Geologic Hazards Evaluation Report
Chabot College Physical Education Building
25555 Hesperian Boulevard
Hayward, California

May 18, 2009
003-09176-18

Prepared for
Chabot-Las Positas Community College District
25555 Hesperian Boulevard
Hayward, California 94545

LFR an ARCADIS company

This Page Intentionally Left Blank for Double-Sided Copying Purposes

LFR an ARCADIS company

May 18, 2009 003-09176-18

Doug Horner, AIA
Project Manager
Chabot-Las Positas Community College District
25555 Hesperian Boulevard
Hayward, California 94545

Subject: Geologic Hazards Evaluation Report, Proposed Chabot College Physical Education Building, 25555 Hesperian Boulevard, Hayward, California

Dear Mr. Horner:

LFR Inc. (LFR) is pleased to present the attached geologic hazards evaluation report for the proposed Chabot College Physical Education Building, to be located at 25555 Hesperian Boulevard in Hayward, California.

LFR appreciates this opportunity to provide consulting services to the Chabot-Las Positas Community College District, and looks forward to being of further assistance as the project proceeds.

If you have any questions concerning this report, please contact either of the undersigned at any time. Steve Beadle can be reached at (805) 349-7180, and Jeff Raines can be reached at (510) 596-9580.

Sincerely,

PROFESSIONAL GEOLOGIST STEVEN C. BEADLE No. EG 2098 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JEFFERY R RAINES 2762 EXP 9-30-09 GEOTECHNICAL STATE OF CALIFORNIA

Steven Beadle, Ph.D., P.G. (6129), CHG (503), CEG (2098), P.E. (C69951)
Senior Associate Engineering Geologist

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

Attachment

510.652.4500 m
510.652.2246 f
1900 Powell Street, 12th Floor
Emeryville, California 94608
Offices Nationwide
www.lfr.com

This Page Intentionally Left Blank for Double-Sided Copying Purposes

CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of the geologic hazards evaluation conducted by LFR Inc. (LFR) for the proposed Chabot College Physical Education (PE) Building, to be located at 25555 Hesperian Boulevard in Hayward, California (“the Site”; Figures 1 and 2).

This report includes our opinions concerning potential geologic hazards that may have an impact on site development and could potentially impede the performance of the proposed project. 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 geologic hazards applicable to sites located in Northern California. These conclusions should not be extrapolated to other areas outside the Site without our prior review.

This report addresses the geological hazards included on the California Geological Survey (CGS) Note 48 (Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings). This report does not address the geotechnical issues included on Note 48 which are addressed in the Geotechnical Report for the Site (LFR 2006, 2008) with the exception of potential liquefaction settlements which are addressed herein.

2.0 LOCATION AND SITE DESCRIPTION

The Site (Figures 1 and 2) is located on the Chabot College campus in the City of Hayward in Alameda County, California. It is currently occupied by a paved and landscaped area on the northwest part of the campus, including areas used for basketball courts. Prior to its use for educational purposes, the Site was used for agriculture.

The center of the Site is located at a latitude of approximately 37.6434° North, and a longitude of approximately 122.1082° West. According to published topographic maps (Figure 3), it lies at an elevation of approximately 43 feet above mean sea level (msl), and is essentially flat. The local topography slopes to the west toward San Francisco Bay (“the Bay”) at a rate of approximately 25 feet per mile.

The Site was inspected by Mr. Jeffery Raines, P.E. (51120), G.E. (2762), on at least five occasions dating back to 2006. No obvious evidence of potential geological hazards was observed at the Site during those visits.

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of this evaluation was to identify major geological and seismic hazards that could potentially preclude the proposed school siting or make the proposed construction economically unfeasible. LFR performed the following scope of services for this geologic hazard evaluation:

- conducted a review of geologic hazards data
- conducted a site inspection
- prepared a report of pertinent findings with respect to seismic, geologic, and preliminary 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

4.0 SITE CONDITIONS

4.1 Geology and Soils

4.1.1 General Setting

The Site is located on a narrow coastal plain, between San Francisco Bay (about 2.5 miles to the west) and the Hayward Hills (about 2.3 miles to the northeast). The surface deposits in the Site area represent Quaternary alluvium (Figure 5).

The surface deposits in the Site area form part of a large alluvial fan complex associated with the combined flows of San Lorenzo and Ward Creeks, which drain the Hayward Hills to the northeast. The surface sediments at the Site have been mapped as Holocene levee deposits (California Geological Survey [CGS] 2003b, pl. 1.1).

The soils at the Site have been classified as Botella Loam (NRCS 2009). The Botella Series includes loamy soils that form on alluvial fan deposits (NRCS 2001). In general, Botella soils are considered to have a moderate shrink-swell potential, and are considered moderately corrosive to concrete and steel.

4.1.2 Site-Specific Data

The existing subsurface data for the building vicinity (Figure 2) include the following:

- Two borings, designated 17 and 20, installed by Woodward Clyde in 1963 (described by LFR 2006, app. B);

- Two Cone Penetration Tests (CPTs), designated 2900-C1 and 2900-C2, installed by LFR in June 2006 (LFR 2006);

- Two CPTs, designated CPT-1 and CPT-2, installed by LFR in July 2008 (LFR 2008).

The maximum depths logged were 33 feet for the borings and 50 feet for the CPTs.

The subsurface conditions in the vicinity of the proposed PE Building are typical of conditions at Chabot College. The previous borings and CPTs generally logged stiff to very stiff interbedded strata of clays and silts down to approximately 15 feet below ground surface (bgs), overlying looser/softer clayey silts and silty clays. The water table was logged at depths of approximately 15 to 16 feet bgs.

Soil testing by LFR (2006) found the local soils to be moderately expansive and corrosive, which is consistent with the general characteristics of the Botella Series as described by NRCS (2009).

4.2 Hydrology and Hydrogeology

There are no "blue line" streams shown on the 1947 USGS 7.5-minute series topographic map for the Site which shows the pre-Campus topography. Stormwater is collected in the campus stormwater system for discharge to local drainage channels and eventually to San Francisco Bay 2.5 miles west of campus.

As is mentioned above, groundwater was reported at depths of approximately 15 to 16 feet below the ground surface in subsurface work conducted at the Site in 1963, 2006, and 2008. CGS (2003b, pl. 1.2) estimated the depth to historically highest groundwater at 10 to 20 feet bgs in the Site area.

4.3 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 4. Activity was determined by slip rates, as per the CGS (Petersen et al. 1996).

Table 1 includes an estimate of the peak ground acceleration (PGA) and the Modified Mercalli Intensity (MMI) likely to be felt at the Site due to earthquakes on the individual faults. The 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 easier to interpret than PGAs.

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 expected ground accelerations at the Site were evaluated with FRISKSP software (Blake 2000b), using the Boore et al. (1997), Abrahamson and Silva (1997), and Sadigh et al. (1997) attenuation relationships for soil, and the calculated distances between the Site and the regional faults. The results are presented in Table 3. The average results for the three attenuation relationships are summarized below.

- **Maximum Considered Earthquake (MCE).** FRISKSP estimated the average MCE acceleration at the Site at approximately 1.10 g. The MCE has a 2,475-year return period (equivalent to a 2% chance of exceedance in 50 years).

USGS (2007b) has published PGAs for the 2% in 50 years seismic event (2,475-year return period) for rock sites. The 0.95 g result from USGS (2007b) for rock sites is reasonably consistent with the 1.1 g result from FRISKSP for alluvium.

Table 4 presents the significant historical earthquakes that have occurred in the site vicinity. Based on these results, the maximum historical PGA at the Site is approximately 0.44 g.

Table 1
Known Active Earthquake Faults within 50 Kilometers of the Site
Chabot College Physical Education Building
Hayward, California

Abbreviated Fault Name	Approx. Distance, miles (km)	Maximum Earthquake Mag. (Mw)	Peak Ground Accel. (g)	Est. Site Intensity, Modified Mercalli
HAYWARD (HS+HN)	2.5 (4.1)	6.9	0.449	X
HAYWARD (HS+HN+RC)	2.5 (4.1)	7.3	0.473	X
HAYWARD (FLOATING)	2.5 (4.1)	6.9	0.448	X
HAYWARD (HS)	2.5 (4.1)	6.7	0.434	X
CALAVERAS (FLOATING)	10.2 (16.4)	6.2	0.155	VIII
CALAVERAS (CS+CC+CN)	10.2 (16.4)	6.9	0.211	VIII
CALAVERAS (CN)	10.2 (16.4)	6.8	0.202	VIII
CALAVERAS (CC+CN)	10.2 (16.4)	6.2	0.159	VIII
HAYWARD (HN)	13.9 (22.4)	6.5	0.144	VIII
HAYWARD (HN+RC)	13.9 (22.4)	7.1	0.175	VIII
MOUNT DIABLO (MTD)	15.5 (25.0)	6.7	0.17	VIII
SAN ANDREAS (SAP+SAN+SAO)	15.8 (25.4)	7.8	0.201	VIII
SAN ANDREAS (SAS+SAP)	15.8 (25.4)	7.4	0.176	VIII
SAN ANDREAS (SAP+SAN)	15.8 (25.4)	7.7	0.189	VIII
SAN ANDREAS (SAP)	15.8 (25.4)	7.2	0.161	VIII
SAN ANDREAS	15.8 (25.4)	7.9	0.206	VIII
SAN ANDREAS (FLOATING)	15.8 (25.4)	6.9	0.148	VIII
SAN ANDREAS (SAS+SAP+SAN)	15.8 (25.4)	7.8	0.196	VIII
MONTE VISTA - SHANNON	16.2 (26.1)	6.7	0.167	VIII
CONCORD/GV (CON)	18.9 (30.4)	6.3	0.093	VII
CONCORD/GV (FLOATING)	18.9 (30.4)	6.2	0.089	VII
CONCORD/GV (CON+GVS+GVN)	18.9 (30.4)	6.7	0.119	VII
CONCORD/GV (CON+GVS)	18.9 (30.4)	6.6	0.114	VII
CALAVERAS (CS+CC)	21.4 (34.5)	6.4	0.091	VII
CALAVERAS (CC)	21.4 (34.5)	6.2	0.081	VII
CALAVERAS (CS+CC FLOATING)	21.4 (34.5)	6.2	0.079	VII
GREENVILLE (GN)	21.5 (34.6)	6.7	0.104	VII
SAN GREGORIO (SGS+SGN)	23.1 (37.2)	7.4	0.133	VIII
SAN GREGORIO (FLOATING)	23.1 (37.2)	6.9	0.108	VII
SAN GREGORIO (SGN)	23.1 (37.2)	7.2	0.122	VII
GREENVILLE (GS+GN)	24.4 (39.3)	6.9	0.104	VII
GREENVILLE (GS)	24.4 (39.3)	6.6	0.091	VII
GREENVILLE (FLOATING)	24.4 (39.3)	6.2	0.069	VI
SAN ANDREAS (SAN+SAO)	27.3 (44.0)	7.7	0.129	VIII
SAN ANDREAS (SAN)	27.3 (44.0)	7.5	0.117	VII

Abbreviated Fault Name	Approx. Distance, miles (km)	Maximum Earthquake Mag. (Mw)	Peak Ground Accel. (g)	Est. Site Intensity, Modified Mercalli
CONCORD/GV (GVS)	27.4 (44.1)	6.2	0.064	VI
CONCORD/GV (GVS+GVN)	27.4 (44.1)	6.5	0.078	VII
GREAT VALLEY 7	28.5 (45.8)	6.7	0.103	VII
GREAT VALLEY 5	31.5 (50.7)	6.5	0.086	VII

Notes: The expected peak ground acceleration (PGA) is the mean value based on Abrahmson & Silva (1997).

