Appendix GEO

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION AND FOUNDATION DESIGN
BERKELEY CITY COLLEGE
2118 MILVIA STREET
BERKELEY, CALIFORNIA

Prepared for

Noll & Tam Architects
729 Heinz Ave. #7
Berkeley, CA 94710

Prepared by

Terraphase Engineering Inc.
1404 Franklin Street, Suite 600
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June 9, 2017

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June 9, 2017 0034.002.001

Merideth Marschak AIA, CSI, LEED AP
Noll & Tam Architects
729 Heinz Ave. #7
Berkeley, CA 94710

Subject: Geotechnical Investigation and Design Report, Proposed Berkeley City College, 2118 Milvia Street, Berkeley, California

Dear Ms. Marschak:

Terraphase Engineering Inc. (Terraphase) is pleased to present the attached Geotechnical Investigation and Foundation Design Report for the proposed Community College Site to be located at 2118 Milvia Street in Berkeley, California.

Terraphase appreciates this opportunity to provide consulting services to Noll & Tam Architects, and looks forward to being of further assistance as the project proceeds.

If you have any comments or questions concerning this report, please contact Jeff Raines at (510) 645-1853.

Sincerely,

Chris Alger, P.G. (C5020), C.E.G. (1564)
Principal Engineering Geologist

Jeff Raines, P.E. (C51120), G.E. (2762)
Principal Geotechnical Engineer

Attachment

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1. INTRODUCTION

This report presents the results of the geotechnical investigation conducted by Terraphase Engineering Inc. (Terraphase) for the site of a proposed Berkeley City College building to be located at 2118 Milvia Street in Berkeley, California (“the Site”; Figure 1).

The report includes an assessment of the capacities of the existing foundation elements. We have included a site-specific seismic hazard assessment performed in accordance with ASCE 41 (2013) for use by the project structural engineers (Appendix D) in evaluating the structural performance of the building during earthquakes.

This report also includes our opinions concerning potential geotechnical constraints and geological hazards that may have an impact on site development and could potentially impede the performance of the proposed project. This assessment covers the requirements of California Geological Survey Note 48 (CGS 2013), *“Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings”.* This report was prepared in general accordance with California Educational Code Section 17212.5. Conclusions presented in this report are based in part on the published data discussed in this report, and on our experience with the types of geotechnical constraints applicable to sites located in Northern California. These conclusions should not be extrapolated to other areas outside the Site without our prior review and concurrence.

2. LOCATION AND SITE DESCRIPTION

The Site (Figure 1) is located in Alameda County in the City of Berkeley. It occupies 11,300 square feet on the northwest corner of Milvia and Center Streets in a commercial district within the city limits of Berkeley. Figure 2 presents the building footprint and the boring locations.

The center of the Site is located at a latitude of approximately 37.8707° North, and a longitude of approximately 122.2707° West. According to published topographic maps (Figure 3), it lies at an elevation of approximately 169 feet above mean sea level (msl), and is essentially flat. The local topography slopes to the west toward San Francisco Bay at an angle of approximately 100 feet per mile.

The 1903 Sanborn Fire Insurance Map of the area (Terraphase 2015) shows that the Site was a vacant lot in 1903 and that Milvia Street had not yet been extended between Addison and Center Streets. The 1929 Sanborn Map shows that the Site was still vacant, though Milvia Street extended between Addison and Center Streets. A gasoline service station was located across Milvia Street from the Site. Strawberry Creek is no longer shown above ground in 1929 having been routed through a subsurface culvert by that date. The 1950 Sanborn Map shows the Site had been developed as a gasoline service station. The RL Polk City Directory indicates that Fairchild and White was located at 1999 Center Street (the Site would be addressed as 1999 Center Street if the building on the Site fronted on Center Street) in 1943 – Environmental Data Resources (EDR) lists Fairchild and White as a former gasoline service station.

The Site was inspected at reconnaissance-level by Mr. Jeffery Raines, PE (51120) GE (2762) on January 26, 2015. No obvious surface evidence of potential geological hazards was evident at the Site on that date.

3. PURPOSE AND SCOPE OF SERVICES

Terraphase's scope of work included:

- conducted a review of geologic hazards data
- conducted a Site inspection
- Installed three borings at the Site to depths up to 50 feet below grade (Appendix B)
- Conducted laboratory testing on representative soil samples (Appendix C)
- Prepared a Site-Specific Seismic Hazard Assessment for the Site (Appendix D)
- prepared a report of pertinent findings with respect to seismic, geologic, and geotechnical engineering issues, including:
    - pertinent site maps showing the approximate project location
    - local geologic setting, faulting, and seismicity
    - site liquefaction potential, ground rupture potential, and other geologic and seismic hazards
    - flood inundation potential
    - Allowable foundation loads

4. SITE CONDITIONS

The local and regional geologic conditions herein are based upon our subsurface investigations, subsurface investigations at neighboring sites and our regional experience and available literature.

4.1 Geology and Soils

4.1.1 Regional Geology

The topography of the Bay Area consists of north- to northwest-trending mountain ranges and intervening valleys that are characteristic of the Coast Range geomorphic province. The Coast Ranges consist of the Mendocino Range to the north of San Francisco Bay, the Santa Cruz Mountains west of the Bay, and the Diablo Range to the east of the Bay.

The San Andreas Fault Zone lies to the west, and represents a major boundary that separates Franciscan Complex rocks on the North American Plate from Salinian basement rocks of the Pacific Plate.

The Coast Ranges represent northwest-southeast trending structural blocks comprised of a variety of lithologies that are juxtaposed by major geologic structures. The Coast Ranges-Sierran Block boundary zone lies many miles to the east of the site. To the west, the major boundary is the San Andreas Fault Zone, which separates Franciscan Complex rocks of the North American plate from the Salinian rocks on the Pacific plate. Oceanic crust Coast Ranges ophiolites within the Franciscan Complex have been deformed by a series of thrust faults, most of which appear to be inactive.

The geology of the San Francisco Bay Area is made up primarily of three different geologic provinces: the Salinian block, the Franciscan complex, and the Great Valley sequence. The Salinian block is located west of the San Andreas fault and is composed primarily of granitic rocks.

The Mesozoic Franciscan Complex is bounded on the east side by the Hayward fault and on the west side by the San Andreas fault. The Franciscan rocks represent terranes of former crust that have been accreted to North America by subduction and collision. These rocks are primarily deep marine sandstone and shale. However, chert and limestone are also found within the assemblage. Certain rocks of the Franciscan complex are prone to landslides.

To the east of the Hayward fault is the Great Valley sequence which in the Bay Area is composed primarily of Cretaceous and Tertiary marine sedimentary rocks in the Bay Area. These rocks are also prone to landsliding.

The Diablo Range extends from the Sacramento River Delta, south along the western side of the San Joaquin Valley. Rocks of the Mesozoic Great Valley are thrust upon Franciscan Complex basement along the San Joaquin Valley margin, and are covered locally by younger sediments of Paleocene to Pleistocene age.

Faults of the San Andreas system separate the Diablo Range from the remainder of the Coast Ranges. Mount Diablo is separated from the western East Bay hills by the Calaveras fault and from the southern extension of the Diablo Range by the Livermore Valley, an east-west-trending Cenozoic basin. The Diablo Range is bounded to the east by the Coast Range-Sierran Block boundary zone, which typically is represented by a series of blind and partially concealed thrust faults (Wong et al., 1988; Unruh and Moores, 1992). The eastern side of Mount Diablo is bounded by the San Joaquin fault (Sowers et al., 1992).

The Diablo Range comprises a series of large asymmetrical anticlines, with intervening synclines. The anticlines are composed of Franciscan Complex rocks, while the synclines contain younger rocks. The folds are frequently cut by east- and west-verging thrust faults. These thrust faults are displaced or truncated by strike-slip movement on the northwest-striking, right-lateral faults of the San Andreas fault system.

The complex arrangement of faults is the result of vigorous tectonic activity which have resulted in locally steep terrain (though not at the Site) with consequent landsliding hazards.

4.1.2 Local Geology

A cross-section of the Site is presented on Figure 5. A geological map (Graymer 2000) is presented on Figure 4. As shown on Figure 4, the local surficial geologic unit is Holocene age (less than 11,000 years old) alluvial fan and fluvial deposits. Graymer describes this unit as:

> *Alluvial fan deposits are brown or tan, medium dense to dense, gravely sand or sandy gravel that generally grades upward to sandy or silty clay. Near the distal fan edges, the fluvial deposits are typically brown, never reddish, medium dense sand that fines upward to sandy or silty clay. The best developed Holocene alluvial fans are on the San Francisco Bay plain. All other alluvial fans and fluvial deposits are confined to narrow valley floors.*

URS (2001) conducted a subsurface exploration approximately 1200 feet west of the Site in the same geologic unit as the Site. URS described the soils they encountered as, “stiff to very stiff silty and sandy clay, overlying hard clay and dense sand below depths of 40 feet.”

Kaldveer (1981) found Franciscan bedrock (sandstone) at 34 feet bgs at a site located 950 feet due east of the Site. Standard Penetration Test (SPT) blow counts ranged from 13 to >100 in Kaldveer’s borings.

Engeo (2013) conducted a geotechnical feasibility study of a site 640 feet southeast of the Site. Their conclusion regarding local geology was,

> Surface soils at the site generally consists of stiff to very stiff gravelly to sandy clay with interbedded layers of medium dense to dense clayey sand and gravels sized rock fragments. These are interpreted as Holocene age alluvial fan deposits and generally extend to depths less than 20 feet deep. The younger alluvium is underlain by older Pleistocene alluvium, generally consisting of similar layers of interbedded clays, sands

and gravels. However, the older granular deposits are dense to very dense and the clayey soils are very stiff to hard.

Engeo identified the soils below 20 feet bgs as Pleistocene-aged which are unlikely to liquefy during seismic events.

Figuers (1998) mapped the bedrock as 50 feet below the ground surface in the vicinity of the Site. CGS (2003b) indicates that the highest historical groundwater elevation in the Site vicinity is between 5 and 10 feet below the ground surface (bgs).

URS's description (2001) of the site 1200 feet to the west of the Site is consistent with Figuers (1998) and CGS (2003b).

4.2 Hydrology and Hydrogeology

CGS (2003b) indicates the highest groundwater level in the Site vicinity has been within 10 feet of the ground surface. Groundwater was encountered at 20 feet bgs in Terraphase boring B-3 on May 10, 2017. However, the groundwater table probably had not stabilized. A groundwater elevation of ten feet bgs was used in the liquefaction susceptibility analysis.

5. GEOLOGICAL HAZARDS

5.1 Faulting and Seismicity

The known regionally active faults within 50 kilometers of the Site that are capable of producing significant ground shaking at the Site are listed in Table 1 and shown on Figure 6. Activity was determined by slip rates, as per the CGS (Petersen et al. 1996 and Cao et al. 2003). The long-term average rate of slip is determined geologically. It is based on the total displacement of a geologic unit divided by the age of the unit. So, the fault is not actually moving other than in earthquakes.

Table 1 includes an estimate of the peak ground acceleration (at the mean plus one standard deviation level) and the Modified Mercalli Intensity likely to be felt at the Site due to earthquakes on the individual faults. The Modified Mercalli Intensity (MMI) scale is described in Table 2. The calculated MMI should be considered to be a rough order of magnitude estimate; it is presented here because it is more understandable for lay readers than peak ground accelerations.

MMI was evaluated using EQFAULT software (Blake 2000a). EQFAULT uses the inverse of the Murphy and O'Brian (1978) acceleration – intensity equation to calculate the MMI:

$$I_{mm} = [\log_{10}(980.7 * a_{Hg}) - 0.29]/0.24$$
$$a_{Hg} = horizontal\ acceleration\ (g)$$

The CGS probabilistic seismic hazard assessment website indicates that the estimated peak ground acceleration for the Site is 1.04 g for alluvium (CGS 2015) for a 2% in 50 years (2,475 year return period[1]) earthquake. This means that a 150 pound person will be subjected to a peak horizontal force of 156 pounds during an earthquake with this peak ground acceleration.

The 2007 Working Group on California Earthquake Open Seismic Hazard Assessment tool predicts that there is a 50% chance that the Site will experience a peak ground acceleration greater than 0.25g in the next 30 years and a 10% chance that the Site will experience a peak ground acceleration of 0.79g in the next 30 years.

Table 3 presents the significant historical earthquakes that have occurred in the site vicinity.

[1] That means that there is only a small chance that an earthquake with a peak ground acceleration greater than 1.04g will occur in a 2,475 year period. The 2007 Working Group on California Earthquake Probabilities indicated that there is a 31% chance that the Hayward Fault will rupture between 2008 and 2038. ABAG believes that the acceleration at the Site from the next event on the Hayward Fault will produce a Modified Mercalli Intensity of X at the Site (please see Table 2).

Table 1
Known Active Earthquake Faults within 50 Kilometers of the Site
Berkeley City College
Berkeley, California

Abbreviated Fault Name	Approx. Distance, miles (km)	Maximum Earthquake Mag. (Mw)	Horizontal Peak Ground Accel. (g)	Est. Site Intensity, Modified Mercalli
HAYWARD (North)	1.0 (1.6)	6.9	0.518	X
HAYWARD (Total Length)	1.0 (1.6)	7.1	0.531	X
HAYWARD (South)	12.2 (19.6)	6.9	0.182	VIII
CALAVERAS (No.of Calaveras Res)	13.0 (20.9)	6.8	0.168	VIII
CONCORD - GREEN VALLEY	14.8 (23.8)	6.9	0.156	VIII
RODGERS CREEK	15.5 (25.0)	7	0.155	VIII
SAN ANDREAS (Peninsula)	17.4 (28.0)	7.1	0.146	VIII
SAN ANDREAS (1906)	17.4 (28.0)	7.9	0.192	VIII
SAN ANDREAS (North Coast)	18.0 (29.0)	7.6	0.169	VIII
GREENVILLE	19.3 (31.0)	6.9	0.126	VIII
SAN GREGORIO	20.1 (32.3)	7.3	0.14	VIII
WEST NAPA	20.4 (32.8)	6.5	0.103	VII
GREAT VALLEY 6	23.7 (38.2)	6.7	0.12	VII
GREAT VALLEY 5	26.7 (42.9)	6.5	0.1	VII
MONTE VISTA - SHANNON	29.9 (48.1)	6.8	0.102	VII
POINT REYES	31.1 (50.0)	6.8	0.099	VII

Notes: The expected peak ground acceleration (PGA) is the mean value
PGA = peak ground acceleration

Table 2
Applicable Portions of Modified Mercalli Intensity Scale
Berkeley City College
Berkeley, California

Intensity	Shaking	Summary	Description
VII	Strong	Nonstructural Damage	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII	Very Strong	Moderate Damage	Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX	Violent	Heavy Damage	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations.) Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluvial areas sand and mud ejected, earthquake fountains, sand craters.
X	Very Violent	Extreme Damage	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.
Masonry B: Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.
Masonry C: Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.
Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Table 3
Historical Earthquakes in Site Vicinity Magnitude > 6
Berkeley City College
Berkeley, California

Latitude	Longitude	Date	Magnitude	PGA (g)	MM	Distance in miles (km)
37.8	122.2	06/10/1836	6.8	0.475	X	6.2 (10.0)
37.7	122.5	4/18/1906	8.25	0.269	IX	17.2 (27.6)
37.8	122.5	06/21/1808	6.3	0.193	VIII	13.4 (21.6)
37.7	122.1	10/21/1868	6.8	0.191	VIII	15.0 (24.1)
37.6	122.4	06/01/1838	7	0.156	VIII	19.9 (32.1)
38.2	122.4	03/31/1898	6.2	0.097	VII	23.8 (38.3)
38	121.9	05/19/1889	6	0.096	VII	22.1 (35.5)
37.5	121.9	11/26/1858	6.1	0.068	VI	32.6 (52.5)
38.4	122	04/19/1892	6.4	0.066	VI	39.4 (63.4)
37.036	121.883	10/18/1989	7	0.06	VI	61.4 (98.8)
37.25	121.75	7/1/1911	6.6	0.057	VI	51.4 (82.8)
37.3	121.9	10/08/1865	6.3	0.056	VI	44.3 (71.3)
38.5	121.9	04/21/1892	6.2	0.049	VI	47.9 (77.1)
37.32	121.698	4/24/1984	6.2	0.048	VI	49.2 (79.2)
36.83	121.57	10/18/1800	7	0.046	VI	81.5 (131.1)
37	121.5	06/20/1897	6.2	0.031	V	73.5 (118.2)
36.9	121.6	04/24/1890	6	0.026	V	76.4 (123.0)
36.61	122.35	10/22/1926	6.1	0.024	V	87.1 (140.2)
36.57	122.17	10/22/1926	6.1	0.024	IV	89.9 (144.7)

Notes: Source: Blake 2000c
Latitude and Longitude are the locations of the assumed epicenters
MM – Mercalli Magnitude (please see Table 2)
Acceleration is the mean expected acceleration at the Site due to the historical earthquake calculated using the Abrahamson & Silva (1997) attenuation relationship.
The Loma Prieta earthquake occurred on October 18, 1989 and produced an acceleration at the Site approximately equal to 6% of the acceleration from an earthquake on the Hayward Fault (see Appendix D, Table 2).

5.2 Ground Rupture Potential

The Site is not located within an Alquist-Priolo Special Studies Earthquake Fault Zone (CGS 1982). There are no known active faults, and therefore no Alquist-Priolo Zones, within 1 mile of the Site (Table 1). The nearest Alquist-Priolo Zone is located at UC Berkeley's Memorial Stadium (Hayward Fault) approximately 1 mile east of the Site.

