GEOTECHNICAL AND GEOLOGICAL INVESTIGATION

for

BUILDING 334 AND PHARMACY VETERANS AFFAIRS MEDICAL CENTER

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GEOTECHNICAL AND GEOLOGICAL INVESTIGATION
BUILDING 334 AND PHARMACY
VETERANS AFFAIRS MEDICAL CENTER
Menlo Park, California

1.0 INTRODUCTION

This report presents the results of the geotechnical and geological investigation performed by Langan Treadwell Rollo for the planned Building 334 renovation and pharmacy to be constructed at the Veterans Affairs Medical Center at 795 Willow Road in Menlo Park, California (VA Menlo Park). This investigation was performed in accordance with our proposal dated 21 March 2013. This report presents our conclusions and recommendations regarding geotechnical aspects of the project.

The pharmacy is part of VA project 640-394, Post Traumatic Stress Diagnosis (PTSD) Expansion & Renovation. The PTSD renovation includes a portion of existing Building 334. We understand Building 334 will receive architectural improvements but the structural loads are not expected to change; therefore, geotechnical recommendations are not needed for this building. Our geotechnical services are limited to the pharmacy, located at another part of the VA Menlo Park campus.

The VA Menlo Park property is bound by Bay Road on the north, Willow Road and Van Buren Road on the east, S. Perimeter Road on the South, and W. Perimeter Road on the west, as shown on Figure 1. The pharmacy will consist of an addition to the south side of the existing Building 360 H Wing, as shown on Figure 2. The pharmacy will be constructed on grade and will be one story with plan dimensions of approximately 74 feet by 41 feet. The site currently consists of a level landscaping area with a ground surface elevation of approximately 24 feet.[1] We understand that existing improvements at the site will be demolished and removed prior to construction of the pharmacy addition. Additional improvements will include new utilities,

1 Elevations discussed in this report refer to Mean Sea Level datum and are based on a topographic plan provided by Polytech Associates, Inc. via email on 3 October 2014.

sidewalks, and landscaping. Based on information provided by the project structural engineer, we understand dead plus live column loads will be about 35 kips for spread footings.

2.0 SCOPE OF SERVICES

The purpose of our investigation was to evaluate subsurface conditions at the site and provide conclusions and recommendations for the geotechnical aspects of the design of the pharmacy. We understand the pharmacy has been designated an essential facility in accordance with the Department of Veterans Affairs Seismic Design Requirements H-18-8, dated August 2013. Per H-18-8, our geotechnical services for design of the pharmacy include preparation of a geologic hazards report and a geotechnical investigation report, which are combined into one document herein. Site-specific response spectra for characterization of earthquake ground motions are included in the geotechnical report per H-18-8.

In accordance with our revised proposal dated 21 March 2013, our scope of services consisted of exploring the subsurface conditions at the site and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including potential for fault rupture, ground shaking, liquefaction, lateral spreading, and seismically induced settlements
- appropriate foundation type(s)
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlements, including total and differential settlements
- site preparation and grading, including criteria for fill quality and compaction
- concrete flatwork
- subgrade preparation and moisture protection for floor slabs

- corrosion potential of near-surface soil
- seismic design parameters in accordance with ASCE 7-10
- site-specific response spectra in accordance with VA Seismic Design Requirements H-18-8, August 2013 edition
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 Current Investigation

Subsurface conditions were explored at the pharmacy site by drilling one boring, performing a dynamic penetrometer test (DPT), and advancing one cone penetration test (CPT). The approximate locations of the boring, DPT, and CPT are presented on Figure 2. Prior to performing our field investigation, we notified Underground Service Alert and retained a private underground utility locating service to check that locations of exploratory points were clear of existing utilities. Because the boring, DPT, and CPT were on federal property, a drilling permit from San Mateo County Environmental Health Services Division was not required.

The boring, designated HA-1, was drilled on 3 October 2014 to a depth of about 10 feet below the existing ground surface (bgs) by our field engineer using a hand auger. During drilling, our field engineer logged the boring and obtained representative samples of the soil encountered for classification. The boring log is presented in Appendix A on Figure A-1. The soil encountered in the boring was classified in accordance with the soil classification system presented on Figure A-2.

The DPT, designated DPT-1, was performed to a depth of about 15 feet bgs and provided soil strength information adjacent to the hand-augered boring. The DPT log is presented in Appendix A on Figure A-3.

The CPT, designated CPT-1, was advanced to a depth of about 100 feet bgs on 3 October 2014 by Middle Earth Geo Testing, Inc. of Fremont, California. The CPT was performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone-tipped probe measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measure soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data was processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered. Additionally, shear wave velocity measurements were obtained to determine the shear wave velocity profile in the upper 100 feet of the site. The CPT log, showing tip resistance, friction resistance, friction ratio, interpreted standard penetration test blow counts, and soil classifications by depth, is presented in Appendix A on Figure A-4.

Upon completion of the field investigation, the borehole was backfilled using soil cuttings and the CPT hole was backfilled with cement grout.

3.2 Laboratory Testing

At the completion of our field investigation, we re-examined the soil samples obtained from our boring in our office to confirm the field classifications and select representative samples for geotechnical laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. The samples were tested to obtain moisture content, Atterberg limits, and corrosion potential. The laboratory test results are presented in Appendix B and on the boring log in Appendix A.

3.3 Previous Investigation by Others

A previous investigation was performed for Building 360 by Fugro West, Inc., the results of which were presented in a report dated 18 November 2005. The approximate locations of the nearest borings and CPTs are presented in Figure 2; the borings and CPT logs are presented in Appendix C.

4.0 SUBSURFACE CONDITIONS

Subsurface information from our field investigation indicate the site is generally underlain by layers of soft to hard clay with variable sand content and medium dense to very dense sand with variable gravel and fines content to the maximum depth explored of 100 feet bgs. Results of Atterberg limits tests performed on the near-surface clay indicate it has a moderate expansion potential.[2]

In boring HA-1, a 3-foot-thick layer of medium stiff to stiff clay was encountered below the ground surface to about Elevation 20.5 feet. The near-surface clay is underlain by soft to medium stiff clay with variable sand and silt content to the termination of the boring at about 10 feet bgs (corresponding to Elevation 13.5 feet).

In the CPT, the interpreted soil types predominantly consist of medium stiff to very stiff clay and silt. A 9-foot-thick layer of medium dense to very dense sand to silty sand was encountered at a depth of about 14 feet bgs (corresponding to approximately Elevation 9.5 feet). A second sand layer, about 7 feet thick, was encountered at a depth of about 45 feet (corresponding to about Elevation -21.5 feet). Occasional, thinner silty sand layers were encountered interbedded with the clay and silt.

Groundwater was encountered in CPT-1 at a depth of 24 feet bgs, corresponding to approximate Elevation -0.5 feet. Groundwater was encountered in borings drilled in the site vicinity by others at about Elevation 8 feet (corresponding to about 15.5 feet bgs at the pharmacy site). The groundwater levels observed during the field investigation do not represent stable groundwater table conditions. The groundwater level is expected to vary due to seasonal fluctuations of rainfall.

[2] Expansive soil undergoes large volume changes with changes in moisture content (i.e. it shrinks when dried and swells when wetted.

5.0 REGIONAL GEOLOGY

The site is located within the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates, and subsequent strike-slip faulting along the San Andreas fault system.

According to the Geologic Map of the Palo Alto and Part of the Redwood Point 7 1/2' Quadrangles, San Mateo and Santa Clara Counties, California (Pampeyan, 1993), the site is located in an area underlain by Holocene age (approximately 11,000 years old to present) medium-grained alluvium (see Figure 3, Regional Geologic Map). This unit is usually up to about 20 feet thick and characterized as unconsolidated to moderately consolidated, moderately sorted sand, silt, and clayey silt deposited along the edge of coarse-grained alluvial fans, forming much of the flatland alluvial plain along the bay margin. The unit interfingers with coarse- and fine-grained alluvium.

The Holocene age alluvium is underlain by older, Pleistocene age (approximately 11,000 to 1.8 million years old) alluvium. The older alluvium generally consists of weathered, unconsolidated to moderately consolidated gravel, sand, and silt, grading coarser to the southwest. A prior borehole shown on Figure 3 drilled along the western side of the campus indicates that the alluvium is underlain by Franciscan assemblage bedrock at about Elevation 536 feet relative to Mean Sea Level datum.

6.0 REGIONAL SEISMICITY AND FAULTING

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras faults. These and other faults of the region are shown on Figure 4. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated maximum

Moment magnitude[3], M_w, [2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) and Cao, et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista - Shannon	8.0	Southwest	6.50
N. San Andreas - Peninsula	11	Southwest	7.23
N. San Andreas (1906 event)	11	Southwest	8.05
Total Hayward	20	Northeast	7.00
Total Hayward - Rodgers Creek	20	Northeast	7.33
San Gregorio Connected	26	West	7.50
Total Calaveras	28	East	7.03
N. San Andreas - Santa Cruz	35	Southeast	7.12
Mount Diablo Thrust	41	Northeast	6.70
Zayante-Vergeles	45	Southeast	7.00
Greenville Connected	49	Northeast	7.00
Green Valley Connected	50	Northeast	6.8

Figure 4 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1, 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 5) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and

[3] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 km in length. It had a maximum intensity of XI (MM), a M_w of about 7.0, and was felt 560 km away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 54 km from the site. The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa Fault, approximately 80 km north of the site, with a M_W of 6.0.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w=6.2).

The site is located about 8 km southwest of the Monte Vista fault zone. This fault zone is a portion of the larger Foothills thrust fault system that bounds the southwest margin of the Santa Clara Valley. The fault zone offsets Quaternary alluvium and Santa Clara Formation materials, and was noted to have experienced slip during the 1989 Loma Prieta Earthquake.

Aeromagnetic surveys performed in the area revealed several linear, positive magnetic anomalies that have been interpreted as steeply dipping, planar serpentinite bodies at depth within the Franciscan assemblage bedrock, formed in zones of crustal weakness (Brabb and Hanna, 1981). A subsequent gravity survey revealed a broad, negative isostatic residual anomaly in the same area, which was interpreted to represent a serpentine body at depth, beneath greywacke sandstone (Carle and Langenhein, 1990). The serpentinite bodies have been interpreted to be occurring in a wide zone of faulting, designated the Redwood City Fault Zone, which has been deemed inactive during Holocene time as they have not been found to offset sediments within the overlying alluvium.

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 30-year probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area to be about 63 percent. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward – Rodgers Creek	31
North San Andreas	21
Calaveras	7
San Gregorio Connected	6
Mount Diablo Thrust	1

7.0 DISCUSSION AND CONCLUSIONS

We conclude that from a geotechnical engineering standpoint, the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications, and are implemented during construction. The primary geotechnical concerns for the pharmacy project are the presence of moderately expansive near-surface soil, soft clay below the foundation level, and potentially liquefiable soil below the water table. Our conclusions regarding seismic hazards, the most appropriate foundation type(s), settlement, and other geotechnical issues are presented in this section.

7.1 Seismic and Geologic Hazards

During a major earthquake on one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[4] lateral spreading,[5] and cyclic densification.[6] We used the results of the boring and CPT to evaluate the potential for these phenomena to occur at the site. The results of our evaluation are presented below.

7.1.1 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no active faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is low.

7.1.2 Soil Liquefaction and Associated Hazards

Liquefaction is a phenomenon in which saturated soil temporarily loses strength from the build-up of excess pore water pressure, especially during earthquake-induced cyclic loading. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. We evaluated liquefaction potential at the site in accordance with Special Publication 117A, *Guidelines for Evaluating and Mitigating Seismic Hazards Zones in California*, dated 11 September 2008. The California Geological Survey (CGS) has prepared a map titled *State of California Seismic Hazard Zones, Palo Alto Quadrangle,* dated 18 October 2006. This map was prepared in

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4 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

5 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

6 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

accordance with the Seismic Hazards Mapping Act of 1990. Based on this map, the site is within a liquefaction hazard zone, as shown on Figure 6.

A peak ground acceleration (PGA) of 0.53g was used for the project; this PGA is from the site-specific response spectrum (discussed in Appendix D) for the geometric mean Maximum Considered Earthquake (MCE_G). We assumed an earthquake magnitude of 8.05, which is the maximum Moment magnitude for the San Andreas Fault, 11 km from the site. A design groundwater depth of 15.5 feet bgs (corresponding to the water level observed at about Elevation 8 feet in the nearby borings by others), was used in our liquefaction analyses.

The liquefaction analysis was performed in accordance with the methodology presented in the publication titled *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, prepared by the National Center for Earthquake Engineering Research (NCEER), dated 31 December 1997. The susceptibility of sand to liquefaction under seismic loading was evaluated in general accordance with the procedure presented by Seed and Idriss (1982). The results of our liquefaction analysis using the CPT data indicates there are thin layers of medium dense granular soil below the groundwater table are susceptible to liquefaction ($FS_{liq}<1.3$) following a major earthquake on a nearby fault.

Based on our liquefaction analysis, we conclude that up to about 2/3 inch of liquefaction-induced total settlement may occur in the vicinity of the pharmacy as a result of a major earthquake on a nearby fault. Differential settlement equivalent to the total settlement of 2/3 inch may occur over short distances.

The potential for liquefaction-induced ground rupture and sand boils to occur at the site depends on the thickness of the liquefiable soil layer relative to the thickness of the overlying non-liquefiable material. Ishihara (1985) presented an empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced surface ruptures and sand boils would be expected to occur under a given level of shaking for a liquefiable layer overlain by a non-liquefiable surficial layer. The potentially liquefiable soil layers encountered in the CPT are between ½ and 2½ feet thick, and are located below a depth of 14 feet bgs. We conclude that the potential for manifestations of liquefaction to occur at the ground surface is low.