CON =Concord
GVS =Green Valley South
GVN =Green Valley North
GV = Green Valley
CS => Calaveras South
HS => Hayward South
RC => Rodgers Creek
SAO => San Andreas Offshore
SAS => San Andreas South
SGN=> San Gregorio North
GN => Greenville North
GS => Greenville South
CC => Calaveras Central
CN => Calaveras North
HN => Hayward North
MTD => Mount Diablo Thrust
SAN => San Andreas North
SAP => San Andreas Peninsula
SGS=> San Gregorio South
PGA = peak ground acceleration

Table 2
Applicable Portions of Modified Mercalli Intensity Scale
Chabot College Physical Education Building
Hayward, California

Intensity	Shaking	Summary	Description
VI	Moderate	Objects Fall	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc., off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle).
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.

Intensity	Shaking	Summary	Description
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.

Notes:

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
Peak Ground Accelerations
Chabot College Physical Education Building
Hayward, California

Relationship	2,475-year Return Period Acceleration (g)
Boore et al. (1997)	1.17
Abrahmson and Silva (1997)	1.14
Sadigh et al. (1997)	1.07
Average	1.10
USGS (2008) Rock Site	0.95

Table 4
Historical Earthquakes in Campus Vicinity Magnitude > 6
Chabot College Physical Education Building
Hayward, California

Latitude	Longitude	Date	Magnitude	PGA (g)	MM	Distance in miles (km)
37.7	122.1	10/21/1868	6.8	0.443	X	3.9 (6.3)
37.8	122.2	06/10/1836	6.8	0.23	IX	11.9 (19.2)
37.5	121.9	11/26/1858	6.1	0.102	VII	15.1 (24.3)
37.6	122.4	06/01/1838	7	0.15	VIII	16.2 (26.1)
37.7	122.5	4/18/1906	8.25	0.188	VIII	21.8 (35.0)
37.8	122.5	06/21/1808	6.3	0.078	VII	24.0 (38.6)
37.3	121.9	10/08/1865	6.3	0.071	VI	26.3 (42.3)
38	121.9	05/19/1889	6	0.051	VI	27.1 (43.6)
37.32	121.698	4/24/1984	6.2	0.054	VI	31.7 (51.0)
37.25	121.75	7/1/1911	6.6	0.068	VI	33.5 (53.9)

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 Campus due to the historical earthquake calculated using the Abrahamson and Silva (1997) attenuation relationship.

4.4 Seismic Hazard Assessment

Probabilistic and Deterministic Seismic Hazard Assessments for the Site are presented in Appendix A.

4.5 Ground Rupture Potential

The Site is not located within an Alquist-Priolo Special Studies Earthquake Fault Zone. There are no known active faults, and therefore no Alquist-Priolo Zones, within 2.5 miles of the Site (Table 1). The nearest Alquist-Priolo Zone is associated with the Hayward Fault, approximately 2 miles ; to the east; it does not trend towards the Site (CGS 1988).

Based on these points, there does not appear to be a significant risk of surface rupture during the expected service life of the proposed school buildings.

4.6 Liquefaction Potential and Settlements

Many parts of the City of Hayward, including the Site area, are located within a liquefaction hazard zone, as mapped by CGS (2003c).

Soils with Soil Behavior Indices (Ic) greater than 2.61 and side friction ratios greater than 1 are unlikely to liquefy (Robertson and Wride 1998). The side friction ratios in the Chabot College clay soils were generally in the 2 to 5% range. LFR used the results of the Cone Penetration Test (CPT) probes to calculate Ic's for the soils at the locations of the 50-foot CPT probe. Strata more than 1 foot thick with Ic greater than 2.61 (liquefiable) with corrected equivalent SPT blow counts less than 50 were not present. There were two approximately 3-inch thick strata, between 12.75 and 13 feet bgs and between 17.8 and 18.1 feet bgs which had Ics less than 2.61 and friction ratios only slightly above 1 which were considered herein to be liquefiable.

The upper stratum could conceivably be below the groundwater table at times during the year and the lower stratum will likely be below the groundwater table for most of the year. Both strata could liquefy during a significant earthquake.

The equivalent average corrected for overburden and fines Standard Penetration Test (SPT) blow count in these strata is16. Assuming the strata will liquefy, the volumetric strain of the stratum can be calculated based on Figure 53 from Seed et al. (2003) as being not greater than 3%

Table 5
Liquefaction Settlement Calculations
Chabot College Physical Education Building
Hayward, California

Depth Range (ft) (5)	Ic (6)	Equivalent SPT Blow Count (1)	Cyclic Stress Ratio (2)	Liquefy? (3)	Expected Deformation (inches) (4)
12.75 – 13	2.17 – 2.34	16	.74	Yes	0.09
17.8 – 18.1	2.1 – 2.43	16	.86	Yes	0.1

Notes (1) Based on fines-corrected CPT data normalized to 1 atmosphere of effective overburden pressure

(2) Calculated based on the MCE peak ground acceleration (PGA) of 1.1g, CSR (13 feet) = 0.65*PGA*Rd*(Total Stress/Effective stress)/ksig/MWF = 0.65*1.1*.99*(1690 psf/1560 psf)/1.07/.96 = 0.74. CSR (18 feet) = 0.65*1.1*.99*(2210 psf/1770 psf)/1.07/.96 = 0.86
Rd and Ksig are weighing factors and MWF is a magnitude weighing factor.

(3) Based on Figure 16 from Seed et al. 2003

(4) Based on Figure 53 from Seed et al. 2003

(5) Ic = [(3.47-log(Q))^2 +(log(F) + 1.22)^2]^0.5 where Q=[(tip resistance – total overburden stress)/Pa](Pa/effective overburden pressure) where Pa is the atmospheric pressure in same units as tip resistance and overburden pressure. F= (Side friction/(tip resistance minus total overburden pressure).

The expected differential settlement due to liquefaction of this stratum is estimated to be less than 0.1 inch between column lines at the site surface. The unsaturated soils at the Site are not expected to settle due to earthquake shaking.

4.7 Landslide Potential

The site area is essentially flat (Figure 3). Given the lack of relief, no significant landslide risk exists. CGS (2003c) did not map any landslide hazard areas near the Site.

4.8 Flood Inundation Potential

4.8.1 Flood Zonation

Based on a review of a Flood Insurance Rate Map of City of Hayward, California, Alameda County, Panel 11 of 29 (Federal Emergency Management Agency 2000), the Site is located within a zone designated as "Zone C – Areas of minimal flooding."

4.8.2 Tsunami

The site is located approximately 2.5 miles inland from San Francisco Bay. Tsunami inundation areas, as mapped by the City of Hayward (2006, pl. 6) are restricted to the coastal parts of the City, at least 2 miles from the Site.

4.8.3 Dam Inundation

According to the Association of Bay Area Governments (ABAG 2009), the campus is not located within the floodplain for a potential dam failure. The floodplains for the Calaveras and Del Valle dams are located approximately 2.6 miles south southeast of Campus.

4.9 Pipeline Risk Analysis

Pipeline risk assessments are not required for community colleges.

4.10 Land Subsidence

The City of Hayward (2006) general plan does not list land subsidence as a significant threat for Hayward. There is no significant petroleum production in the Hayward area, and so there are no large-scale oil or natural gas withdrawals. Large scale agriculture in the Campus vicinity has ended, so significant groundwater withdrawal is unlikely to occur in the Campus vicinity in the future. Should groundwater extraction for municipal use become important in the future, it is likely that it would be controlled to mitigate land subsidence.

4.11 Naturally Occurring Asbestos

Naturally occurring asbestos in California is most often associated with ultramafic (or "ultrabasic") rock bodies. Asbestos occurs less commonly in metagabbro (CGS 2002b). CGS (2000b) has mapped significant areas of serpentinite in the Pleasanton area approximately 12 miles east of Campus and in a different drainage.

Graymer et al. (1996) mapped a small area of serpentinite in the Hayward Hills, approximately 4 miles east of Campus (Figure 5). The serpentinite area is drained by

Ward Creek, which does not currently run towards the Site. However, the Site is located on an alluvial fan which historically received drainage from San Lorenzo and Ward Creeks (Section 4.1.1). It is therefore possible that Ward Creek may have drained towards the Site in the geologic past. Given the small area of the upgradient serpentinite and its distance from the Site, it is unlikely to have significantly affected the local soils at the Site.

4.12 Other Hazards

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

- **Naturally Occurring Radon.** The California Department of Health Services (2008) (DHS) maintains a database of radon measurements in California, based on zip code. No elevated radon results (greater than or equal to 4.0 picoCuries per liter [pCi/L]) have been reported in 114 measurements from the 94545 (Hayward) 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; 1999). The closest historical field is the abandoned Hospital Nose Gas field located approximately 14 miles east of the Site.
- **Mineral Resources and Producers.** The Site is not located near a mineral resource or producer, as listed by the Mineral Resources Data System (MRDS) of the US Geological Survey (USGS 2009). The nearest MRDS locality is a historic facility in the Hayward Hills, approximately 2.5 miles to the northeast of the Site, that processed stone.

5.0 CONCLUSIONS

The conclusions reached by LFR are based on the published data reviewed for this geologic hazards evaluation. Our findings are summarized below.

- The Site is not located within an Alquist Priolo Special Studies Earthquake Fault Zone. Surface rupture should not reasonably be expected during the life of the proposed school buildings.
- 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.
- Liquefaction settlements at the Site are expected to be on the order of 0.1 inches.
- There is a small outcrop of serpentinite approximately 4 miles east of the Site. This area does not currently drain towards the Site, but may have done so in the

geologic past. Given the small area of the upgradient serpentinite and its distance from the Site, it unlikely to have significantly affected the local soils at the Site.

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

6.0 LIMITATIONS STATEMENT

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 LFR 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 environmental consulting industry. No representation, warranty, or guarantee, express or implied, is intended or given. To the extent that LFR relied upon any information prepared by other parties not under contract to LFR, LFR 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. 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 document shall relieve any other party of its responsibility to abide by contract documents and applicable laws, codes, regulations, or standards.

7.0 REFERENCES

Abrahamson, N.A. and Silva, W.J. 1997. "Empirical response spectral attenuation relations for shallow crustal earthquakes," Seism. Research Letters, 68(1), 94-127.

American Society of Civil Engineers (ASCE). 2005. Minimum Design Loads for Buildings and Other Structures. Standard ASCE/SEI 7-05.

Association of Bay Area Governments. 2009. Dam Inundation Maps.

Blake, Thomas F. 2000a. EQFAULT: A Computer Program for the Estimation of Peak Horizontal Acceleration from 3-D Fault Sources. Thomas F. Blake Computer Services & Software. April.

———. 2000b. FRISKSP: A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources. Thomas F. Blake Computer Services & Software. April.

———. 2000c. EQSEARCH: A Computer Program for the Estimation of Peak Acceleration from California Historical Earthquake Catalogs. Thomas F. Blake Computer Services & Software. April.

Boore, D.M., W.B. Joyner, and T.E. Fumal. 1997. Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work. Seismological Research Letters. Vol. 68, No. 1, pp. 128-153.

Bryant, W. A. (compiler). 2005. Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, Version 2.0. California Geological Survey. http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.

California Department of Health Services (DHS). 2008. California Indoor Radon Measurements Sorted by Zip Code. July 1. http://ww2.cdph.ca.gov/HealthInfo/environhealth/Documents/Radon/CaliforniaRadonDatabase.pdf

California Department of Oil, Gas, and Geothermal Resources (DOGGR). 1999. California Digital Map Data. District 6 Oil and Gas Fields. June 17.

California Department of Toxic Substances Control (DTSC). 2004. Interim Guidance, Naturally-Occurring Asbestos (NOA) at School Sites. September 24. http://www.dtsc.ca.gov/Schools/upload/SMBRP_POL_Guidance_Schools_NOA.pdf.