Since the Site is remote from these faults, there does not appear to be a significant risk of surface rupture during the expected service life of the buildings

5.3 Liquefaction Potential

Liquefaction can be induced by cyclic loading (shaking) from an earthquake, which can cause granular materials to lose their inherent shear strength due to increased pore water pressures. Some of the factors that typically contribute to liquefaction risk include a shallow water table, low relative density of granular materials below the groundwater table, low soil cohesion or plasticity, low percentage of fine-grained material in soil, relatively long seismic shaking duration, and high ground acceleration during earthquakes.

CGS (2003a, Figure 7) does not map the Site as being in a liquefaction hazard zone.

5.3.1 Liquefaction Resistance

Terraphase encountered one potentially liquefiable strata in Boring 3 between 25 and 33 feet bgs. The soil was a gravelly sand with SPT blow counts of 38 (25 feet bgs) and 25 (30 feet bgs). SPT blow counts were adjusted as shown in Table 4.

Table 4
SPT Correction Factors
Berkeley City College
Berkeley, California

Factor	Value (25' / 30') bgs	Explanation
CS	1.3 / 1.25	Sampler did not contain rings or sleeves
CB	1.150	Borehole size (8 inch)
CE	1.000	Hammer efficiency
Cr	0.97 / 1	Rod Length
CN	0.87 /.83	Overburden
Total	1.26 / 1.19	

Overburden based on 125 pcf total unit weight with the water table at 20 ft bgs – the water level at the time of the boring

Hence, the corrected SPT blow counts are 48 and 30. These are consistent with blow counts in this geologic strata found in the adjacent sites.

Seed et al. (2003) recommends an SPT correction factor for fines equal to:

C_{fines} = (1+0.004*FC)+0.05*(FC/N1,60) = 1.09 for a fines content of 16.1% and an SPT blow count of 30 and 1.1 for an SPT blow count of 48. So the final corrected blow counts are 53 and 32.

5.3.2 Liquefaction Potential

The strata from 25 to 30 feet bgs will not liquefy, but the strata from 30 to 33 feet bgs may. Specifically:

The cyclic stress ratio at 31.5 feet is

$$CSR_{peak} = \frac{a_{max}}{g}\left(\frac{\sigma_v}{\sigma'_v}\right)r_d$$

a_{max} is the maximum credible earthquake peak ground acceleration (0.90 g, see Appendix D)

σ_v is the total vertical stress at 31.5 feet = 125 pcf * 10 feet + 130pcf * 21.5 feet = 4,045 pounds per square foot (psf) – based on the groundwater table at 10 feet bgs (worst case) and a saturated unit weight of the soil of 130 pcf.

σ'_v is the effective vertical stress at 31.5 feet = 125 pcf * 10 feet + (130-62.4) pcf * 21.5 feet = 2,700 pounds per square foot (psf)

r_d is the shear mass participation factor (1.0 from Seed et al. 2003 equation 2)

CSR_{peak} = 0.9*(4045/2700)*1 = 1.3

CSR_{eq} = 0.65* CSR_{peak} = .85 –> strata between 30 and 32.5 feet potentially liquefies

Figure 53 in Seed et al. (2003) indicates the volumetric strain for this strata will be approximately 1.0% resulting in a settlement of 0.3 inches. Given the depth of the liquefiable strata we do not expect there to be any significant differential settlement at the surface.

ASCE 41 regards the entire strata as non-liquefiable if:

The soils are cohesionless with a minimum normalized standard penetration test (SPT) resistance, $(N_1)_{60}$, value of 30 blows/0.3 m (30 blows/ft), as defined in ASTM D1586, for depths below the groundwater table;

As there is only one boring in the strata that is potentially liquefiable, the existing building should be conservatively assessed for a differential settlement of 0.3 inches. Based on ASCE 41 criteria, SPT blow count greater than 30, the stratum would not liquefy.

5.4 Landslide Potential

The Site area is essentially flat (Figure 3). Given the lack of relief, no significant landslide risk exists.

5.5 Flood Inundation Potential

5.5.1 Flood Zonation

The local Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FEMA 2009) indicate that the Site is not located within a 100-year flood zone. The nearest 100-year flood zone ("Zone A") is located approximately 1,500 feet east of the Site (Figure 8).

5.5.2 Dam Inundation

The Site is not within any dam inundation zones as mapped by the City of Berkeley (2015).

5.6 Land Subsidence

Land subsidence, generally caused by excessive groundwater withdrawal, is unlikely to occur in downtown Berkeley. Because of environmental concerns the groundwater in Berkeley is not a resource likely to be tapped. Should land subsidence occur, the building currently constructed on the Site is likely to be less susceptible than the adjacent buildings, which are taller and older, and hence the subsidence would likely begin to damage those buildings before it damaged the Site building and hence subsidence would be stopped before it affected the Site.

5.7 Naturally Occurring Asbestos

CGS (2000) does not map significant outcrops of serpentine-bearing (e.g., ultramafic) rocks in the watershed above the project Site. The chances of finding significant quantities of naturally-occurring asbestos (NOA) in alluvium derived from upslope bedrock at the Site are negligible.

5.8 Other Hazards

Certain other potential geologic hazards, including tsunamis, seiches, naturally occurring radon, and oil and gas fields, do not appear to pose significant risks at the Site, for the reasons discussed briefly below.

> **Tsunamis and Seiches.** Tsunamis do not pose an appreciable risk at this inland location (California Emergency Management Agency 2009). Seiches do not pose an appreciable risk given the absence of adjacent surface water bodies.
>
> **Naturally Occurring Radon.** The California Department of Health Services (DHS 2010) maintains a database of radon measurements in California, based on zip code. No elevated radon results (greater than or equal to 4.0 pCi/L) have been reported in 34 measurements from the 94704 (Berkeley) zip code, which includes the Site.

Oil and Gas Fields. The Site is not located within an oil or gas field, as recognized by the California Department of Oil, Gas, and Geothermal Resources (DOGGR 2015) maintains a Geographical Information System (GIS) map of all active and abandoned oil and natural gas wells in California. No wells have been drilled in the Berkeley area. The nearest abandoned well is approximately 3 miles east of the Site.

6. SEISMIC HAZARD PARAMETERS

Seismic design information is presented in Appendix D.

7. FOUNDATION DESIGN

7.1 General

The building is approximately 50 years old and shows no visible signs of foundation problems (cracking in masonry walls, separation of flatwork, non-planar floors) so it's existing foundation appears to be performing well. ASCE 41 (2013) requires that the following parameters be reported for the building:

1. Foundation type;
2. Foundation configuration, including dimensions and locations; and
3. Material composition and details of construction.

ASCE 41 (2013) requires the following soil information:

1. unit weight, γ;
2. the effective stress friction angle, φ' or the undrained shear strength of clays, su;
3. soil compressibility characteristics;
4. small-strain soil shear modulus, Gmax ; and
5. Poisson's ratio, ν .

7.2 Existing Foundation

The current structure was constructed in the mid-1960s – the building construction plans are dated August 18, 1966 and a 1968 aerial photograph shows the existing building. It is a three-story, approximately 11,000 square foot footprint facility. Interior columns are supported on 9-feet by 9-feet spread footings (Appendix A). The western and southern walls of the facility are supported on spread footings varying in size from 9-feet by 3-feet to 7 feet by 7 feet. The building construction plans show the eastern wall being supported on a strip footing 3.33 feet by 92-feet. The north wall is supported on 14, 18-inch diameter drilled piers of unknown depth. The building plans (Appendix A) indicated the drilled piers were to be installed 4 feet into the dense cohesionless strata which would indicate they could be installed to approximately 30 feet below existing grade.

Interior column loads are 70 kips (Shea 2017) which corresponds to a bearing pressure of 860 pounds per square foot (psf) which is about half of the presumptive building code allowable bearing pressure.

7.3 Soil Properties

The clay soils on which the spread footings derive support can be modeled with the following properties

1. moist unit weight, γ – 125 pounds per cubic foot (pcf)
2. saturated unit weight – 130 pcf
3. the undrained shear strength of clays, su; - 2,500 psf (the actual shear strength of the clays supporting the existing footings is likely to be higher due to the 50 years of consolidation that has taken place since the building was constructed).
4. soil compressibility characteristics; k_{sv}, use 150 pounds per cubic inch (pci) divided by the width of the footing (least dimension) for static analyses and 240 pci divided by the width of the footing for dynamic analyses (Johnson and Ireland, 1963, found that clays loaded dynamically were 1.6 times stiffer than the same clays loaded statically).
5. small-strain soil shear modulus, Gmax – 1,700 tons per square foot (tsf) (Ohsaki & Iwasaki 1973 – Gmax = $(78*(N_{60})^{0.39})^2$ times (3.28 feet/meter) times soil density = $(286 \text{ m/s} *3.28 \text{ ft/m})^2 * 125 \text{ pcf} / 32.2 \text{ ft/s}^2$ = 1,700 tsf; and
6. Poisson's ratio,ν – use 0.35 for soil above 10 feet bgs and 0.5 for soils below 10 feet bgs.

Unit weights are based on the material types and our experience in the site vicinity. Undrained shear strength is based on pocket penetrometer values in shallow soils from Boring 1. Soil compressibility characteristics are based on the low end of the range of soil compressibilities from published data for clay soils (USACE 1984, Page 2-4). Poisson's ratio is from ASCE 41 (ASCE 2013).

7.4 Fill Recommendations

Imported fill materials should be approved by the Engineer before being brought to the Site. Imported fill shall be certified as clean from the source (not from former industrial sites or similar locations; not chemically affected). Imported fill should be nonexpansive, granular in nature and meet the following requirements: minimum R-Value of 35 (Caltrans Test Method 301), maximum expansion index of 25 (UBC 18-2), and maximum plasticity index of 12 (ASTM D4318). The soil should be compacted in lifts no greater than 8 inches loose to a minimum of 90% of the soil's maximum dry density. Native soil below the fill should be scarified to a depth of 12 inches, moisture conditioned to a minimum of 12% above optimum and be compacted to 90% of it's maximum dry density. A representative of the geotechnical engineer should observe placing and compacting of fill and backfill.

Controlled density fill shall be composed of cementitious materials, aggregate, water, and an air-entraining admixture, as follows:

1. Cementitious materials shall be portland cement in combination with fly ash.
2. Admixture shall be an air-entraining agent.

3. Aggregate Content: CDF mixture shall contain no aggregate larger than 3/8 inch. Amount passing a No. 200 sieve shall not exceed 12 percent. No plastic fines shall be present.

4. Air Content: Total calculated air content of the sample, prepared in accordance with ASTM C231, shall not exceed 30 percent

5. Strength: Controlled density fill shall have an unconfined compressive strength at 28 days of from 50 psi to a maximum of 150 psi.

7.5 Trench Excavation and Backfilling

Trenches should be excavated as required by the plans and specifications, using appropriate equipment. Where necessary, trenches should be sloped or shored by the contractor, in accordance with the governing safety standards to provide a safe work site. The contractor shall be responsible for any temporary slopes and trenches excavated at the Site and for design of shoring, should it be required.

Trenches should be backfilled with compacted fill, in accordance with the stricter of the recommendations contained in this section or in accordance with local requirements. Fill material should be placed in lifts no greater than 8 inches in loose thickness and compacted by mechanical means. Trench backfill should be compacted to at least 90% relative compaction.

7.6 Excavations Adjacent to Buildings

Trenches and other excavations located adjacent to existing foundations should be located such that an imaginary line drawn at a 45 degree angle from the bottom of the outer edge of the spread footing does not intersect the trench.

Trenches and other excavations that will pass within an imaginary 45-degree angle to a spread footing or slab-on-grade foundation that will be constructed in the future should be backfilled with clean fill compacted to at least 95% relative compaction or with controlled density fill prior to constructing the foundation or slab.

Trenches to be excavated parallel to an existing slab-on-grade foundation should be located such that an imaginary line drawn at a 45 degree angle from the bottom of the outer edge of the slab does not intersect the trench. If this is not possible, the trench can be installed in 5-foot long sections with each section backfilled with clean fill compacted to at least 95% relative compaction or with controlled density fill prior to excavating the next segment of the trench.

For other trench/foundation layouts, please consult with the engineer.

7.7 Foundations

7.7.1 Spread or Continuous Footings

The existing footings vary between 3 and 9 feet wide and are based 3 feet below the top of slab (Appendix A). Per Section 8.4.2.1, the soil properties between 5 and 8 feet below the top of slab can be used to assess bearing capacity. Based on pocket penetrometer and blow counts in this vicinity, the undrained shear strength of the clay bearing strata is approximately 2,500 psf. The following are recommended allowable bearing pressures for foundation elements:

Table 5
Spread Footing Allowable Bearing Pressures
Berkeley City College
Berkeley, California

Loading Condition	Allowable Bearing Pressure
Dead Loads	3,300 psf
Dead plus Live Loads	5,000 psf
All Loads, including Wind or Seismic	6,500 psf

Notes: psf = pounds per square foot; Factor of safety = 4

If additional footings are required, footing concrete should be poured neat against native soil. Footings excavations should not be allowed to dry out prior to pouring concrete. Cracks in footing excavations more than ¼ inch wide should be dug out. Any disturbed or softened material encountered at the bottom of the footing excavations should be removed to expose firm bearing material. Overexcavated areas should be backfilled with lean or structural concrete. Footing excavations should be kept moist before concrete placement.

Continuous footings should be reinforced with a minimum of at least two (2) #4 bars top and bottom in the longitudinal direction unless otherwise determined by the structural engineer. Isolated spread footings should be reinforced with a minimum of two (2) #4 bars in each direction. Reinforcement should be spaced 12 inches on center in each direction unless otherwise determined by the structural engineer.

Before issuing the construction bids, the geotechnical engineer should review the foundation plans and prepare a review letter. In addition, the geotechnical engineer should observe foundation operations.

7.7.2 Concrete Slabs-on-Grade

Slab-on-grade floors should be supported on a minimum of 4 inches of clean gravel or crushed rock. We recommend that moisture sensitive foundations in direct contact with the subsurface (mechanical rooms, elevator shafts, lobbies and commercial and residential units on the ground floor) be underlain by a moisture barrier. A typical moisture barrier should include a capillary moisture break consisting of at least four inches of clean, free-draining gravel or crushed rock (1/2 to 3/4 inch gradation) overlain by a moisture-proof membrane of at least 10 mils thick (15-mil Stego, Grace FlorPrufe or equivalent – for shallow groundwater, require Grace PrePrufe). The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. Water should not be allowed to accumulate in the capillary break or sand prior to casting the slab.

The vapor retarder should meet the requirements for Class C vapor retarders as given in ASTM Standard E1745-97. The vapor retarder should be installed in general accordance with the methodology documented in ASTM Standard E1643-98. These requirements include overlapping seams by at least six inches, taping seams, and sealing penetration through the vapor retarder. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in the following table.

Material for support of slabs should conform to the gradation specification shown in Table 6.

Table 6
Subslab Foundation Materials
Berkeley City College
Berkeley, California

	Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	1 inch	90 – 100
	¾ inch	30 – 100
	½ inch	5 – 25
	3/8 inch	0 – 6
Sand	No. 4	100
	No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed. There should be no free liquid in the sand.

It is recommended that slabs-on-grade be reinforced with reinforcing bars instead of mesh. Slabs should be constructed with frequently spaced construction joints to reduce the potential for uncontrolled shrinkage cracking. Spacing and type of joints should be designed by the structural engineer. The slab subgrade should be prepared as described in Section 7.4.

7.7.3 Drilled Piers

The drilled piers were designed as end-bearing in the dense gravelly-sand stratum located approximately 30 feet bgs. Based on corrected SPT blow counts of 48 (see Section 5.3 of this report), the friction angle of the gravelly-sand is approximately 40^{o}.

Based on a friction angle of 40^{o} and a depth below the ground surface of 25 feet, an 18-inch diameter drilled pier would have an allowable bearing capacity (factor of safety of 3) of 100 kips. Even neglecting side friction, the drilled piers appear to have sufficient capacity.

Additional drilled piers would not be an economical foundation type if additional deep foundation elements are required. In the event that additional deep foundation elements are required, we recommend that micropiles embedded into the dense gravelly-sand below 25 feet bgs be used. A six-inch diameter, concrete-filled micropile would have a capacity of 40 kips (including side friction) using a factor of safety of 2 based on a shear strength of 1.5 kips per square foot (ksf). A lower factor of safety is appropriate given that the jacking pressure used to install the micropile is known. For uplift control, the micropiles would have a capacity of 29 kips.

7.8 Soil Corrosivity

Examination of the concrete cores removed from the building slab did not indicate any deterioration of the concrete after 50 years in contact with Site soils. New metal utilities should be corrosion protected.

7.9 Soil Expansion

The plasticity index of a soil sample collected between 1 and 3 feet bgs was 18 (low expansion potential) while the plasticity index of a soil sample collected between 5 and 6 feet bgs was 38 (very high expansion potential). Given that the water table below the Site is very shallow and the entire site is paved, we would not expect that the foundation soil moisture content would change significantly and hence expansion/shrinkage of the clay soils is unlikely. No indications of building distress indicative of differential settlements (e.g., diagonal cracks in masonry walls) were noted.

7.10 Exterior Flatwork

It is recommended that exterior concrete flatwork be a minimum of 4 inches thick and reinforced with reinforcing bars. Exterior flatwork should be underlain by at least 4 inches of aggregate base rock conforming to Caltrans Class 2 standards that is compacted to a minimum of 92% relative compaction. The exterior flatwork should be poured separately from building foundations so that they act independently of the walls and foundations. Exterior finish grades should be sloped a minimum of 2% percent away from interior slab areas to preclude ponding of water adjacent to the structures. Soils below exterior flatwork should scarified to a depth of 6 inches and be compacted to a minimum of 92% of the Modified Proctor Maximum Density at a moisture content at least 2% greater than the optimum moisture content. This may require moisture conditioning the soil.

