included grading, drainage improvements and loss of native vegetation) can obscure or obliterate geomorphic features that are characteristic of active faulting. Perhaps the most significant geologic investigation of the neighborhood was performed for the San Pablo Redevelopment Agency by Woodward Clyde Consultants (WCC) during the late 1970's.[4] That investigation included a) a detailed field reconnaissance of the WCC study area b) geologic interpretation of aerial photographs, and c) logging on exploratory trenches and borings. The primary objectives of that study were to characterize geologic hazards within a redevelopment area and to serve as the primary source of information for the "Geologic and Soils" chapter of the CEQA document that was to be prepared by the Lead Agency (i.e., City of San Pablo). The WCC investigation failed to accurately establish the location of any active fault traces within their study area.

The A-P Act, California Public Resources Code, Division 2, Chapter 7-5, commencing with Section 2621, requires a geologic investigation directed to the hazard of surface fault rupture for "projects" located within the official EFZ. All proposed subdivisions of land (including minor subdivisions) are subject to the provisions of the A-P Act. The Act also allows local jurisdictions to have more stringent standards. The Contra Costs 2045 Health and Safety Element presents policies aimed at the protection of human life and reducing the potential for serious injuries from earthquakes, including ground shaking and surface fault rupture. General Plan policies pertaining to geologic and seismic hazards are presented in Table 1. Those policies indicate that fault hazard investigations are warranted in areas of suspected active faulting. In the case of the proposed construction of single-family, one-to-two story dwelling on a legally established parcel, it has been the practice of the County to only trigger fault hazard investigations where there is evidence of an active fault trace within approximately 200 ft. When required, the intent of the investigation is to determine if there is evidence of an active trace within the subject parcel. There has also been flexibility

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[3] Pacific Aerial Surveys, 1973, Photographs #CC3526-1-55 thru 1-57, scale 1 in.= 1,000 ft. (flown on May 7, 1973).

[4] Woodward-Clyde and Associates, 1978, *Phase II Geologic Assessment, San Pablo Redevelopment Agency Hillside Neighborhood, San Pablo, California* (report on file with the CGS; file #AP-727).

in the amount of the required structural setback. The County's objective is to allow development of the site provided that structures for human occupancy are setback from the active trace. The amount of structure setback is provided by fault investigation report, prepared by the applicant's consultant, based on appropriately detailed information. In one case in the East Richmond Heights area, a residential lot was developed with a detached garage constructed within the structure setback zone, and a variance to the rear yard setback standard of the prevailing zoning district was granted to facilitate development of the residence.

Table 1
Health & Safety Element Geologic Hazard Policies

HS-P11.1 For projects in Alquist -Priolo Earthquake Fault Zones or Seismic Hazard Zones (areas considered at-risk of earthquake triggered liquefaction or landslide displacement) delineated by the California Geological Survey, as well as any other areas of steep slopes or areas of suspected ground failure known to the County, require submittal of appropriately detailed engineering geologic or geotechnical investigations. The reports must be compliant with State Guidelines and include: a) A map showing the outline of any geologic or potentially hazardous soil conditions and areas subject to inundation. b) Recommended means of mitigation of any adverse condition representing a hazard to improvements. c) Recommendations to assure proper implementation of mitigation measures during construction.
HS-P11.2 Prohibit construction of buildings intended for human occupancy in areas where seismic and other geologic hazards (e.g. landslides, liquefaction and fault lines) cannot be adequately mitigated.
HS-P11.3 Discourage construction of critical facilities and buildings intended for human occupancy in Alquist-Priolo Fault Zones and encourage earthquake retrofitting where such development already exists. If there is no feasible alternative to siting critical facilities and buildings intended for human occupancy in the Fault Zones, buildings must be sited, designed and constructed to withstand the anticipated seismic stresses.
HS-P11.4 Refer geotechnical and engineering geologic reports to the County Peer Review Geologist for evaluation of their adequacy, as required by State Law for projects in State-designated hazard zones. Reports deemed inadequate will require further engineering analysis and revision until the findings/ opinions of the Peer Revie Geologist have been addressed to the County's satisfaction.
HS-P11.5 Discourage development on slopes exceeding 15 percent and prohibit development on slopes of 26 percent or greater to avoid slope instability, unnecessary grading and extensive land disturbance, and facilitate long-term control of erosion and sedimentation. Exceptions may be considered for infrastructure projects and development on existing legal lots where no other feasible building sites exist.
HS-P11.6 Require projects to form a Geologic Hazard Abatement District (GHAD) or join an existing GHAD whenever necessary to adequately mitigate anticipated or residual geologic hazards.
HS-P11.7 Do not accept public road dedications or allow construction of private roads on unstable hillsides or in landslide hazard areas unless potential hazards have been mitigated to the County's satisfaction. All private roads constructed in such areas must be fully compliant with private road standards adopted by the County and fire protection district with jurisdiction.
Source: Contra Costa County 2045 General Plan – Health and Safety Element, pages 9-52 & -53

In summary, the location of surface rupture generally can be assumed to be along an active major fault trace. Although mapping of the CGS and USGS have not confirmed the presence of any active or inactive faults on the project site. Furthermore, maps issued by those agencies indicate that the project site is not within the EFZ. Consequently, the risk of surface rupture on the project site is regarded as less-than-significant. Because of the proximity of the Hayward fault, we have provided the following discussion:

A. Guidelines for Investigations. It should also be recognized that the State has adopted Guidelines for evaluation of the Hazard of Surface Fault Rupture (DMG Note #49), and the State Mining and Geology Board adopted *Policies and Criteria* that are intended to guide implementation of the State Law. The provisions of the State Law are presented in Appendix A, and the Board's Policies and Criteria are presented in Appendix B. Additionally, the CGS has issued Guidelines for Reviewing Geologic Reports (CGS Note #41). Fault hazard reports prepared in compliance with the A-P Act must be peer reviewed by a registered geologist acting in behalf of the local jurisdiction, and copies of the fault hazard report along with a copy of the peer review must be provided to the CGS. Peer review is a critical part of the evaluation process. It is the duty of the peer reviewer to assure that the geologic investigation and resulting report adequately addresses the geologic conditions that exist on the site. CGS Note #41 requires that a) the reviewer must have the courage of his/ her convictions and not approve a report that is inadequate, b) the reviewer must bear in mind that some competent investigators are not accomplished writers, and that an important task of the reviewer is distinguish between the important and the insignificant. The guidelines also note that the best reviews generally are performed by experienced reviewers. (Use of multiple, part-time reviewers by a lead agency tends to prevent development consistently high-quality reviews.) This is, at least in part, attributable to different reviewers having differing standards, which can result in inconsistent treatment of development projects.

B. Historic Perspective. The Hayward fault zone comprises a northwest-trending zone of faults along the western front of the hills bordering the east side of San Francisco Bay. The fault zone can be traced nearly continuously northwesterly from the Warm Springs District in Southern Alameda County to San Pablo Bay at Point Pinole in Contra Costa County. It cannot be traced south of the Warm Springs District of Fremont with any degree of certainty. Southeast of the San Francisco Bay Area, the Hayward fault is inferred to merge with the Calaveras fault in the vicinity of the Calaveras Reservoir._Fault traces within this zone have experienced surface fault rupture during historic earthquakes, including the 1868 earthquake. That seismic event resulted in surface fault rupture on the segment of the Hayward fault between Mills College, Oakland and the Warm Springs District of Fremont. Faulting was reported as far north as the campus of the University of California, Berkeley. No large historic earthquakes are known to have occurred on the northern segment of the Hayward fault in historic time (i.e., segment that bisects Contra Costa County). In 2001 a paleo-seismicity study of Late Holocene deposits was conducted within the Mira Vista Country Club in El Cerrito (located approximately 2½ miles SE of the project site). The purpose of the study was to obtain scientific data on the occurrence of large earthquakes on the northern segment of the Hayward fault during the Late Holocene. Evidence of several seismic "events" that triggered fault rupture were confirmed in an exploratory trench that crossed the Hayward fault. Radiocarbon dating indicated that these seismic events (and associated surface fault rupture) occurred during the past 2,130 years. Detailed study of trench exposures yielded a recurrence interval ranging from 270 years to approximately 710 years for large earthquakes.[5]

C. Seismicity. Existing seismic records provide compelling evidence that the Hayward fault remains seismically active. For example, the earthquakes recorded in the San Francisco Bay Region show a good correlation between earthquake epicenters and known active traces, including the Hayward fault. These epicenters are evidence of adjustments taking place at-depth along active Bay faults. In a typical year, 20 or more felt seismic events are recorded in the immediate vicinity of the Hayward fault.

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[5] Lettis, W.R., 2001, *Late Holocene Behavior and Seismogenic Potential of the Hayward- Rogers Creek Fault System in the San Francisco Bay Area, California,* in Engineering Geology Practice in Northern California, C.G.S. Bulletin 201 and Assoc. of Engineering Geologists Spec. Pub. 12 (pages 167-177).

D. Fault Creep. Fault creep has been observed at various places along the length of the Hayward fault from Point Pinole to Fremont. Fault creep has cracked and offset curbs, streets, fences, railroad tracks, pipelines and buildings. All creep movement appears to be right-lateral. Near Point Pinole, a series of benchmarks were placed perpendicular to the fault by the USGS. During the period 1968 to 1980, 65 mm. of aseismic displacement was recorded (5.3 mm/year; Harsh and Buford, 1982). The long-term slip rate on the Hayward fault is believed to be on the order of 8 mm/year. No fault creep has been confirmed by the USGS on the segment of the Hayward fault that passes through the vicinity of the project site. Assuming an 8 mm/year average rate of displacement, 8 mm/ yr. displacement would translate into 31.5 inches of slippage on the fault per century. At Point Pinole, approximately two-thirds of the displacement may be occurring through fault creep. In the San Pablo Hills neighborhood of the amount and rate of fault creep has not been established.

E. Recently Active Traces of Hayward Fault. According to the CGS, recently active and potentially active traces of the Hayward fault may exist anywhere in the A-P Zone. Because the EFZ is centered on the discontinuous but aligned geomorphic features considered to be possible evidence surface fault rupture, the center of the EFZ may represent a higher risk area within the EFZ. In summary, evidence of the active trace(s) of the Hayward fault is not well defined in the San Pablo Hills, and there is potential for branching or en echelon traces / step-overs.

In the 1990s the USGS attempted to map the *recently active trace* [6] of the Hayward fault using three lines of evidence:[7] (a) geomorphic expression (i.e. terrain features that are aligned and are typically associated with fault displacement at the surface), (b) fault creep (i.e. aseismic fault slip), and (c) fault exposures in exploratory trenches excavated by consultants who were performing EFZ investigation for land development projects that were located in the EFZ. The major scientific goal of the USGS was to learn how the distribution of fault creep features and creep rate varied both along the fault and transverse to the fault. The text accompanying the report cautions engineers and land use planners that the clarity of the features along the fault is variable, and that subsidiary traces (i.e., branching or en echelon) may not be recognized because many sections of the fault were urbanized prior to enactment of the Alquist-Priolo Earthquake Fault Zone Act by the State of California. Furthermore, geomorphic features indicative of active faulting may have been obliterated by human activity (e.g., grading, drainage improvements, construction, urban vegetation). Additionally, both active and dormant landslides may have obliterated tectonic creep features. For these reasons, the main method of recognizing and precise location active fault strands for segments of the fault that lack reliable creep data will continue to be the subsurface data gathered by fault hazard investigations.

Figure 2 presents an enlargement of the latest revision of Lienkaemper map for the segment of the fault that passes through the project vicinity. This map identifies two subparallel traces in the vicinity of the project site. The western trace is shown passing approximately 1,600 ft. southwest of the site; the eastern trace is shown to passing approximately 1,100 ft. southwest of the site. Both traces are represented by dashed-and-queried symbols, indicating considerable uncertainty regarding the precise location of these fault traces. Approximately 1,500 ft. west-northwest of the CDVR24-01044 project site, the eastern trace of the Hayward fault is represented by a solid red line, indicating that the location shown is considered well defined. This line terminates where this trace crosses Bayo Vista Ave. Although the recently active trace of the Hayward fault is weakly

[6] *The term "recently active fault trace", as used in this USGS report, is defined as a fault trace that has evidence of movement during Holocene time (approximately the last 11,700 years).*

[7] Lienkaemper, J.J., 1992, *Map of Recently Active Traces of the Hayward Fault, Alameda and Contra Costa Counties, California,* USGS Miscellaneous Field Studies Map MF-2196 (1992; date of most recent revision 2008)

defined in the San Pablo hills, the evidence gathered to date indicate that the Hayward fault passes more than ¼ mi. southwest of the site.

3. Bedrock Geology

In 1994 the USGS issued a digitized geologic map of Contra Costa County that emphasized bedrock formations.[8] Figure 3, presents a clean Topographic Map, showing topography in the San Pablo hills and vicinity, but without the interpretation of bedrock geology, and without the local road network in the hills and without parcels This resulting topographic map indicates the project site is located on the backbone of a prominent ridge, whose axis trends northwesterly, along with the bedrock faults mapped by the USGS in the vicinity of the project site. This map does not classify faults by their activity status. However, the sub-parallel faults that pass southwest of the project site correspond to the twin traces of the Hayward fault; faults shown northeast of the project site are not considered active. No faults are shown crossing the project site.

Figure 4 shows the same area as was presented in Figure 3 but with roads, parcels and bedrock formations added to the topographic base map. The project site is indicated to be in the outcrop belt of the Orinda Formation (Tor). Note that this USGS map does not attempt to show the distribution of landslide deposits. The Orinda Formation consists of chiefly of sandstone, much of it clean, much clayey, along with some conglomerate and mudstone, and locally minor tuff and tuff breccia have been mapped in Tor. This formation is generally characterized as weakly consolidated, and weathered to depths of up to 30 ft. Most bedrock is unexpansive; some expansive (mudstone). Most soil mantle is severely expansive. The intergranular permeability of clean sandstones is high where the rock is clay free or nearly clay free; moderate where sand grains are clay coated; conglomerate is mostly high. Where clayey matrix material is abundant in fine-grained sandstone and siltstone, permeability ranges from low to very low; in tuffaceous material the permeability is low.