California Geological Survey (CGS). 1982. State of California Special Studies Zones, Hayward, Revised Official Map. January 1.

———. 2000. A General Location Guide for Ultramafic Rocks In California - Areas More Likely To Contain Naturally Occurring Asbestos. California Division of Mines and Geology, Open-File Report 2000-19. August. ftp://ftp.consrv.ca.gov/pub/dmg/pubs/ofr/ofr_2000-019.pdf.

———. 2002a. California Geomorphic Provinces. Note 36. http://www.consrv.ca.gov/cgs/information/publications/cgs_notes/note_36/note_36.pdf.

———. 2002b. Guidelines for Geologic Investigations of Naturally Occurring Asbestos in California, Special Publication 124.

———. 2003a. Geologic Controls on the Distribution of Radon in California. Report prepared for the California Department of Health Services. December. http://www.consrv.ca.gov/cgs/minerals/hazardous_minerals/radon/Geo_Controls_Dist_Radon.pdf.

———. 2003b. Seismic Hazard Zone Report 091, Seismic Hazard Zone Report For The Hayward 7.5-Minute Quadrangle, Alameda County, California.

———. 2003c. Seismic Hazard Zones, Hayward Quadrangle, Official Map. July 2.

———. 2006. Seismic Shaking Hazards in California Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). May 1. http://www.consrv.ca.gov/CGS/rghm/pshamap/pshamain.html.

———. 2007. Interactive Fault Parameter Map of California. http://www.consrv.ca.gov/CGS/rghm/psha/fault_parameters/htm/index.htm.

Cao, T., W. A. Bryant, B. Rowshandel, D. Branum, and C.J. Wills. 2003. The Revised 2002 California Probabilistic Seismic Hazard Maps, June 2003. California Geological Survey. http://www.consrv.ca.gov/cgs/rghm/psha/fault_parameters/pdf/2002_CA_Hazard_Maps.pdf.

City of Hayward. 2006. General Plan. June. http://www.ci.hayward.ca.us/about/general.shtm

Federal Emergency Management Agency (FEMA). 2000. National Flood Insurance Program. Flood Insurance Rate Map, City of Hayward, California. Community-Panel Number 065033 0011 E. February 9.

Graymer, R.W., Jones, D.L., and Brabb, E.E. 1996. Preliminary geologic map emphasizing bedrock formations in Alameda County, California: A digital database: U.S. Geological Survey Open-File Report 96-252.

LFR Inc. (LFR). 2006. Geotechnical Investigation and Design Report, Chabot College Expansion, Hayward, California. August 11.

________________. 2008. Letter to Mr. Douglas Horner: Geotechnical Recommendations, New Physical Education Building, Chabot College, Hayward, California. August 11.

Murphy, J.R., and L.J. O'Brian. 1978. Analysis of Worldwide Strong Motion Data Sample to Develop an Improved Correlation Between Peak Acceleration, Seismic Intensity, and other Physical Parameters, US Nuclear Regulatory Commission, NUREG-0402.

National Resources Conservation Service (NRCS). 2001. Official Series Description, Botella Series. June. http://www2.ftw.nrcs.usda.gov/osd/dat/B/BOTELLA.html

--------. 2009. Web Soil Survey. http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx

OpenSHA. 2009. Attenuation Relationship Plotter, Version 0.10.22. http://www.opensha.org/

Petersen, M.D., W.A. Bryant, C.H. Cramer, T. Cao, and M. Reichle. 1996. Probabilistic Seismic Hazard Assessment for the State of California. California Division of Mines and Geology, Open-File Report 96-08; U.S. Geological Survey, Open-File Report 96-706.

Petersen, Mark D., Frankel, Arthur D., Harmsen, Stephen C., Mueller, Charles S., Haller, Kathleen M., Wheeler, Russell L., Wesson, Robert L., Zeng, Yuehua, Boyd, Oliver S., Perkins, David M., Luco, Nicolas, Field, Edward H., Wills, Chris J., and Rukstales, Kenneth S., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008–1128, 61 p.

Robertson, P. K. and Wride, C. E. 1998. Evaluating Cyclic Liquefaction Potential Using The Cone Penetration Test. Canadian Geotechnical Journal, Vol. 35, No. 3, pp. 442-459.

Sadigh, K., Chang, C.-Y., Egan, J.A., Makdisi, F., and Youngs, R.R. (1997). "Attenuation relations for shallow crustal earthquakes based on California strong motion data," Seism. Res. Letters, 68(1),180-189.

Seed1, R. B., K. O. Cetin, R. E. S. Moss, A. M. Kammerer, J. Wu, J. M. Pestana, M. F. Riemer, R.B. Sancio, J.D. Bray, R. E. Kayen, and A. Faris. 2003. 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, H.M.S. Queen Mary, Long Beach, California, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. April 30,

United States Environmental Protection Agency (U.S. EPA). 2006. EPA Map of Radon Zones, California. March 8. http://www.epa.gov/radon/zonemap/california.htm.

United States Geological Survey (USGS). 1947. Topographic Map, 7.5-Minute Series, Hayward, California Quadrangle. Scale 1:24,000.

———. 1980. Topographic Map, 7.5-Minute Series, Hayward, California Quadrangle. Scale 1:24,000.

———. 2006. Quaternary Fault and Fold Database of the United States. http://earthquake.usgs.gov/regional/qfaults/

———. 2008. Earthquake Ground Motion Tool, Version 5.09. http://earthquake.usgs.gov/research/hazmaps/design/

--------. 2009. Mineral Resources Spatial Data. http://mrdata.usgs.gov/website/MRData-US/viewer.htm