7.1.3 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open slope cut, or in the direction of a regional slope or gradient. The potential for lateral spreading to occur at a site is typically evaluated using an empirical relationship developed by Youd et al. (2002). This relationship incorporates the thickness of the liquefiable layer, the fines content and mean grain-size diameter of the liquefiable soil, the relative density of the liquefiable soil, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions (such as a free face or edge of shoreline), to estimate the horizontal ground movement. The ground surface at the site is relatively level. The potentially-liquefiable soil layers encountered at the site are thin (2 feet thick or less), with interpreted $(N_1)_{60}$ values greater than 15. Typically layers with interpreted $(N_1)_{60}$ values greater than 15 are not considered to have the potential for lateral spreading. Therefore, we conclude the potential for lateral spreading at the site is low.

7.1.4 Cyclic Densification

Cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in settlement. Granular soil was not encountered in boring HA-1. In CPT-1, about 2 feet of soil interpreted as sandy silt, silty sand, and sand was encountered above the design groundwater level of Elevation 8 feet. We compute that cyclic densification settlement of less than ¼ inch may occur as a result of strong shaking from a large earthquake on a nearby fault, with a possibility of equivalent differential settlement between building columns.

7.1.5 Landslides, Erosion, and Seepage

The site is relatively level and groundwater is anticipated to be below about 15.5 feet bgs. We conclude the potential for geologic hazards such as landsliding, erosion, and seepage to occur at the site is very low.

7.2 Foundation Support

Based on our field investigation, we anticipate the soil exposed at the foundation level of the pharmacy addition will be soft to stiff clay. The soil at the foundation level has low to moderate strength and relatively low compressibility.

The site is underlain by moderately expansive near-surface soil. Expansive soil is subject to high volume changes during seasonal fluctuations in moisture content, which can cause cracking of foundations and floor slabs. The detrimental effects of near-surface expansive soil can be mitigated by moisture conditioning the expansive soil below slabs, placing non-expansive fill below slabs, supporting foundations below the zone of severe moisture change, and/or designing foundations to resist the movements associated with the volume changes. Because expansive soil is present, we conclude the pharmacy foundation should be deepened to reduce the potential for movement due to moisture change. We conclude the pharmacy building can be supported on deepened isolated spread footings at interior column locations and continuous, deepened perimeter footings bearing on native soil.

We estimate the total static settlement of properly constructed spread footings due to static loading, designed using the recommendations presented in Section 8.2, will be less than ½ inch, with up to ¼ inch of differential settlement across a horizontal distance of 25 feet. In addition to the static settlements, seismically-induced settlement may occur during a major earthquake, as discussed in Section 7.1.

7.3 Floor Slabs

The near-surface soil is generally medium stiff to stiff clay, and we conclude the floor slab may be supported on grade. Because the near-surface soil is moderately expansive, the floor slab and capillary break/vapor retarder (recommended in Section 8.3) should be underlain by at least 12 inches of non-expansive soil to mitigate the potential for movement of the slab. The non-expansive soil may consist of imported select fill or lime-treated native soil.

7.4 Corrosion Potential

We performed corrosivity tests on soil samples collected from boring HA-1 at 3 feet bgs. The soil samples were tested in accordance with Caltrans and ASTM protocols by Environmental Technical Services (ETS) of Petaluma, California. The corrosivity test results are presented in Appendix C on Figure C-2.

8.0 RECOMMENDATIONS

Our recommendations regarding earthwork, foundation design, floor slabs, pavement design, and other geotechnical aspects of this project are presented in this section.

8.1 Earthwork

8.1.1 Site Preparation

Site preparation should include removal of all existing structures, foundations, slabs, pavements (if any), and underground utilities within the footprint of the planned development. All areas to receive improvements should be stripped of vegetation and organic topsoil. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the landscape architect. Any existing fill encountered should be removed and recompacted. If soft clay is exposed during site preparation, it may need to be stabilized by overexcavating and replacing it with a layer of reinforcing geotextile and crushed rock.

Underground utilities should be removed to the service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the planned construction, they may be abandoned in-place, provided the lines are filled with lean concrete or cement grout to the limits of the project. Voids resulting from demolition activities should be properly backfilled with engineered fill as described in Section 8.1.3.

From a geotechnical standpoint, concrete and asphalt generated by demolition (if any) may be crushed and reused providing it is free of organic material and rocks or lumps greater than three inches in greatest dimension. The acceptability of using crushed asphalt at the site should be verified by the VA and architect. Where crushed asphalt pavement materials are used as fill, particles between 1-1/2 and 3 inches in greatest dimension should comprise no more than 30 percent of the fill by weight.

8.1.2 Subgrade Preparation

The floor slab and underlying capillary break/vapor retarder (recommended in Section 8.3) should be underlain by at least 12 inches of non-expansive soil consisting of either select fill or lime-treated native soil, as described in the following sections. The soil subgrade should be kept moist until it is covered by fill or improvements.

Slab-on-Grade

Placement of Select Fill (Alternative No. 1)

If Alternative No. 1 is selected, we recommend the building pad be overexcavated to allow placement of at least 12 inches of select fill beneath the slab-on-grade floor and underlying capillary moisture break. The limit of overexcavation for select fill placement should extend at least five feet beyond the building footprint. If the building pad will be raised at least 12 inches above the existing ground surface following demolition, overexcavation will not be necessary. However, the fill used to raise the grade should be non-expansive. The native expansive soil at the base of the overexcavation or at current grade prior to raising grades should be scarified to a depth of at least 12 inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to between 88 and 92 percent relative compaction.[7] If non-expansive soil is exposed at the base of the overexcavation, it should be scarified to a depth of at least 6 inches, moisture-conditioned to at least two percent above optimum moisture content, and compacted to at least 90 percent relative compaction.

Lime Treatment (Alternative No. 2)

If Alternative No. 2 is selected, the upper 12 inches of the building pad (measured below the capillary moisture break) should be treated in place with between four to eight percent (to be determined by the contractor) dolomitic quicklime by dry weight of soil. The limit of lime treatment should extend at least five feet beyond the building footprint. A specialty subcontractor typically performs lime treatment, and we recommend this work be performed only by an experienced contractor. The contractor should determine the percent lime to be used. Prior to lime treatment, we recommend the site be graded to a level pad elevation in accordance with our previous recommendations and all below-grade obstructions be removed. The soil treated with lime should be mixed and compacted in one lift. The lime should be thoroughly blended with the soil and allowed to cure for 24 hours prior to compaction. The lime-treated soil should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. Lime-treated soil should be removed from landscaping areas as it will inhibit growth of vegetation. It should be noted that disposal of lime-treated soil is typically expensive because of the high pH of the treated soil.

Concrete Flatwork Areas

As a minimum preparation for exterior concrete flatwork, including patio slabs and sidewalks, the upper 12 inches of expansive native soil at subgrade should be moisture-conditioned to at

7 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the latest ASTM D1557 laboratory compaction procedure.

least three percent above optimum moisture content and compacted to between 88 and 92 percent relative compaction.

If it is desirable to reduce the potential for differential movement and cracking, exterior concrete flatwork should be underlain by at least 12 inches of select fill, lime-treated soil, or Caltrans Class 2 aggregate base (AB), as recommended in Section 8.4. Select fill and Class 2 AB in concrete flatwork areas should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction. Lime-treated soil should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction.

8.1.3 Fill Placement

We anticipate fill placement at the site will consist primarily of minor grading for the building pad and backfill for utility trenches. Prior to placement of general site fill, the subgrade soil should be scarified, moisture-conditioned, and recompacted as recommended in Section 8.1.2.

If native expansive clay is to be used as general site fill, it should be moisture-conditioned to at least three percent above optimum moisture content, placed in horizontal lifts not exceeding eight inches in loose thickness, and compacted to between 88 and 92 percent relative compaction.

Select fill should consist of imported or on-site soil that is non-corrosive, free of organic matter and hazardous material, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. In addition, select fill placed outside the building footprint should contain at least 20 percent fines (particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate beneath slabs. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction for fill thickness equal to or less than five feet and 95 percent relative compaction for fill thickness greater than five feet.

We should approve all sources of engineered fill at least three days before use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material.

8.1.4 Utility Trenches

Excavations for utility trenches can be made with a backhoe. All trenches should conform to the current CAL-OSHA requirements for slopes, shoring, and other safety concerns. Site Condition C (swelling soils) should be used for utility boxes designed using Section 33 63 00, Appendix II, of the VA Specifications.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. If groundwater is encountered during trench excavation, the gravel used as bedding and cover should be replaced with Caltrans Class 2 permeable material below the water level, or the open-graded gravel used as bedding and cover should be wrapped in filter fabric (Mirafi 140N or equivalent) to reduce the potential for infiltration of fines.

Backfill for utility trenches and other excavations is also considered fill and should be placed and compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where utility trenches backfilled with sand or gravel enter the building pads, an impermeable plug consisting of native clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The plug should extend from the bottom of the trench to the subgrade elevation. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

8.2 Foundation Support

Based on our field investigation, we anticipate the soil exposed at the foundation level of the pharmacy addition will be soft to stiff clay. The soil at the foundation level has low to moderate strength and relatively low compressibility. Considering the building loads are relatively light, we conclude the pharmacy can be supported on a combination of isolated interior spread footings and deepened continuous perimeter footings bearing on native soil. Because of the presence of near-surface expansive soil, foundations should be deepened below the zone of severe moisture change, as discussed in Section 7.2.

Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. To reduce the potential for movement of the footings due to shrink and swell of the expansive clay, we recommend the perimeter footings be bottomed at least 24 inches below the lowest adjacent exterior soil subgrade or the top of the select fill or lime-treated layer (recommended in Section 8.1.2), whichever is deeper. Interior footings should extend at least 24 inches below the lowest adjacent soil subgrade (measured from the top of the select fill or lime-treated soil). The footings should bear at the same level as or below the adjacent Building 360 footings to reduce the potential for surcharging them. If the new footings will bear below adjacent Building 360 footings, the existing footings will need to be underpinned, and we should be contacted to provide recommendations for underpinning if this condition exists. In addition, footings located adjacent to utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent trench.

Footings should be designed using an allowable bearing pressure for dead plus live loads of 1,500 pounds per square foot (psf). The allowable bearing pressure may be increased by one-third for total loads, including wind or seismic forces, and includes factors of safety of at least 2.0 and 1.5 for dead plus live loads and total loads, respectively. To design footings using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 36 kips per cubic foot; this modulus is representative of the anticipated settlement under the building loads provided.

Lateral loads can be resisted by a combination of passive pressure acting on the vertical faces of the footings and friction along the base of the footings. We recommend passive resistance be calculated using the following values:

- an equivalent fluid weight (triangular distribution) of 350 pounds per cubic foot (pcf) in select fill
- a uniform pressure of 2,000 psf in lime-treated soil
- a uniform pressure of 650 psf for on-site soil.

The passive resistance gained in the upper foot of soil should be ignored unless the soil is confined by slabs or pavement. Frictional resistance at the base of the footings should be computed using a friction coefficient of 0.25. These values include a factor of safety of about 1.5. Uplift loads may be resisted by the weight of the footing and any overlying soil.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If soft soil is encountered at the bottom of a footing excavation, it should be overexcavated to a depth of one foot and the overxcavation should be backfilled with a layer of reinforcing geotextile (Mirafi 500X or equivalent) and Caltrans class 2 aggregate base, which should be compacted to at least 90% relative compaction. If non-engineered fill is encountered in a footing excavation, the fill should be overexcavated to expose native soil. The excavated material should be replaced with either structural concrete or sand-cement slurry with a 28-day compressive strength of at least 50 pounds per square inch (psi). The bottoms and sides of excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will eventually heave, which may result in cracking and distress. We should check foundation excavations prior to placement of reinforcing steel to confirm suitable bearing material is present. We should recheck the condition of the excavations just prior to concrete placement to confirm the excavations are sufficiently moist.

8.3 Floor Slabs

The near-surface soil is generally medium stiff to stiff clay, and we conclude the floor slab may be supported on grade. To mitigate the potential for movement of the slab where expansive soil is present, the building slab-on-grade and capillary break/vapor retarder should be underlain by at least 12 inches of non-expansive soil consisting of either select fill or lime-treated native soil, as recommended in Section 8.1.2. If the previously-compacted soil subgrade is disturbed during foundation and utility excavation, the subgrade should be scarified, moisture-conditioned, and rerolled to provide a firm, unyielding surface prior to construction of the slab-on-grade floor. To further reduce the potential for cracking of slab-on-grade floor, we recommend the slab be reinforced with at least No. 4 bars spaced at 18 inches, each way.

Where moisture on the floor slab is undesirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor to reduce water vapor transmission through the floor slab. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. 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. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 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; however, there should be no free water present in the sand. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio – less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

8.4 Concrete Flatwork

If it is desirable to reduce the potential for differential movement and cracking, exterior concrete flatwork should be underlain by at least 12 inches of select fill, lime-treated soil, or Class 2 aggregate base (AB), which should extend at least two feet beyond the slab edges. Even with 12 inches of select fill, lime-treated soil, or AB, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to the building, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries. Recommendations for subgrade preparation for concrete flatwork are provided in Section 8.1.2.

8.5 Drainage

Positive surface drainage should be provided around the building to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the structures, we recommend the ground surface within a horizontal distance of ten feet from the building slope down away from the building with a surface gradient of at least five percent in unpaved areas and two percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. Infiltration basins or bioswales should not be placed within five feet of the foundations. Because the subgrade soil consists predominantly of clay, it will have a relatively low permeability. If infiltration basins, bioswales, or permeable pavement are planned, they should be lined with an impermeable membrane and drains should be provided that direct the water to an appropriate outlet.