8. CONCLUSIONS

Our findings are summarized below.

- Existing foundation elements are loaded well below their static capacities
- Liquefaction settlements are likely to be less than 0.3 inches
- The Site is not located within or near an Alquist Priolo Special Studies Earthquake Fault Zone. Surface rupture should not reasonably be expected during the life of the building.
- The Site is not located within the 100-year flood zone.
- School buildings constructed on the Site will likely be subjected to strong shaking during earthquakes during their useful economic lives.

Based on the above findings, it is Terraphase's opinion that the Site is suitable for the proposed school development.

9. DESIGN REVIEW AND CONSTRUCTION MONITORING

Terraphase recommends that the geotechnical aspects of the project be reviewed by Terraphase during the design process. The scope of services may include:

- assisting the design team in providing specific recommendations for special cases
- reviewing the foundation design and evaluating the overall applicability of our recommendations
- reviewing the geotechnical portions of the project for possible cost savings through alternative approaches
- reviewing the proposed construction techniques to evaluate whether they satisfy the intent of our recommendations
- reviewing and stamping drawings

Terraphase recommends that foundation construction and earthwork performed during construction, if any, be monitored by a qualified representative from our office, including:

- site preparation (stripping and grading)
- placement of compacted fill and backfill
- all foundation excavations
- construction of slab, roadway, and/or parking-area subgrade

Terraphase's representative should be present to observe the soil conditions encountered during construction to evaluate the applicability of the recommendations presented in this report to the soil conditions encountered and to recommend appropriate changes in design or construction procedures, if conditions differ from those described herein.

10. LIMITATIONS

The opinions and recommendations presented in this report are based upon the scope of services, information obtained through the performance of the services, and the schedule as agreed upon by Terraphase and the party for whom this report was originally prepared. This report is an instrument of professional service and was prepared in accordance with the generally accepted standards and level of skill and care under similar conditions and circumstances established by the geotechnical consulting industry. No representation, warranty, or guarantee, express or implied, is intended or given. To the extent that Terraphase relied upon any information prepared by other parties not under contract to Terraphase, Terraphase makes no representation as to the accuracy or completeness of such information. This report is expressly for the sole and exclusive use of the party for whom this report was originally prepared for a particular purpose and only in it's entirely. Only the party for whom this report was originally prepared and/or other specifically named parties have the right to make use of and rely upon this report. Reuse of this report or any portion thereof for other than its intended purpose, or if modified, or if used by third parties, shall be at the user's sole risk.

Furthermore, nothing contained in this report shall relieve any other party of its responsibility to abide by contract documents and applicable laws, codes, regulations, or standards.

<u>Subsurface Explorations and Testing</u>

Results of any observations, subsurface exploration or testing, and any findings presented in this report apply solely to conditions existing at the time when Terraphase's exploratory work was performed. It must be recognized that any such observations and exploratory or testing activities are inherently limited and do not represent a conclusive or complete characterization. Conditions in other parts of the project site may vary from those at the locations where data were collected and conditions can change with time. Terraphase's ability to interpret exploratory and test results is related to the availability of the data and the extent of the exploratory and testing activities.

The findings and recommendations submitted in this report are based, in part, on data obtained from subsurface borings, test pits, and specific, discrete sampling locations. The nature and extent of variation between these test locations, which may be widely spaced, may not become evident until construction. If variations are subsequently encountered, it will be necessary to re-evaluate the conclusions and recommendations of this report.

Correlations and descriptions of subsurface conditions presented in boring logs, test pit logs, subsurface profiles, and other materials are approximate only. Subsurface conditions may vary significantly from those encountered in borings and sampling locations and transitions between subsurface materials may be gradual or highly variable.

Conditions at the time water level measurements and other subsurface observations were made are presented in the boring logs or other sampling forms. This field data have been reviewed and interpretations provided in this report. However, groundwater levels may be variable and may fluctuate due to variations in precipitation, temperature, and other factors. Therefore, groundwater levels at the site at any time may be different than stated in this report.

Review

In the event that any change in the nature, design, or location of the proposed structure(s) is planned, the conclusions and recommendations in this report shall not be considered valid nor relied upon unless the changes are reviewed and the conclusions and recommendations of this report are modified or verified in writing.

Terraphase should be provided the opportunity for a general review of final design plans and specifications to assess that our recommendations have been properly interpreted and included in the design and construction documents.

Construction

To verify conditions presented in this report and modify recommendations based on field conditions encountered in the field, Terraphase should be retained to provide geotechnical engineering services during the construction phase of the project. This is to observe compliance with design concepts, specifications, and recommendations contained in this report, and to verify and refine our recommendations as necessary in the event that subsurface conditions differ from those anticipated prior to the start of construction.

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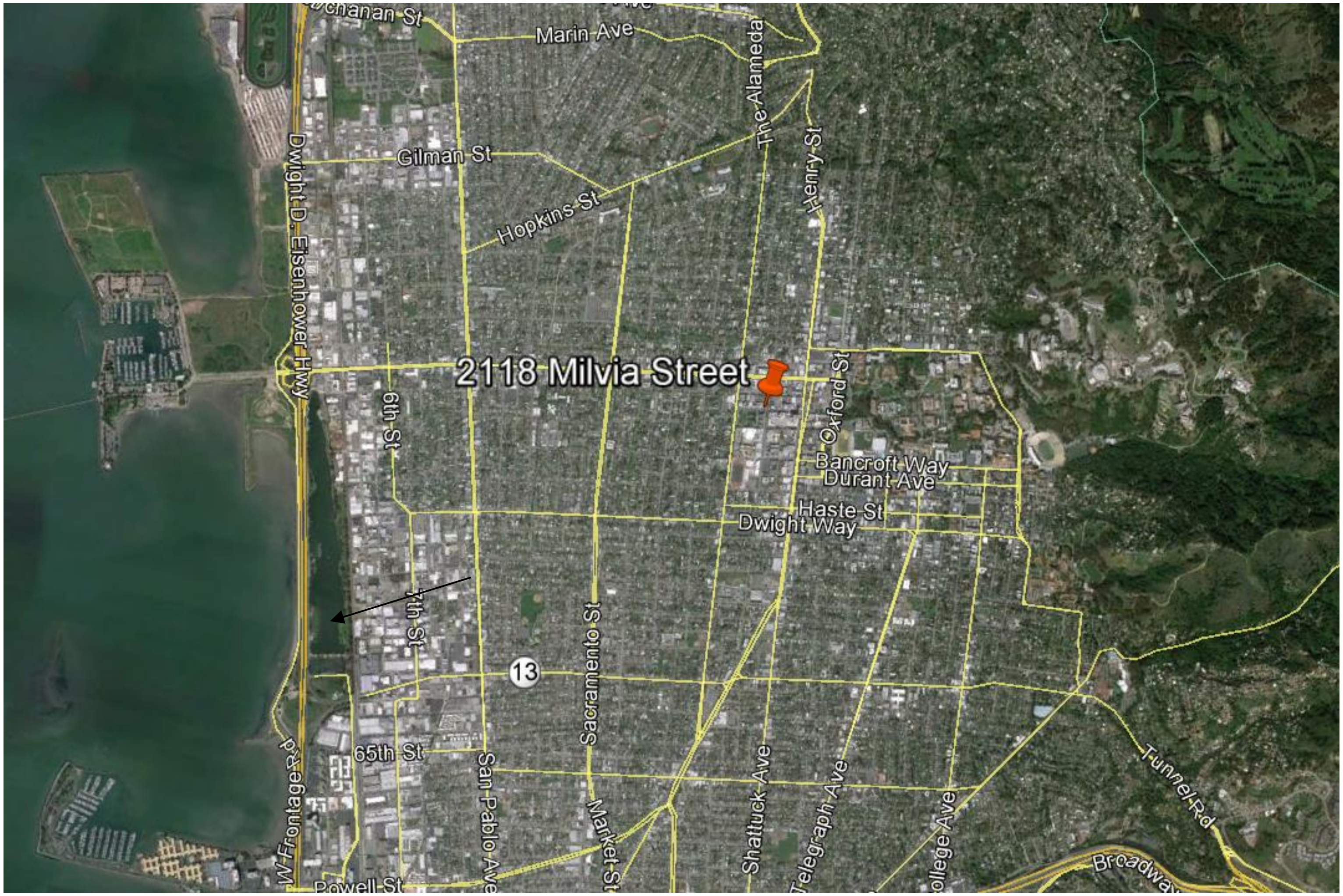


Source: GoogleEarth Professional

N

0' 3,500' 7,000'

SAFETY FIRST	CLIENT: Peralta Community College Dist	SITE LOCATION
terraphase engineering	PROJECT: 2118 Milvia Street	
	PROJECT NUMBER: 0034.001.001	FIGURE 1

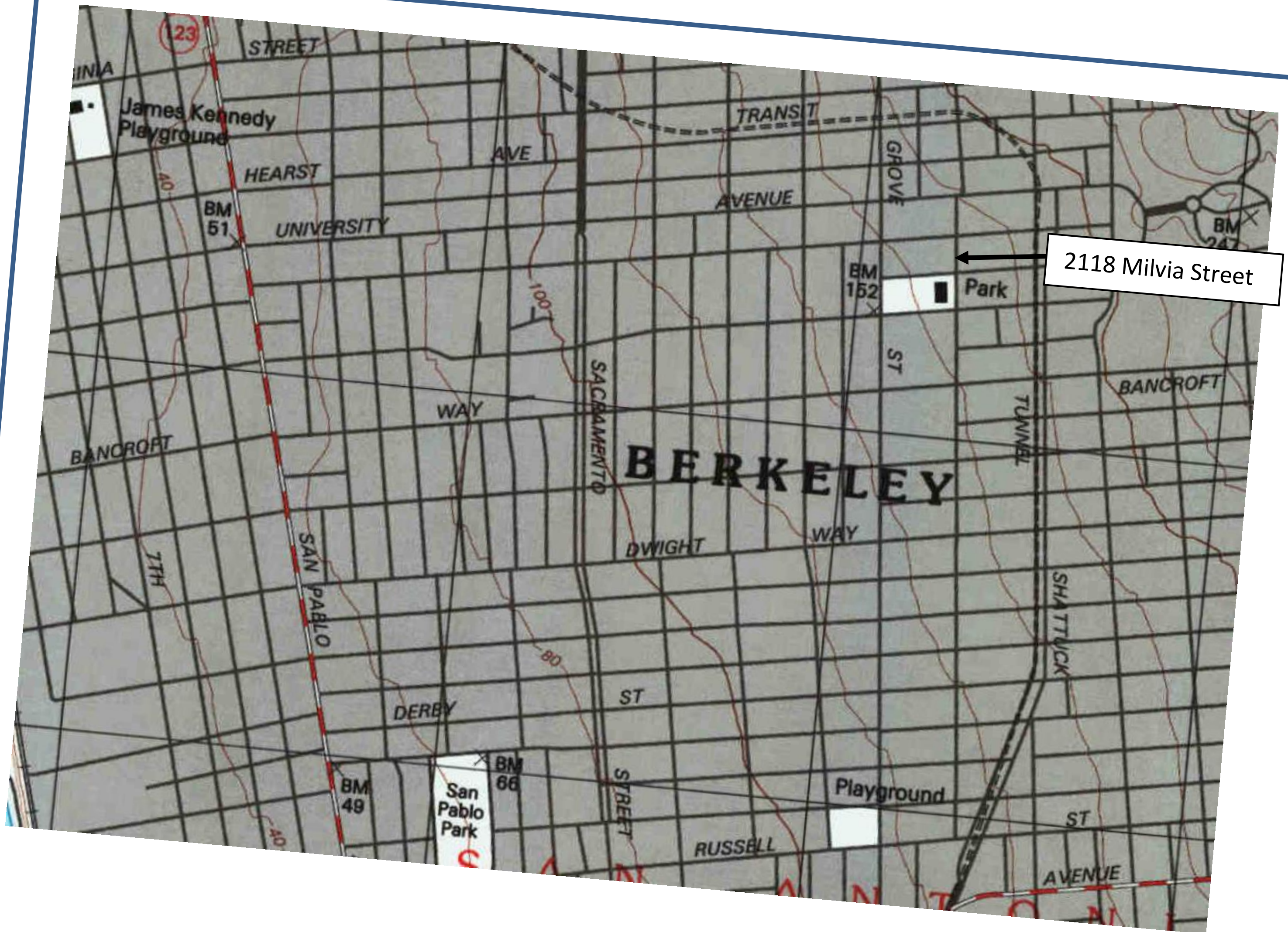
File Name: figure 1 - site location.docx drawn by: jrr checked by: ag

Reference: GoogleEarth Professional

SAFETY FIRST	CLIENT: Peralta Community College District	SITE LAYOUT
terraphase engineering	PROJECT: 2118 Milvia Street, Berkeley	
	PROJECT NO.: 0034.002.001	FIGURE 2

File Name: figure 3 site map.docx drawn by: jrr checked by: em

File Name: figure 2 -topography.docx drawn by: jrr checked by: ag



Source: USGS 7.5-minute quadrangle, Oakland West, 1993

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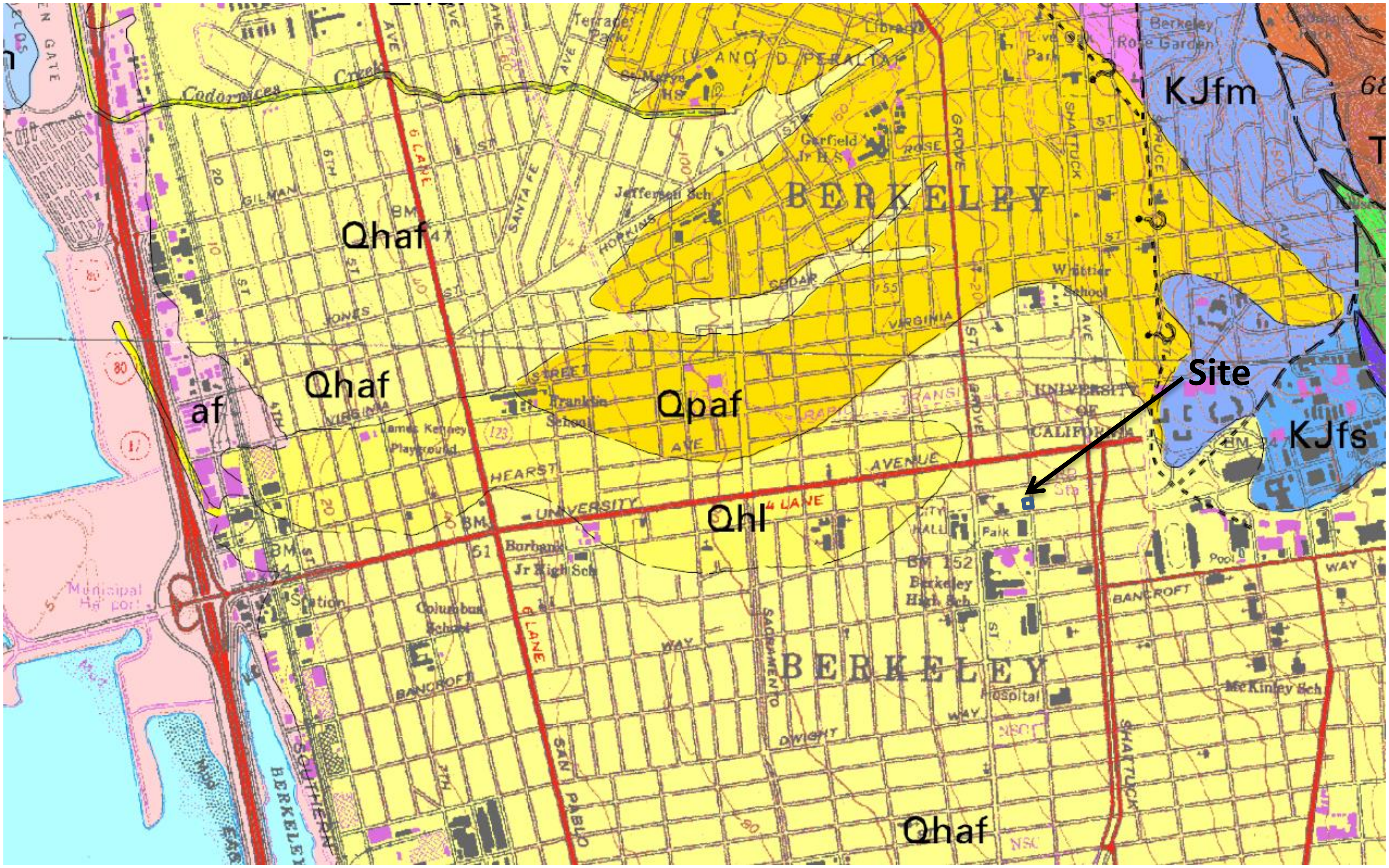
0' 1,500' 3,000'

SAFETY FIRST	CLIENT: Peralta Community College Dist	LOCAL TOPOGRAPHY
terraphase engineering	PROJECT: 2118 Milvia Street	
	PROJECT NUMBER: 0034.001.001	FIGURE 3