This Page Intentionally Left Blank for Double-Sided Copying Purposes

SOURCE: GOOGLE EARTH PRO 2006

0 500 FEET

Site Location

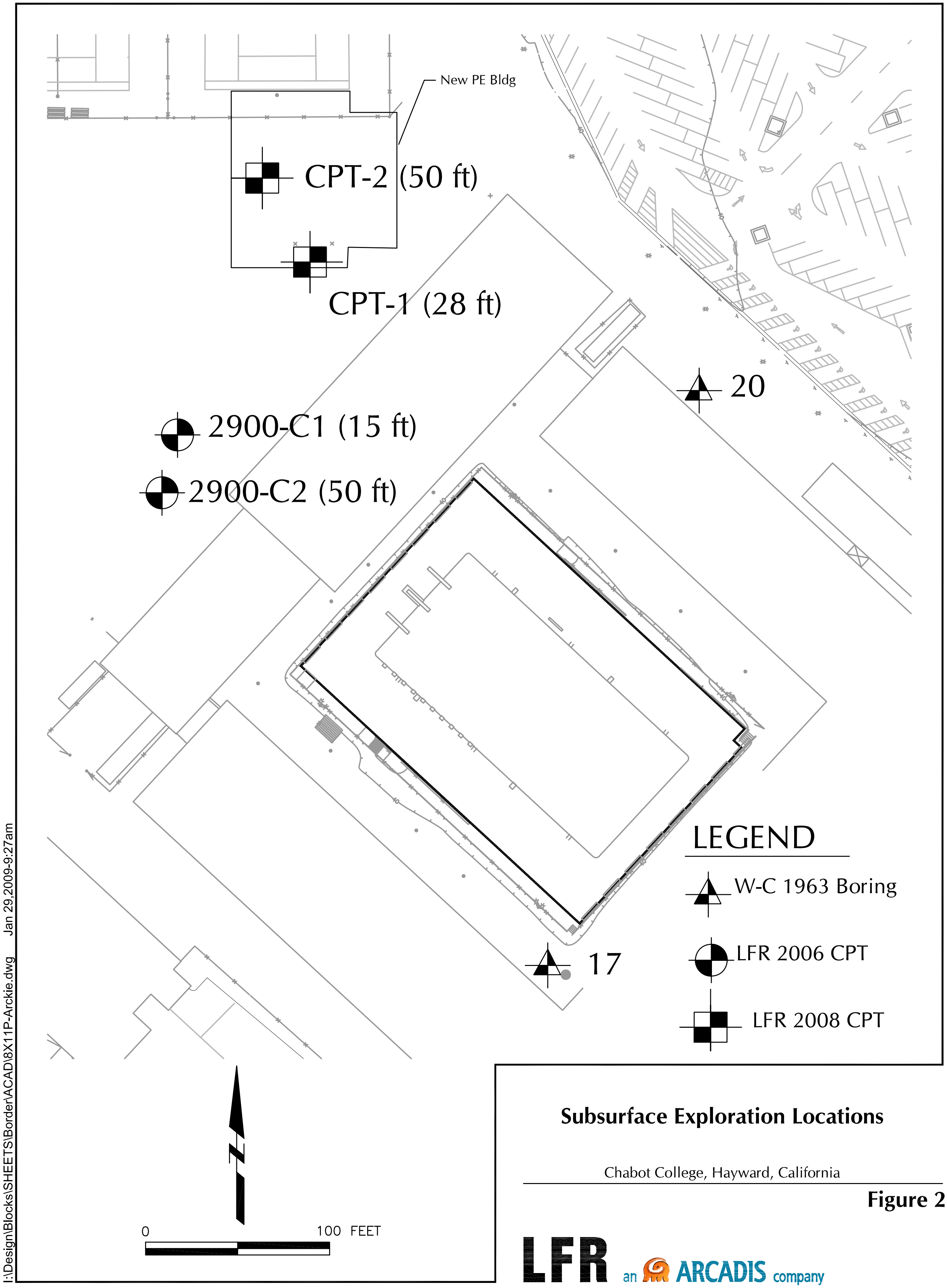
Chabot College, Hayward, California

Figure 1

LFR an ARCADIS company

I:\Design\Blocks\SHEETS\Border\ACAD\8X11P-Arckie.dwg Jan 29,2009-9:27am

This Page Intentionally Left Blank for Double-Sided Copying Purposes



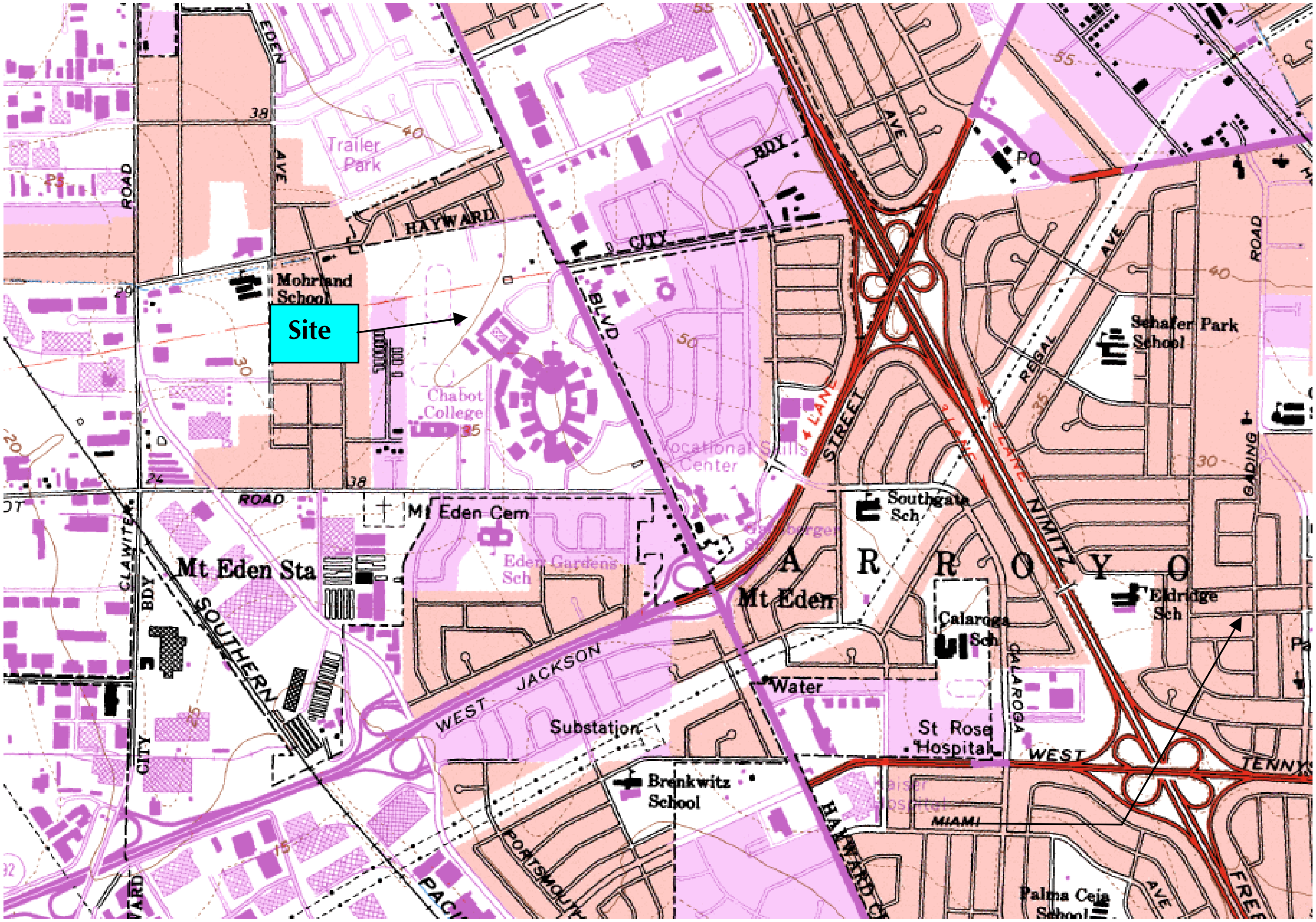
Subsurface Exploration Locations

Chabot College, Hayward, California

Figure 2

LFR an ARCADIS company

This Page Intentionally Left Blank for Double-Sided Copying Purposes



1000	0	1000	2000

feet

Source: USGS 7.5-minute series Hayward Quadrangle (1980)

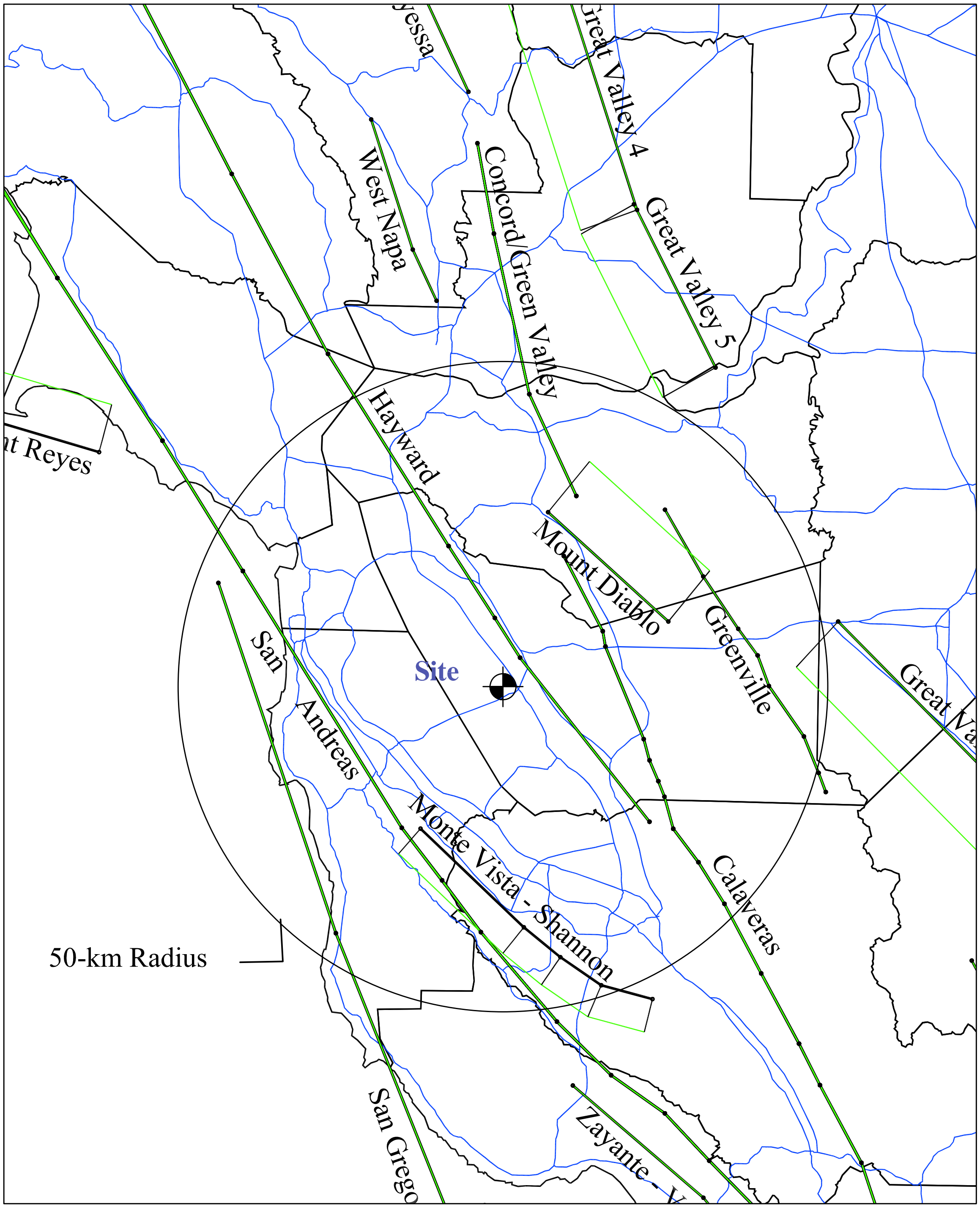
Topography

Chabot College, Hayward California

Figure 3

LFR an ARCADIS company

This Page Intentionally Left Blank for Double-Sided Copying Purposes



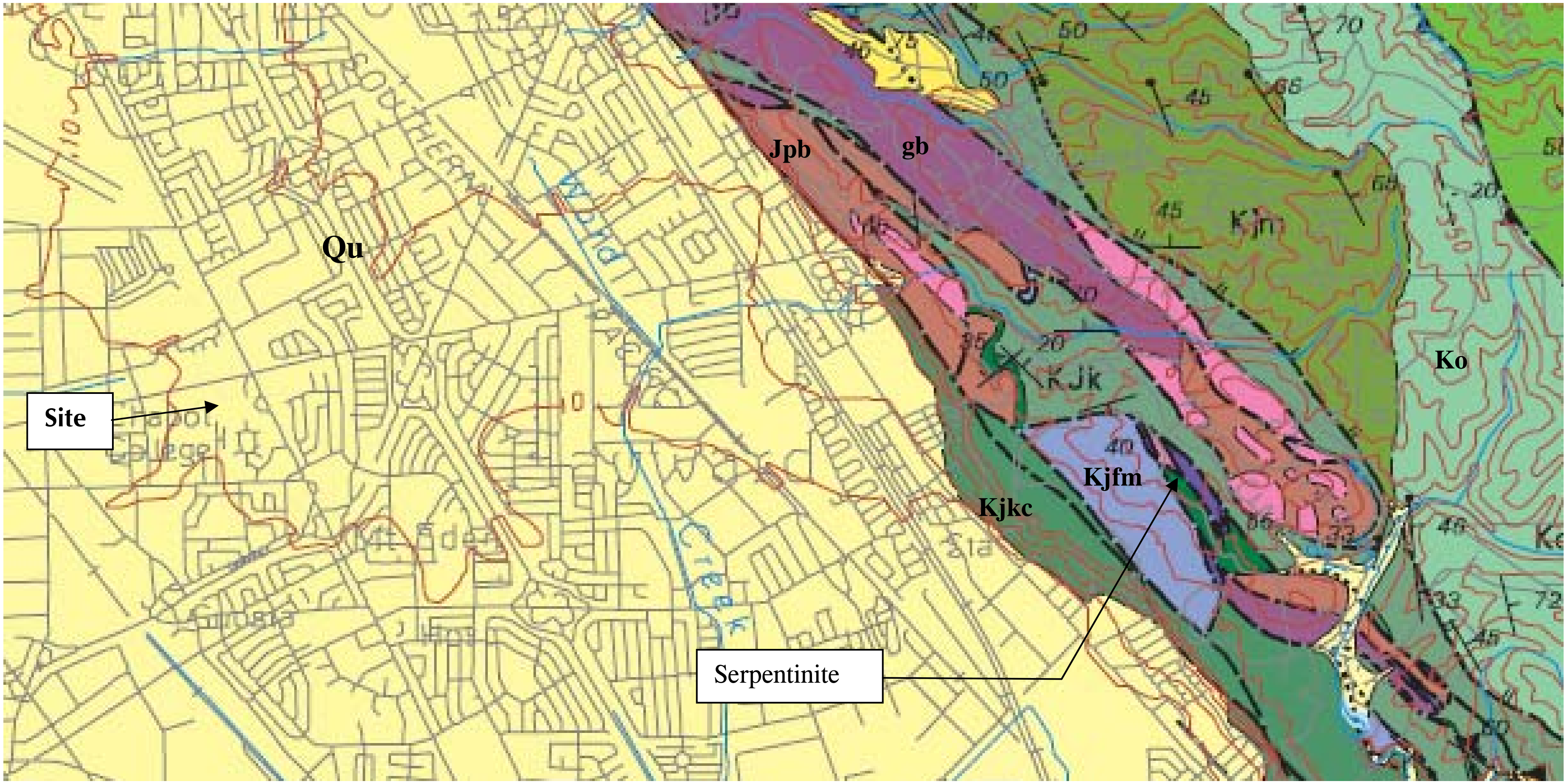
Fault Locations

Chabot College, Hayward, California

Figure 4

LFR an ARCADIS company

This Page Intentionally Left Blank for Double-Sided Copying Purposes



KJkc Knoxville Formation - conglomerate member
KJk Knoxville Formation - sandstone and shale
KJfm Melange terrane
Qu Undivided surficial deposits
Jpb Pillow basalt
Ko Oakland Formation - conglomerate and sandstone
Kjm Joaquin Miller Formation - fine sandstone and shale
Jgb Gabbro

5,000 ft

Source: Graymer et al (1996)

Geological Map

Chabot College, Hayward California

Figure 5

LFR an ARCADIS company

This Page Intentionally Left Blank for Double-Sided Copying Purposes

APPENDIX A

Probabilistic and Deterministic Seismic Hazard Assessment

This Page Intentionally Left Blank for Double-Sided Copying Purposes

A.1 Introduction

This Appendix provides general ground motion parameters, as required by the 2007 California Building Code (CBC). It also includes a site-specific ground motion analysis, as required by 2007 CBC 1614A.1.2 for sites located within 10 kilometers (km) of an active fault. The ground motion analysis was conduced in accordance with American Society of Civil Engineers (ASCE) Standard 7-05, Sections 21.2 to 21.4 (ASCE 2005).

The evaluation addressed fundamental building periods ranging from 0 to 5 seconds. Evaluation of longer periods was deemed unnecessary, since the proposed school development will consist only of one- or two-story buildings, with relatively low fundamental periods.