8.6 Irrigation and Landscaping Limitations

The use of water-intensive landscaping around the perimeter of the building should be avoided to reduce the amount of water introduced to the expansive clay subgrade. In addition, irrigation of landscaping around the building should be limited to drip or bubbler-type systems. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which has been known to cause large differential settlement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

Moderately expansive native clay is expected to be present at or near the subgrade level. For this condition, prior experience and industry literature indicate some species of high water-demand[8] trees can induce ground surface settlement by drawing water from the expansive soil and causing it to shrink. Where these types of trees are planted adjacent to structures, the ground-surface settlement may result in damage to the structures. This problem usually occurs ten or more years after project completion as the trees reach mature height. To reduce the risk of tree-induced, ground-surface settlement, we recommend trees of the following genera not be planted within a horizontal distance from the buildings equal to the mature height of the tree: Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus (simple-leafed), Ulmus, Cupressus, Chamaecyparis, and Cupressocyparis. Because this is a limited list and does not include all genera that may induce ground-surface settlement, the project landscape architect should use judgment in limiting other types of trees with similar properties in the vicinity of the building.

8.7 Seismic Design Criteria

We developed site-specific response spectra with the guidelines in ASCE 7-10/2012 IBC for the pharmacy addition site and provided mapped values per ASCE 7-10/2012 IBC.

8 "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil subgrade, rather than soil suction exerted by the root system.

8.7.1 2012 International Building Code/ASCE 7-10 Mapped Values

On the basis of the results of our subsurface investigation, we conclude the site is classified as a stiff soil site with an average shear wave velocity in top 30 meters (100 feet), VS30, of 245 meters per second (803 feet per second). For seismic design in accordance with the provisions of ASCE 7-10/2012 IBC we recommend the following mapped parameters be used:

- site class D
- site coefficient values F_a and F_v of 1.0 and 1.5, respectively
- mapped short (S_s) and one-second (S_1) spectral acceleration values for the risk targeted Maximum Considered Earthquake (MCE_R) of 1.500g and 0.639g, respectively
- spectral acceleration values S_{Ms} and S_{M1} for the MCE_R of 1.500g and 0.959g, respectively
- spectral acceleration values for the Design Earthquake (DE) of S_{Ds} and S_{D1} of 1.000g and 0.639g, respectively.

8.7.2 Site-Specific Spectra

We performed probabilistic seismic hazard analysis and deterministic seismic hazard analysis to develop site-specific horizontal response spectra consistent with the provisions of Chapter 21 of ASCE 7-10 and 2012 IBC.

The details of our analyses are presented in Appendix D. We used appropriate Next Generation Attenuation (NGA) relationships to develop the site-specific spectra. Table 4 presents the recommended horizontal MCE_R and DE spectra in accordance with ASCE 7-10. The recommended spectra are provided on Figure 7.

TABLE 4
Recommended MCE_R and DE Horizontal Acceleration Response Spectra (g)
Damping Ratio of 5 percent

Period (seconds)	MCE_R	DE
0.01	0.630	0.420
0.05	0.960	0.640
0.10	1.347	0.898
0.14	1.500	1.000
0.20	1.500	1.000
0.30	1.500	1.000
0.40	1.500	1.000
0.50	1.500	1.000
0.60	1.500	1.000
0.75	1.346	0.897
1.00	1.209	0.806
1.50	1.003	0.669
2.00	0.839	0.559
3.00	0.612	0.408
4.00	0.460	0.307

8.7.3 Site-Specific Site Coefficients

Because the site-specific procedure was used to determine the recommended MCE_R and DE response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-10 should be used as shown in Table 5.

TABLE 5

Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g)
S_{MS}	1.500
S_{M1}	1.678[9]
S_{DS}	1.000
S_{D1}	1.118[9]

9.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Langan Treadwell Rollo should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing services during subgrade preparation, excavation and installation of footings, and fill placement and compaction, including utility trench backfill. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the time of the investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan Treadwell Rollo should be notified to make supplemental recommendations, if necessary.

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9 The two second spectral values govern the determination of S_{M1} and S_{D1}.

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FIGURES

NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

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BUILDING 334 AND PHARMACY VETERANS AFFAIRS MEDICAL CENTER Menlo Park, California	SITE LOCATION MAP		
LANGAN TREADWELL ROLLO	Date 10/09/14	Project No. 750612403	Figure 1

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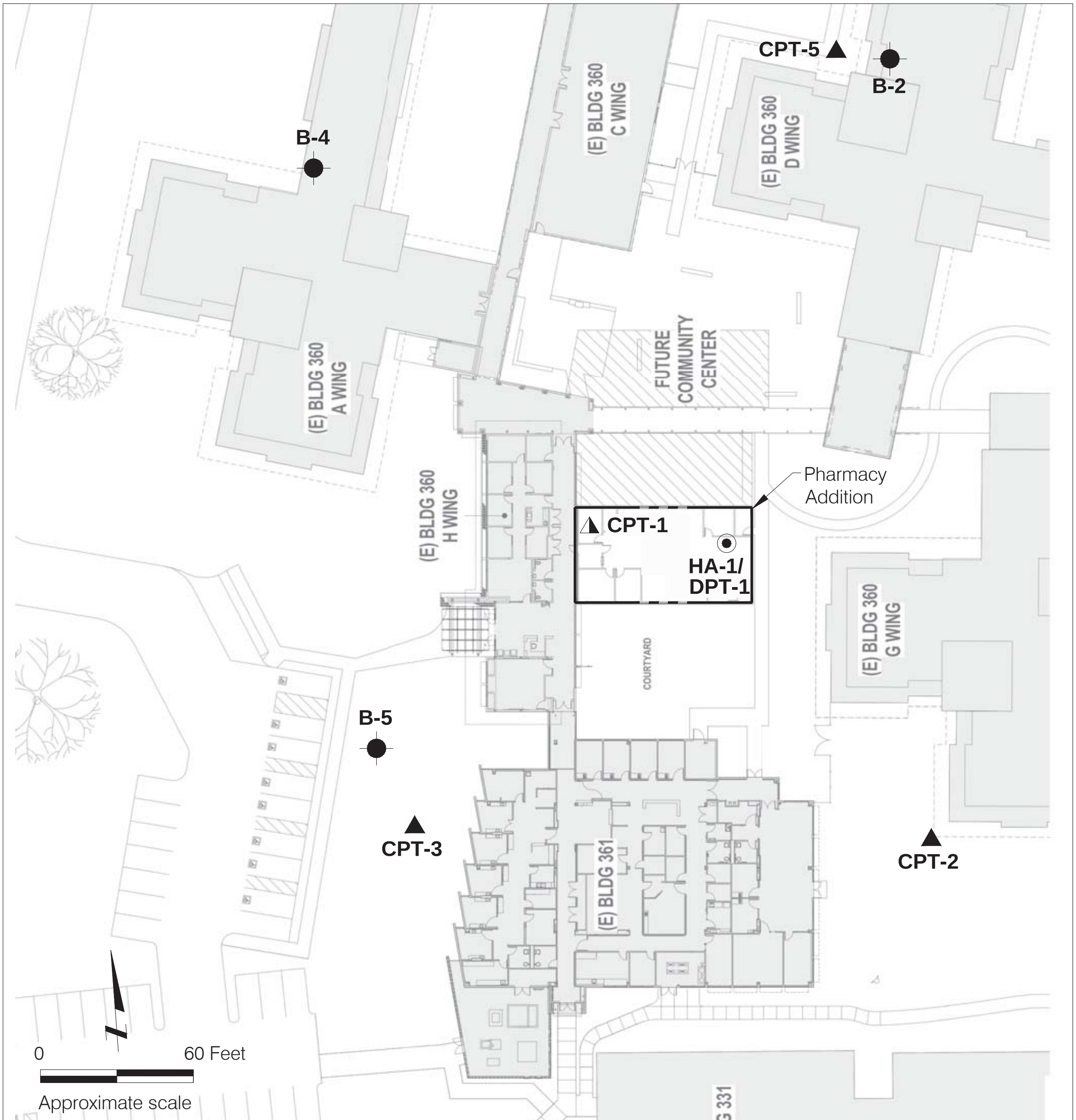

EXPLANATION

CPT-1 — Approximate location of cone penetration test by Langan Treadwell Rollo, November 2014

HA-1/ DPT-1 — Approximate location of hand auger/dynamic penetrometer test by Langan Treadwell Rollo, November 2014

B-5 — Approximate location of boring by Fugro, November 2005

CPT-3 — Approximate location of cone penetration test by Fugro, November 2005

— — — — Site boundary

Reference: Base map from a drawing titled "Site Plan Option D - New Pharmacy Building," by Polytech Associates, Inc., dated 04/11/14.

BUILDING 334 AND PHARMACY **VETERANS AFFAIRS MEDICAL CENTER** Menlo Park, California	**SITE PLAN**
LANGAN TREADWELL ROLLO	Date 10/15/14 | Project No. 750612403 | Figure 2

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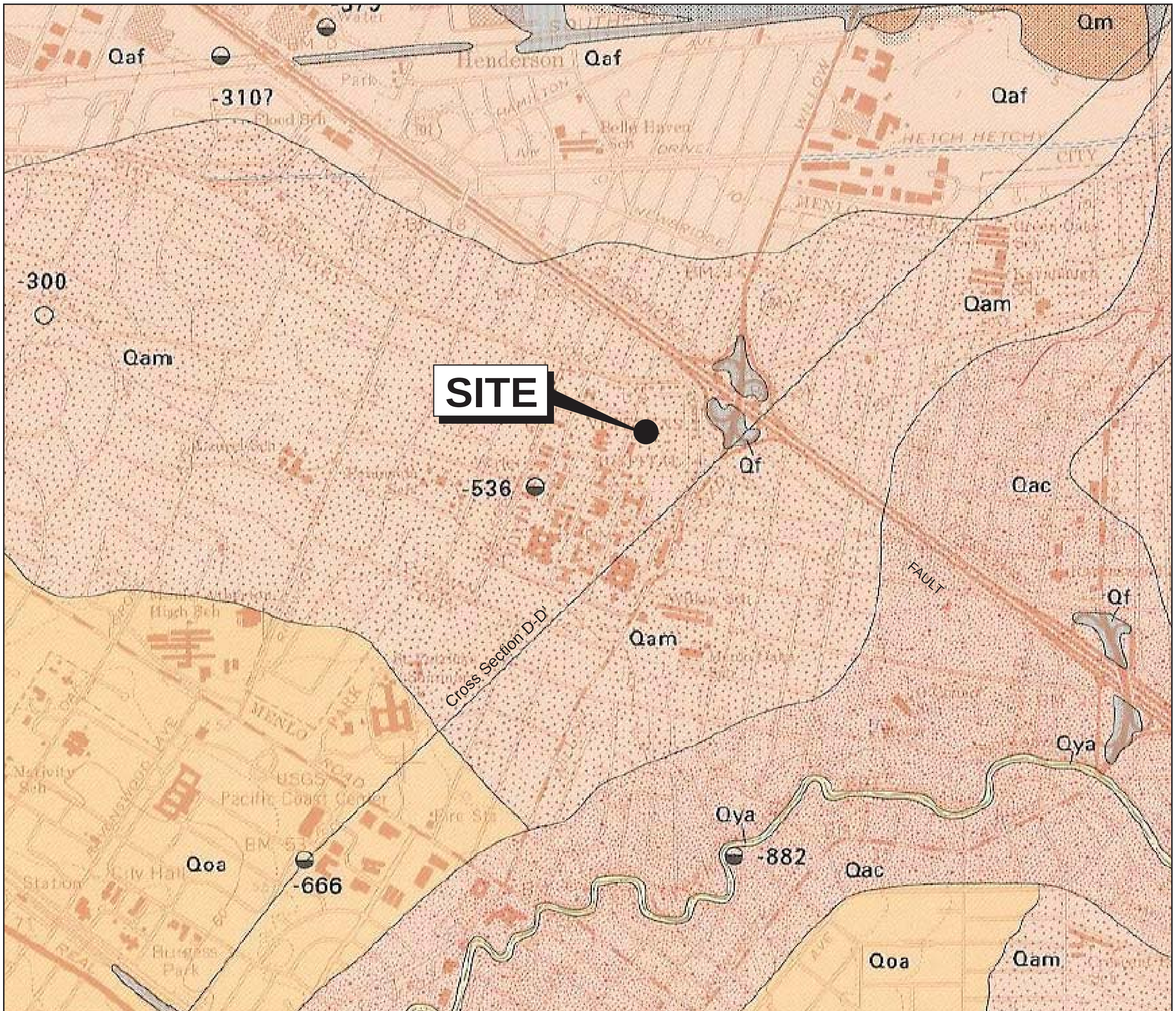


Base map: Geology of the Palo Alto and Part of the Redwood Point 7-1/2' Quadrangles, San Mateo and Santa Clara Counties, California.
by Earl H. Pampeyan, 1993.