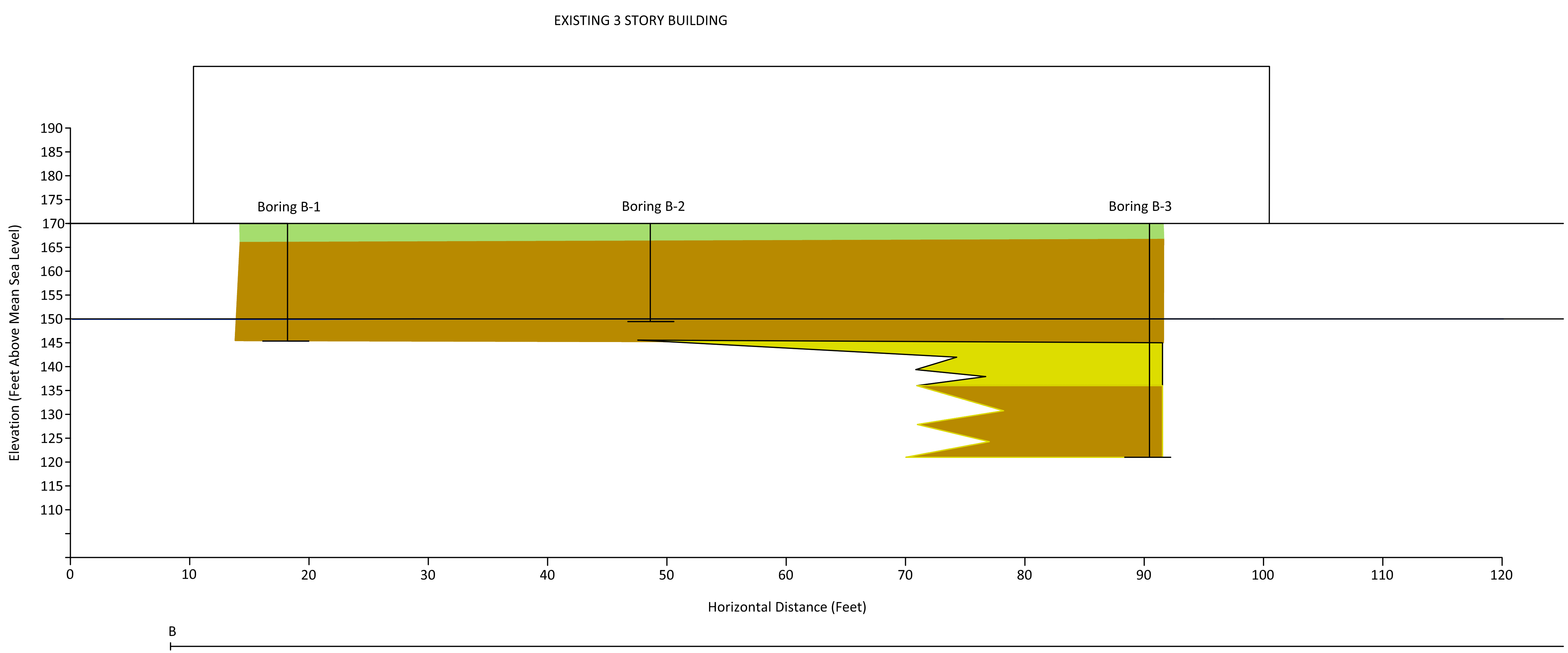


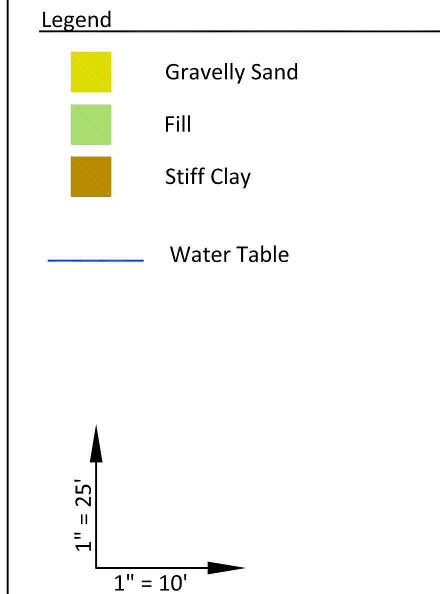
Qhaf	Alluvial fan and fluvial deposits (Holocene)
Qhl	Natural levee deposits (Holocene)
KJfm	Franciscan complex, mélange (Cretaceous Late Jurassic), includes mapped locally: Graywacke and meta-graywacke blocks
Qpaf	Pleistocene) Alluvial fan and fluvial deposits

SAFETY FIRST	CLIENT: Peralta Community College District	G9h[hDL/ ! [a ! t
terraphase engineering	PROJECT: 2118 Milvia Street, Berkeley	
	PROJECT NO.: 0034.002.001	FIGURE 4

File Name: figure 3 geological map.docx drawn by: jrr checked by: em

J:\CADD FILES\0034 Peralta\Cross Section berkeley city college.dwg Drawn by: JRR ; Checked by:____





SAFETY FIRST terraphase engineering	CLIENT: Peralta Community College District PROJECT: 2118 Milvia Street PROJECT NUMBER: 0062.004.001	**Cross-Section A-A'** **FIGURE 5**

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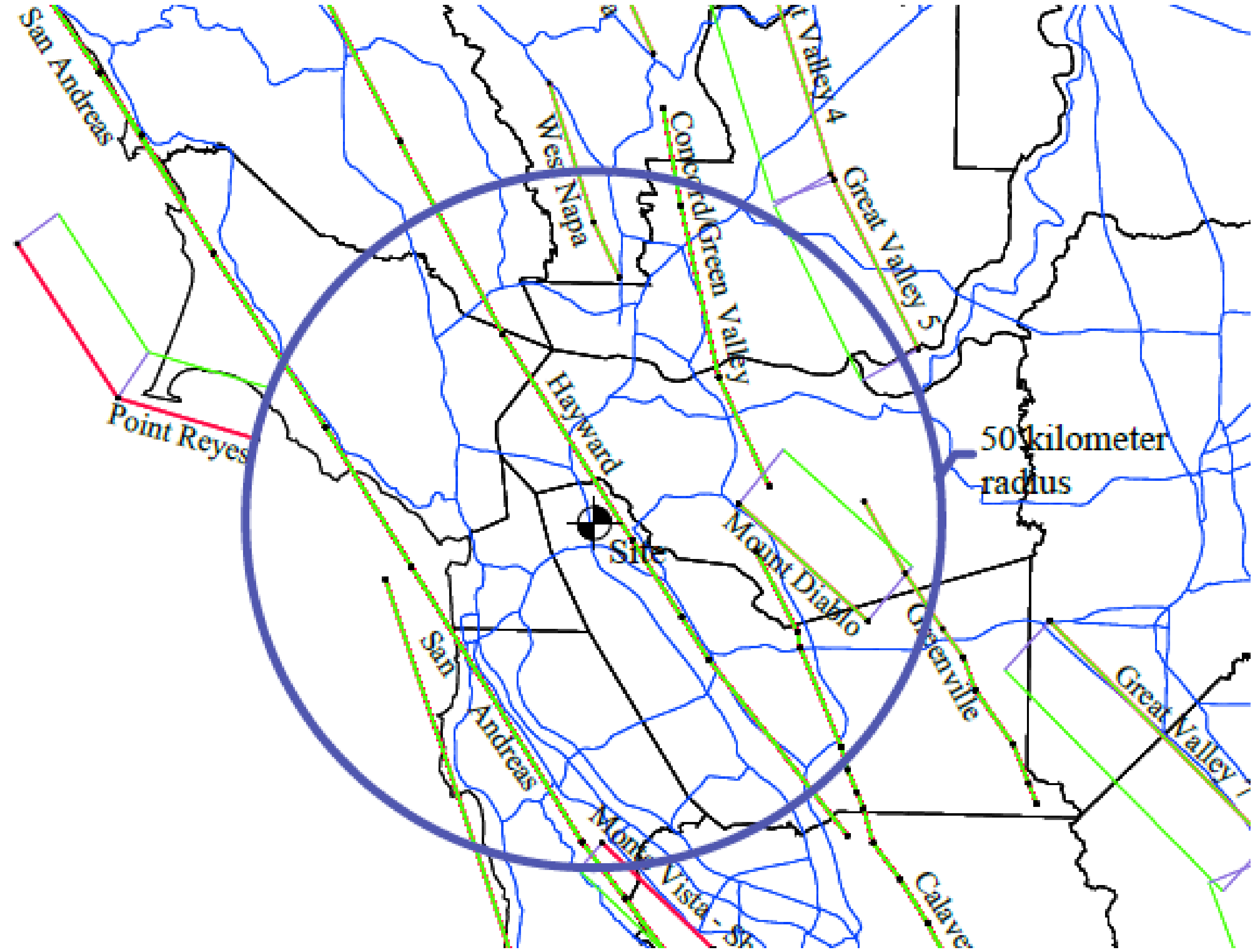


Fault Bend

Fault Trace

Fault Bottom

County Line

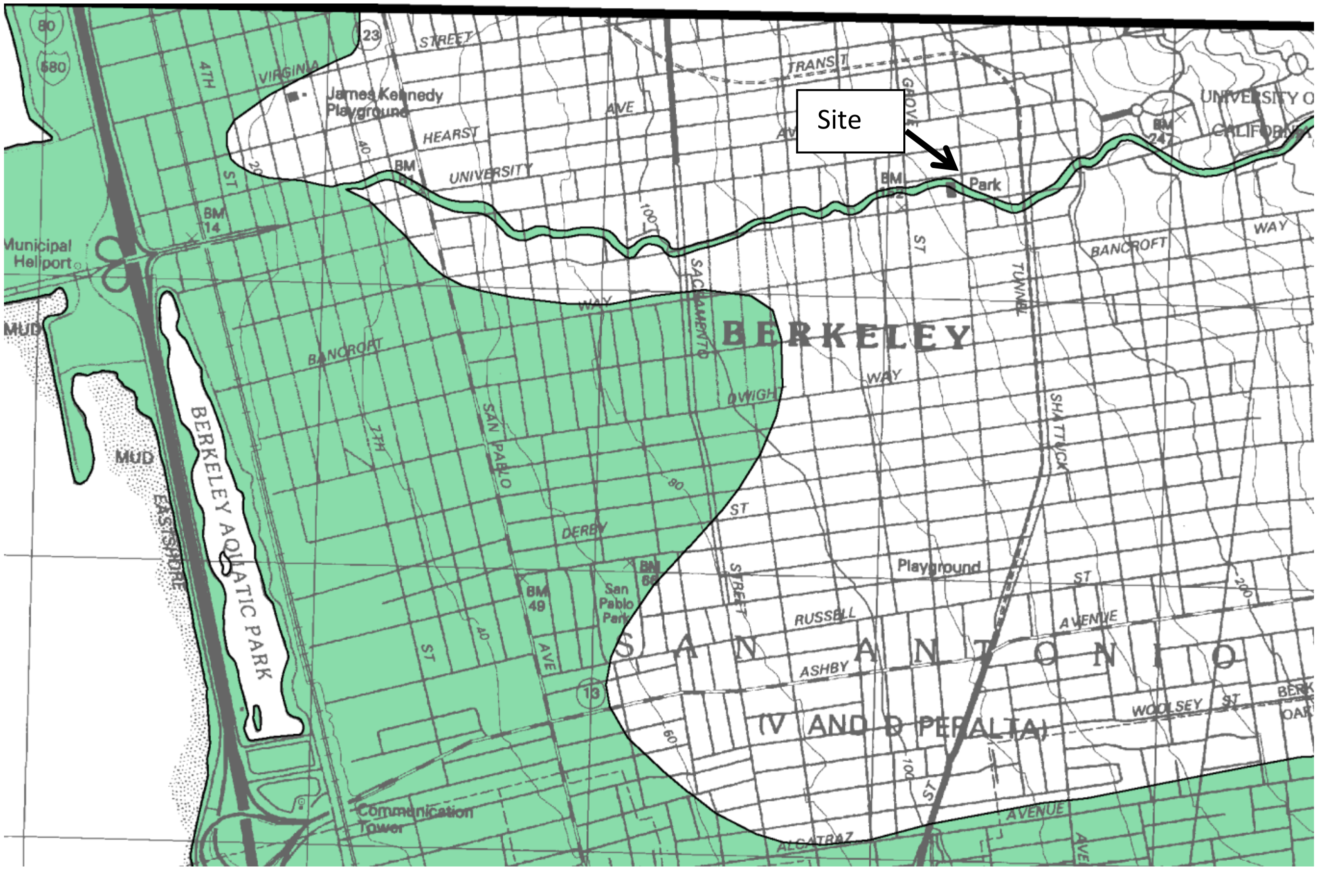
Major Road

N

SAFETY FIRST	CLIENT: Peralta Community College	**FAULT MAP**
terraphase engineering	PROJECT: Berkeley City College 2118 Milvia Street	
	PROJECT NUMBER: 0034-002-001	**FIGURE 6**

File Name: figure 4 - fault map.docx drawn by: jrr checked by: em

File Name: figure 2 liquefaction and landslide hazards.docx drawn by: jrr checked by: ag



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

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

SCALE 1:24,000

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

Source: State of California Seismic Hazard Zones, Oakland West Quadrangle, February 14, 2003

SAFETY FIRST	CLIENT: Peralta Community College	LIQUEFACTION HAZARD
terraphase engineering	PROJECT: Berkeley City College 2118 Milvia Street	
	PROJECT NUMBER: 0034-001-001	FIGURE 7

N

350 ft

Source: FEMA 2009

SAFETY FIRST	CLIENT: Peralta Community College	FLOOD MAP
terraphase engineering	PROJECT: Berkeley City College 2118 Milvia Street	
	PROJECT NUMBER: 0034.001.00	FIGURE 8

File Name: figure 7 - flood map.docx drawn by: jrr checked by: em

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APPENDIX A
EXISTING BUILDING PLAN

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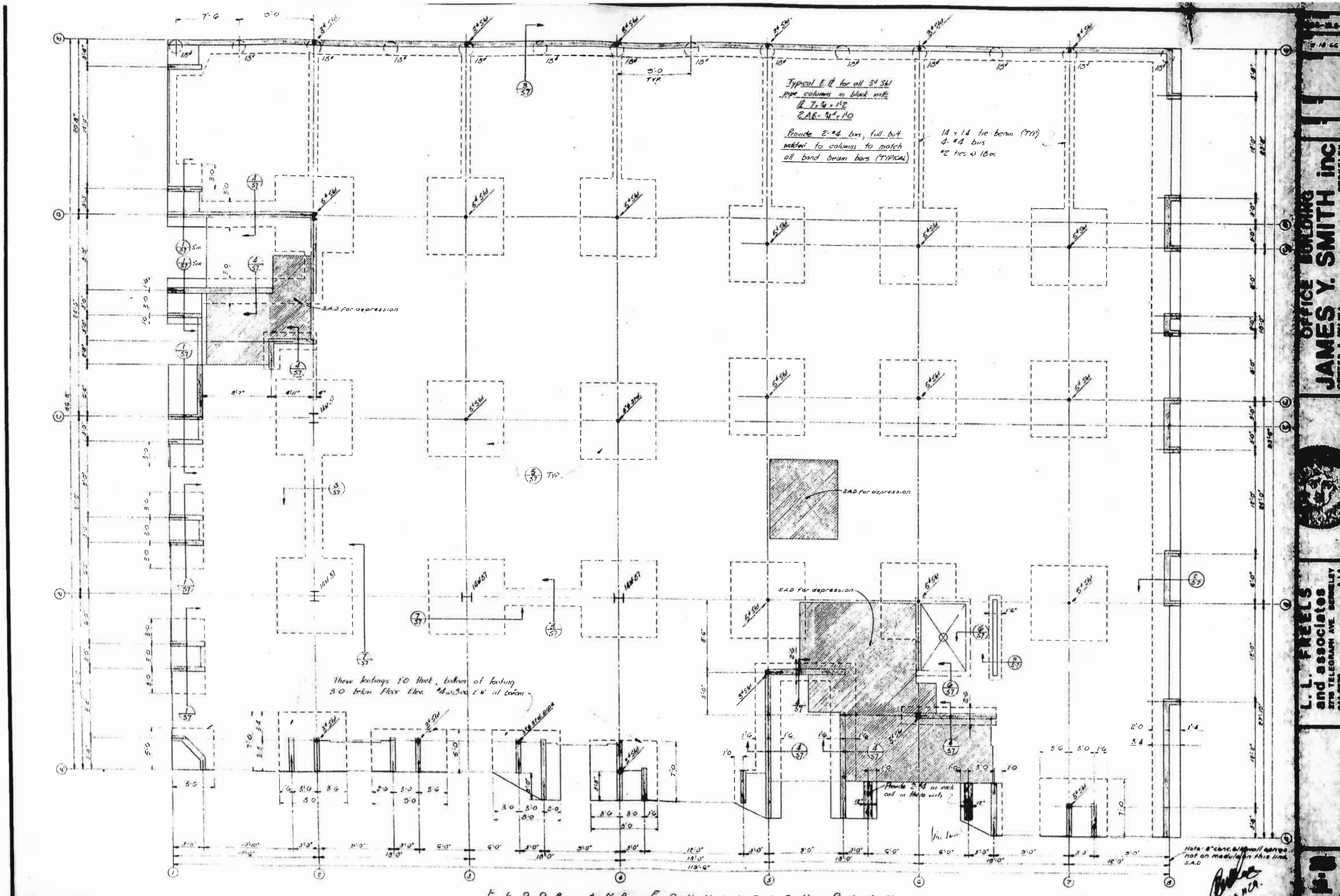
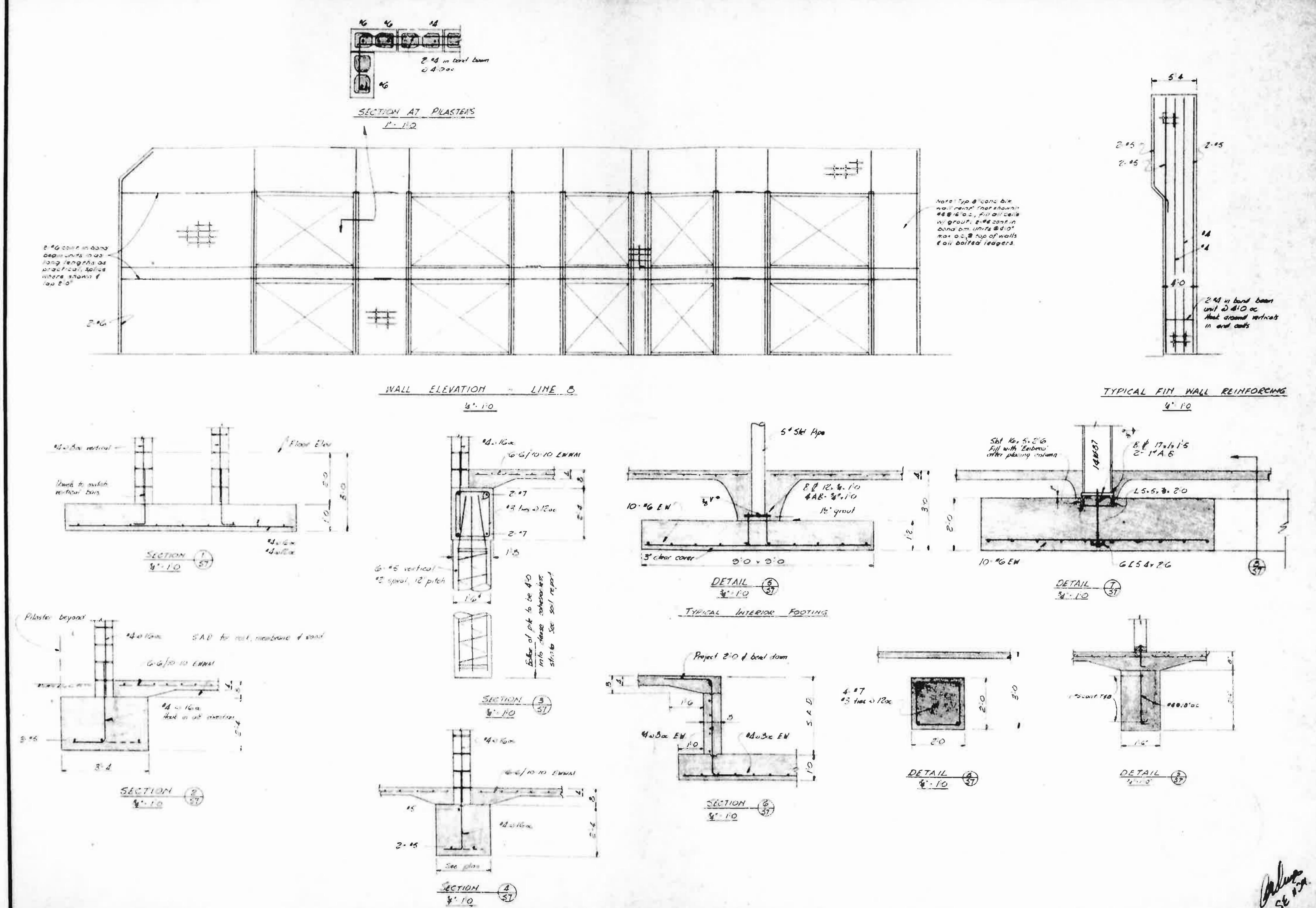