4. Landslides

Publications that provide mapping of landslides in the site vicinity include mapping of the CGS [9] and USGS.[10] These maps are presented in Figures 5 and 6. The scope of those investigations and their findings can be summarized as follows:

A. California Geological Survey. In 1973 the CGS (formerly the California Department of Mines & Geology) was retained by Contra Costa County and the Cities of El Cerrito, Richmond and San Pablo to perform a broad-scoped reconnaissance of the hills overlooking the Bay Plain in the three cities, along with the adjoining unincorporated area. The scope of work included (a) field mapping, (b) geologic interpretation of historic aerial photographs, (c) geophysical surveys, and (d) geologic evaluation of the data gathered and (e) publishing a report that provided the evaluation and recommendations of the CGS. The purposes of the investigation were to provide information on potential geologic and seismic hazards that could be incorporated into Safety Elements of the local jurisdictions; and provide data to planners, engineers, property owners and developers on potential geologic and seismic hazards. The CGS report was not intended to serve as a substitute for site-specific geologic / geotechnical investigations of individual parcels of land.

[8] Graymer, R., D.L. Jones & E.E. Brabb, 1994. *Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California.* U.S. Geological Survey Open File Report 94-622.

[9] Bishop, C.C., Knox, R.D., et. al., 1973, *Geological and Geophysical Investigation for Tri-Cities Seismic Safety and Environmental Resources Study,* California Division of Mines & Geology, Preliminary Report # 19

[10] Dibblee, T.W., 1980, *Preliminary Geologic Map of the Richmond Quadrangle, Alameda and Contra Costa Counties:* U.S.G.S. Open-File Map 80-1100 (scale 1:24,000).

Figure 5 presents a portion of the Bishop Landslide Map. According to this map the site is located approximately 500 ft. east of a 25 ac.± area classified "dg" (i.e., disturbed ground). The "dg" designation indicates that the CGS geologists concluded the area has experienced recently active ground movement, based on geomorphic features, which included scarps, other inferred landslide-related terrain features, and evidence of slope creep. Additionally, the Bishop Map shows several large landslides (outlined in red). The red arrow within the slide indicates the general direction of downslope displacement. The nearest of these landslides is approximately 400 ft. south of the site.

B. U.S. Geological Survey. In 1980 USGS geologist Tom Dibblee prepared a bedrock geology map of the Richmond Quadrangle that was based on field mapping. In areas where the bedrock was obscured by landslide deposits, the approximate boundary of landslides was plotted. Figure 6 shows key features mapped by Dibblee in the vicinity of the project site. The base map shows topographic contours, the local road network and parcel boundaries, and the site boundary is outlined in green. Geologic features that are shown include a) landslides (which are shaded pink, with a black arrow indicating the general direction of down slope movement), b) the axis of a synclinal fold (represented by black dashes (shaded yellow) which trend N40°W. A syncline is a *canoe-shaped* fold; as shown the axis of the fold passes approximately 125 ft northeast of the project site. The orientation of bedding on the site can be inferred to dip to the northeast (toward the synclinal axis). Additionally, Dibblee maps the location of two sub-parallel faults which generally match the inferred location of the Hayward fault traces mapped by Lienkaemper and the traces shown of the EFZ map issued by the CGS. These subparallel, northwest-trending faults pass approximately 1,500 and 1,800 ft. southwest of the project site. They represent the inferred location of Hayward fault traces. In summary, the landslides mapped by Dibblee do not pose a hazard to the project site. The location of landslides in mapped by Dibblee closely match the interpretation of landslides presented in Figure 5.

5. Soils

According to the Soil Survey of Contra Costa County,[11] the soil series mapped on the site is the Los Osos clay loam (LhF; 30 to 50% slopes). This soil series consists of well-drained soils underlain by soft fine-grained sandstone and shale. The soil profile is typically 32 inches thick, consisting of an A-horizon that extends from to surface to a depth of 10 inches. The B-horizon extends from the base of the A-horizon to the weathered bedrock. The A-horizon is described as *a very dark gray clay loam that is weak, fine, and with a sub-angular blocky structure*; the B-horizon is *similar to the A-horizon but with a higher clay content, and with a weak, coarse blocky prismatic structure*. LhF is a non-prime agricultural soil (Class IV). This soil is well drained, runoff is medium to rapid, and the hazard of erosion is "moderate" to "high" where the soil is bare.

The Soil Survey map of Contra Costa County considers the native soils on the project site to be *Highly Expansive* and *Highly Corrosive*. Expansive soils are soils that expand when water is added and shrink when they dry out. This continuous change in soils volume causes homes and other structures to move unevenly and crack. It should also be recognized that corrosive soils tend to damage concrete and/or uncoated steel that is in contact with the ground. Following grading and site preparation work for the proposed improvements, laboratory testing is warranted to confirm foundation conditions. Ideally, corrosivity testing is performed after clearing and any rough grading so that the laboratory testing addresses the properties of the foundation soils. The risks of damage associated with corrosive soils can be avoided / minimized by proper site preparation work, in combination with foundation and drainage design that is sensitive to the prevailing soils conditions.

[11] Welch, L.E. et. al., 1977, Soil Survey of Contra Costa County, California, USDA Soil Conservation Service

Seismic Hazard Zone Mapping Act

1. State Law

The provisions of the Seismic Hazard Mapping Act can be found in the California Public Resources Code, Chapter 7.8, Sections 2690-2699.6. This law is similar in many respects to the Alquist-Priolo Earthquake Fault Zone Mapping Act, which has been implemented by the County for 50 years. However, the official Seismic Hazard Zone (SHZ) maps issued by the California Geological Survey (CGS) identify areas that are at-risk of earthquake induced landslide displacement and earthquake induced liquefaction. The procedure for issuance of official SHZ maps is to distribute preliminary review copies of the SHZ maps and invite local jurisdictions, public agencies, and property owner/ general public to provide comments, particularly submittal of technical data. The CGS professional staff reviews the comments/ technical data provided. Based on input provided on the preliminary map(s), the CGS may modify the Preliminary Map. Finally, a public hearing is held before the State Mining and Geology Board with a recommendation from the CGS that the map(s) be approved. When SHZ maps are accepted as adequate by the Mining and Geology Board, they are distributed to local jurisdictions and public agencies. Nearly all land development projects that are located within areas at-risk of earthquake-triggered landslide displacement or liquefaction (or both) and which will eventually lead to construction of structures for human occupancy (including all major and minor subdivisions), require comprehensive geological/ geotechnical investigation. The SHZ Mapping Act has relatively few exemptions. However, construction of a single-family residence up to 2½ stories is exempt from the provisions of the State Law requiring a comprehensive landslide or liquefaction hazard investigation, provided the parcel was legally established prior to the issuance of the SHZ map. The project site is located within the Richmond Quadrangle. The official SHZ map was issued in 2024. [12] Figure 7 presents a portion of this SHZ map showing the San Pablo Hills and vicinity. The boundary of the project site is indicated with a green line. The area subject to potential hazard from earthquake induced landslide displacement is identified in a muted red color and the area subject to liquefaction is indicated in a yellow ochre color. According to Figure 7, the project site is not located in a SHZ.

2. Standards and Criteria

Accompanying each official SHZ map is a Seismic Hazard Zone Report. [13] The SHZ report describes the approach used by the CGS staff in their analysis and it presents technical data pertaining to the a) geology, b) groundwater, c) the probabilistic seismic hazard analysis model and its application to landslide hazard assessment d) results of materials testing, d) ground motion assessment, e) lists key references and f) explains the associated zoning techniques. Note that for a project site to be designated within an area of potential earthquake induced landslide displacement does not necessarily imply the presence of landslide debris on the property. The hazard designation implies that based on review of topographic, geologic, geotechnical and subsurface water conditions by the CGS geologists, there is a potential for permanent ground displacements such that mitigation as defined in California Public Resources Code Section 2693 (c) would be required.

3. Regarding the Relationship of SHZ Map to CEQA

The relationship of SHZ's to the CEQA process, the State of California CEQA Guidelines indicate the

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[12] California Geological Survey, 2024, *Official Map of Earthquake Zones of Required Investigation, Richmond Quadrangle Released: February 22, 2024.*

[13] California Geological Survey, 2024, *Seismic Hazard Zone Report 134 for the Richmond, Mare Island and San Quentin 7.5-Minute Quadrangles, Contra Costa County California,* (37 pgs. and 7 Plates).

following:

Nothing in these guidelines is intended to negate, supersede or duplicate any requirement of the SHZ. At the discretion of the lead agency, some or all of the investigations required by the Seismic Hazzard Mapping Act may occur either before, concurrent with or after the CEQA process.

The State CEQA Guidelines go on to indicate that if the SHZ investigation does not precede CEQA, it may be desirable for the CEQA document to describe the full range of mitigation measures that may be required to stabilize the land development project. <u>However, if all or part of the investigation is performed prior to completion of the CEQA process, it may be possible to narrow the discussion of mitigation alternatives to only those that would provide reasonable protection of the public safety given site-specific knowledge of the field conditions</u>.

Investigation of John Campbell + Associates

1. <u>Background</u>

In 2024 John Campbell + Associates (hereafter referred to as JCA) were retained to provide an overview of the geologic setting of the site, characterize foundation conditions, and provide conceptual recommendations for site grading, drainage and foundation design. The scope of the investigation was limited to a review of readily available geologic and geotechnical reports and maps and a field reconnaissance performed by a California licensed Civil Engineer. The consultant's scope of work did not include the following:

(a) Reference/ review civil engineering drawings prepared for the project (grading & drainage plans)
(b) Reference/ present data from a relevant investigation performed for nearby land development project in the immediate site vicinity
(c) Include analysis of stereo pairs of historic aerial photographs, which can provide the opportunity to view the site during buildout of the neighborhood; and opportunity to view of the site in different seasons of the year and in different years
(d) Subsurface exploration was no included in the scope of the investigation and no laboratory data on the engineering properties of the native soils or bedrock
(e) Did not present an original geologic map of JCA's study are their interpretation of site conditions.

2. <u>JCA Findings</u>

The consultant's findings were presented on pg. 6 of JCA report. They provide brief, rather generalized discussion of the following:

a) The site description indicates the parcel is a vacant lot and is a relatively steep downhill lot.
b) The geologic setting is overly simplified but identifies the bedrock as sandstone. No information is provided on its age, engineering properties or geologic structure of the sandstone.
c) At the surface, JCA indicates the presence of <u>silty sand</u> at the ground surface, underlain by bedrock. JCA references geotechnical borings on an adjacent parcel but fails to identify the property, the geotechnical firm or provide details of the soil and rock that was penetrated. JCA concludes that soil conditions on the project site are consistent with those encountered on the adjacent parcel.
d) JCA states that groundwater levels are not anticipated to pose an issue for development of the project site.

3. JCA Conclusions

a) Faulting The subject property is not located in the Earthquake Fault Zone delineated by the CGS, and no evidence of faulting was found during the site reconnaissance. JCA concludes the risk of surface fault rupture is less-than-significant (i.e., further evaluation of the risks associated with the potential for surface fault rupture is deemed to be necessary by the consultant).

b) Regional Seismicity. JCA provides a brief review of the seven (7) historic earthquakes that have impacted the San Francisco Bay Region. Additionally, the forecasts of the USGS for major earthquakes in the Bay Area are presented, including a 32% probability of a 6.7 or greater magnitude earthquake in the Hayward Fault by the year 2030.

c) Ground Shaking. Based on the seismicity of the Bay Area, JCA indicates there is potential for very strong to violent ground shaking on the project site. JCA recommends the improvements on the project be designed and constructed in compliance with the provisions of the California Building Code.

d) Liquefaction. After presenting a brief overview of the liquefaction hazard, JCA states the following:

> *Due to the density of the sandy clays and the clays are cohesive, soils on the site are not susceptible to liquefaction.*

However, JCA's **Findings** state that the sandstone bedrock on the site is overlain by silty sand. That discussion makes no mention of silty clay / cohesive clay overlying the bedrock. Unfortunately, gradation testing was not performed on the soils that overlies sandstone on the project site. Note that no information is provided by JCA on the thickness of the soils that overlie the bedrock. JCA concludes that the liquefaction potential is very low on the project site. We do not disagree with JCA's conclusion, but the report lacks data to support the conclusion.

e) Lateral Spreading, Densification and Landslides. JCA provides their conclusions regarding these potential hazards. The impacts are regarded as less-than-significant for each. We have one comment on the assessment of landslide hazards. JCA indicates that based on their review of landslide maps of the area, there is no landslide risk. We would note that the maps reviewed were not listed in Appendix A of their report, Furthermore, those published maps can only indicate that no landslides have been identified on or in the immediate vicinity to the site. On a relatively steep site, with an average slope of 45%, the risks of slope failure cannot be dismissed. Moreover, JCA has not gathered sufficient information to prove that the sandstone bedrock is present throughout the property. The lack of an original geologic map on the site which identifies sandstone outcrop areas within their study area, the lack of information on orientation of bedding, the absence of data on the weathering profile weakens JCA's assessment of slope stability.

f) Sandstone at Proposed Foundation Levels. JCA indicates that there will be competent sandstone at the proposed foundation levels. We do have some reservations, even though the consultant had the opportunity to review data from an adjacent parcel. That is because the JCA report a) did not include an Original Geologic of the project site, b) the average slope gradient on the site is 45% and total relief on the site is nearly 30 ft., c) nearby properties in the San Pablo Hills (located within the outcrop belt of the same formation that occurs on the site) have been mapped as landslide deposits (i.e., mapping of Bishop et. al., 1973), Dibblee, 1980 and Nilsen, 1975. [14] Additionally, d) no information is presented by JCA on the strike and dip of bedding, e) the thickness of the sandstone bed that occurs on the project site is unknown, f) the depth of weathering in the sandstone and its effect on the strength of the sandstone is unknown, and g) the distribution, thickness and engineering properties of the residual soil is unknown. A USGS report characterizes the

[14] Nilsen, T.H., 1975. *Preliminary Photointerpretation Map of Landslide and Other Surficial Deposits of the Richmond 7.5-Minute Quadrangle, Contra Costa & Alameda Counties,* U.S. Geological Survey, Open File Map 75-277-47.

stratigraphy of Orinda formation (identified by Ellen and Wentworth as Unit #131) in the San Pablo Hills as follows: [15]

> .... *interbedded conglomerate, sandstone, siltstone and mudstone. Much of the soil mantle and at least some of the bedrock is highly expansive. Within this formation the sandstone in places contains pebble trains and consists poorly sorted sand with matrix material that contains silt and clay to saturation. Clayey fine-grained sandstone and siltstone grade into one another and into mudstone. Much of this formation has irregular or lenticular bedding, some sandstone as laminated bedding and cross-bedded sandstone is also present. With regard to hardness, conglomerate and medium- to coarse-grained sandstone are mostly soft where weathered and fresh, but some of these materials are anomalously quite firm, especially where partially cemented by calcite* ....