EXPLANATION

Contact—Depositional or intrusive contact, dashed where approximately located, dotted where concealed

-666 **Borehole to basement (Franciscan Complex), with bedrock elevation (Mean Seal Level Datum)**

-300 **Borehole that did not reach basement (Franciscan Complex), with bottom of hole elevation (MSL Datum)**

Qya	Younger alluvium (Holocene)
Qf	Artifical Fill (Holocene)
Qaf	Fine-grained alluvium (Holocene)
Qam	Medium-grained alluvium (Holocene)
Qac	Coarse-grained alluvium (Holocene)
Qm	Bay mud (Holocene)
Qoa	Older alluvium (Pleistocene)

0 1/4 1/2 Mile

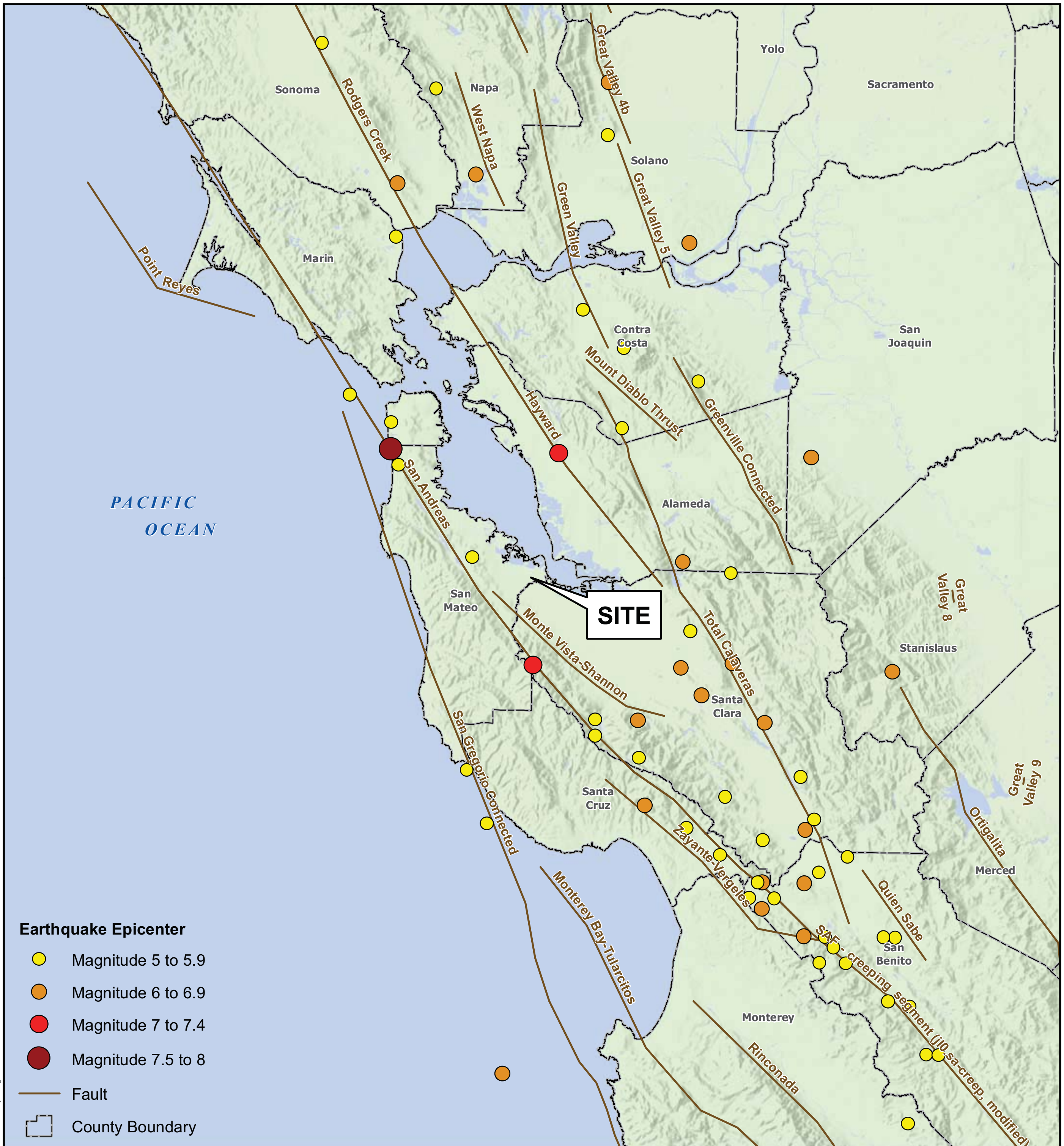
Approximate scale

BUILDING 334 AND PHARMACY
VETERANS AFFAIRS MEDICAL CENTER
Menlo Park, California

LANGAN TREADWELL ROLLO

REGIONAL GEOLOGIC MAP

Date 10/15/14 | Project No. 750612403 | Figure 3



\\langan.com\data\OA\data4\750612403\ArcGIS\ArcMap_Documents\Fault Map.mxd User: cyoung

Notes:

1. Quaternary fault data displayed are based on a generalized version of USGS Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 08/26/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

BUILDING 334 AND PHARMACY
VETERANS AFFAIRS MEDICAL CENTER
Menlo Park, California

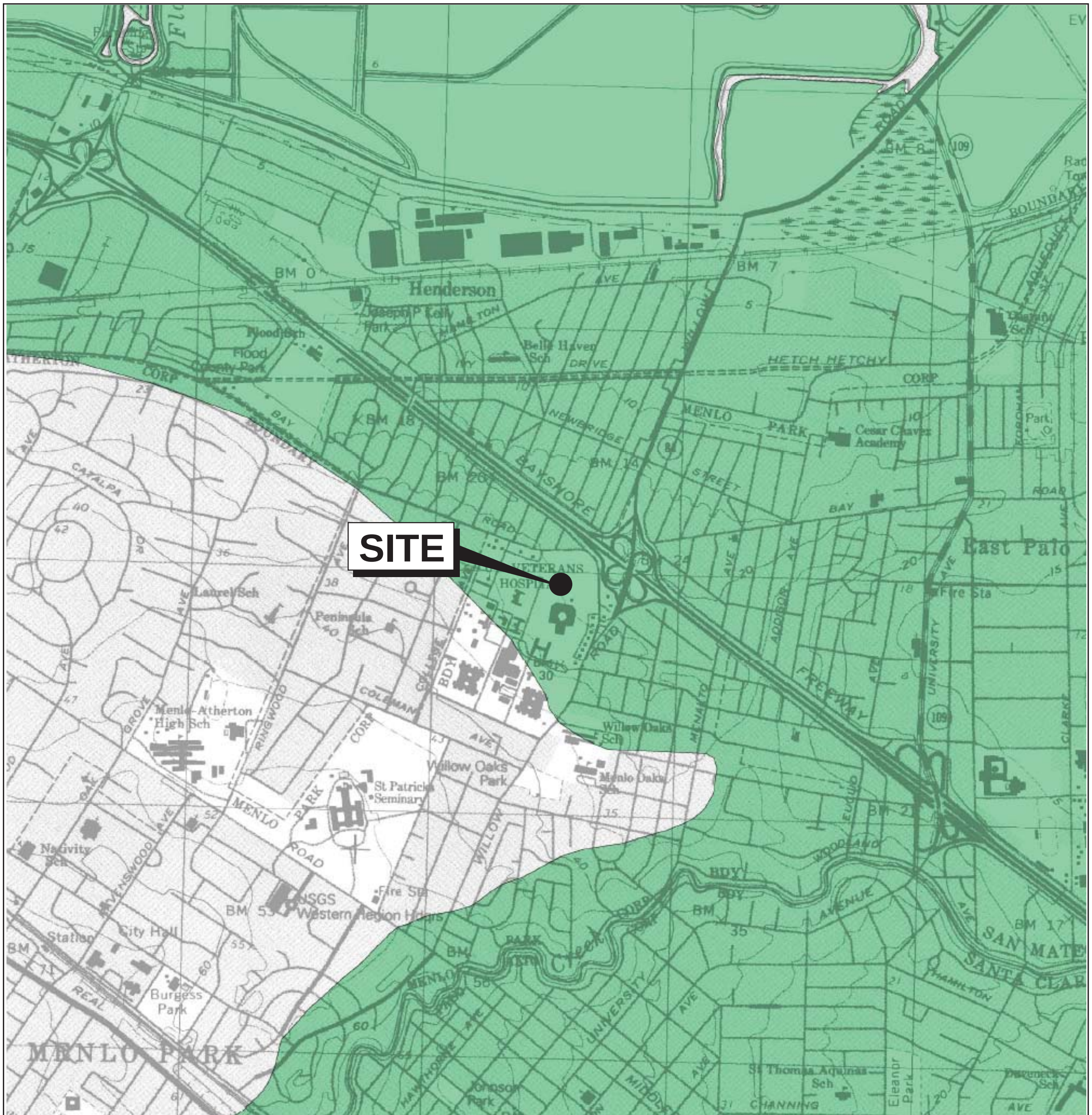
LANGAN TREADWELL ROLLO

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 10/9/2014	Project No. 750612403	Figure 4

I	**Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.** Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.
II	**Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.** As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.
III	**Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.** Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.
IV	**Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.** Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.
V	**Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.** Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.
VI	**Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.** Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.
VII	**Frightens everyone. General alarm, and everyone runs outdoors.** People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.
VIII	**General fright, and alarm approaches panic.** Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.
IX	**Panic is general.** Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.
X	**Panic is general.** Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.
XI	**Panic is general.** Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.
XII	**Panic is general.** Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

BUILDING 334 AND PHARMACY **VETERANS AFFAIRS MEDICAL CENTER** Menlo Park, California	**MODIFIED MERCALLI INTENSITY SCALE**		
LANGAN TREADWELL ROLLO	Date 10/09/14	Project No. 750612403	Figure 5



EXPLANATION

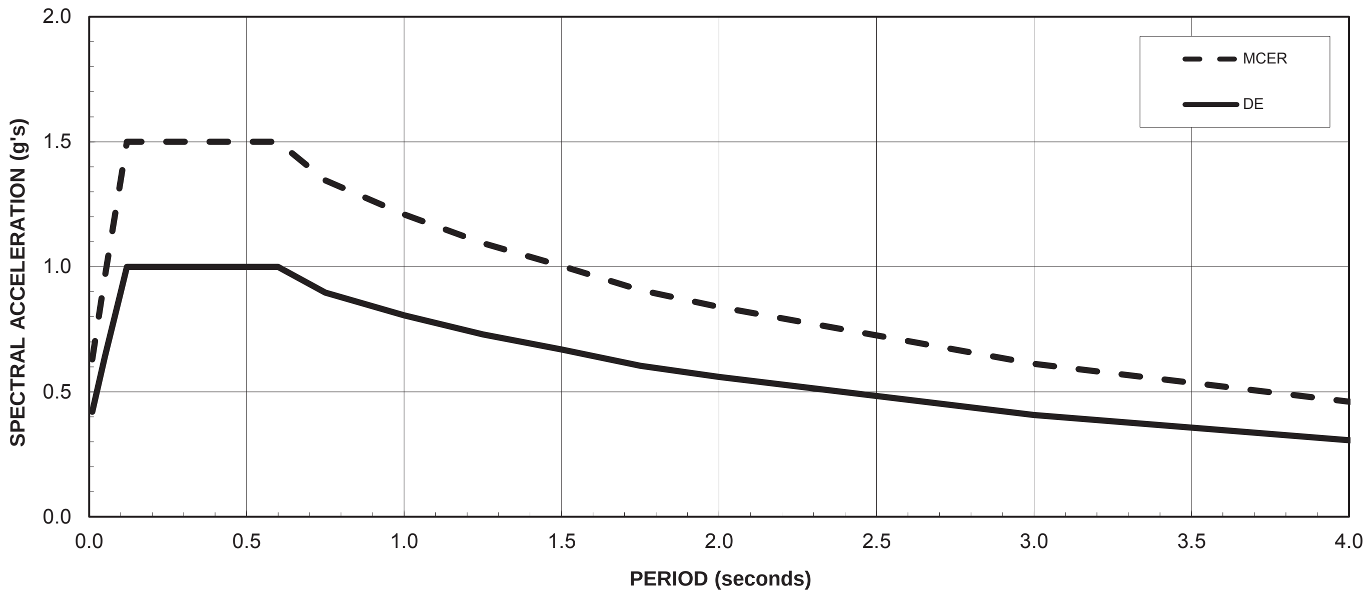
Zone of Liquefaction Potential

0 2000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones" Palo Alto Quadrangle, Released on October 18, 2006

BUILDING 334 AND PHARMACY **VETERANS AFFAIRS MEDICAL CENTER** Menlo Park, California	**REGIONAL SEISMIC HAZARD ZONES MAP**
LANGAN TREADWELL ROLLO	Date 10/15/14 | Project No. 750612403 | Figure 6



Damping Ratio = 5%

Note:
1. Estimated site avg. V_{s30} = 245 m/s

BUILDING 334 AND PHARMACY
VETERANS AFFAIRS MEDICAL CENTER
Menlo Park, California

RECOMMENDED HORIZONTAL MCE_R AND DE SPECTRA

Date 11/06/14	Project No. 750612403	Figure 7

LANGAN TREADWELL ROLLO

APPENDIX A

LOG OF BORING, DYNAMIC PENETRATION TEST, AND CONE PENETRATION TEST

PROJECT: **BUILDING 334 AND PHARMACY**
VETERANS AFFAIRS MEDICAL CENTER
Menlo Park, California

Log of Boring HA-1

Boring location:	See Site Plan, Figure 2
Date started:	10/3/14
Date finished:	10/3/14
Drilling method:	Hand Auger
Hammer weight/drop:	NA
Hammer type:	NA
Sampler:	Hand Auger (HA)
Logged by:	S. Magallon

Ground Surface Elevation: 23.5 feet[1]

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–3	HA (2)	X			CL	CLAY (CL) olive-brown, medium stiff to stiff, moist LL = 42, PI = 23, see Figure C-1 Corrosion Test, see Figure C-2					21.1	
3–5.5	HA (3–4)	X			CL	CLAY with SAND (CL) yellow-brown, soft to medium stiff, moist, very fine-grained sand						
5.5–6	HA (6)	X			CL	SANDY CLAY with SILT (CL) yellow-brown, medium stiff, moist, very fine- to fine-grained sand						
6–10	HA (8); HA (10)	X; X			CL	CLAY (CL) yellow-brown with red-orange mottling, soft to medium stiff, moist, trace fine-grained sand and occasional subrounded fine gravel yellow-brown with olive to olive-brown mottling, no gravel						

Boring terminated at a depth of 10 feet below ground surface.
Boring backfilled with cuttings.
Groundwater not encountered while hand augering.

[1] Elevations based on Mean Sea Level Datum.