A.2 ASCE 7-05, Section 11.4: Seismic Ground Motion Parameters

The Site Class was determined based on previous investigations conducted by LFR (2006). Other ground motion parameters were obtained using the U.S. Geological Survey's "Earthquake Ground Motion Tool", version 5.09 (USGS 2008), using the following options:

- Geographic Region = 48 Conterminous States
- Data Edition= 2005 ASCE 7 Standard
- Latitude (Degrees) = 37.6434
- Longitude (Degrees) = -122.1082

The resulting ground motion parameters may be summarized as follows:

- $\mathbf{S_S}$ = 1.636
- $\mathbf{S_1}$ = 0.607
- **Site Class:** D (stiff soil)
- $\mathbf{F_a}$ = 1.0
- $\mathbf{F_v}$ = 1.5
- $\mathbf{S_{MS}} = F_a \times S_S = 1.0 \times 1.636 = 1.636$
- $\mathbf{S_{M1}} = F_v \times S_1 = 1.5 \times 0.607 = 0.910$
- $\mathbf{S_{DS}} = (2/3) \times S_{MS} = (2/3) \times 1.636 = 1.091$
- $\mathbf{S_{D1}} = (2/3) \times S_{M1} = (2/3) \times 0.896 = 0.607$

A.3 ASCE 7-05, Section 21.2.1: Probabilistic MCE Response Spectrum

Section 21.2.1 of ASCE (2005) requires a probabilistic maximum considered earthquake (MCE) response spectrum. Three probabilistic response spectra were generated; they are shown in Table 1 and on Figure 1.

- A **Probabilistic Design Response Spectrum** was generated in accordance with Section 11.4.5 and Figure 11.4-1 of ASCE (2005), using the ground motion parameters in Section 1.0 above.
- A **Probabilistic MCE Response Spectrum** was then generated by multiplying the Probabilistic Design Response Spectrum by 1.5, in accordance with Section 11.4.6 of ASCE (2005).
- A **Lower-Limit Probabilistic Response Spectrum** was also generated, by multiplying the Probabilistic Design Response Spectrum by 80%, in accordance with Section 21.3 of ASCE (2005).

A.4 ASCE 7-05, Section 21.2.2: Deterministic MCE Response Spectrum

Section 21.2.2 of ASCE (2005) requires a deterministic MCE response spectrum, based on 150% of the largest spectral accelerations associated with active faults within the region. The local faults with the highest associated PGAs (≥ 0.17 g) and MMIs (≥ VIII) are summarized in the following table:

Abbreviated Fault Name	Approx. Distance, miles (km)	Maximum Earthquake Mag. (Mw)	Peak Ground Accel. (g)	Est. Site Intensity, Modified Mercalli
HAYWARD (HS+HN+RC)	2.5 (4.1)	7.3	0.473	X
CALAVERAS (CS+CC+CN)	10.2 (16.4)	6.9	0.211	VIII
MOUNT DIABLO (MTD)	15.5 (25.0)	6.7	0.17	VIII
SAN ANDREAS (SAP+SAN+SAO)	15.8 (25.4)	7.8	0.201	VIII

Because the Hayward fault is so close to the Campus, it dominates the deterministic seismic hazard assessment. Only the Hayward fault was considered further. The Hayward Fault's fault characteristics as described by Cao et al. (2003):

- The Hayward fault is a right lateral strike slip fault. The combined lengths of the northern and southern sections of the Hayward fault and the associated Rogers Creek fault are150 km. The slip rate of the fault is 9 millimeters (mm) +/- 2 mm per year.

Deterministic response spectra for the maximum earthquakes on the Hayward fault were calculated using the OpenSHA (2009) Attenuation Relationship Plotter. Three deterministic response spectra were calculated for the fault using the attenuation relationships of Boore, Joyner and Fumal (1997), Abrahamson and Silva (1997), and Sadigh et al. (1997). Complete attenuation relationship parameters are shown in Table 2A. Linear interpolation was used as necessary to provide comparable response spectra for each attenuation relationship.

The spectral accelerations generated by the three attenuation relationships at each period were then averaged, to provide a Mean Deterministic Response Spectrum for the fault.

The Boore, Joyner, and Fumal (1997) attenuation relationship yielded significantly higher response accelerations than the other attenuation relationships (Figure 2A).

The Mean Deterministic Response Spectra for the four evaluated faults are shown in Table 2B and on Figure 2B. Three deterministic MCE response spectra were then generated, in accordance with Section 21.2.2 and Figure 21.2-1 of ASCE (2005):

- A **Maximum Fault-Based Deterministic MCE Response Spectrum** was generated, by multiplying the maximum deterministic responses for each period by 150%. The resulting Fault-Based Deterministic MCE Response Spectrum is presented in Table 2B and shown on Figures 2B and 2C.

- A **Lower Limit Deterministic MCE Response Spectrum** was generated, based on the previously determined values of F_a and F_v (Section 1.0) with $S_s = 1.5$ and $S_1 = 0.6$. The resulting Lower-Limit Deterministic MCE Response Spectrum is shown in Table 2C and on Figure 2C.

- A **Final Deterministic MCE Response Spectrum** was then generated, using the greater of the Maximum and Lower Limit Deterministic MCE Response Spectra. The resulting Final Deterministic MCE Response Spectrum is shown in Table 2C and on Figure 2C.

  The Lower Limit Deterministic MCE Response Spectrum governs the Final Deterministic MCE Response Spectrum at periods less than approximately 0.14 seconds.

A.5 ASCE 7-05, Section 21.2.3: Site-Specific MCE Response Spectrum

Section 21.2.3 of ASCE (2005) requires a Site-Specific MCE Response Spectrum, to be taken as the lesser of the Probabilistic MCE Response Spectrum (from Figure 1) and the Deterministic MCE Response Spectrum (from Figure 2C). These two spectra are plotted together on Figure 3A.

The probabilistic MCE spectral accelerations are typically lower, and therefore govern the Site-Specific MCE Response Spectrum at most periods (Figure 3A). The only exception is at periods of approximately 0.1 to 0.17 seconds (Figure 3B). For this interval, the deterministic MCE spectral accelerations are slightly lower; the maximum difference is approximately 9%.

In summary, the **Site-Specific MCE Response Spectrum** (Table 3A and Figure 3A) is equivalent to the Probabilistic MCE Response Spectrum (Figure 1), except at periods of approximately 0.1 to 0.17 seconds (Figure 3B).

A.6 ASCE 7-05, Section 21.2.3: Design Response Spectrum

Section 21.3 of ASCE (2005) requires a design response spectrum, where the design response accelerations are two-thirds of the site-specific MCE response accelerations (from Figure 3A). The Site-Specific MCE Response Spectrum and the corresponding **Final Design Response Spectrum** are shown in Table 4 and on Figure 4.

The Final Design Response Spectrum is nearly identical to the Probabilistic Design Response spectrum (from Figure 1). This is because:

- The Final Design Response Spectrum is two-thirds of the Site-Specific MCE Response Spectrum, as noted above;
- The Site-Specific MCE Response Spectrum, in turn, is equivalent to the Probabilistic MCE Response Spectrum, except at periods of approximately 0.1 to 0.17 seconds (Section 4.0);
- The Probabilistic MCE Response Spectrum, in turn, is 1.5 times the Probabilistic Design Response Spectrum (Section 2.0).

The factors of two-thirds and 1.5 cancel each other out, and so the Final Design Response Spectrum is nearly identical to the Probabilistic Design Response Spectrum. The only difference is at periods of approximately 0.1 to 0.17 seconds, where the Final Design Response Spectrum is ultimately based upon deterministic MCE spectral accelerations (Figure 3B).

Section 21.3 of ASCE (2005) defines a lower limit on the Final Design Response Spectrum, based on the Lower Limit Probabilistic Design Response Spectrum generated in Section 2.0 (Figure 1). The Lower Limit Design Response Spectrum is 80% of the Probabilistic Design Response Spectrum; it is shown on Tables 1 and 4 and on Figure 4.

The Lower Limit Design Response Spectrum constrains any reductions to the Probabilistic Design Response Spectrum to factors of less than 20%. In this case, the only reductions to the Probabilistic Design Response Spectrum are at periods of approximately 0.1 to 0.17 seconds, and these reductions are no more than 9% (Section 4.0). The Lower-Limit Design Response Spectrum therefore has no effect on the Final Design Response Spectrum.

A.7 ASCE 7-05, Section 21.4: Design Acceleration Parameters

Section 21.4 of ASCE (2005) requires recalculation of **design acceleration parameters**, based on the final design response accelerations at periods of 0.2, 1, and 2 seconds.

In this case, the Final Design Response Spectrum (Figure 4) is identical to the original Probabilistic Design Response Spectrum (Figure 1) at these periods, as described in Section 5.0. The two spectra differ only at periods of approximately 0.1 to 0.17 seconds. These differences do not affect the design acceleration parameters at periods of 0.2, 1, or 2 seconds.

The design acceleration parameters as required by ASCE (2005), Section 21.4 therefore remain unchanged from those defined above in Section 1.0:

- $\mathbf{S_{MS}}$ = 1.636
- $\mathbf{S_{M1}}$ = 0.910
- $\mathbf{S_{DS}}$ = 1.091
- $\mathbf{S_{D1}}$ = 0.607

Table 1
Probabilistic MCE Response Spectrum

Period (sec)	Probabilistic Spectral Acceleration		
	Design (g)	MCE (150% Design) (g)	Lower-Limit (80% Design) (g)
0.00	0.436	0.654	0.349
0.11	1.091	1.637	0.873
0.20	1.091	1.637	0.873
0.56	1.091	1.637	0.873
0.60	1.011	1.517	0.809
0.70	0.867	1.301	0.694
0.80	0.759	1.139	0.607
0.90	0.674	1.011	0.539
1.00	0.607	0.911	0.486
1.10	0.552	0.828	0.442
1.20	0.506	0.759	0.405
1.30	0.467	0.701	0.374
1.40	0.433	0.650	0.346
1.50	0.405	0.608	0.324
1.60	0.379	0.569	0.303
1.70	0.357	0.536	0.286
1.80	0.337	0.506	0.270
1.90	0.319	0.479	0.255
2.00	0.303	0.455	0.242
2.50	0.243	0.364	0.194
3.00	0.202	0.304	0.162
3.50	0.173	0.260	0.139
4.00	0.152	0.228	0.121
4.50	0.135	0.202	0.108
5.00	0.121	0.182	0.097

Design Spectral Accelerations: As per ASCE (2005), Sec. 11.4.5,
S_{DS} = 1.091, S_{D1} = 0.607

MCE Spectral Accelerations: 150 % Design, as per ASCE (2005), Sec. 11.4.6

Lower-Limit Spectral Accelerations: 80% Design, as per ASCE (2005), Sec. 21.3

Response Spectra shown graphically in Figure 1

Table 2A

Attenuation Relationship Parameters for Earthquakes on Selected Faults

Boore, Joyner, and Fumal (1997) Attenuation Relationship

Fault	DIstanceJB (km)	Magnitude	Vs30 (m/s)	Type	Component
Hayward Fault	4.1	7.3	270	Strike Slip	Average Horizontal

Abrahamson and Silva (1997) Attenuation Relationship

Fault	DIstanceRup (km)	Magnitude	AS Site Type	Type	Component	On Hanging Wall?
Hayward Fault	4.1	7.3	Deep-Soil	Strike Slip	Average Horizontal	No

Sadigh et al. (1997) Attenuation Relationship

Fault	DIstanceRup (km)	Magnitude	Sadigh Site Type	Type
Hayward Fault	4.1	7.3	Deep-Soil	Strike Slip

Deterministic response accelerations calculated using the OpenSHA (2009) Attenuation Relationship Plotter and the parameters above

Table 2B
Fault-Based Deterministic MCE Response Spectrum
Chabot College Physical Education Building

Period (sec)	Deterministic Spectral Acceleration: Hayward Fault (g)	Deterministic Spectral Acceleration: Fault-Based MCE (150% of Max.) (g)
0.00	0.511	0.766
0.10	0.875	1.312
0.20	1.175	1.762
0.30	1.276	1.914
0.40	1.279	1.918
0.50	1.224	1.835
0.60	1.160	1.740
0.70	1.081	1.622
0.80	1.006	1.508
0.90	0.933	1.400
1.00	0.861	1.292
1.10	0.806	1.209
1.20	0.753	1.130
1.30	0.703	1.055
1.40	0.656	0.984
1.50	0.610	0.915
1.60	0.572	0.859
1.70	0.538	0.807
1.80	0.503	0.754
1.90	0.470	0.705
2.00	0.438	0.657
3.00	0.262	0.393
4.00	0.173	0.259
5.00	0.109	0.164

Fault-Based Spectral Accelerations: Three response spectra calculated for each fault, using the attenuation relationships and parameters listed in Table 2A. Mean values are shown.