In summary, the bedrock is not isotropic and homogeneous. In our experience where simplifying assumptions must be made due to the limited scope of work, it is important that geotechnical recommendations be conservative on the side of safety, and geotechnical monitoring during construction becomes especially important. Due to the steepness and height of the slope on the subject parcel, measures are required to control the long-term risk of active erosion and sedimentation. To date there are no grading, drainage or foundation plans have been provided for our review and none are referenced in Appendix A of the JCA report.

One final comment: On some occasions project sites that appear to have no serious geotechnical constraints can pose significant issues during or following the construction period.

g) Earthquake Hazards. JCA acknowledges that the project site is located within the highly seismic San Francisco Bay Area and indicates the foundation recommendations in the report are expected to provide a significant improvement in performance during earthquake ground shaking *over the existing foundation system* (?). The discussion of Earthquake Hazards on pg. 7 makes reference to the California Building Code (CBC) as it pertains to seismic design provisions. Note that on pg. 8 JCA provides 2022 seismic design criteria for the proposed project. Based on JCA's review of the geologic setting of the project site, the seismic design criteria include classifying the building site as Type D, and spectral accelerations are provided for use by the project structural engineer.)

h) Post Construction Settlement. JCA cautions that settlement of foundation improvements is to be expected. Assuming the project foundations are designed and constructed in compliance with JCA's recommendations, the total post construction settlement is anticipated to be less than 2 inches, implying that the structural design of the proposed improvements should be designed to mitigate the adverse effects of the anticipated settlement.

4. JCA Recommendations

Commencing on pg. 10 of the report, recommendations are provided for site grading, drainage and foundation design, commencing with plan review to assess compliance of the construction drawings with the geotechnical recommendations of JCA. Those recommendations, which extend onto pg. 16, appear to be generally compliant with the prevailing regulations governing the construction of a two-story, wood-frame single family residence. Note that project building plans are subject to review by the professional staff of the Building Inspection Division of DCD. It would not be surprising if BID required supplemental calculations, general notes, plan revisions or other clarifications.

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[15] Ellen, S.D. and C.M. Wentworth, 1995. *Hillside Materials and Slopes in the San Francisco Bay Region, California*. U.S. Geological Survey Professional Paper 1357.

DMA Evaluation

The purpose of our review was to provide a professional opinion on the adequacy of the geotechnical report to serve a suitable basis to allow for the full processing of the application, which is a request for approval of construction permits for a new single-family residence on the project site located on a steep slope and any justification it might provide for granting approval of the variances being requested. The primary geologic, geotechnical and seismic hazards to this project include the following: a) earthquake ground shaking, b) slope failure, c) erosion and sedimentation, d) and the native soils on the project site are classified as highly expansive and highly corrosive by the Soil Survey of Contra Costa County.

1. <u>Slope Stability and Drainage</u>

The density of landslides and the San Pablo hills neighborhood is evidence that existing slopes this portion of the outcrop belt of the Orinda Formation bedrock is marginally stable; and existing natural slopes are likely to be sensitive to grading. However, the project site <u>is not</u> within a mapped landslide area, and it <u>is not</u> located within the Seismic Hazard Zone (SHZ) delineated by the California Geological Survey (see attached Figures 5, 6 and 7). Because of the steepness of the slope gradient and total relief on the project site (45% gradient & 28 ft. of relief), there is potential for the project grading and drainage to create slope stability, sedimentation and/or erosion problems that could impact developed residential lots that are adjacent to the project site. Given the potential for the project to impact downslope lots, it is important that a) grading be minimized, b) use of sliver fills avoided and c) engineered/permanent retaining walls constructed with building permits in lieu of graded slopes.

Control of runoff is another factor that is critical to the success of the project. In our opinion adequate protection of downslope lots following buildout of the project should be designed with the intent to not increase either peak flows or total volume of runoff (i.e. design the project with the intent to collect runoff from graded and developed areas and convey it in a closed conduit to adequate existing drainage facilities.

2. <u>Seismicity</u>

Although the project site is not within the official Earthquake Fault Zone (EFZ) delineated by the CGC, the project site is within approximately ¼ mile of the active trace of the Hayward fault. Although the risk of surface fault rupture is less-than-significant on the project site, the property is within an area where violent ground shaking is a potential hazard, particularly in the event a major earthquake on the Haward fault. The JCA report provides seismic parameters that are used by the project structural engineer to design improvements that mitigate the ground shaking hazard. The intent of the CBC is to mitigate ground shaking damage as described below:

> The site is within the seismically active San Francisco Bay Region area, where a moderate to high magnitude earthquake is a foreseeable event. The risk of damage from ground shaking is controlled by using sound engineering judgement and compliance with the latest provisions of the California Building Code (CBC), as a minimum. The seismic design provisions of the CBC prescribe minimum lateral forces applied statistically to the structure(s), combined with the gravity forces and dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. The intent of the code is to enable structures to *(i)* resist minor earthquakes without damage, *(ii)* resist moderate earthquakes without structural damage but with some non-structural damage, and *(iii)* resist major earthquakes without collapse but with some structural as well as non-structural damage.

3. Soils

The Soil Survey of Contra Costa County classifies the soils that occur in the San Pablo hills as highly expansive and highly corrosive. The geotechnical report for the project did not include any subsurface exploration of the site and no laboratory testing. In our opinion there is a need for a geotechnical report update prior to issuance of any construction permits requiring evaluation of these adverse soil properties and the report shall provide mitigation measures for any adverse soil conditions that are confirmed to be present on the site.

DMA Recommendations

Based on review of our review of the materials provides by the project proponent, including the JCA report, along with review of the pertinent published geologic and seismic hazard maps and the General Plan Policies presented in Table 1, it is our opinion that the Community Development Divisions of DCD should require submittal of an update geotechnical report prior to issuance of an Update Geotechnical Report as a Condition of Approval. The intent of the report is to present subsurface and laboratory test date that is adequate to confirm/ modify the preliminary conclusions and recommendation of the project geotechnical engineer. The following are recommended Conditions of Approval.

GEO-1 At least 30 days prior to issuance of Construction Permits, the project proponent shall submit an updated geotechnical report that provides adequate subsurface and laboratory test data. The expectations of the County for the scope of the investigation include the following:

A. The project geotechnical engineer shall review design-level grading, drainage and foundation plans, referencing the date of the plans reviewed.
B. The geotechnical engineer shall excavate and log borings or test pits at/near the four corners of the area proposed for grading to establish the depth to bedrock, characterize. The report shall include logs showing the details of the earth materials penetrated. The logs shall not be diagrammatic or generalized. Representative samples shall be retrieved for laboratory testing. The logs should show the weathering profile, and comment on the the effect of weathering on engineering properties of the units penetrated.
C. Samples of the samples collected shall be subject to laboratory testing (moisture/density, compressive strength, shear strength, expansion potential, gradation testing of native soils, and corrosion potential testing of soil and bedrock. and gradation),
D. Provide an original geologic map of the site that represents the geotechnical engineer's and/or engineering geologist's interpretation of site conditions (i.e., bedrock stratigraphy, presence of any significant features (shear zones, bedding, deeply weathered zones, properties of native soils).
E. The geotechnical update report shall provide mitigation measures for any significant impacts that are confirmed to be present of the site,
F. Restate recommendations for geotechnical monitoring and testing during the construction period.

GEO-2 The geotechnical report shall be subject to review by the County's peer review geologist, and review/approval of the Zoning Administrator. Improvement, grading and building plans shall carry out the recommendations of the approved report.

GEO-3 The geotechnical report required by GEO-1 routinely includes recommended geotechnical observation and testing services during construction. These services are essential to the success of the project. They allow the geotechnical engineer to *(i)* ensure geotechnical recommendations for the project are properly interpreted and implemented by contractors, *(ii)* allow the geotechnical engineer to view exposed conditions during construction to ensure that field conditions match those that were the basis of the design recommendations in the approved report, and *(iii)* provide the opportunity for field modifications of geotechnical recommendations (with BID approval), based on exposed conditions. The monitoring shall commence during clearing, and extend through grading, installation of recommended drainage facilities, and foundation related work, including retaining wall construction. A ***hard hold*** shall be placed on the "final" building inspection, pending submittal of a report(s) from the project geotechnical engineer that documents their observation and testing services, including the testing of any required backfill (e.g., backfilling of utility trenches). The monitoring report shall also include the geotechnical engineer's opinion on the compliance of the as graded, as-built project with all recommendations in the design level report.

GEO-4 All grading, excavation and filling shall be conducted during the dry season (April 15 through October 15) only, and all areas of exposed soils shall be revegetated to minimize erosion and subsequent sedimentation. After October 15, only erosion control work shall be allowed. Any modification to the above schedule shall be subject to review by the BID Grading Inspector, and the review / approval of the Zoning Administrator.

GEO-5 The project proponent shall record a deed disclosure that is intended to *(i)* identify the project geotechnical engineers and reference all reports and letters issued by the geotechnical engineers (i.e., provide full bibliographic citation to these documents), *(ii)* provide information that explains on how an interested party could access these documents, *(iii)* state that no changes to site grading or drainage can be allowed without prior review and approval of the Department of Conservation and Development. Note that DCD's review/ approval may require justification from the property owners geotechnical engineer, and *(iv)* explain that the property owner assumes monitoring and maintenance responsibility for all drainage improvements on the parcel. A draft of the proposed Deed Disclosure language must be reviewed and approved by the Community Development Division (CDD) prior to recordation; after the Deed Disclosure is recorded, the project proponent must provide CDD with a copy of the recorded document to serve as evidence that the requirements of GEO-5 were satisfied.

Purpose and Limitations

The purpose of our review was to provide a professional opinion on the adequacy of the geotechnical reconnaissance report submitted with the application to construction of a single-family residence on the captioned property. Specifically, we provide technical advice to assist the Community Development Division with discretionary permit decisions. Our services have been limited to interpretation of 1973 aerial photographs and review of the referenced reports and maps. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the engineering geology profession.

We trust this letter provides the evaluation and comments that you requested. Please call if you have any questions.

Sincerely,
DARWIN MYERS ASSOCIATES

REGISTERED GEOLOGIST
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Principal

JOHN CAMPELL + ASSOCIATES
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL INVESTIGATION

Proposed Residence

1518 Barth Ave.
San Pablo, CA

Prepared by

John Campbell + Associates

REGISTERED PROFESSIONAL ENGINEER
ARTUR DA ROSA
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STATE OF CALIFORNIA

10/30/2024

- October 30, 2025
- Project No. 2024.9.2035

Table of Contents

Approval

The following Geotechnical Investigation was conducted under the supervision of

INTRODUCTION

Project Description

A geotechnical study has been completed for a new residence at 1518 Barth Avenue in San Pablo, California. The project involves new foundations for the new residence.

Purpose and Scope of Work

The purpose of this study is to gather information on the nature, distribution, and characteristics of the earth materials and the groundwater conditions at the site to prepare this report, which includes conclusions and recommendations for the design and construction of the new foundations for the residence. The design criteria are intended for use by your structural engineer. In addition, we have evaluated the site's exposure to primary geologic hazards, including faulting and ground shaking; and to secondary geologic hazards, including landsliding, liquefaction, subsidence, and ground spreading during future earthquakes.

Our investigation included an engineering reconnaissance of the site and surrounding areas; a review of published geologic data pertinent to the project area; engineering analyses; and this report’s preparation.

This report contains the results of our investigation, including our findings regarding site, soil, geologic, and groundwater conditions; conclusions pertaining to geologic hazards and geotechnical considerations; and recommendations for foundation and drainage construction.

Pertinent exhibits appear in Appendix A.

Review of Geotechnical Data

Several published and unpublished sources of data were reviewed to evaluate geotechnical information regarding the subject parcel. This information included geotechnical literature, topographic and geologic maps, and preliminary photo interpretive landslide maps prepared by the United States Geological Survey, also including geologic, landslide, and fault maps prepared by the California Geological Survey (formerly the California Division of Mines and Geology - CDMG).

A list of the published sources used is presented at the end of this report.

FINDINGS

Site Description

The subject property is currently vacant and is on a relatively steep downhill lot.

Geologic Setting

Bedrock and Structure

Wagner, (1991), has mapped the underlying deposits as Sandstone derived from the underlying bedrock.

Earth Materials

Soils and Bedrock

Silty sands were encountered at the surface underlain by sandstone bedrock. relatively shallow foundations are anticipated. previous borings were performed on an adjacent site, and the soil conditions are consistent with the subject site.

Groundwater

Groundwater levels are not anticipated to be an issue for the proposed construction.

CONCLUSIONS

Geologic Hazards

Faulting

The property is not within a current *Alquist-Priolo Special Studies Zone*, and we did not observe any evidence of active faulting during our reconnaissance of the property. In addition, no evidence of faulting or fault-related features were noted on the property. We believe that there is little risk of ground rupture along a fault trace at the site.

Regional Seismicity

The site is within the Coast Range Province which is considered seismically active. Historical earthquake records indicate a potential for strong earthquake shaking throughout the entire East Bay area. Large magnitude earthquakes have historically impacted the Bay area in 1836, 1938, 1852, 1861, 1868, 1906, and most recently, 1989 (the Loma Prieta earthquake).

Studies by the United States Geological Surveys Working Group on California Earthquake Probabilities (United States Geological Survey, 1999) have estimated a 70 percent probability of at least one magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region before the year 2030. As part of their prediction, they estimated the probability to be 32 percent for a magnitude 6.7 or greater earthquake to occur on the Hayward fault by the year 2030.

Ground Shaking

The greatest short-term risk to the property is from ground shaking from forecast activity along the Hayward Fault, Calaveras, and the San Andreas Faults along the Peninsula. Future strong earthquake shaking should be anticipated at the site from these faults are active and trend in a northwest direction through the Bay Area and are capable of producing very strong to violent earthquakes. It will be necessary to design and construct the project improvements in strict adherence to current standards for earthquake-resistant construction.

Liquefaction

Liquefaction is a sudden loss of shear strength experienced in saturated granular soils below the groundwater level during strong earthquake ground shaking. According to Seed (1983), the likelihood of this phenomenon is dependent on many factors, including the intensity of duration of ground shaking, soil density and particle size distribution, and position of the groundwater table. Due to the density of the sandy clays and the clays are cohesive the soils at the site are not susceptible to liquefaction. We anticipate that the groundwater table and/or phreatic (saturation) level beneath the residence may possibly rise during winter storm periods. Even under these conditions, however, it is estimated that the risk of liquefaction beneath the property is very low.