LANGAN TREADWELL ROLLO

Project No.: 750612403
Figure: A-1

TEST GEOTECH LOG 750612403.GPJ TR.GDT 11/10/14

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	**Grain Size in Millimeters**
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push or Drive sampler

SAMPLER TYPE

C — Core barrel

CA — California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M — Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O — Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT — Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H — Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT — Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST — Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

BUILDING 334 AND PHARMACY
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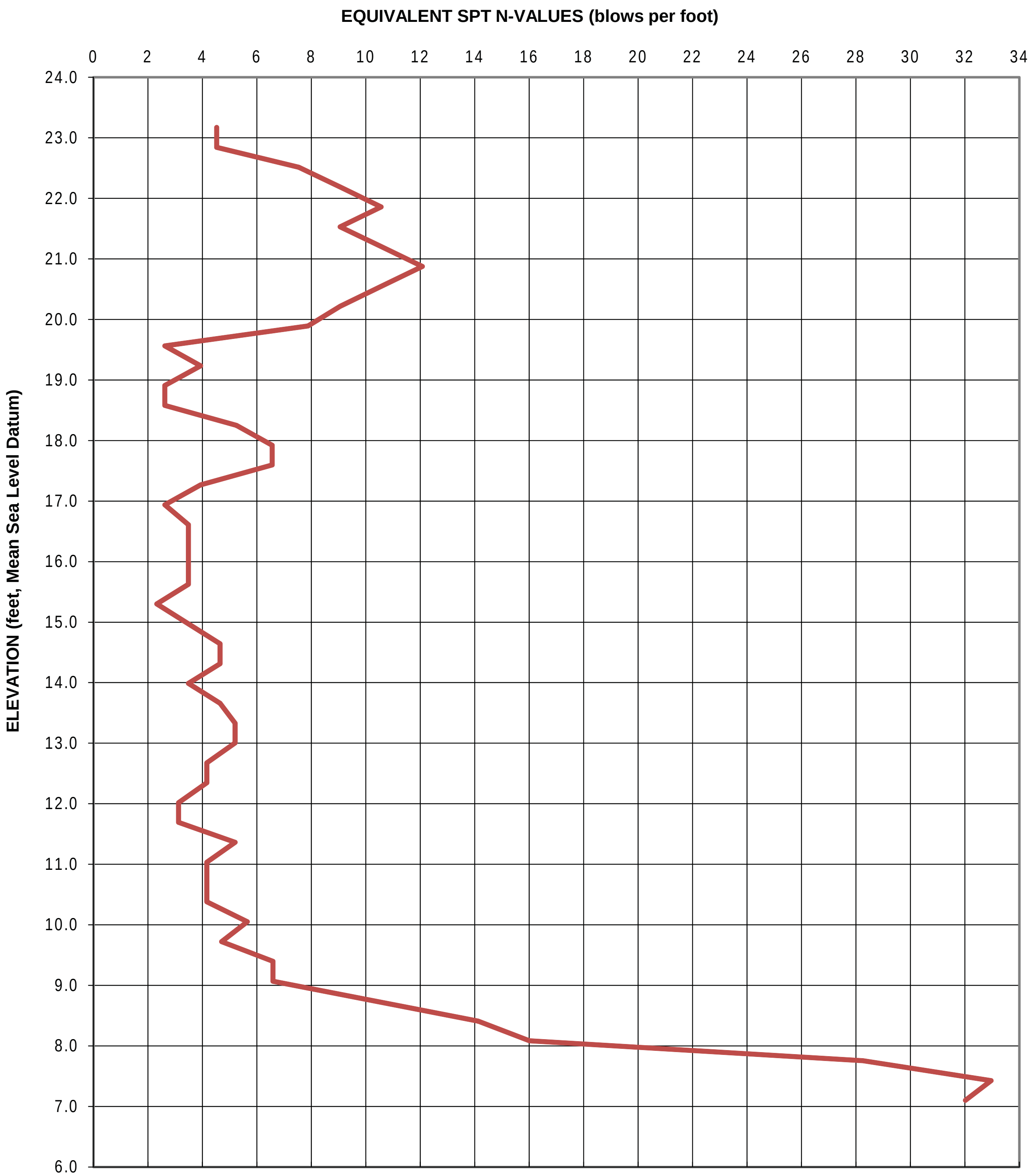
Menlo Park, California

LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 10/09/14 | Project No. 750612403 | Figure A-2



BUILDING 334 AND PHARMACY
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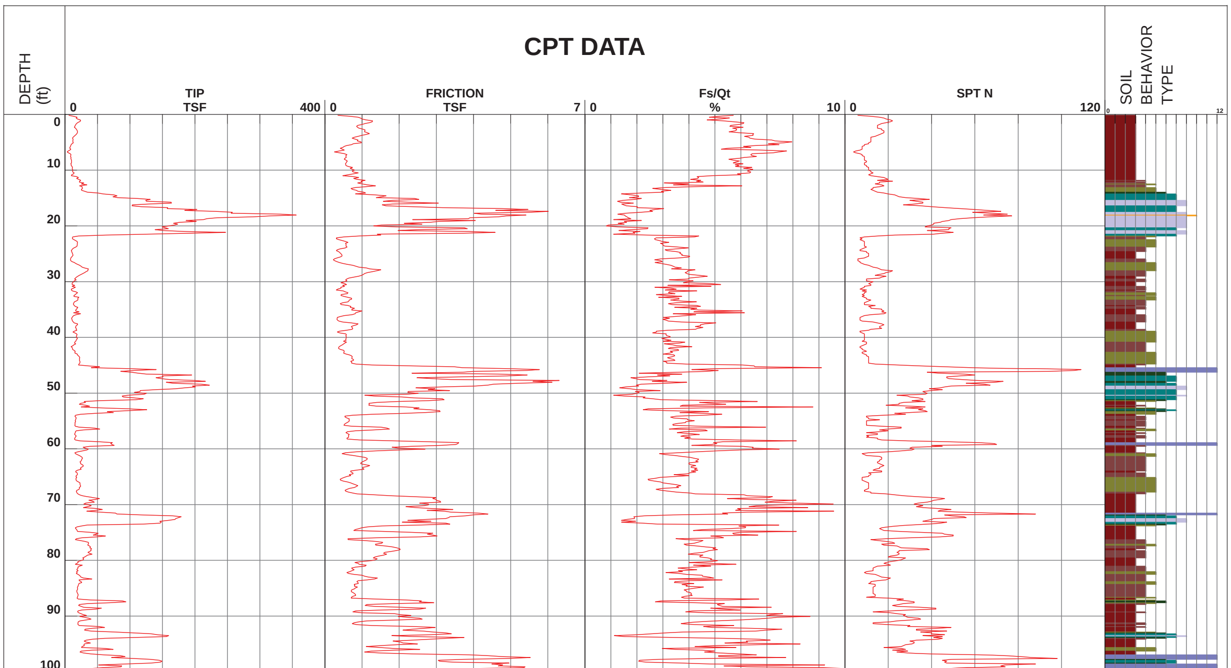
Menlo Park, California

LANGAN TREADWELL ROLLO

DYNAMIC PENETROMETER TEST
DPT-1

Date 10/19/14	Project No. 750612403	Figure A-3



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

*Soil behavior type and SPT based on data from UBC-1983

Terminated at 100.5 feet.
Groundwater encountered at 24 feet.
Date performed 10/03/14.
Ground surface elevation: 23.5 feet, Mean Sea Level Datum.

BUILDING 334 AND PHARMACY
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Menlo Park, California

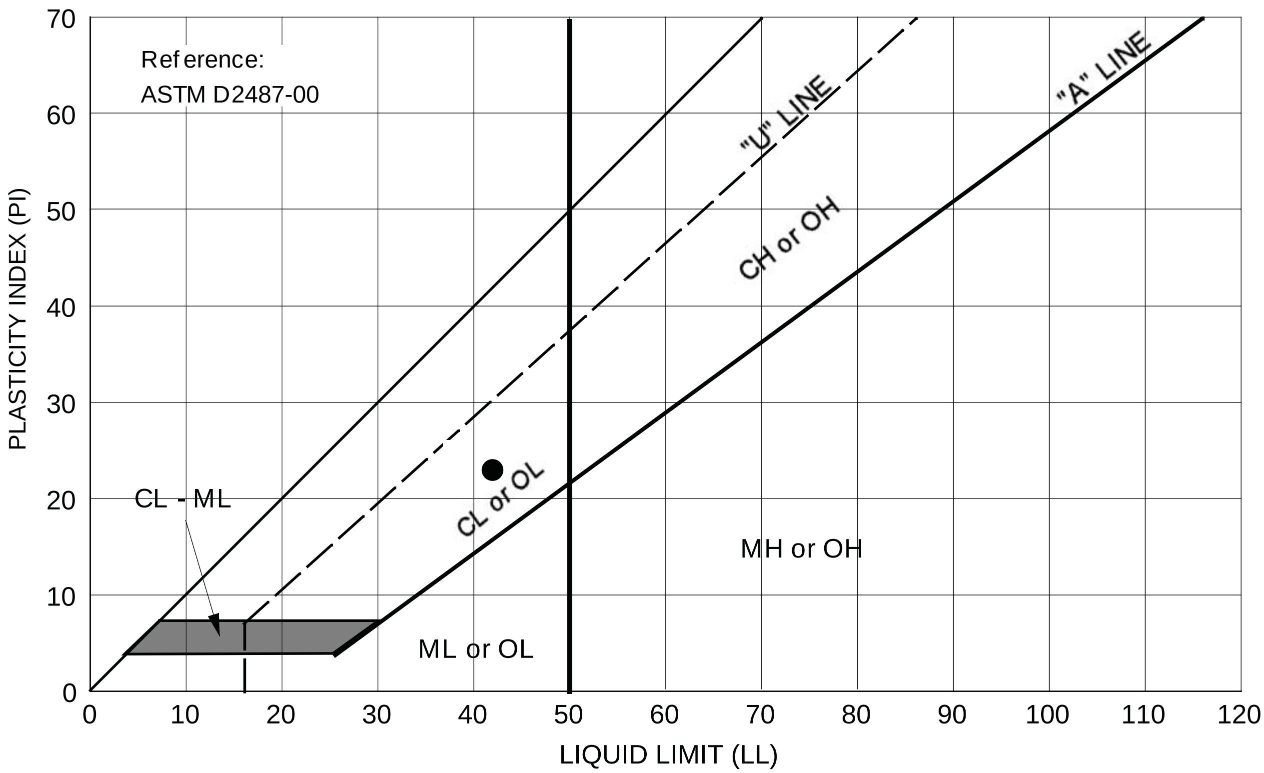
CONE PENETRATION TEST RESULTS
CPT-1

Date 10/16/14	Project No. 750612403	Figure A-4

LANGAN TREADWELL ROLLO

APPENDIX B

LABORATORY TEST RESULTS



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	HA-1 at 2 feet	CLAY (CL), olive-brown	21.1	42	23	--

BUILDING 334 AND PHARMACY
VETERANS AFFAIRS MEDICAL CENTER
Menlo Park, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 10/17/14 | Project No. 750612403 | Figure B-1

ETS

Environmental Technical Services

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: *entech@pacbell.net*

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COMPANY: Treadwell & Rollo, 501 14th Street, 3rd Floor, Oakland, CA 94612			ANALYST(S)	SUPERVISOR
ATTN: Elena Ayers		DATE of	D. Salinas	D. Jacobson
JOB SITE: VA Pharmacy, Menlo Park, California	DATE RECEIVED	COMPLETION	S. Santos	LAB DIRECTOR
JOB #: 750612403	4/18/1904	10/17/2014		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
06073-1	VA1-P/MP	HA-1-2 @ 3.0'	**6.06**	**1,247**	**[802]**	**120**	**126**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
06073-1	VA1-P/MP	HA-1-2 @ 3.0'				**+256.8**	
Method	Detection	Limits --->	---	0.1	0.1	-400 -> +800	0.1

** COMMENTS **

Resistivity is just below 1,250 ohm-cm, i.e., very low (assign 2-10 pts, depending on specs); soil reaction (i.e., pH) is mildly acidic assign Ø pts); sulfate level is low (SO4 @ <200 ppm, assign Ø pts); but chloride is mildly elevated (Cl @ >100 ppm, assign Ø-3 pts, depending on specs); soil is mildly reduced (assign Ø-3.5 pts, depending on specs). Standard CalTrans times to perforation are as follows: for 18 ga steel the time is <11 yrs, and for 12 ga it goes up to ≈23 yrs. For gray/ductile/mild steels and cast iron the calculated average pitting rate for this soil (according to Uhlig) is 0.24 mm/yr putting the 2 mm depth time at >8 yrs, and the 4 mm depth time would be <17 yrs. Sulfate level is low enough that there would be no measurable adverse impact on concrete, cement, mortar & grout; chloride level is slightly elevated so there could be a very minor adverse impact on steel reinforcement. Soil redox level is such that there could be some mild adverse impact on some construction materials (i.e., concrete or steel). This soil, in principle, could benefit from alkaline (i.e., lime or cement) treatment in that raising its pH to the 7.5-8.5 range would increase the CalTrans 18 ga time to perforation to ~27 yrs; and the pitting rate would decline to 0.10 mm/yr putting the 2 mm depth time at >20 yrs. But lime treatment only persists in protected locations (i.e., underneath slabs, bldgs, etc.); and while cement treatment is more permanent, there can be practical limitations. Otherwise, metal longevity can be improved by upgrading (e.g. increased gauge or more resistant steels, etc.). In fact, many times strength considerations will require use of heavier steel than used in the presented examples such that perforation or pitting times can be well beyond specified life span. Where this is not the case, cathodic protection of coated steel assets could be done as a potential solution for buried assets. Other alternatives include increased or specialized engineering fill, and/or use of plastic, fiberglass or concrete assets. Based on these results, standard concrete mixes should be fine in this soil. Total points for buried steel is in the range of 2-16.5 pts, depending on the specifications. Therefore, depending on specs, this soil could exceed 10 pts requiring remediation or replacement; or specific results could cause outright rejection (e.g. resistivity @ <1,500 ohm-cm, or @ <2,000 ohm-cm, etc.).