Response Spectra shown graphically in Figure 2B

Table 2C
Final Deterministic MCE Response Spectrum

Period (sec)	Deterministic Spectral Acceleration Fault-Based MCE (g)	Lower-Limit MCE (g)	Final MCE (g)
0.00	0.766	*1.500*	1.500
0.05	0.985	*1.500*	1.500
0.09	1.236	*1.500*	1.500
0.10	1.312	*1.500*	1.500
0.13	1.447	*1.500*	1.500
0.20	*1.762*	1.500	1.762
0.30	*1.914*	1.500	1.914
0.40	*1.918*	1.500	1.918
0.50	*1.835*	1.500	1.835
0.60	*1.740*	1.500	1.740
0.70	*1.622*	1.286	1.622
0.80	*1.508*	1.125	1.508
0.90	*1.400*	1.000	1.400
1.00	*1.292*	0.900	1.292
1.10	*1.209*	0.818	1.209
1.20	*1.130*	0.750	1.130
1.30	*1.055*	0.692	1.055
1.40	*0.984*	0.643	0.984
1.50	*0.915*	0.600	0.915
1.60	*0.859*	0.563	0.859
1.70	*0.807*	0.529	0.807
1.80	*0.754*	0.500	0.754
1.90	*0.705*	0.474	0.705
2.00	*0.657*	0.450	0.657
3.00	*0.393*	0.300	0.393
4.00	*0.259*	0.225	0.259
5.00	0.164	*0.180*	0.180

Fault-Based MCE Spectral Accelerations: From Table 2B

Lower-Limit Deterministic MCE Spectral Accelerations: As per ASCE (2005), sec. 21.2.2

Governing Values: Based on maximum, as shown in *italic* font

Response Spectra shown graphically in Figure 2C

Table 3
Site-Specific MCE Response Spectrum

Period (sec)	Spectral Acceleration Probabilistic MCE (g)	Deterministic MCE (g)	Site-Specific MCE (g)
0.00	*0.655*	1.500	0.655
0.05	*1.096*	1.500	1.096
0.09	*1.449*	1.500	1.449
0.10	1.537	*1.500*	1.500
0.11	1.637	*1.500*	1.500
0.14	1.637	*1.500*	1.500
0.15	1.637	*1.540*	1.540
0.20	*1.637*	1.760	1.637
0.30	*1.637*	1.910	1.637
0.40	*1.637*	1.920	1.637
0.50	*1.637*	1.840	1.637
0.56	*1.637*	1.780	1.637
0.60	*1.518*	1.740	1.518
0.70	*1.301*	1.620	1.301
0.80	*1.138*	1.510	1.138
0.90	*1.012*	1.400	1.012
1.00	*0.911*	1.290	0.911
1.10	*0.828*	1.209	0.828
1.20	*0.759*	1.130	0.759
1.30	*0.700*	1.055	0.700
1.40	*0.650*	0.984	0.650
1.50	*0.607*	0.915	0.607
1.60	*0.569*	0.859	0.569
1.70	*0.536*	0.807	0.536
1.80	*0.506*	0.754	0.506
1.90	*0.479*	0.705	0.479
2.00	*0.455*	0.657	0.455
3.00	*0.304*	0.393	0.304
4.00	*0.228*	0.259	0.228
5.00	0.182	*0.180*	0.180

Probabilistic MCE Spectral Response Accelerations: From Table 1

Deterministic MCE Spectral Response Accelerations: From Table 2C

Response Spectra shown graphically in Figure 2C

Table 4

Final Design Response Spectrum

Period (sec)	Spectral Acceleration Site-Specific MCE (g)	Probabilistic Design (g)	Lower-Limit Design (g)	Final Design (2/3 Site-Specific MCE) (g)
0.00	0.654	0.436	0.349	0.436
0.05	1.170	0.731	0.585	0.731
0.09	1.488	0.966	0.773	0.966
0.10	1.500	1.030	0.824	1.030
0.11	1.500	1.091	0.873	1.000
0.13	1.500	1.091	0.873	1.000
0.20	1.637	1.091	0.873	1.091
0.30	1.637	1.091	0.873	1.091
0.40	1.637	1.091	0.873	1.091
0.50	1.637	1.091	0.873	1.091
0.60	1.517	1.011	0.809	1.011
0.70	1.279	0.867	0.694	0.867
0.80	1.119	0.759	0.607	0.759
0.90	1.011	0.674	0.539	0.674
1.00	0.896	0.607	0.486	0.607
1.10	0.828	0.552	0.442	0.552
1.20	0.759	0.506	0.405	0.506
1.30	0.701	0.467	0.374	0.467
1.40	0.650	0.433	0.346	0.433
1.50	0.608	0.405	0.324	0.405
1.60	0.569	0.379	0.303	0.379
1.70	0.536	0.357	0.286	0.357
1.80	0.506	0.337	0.270	0.337
1.90	0.479	0.319	0.255	0.319
2.00	0.455	0.303	0.242	0.303
3.00	0.299	0.202	0.162	0.202
4.00	0.224	0.152	0.122	0.152
5.00	0.179	0.121	0.097	0.121

Site-Specific MCE Spectral Response Accelerations: From Table 3

Probabilistic Design Spectral Response Accelerations: From Table 1

Lower-Limit Design Spectral Response Accelerations: From Table 1

Final Design Spectral Response Accelerations: 2/3 of Site-Specific MCE Spectral Response Accelerations, as per ASCE (2005), Section 21.3

Response Spectra shown graphically in Figure 4

Figure 1. Probabilistic MCE Response Spectrum
Chabot College Physical Education Building
2% Exceedance in 50 Years, 5% Damping

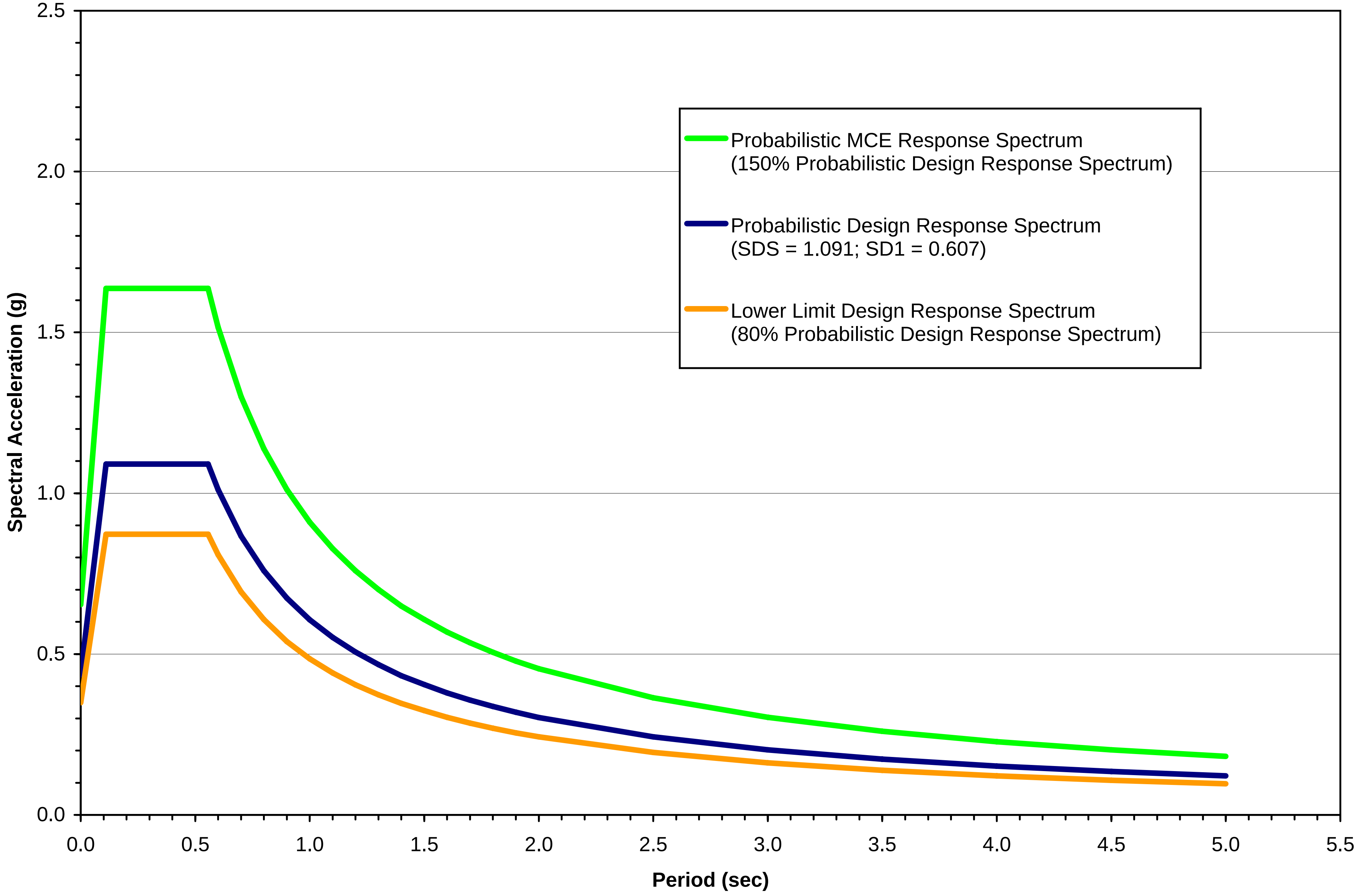


Fig. 2A. Deterministic Response Spectra, Hayward Fault
Chabot College Physical Education Building
5% Damping