Lateral Spreading

Lateral spreading or lurching is generally caused by seismically induced liquefaction of marginally stable soils underlying gentle slopes and is usually accompanied by fissures. Because the soils underlying the site are not subject to liquefaction during a future earthquake, and the site is on a relatively gentle slope there is a very low risk associated with seismically-induced lateral spreading affecting the structure.

Densification

Earthquake-induced densification and settlement of soils above the groundwater table are considered unlikely due to the density of the clays.

Landsliding Risk

Although the area is relatively steep, we checked landslide maps of the area, there is no risk of landsliding.

General Geotechnical Considerations for Proposed Foundation Improvements

Based on our investigation, it is our opinion the site is suitable for the proposed foundation improvements from a geotechnical standpoint. All the conclusions and recommendations presented in this report should be incorporated into the design and construction of the project to reduce potential geotechnical risks.

- The presence of relatively dense Sandstone at proposed foundation levels;
- and Earthquake hazards.

These considerations are discussed as follows.

Sandstone at Proposed Foundation Levels

Sandstone will be found at the surface and well below foundation levels. These materials are relatively easy to excavate and are suitable for relatively high bearing pressures as required by foundations and retaining walls. The foundation depths for the structure will be extended approximately 18 inches below grade to provide lateral resistance to movement during seismic activity. New foundations, foundation improvements, and retaining walls can be supported on conventional spread footing foundations. Provided that the foundations are extended to the recommended depths conventional spread footings will provide adequate support for the structures.

Earthquake Hazards

The subject site is located in the highly seismic San Francisco Bay Area and there is a strong probability that a moderate to severe earthquake will occur during the life of the structure. The foundations recommended in this report would be expected to generally provide a significant improvement in performance over the existing foundation system. The 2024 California Building Code has adopted provisions for the incorporation of ground shaking into the design of all structures. Recommendations for geotechnical parameters to be used in the structural seismic design of the new foundations are presented in the Recommendations Section.

Post Construction Settlement

Minor cracking during the settlement of foundation improvements is to be expected following construction.

Provided that the foundation supports are designed and constructed in accordance with our recommendations, we estimate that short-term maximum total post-construction settlements will be less than about 2 inches.

RECOMMENDATIONS

General

It is the responsibility of the owner or his representative to confirm that the recommendations presented in this report are called to the attention of the contractor, subcontractors, and any governmental body that may have jurisdiction and that these recommendations are carried out in the field.

It will be necessary to design and construct the project improvements in strict adherence to current standards for earthquake-resistant construction. It is our understanding that the project is to be designed in accordance with the 2024 California Building Code.

In order to determine the soil profile in the upper 100 feet, we reviewed the hardness data from typical bedrock materials in the area. We are recommending an SC soil/rock profile for the seismic design of the improvements to the site and structure.

Seismic Design

The closest major active faults in the area, San Andreas and Hayward, are capable of producing an earthquake with a magnitude equal to or greater than 7.5.

Based on our review of the site location, geology, and the newly adopted 2022 California Building Code (CBC), we recommend the following parameters be used for the seismic design of the foundations for the residence.

- Site Class D Mapped Spectral Acceleration for Short Period (Ss,=2.610g)
- Site Class D S1= 1.02g
- Site Class D Sms= 2.74g
- Site Class D Sm1 = 1.82
- Site Class D (Sds=2.15g)
- Site Class D Sd1= 1.44g.

Wet Weather Construction

Construction is most economically performed during the summer months when the on-site soils are driest. Delays should be anticipated in site construction performed during the rainy season due to excessive moisture. Special and comparatively expensive construction procedures should be anticipated if construction must be completed during the winter.

If utility trenches or excavations are open during winter rains, then caving of the trenches or foundation walls may occur. Also, if the foundation trenches fill with water during construction, or if saturated materials are encountered at the anticipated bottom of the foundations, they may need to be extended to greater depths to reach adequate support capacity than would be necessary if dry weather construction took place. We should also note that it has been our experience that increased clean-up costs will occur, and greater safety hazards will exist if the work proceeds during the wet winter months. Furthermore, engineering costs to observe construction are increased because of project delays, modifications, and rework.

Clearing and Grading

Those areas where new foundations will be constructed should be stripped of any topsoil or loose materials and removed from the site. After the selected areas of the site have been stripped, excavations required can be made.

Structural Fills

No fills are anticipated at the project. If fills are needed, only select, non-expansive soils, should be used as fill below slabs, (if necessary), and behind retaining walls. Fill material should have a Plasticity Index of less than 15. The surficial sands and clays on site would appear to meet these criteria. Non-expansive import may be used.

New utility trenches beneath the foundations should be compacted to avoid settlement and sealed to keep external water sources from entering the trenches below the structure.

Backfill materials should be approved by the soil engineer prior to use. All backfill should be placed in lifts not exceeding 8 inches in loose thickness. Each lift should be brought to at least the optimum moisture content and compacted to at least 90 percent relative compaction, in accordance with ASTM Designation D 1557.

Cut slopes

No finished cuts are anticipated for the project. In general, finished permanent cut slopes should be no steeper than 2:1 (2 horizontal to 1 vertical). Fills should be no steeper than 2:1. Where steeper banks are required, retaining walls should be used.

Foundations

Foundations for the new residence, including retaining walls, should consist of continuous spreadtype footings stepped into the hillside. There may also be isolated interior column loads supporting floor joists and beams. These loads should be supported upon conventional isolated spread footing pads as well. As noted, an alternative to spread footings for the foundation system can consist of a matt slab. Recommendations for the matt slab should be prepared by your structural engineer.

Spread Footing Foundations

The design of the foundation system should be performed by the structural engineer. Groundlevel floors can consist of raised wood floors or slab floors.

Any basement and additional foundations for the residence should consist of spread footings supported on stiff clays at design foundation depths indicated on the plans. Perimeter foundations at ground level, as well as interior foundations supporting walls and isolated interior column supports should be on spread footings designed in accordance with the criteria given below.

Isolated column footings should be at least 24 inches wide.

There may be isolated interior column loads from posts supporting floor beams. These loads should be supported upon conventional isolated spread footings excavated into the bedrock as outlined above. An allowable passive pressure of 400 psf should be used against the vertical projection of the footings for earthquake loads.

Foundations so established should be designed for the following maximum allowable bearing pressures.

Table 1

Dead Load	Dead + Live Load	Total Load
2,500 psf	3000 psf	3,500 psf

Dead-plus live loads are defined as "real" loads, including permanently applied live loads, and total loads are defined as "real" loads plus the effects of seismic or wind forces. The weight of foundation concrete extending below grade may be disregarded in sizing computations.

The plans should indicate that the soil engineer should provide observation of the excavations for spread footing foundations prior to the placement of foundation steel and concrete.

Retaining Walls

Retaining walls should be founded on spread footings in accordance with the criteria given previously. For walls supporting level backfill, walls should be designed to resist active earth pressures equivalent to those exerted by a fluid weighing 45 pounds per cubic foot. For backslopes steeper than 3:1, to a maximum of 2:1, the walls should be designed for lateral pressures equivalent to a fluid weighing 55 pounds per cubic foot. These pressures are based upon Rankine coefficients for an active state of stress.

Retaining walls restrained from movement at the top should be designed for "at rest" lateral earth pressures equivalent to a fluid weighing 60 pounds per cubic foot for level backfill conditions, and 75 pounds per cubic foot backslopes steeper than 3:1, to a maximum of 2:1. The walls must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls.

Where an imaginary plane inclined at 1½:1 extends downward from the nearest edge of any foundation through a retaining wall, the portion of the affected wall below the intersection should be designed for an additional horizontal surcharge load, such as for adjacent neighboring foundations. A lateral frictional coefficient of 0.35 for sliding should be used.

Retaining Wall Drainage

Retaining walls should be fully back drained. The back drains should consist of a 4-inch diameter, rigid perforated pipe embedded in drain rock. The pipe should be PVC Schedule 80 or ABS (SDR 35 or better), and the pipe should be sloped to drain to appropriate outlets by gravity. Drain rock should consist of clean, free-draining crushed rock or gravel. The rock should be wrapped in filter fabric such as Mirafi 140N® or equivalent. Alternatively, the drainage blanket may consist of Class 2 "Permeable Material", per Section 68-1.025 of the Caltrans Standard Specifications (1991). Miradrain® geotechnical drainage product can be used on excavations where excavating for a gravel drain behind the wall is prohibitive.

The top of the collector pipe should be at least 8 inches below the lowest adjacent grade and at least 8 inches below the base of the floor slabs where retaining walls abut floor slabs. The crushed rock or gravel should extend to within 1 foot of the surface. The upper 1 foot should be backfilled with compacted soil to exclude surface water. The ground surface behind retaining walls should be sloped to drain.

Retaining Wall Backfill

Only select, non-expansive soils should be used as wall backfill. Wall backfill should be placed in the wedge-shaped zone described by the rear of the wall or wall drainage blanket, a plane extending upward at an inclination not exceeding 1:1 from the heel of the wall, and a finished grade behind the wall. Drainage rock consisting of clean 3/4" rock can be used for backfill without compaction. All backfill should be placed in lifts not exceeding 8 inches in loose thickness. Each lift should be brought to at least the optimum moisture content and compacted to at least 90 percent relative compaction, in accordance with ASTM Designation D 1557. We should observe the foundation excavations prior to the placement of reinforcing steel for concrete. Also, if soil conditions other than those observed at the site are encountered during foundation excavations we should be notified in order to evaluate the possible modifications to our recommendations. The retaining walls should be backfilled with clean 3/4 drain rock. A back drain should be incorporated into the design of the wall to reduce the risk of seepage through the retaining walls.

Floor Structure

Floors in the new residence can consist of standard wood floors or concrete slabs on grade.

Concrete Slabs-on-Grade

If floors in the structure are to be concrete slab on grade the following recommendations apply. Interior slabs-on-grade will be supported on the stiff undisturbed soils, bedrock, or on a minimum of 12 inches of compacted, nonexpansive soil. Minor structural fills may be required to bring the slab subgrades to the proper elevation for the construction of the slabs. In general, fill and backfill materials for slabs and retaining walls should be approved by the soil engineer prior to use. Prior to the construction of the slabs, any disturbed subgrade surface should be compacted to provide a smooth, firm surface for slab support.

Only select, non-expansive soils should be used as fill below slabs. Fill material should have a Plasticity Index of less than 15 and be crushed to a size smaller than 2.5 inches. Clean 3/4-inch drain rock can be used as slab backfill. All fill and backfill should be placed in lifts not exceeding 8 inches in loose thickness. Each lift should be brought to at least the optimum moisture content and compacted to at least 90 percent relative compaction, in accordance with ASTM Designation D 1557-78.

Slab Design

The slabs should be appropriately reinforced according to structural requirements; concentrated loads may require additional reinforcing. Minor movement of the concrete slab with resulting cracking should be expected. Steps to the house from the slab area should be created with an expansion joint between the steps and the house foundations. The recommendations presented above, if properly implemented, should help minimize the magnitude of this cracking. It has been our experience that the installation of wire mesh for slab reinforcement has often not been performed properly during the construction of the slab. As a result, we recommend that steel bar reinforcement be used to reinforce any proposed slabs. Any new exterior slabs should be placed on non-expansive materials which are properly compacted.

Slab Construction and Cracking Control

Slabs, including any new slab, should be at least 5 inches thick and should be reinforced to reduce cracking. Slabs should be provided with construction joints spaced no farther apart than 10 feet on centers (both ways) to control cracking. The subgrade soils should be rolled to produce a dense, uniform, and essentially unyielding surface. The subgrade should not be allowed to dry out prior to the placement of concrete. Concrete is a rigid construction material, and even though reinforced to reduce cracking minor cracks will occur in slabs from temperature expansion and minor settlements of the slab. Slab reinforcement should be designed by the structural engineer.

TABLE 2

Percent Passing by Weight	Sieve Size
1 inch 100 No. 4	0

Vapor Barrier

A moisture vapor barrier membrane should be included below the slabs. The membrane should be placed between the drain rock and the slab and should be covered with 2 inches of damp, clean sand to protect it during construction.

Capillary Break - Slabs should be underlain by a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel graded as follows when tested in accordance with ASTM Designation D 422-73.

Geotechnical Drainage

In general, surface water should be diverted away from slopes and foundations. Roofs should be provided with gutters, and the downspouts should be connected to closed conduits discharging well away from foundations and slopes. Roof downspouts and surface drains must be maintained entirely separate from any foundation drains that may exist. Drainage from retaining walls should be channeled into an appropriate drainage collection facility in accordance with the requirements of the County.

Where any perforated sub-drain pipe connects with the solid discharge drainpipe, the drainrock backfill should be discontinued. A clay plug should be constructed out of relatively impervious soils to direct collected water into the perforated pipe and minimize the potential for water collecting around the solid drainpipe and saturating the adjacent soils. We recommend waterproofing be applied to any proposed retaining walls where applicable. The specification of the type of waterproofing and the observation of its installation should be performed by the architect and/or structural engineer. In addition to the drainage details noted above, the high end and all 90-degree bends of the sub-drain pipe should be connected to a riser that extends to the surface and acts as a cleanout. The number of cleanouts can be reduced by installing "sweep" 90degree bends or pairs of 45-degree bends in succession instead of using "tight" 90-degree bends.

Maintenance

Some nominal maintenance of the drainage facilities should be expected after the initial construction has been completed. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary. Downspout sub-drain pipes should be checked by flushing with a hose once every two years. If blockages develop, the lines should be cleared by a contractor who specializes in such work.

Should ownership of this property change hands, the new owner should be informed of the existence of this report, not adversely change the grading or drainage facilities, and understand the importance of maintaining proper surface drainage.

Construction Observation

On-site geotechnical observation during construction is recommended in order to ensure that the subsurface conditions. Encountered during construction are consistent with those encountered during the investigation to assure that your contractor follows the recommendations in the report and on the approved plans, and to submit a final summary letter to the Contra Costa Building Department as required. If variations in field conditions become apparent, it may be necessary to re-evaluate the recommendations of this report., If we are not retained to provide the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

Supplemental Services

The following are supplemental services we recommend during project development. We recommend that we be retained to review the geotechnical aspects of the project plans and specifications to determine if they are consistent with our recommendations. In addition, we should be retained to observe the geotechnical aspects of the construction, and foundation excavations and to perform appropriate field and laboratory testing. Special geotechnical inspection may be required by the County Building Department.