FLOOR AND FOUNDATION PLAN



L. L. FREELS and associates
2718 TELEGRAPH AVE. BERKELEY
841-8670 A.I.A.

OFFICE BUILDING
JAMES Y. SMITH inc
CENTER & MILVIA BERKELEY

APPENDIX B
BORING LOGS

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Project: **2118 Milvia Street, Berkeley California**	Log of Boring 1
Project Location: **Berkeley, California**	**Sheet 1 of 1**
Project Number: **0062.004.001**	

Date(s) Drilled **March 21, 2017**	Logged By **ng**	Checked By **jr**
Drilling Method **Direct Push**	Drill Bit Size/Type **2 inch**	Total Depth of Borehole **25**
Drill Rig Type **Limited Access**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation **170**
Groundwater Level and Date Measured **20**	Sampling Method(s) **Continuous**	Hammer Data **Not Applicable**
Borehole Backfill **Cement Grout**	Location	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
170	0				CL		Brown, clayey-silt (CL), trace gravel, stiff, pp=3.5 tsf	
165	5				CH		Dark brown fat clay (CH), LL=58, PL=20, PI=38; pp= 4.0 tsf	
					CL		Lighter brown color, ~10% sand, some gravel (<3%)	
160	10							
					CL		Same as above, wet, pp=1.0 tsf	
155	15							
150	20							
					CL		Same as above, pp=1.75 tsf	
145	25							
140	30							

C:\Users\Jeff\AppData\Local\Temp\borings_temp\tmpfile.bgs[terraphase.tpl]

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Figure 1

Project: **2118 Milvia Street, Berkeley California**	Log of Boring 2
Project Location: **Berkeley, California**	**Sheet 1 of 1**
Project Number: **0062.004.001**	

Date(s) Drilled **March 21, 2017**	Logged By **ng**	Checked By **jr**
Drilling Method **Direct Push/hand auger 0 to 10 feet**	Drill Bit Size/Type **2 inch**	Total Depth of Borehole **21**
Drill Rig Type **Limited Access**	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation **170**
Groundwater Level and Date Measured **none**	Sampling Method(s) **Continuous/hand auger where refusal**	Hammer Data **Not Applicable**
Borehole Backfill **Cement Grout**	Location	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
170	0				CL		Brown, clayey-silt (CL), trace gravel, stiff	
165	5							
					CL		Hand Auger - Light Brown Silty Clay (CL) pp = 1 tsf	
160	10							
					ML		Sandy silt, trace gravel, low recovery	
155	15				CL		Stiff silty clay, brown and dark brown, gravel 1 inch; turns lighter brown below 17 feet	
150	20				CL		Bottom of Boring	
145	25							
140	30							

J:\Projects\0034 Peralta\Berkeley City College Milvia Site\Technical\Boring Logs\Milvia Boring 1.bg4[master 0 lab).tpl]

Figure 2

Project: **2118 Milvia Street, Berkeley California**	Log of Boring 3
Project Location: **Berkeley, California**	**Sheet 1 of 2**
Project Number: **0062.004.001**	

Date(s) Drilled **March 21, 2017**	Logged By **ng**	Checked By **jr**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8 inch**	Total Depth of Borehole **25**
Drill Rig Type	Drilling Contractor **Gregg Drilling**	Approximate Surface Elevation **170**
Groundwater Level and Date Measured **20**	Sampling Method(s) **SPT and Cal-Mod (all unlined)**	Hammer Data **Safety, 140# falling 30 inches**
Borehole Backfill **Cement Grout**	Location	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
170	0				GW		Road base	
					CL		Dark brown clay (CL), thin sand band at 57"	
165	5		1					
160	10		2	5 ,7, 8	CL		stiff, light brown clay, 8 inch gravel/sand band at 10'-4"	
155	15		3	3, 3, 4	CL		Same as above, softer between 15.5 and 16.5 ft	
150	20		4	9, 10, 19	CL		with gravel, water table at 20 feet bgs	
145	25		5	11, 20, 18	SC		Clayey sand with gravel (19.8% gravel, 59% sand, 21.2% fines)	
					CL		Light brown stiff clay (CL) wet	
140	30							

J:\Projects\0034 Peralta\Berkeley City College Milvia Site\Technical\Boring Logs\Milvia Boring 3.bg4[(master 0 lab).tpl]

Figure 3

Project: **2118 Milvia Street, Berkeley California**
Project Location: **Berkeley, California**
Project Number: **0062.004.001**

Log of Boring 3

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
140	30		6	10, 18, 7	CL		Light brown stiff clay (CL) wet	
					SC		same as above (19% gravel, 64.9% sand, 16.1% fines)	
					CL		very stiff clay (CL), light brown	
135	35		7	16, 12, 15				
130	40		8	4, 10, 11	CL		same as above	
125	45		9	8, 10, 19	CL		Same as above	
120	50		10	7, 15, 16	CL		Same as above, thin band of sand at 49 feet, soft clay at 50 feet	
115	55							
110	60							
105	65							

Figure 3

Project: **2118 Milvia Street, Berkeley California** Project Location: **Berkeley, California** Project Number: **0062.004.001**	# Key to Log of Boring **Sheet 1 of 1**

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9

COLUMN DESCRIPTIONS

1 Elevation (feet): Elevation (MSL, feet).
2 Depth (feet): Depth in feet below the ground surface.
3 Sample Type: Type of soil sample collected at the depth interval shown.
4 Sample Number: Sample identification number.
5 Sampling Resistance, blows/ft: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.
6 Material Type: Type of material encountered.
7 Graphic Log: Graphic depiction of the subsurface material encountered.
8 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
9 REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
Well graded GRAVEL (GW)
Clayey SAND (SC)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Auger sampler
CME Sampler
Pitcher Sample
Bulk Sample
Grab Sample
2-inch-OD unlined split spoon (SPT)
3-inch-OD California w/ brass rings
2.5-inch-OD Modified California w/ brass liners
Shelby Tube (Thin-walled, fixed head)

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)
Water level (after waiting)
Minor change in material properties within a stratum
Inferred/gradational contact between strata
?
Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

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APPENDIX C
LABORATORY TEST RESULTS

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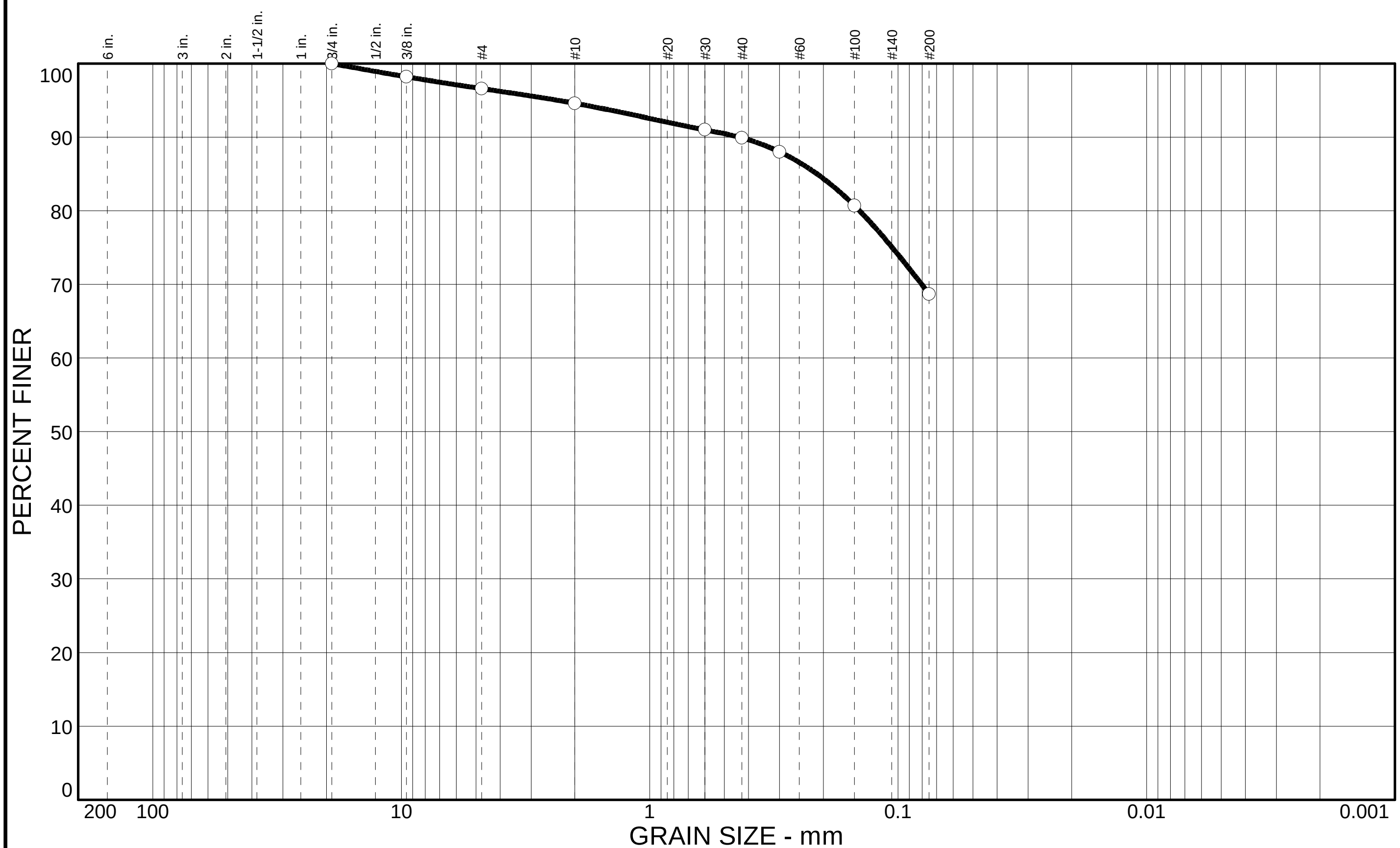
Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		3.4	27.9	68.7		CL		21	39

SIEVE inches size	PERCENT FINER ○		
3/4"	100.0		
3/8"	98.2		
	GRAIN SIZE		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○		
#4	96.6		
#10	94.6		
#30	91.0		
#40	89.9		
#50	88.0		
#100	80.7		
#200	68.7		

SOIL DESCRIPTION

○ Dark Reddish Brown Sandy Lean CLAY

REMARKS:

○

○ Source: 2

Elev./Depth: 1-3'

COOPER TESTING LABORATORY

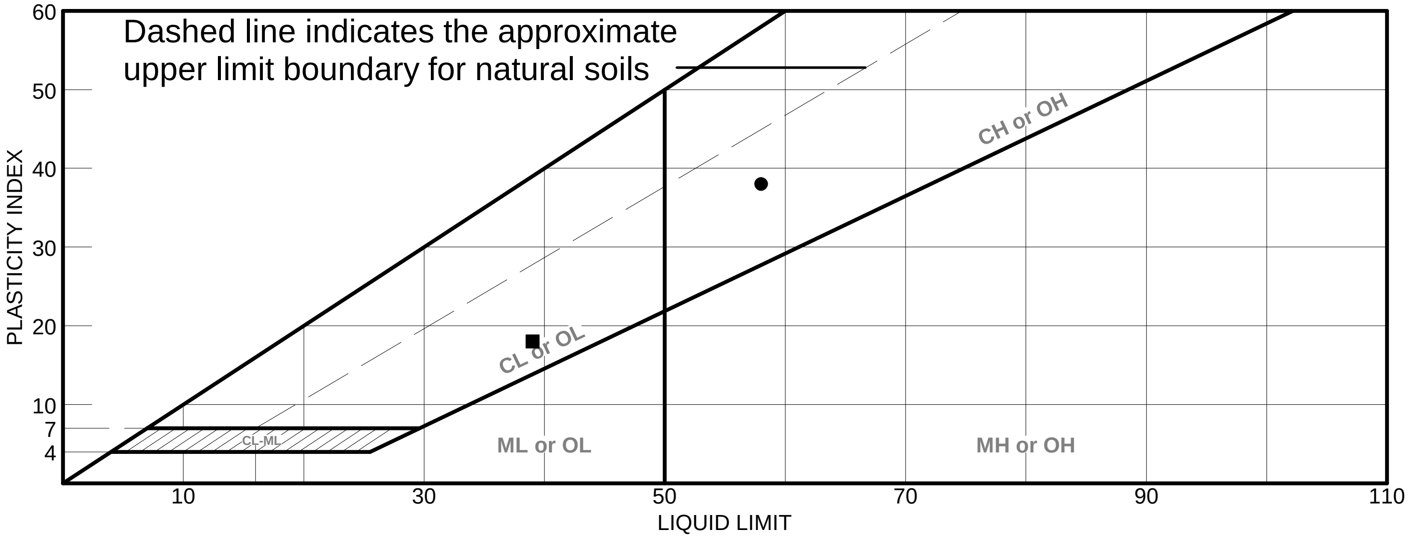
Client: Terraphase

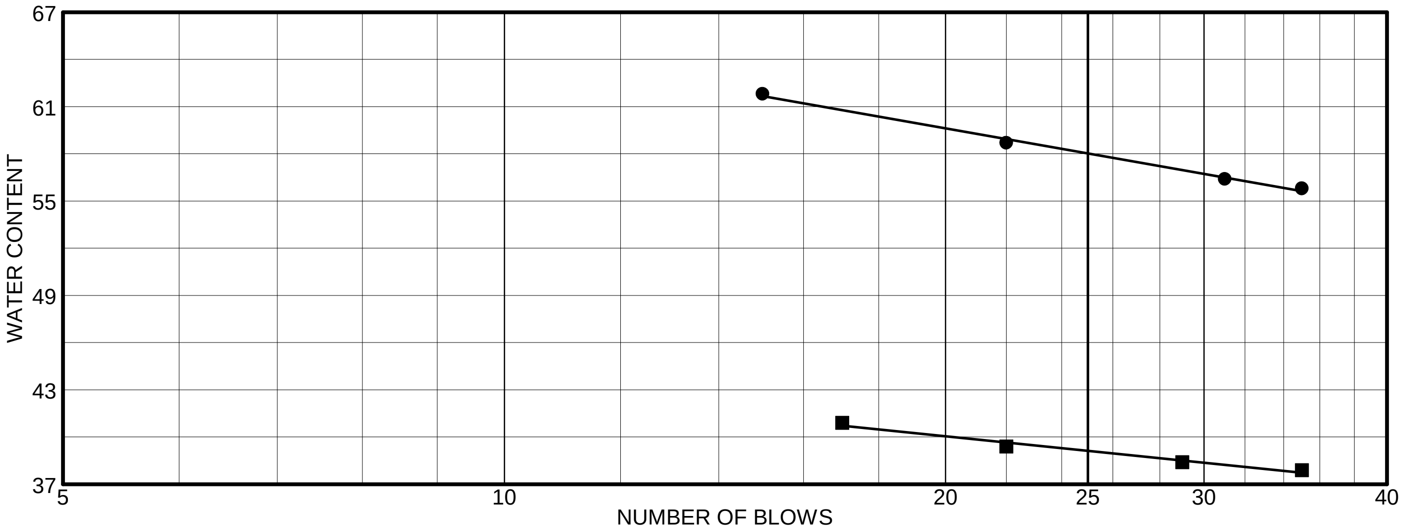
Project: 2118 Milvia - 0062.004.001

Project No.: 734-052

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Olive Brown Fat CLAY w/ Sand	58	20	38			
■	Dark Reddish Brown Sandy Lean CLAY	39	21	18	89.9	68.7	CL