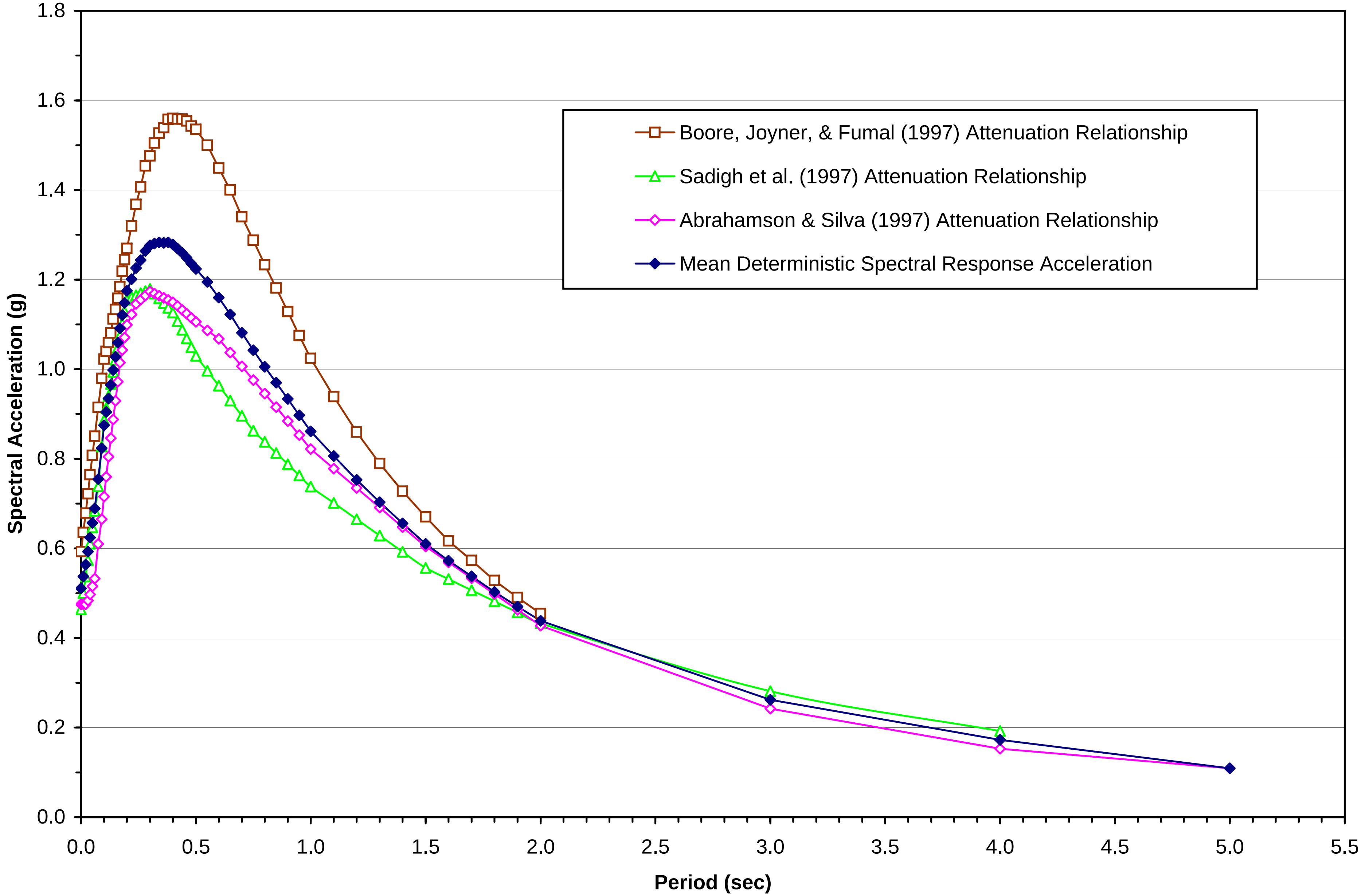


Figure 2B. Deterministic Response Spectra, All Evaluated Faults
Chabot College Physical Education Building
5% Critical Damping

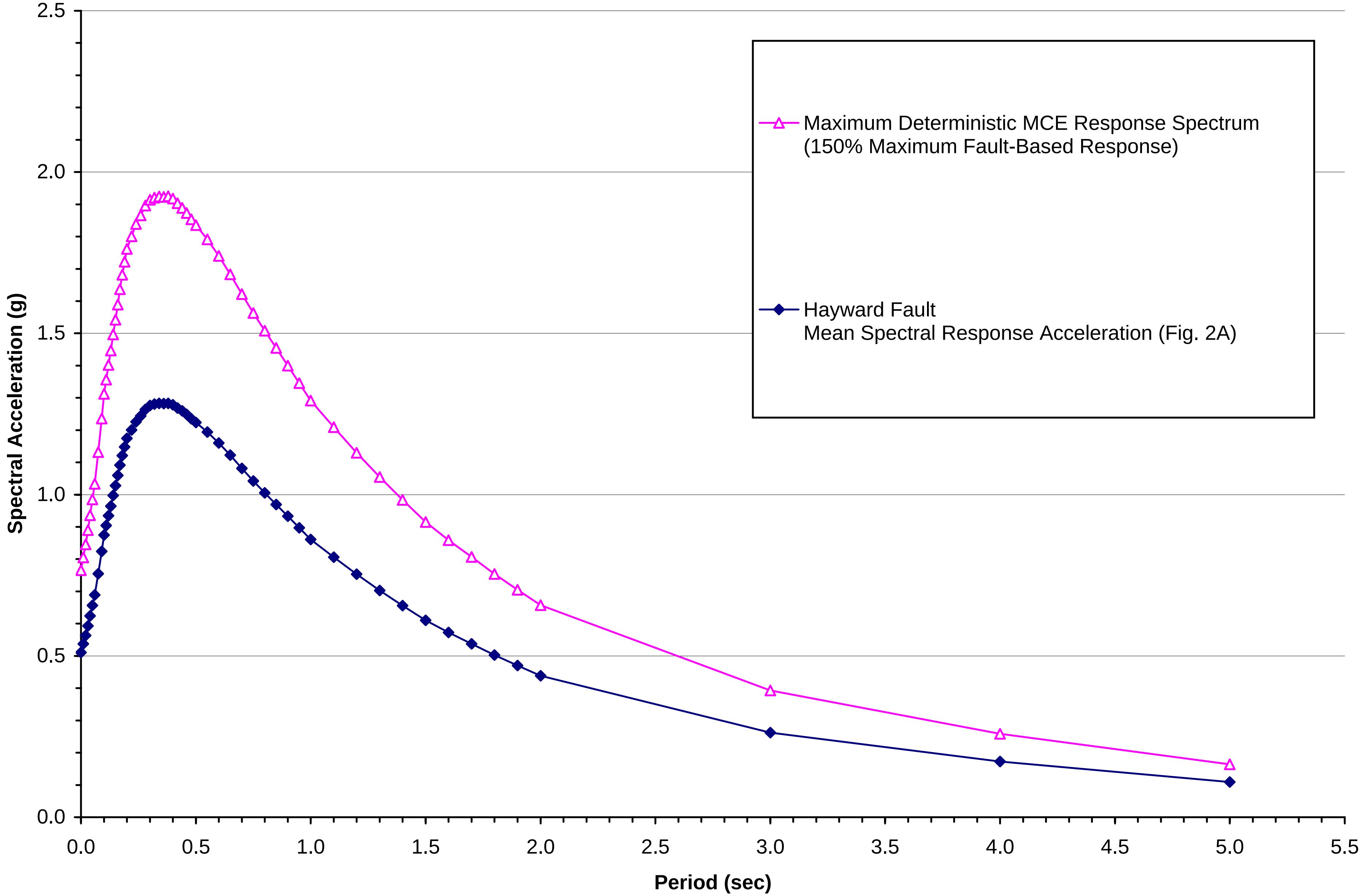
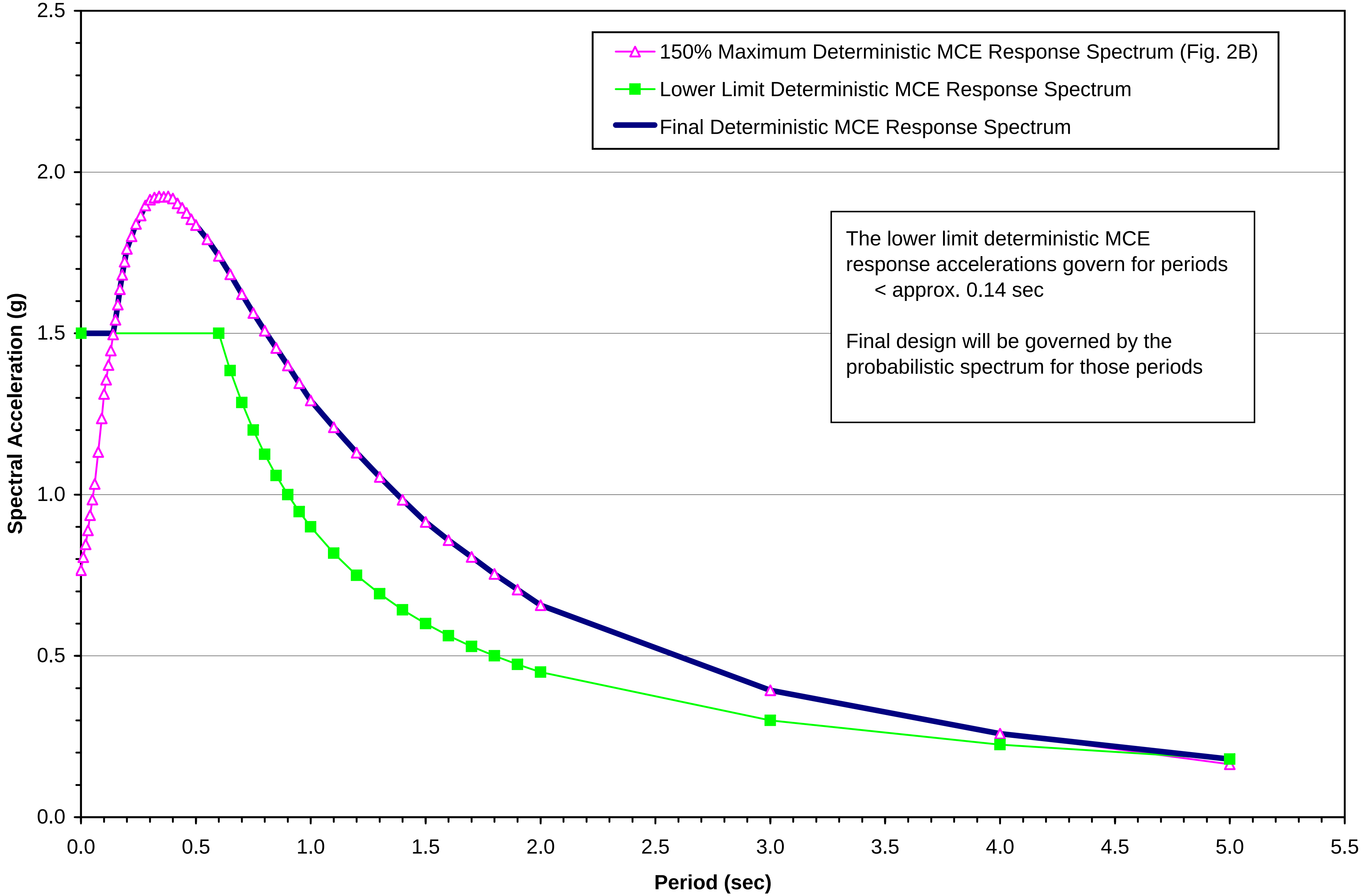


Figure 2C. Final Deterministic MCE Response Spectrum
Chabot College Physical Education Building
5% Damping