If, during construction, subsurface conditions different from those encountered in the explorations are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon such notification and review of the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at, or adjacent to, the site, the recommendations made in this report may no longer be valid or appropriate. In such a case, we recommend that we review this report to determine the applicability of the conclusions and recommendations, considering the elapsed time or changed conditions. T h e recommendations made in this report are contingent upon such a review.

These supplemental services are performed on an as-requested basis and are in addition to this geotechnical investigation. Subsequent additional work anticipated, but not included in this proposal, would consist of a plan review of the structural plans for mitigation, and geotechnical construction observation services during the actual construction. We cannot render an opinion for conditions, situations, or stages of construction that we are not retained to observe.

LIMITATIONS

This report has been prepared for the exclusive use of the owner, and their consultants for development of the proposed project described in this report.

Our services have consisted of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no other warranty, either express or implied. Our conclusions and recommendations are based on the information provided to us regarding the proposed construction, the results of our field observation, geologic mapping, and professional judgment. Site conditions and cultural features described in the text of this report are those existing at the time of our field observations and reconnaissance and may not necessarily be the same or comparable at other times.

The scope of our services did not include an environmental assessment or investigation for the presence of absence of wetlands, corrosive soils or groundwater, hazardous or toxic materials in the soil, surface water, groundwater or air, on or below, or around the site. Any statements contained in this report, regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client.

APPENDIX A - REFERENCES

Caltrans, 1991, *Standard Specifications*, State of California Department of Transportation.

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U.S. Geological Survey, 1999, "Earthquake Probabilities in the San Francisco Bay Region: 2000 to 2030 – A Summary of Findings," Open File Report 99-517.

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Wagner, D. L., Bortugno, E. J., and McJunkin, R. D., 1990, Geologic Map of the Santa Rosa Quadrangle, California Division of Mines and Geology, scale 1:250,000.

Working Group on California Earthquake Probabilities, (1999), *Earthquake Probabilities in the San Francisco Bay Region: 2000 to 2030 –*

A Summary of Findings, U.S. Geological Survey, Open File Report 99-517, Online Version 1.0

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GEOTECHNICAL INVESTIGATION PROPOSED NEW RESIDENCE 1518 BARTH AVENUE RICHMOND, CALIFORNIA

CE&G Project No. 070780
13 May 2008

Prepared for:

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13 May 2008

Rene Lucchini
801 Yuba Street
Richmond, California 94805

RE: Subsurface Exploration
Proposed New Residence
1518 Barth Avenue
Richmond, California

Dear Ms. Lucchini:

In accordance with our contract, we have completed our subsurface exploration and report for the proposed new residence to be constructed on your property located at 1518 Barth Avenue in Richmond, California (Figure 1). The purposes of this project were to develop information regarding the surface and subsurface soil conditions near the proposed improvements and to provide geotechnical engineering recommendations for the proposed project.

1.0 SCOPE OF WORK

The scope of work for this project has included but was not limited to the following tasks:

- review of published soil and geologic maps of the area;
- review of engineering reports by others provided by the client;
- drilling and sampling of four exploratory borings;
- evaluation of the materials encountered in the borings;
- laboratory testing of selected samples recovered from the borings;
- engineering analyses;
- development of geotechnical design parameters for the project; and
- preparation of this summary report of findings and recommendations.

This report presents the results of the field exploration, laboratory testing program, data analysis, and conclusions and recommendations pertaining to the geotechnical engineering aspects of the design and construction of the proposed project. Evaluation or identification of the potential presence of hazardous materials at the site was not requested and is beyond the authorized scope of this project.

**SUBSURFACE EXPLORATION FOR PROPOSED NEW RESIDENCE
1518 BARTH AVENUE
RICHMOND, CALIFORNIA**

TABLE OF CONTENTS

2.0 SITE DESCRIPTION AND HISTORY

2.1 Site Description

For the purposes of this report, the reader should assume that the property is located on the north side of Barth Avenue between Harbor View Avenue and Capitol Hill Avenue (Figure 1). Barth Avenue generally trends east-west in front of the property. The property consists of a 4,685 square foot, pentagon-shaped, undeveloped lot that slopes down to the north-northwest at an estimated average inclination of approximately 2.2H:1V.

Past grading on the property appears to have consisted of conventional cut-and-fill grading associated with the construction of Barth Avenue and the parcels along the downslope side of the roadway. Such grading is evidenced by the uniform nature of the slope and the presence of a mid-slope bench. In addition, a landslide repair was completed on the property in 1984 (see below).

2.2 Project Description

We understand that consideration is being given to developing the site for a single family residence. It is anticipated that the residence will be of modular home construction and supported on a pier-and-grade beam type foundation.

2.3 Site History

We reviewed the following geotechnical reports prepared for the subject property by other consultants:

2.3.1 "Geotechnical Engineering Services, Barth Avenue Slide, Richmond, California," prepared by Harding Lawson Associates (HLA), dated 4 April 1984.

According to this report, a landslide developed on the property in March 1983. The landslide was characterized as a wedge-type failure that formed in residual soil and overlying fill. As part of their investigation, HLA drilled and sampled two exploratory borings. HLA did not determine the location of a discrete slide plane from the boring data. The original repair for the landslide designed by HLA consisted of a buttress fill across the lower half of the slide mass. The upper portion of the slide was to remain intact and stabilized in-place by the buttress fill. The HLA report indicates that the actual repair was modified to include a smaller buttress fill at the toe, with benched levels of compacted fill up to the headscarp of the slide below Barth Avenue. Two subdrains were also placed beneath the fill. The repair was completed by the end of August 1983.

2.3.2 "Slope Stability Investigation Report for Barth Avenue, City of Richmond, California," prepared by Questa Engineering Corporation (QEC), dated 7 November 2001

The QEC report documented cracking distress in the pavement of Barth Avenue directly above the subject property following the heavy El Nino rains in 1997 to 1999. The distress was attributed to slope movement which occurred after the 1983 HLA repairs to the hillside. QEC drilled and sampled four borings in the Barth Avenue pavement during their investigation. They concluded that the "observed deformation is most likely due to settlement and rotational slumping of low density and poorly compacted fill underlying Barth Avenue and movement due to saturation of the fill soil during prolonged heavy winters." QEC further concluded that the movement may also be "partially due to settlement of the fill soil placed during the previous landslide repair and the underlying residual soil left in place in the upper portion of the repair." QEC recommended that the soil underlying the affected portion of the Barth Avenue be removed and replaced as an engineered fill, with a subdrain placed along the contact between the new fill and underlying fill and/or residual soil. The recommended repairs to Barth Avenue had not been implemented at the time of this report.

2.3.3 "Report - Geotechnical Investigation, Planned Residence on Barth Avenue, San Pablo, California," prepared by Earth Mechanics Consulting Engineers, dated 25 May 2005.

This report provided geotechnical recommendations for drilled pier and spread footing foundation systems, slabs-on-grade, retaining walls, and drainage. Earth Mechanics Consulting Engineers drilled and sampled one boring as part of their investigation. The report noted evidence of slope movement at the site, and concluded the following:

> *It is our opinion that further studies of the potential limits of landsliding within the site and the potential rate of movement should be conducted prior to developing the subject site. We recommend that the work be performed by a certified engineering geologist and should include a field reconnaissance and review of historical aerial photographs. Depending on the engineering geologist's findings, it may be necessary to install slope inclinometers and monitor them over a period of years to determine the rate of landslide movement. The certified engineering geologist may determine based on the collected data that the subject lot may or may not be suitable for support of residential construction in its current condition.*

3.0 SOIL AND GEOLOGIC CONDITIONS

3.1 Surficial Soils

The surficial soil in the vicinity of the property has been mapped by the USDA Soil Conservation Service as clay loam belonging to the Los Osos Series on 30 to 50 percent slopes (SCS, 1977). Soils of this series form on soft, fine-grained sandstone and shale bedrock materials and are found on upland areas. The Los Osos Series classifies as a low plasticity clay (CL) which a high shrink-swell potential. Runoff is medium to rapid, and the hazard of erosion is moderate to high where the soil is bare.

3.2 Bedrock Geology

According to published geologic maps of this area (Dibblee, 2005 and Crane, 1995), the area encompassing the subject property is underlain by Pliocene age bedrock materials (Figure 2). The bedrock is described as consisting of interbedded pebble conglomerate, sandstone, and claystone of the Orinda Formation. The Crane mapping shows that the bedrock locally dips 85 degrees to the northeast. This mapping is consistent with the materials encountered in our exploratory borings.

3.3 Landslides

The U.S. Geological Survey has developed regional landslides features maps for a significant portion of the greater San Francisco Bay Area. The landslide features map for this area does not show any landslide features on the property (Nilsen, 1975). According to a geological and geophysical seismic safety report of this area (CDMG, 1973), the area encompassing the subject property is classified as "Urban Area C: Areas underlain by incompetent formations, but having no well-defined landslide features." A landslide features map contained in the report also does not show any mapped slides on or adjacent to the property.

3.4 Liquefaction

According to the seismic safety report referenced above (CDMG, 1973), the area encompassing the subject property is classified as a "Zone III; Liquefaction Potential Probably Absent."

3.5 Seismicity

The project site is located within the greater San Francisco Bay Area which is recognized as one of the more seismically active regions of California. The seismic activity of the greater Bay Area results from the complex movements along the transform boundary between the Pacific Plate and the North American Plate. Studies have shown that the Pacific Plate is slowly moving to the northwest relative to the more stable North American Plate at an average rate of about 49 mm/yr (Page, 1992). The differential movements between the two crustal plates have caused the formation of a series of active fault systems within the transform boundary. The transform boundary between the two plates extends across a broad zone of the North American Plate within which right lateral strike slip faulting predominates. In this broad zone, the San Andreas fault accommodates less than half of the average total relative plate motion. Much of the remainder in the greater San Francisco Bay Area is distributed across the Calaveras, Hayward, Concord- Greenvalley, Greenville, West Napa, and Rodgers Creek Fault systems.

The site is not located within an Earthquake Fault Zone for active[1] faults as designated by the State Geologist (CGS, 1992). However, the active Hayward fault system has been mapped approximately 1000 feet (0.3 km) west of the subject property (CDMG, 1998). Other nearby active faults systems which could induce strong ground shaking at the site include the Calaveras, Greenville, Concord-Green Valley, West Napa, Rodgers Creek, and San Andreas faults. These active faults and their distances from the project site are presented in Table A (CDMG, 1998).

[1]Earthquake Fault Zones (EFZ) have been delineated by the California Geological Survey (formerly the California Division of Mines and Geology) around all known active faults throughout the State. A fault is defined as being active if it exhibits evidence of surface rupture due to faulting within the last 11,000 years (Holocene time). The land within an EFZ is believed to have an elevated potential for experiencing surface rupture due to faulting.

A large magnitude earthquake on any of these fault systems has the potential to cause significant ground shaking at the site. The intensity of ground shaking that is likely to occur at the property will generally be dependent upon the magnitude of the earthquake and the distance to the epicenter. In general, the greater the distance to the epicenter, the lesser the intensity of the ground shaking that is anticipated to occur at the site.

TABLE A - DISTANCES TO KNOWN ACTIVE FAULTS

Fault	Fault Type	Distance To Fault
Hayward	Type A	0.3 km southwest
Calaveras	Type B	25 km southeast
Greenville	Type B	30 km east
Concord-Green Valley	Type B	21 km east
West Napa	Type B	22 km northeast
Rodgers Creek	Type A	12 km northeast
San Andreas	Type A	27 km west

4.0 SUBSURFACE EXPLORATION

The subsurface conditions in the vicinity of the proposed new residence were explored by drilling and sampling four exploratory borings. The exploratory drilling was performed on 12 July 2007, using a "Minuteman" portable-type drilling rig equipped with 4-inch diameter solid stem, continuous flight augers. The borings were drilled to depths between 14.5 and 26.5 feet below the existing grade. Drive samples of the site materials were obtained using California Modified (CM) and Standard Penetration (SPT) samplers. The samplers were advanced using a 140-pound hammer with a free fall drop of 30 inches. The number of blows required to drive each of the samplers 6 inches was recorded and the uncorrected blow counts are shown on the boring logs in Appendix A. Samples obtained from the borings were retained for possible laboratory testing and analyses. The approximate locations of the borings are shown on Figures 3 and 4.

The materials encountered in the borings were continuously logged in the field by a geologist from our office. The soils were visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) (Figure 5) and the Bedrock Characteristics Chart (Figure 6). The logs of the borings are included in Appendix A.

We encountered between 4.5 feet and 14 feet of artificial fill in the borings. The artificial fill consisted of a variable mixture of silts, clays, and gravels. The artificial fill is underlain by clayey silt, sandy silt, and silty clay native/residual soil and landslide debris. These soil materials are underlain by siltstone and sandstone bedrock to the depths explored. Free groundwater was not encountered in the borings.

5.0 LABORATORY TESTING

Laboratory testing was performed to obtain information concerning the physical properties of the samples recovered from the exploratory borings. Tests were performed in general conformance with applicable ASTM standards. Tests performed included determination of moisture content, dry unit weight, and Atterberg Limits. The results of the laboratory tests are summarized on the boring logs in Appendix A.

6.0 CONCLUSIONS

6.1 General

Based on the results of our work, it is our opinion that there has been ongoing subsurface movement at the site since the 1983 slope repair. This movement can be mitigated by either utilizing an earthwork-type repair or installing a deep pier foundation system for the new residence that effectively pins the upper portion of the slide mass.

6.2 Geotechnical Considerations

6.2.1 Soil Conditions

The results of our limited subsurface exploration indicate that the surficial soils in the vicinity of the proposed improvements consist of gravel, silt, and clay soils. Our laboratory soil testing indicates that the clayey soils have a high expansion potential. The expansion potential of these soils must be taken into consideration when designing the foundations systems for the proposed improvements and shallow founded appurtenant structures.

6.2.2 Landsliding

The landslide that occurred on the property in 1983 was partially stabilized in 1984 utilizing a buttress fill at the toe, with benched levels of compacted fill up to the headscarp below Barth Avenue, and two subdrains. This repair did not remove and replace the entire slide mass on the subject property as an under drained engineered fill. Most of the fill at the property is underlain at depth by landslide debris. Between 1997 and 1999 additional movement was documented by Questa Engineering Corporation in the street above the project site. It appears that this more recent movement is likely the result of a combination of continued movement of the slide mass and settlement of the fill soils placed during the 1984 partial stabilization. Our interpretation of the subsurface geometry is presented in Figure 4. In order to develop the property with a single family dwelling, the identified subsurface movement will need to be abated. Recommendations for slope stabilization measures are presented in Section 7.0.