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

Figure B-2

APPENDIX C

LOGS OF BORINGS AND CONE PENETRATION TESTS BY OTHERS

Project No. 1150.003

Sheet 1 of 1

LOCATION: Middle of the Site

SURFACE EL: 22.0 ft +/- (rel. MSL datum)

SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
(25)	Lean CLAY (CL): stiff, dark brown, dry to moist, with a trace of fine-grained sand, silt and rootlets.	93	23				3.3 P	
16								
(15)	- grades brown below 3.5 ft.						2.3 P	
(14)	Clayey SAND (SC): loose, brown, moist, with fine-grained sand and a trace of gravel.						3.3 P	
	Poorly-graded GRAVEL with silt (GP-GM): medium dense, brown, moist to wet, fine to coarse, subangular to subrounded, some fine- to coarse-grained sand, trace silt							
20				6				
30								
	Lean CLAY with sand (CL): firm, brown, with fine-grained sand.							
(9)								
(18)	- grades stiff below 30 ft.	97	26	75	33	19	1.3 P	
	SAND (SP): medium dense, brown, with fine to coarse grained sand.							
12				3				
15								
27	Well-graded SAND (SW) with silt and gravel: medium dense, brown, fine- to coarse-grained, some silt, some fine, subangular to subrounded gravel			14				
32	- grades dense at 50 ft.							

ELEVATION, ft: 20, 15, 10, 5, 0, -5, -10, -15, -20, -25

DEPTH, ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

BORING DEPTH: 51.5 ft
DEPTH TO WATER: 14.0 ft
BACKFILL: Grout
COMPLETION DATE: August 9, 2005
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Manual Trip
RIG TYPE: B-53
DRILLED BY: Exploration Geoservices, Lauren
LOGGED BY: LA

LOG OF BORING NO. B2

Gero-psychiatric Replacement Facility
VA Hospital - Menlo Park, California

LOCATION: West Side of the Site

SURFACE EL: 23.0 ft +/- (rel. MSL datum)

SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
(36)	Lean CLAY (CL): very stiff, dark brown, dry to moist, with a trace of fine-grained sand.						+4.5 P	
37	- grades hard at 2 ft.							
(19)	- grades stiff and brown below 3.5 ft.	104	20				4.3 P	
(14)	Clayey SAND (SC): loose, brown, moist, fine-grained.						3.8 P	
27	Poorly-graded GRAVEL with silt (GP-GM): medium dense, brown, moist to wet, fine to coarse, subangular to subrounded, some fine- to coarse-grained sand, trace silt							
35	- grades dense at 20 ft.							
7	Sandy Lean CLAY (CL): firm, brown, fine-grained sand							
(23)	- grades stiff with a decrease in sand content at 30 ft						2.0 P	

BORING DEPTH: 31.5 ft
DEPTH TO WATER: 15.0 ft
BACKFILL: Grout
COMPLETION DATE: August 9, 2005
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Manual Trip
RIG TYPE: B-53
DRILLED BY: Exploration Geoservices, Lauren
LOGGED BY: LA

LOG OF BORING NO. B4

Gero-psychiatric Replacement Facility
VA Hospital - Menlo Park, California

Project No. 1150.003

FUGRO

Sheet 1 of 1

LOCATION: Southwest Side of Site

SURFACE EL: 24.0 ft +/- (rel. MSL datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLER TYPE	SAMPLER BLOW COUNT/ PRESSURE, psi	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
					(24)	Lean CLAY (CL): stiff, dark brown, dry to moist, with a trace of fine-grained sand and rootlets							R Value = 11
					14								
20	5				(17)	- grades brown							
15	10												
10	15												
5	20												
0	25												
-5	30												
-10	35												
-15	40												
-20	45												
-25	50												

BORING DEPTH: 5.0 ft
DEPTH TO WATER: Not Encountered
BACKFILL: Grout
COMPLETION DATE: August 9, 2005
NOTES: 1. Terms and symbols defined on Plate A-1.

DRILLING METHOD: 8-in. dia. Hollow Stem Auger
HAMMER TYPE: Manual Trip
RIG TYPE: B-53
DRILLED BY: Exploration Geoservices, Lauren
LOGGED BY: LA

LOG OF BORING NO. B5
Gero-psychiatric Replacement Facility
VA Hospital - Menlo Park, California

FUGRO

MAJOR DIVISIONS			GROUP NAMES	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded Gravel
			GP	Poorly Graded Gravel
		Gravels with more than 12% fines	GM	Silty Gravel
			GC	Clayey Gravel
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded Sand
			SP	Poorly Graded Sand
		Sands with more than 12% fines	SM	Silty Sand
			SC	Clayey Sand
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	Silt
			CL	Lean Clay
			OL	Organic Silt
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic Silt
			CH	Fat Clay
			OH	Organic Clay
HIGHLY ORGANIC SOILS			PT	Peat or Highly Organic Soils
			FILL	Debris or Mixed Fill
			AC	Asphalt Concrete Pavement with Aggregate Base

GENERAL NOTES

Classification of Soils per ASTM D2487 or D2488

Geologic Formation noted in bold font at the top of interpreted interval

Sloped line in break column indicates transitional boundary

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

SAMPLER DRIVING RESISTANCE

Number of blows with 140 lb. hammer, falling 30-in. to drive sampler 1-ft. after seating sampler 6-in.; for example,

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
50/7"	50 blows drove sampler 7" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval (Ref=Refusal)

STRENGTH TEST METHOD

U = Unconfined Compression
Q = Unconsolidated Undrained Triaxial
T = Torvane
P = Pocket Penetrometer
M = Miniature Vane
F = Field Vane

OTHER TESTS

k = Permeability
Consol = Consolidation
Gs = Specific Gravity
MA = Particle Size Analysis
EI = Expansion Index
OVM = Organic Vapor Measurement

WATER LEVEL SYMBOLS

Initial or perched water level
Final ground water level
Seepages encountered

SAMPLER TYPE

1 SPT, 2 MC, 3 CA, 4 SH, 5 BB, 6 HA, 7 LS, 8 PS, 9 VS, 10 NR, 11 RC

Samplers and sampler dimensions (unless otherwise noted in report text) are as follows:

1 SPT Sampler, driven 1 3/8" ID, 2" OD
2 MOD CA Liner Sampler 2 3/8" ID, 3" OD
3 CA Liner Sampler 1 7/8" ID, 2.5" OD
4 Thin-walled Tube, pushed 2 7/8" ID, 3" OD
5 Bulk Bag Sample (from cuttings)
6 Hand Auger Sample
7 Lexan Sample
8 Pitcher Sample
9 Vibracore Sample
10 No Sample Recovered
11 Rock Core

SOIL STRUCTURE

Fissured: Containing shrinkage or relief cracks, often filled with fine sand or silt, usually more or less vertical.

Pocket: Inclusion of material of different texture that is smaller than the diameter of the sample.

Parting: Inclusion less than 1/8 inch thick extending through the sample.

Seam: Inclusion 1/8 inch to 3 inches thick extending through the sample.

Layer: Inclusion greater than 3 inches thick extending through the sample.

Laminated: Soil sample composed of alternating partings or seams of different soil types.

Interlayered: Soil sample composed of alternating layers of different soil type.

Intermixed: Soil sample composed of pockets of different soil type, and layered or laminated structure is not evident.

CONSISTENCY

Clays	Blows/Foot SPT	Undrained Shear Strength (ksf)
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.5
Firm	5 - 8	0.5 - 1
Stiff	9 - 16	1 - 2
Very Stiff	17 - 32	2 - 4
Hard	Over 32	Over 4

RELATIVE DENSITY

Sands and Gravels	Blows/Foot SPT
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	Over 50

INCREASING VISUAL MOISTURE CONTENT

Dry
Moist
Wet

Information on each boring log is a compilation of subsurface conditions and soil or rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the time and places indicated, and can vary with time, geologic condition, or construction activity.

TERMS AND SYMBOLS USED ON BORING LOGS

PLATE A-1

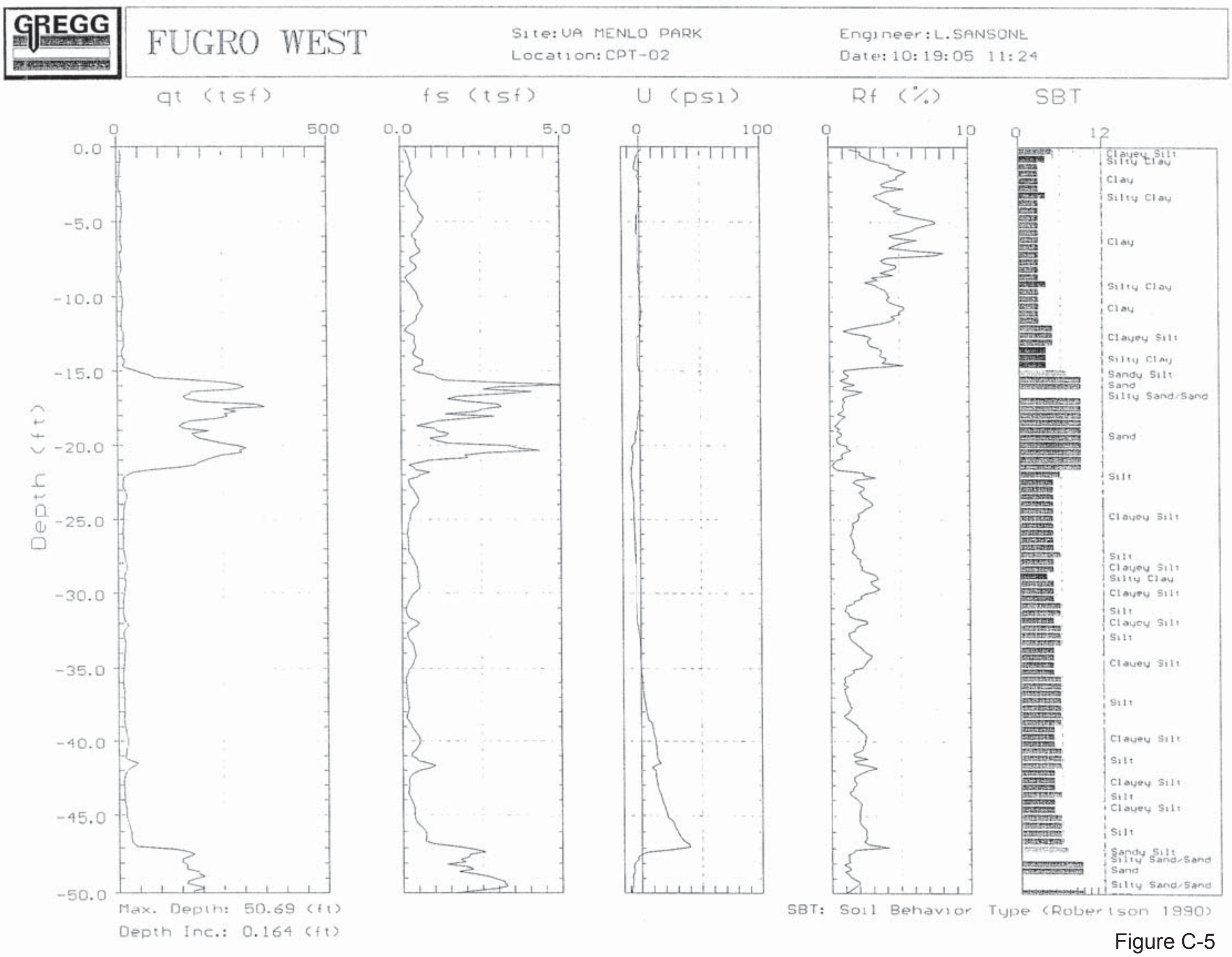

GREGG
FUGRO WEST
Site: VA MENLO PARK
Location: CPT-02
Engineer: L.SANSONE
Date: 10:19:05 11:24
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 50.69 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

Figure C-5

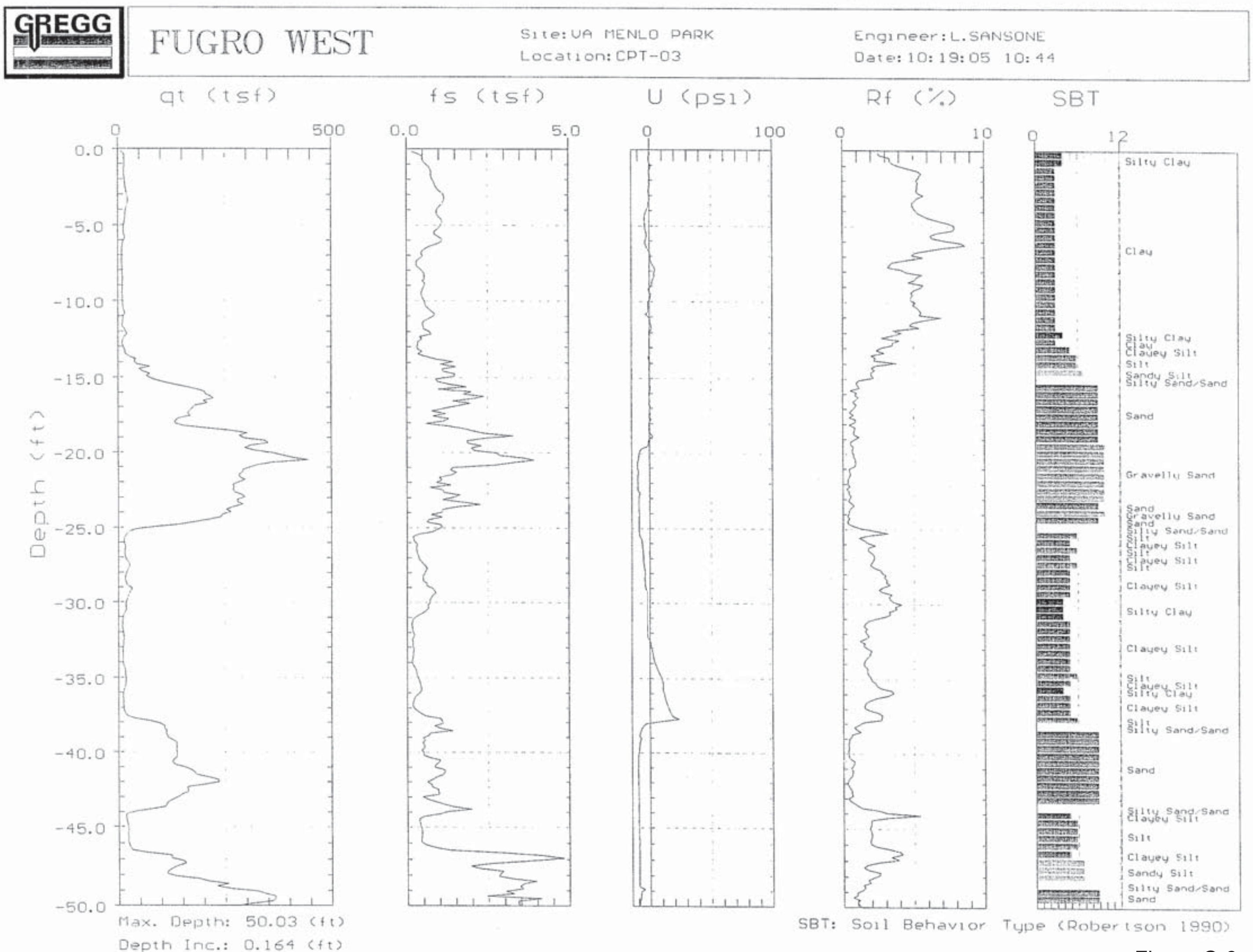

GREGG
FUGRO WEST
Site: VA MENLO PARK
Location: CPT-03
Engineer: L.SANSONE
Date: 10:19:05 10:44
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)
Silty Clay
Clay
Silty Clay
Clayey Silt
Silt
Sandy Silt
Silty Sand/Sand
Sand
Gravelly Sand
Clayey Silt
Silty Clay
SBT: Soil Behavior Type (Robertson 1990)