Project No. 734-052 **Client:** Terraphase

Project: 2118 Milvia - 0062.004.001

● **Source:** 1 **Elev./Depth:** 5-6'

■ **Source:** 2 **Elev./Depth:** 1-3'

Remarks:

●

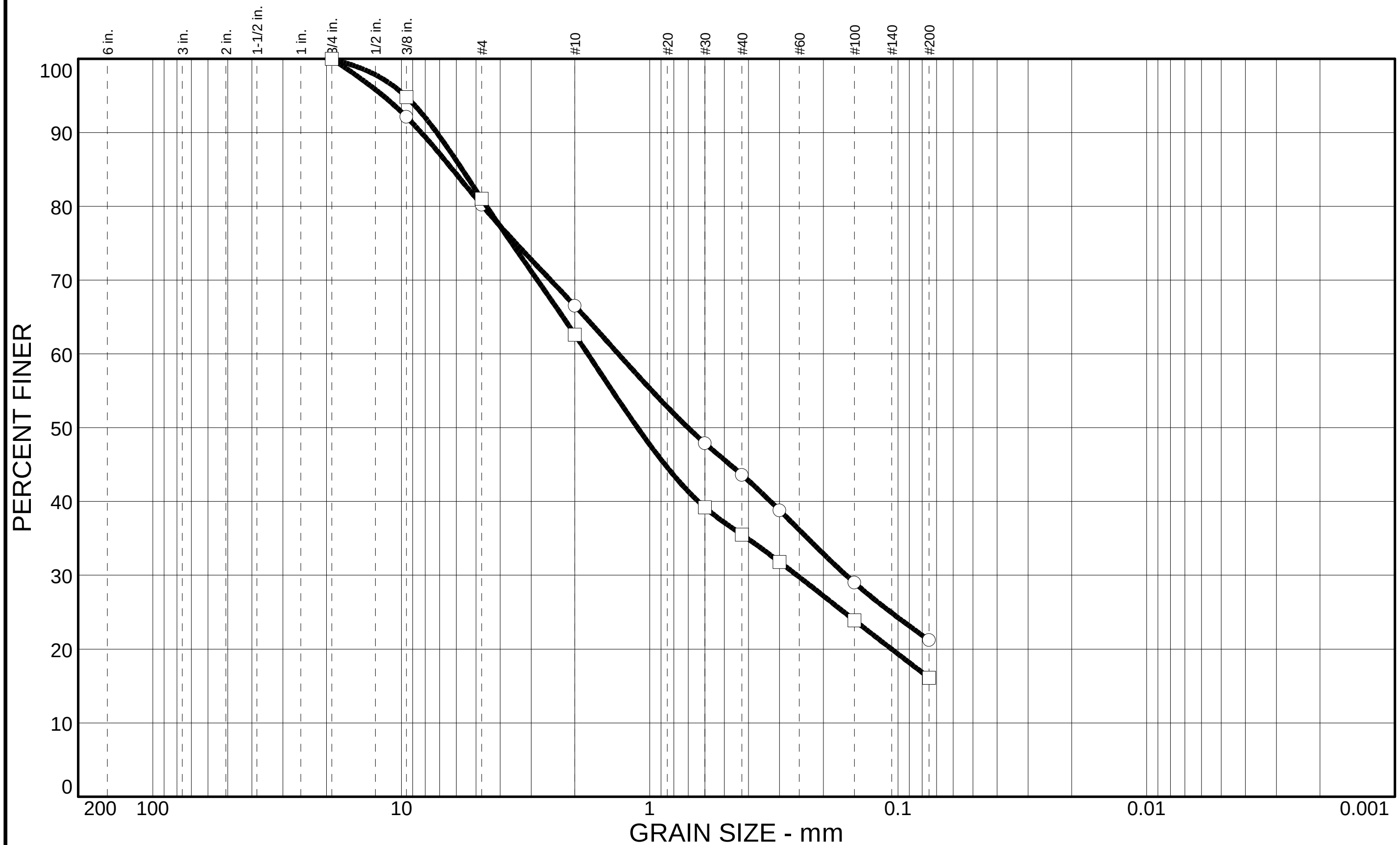
■

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		19.8	59.0	21.2					
□		19.0	64.9	16.1					

SIEVE inches size	PERCENT FINER ○	□	
3/4"	100.0	100.0	
3/8"	92.1	94.8	
	GRAIN SIZE		
D_{60}	1.34	1.78	
D_{30}	0.162	0.255	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	
#4	80.2	81.0	
#10	66.5	62.6	
#30	47.9	39.2	
#40	43.6	35.5	
#50	38.8	31.8	
#100	29.0	23.9	
#200	21.2	16.1	

SOIL DESCRIPTION

○ Olive Brown Clayey SAND w/ Gravel

□ Olive Clayey SAND w/ Gravel

REMARKS:

○

□

○ Source: B3 Elev./Depth: 26.3'

□ Source: B3 Elev./Depth: 31.3'

COOPER TESTING LABORATORY

Client: Terraphase

Project: Berkeley City College - 0062-004-001

Project No.: 734-053

Figure

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APPENDIX D
SITE SPECIFIC SEISMIC HAZARD ASSESSMENT

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DRAFT

May 17, 2017

Merideth Marschak AIA, CSI, LEED AP
Noll & Tam Architects
729 Heinz Ave. #7
Berkeley, CA 94710

sent via email to: merideth.marschak@nollandtam.com

Subject: Site-Specific Seismic Risk Assessment, 2118 Milvia Street, Berkeley, California

Dear Ms. Marschak:

This letter report contains a site-specific seismic risk assessment for the proposed seismic retrofit of the existing structure located at 2118 Milvia Street, Berkeley, California (the "Site", Figure 1). This letter report supplements the Geotechnical Investigation report for the Building currently in development.

1.0 CBC SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapters 16A and 18A of the 2016 California Building Code (CBC), Chapters 11 and 21 of ASCE 7-10 and Chapter 2 of ASCE 41-13.

1.1 SITE CLASSIFICATION

Subsurface investigation of the Site indicates that it falls into Soil Class D (Stiff Soil). The USGS shear wave velocity maps (https://earthquake.usgs.gov/data/vs30/us/) indicate the shear wave velocity in the top 30 meters at the Site have an average shear wave velocity of 330 meters per second (m/s).

1.2 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum Considered Earthquake (MCE_R) spectral acceleration parameters at short periods and at 1 second period (S_S and S_1) were calculated using the USGS Seismic Design Maps on-line hazard calculator (USGS 2013).

The mapped acceleration parameters were adjusted for local site conditions based on the average soils conditions for the upper 30 meters of the soil profile. MCE spectral response acceleration parameters adjusted for site effects (S_{Ms} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table 1. These are equal to the ASCE-41 BSE-2N and BSE-1N spectra. The USGS Seismic Design Maps on-line hazard calculator also provides the ASCE-41 spectra (BSE-2E and BSE-1E) which are also presented in Table 1.

In accordance with CBC Section 1613A.3.5, Risk Category I, II, or Ill structures with mapped spectral response acceleration parameter at the 1-second period (S_1) greater than 0.75, are assigned Seismic Design Category E. In accordance with CBC 1616A.1.3, Seismic Design Category E structures require a site-specific ground motion hazard analysis performed in accordance with ASCE 7 chapter 21 and section

1803A.6 of the California Building Code. Therefore, the values in Table 1 should not be used for design. Values are provided only for determination of Seismic Design Category and comparison with minimum code requirements in our site-specific ground motion hazard analysis.

2.0 SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

We performed a site-specific hazard analysis in accordance with ASCE 7-10 Chapter 21.2 and 2013 CBC Section 1803A.6. Our analyses were performed using the computer program EZFrisk, version 7.65, Build 4 (Risk Engineering, 2012) and the 2008 USGS fault model (Petersen, et al. 2008).

Our analysis utilized the mean ground motions predicted by three of the Next Generation Attenuation (NGA) relationships: Boore and Atkinson (2008), Campbell-Bozorgnia (2008), Chiou-Youngs (2007), and Abrahamson-Silva (2007). Our analysis used the FEMA P-750 (2009) method for calculating the maximum rotated component of ground motions, which is based on Huang et al. (2008).

2.1 Deterministic MCE_R

We performed deterministic seismic hazard analyses in accordance with ASCE 7-10 Section 21.2.2. The deterministic MCE_R acceleration response spectrum is defined as the largest 84th percentile ground motion in the direction of maximum horizontal response for each period of characteristic earthquakes on all known active faults within the region. Our analysis considered all known active faults within 170 kilometers of the site.

The 84th percentile ground motion in the direction of maximum horizontal response for this event is presented on Figure 2. Spectral ordinates are tabulated in Table 2, Column 4. ASCE 7-10 specifies that the deterministic MCE_R shall not be less than the Deterministic Lower Limit MCE response spectrum (ASCE 7-10 Figure 21.2-1). The Deterministic Lower Limit spectrum is presented on Figure 2. Spectral ordinates are tabulated in Table 2, Column 5.

The deterministic MCE_R spectrum was calculated by taking the greater of Table 2, Columns 4 and 5. Spectral ordinates for the deterministic MCE_R are tabulated in Table 2, Column 6. The deterministic MCE_R is presented graphically on Figure 2.

2.2 Probabilistic MCE_R

We performed a probabilistic seismic hazard analysis (PSHA) in accordance with ASCE 7-10 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50-year period (2,475-year return period). Our PSHA considered all known active faults within 170 kilometers of the site as well as a gridded seismic source modeled by the USGS (2008) which governed risk for spectral periods less than 0.75 seconds. The rotated probabilistic MCE_R spectrum was multiplied by Risk Coefficients (CR) to determine the uniform risk probabilistic MCE_R. We used Risk Coefficients (CRs and CR1) of 1.008 and 0.984, respectively, based on ASCE 7-10 Section 21.2.1.1 - Method 1 and the USGS on-line calculator.

The resulting probabilistic MCE_R is presented on Figure 2. Spectral ordinates for the uniform risk probabilistic spectra are tabulated in Table 2, Column 11.

2.3 Site-Specific MCE_R

The site-specific MCE_R is defined by ASCE 7-10 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE_R's at each period. The site-specific MCE_R spectrum was calculated by taking the lesser of the deterministic MCE_R (Table 2, Column 6, MCE, Figure 2) and the probabilistic MCE_R (Table 5, Column 11, Figure 2). Spectral ordinates for the site-specific MCE_R are tabulated in Table 2, Column 12 and shown graphically on Figure 3. The deterministic spectrum governed for every spectral period (the site is only 1.7 kilometers from the Hayward Fault).

The site-specific Design Response Spectrum (DRS) is defined in ASCE 7-10 Section 21.3 as 2/3^{rds} of the site-specific MCE_R, spectrum but not less than 80% of the general design response spectrum. Spectral accelerations corresponding to the 2/3^{rds} of the MCE_R are tabulated in Table 2, Column 14. Ordinates corresponding to 80% of the general response spectrum are tabulated in Table 2 Column 15. Ordinates of the site-specific DRS are tabulated in Table 2, Column 16. Development of the site-specific DRS is presented graphically on Figure 3.

2.4 DESIGN ACCELERATION PARAMETERS

Site-specific design acceleration parameters (S_{DS} and S_{D1}) were determined in accordance with Section 21.4 of ASCE 7-10. S_{DS} is defined as the design spectral acceleration at a period of 0.2 seconds, but not less than 90% of the spectral acceleration at any period greater than 0.2 seconds. S_{D1} is defined as the greater of the design spectral acceleration at a period of 1 second or two times the spectral acceleration at a period of 2 seconds.

Site-specific MCE spectral response acceleration parameters (S_{MS} and S_{M1}) are calculated as 1.5 times the S_{DS} and S_{D1} values, respectively, but not less than 80% of the code-based values presented in Table 1, Column 15. Site-specific design acceleration parameters are summarized below.

S_{DS} = 1.401

S_{D1} = 1.27

S_{MS} = 2.10

S_{M1} = 1.91

When using the Equivalent Lateral Force Procedure, ASCE 7-10 Section 21.4 allows using the spectral acceleration at the building fundamental period (T) in lieu of S_{D1}/T in Eq. 12.8-3. The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table 2, Column 16.

3.0 SEISMIC PARAMETERS FOR ASCE/SEI 41

3.1 General

The spectra for ASCE/SEI 41-13 are:

- BSE-2N (equal to MCE_R of ASCE/SEI 7-10)

- BSE-1N (equal to 2/3rds times MCE_R (Design level) of ASCE/SEI 7-10)
- BSE-2E (equal to 5% probability of exceedance in 50 years ground motion level – 974-year return period)
- BSE-1E (equal to 20% probability of exceedance in 50 years ground motion level – 224-year return period)

3.2 USGS Tool

In accordance with the 2016 CEBC and ASCE/SEI 41, the following seismic design parameters may be used for the project. The values of Ss, S1, Fa, and Fv used in development of the site-adjusted Basic Safety Earthquake (BSE) spectral parameters (described below) are obtained from the USGS online tool, U.S. Seismic Design Maps (http://earthquake.usgs.gov/hazards/designmaps/usdesign.php). The values of Fa and Fv are for Site Class D. For ASCE/SEI 41, the site-adjusted short and long period spectral parameters are referred to as Sxs and Sx1 , respectively.

$S_{XS,BSE\text{-}2N} = FaS_{S,BSE\text{-}2N} = 1.000 \times 2.318$ g = 2.318 g

$S_{X1,BSE\text{-}2N} = Fv_{S1,BSE\text{-}2N} = 1.500 \times 0.963$ g = 1.445 g

$S_{XS,BSE\text{-}2E}$ = 2.317 g

$S_{X1,BSE\text{-}2E}$ = 1.313 g

3.3 BSE-2N and BSE-1N

See Section 2.0

3.4 BSE-2E

The 5% probability of exceedance in 50 years ground motion level – 974-year return period – spectra is presented in Table 2. The spectra values were multiplied by 1.1 at periods less than 0.2 seconds and 1.3 at periods greater than 1.0, with linearly interpolated values between 0.2 and 1.0 seconds, to obtain the maximum rotated component. The spectral values were capped at the MCE_R values.

S_{XS} = spectral acceleration at 0.2 seconds (not less than 90% of higher spectral values) = 2.10g
S_{X1} = larger of spectral acceleration at 1 second or twice that at 2 seconds = 1.42g
$T_0 = 0.2*S_{X1} / S_{XS}$ = 0.135 seconds
$T_S = S_{X1} / S_{XS}$ = 0.676 seconds

3.5 BSE-1E

The 20% probability of exceedance in 50 years ground motion level – 224-year return period – spectra is presented in Table 3. The spectra values were multiplied by 1.1 at periods less than 0.2 seconds and 1.3 at periods greater than 1.0, with linearly interpolated values between 0.2 and 1.0 seconds, to obtain the maximum rotated component. The spectral values were capped at the MCE_R values.

S_{XS} = spectral acceleration at 0.2 seconds (not less than 90% of higher spectral values) = 1.22g
S_{X1} = larger of spectral acceleration at 1 second or twice that at 2 seconds = 0.8g

$T_0 = 0.2*S_{X1} / S_{XS} = 0.131$ seconds
$T_S = S_{X1} / S_{XS} = 0.656$ seconds

3.6 Vertical Spectra

If a vertical spectra is required, Chapter 23 of FEMA (2009) recommends:

Period < 0.025 seconds (sec): $S_{aV} = 0.3C_V*S_{DS}$

0.025 sec < Period < 0.05 sec: $S_{aV} = 20*C_V*S_{DS}(Tv-0.025)+ 0.3C_V*S_{DS}$

0.05 sec < Period < .15 sec: $S_{aV} = 0.8C_V*S_{DS}$

0.15 sec < Period < 2 sec: $S_{aV} = 0.8C_V*S_{DS}*(0.15/Tv)$^0.75

$S_{DS} = 1.401$

$C_V = 1.5$

The resulting vertical spectrum is presented in Table 4.

The vertical spectrum calculated above is less than 2/3rds of the design spectrum (MCER) at periods greater than 0.5 seconds. We have included the 2/3rds of the design spectrum in Table 4 and recommend using that value for periods greater than 0.5 seconds.

4.0 MCE$_G$ PEAK GROUND ACCELERATION

We calculated the MCE Geometric Mean Peak Ground Acceleration (MCE$_G$) in accordance with ASCE 7-10 Section 21.5. The MCE$_G$ is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The 2% in 50-year probabilistic geometric mean PGA is 1.13g. The deterministic MCE$_G$ is considered the greater of the largest 84th percentile deterministic geometric mean PGA (0 .90g) or one-half of the tabulated F_{PGA} value from ASCE 7- 10 Table 11.8.1. For the site, F_{PGA} is 1.0g and one half of the F_{PGA} is 0.50g; therefore, the deterministic MCE$_G$ is 0.90 g. Additionally, the MCE$_G$ may not be less than 80% of the mapped PGA_M determined from ASCE -10 Equation 11.8-1. The PGA_M for the site is 0.89g; 80% of PGAM is 0.71g. Therefore, the MCE$_G$ for the site may be considered 0.90g.