6.2.3 Groundwater

Groundwater was not encountered in the four exploratory borings. However, groundwater levels can fluctuate seasonally and/or over a period of years. Note that the exploratory boring sampled by

Earth Mechanics Consulting Engineers in 2005 encountered a perched water table at a depth of 3.5 feet.

Therefore, it is possible the adverse groundwater conditions could be encountered during construction. The contractor should be prepared for this possibility.

6.3 Seismic Hazards

6.3.1 Fault Rupture

The site is not located within an Earthquake Fault Zone for active faults as defined by the State Geologist. The nearest mapped active fault (Hayward Fault) is located approximately 0.3 kilometer west of the site. Therefore, it is our opinion that the potential for surface rupture due to primary faulting at the site is low.

6.3.2 Seismically-Induced Ground Shaking

Due to the proximity of the site to numerous active fault systems which traverse the greater San Francisco Bay Area, it is likely that the property will be subjected to the effects of a major earthquake during the design life of the proposed improvements. The effects are likely to consist of significant ground accelerations. These ground movements may cause damage to the proposed improvements. This potential hazard should be taken into consideration when designing the structural systems of the proposed structures and associated appurtenances.

6.3.3 Liquefaction

Due to the shallow depth of the soils, the high clay and silt content, and the absence of elevated groundwater conditions underlying the site, it is our opinion that there is no potential for liquefaction.

6.3.4 Lateral Spreading

Lateral spreading is a type of ground instability that results in ground displacements that occur when liquefaction of a soil layer causes insufficient strength for lateral stability. This phenomenon occurs when either the ground surface or the soil layer subject to liquefaction is sloped, or when there is an open slope face or stream channel adjacent to a potentially liquefiable soil layer. These conditions are not known to be present at the site. It is our opinion that there is no potential for lateral spreading to occur at the site.

6.3.5 Seismically-Induced Subsidence

Seismically-induced ground shaking can cause vertical subsidence of specific types of soils. Seismically related settlement generally results from the densification of loose sands and sandy silts due to vibrations or liquefaction. Due to the nature of the soils encountered during our exploratory borings, it is our opinion that the potential for seismically induced subsidence is very low.

6.3.6 Ground Lurching

Ground lurching is a phenomenon whereby strong seismic shaking causes cracking and deformation of the ground surface in areas underlain by soft weak soils. The cracking and deformation are the result of the disruption of the passing earthquake waves. Based on the known site soil conditions and our analysis, it is our opinion that the potential for ground lurching to occur at the site is moderate unless the recommended stabilization measures are constructed.

7.0 RECOMMENDATIONS

7.1 General

We recommend that the foundation system for the new residence consist of pier supported grade beam system, tie beams, and retaining walls. The foundation system should be specifically designed and constructed to resist future movement of the landslide debris which underlies the site. This can be accomplished by either 1) removing the landslide and replacing it with a drained engineered fill, or 2) constructing a foundation system for the new residence which serves the dual purpose of stabilizing the underlying landslide debris and supporting the house. The specific system utilized at the site should be selected by considering cost, the potential for affecting adjacent parcels, and potential future maintenance costs. These landslide mitigation options are discussed in more detail below.

7.2 Option 1: Remedial Grading/Landslide Removal

The most straightforward way to abate the landslide debris underlying the site would be to remove it entirely and regrade the hillside with a well-drained engineered fill. It is anticipated that the existing fill and underlying landslide debris could be reused as engineered fill. It would also be necessary to install temporary shoring piers to provide lateral support to Barth Avenue prior to the start of the grading. In our opinion, it is unlikely that this option would prove feasible considering the proximity of the adjacent properties and Barth Avenue, and lack of area to temporarily stockpile materials on site. As a result, we have not developed specific recommendations for this option.

7.3 Option 2: Landslide Stabilization Foundation

The second option for abating the landslide debris which underlies the site is to design and construct a foundation system for the new house to stabilizing it in place. This would be accomplished by designing the new foundation system to extend at least 15 feet below the interpolated depth of active movement identified in this report and using grade beams with tiebacks to resist the lateral forces exerted by the landslide debris. Although the design and construction of this option would require relatively minimal grading, the loads required to retain the landslide debris in-place would be significant and, therefore, expensive.

7.3.1 Design Criteria

It is recommended that the residence for the new foundation system consist of a perimeter and interior grade beams, tie beams, and retaining walls supported by drilled piers with permanent tiebacks.

The design parameters for the foundation systems are as follows.

Minimum pier diameter:	30 inches for piers supporting grade beams with tiebacks
	18 inches supporting all other grade beams and tie beams retaining less than 3 feet of soil
	24 inches supporting all grade beams/retaining walls retaining more than 3 feet of soil
Allowable skin friction:	600 psf in firm residual soil and/or competent bedrock below the slide plane
Pier spacing:	Minimum three diameters on center
Minimum reinforcing steel:	30-inch diameter piers: ten #10 bars w/ #5 stirrups at 6 inches on center (see Appendix B)
	24-inch diameter piers: eight #8 bars w/ #4 stirrups at 6 inches on center
	18-inch diameter piers: six #6 bars with #4 stirrups at 6 inches on center

Final steel requirements and pier depths shall be determined by the project Structural Engineer

Minimum pier depths:	For grade beams with tiebacks: 40 feet or as otherwise required by the project Structural Engineer

For all other foundation elements:

Extend pier to at least 15 feet below the slide plane or as otherwise required by the Project Structural Engineer

Lateral soil loads:	Grade beams with tiebacks: The lowermost grade beam or a separate external grade beam with tiebacks should be designed to resist lateral pressures exerted from the in place landslide above the house.
	The piers and tieback system should be designed to resist a lateral

load equal to an equivalent fluid pressure of 85 psf/ft acting from the original ground surface (or as regraded) to a depth of 23 feet below the original ground surface. To resist the lateral loading, a passive equivalent fluid pressure of 500 psf/ft acting over two pier diameters can be assumed for the soil and bedrock below a depth of 23 feet. The passive resistance of the upper 23 feet soil should be neglected in the structural calculations. Alternatively, the piers and tiebacks can be designed by determining loading requirements based on the results of slope stability analyses as shown in the preliminary stabilization design analyses included in Appendix B.

All other piers and grade beams:
All other piers and grade beams should be designed to resist a lateral load equal to an equivalent fluid pressure of 50 psf/ft acting from the final ground surface to a depth of 10 feet. To resist the lateral loading, a passive equivalent fluid pressure of 500 psf/ft acting over two pier diameters can be assumed for the soil and bedrock below a depth of 8 feet. The passive resistance of the upper 8 feet soil should be neglected in the structural calculations.

Tieback design parameters: Additional lateral resistance forces can be provided by the use of permanent double corrosion protected tiebacks which are structurally connected to the piers/grade beams. Preliminary tieback design analyses included in Appendix B indicate that for tiebacks spaced at 8 feet on center at a depth of 6 feet below grade and inclined at 20 degrees, a design lock of load of 155 kips per tieback will be required.

Tiebacks should be designed by the contractor assuming an allowable adhesion of 1500 psf. Actual forces developed by the tiebacks should be determined by proof testing of the tiebacks. Tiebacks should be designed in conformance with current Caltrans methodology or equivalent.

Final design pier depths and spacing and tieback requirements should be based on structural design considerations. All perimeter piers and interior piers should be structurally connected with grade beams and tie beams. Care should be taken to ensure that the interior grade beams do not adversely impact the cross flow ventilation of the underfloor areas. The grade beams and tie beams should be designed by the project Structural Engineer. Grade beam and tie beam dimensions and steel reinforcing requirements should be determined based on the design structural loads. We recommend that all tiebacks be required to be contractor-designed and that the design calculations and plans be provided to our office for review.

7.3.2 Piers

The bottoms of the all pier holes should be dry and free of loose cuttings and debris prior to installation of the reinforcing steel and concrete. This shall be done to the satisfaction of the engineer or geologist from Cal Engineering & Geology, Inc. observing the drilling operations. The concrete should be placed carefully in the pier holes so that over pouring of the piers (mushrooming at the top) does not occur and the concrete does not have a free fall drop in excess of 4 feet.

Free groundwater was not encountered during the exploratory drilling at the site. However, a perched groundwater condition was reported in the borings drilled in the street by QEC. As groundwater levels can fluctuate seasonally and over a period of years. It is therefore possible that adverse groundwater conditions will be encountered during the drilling of the foundation piers. The contractor must be prepared to drill and place the steel and concrete for the foundation piers on the same day, should adverse groundwater conditions be encountered during construction. Under no circumstances shall water be allowed to remain in a drilled pier hole overnight. Should this occur, it will be necessary for the contractor to enlarge the hole to a wider diameter and a greater depth to the satisfaction of the engineer or geologist from our office who is observing the drilling operation.

7.3.3 Tiebacks

All tiebacks should be constructed and tested in accordance with current Caltrans guidelines or equivalent methods if approved by our office.

7.3.4 Foundation Retaining Walls

It is our understanding foundation retaining walls may be required as part of the foundation system for the new residence. It is recommended that all foundation retaining walls be designed for at-rest conditions (restrained walls). Based on the information obtained from the field work, it is our opinion that these retaining walls should be supported drilled piers. The retaining wall foundations can be designed using the parameters provided in the Design Criteria section of this report.

All foundation retaining walls should be waterproofed to prevent moisture migration through the concrete walls. The waterproofing system should be designed by the project Architect and/or Structural Engineer and reviewed by our office.

For level conditions above the walls, a restrained at rest equivalent fluid pressure of 70 pcf should be assumed to be acting over the full height of the wall. For back slopes up to 2:1V (horizontal to vertical) above the wall, an active equivalent fluid pressure of 90 pcf should be used. For back slopes between level and 2H:1V, the active equivalent fluid pressure should be interpolated between 70 and 90 pcf.

The above active pressures assume fully drained conditions behind the retaining walls. Therefore, the retaining walls should be provided with a full height back wall drainage consisting of a 12 inch wide layer of Caltrans Class 2 permeable drain material or a 12 inch wide layer of ¾ inch crushed rock with a 6 ounce per square yard non-woven filter fabric. A perforated ABS SDR 35 plastic pipe should be placed at the heel of the wall. The pipe should be placed with the perforations down and should be installed with a cross gradient of not less than 2%.

All retaining wall backfill materials should be compacted to minimum relative compaction of 90% as determined by the ASTM D-1557 test procedure at a moisture content of approximately of 3% above optimum. Soils which do not meet the required relative compaction should be scarified, moisture conditioned, and reworked until the required relative density is attained.

The recommendations for fill placement provided above are based on the assumption that adequate surface and subsurface drainage will be incorporated into the project and that water will not be allowed to build-up within or behind the fills. Therefore, all fills in excess of 5 feet thick should be well-drained. Subsurface drains for the fills should be at least 12 inches wide and extend a minimum of 3 feet up the back cut of the excavation. The subdrain should consist of a perforated ABS SDR-35 pipe (with perforations down), surrounded with Class 2 permeable material. All excavations, trenches, and pipes should have a minimum fall of 2%.

7.3.5 Garage Slab

It is anticipated that the garage floor will be constructed on grade as a concrete slab-on-grade floor. The slab-on-grade floor should be structurally independent from the perimeter foundation elements. We recommend that the concrete slab floor be a minimum of 5 inches thick and reinforced with a minimum of #3 bars at 12-inches on center in both directions or #4 bars in both directions at 18-inch centers. The steel reinforcement shall be placed in the middle of the slab and should be held in place by dobie blocks or other suitable means. Actual dimensions and reinforcement should be determined by the project Structural Engineer. Even with the steel reinforcement, it should be recognized that some cracking of the slab will likely occur.

It is recommended that the floor slab for the garage be supported by on a minimum of 6 inches of non-expansive, granular fill (Caltrans Class 2 aggregate base rock or equivalent), compacted to a minimum relative compaction of 95% as determined by the ASTM D-1557 test procedure. We recommend that the upper 6 inches of the soil below the base course material be removed and re-compacted as engineered fill. The soils should be compacted to a minium relative density of 90% at a moisture content of at least 3% over optimum as determined by the ASTM D-1557 test procedure.

If it is desirable to reduce the potential for moisture migration through the garage floor slab, then consideration should be given to the installation of a moisture vapor barrier. The vapor barrier should consist of a double layer of 10 mil polyethylene membrane. The vapor barrier should be underlain by a 3 inch layer of pea gravel placed after proof rolling of the base course. The membrane should be covered with a 2 inch thick, uniform layer of sand to protect the membrane during construction.

7.4 Appurtenant Slabs

To reduce the potential for cracking of the appurtenant concrete slabs (such as walkways and patios), we recommend that slabs be a minimum of 5 inches thick. The slabs should include minimum reinforcement of #3 bars in both directions at 12-inch centers or #4 bars in both directions at 18-inch centers. The steel should be placed in the middle of the slab and should be held in place by dobie blocks or other suitable means. Actual dimensions and reinforcement should be determined by the project Structural Engineer. The concrete slabs should be underlain by at least 4 inches of Class 2 aggregate base rock compacted to a minimum relative compaction of 95%. Even with the steel reinforcement and base rock, it should be recognized that some cracking and heaving of the slabs will likely occur. All exterior slabs should be isolated from the building foundations.

All exterior concrete slabs-on-grade located within 10 feet of a downslope will be subject to the effects of long-term and on-going downslope creep of the near surface soils. Downslope soil creep of the near surface soils has the potential to cause sloping and cracking of the concrete slabs. If it desirable to minimize the potential impacts of soil creep on these structures, then the slabs should be supported on drilled piers. The piers can be designed using the parameters provided above.

7.5 Site Grading

It is anticipated that conventional cut and fill grading will be required to establish the desired grades.

It is recommended that all fill placed at the site be engineered and compacted to the following specifications.

7.5.1 Fill Placement

Prior to commencement of the grading operation, the site should be cleared and grubbed of existing vegetation. All existing structures and debris should be removed from the site, including but not limited to: foundation systems, leach fields, septic tank, basements, groundwater well, buried pipes etc. Prior to placement of engineered fill, all loose soil and vegetation should be removed from the areas to receive fill. All fills should be keyed at least 2 feet into competent soil and/or bedrock materials.

All fill shall be placed as engineered fill. All fills should be compacted to a minimum relative compaction of 90 percent as determined by the ASTM D-1557 (latest revision) test procedure at a moisture content of approximately 3 percent above optimum. Fill materials shall be spread evenly and compacted in uniform lifts not exceeding 8 inches in uncompacted thickness. Fill materials which do not meet the specified relative compaction shall be ripped, moisture conditioned, and re-compacted until the required relative compaction and moisture content are obtained.