Figure C-6

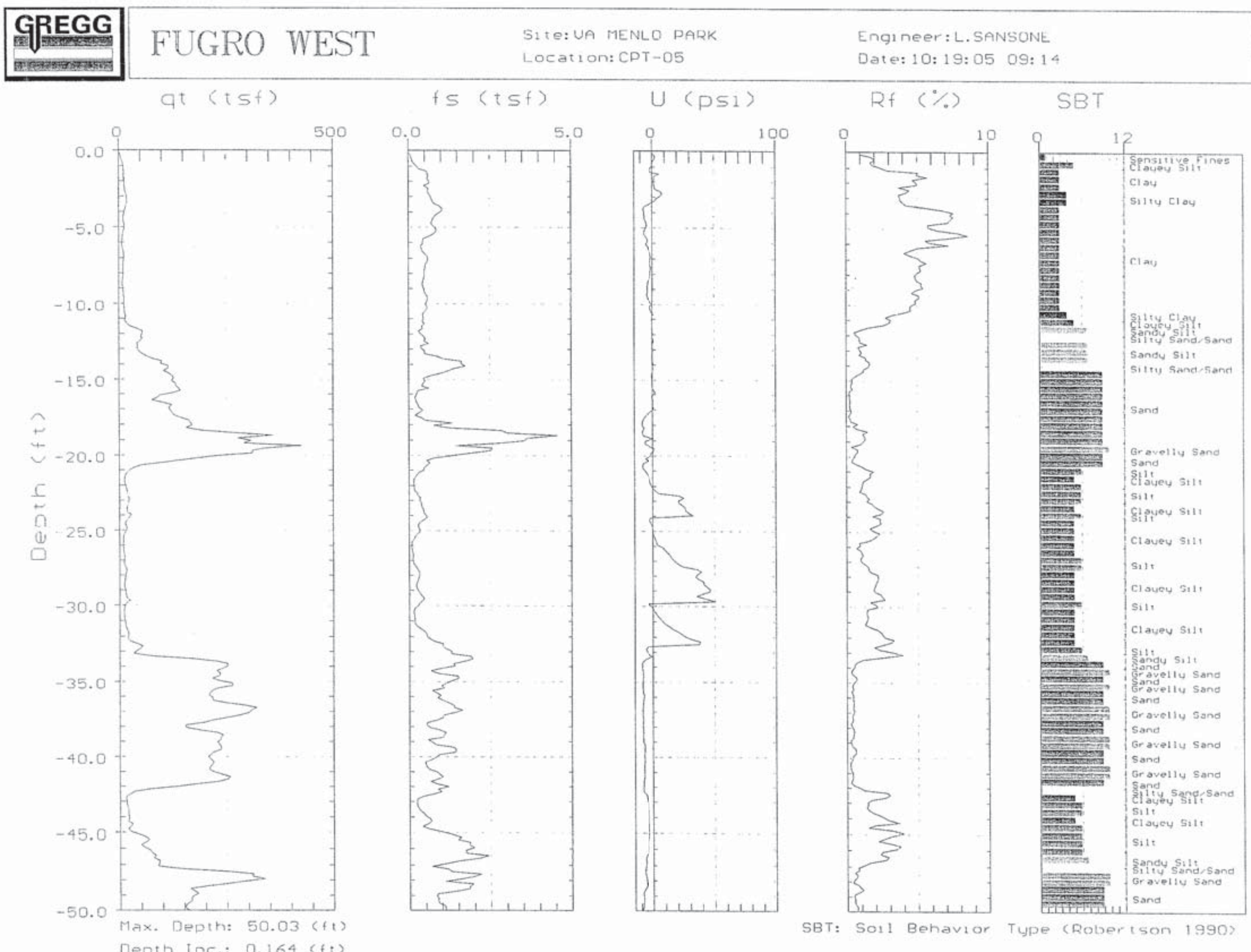

GREGG
FUGRO WEST
Site: VA MENLO PARK
Location: CPT-05
Engineer: L.SANSONE
Date: 10:19:05 09:14
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Depth (ft)
Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
Sensitive Fines
Clayey Silt
Clay
Silty Clay
Sandy Silt
Silty Sand/Sand
Sand
Gravelly Sand
Silt

Figure C-7

APPENDIX D

SITE-SPECIFIC RESPONSE SPECTRA

APPENDIX D
SITE-SPECIFIC RESPONSE SPECTRA

This appendix presents the details of our estimation of the level of ground shaking at the site of the new pharmacy addition at the Veterans Affairs Medical Center in Menlo Park, California during future earthquakes. We developed site-specific response spectra for two levels of shaking corresponding to the Risk Targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE) consistent with the 2012 International Building Code (IBC)/ASCE 7-10.

For the development of MCE_R and DE in accordance with the 2012 IBC/ASCE 7-10 criteria, we performed a Probabilistic Seismic Hazard Analysis (PSHA) and deterministic analysis to develop smooth, site-specific horizontal spectra for two levels of shaking, namely:

- MCE_R, which corresponds to the lesser of the risk targeted probabilistic spectrum in the maximum direction having 2 percent probability of exceedance in 50 years or the 84^{th} percentile in the maximum direction of the deterministic event on the governing fault(s);
- DE, which corresponds to 2/3 of the MCE_R.

These procedures comply with the Department of Veterans Affairs Seismic Design Requirements H-18-8 dated August 2013. Details regarding our study are presented in the following sections of this appendix.

D1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data;
- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake;
- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

As part of the development of the MCE_R spectrum, we performed a PSHA to develop a site-specific response spectrum for a 2 percent probability of exceedance in 50 years. The ground surface spectrum was developed using the computer code EZFRISK 7.62 (Risk Engineering 2011). The approach used in EZFRISK is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using Next Generation Attenuation (NGA) relationships that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault.

D1.1 Probabilistic Model

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i \nu_i \iint P[Z > z \mid m, r] f_{M_i}(m) f_{R_i|M_i}(r; m) dr dm$$

where:

ν_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i

P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z

$f_{Mi}(m)$ and $f_{Ri|Mi}(r;m)$ = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

D1.2 Source Modeling and Characterization

The segmentation of faults, mean characteristic magnitudes, and recurrence rates were modeled using the data presented in the WGCEP (2008) and Cao et al. (2003) reports. We also included the combination of fault segments and their associated magnitudes and recurrence rates as described in the WGCEP (2008) in our seismic hazard model. Table D-1 presents the distance and direction from the site to the fault, mean characteristic magnitude, mean slip rate, and fault length for individual fault segments within 100 km from the site. We used the California fault database identified as "USGS08" in EZFRISK 7.62. We understand EZFRISK obtained this database directly from USGS and models the faults with multiple segments. Each segment is characterized with multiple magnitudes, occurrence or slip rates and weights. This approach takes into account the epistemic uncertainty associated with the various seismic sources in our model.

TABLE D-1
Source Zone Parameters

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Fault Length (km)
Monte Vista-Shannon	8	Southwest	6.50	0.4	45
N. San Andreas - Peninsula	11	Southwest	7.23	17	85
N. San Andreas (1906 event)	11	Southwest	8.05	22	472
Total Hayward	20	Northeast	7.00	9	87
Total Hayward-Rodgers Creek	20	Northeast	7.33	9	150
San Gregorio Connected	26	West	7.50	5.5	176
Total Calaveras	28	East	7.03	12	123
N. San Andreas - Santa Cruz	35	Southeast	7.12	17	62
Mount Diablo Thrust	41	Northeast	6.70	2	25
Zayante-Vergeles	45	Southeast	7.00	0.1	58
Greenville Connected	49	Northeast	7.00	2	50
Green Valley Connected	50	Northeast	6.80	4.7	56
N. San Andreas - North Coast	51	Northwest	7.51	24	189
Monterey Bay-Tularcitos	61	South	7.30	0.5	83
Great Valley 7	63	East	6.90	1.5	45
Great Valley 5, Pittsburg Kirby Hills	67	Northeast	6.70	1	32
Rodgers Creek	73	North	7.07	9	62
West Napa	78	North	6.70	1	30
Point Reyes	80	Northwest	6.90	0.3	47
Ortigalita	81	East	7.10	1	70
Great Valley 8	89	East	6.80	1.5	41
Quien Sabe	93	Southeast	6.60	1	23
Great Valley 4b, Gordon Valley	93	North	6.80	1.3	28
SAF - creeping segment (jl0.sa-creep, modified)	95	Southeast	6.70	34	125
Rinconada	95	Southeast	7.50	1	191

D1.3 Attenuation Relationships

Subsurface information from our geotechnical investigation indicates the site is generally underlain by soft to hard clay with variable sand and silt content with thin layers of medium dense to very dense sand with variable gravel and fines content to the maximum depth explored of 100 feet below the ground surface. Using subsurface data from our geotechnical investigation and the measurements of shear wave velocity at the site, we estimate the average shear wave velocity of the upper 100 feet to be approximately 803 feet per second (245 meters per second). Based on the subsurface conditions, the site is classified as a stiff soil site, site class D.

Pacific Earthquake Engineering Research Center (PEER) embarked on the NGA project to update the previously developed attenuation relationships which were mostly published in 1997. We used the relationships by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). These attenuation relationships include the average shear wave velocity in the upper 100 feet. Furthermore, these relationships were developed using different earthquake databases, therefore, the average of the relationships was used to develop the recommended spectra.

The NGA relationships were developed for the orientation-independent geometric mean of the data. Geometric mean is defined as the square root of the product of the two recorded components.

D1.4 PSHA RESULTS

Figure D-1 presents the results of the PSHA for 2 percent probability of exceedance in 50 years. The average of the four attenuation relationships is also shown on these figures. The results presented on these figure are for the geometric mean as discussed in Section D1.3.

ASCE 7-10 specifies the development of MCE site-specific response spectra in the maximum direction. Shahi and Baker (2013) provide scaling factors that modify the geometric mean spectra to provide spectral values for the maximum response (maximum direction). We used the scaling factors presented on Figure 3.1 of Shahi and Baker (2013) ratios $Sa_{RotD100}$/ $Sa_{GMRotI50}$ to modify the average of the PSHA results. The maximum direction spectrum is also shown on Figure D-1.

Figure D-2 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, it can be seen that the San Andreas Fault dominates the hazard at the site at different periods of interest.

D2.0 DETERMINISTIC ANALYSIS

We performed a deterministic analysis to develop the MCE_R spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. The governing earthquake scenario was defined as an earthquake on the San Andreas Fault having a Moment Magnitude of about 8.0 occurring at a distance of about 10.7 km. This is consistent with the deaggregation results discussed in Section D1.4.

The same attenuation relationships as discussed in Section D1.3 were used in our deterministic analysis. The 84^{th} percentile deterministic results of the four attenuation relationships as well as the averages of the four relationships are presented on Figure D-3 for the San Andreas Fault. We also developed the 84^{th} percentile deterministic spectrum in the maximum direction which is shown on Figure D-3 using the same scaling factors by Shahi and Baker (2013) as discussed in Section D1.4.

D3.0 RECOMMENDED SPECTRA

The MCE_R as defined in ASCE 7-10 is the lesser of the maximum direction PSHA spectrum having a two percent probability of exceedance in 50 years (2,475-year return period) or the maximum direction 84^{th} percentile deterministic spectrum of the governing earthquake scenario. Furthermore, the MCE_R spectrum is defined as a risk targeted response spectrum, which corresponds to a targeted collapse probability of one percent in 50 years. According to the USGS website the risk coefficients for the PSHA spectra for short and long periods are

1.035 and 0.980, respectively. The corresponding risk coefficients for the deterministic spectrum are 1.0. We used these risk coefficients to develop the risk targeted PSHA and deterministic response spectra.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-10 to develop the site-specific spectra for MCE_R. Chapter 21 of ASCE 7-10 requires the following checks:

1. the deterministic spectrum used to develop the MCE_R not fall below the Deterministic Lower Limit spectrum as shown on Figure 21.2-1 of ASCE 7-10;

2. the DE spectrum shall not fall below 80 percent of general design spectrum (Section 21.3 of Chapter 21 ASCE 7-10).

D3.1 Recommended Horizontal Spectra

Figure D-4 and Table D-2 present a comparison of the site-specific spectra for the Risk Targeted PSHA 2,475 year return period and the 84^{th} percentile deterministic (both in maximum direction), and the Deterministic Lower Limit spectra for Site Class D per ASCE 7-10. In this case, for periods greater than 0.01 seconds and less than 0.75 seconds the Deterministic Lower Limit spectra for Site Class D per ASCE 7-10 is larger than the 84^{th} percentile deterministic (maximum direction). Therefore, for periods within this range we recommend that the lesser spectrum of the deterministic lower limit and 2,475 year return period be used to develop the MCE_R spectrum and for periods outside this range we recommend that the lesser spectrum of the 84^{th} percentile deterministic and 2,475 year return period be used to develop the MCE_R spectrum. The recommended MCE_R spectrum is presented on Figure D-4 and in Table D-2.