5.0 REFERENCES

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Boore, D.M., and Atkinson, G.M., 2008, Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01 s and 10.0 s, Earthquake Spectra, Vol. 24 (1): 99-138.

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Campbell, K.W., and Bozorgnia, Y., 2008, NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD, and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01to10.0 S, Earthquake Spectra Vol. 24 (1): 139-171 .

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USGS, 2013, U.S. Seismic Design Maps, revision date April 2, 2013 - A Computer Program for determining mapped ground motion parameters for use with IBC 2006 available at https://earthquake.usgs.gov/designmaps/us/application.php .

6.0 CLOSING

Terraphase is grateful for the opportunity to offer our services on this important project. If you have any question or comments regarding this submittal, please contact Jeff Raines at (510) 507-3086.

Sincerely,
For Terraphase Engineering Inc.

Jeff Raines, P.E. (C51120), G.E. (2762)
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JEFFERY R RAINES
2762
EXP 9-30-17
GEOTECHNICAL
STATE OF CALIFORNIA

Christopher Alger, P.G. (5020), E.G. (1564)
Principal Geologist

CERTIFIED ENGINEERING GEOLOGIST
CHRISTOPHER S. ALGER
NO. 1564
exp 2/19
STATE OF CALIFORNIA

Attachments:

Table 1 – Mapped Acceleration Parameters
Table 2 – Site-specific Acceleration Parameters
Table 3 - BSE-2E Spectra
Table 4 - BSE-1E Spectra
Table 5 – Recommended Vertical Spectra

Figure 1 – Site Location
Figure 2 – Probabilistic & Deterministic Spectra
Figure 3 - MCER and Design Spectra

Attachment A – EZ-Frisk Output

Table 1
Mapped Acceleration Parameters
2118 Milvia Street, Berkeley, California,

Item	Value
Geographic Region	48 Conterminous States
Data Edition=	2010 ASCE 7 Standard
Longitude	122.27076° W
Latitude	37.8701° N
S_S	2.317g
S_1	0.963g
Fa	1.0
Fv	1.5
S_{MS} Fa × SS 0.9 × 1.901	2.317g
S_{M1} Fv × S1 2.4 × 0.766	1.444g
S_{DS} =(2/3) × S_{MS} (2/3) × 2.317 Section 1613.5.4	1.544g
S_{D1} = (2/3) × S_{M1} (2/3) × 1.444 Section 1613.5.1	0.963g
TL	8 seconds
PGA	0.89g
PGA_M = F_{PGA}PGA – 1.0*0.89g = 0.89 g (used for liquefaction analysis)	0.89 g
Seismic Design Category	E
CRS	1.008
CR1	0.984
$S_{XS,BSE2E}$	2.317
$S_{X1,BSE2E}$	1.313
T_0	0.113
T_S	0.567
B_1	1.0
$S_{XS,BSE1E}$	1.245
$S_{X1,BSE1E}$	0.699
T_0	0.112
T_S	0.561
B_1	1.0

Table 2
Site-specific Spectral Acceleration Parameters
2118 Milvia Street, Berkeley, California,

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Period	**Deterministic 84% (g)**	**multiplier**	**Max Rotated deterministic (g)**	**Lower Limit (g)**	**Deterministic (g)**	**Probabilistic (2% in 50 years) (g)**	**multiplier**	**Max Rotated Probabilistic (g)**	**Uniform Risk Factor Cr**	**Uniform Risk Probabilistic (g)**	**Site-specific MCE_R (g)**	**General Response Spectrum (g)**	**Design Spectrum (g)**	**80% General Response Spectrum (g)**	**Design Spectrum (final) (g)**
0	0.896	1.100	0.986	0.600	0.986	1.13E+00	1.100	1.247	1.008	1.257	0.986	0.618	0.657	0.494	0.657
0.05	1.117	1.100	1.229	0.975	1.229	1.44E+00	1.100	1.579	1.008	1.591	1.229	0.988	0.819	0.791	0.819
0.1	1.501	1.100	1.651	1.350	1.651	2.05E+00	1.100	2.251	1.008	2.269	1.651	1.359	1.101	1.087	1.101
0.120	1.583	1.100	1.741	1.500	1.741	2.14E+00	1.100	2.356	1.008	2.375	1.741	1.507	1.161	1.206	1.206
0.125	1.603	1.100	1.764	1.500	1.764	2.17E+00	1.100	2.383	1.008	2.402	1.764	1.544	1.176	1.235	1.235
0.2	1.910	1.100	2.101	1.500	2.101	2.53E+00	1.100	2.780	1.008	2.802	2.101	1.544	1.401	1.235	1.401
0.215	1.925	1.104	2.124	1.500	2.124	2.535	1.104	2.798	1.008	2.819	2.124	1.544	1.416	1.235	1.416
0.3	2.008	1.125	2.259	1.500	2.259	2.59E+00	1.125	2.909	1.005	2.924	2.259	1.544	1.506	1.235	1.506
0.4	1.972	1.150	2.268	1.500	2.268	2.54E+00	1.150	2.916	1.002	2.922	2.268	1.544	1.512	1.235	1.512
0.5	1.871	1.175	2.198	1.500	2.198	2.40E+00	1.175	2.818	0.999	2.815	2.198	1.544	1.466	1.235	1.466
0.6	1.760	1.200	2.112	1.500	2.112	2.24E+00	1.200	2.690	0.996	2.679	2.112	1.544	1.408	1.235	1.408
0.624	1.733	1.206	2.090	1.442	2.090	2.20E+00	1.206	2.658	0.995	2.646	2.090	1.544	1.393	1.235	1.393
0.75	1.593	1.238	1.971	1.200	1.971	2.01E+00	1.238	2.484	0.992	2.463	1.971	1.284	1.314	1.027	1.314
1	1.309	1.300	1.702	0.900	1.702	1.57E+00	1.300	2.045	0.984	2.012	1.702	0.963	1.134	0.770	1.134
1.5	1.021	1.300	1.327	0.600	1.327	1.207	1.300	1.569	0.984	1.544	1.327	0.642	0.885	0.514	0.885
2	0.733	1.300	0.952	0.450	0.952	8.41E-01	1.300	1.093	0.984	1.075	0.952	0.482	0.635	0.385	0.635
3	0.476	1.300	0.618	0.300	0.618	5.21E-01	1.300	0.677	0.984	0.667	0.618	0.321	0.412	0.257	0.412
4	0.340	1.300	0.442	0.225	0.442	3.66E-01	1.300	0.476	0.984	0.468	0.442	0.241	0.295	0.193	0.295

Table 3
BSE-2E Spectra
2118 Milvia Street, Berkeley, California,

Period	Probabilistic (5% in 50 years) (g)	Multiplier	Maximum Rotated (g)	MCE_R (g)	BSE-2E (g)
PGA	8.92E-01	1.100	0.981	0.986	0.981
0.05	1.12E+00	1.100	1.227	1.229	1.227
0.1	1.51E+00	1.100	1.665	1.651	1.651
0.2	1.95E+00	1.100	2.141	2.101	2.101
0.3	1.98E+00	1.125	2.228	2.259	2.228
0.4	1.92E+00	1.150	2.206	2.268	2.206
0.5	1.78E+00	1.175	2.095	2.198	2.095
0.75	1.43E+00	1.238	1.765	1.971	1.765
1	1.16E+00	1.300	1.505	1.702	1.505
2	6.06E-01	1.300	0.788	0.952	0.788
3	3.74E-01	1.300	0.486	0.618	0.486
4	2.60E-01	1.300	0.338	0.442	0.338

BSE-2E spectra is capped at the MCE_R acceleration

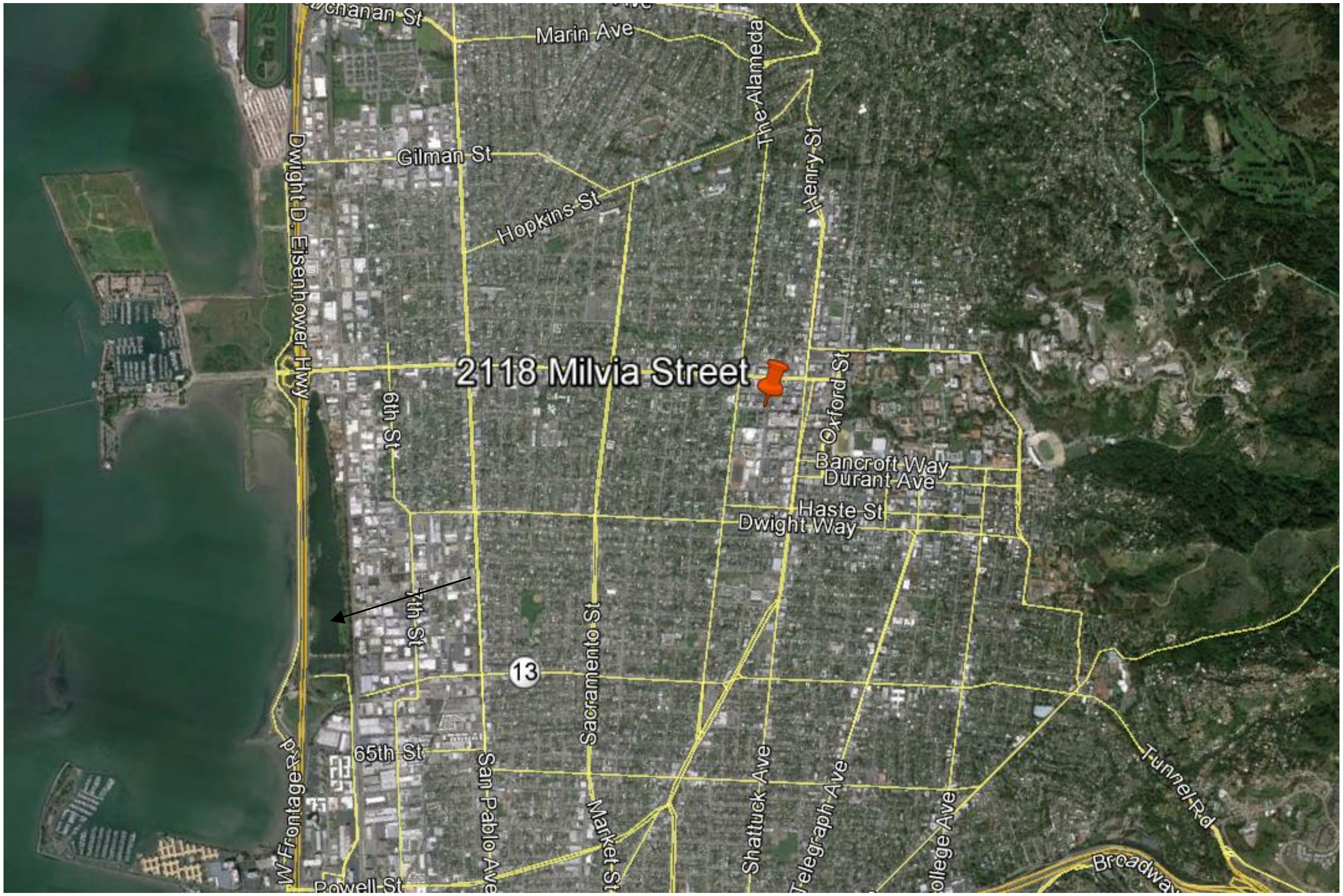
Table 4
BSE-1E Spectra
2118 Milvia Street, Berkeley, California

Period	Probabilistic (20% in 50 years) (g)	Multiplier	Maximum Rotated BSE-1E (g)	BSE-1N (g)	BSE-1E (g)
PGA	0.523	1.100	0.575	0.657	0.575
0.05	0.651	1.100	0.716	0.819	0.716
0.1	0.902	1.100	0.992	1.101	0.992
0.2	1.111	1.100	1.222	1.401	1.222
0.3	1.101	1.125	1.239	1.506	1.239
0.4	1.055	1.150	1.213	1.512	1.213
0.5	0.987	1.175	1.160	1.465	1.160
0.75	0.772	1.238	0.955	1.314	0.955
1	0.612	1.300	0.796	1.135	0.796
2	0.307	1.300	0.400	0.635	0.400
3	0.186	1.300	0.241	0.412	0.241
4	0.127	1.300	0.164	0.295	0.164

Table 5
Recommended Vertical Spectra
2118 Milvia Street, Berkeley, California,

Period	FEMA 750 (g)	2/3rds Design Spectra (g)
0	0.630	0.438
0.025	0.630	0.492
0.05	1.681	0.546
0.1	1.681	0.734
0.15	1.681	0.834
0.2	1.355	0.934
0.3	1.000	1.004
0.4	0.806	1.008
0.5	0.681	0.977
0.75	0.503	0.876
1.0	0.405	0.756
2.0	0.241	.423

Use the larger of the FEMA 750 and 2/3rds Design Spectra value



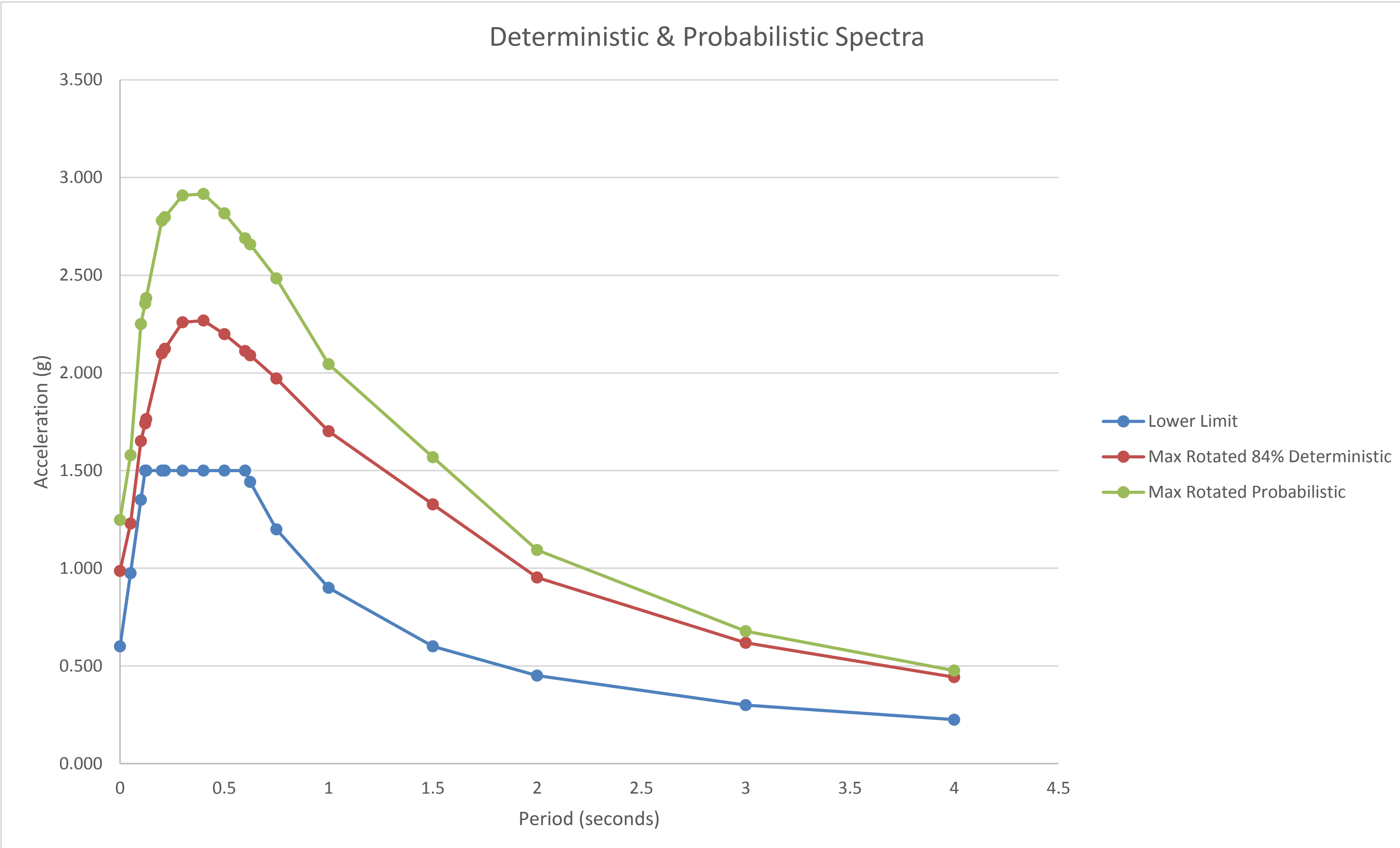
Source: GoogleEarth Professional

N

0' 3,500' 7,000'