7.5.2 Imported Fill Materials

All imported fill must be reviewed and approved by the geotechnical engineer prior to importation to the site. A minimum of three to four days will be required to evaluate and test the suitability of all proposed imported materials. All imported materials should meet the following criteria.

The imported materials shall be non-expansive and have a Plasticity Index less than 15 percent and a Liquid Limit of 30 percent or less. The imported material shall be free of organic debris or contaminated materials.

7.5.3 Fill Slopes

Permanent cut and fill slopes less than 10 feet tall can be made at a maximum gradient of 2.1 H:1V or flatter.

7.5.4 Fill Drainage

The fills in excess of 5 vertical feet in thickness shall be bench keyed and under drained. Subsurface drains should be at least 12 inches wide and extend a minimum of 3 feet up the back cut. The subdrain should consist of perforated ABS SDR-35 pipe (with perforations down), surrounded with Caltrans Class 2 permeable material. All excavations, trenches, and pipes should have a minimum fall of 2 percent. The subdrains should discharge into an appropriate erosion resistant drainage facility.

7.6 Surface Drainage

The areas adjacent to the proposed residence should be positively sloped away from the building to provide for rapid removal of surface water runoff. Ponding of water in the underfloor area or seepage toward foundation systems at any time during or after construction should be prevented. To reduce the potential for ponding of water adjacent to the foundation system, we recommend the following be included in the project plans.

- At a minimum, we recommend that at least 8 inches of soil be placed and compacted on the outside of the grade beams and sloped away from the foundations at a gradient of 5 percent for a distance of at least 5 feet to provide for rapid removal of surface water runoff.

- Finished grades within 5 feet of the structure should slope away from the house and garage at a minimum gradient of 5% to allow surface water to drain positively away from the structures.

- All storm water from roof downspouts should be collected in a solid pipe and outlet into an appropriate discharge facility.

- Planted areas should be avoided immediately adjacent to the buildings. If planting adjacent to the building is desired, the use of plants that require very little moisture is recommended. Irrigation of landscape areas should be limited strictly to that necessary for plant growth. Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 5 feet from walls or under the structures.

If it is desirable to reduce the potential for near surface water entering into the underfloor area, then a subdrain should be installed around the exterior of the residence. The subdrain should consist of 4 inch diameter perforated ABS SDR 35 pipe encapsulated in 3/4 inch clean crushed rock wrapped in a non-woven geotextile. The subdrain trench should extend at least 12 inches below the bottom of the perimeter grade beams. The drain pipe should be connected to a solid wall pipe which discharges to an erosion resistant discharge point located away from the building.

7.7 Seismic Design Parameters

Due to the proximity of the site to numerous active fault systems, it is likely that the property will be subjected to the effects of a major earthquake during the design life of the proposed improvements. The effects are likely to consist of significant ground accelerations. These ground type movements may cause damage to the proposed improvements. We therefore recommend that, at a minimum, the structural systems for the new residence be designed in accordance with the requirements of Chapter 16 of the 2007 California Building Code for Site Class B and a Seismic Source Type B at a distance of 0.3 kilometer from the source. The California Building Code seismic design parameters for Site Class B are included in Table B below. These parameters were determined by using the computer program titled Java Earthquake Ground Motion Parameter Calculator, version 5.0.8, which was downloaded from the U.S. Geological Survey Website (http://earthquake.usgs.gov/research/hazmaps/design/).

TABLE B - California Building Code Seismic Design Parameters - Site Class B

Item	Design Value	Source
Site Class Definition	B	Table 1613.5.2
0.2 Second Spectral Response Acceleration, S_s	2.033	Figure 1613.5(3)
1.0 Second Spectral Response Acceleration, S_1	0.795	Figure 1613.5(4)
Values of Site Coefficient, F_a	1.0	Table 1613.5.3(1)
Value of Site Coefficient, F_v	1.0	Table 1613.5.3(2)
Designed Spectral Response Acceleration for Short Periods, S_{DS}	1.356	Equation 16-39 (S_{DS}=2/3(F_a S_s))
Designed Spectral Response Acceleration for 1-Sec Periods, S_{D1}	0.530	Equation 16-40 (S_{DS}=2/3(F_v S_1))

* Hayward Fault is located 0.3 kilometer west of the project site.

7.8 Excavations and Trenches

All excavations made during development of the site should be backfilled with engineered fill. This includes excavations created during the installation of the utility lines, the sewer lines, etc. The engineered fill should be compacted to a minimum relative compaction of 90 percent as determined by the ASTM D-1557 test procedure. It is anticipated that the on-site soil and bedrock materials will be suitable for use as trench backfill. Furthermore, no loose or uncontrolled backfilling of depressions resulting from stripping or tree removal should be permitted.

It is recommended that an impervious seal be created wherever a trench passes beneath the perimeter foundation of a house. The impermeable seal should be designed to prevent near surface water from reaching the exterior areas of the property from flowing beneath the house along the utility trench. The impermeable seal should extend for a distance of at least 3 feet on both sides of the perimeter foundation.

8.0 LIMITATIONS

The conclusions and recommendations of this report are based upon information provided to us regarding the proposed improvements, subsurface conditions encountered at the boring locations, our geologic reconnaissance, the results of the laboratory testing program, and professional judgement. We have employed accepted geotechnical engineering and engineering geologic procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering and engineering geologic principles and practices. This standard is in lieu of all warranties, either expressed or implied.

It is the owner's responsibility ensure that the recommendations contained in this report are brought to the attention of the architect, engineers, and contractors working on the project. Furthermore, it is the owner's responsibility to make sure that these recommendations are carried out during the design and construction phases of the project.

The locations of the borings were determined by taping from established site features and other points of reference and are considered to be approximate only. Site conditions described in the text are those existing at the time of our last field exploration and reconnaissance in July 2007 and are not necessarily representative of the site conditions at other times or locations.

Unanticipated soil conditions are frequently encountered during construction and cannot be fully determined by drilling and sampling a limited number of exploratory borings. Additional expenditures may be required during the construction phases of the project as conditions vary. It is recommended that a contingency fund be established to cover potential adverse soil and groundwater conditions which may be encountered during site development. If it is found during construction that subsurface conditions differ from those described on the borings logs, then the conclusions and recommendations in this report shall be considered invalid, unless the changes are reviewed and the conclusions and recommendations modified and approved in writing by Cal Engineering & Geology, Inc.

The findings of this report should be considered valid for period of three years unless the conditions of the site change. After a period of three years, we should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

Cal Engineering & Geology, Inc. should be accorded the opportunity to review the final plans and specifications to determine if the recommendations of this report have been implemented in those documents.

Field observation and testing services are essential parts of the proposed project. It is important that Cal Engineering & Geology, Inc. be retained to observe the earthwork, foundation drilling and excavation, and other relevant construction operations. The recommendations of this report are contingent upon this stipulation.

The evaluation or identification of the potential presence of hazardous materials at the site was not requested and is beyond the scope of this project.

If you have questions regarding this report, or if we may be of further service, please contact us.

Yours truly,

CAL ENGINEERING & GEOLOGY, INC.

Dave Buscheck, P.E.
Senior Engineer

Mitchell Wolfe, P.G., E.G.
Principal Geologist

Phillip Gregory, P.E., G.E.
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CIVIL-GEOTECHNICAL
STATE OF CALIFORNIA

9.0 REFERENCES

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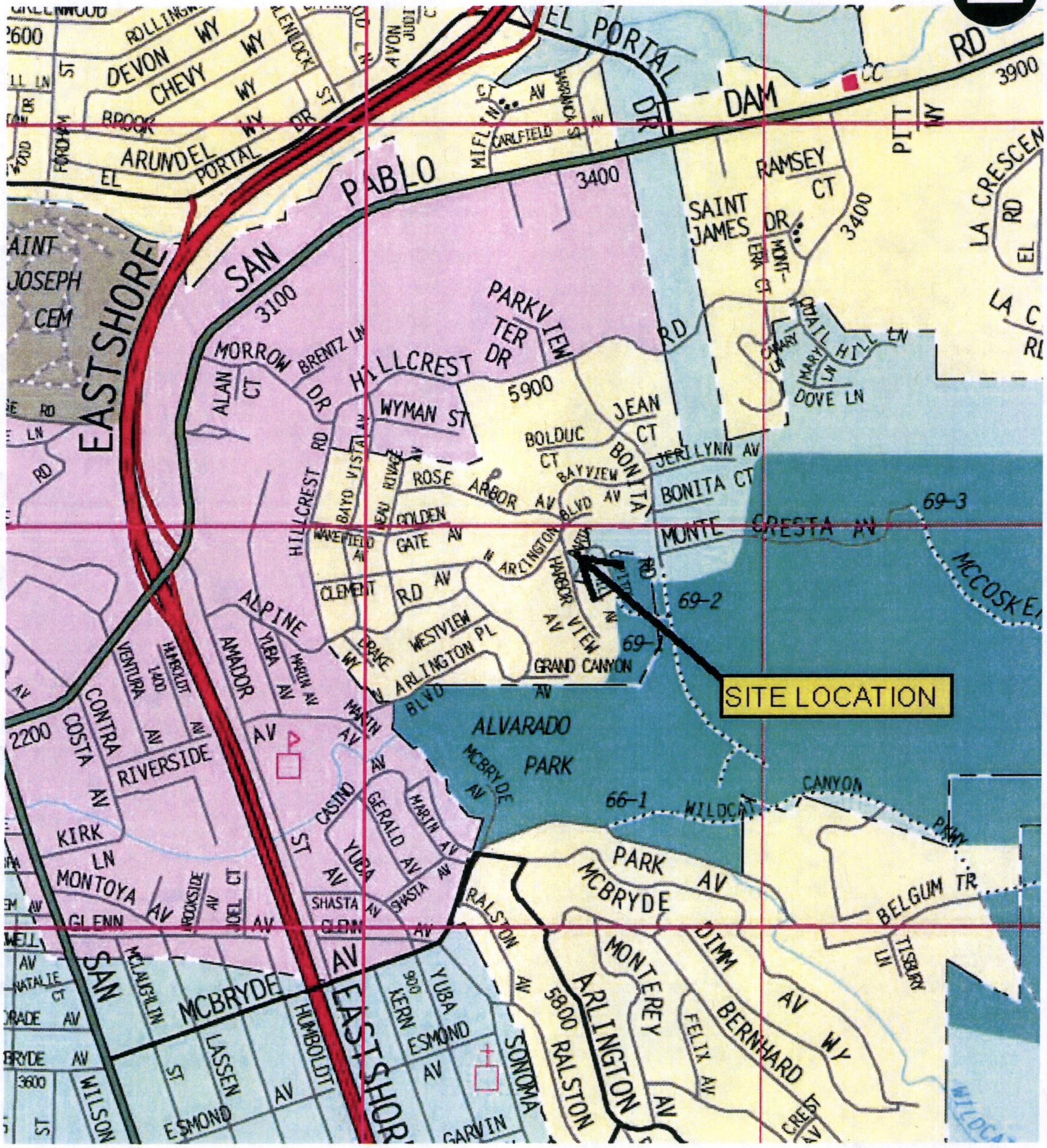