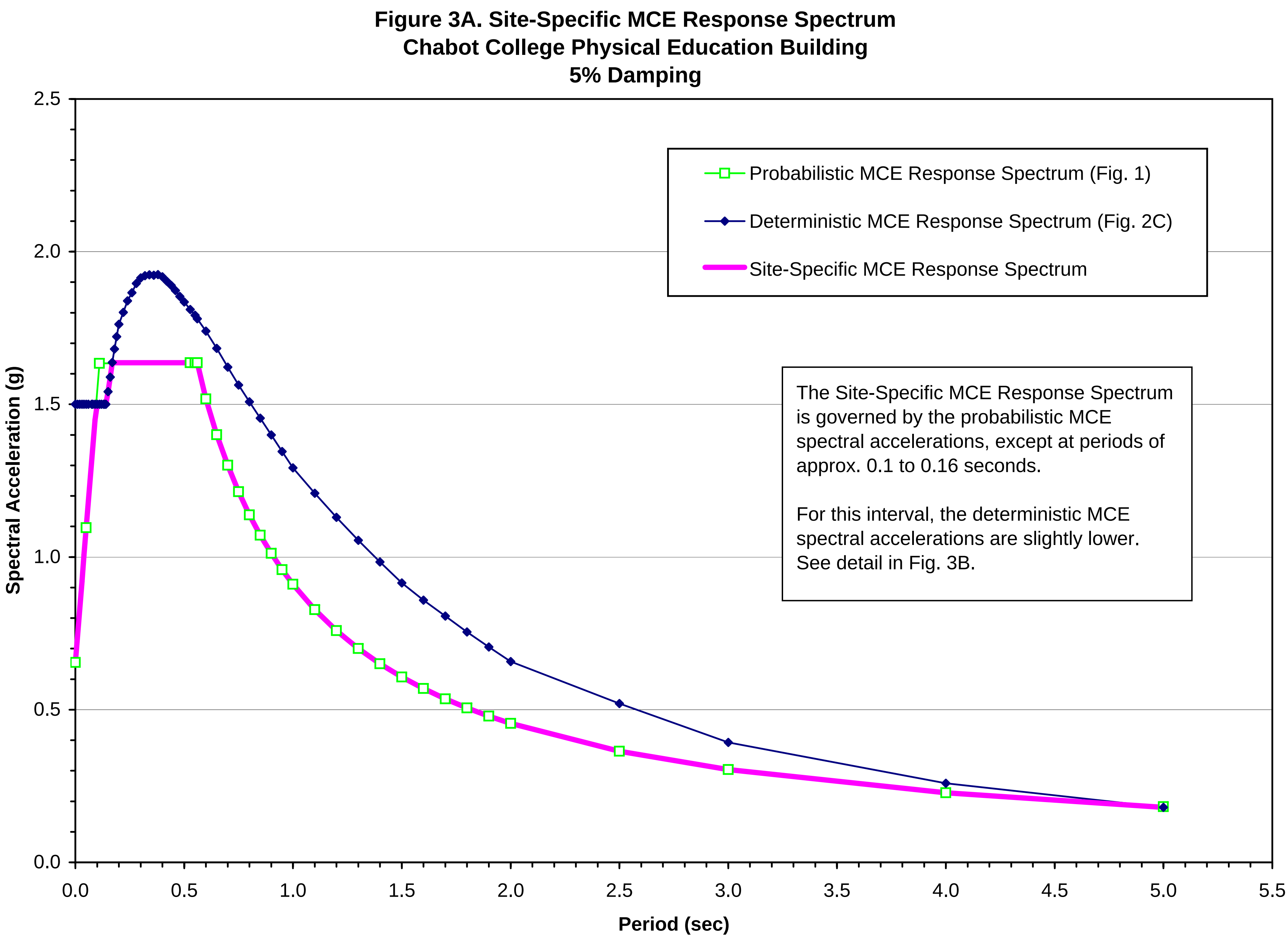
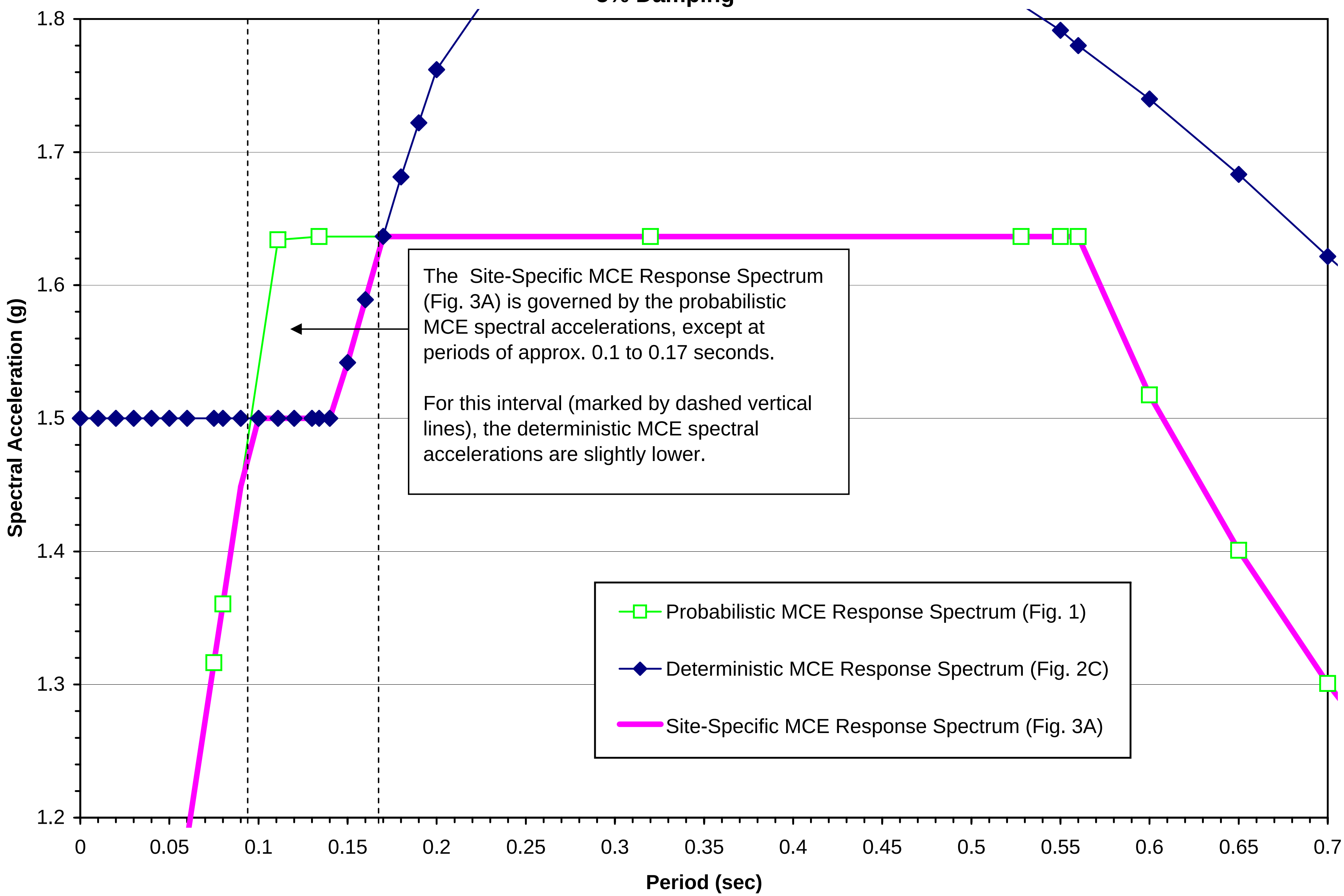


Figure 3A. Site-Specific MCE Response Spectrum
Chabot College Physical Education Building
5% Damping
Spectral Acceleration (g)
Period (sec)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
Probabilistic MCE Response Spectrum (Fig. 1)
Deterministic MCE Response Spectrum (Fig. 2C)
Site-Specific MCE Response Spectrum
The Site-Specific MCE Response Spectrum is governed by the probabilistic MCE spectral accelerations, except at periods of approx. 0.1 to 0.16 seconds.
For this interval, the deterministic MCE spectral accelerations are slightly lower. See detail in Fig. 3B.

Figure 3B. Site-Specific MCE Response Spectrum, Detail
Chabot College Physical Education Building
5% Damping



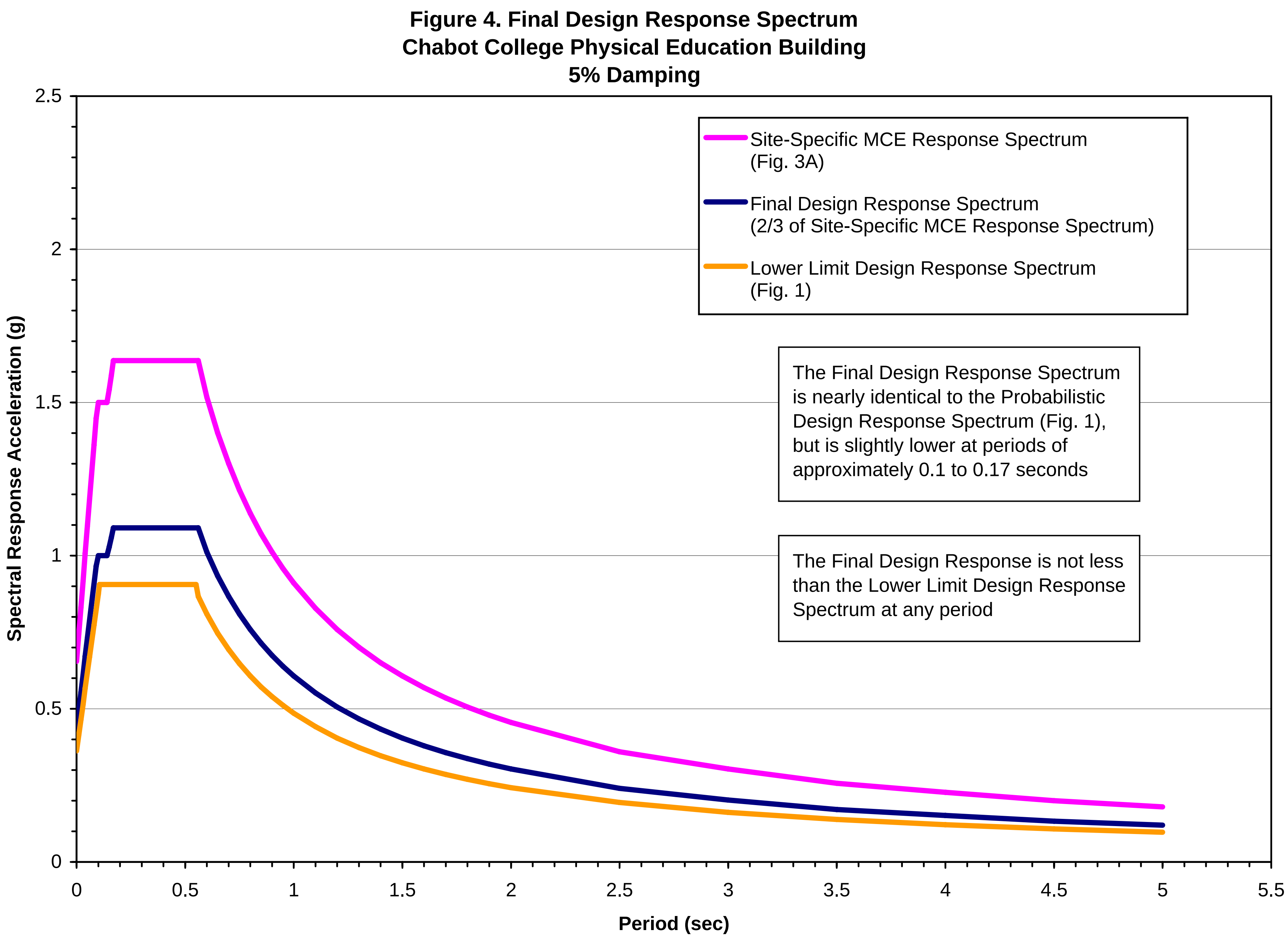


Figure 4. Final Design Response Spectrum
Chabot College Physical Education Building
5% Damping
Site-Specific MCE Response Spectrum (Fig. 3A)
Final Design Response Spectrum (2/3 of Site-Specific MCE Response Spectrum)
Lower Limit Design Response Spectrum (Fig. 1)
The Final Design Response Spectrum is nearly identical to the Probabilistic Design Response Spectrum (Fig. 1), but is slightly lower at periods of approximately 0.1 to 0.17 seconds
The Final Design Response is not less than the Lower Limit Design Response Spectrum at any period
Spectral Response Acceleration (g)
Period (sec)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5

This Page Intentionally Left Blank for Double-Sided Copying Purposes