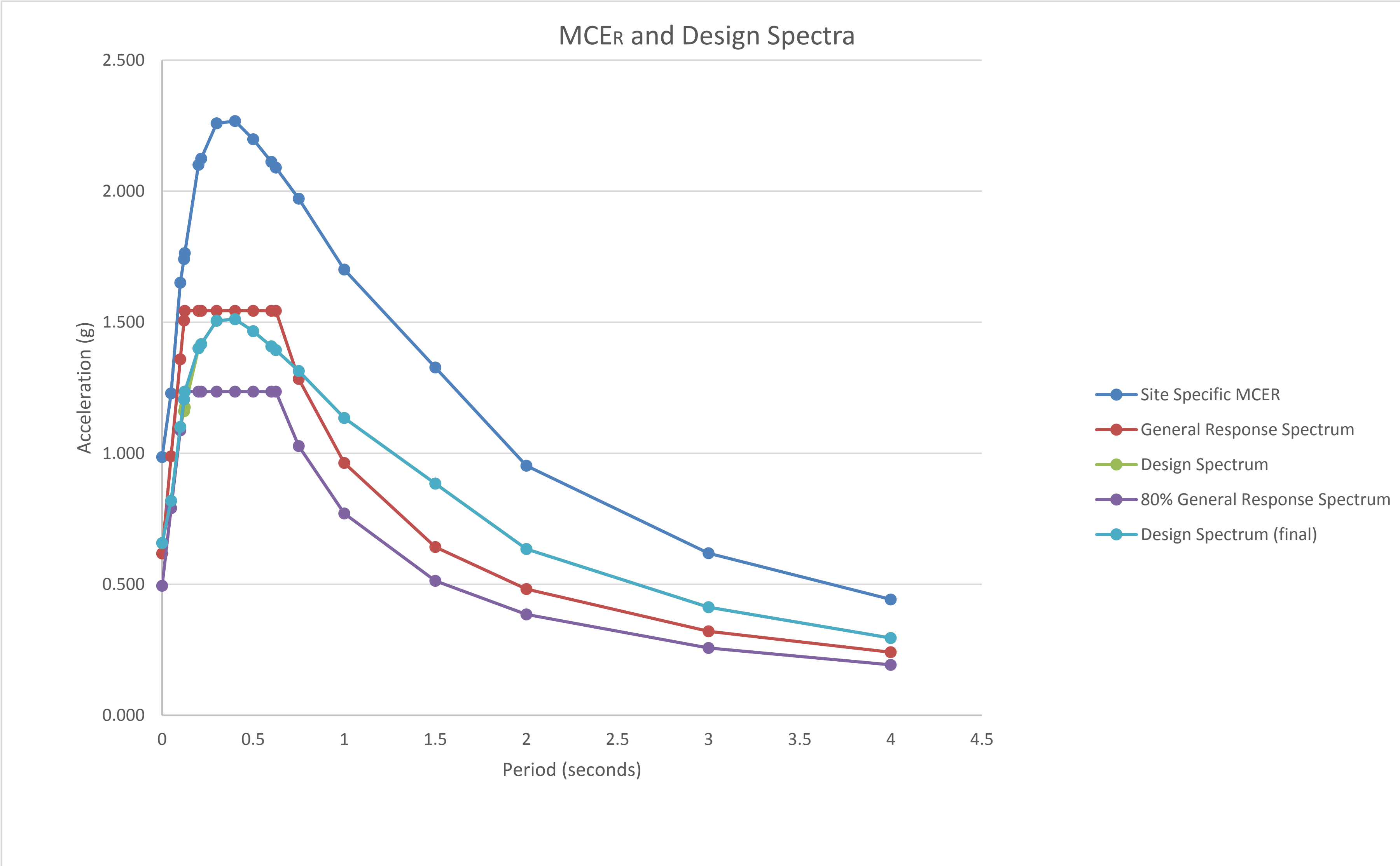
SAFETY FIRST	CLIENT: Peralta Community College Dist	SITE LOCATION
terraphase engineering	PROJECT: 2118 Milvia Street	
	PROJECT NUMBER: 0034.001.001	FIGURE 1

File Name: figure 1 - site location.docx drawn by: jrr checked by: ag



File Name: figure 2 det and prob spectra.doxc drawn by: jrr checked by: ca

SAFETY FIRST	CLIENT: Peralta Community College District	Probabilistic & Deterministic Spectra
terraphase engineering	PROJECT: 2118 Milvia Street	
	PROJECT NUMBER: 0062.004.001	FIGURE 2



SAFETY FIRST	CLIENT: Peralta Community College District	MCER and Design Spectra
terraphase engineering	PROJECT: 2118 Milvia Street	
	PROJECT NUMBER: 0062.004.001	FIGURE 3

File Name: figure MCE.doxc drawn by: jrr checked by: ca

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Attachment A

EZ-Frisk Output

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**********************************************
*****               EZ-FRISK            *****
***** SEISMIC HAZARD ANALYSIS DEFINITION *****
*****       FUGRO CONSULTANTS, INC.      *****
*****        WALNUT CREEK, CA  USA       *****
**********************************************

PROGRAM VERSION
  EZ-FRISK 7.65 Build 004

ANALYSIS TITLE:
  Seismic Hazard Analysis 1

ANALYSIS TYPE:
  Single Site Analysis

SITE COORDINATES
  Latitude 37.8701
  Longitude -122.271

INTENSITY TYPE:  Spectral Response @ 5% Damping

HAZARD DEAGGREGATION
  Status: OFF

SOIL AMPLIFICATION
  Method: Do not use soil amplification

ATTENUATION EQUATION SITE PARAMETERS
  Depth[Vs=1000m/s] (m): 40
  Estimate Z1 from Vs30 for CY NGA: 1
  Vs30 (m/s): 330
  Vs30 Is Measured: 0
  Z25 (km): 2

AMPLITUDES - Acceleration (g)
  0.0001
  0.001
  0.01
  0.02
  0.05
  0.07
  0.1
  0.2
  0.3
  0.4
  0.5
  0.7
  1
  2
  3

PERIODS (s)
  PGA
  0.05
  0.1
  0.2
  0.3
  0.4
  0.5
  0.75
  1
  2
  3
  4

DETERMINISTIC FRACTILES
  0.5
  0.84

PLOTTING PARAMETERS
```

```
  Period at which to plot PGA: 0.005

CALCULATIONAL PARAMETERS
  Fault Seismic Sources -
    Maximum inclusion distance          :   200 km
    Down dip integration increment      :   1 km
    Horizontal integration increment    :   1 km
    Number rupture length per earthquake :   1
  Subduction Interface Seismic Sources -
    Maximum inclusion distance          :   1000 km
    Down dip integration increment      :   5 km
    Horizontal integration increment    :   20 km
    Number rupture length per earthquake :   1
  Subduction Slab Seismic Sources -
    Maximum inclusion distance          :   1000 km
    Down dip integration increment      :   5 km
    Horizontal integration increment    :   20 km
    Number rupture length per earthquake :   1
  Area Seismic Sources -
    Maximum inclusion distance          :   200 km
    Vertical integration increment      :   3 km
    Number of rupture azimuths          :   3
    Minimum epicentral distance step    :   0.5 km
    Maximum epicentral distance step    :   10 km
  Gridded Seismic Sources -
    Maximum inclusion distance           :   200 km
    Default number of rupture azimuths   :   20
    Maximum distance for default azimuths :   40 km
    Minimum distance for one azimuth     :   150
    Use binned calcuations if possible   :   true
    Bins per decade in distance (km)     :   20
  All Seismic Sources -
    Magnitude integration step          :   0.1 M
    Apply magnitude scaling             :   NO
    Include near-source directivity     :   NO

ATTENUATION EQUATIONS

  Name: Boore-Atkinson (2008) NGA USGS 2008
  Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
  Base: Boore-Atkinson 2007 NGA
  Truncation Type: Trunc Sigma*Value
  Truncation Value: 3
  Magnitude Scale: Moment Magnitude
  Distance Type: Horizontal Distance To Rupture

  Name: Campbell-Bozorgnia (2008) NGA USGS 2008
  Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
  Base: Campbell-Bozorgnia 2008 NGA
  Truncation Type: Trunc Sigma*Value
  Truncation Value: 3
  Magnitude Scale: Moment Magnitude
  Distance Type: Distance To Rupture

  Name: Chiou-Youngs (2008) NGA
  Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
  Base: Chiou-Youngs 2008 NGA
  Truncation Type: No Truncation
  Truncation Value: 0
  Magnitude Scale: Moment Magnitude
  Distance Type: Distance To Rupture

  Name: Youngs (1997) Subduction Soil
  Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
  Base: Youngs 1997 Soil
  Truncation Type: No Truncation
  Truncation Value: 0
  Magnitude Scale: Moment Magnitude
  Distance Type: Distance To Rupture
```

SEISMIC SOURCE SUMMARY TABLE

Source	Region	Closest Distance	Deterministic Magnitude	Fault Mechanism	Dip Angle	Dips To	Site Lies
Calaveras	USGS 2008 California	23.71	7.0250	Strike Slip	90.0000	--	W
California Gridded	USGS 2008 California	0.00	7.0000	SS|R	90.0000	--	Above
California Gridded Deep	USGS 2008 California	26.06	7.2000	Intraslab	90.0000	--	S
Great Valley 3, Mysterious Ridge	USGS 2008 California	87.09	7.1000	Reverse	20.0000	SW	S
Great Valley 4a, Trout Creek	USGS 2008 California	69.89	6.6000	Reverse	20.0000	SW	S
Great Valley 4b, Gordon Valley	USGS 2008 California	45.82	6.8000	Reverse	20.0000	W	S
Great Valley 5, Pittsburg Kirby Hills	USGS 2008 California	41.91	6.7000	Strike Slip	90.0000	--	SW
Great Valley 7	USGS 2008 California	63.58	6.9000	Reverse	15.0000	SW	W
Green Valley Connected	USGS 2008 California	23.87	6.8000	Strike Slip	90.0000	--	SW
Greenville Connected	USGS 2008 California	38.45	7.0000	Strike Slip	90.0000	--	W
Greenville Connected U	USGS 2008 California	38.45	7.0000	Strike Slip	90.0000	--	W
Hayward-Rodgers Creek	USGS 2008 California	1.72	7.3340	Strike Slip	90.0000	--	SW
Hunting Creek-Berryessa	USGS 2008 California	65.10	7.1000	Strike Slip	90.0000	--	S
Maacama-Garberville	USGS 2008 California	86.67	7.4000	Strike Slip	90.0000	--	SE
Monte Vista-Shannon	USGS 2008 California	48.11	6.5010	Reverse	45.0000	SW	N
Mount Diablo Thrust	USGS 2008 California	21.87	6.7000	Reverse	38.0000	NE	W
Northern San Andreas	USGS 2008 California	27.39	8.0500	Strike Slip	90.0000	--	NE
Point Reyes	USGS 2008 California	49.94	6.9000	Reverse	50.0000	NE	E
San Andreas Creeping Section Gridded	USGS 2008 California	99.28	6.0000	Strike Slip	90.0000	--	NW
San Gregorio Connected	USGS 2008 California	32.42	7.5000	Strike Slip	90.0000	--	E
West Napa	USGS 2008 California	32.79	6.7000	Strike Slip	90.0000	--	S
Zayante-Vergeles	USGS 2008 California	90.71	7.0000	Strike Slip	90.0000	--	N
Extensional Gridded	USGS 2008 Western US	0.00	7.0000	N|SS	90.0000	--	Above
Nonextensional Gridded	USGS 2008 Western US	76.18	10.0000	SS|R	90.0000	--	S

Deterministic Spectra Results using EZ-FRISK 7.65 Build 004

Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations
Amplitude Units: Acceleration (g)

Fractile: 0.5

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	5.412e-001	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.05	6.586e-001	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.1	8.763e-001	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.2	1.117e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.3	1.156e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.4	1.125e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.5	1.051e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.75	8.670e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
1	7.026e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
2	3.766e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
3	2.420e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
4	1.715e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek

Fractile: 0.84

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	8.962e-001	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.05	1.117e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.1	1.501e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.2	1.908e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.3	2.008e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.4	1.972e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.5	1.871e+000	7.00 Mw	5.00	USGS 2008 California	California Gridded
0.75	1.593e+000	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
1	1.309e+000	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
2	7.326e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
3	4.757e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek
4	3.403e-001	7.33 Mw	1.72	USGS 2008 California	Hayward-Rodgers Creek

Probabilistic Spectra results for EZ-FRISK 7.65 Build 004

ANNUAL FREQUENCY OF EXCEEDANCE: 2.107e-003
RETURN PERIOD: 474.6
PROBABILITY OF EXCEEDENCE: 10.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008
Column 4: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008
Column 5: Acceleration (g) for: Chiou-Youngs (2008) NGA
Column 6: Acceleration (g) for: Youngs (1997) Subduction Soil

1	2	3	4	5	6
PGA	7.094e-001	7.431e-001	5.890e-001	7.734e-001	3.845e-003
0.05	8.845e-001	9.473e-001	7.414e-001	9.742e-001	5.366e-003
0.1	1.192e+000	1.318e+000	1.016e+000	1.248e+000	7.702e-003
0.2	1.479e+000	1.623e+000	1.230e+000	1.592e+000	1.159e-002
0.3	1.486e+000	1.602e+000	1.247e+000	1.615e+000	1.129e-002
0.4	1.432e+000	1.553e+000	1.243e+000	1.506e+000	1.018e-002
0.5	1.337e+000	1.422e+000	1.222e+000	1.373e+000	5.746e-003
0.75	1.095e+000	1.140e+000	1.023e+000	1.123e+000	3.130e-003
1	8.834e-001	8.978e-001	8.194e-001	9.359e-001	2.043e-003
2	4.497e-001	4.672e-001	4.317e-001	4.511e-001	3.051e-004
3	2.742e-001	2.821e-001	2.722e-001	2.680e-001	1.282e-004
4	1.914e-001	1.931e-001	2.012e-001	1.784e-001	* 8.350e-005

ANNUAL FREQUENCY OF EXCEEDANCE: 1.026e-003
RETURN PERIOD: 974.8
PROBABILITY OF EXCEEDENCE: 5.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008
Column 4: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008
Column 5: Acceleration (g) for: Chiou-Youngs (2008) NGA
Column 6: Acceleration (g) for: Youngs (1997) Subduction Soil

1	2	3	4	5	6
PGA	8.916e-001	9.561e-001	7.320e-001	9.793e-001	1.649e-002
0.05	1.115e+000	1.201e+000	9.181e-001	1.189e+000	2.065e-002
0.1	1.514e+000	1.757e+000	1.207e+000	1.562e+000	2.376e-002
0.2	1.946e+000	2.165e+000	1.510e+000	2.066e+000	3.080e-002
0.3	1.980e+000	2.156e+000	1.560e+000	2.110e+000	2.987e-002
0.4	1.918e+000	2.114e+000	1.584e+000	2.013e+000	2.644e-002
0.5	1.783e+000	1.957e+000	1.573e+000	1.825e+000	2.156e-002
0.75	1.426e+000	1.521e+000	1.300e+000	1.457e+000	1.461e-002
1	1.158e+000	1.183e+000	1.075e+000	1.214e+000	9.500e-003
2	6.059e-001	6.336e-001	5.749e-001	6.102e-001	1.485e-003
3	3.735e-001	3.835e-001	3.668e-001	3.702e-001	5.144e-004
4	2.600e-001	2.615e-001	2.684e-001	2.491e-001	2.598e-004

ANNUAL FREQUENCY OF EXCEEDANCE: 4.041e-004
RETURN PERIOD: 2474.9
PROBABILITY OF EXCEEDENCE: 2.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008
Column 4: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008
Column 5: Acceleration (g) for: Chiou-Youngs (2008) NGA
Column 6: Acceleration (g) for: Youngs (1997) Subduction Soil

1	2	3	4	5	6
PGA	1.134e+000	1.216e+000	9.154e-001	1.205e+000	3.484e-002
0.05	1.435e+000	1.599e+000	1.125e+000	1.520e+000	4.468e-002
0.1	2.046e+000	2.376e+000	1.508e+000	2.060e+000	5.386e-002
0.2	2.527e+000	2.919e+000	1.969e+000	2.611e+000	7.158e-002
0.3	2.586e+000	2.930e+000	2.056e+000	2.712e+000	6.961e-002
0.4	2.536e+000	2.875e+000	2.109e+000	2.592e+000	6.088e-002
0.5	2.398e+000	2.652e+000	2.122e+000	2.412e+000	4.933e-002
0.75	2.007e+000	2.150e+000	1.773e+000	2.030e+000	3.335e-002
1	1.573e+000	1.631e+000	1.422e+000	1.657e+000	2.272e-002
2	8.407e-001	8.863e-001	7.884e-001	8.475e-001	4.279e-003
3	5.211e-001	5.339e-001	5.068e-001	5.229e-001	1.810e-003
4	3.662e-001	3.680e-001	3.719e-001	3.578e-001	1.069e-003

ANNUAL FREQUENCY OF EXCEEDANCE: 4.463e-003
RETURN PERIOD: 224.1
PROBABILITY OF EXCEEDENCE: 20.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008
Column 4: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008
Column 5: Acceleration (g) for: Chiou-Youngs (2008) NGA
Column 6: Acceleration (g) for: Youngs (1997) Subduction Soil

1	2	3	4	5	6
PGA	5.230e-001	5.430e-001	4.517e-001	5.731e-001	* 7.043e-032
0.05	6.513e-001	6.809e-001	5.616e-001	7.192e-001	* 2.674e-046
0.1	9.021e-001	9.689e-001	7.806e-001	9.797e-001	* 2.178e-068
0.2	1.111e+000	1.164e+000	9.895e-001	1.191e+000	* 1.241e-152
0.3	1.101e+000	1.142e+000	9.802e-001	1.187e+000	* 5.048e-140
0.4	1.055e+000	1.100e+000	9.505e-001	1.109e+000	* 1.576e-097
0.5	9.873e-001	1.019e+000	9.119e-001	1.021e+000	* 1.316e-048
0.75	7.718e-001	7.965e-001	7.261e-001	7.965e-001	* 7.913e-021
1	6.123e-001	6.190e-001	5.772e-001	6.442e-001	* 1.181e-010
2	3.074e-001	3.182e-001	2.997e-001	3.045e-001	* 2.782e-005
3	1.857e-001	1.929e-001	1.869e-001	1.767e-001	* 3.012e-005
4	1.265e-001	1.287e-001	1.335e-001	1.167e-001	* 2.558e-005