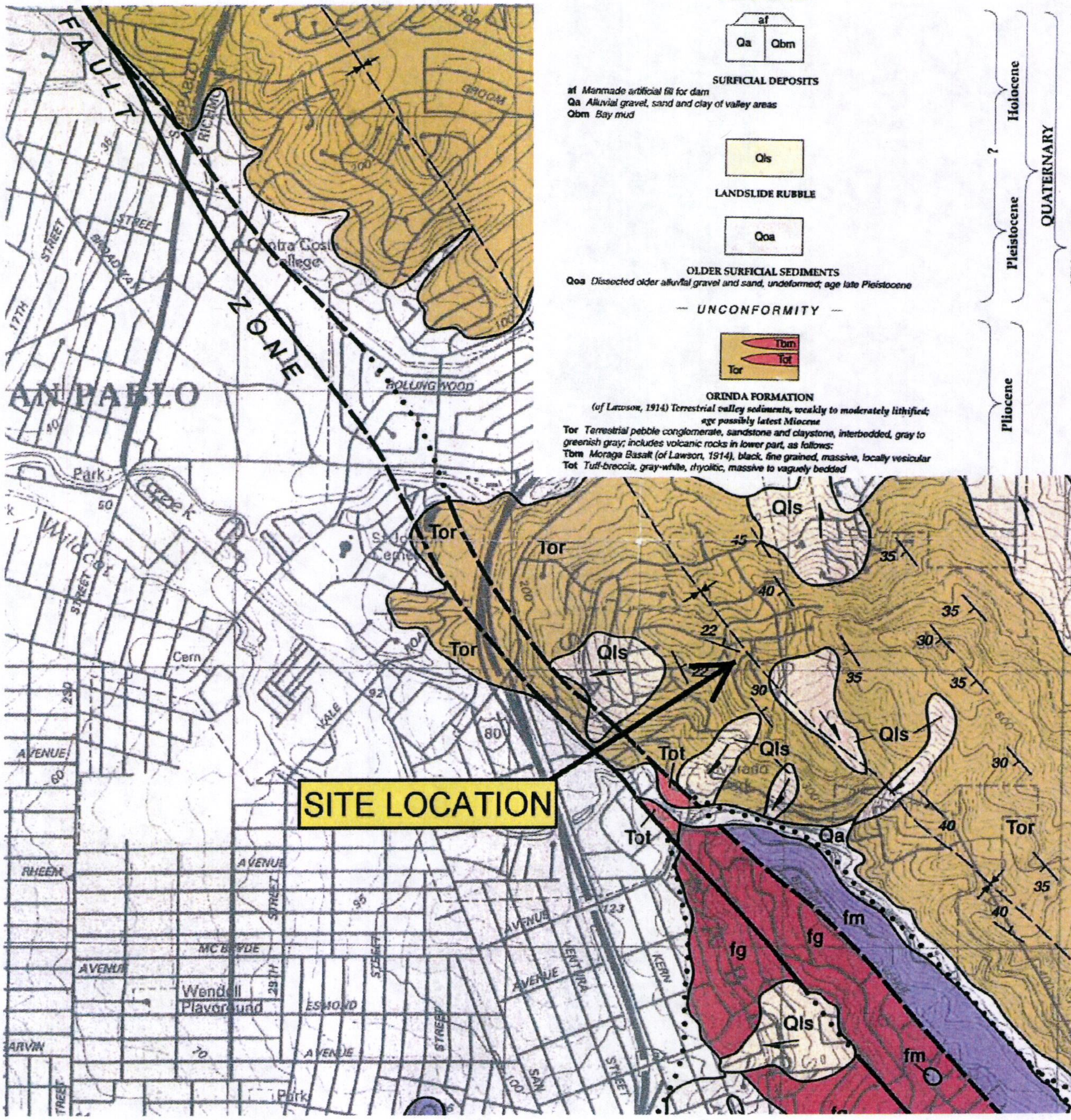
0 .125 .25 .375 .5 miles

FROM THOMAS BROTHERS MAPS, 1997

CE&G
CAL ENGINEERING & GEOLOGY

SITE LOCATION MAP	
1518 BARTH AVENUE, RICHMOND, CA	
JN: 070780	FIGURE 1

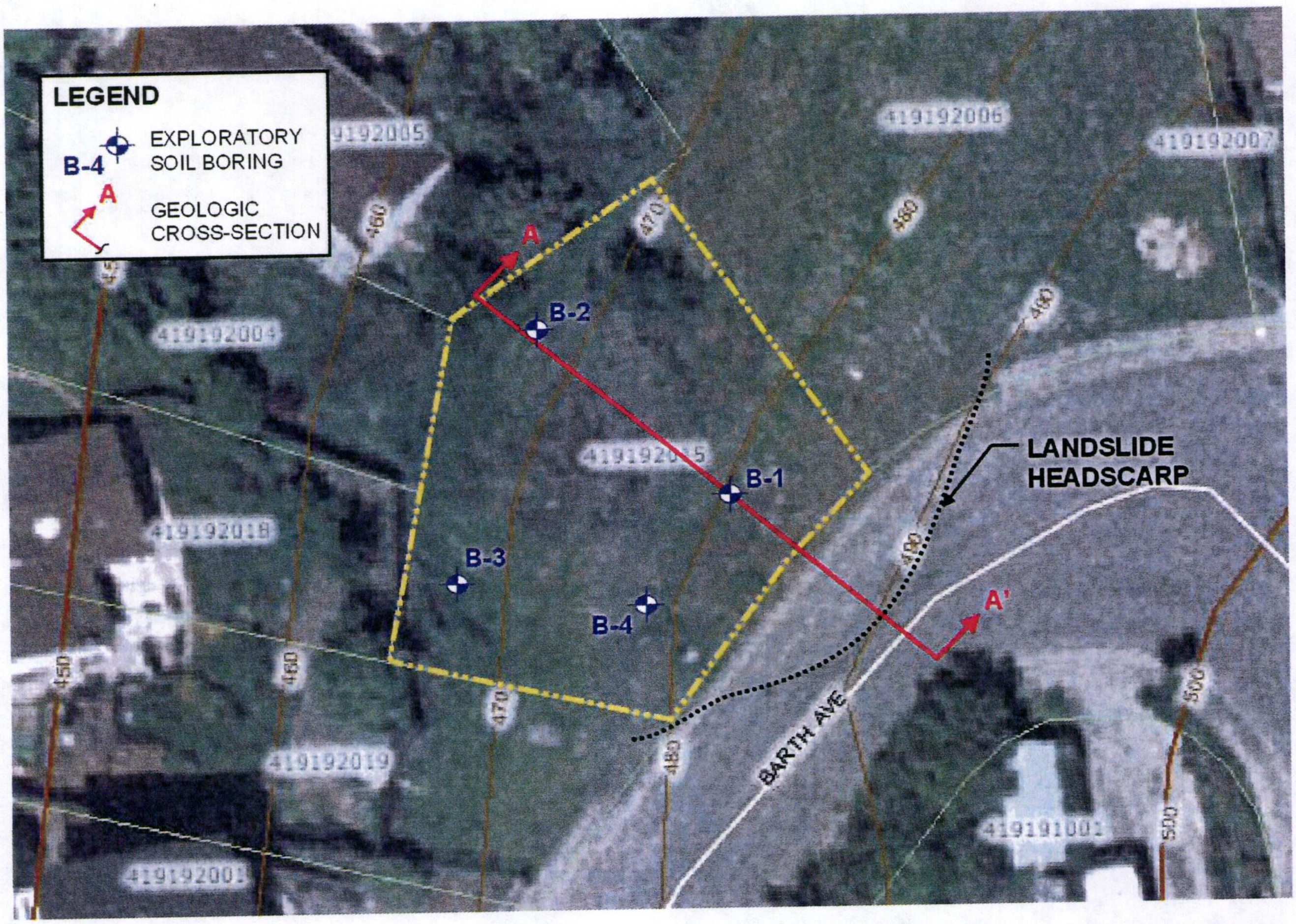
RICHMOND MAP (DF-147)



NO SCALE

AFTER DIBBLEE, (2005)

CE&G CAL ENGINEERING & GEOLOGY	REGIONAL GEOLOGY MAP
	1518 BARTH AVENUE, RICHMOND, CA
	JN: 070780 — FIGURE 2



NOT TO SCALE

SOURCE: CONTRA COSTA COUNTY GIS

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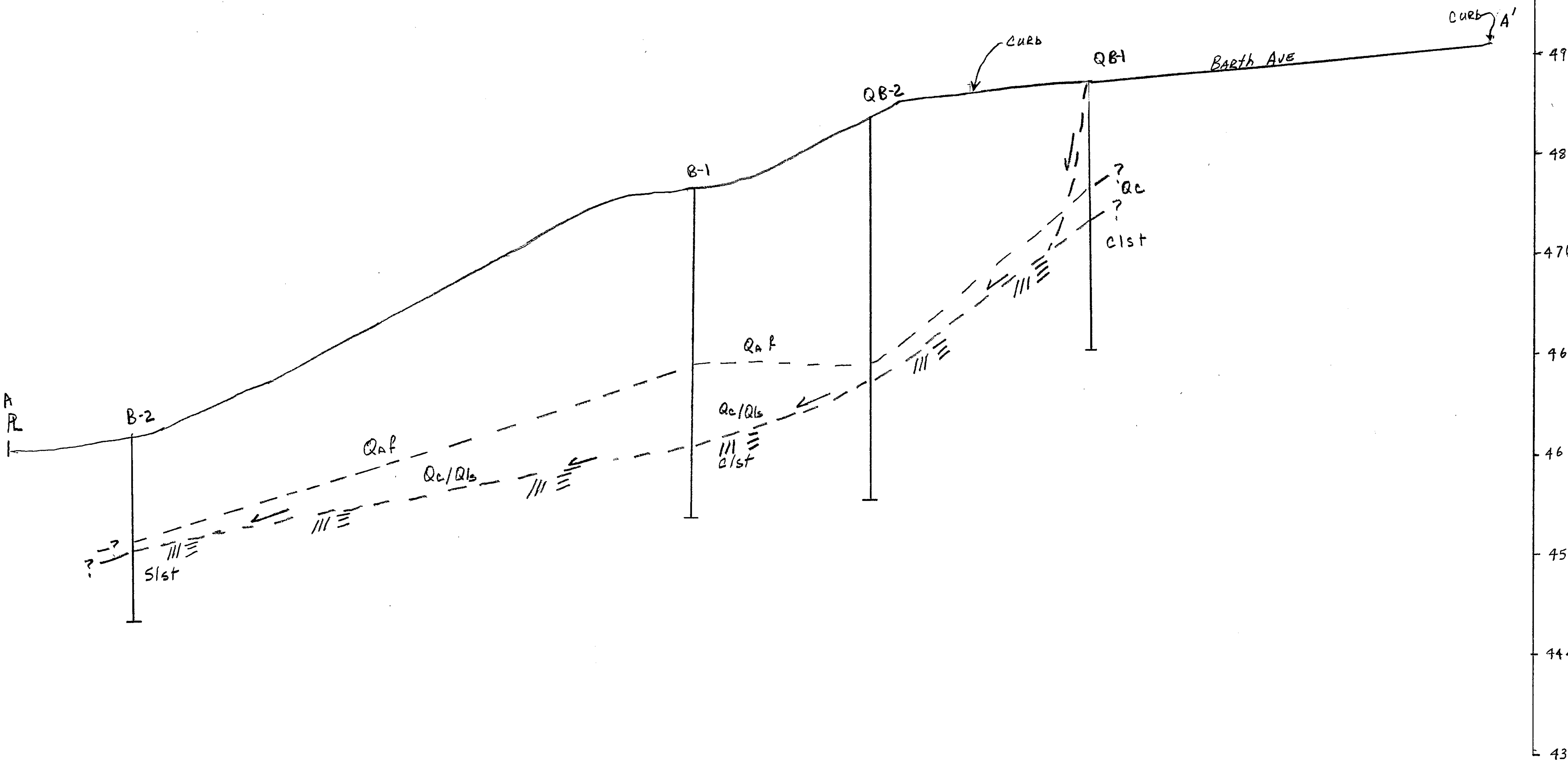
BORING LOCATION MAP	
1518 BARTH AVENUE, RICHMOND, CA	
JN: 070780	FIGURE 3



Scale as shown.

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GEOLOGIC CROSS SECTION

1518 Barth Avenue, Richmond, California

JN: 070780.001

Figure 4

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Field Identification			Group Symbol	Typical Names	Laboratory Classification Criteria	
Coarse-Grained Soils More than 50% of material is retained on the No. 200 sieve.	**Gravels** More than 50% coarse fraction retained on the No. 4 sieve	Clean Gravels < 5% Fines	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_U = D_{60} \div D_{10} \geq 4$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1$ & ≤ 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	$C_U = D_{60} \div D_{10} < 4$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1$ & > 3	
		Gravels with Fines >12% Fines	GM	Silty gravels, poorly graded gravel-sand-silt mixtures	Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol GC/GM
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	Fines classify as CL or CH	
	Sands More than 50% coarse fraction passes the No. 4 sieve	Clean Sands < 5% Fines	SW	Well-graded sands, gravelly sands, little or no fines	$C_U = D_{60} \div D_{10} \geq 6$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1$ & ≤ 3	
			SP	Poorly graded sands, gravelly sands, little or no fines	$C_U = D_{60} \div D_{10} < 6$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1$ & > 3	
		Sands with Fines >12% Fines	SM	Silty sands, poorly graded sand-silt mixtures	Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol SC/SM
			SC	Clayey sands, poorly graded sand-clay mixtures	Fines classify as CL or CH	
Fine-Grained Soils More than 50% of material passes the No. 200 sieve.	Identification Procedures on Percentage Passing the No. 40 Sieve					
	Silts & Clays Liquid Limit less than 50%		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands with slight plasticity		
			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and/or silty clays, lean clays		
			OL	Organic silts, organic silty clays of low plasticity		
	Silts & Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy/-silty soil, elastic silts		
			CH	Inorganic clays of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity		
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils		

CLASSIFICATION OF GRAVELS & SANDS WITH 5% TO 12% FINES REQUIRES DUAL SYMBOLS

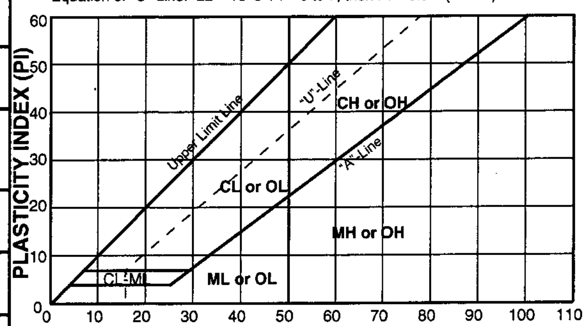
Gravel/Silty Gravel
Gravel/Clayey Gravel
Sand/Silty Sand
Sand/Clayey Sand

GW/GM or GP/GM:
GW/GC or GP/GC:
SW/SM or SP/SM:
SW/SC or SP/SC:

PLASTICITY CHART

For Classification of Fine-Grained Soils and Fine-Grained Fraction of Coarse-Grained Soils

Equation of "A"-Line: PI = 4 @ LL = 4 to 25.5, then PI = 0.73 x (LL - 20)
Equation of "U"-Line: LL = 16 @ PI = 0 to 7, then PI = 0.9 x (LL - 8)



KEY TO SAMPLER TYPES AND OTHER LOG SYMBOLS

CS	California Standard Sampler
CM	California Modified Sampler
SPT	Standard Penetration Test Sampler
SHL	Shelby Tube Sampler
BU	Bulk Sample
LL	Liquid Limit of Sample (ASTM D-4318)
PI	Plasticity Index of Sample (ASTM D-4318)
Q_U	Unconfined Compression Test (ASTM D-2166)
▽	Depth at which Groundwater was Encountered During Drilling
▼	Depth at which Groundwater was Measured After Drilling
PP	Pocket Penetrometer Test
PTV	Pocket Torvane Test
-#200	% of Material Passing the No. 200 Sieve Test (ASTM D-1140)
PSA	Particle-Size Analysis (ASTM D-422 & D-1140)
C	Consolidation Test (ASTM D-2435)
TXUU	Unconsolidated Undrained Compression Test (ASTM D-2850)

KEY TO SAMPLE INTERVALS

CS	Length of Sampler Interval with a CS Sampler
☒	Length of Sampler Interval with a CM Sampler
▥	Length of Sampler Interval with a SPT Sampler
SHL	Length of Sampler Interval with a SHL Sampler
BU	Bulk Sample Recovered for Interval Shown (i.e., cuttings)
⌶	Length of Coring Run with Core Barrel Type Sampler
NR	No Sample Recovered for Interval Shown

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UNIFIED SOIL CLASSIFICATION SYSTEM AND KEY TO BORING LOGS

FIGURE 5

Rock Hardness Descriptions

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimen requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to 1/4-inch deep can be excavated by hard blow of geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16-inch deep by firm pressure of knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small tin pieces can be broken by finger pressure.
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Bedding Thickness & Joint/Fracture Spacing Descriptions

Centimeters	Inches	Bedding	Joints/Fractures
< 2	< ¾	Laminated	Extremely Close
2-5	¾-2	Very Thin	Very Close
5-30	2-12	Thin	Close
30-90	12-36	Medium	Moderate
90-300	36-120	Thick	Wide
> 300	> 120	Very Thick	Very Wide

Rock Weathering Descriptions

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dulled and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately Severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible. But mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

The above Bedrock Characteristics are based on the ASCE Manual No. 56, "Subsrface Investigation For Design And Construction Of Foundations Of Buildings," 1976.

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Bedrock Characteristics Chart

FIGURE 6