TABLE D-2
Comparison of Site-specific and Code Spectra for Development of MCE_R Spectrum per ASCE 7-10
S_a (g) for 5 percent damping

Period (seconds)	Risk Targeted Maximum Direction PSHA – 2,475 Year Return Period	Risk Targeted Maximum Direction Deterministic – 84th Percentile	ASCE 7-10 Deterministic Lower Limit Site Class D	Recommended MCE_R
0.01	0.804	0.630	0.600	0.630
0.05	0.960	0.725	0.975	0.960
0.10	1.347	0.961	1.350	1.347
0.12	1.568	1.052	1.500	1.500
0.20	1.781	1.277	1.500	1.500
0.30	1.864	1.407	1.500	1.500
0.40	1.827	1.423	1.500	1.500
0.50	1.771	1.422	1.500	1.500
0.60	1.683	1.391	1.500	1.500
0.75	1.578	1.346	1.200	1.346
1.00	1.450	1.209	0.900	1.209
1.50	1.024	0.907	0.600	1.003
2.00	0.936	0.839	0.450	0.839
3.00	0.652	0.612	0.300	0.612
4.00	0.485	0.460	0.230	0.460

Table D-3 presents the development of recommended DE spectrum following the procedures outlined in Chapter 21 of ASCE 7-10. The DE is defined as 2/3 of the MCE_R; however, the recommended DE may not be below 80 percent of the general spectrum at any period (ASCE 7-10 Section 21.3). Figure D-4 and Table D-3 presents a comparison of 2/3 of the MCE_R spectrum and 80 percent of the general spectrum for Site Class D. As shown in Table D-3 and Figure D-4, 80 percent of the general spectrum is lower than 2/3 of the MCE_R spectrum for all periods. Therefore, we recommend that 2/3 of the MCE_R spectrum be used to develop the DE spectrum for all periods.

TABLE D-3
Comparison of Site-specific and Code Spectra for Development of DE Spectrum per ASCE 7-10
S_a (g) for 5 percent damping

Period (seconds)	Recommended MCE_R	2/3 times MCE_R	80% of General Design Spectrum	Recommended DE
0.01	0.630	0.420	0.320	0.420
0.05	0.960	0.640	0.508	0.640
0.10	1.347	0.898	0.696	0.898
0.12	1.500	1.000	0.771	1.000
0.20	1.500	1.000	0.800	1.000
0.30	1.500	1.000	0.800	1.000
0.40	1.500	1.000	0.800	1.000
0.50	1.500	1.000	0.800	1.000
0.60	1.500	1.000	0.800	1.000
0.75	1.346	0.897	0.682	0.897
1.00	1.209	0.806	0.511	0.806
1.50	1.003	0.669	0.341	0.669
2.00	0.839	0.559	0.256	0.559
3.00	0.612	0.408	0.170	0.408
4.00	0.460	0.307	0.128	0.307

The recommended spectra for the MCE_R and DE are presented on Figure D-5. Also digitized values of the recommended spectra are presented in Table D-4 for damping ratios of 5 percent. For comparison, the recommended MCE_R and DE and the code mapped MCE_R and DE spectra are also presented on Figure D-5.

TABLE D-4
Recommended MCE_R and DE Spectra for Damping Ratio of 5 percent
Spectral Acceleration (g)

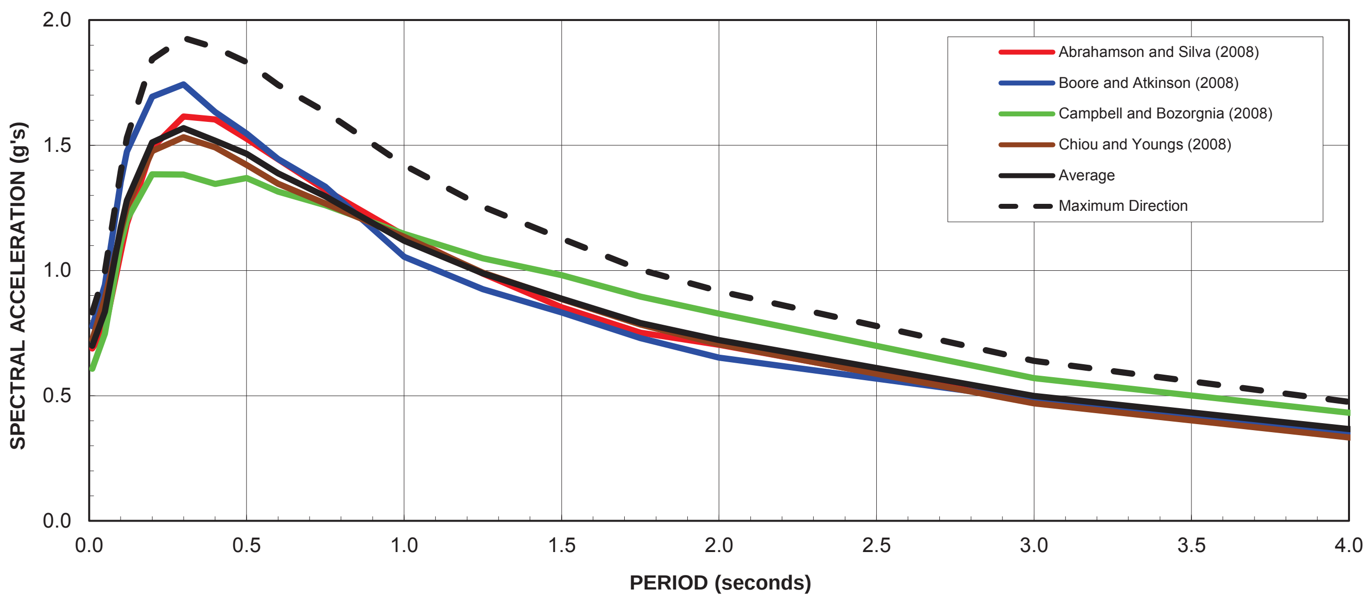
Period (seconds)	Recommended MCE_R	Recommended DE
0.01	0.630	0.420
0.05	0.960	0.640
0.10	1.347	0.898
0.12	1.500	1.000
0.20	1.500	1.000
0.30	1.500	1.000
0.40	1.500	1.000
0.50	1.500	1.000
0.60	1.500	1.000
0.75	1.346	0.897
1.00	1.209	0.806
1.50	1.003	0.669
2.00	0.839	0.559
3.00	0.612	0.408
4.00	0.460	0.307

Because the site-specific procedure was used to determine the recommended MCE_R and DE response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS}, and S_{D1} per Section 21.4 of ASCE 7-10 should be used as shown in Table D-5.

TABLE D-5
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.500
S_{M1}	1.678*
S_{DS}	1.000
S_{D1}	1.118*

*The two second spectral values govern the determination of S_{M1} and S_{D1}.



Damping Ratio = 5%

Note:
1. Estimated site avg. V_{s30} = 245 m/s
2. Maximum direction based on Shahi and Baker (2013)

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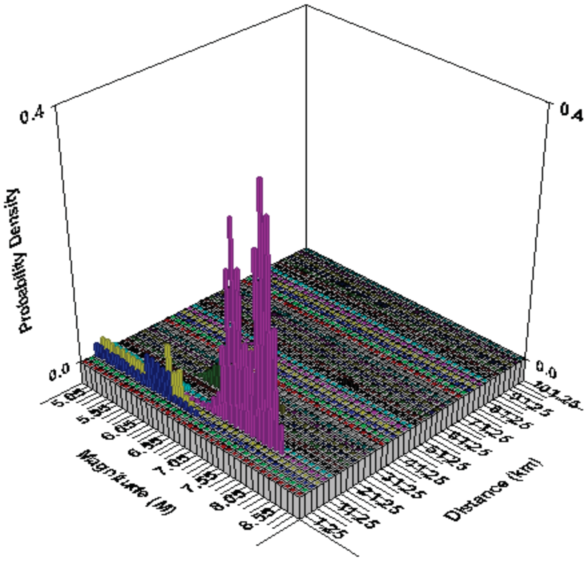
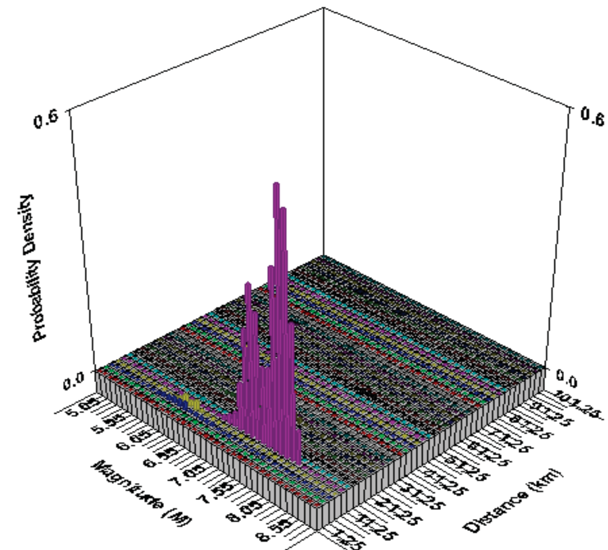
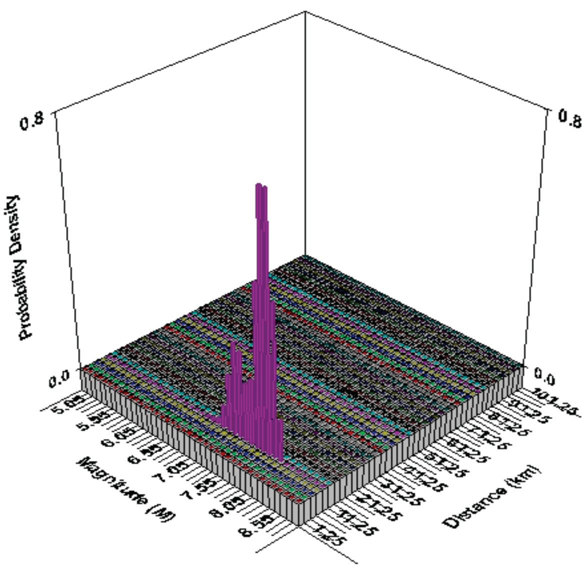
RESULTS OF PSHA FOR 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS - MAXIMUM DIRECTION		
Date 11/06/14	Project No. 750612403	Figure D-1
LANGAN TREADWELL ROLLO		



(a) PGA



(b) S_a, T = 1.0 seconds



(c) S_a, T = 4.0 seconds

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2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS DEAGGREGATION

Date 11/06/14	Project No. 750612403	Figure D-2

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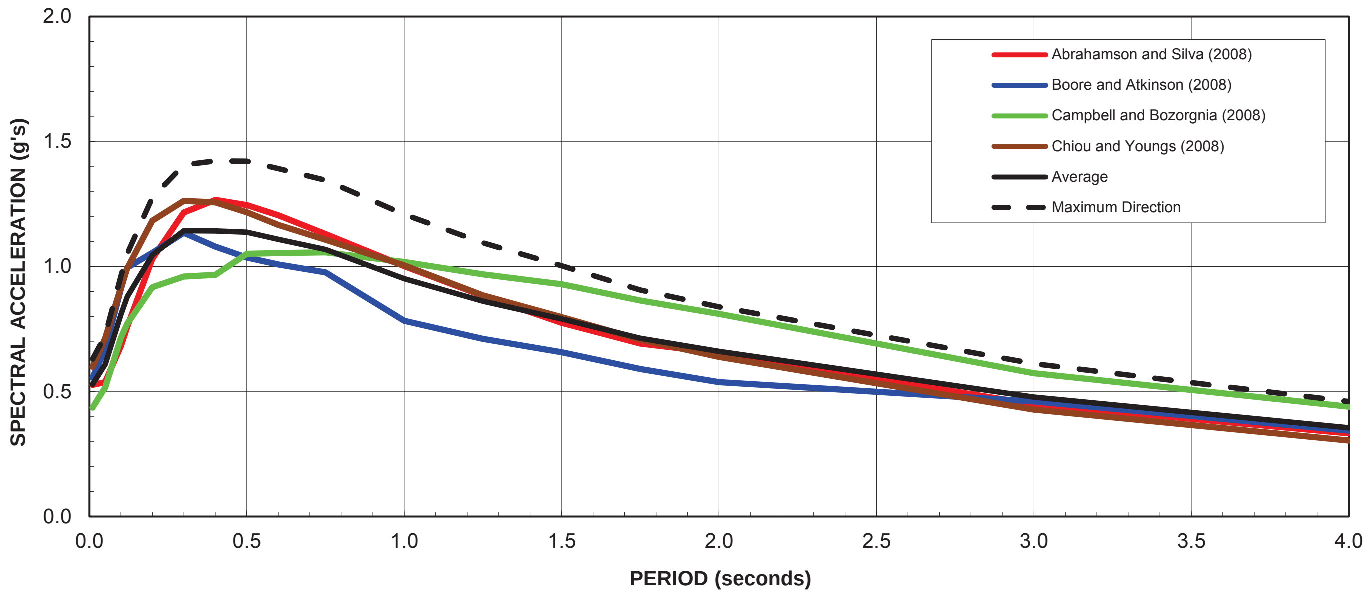

Damping Ratio = 5%

Note:

1. Estimated site avg. V_{s30} = 245 m/s
2. Maximum direction based on Shahi and Baker (2013)

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RESULTS OF DETERMINISTIC ANALYSIS - 84th PERCENTILE FOR SAN ANDREAS EVENT
(M_w = 8.0, Dist. = 10.7 km)

Date 11/06/14	Project No. 750612403	Figure D-3

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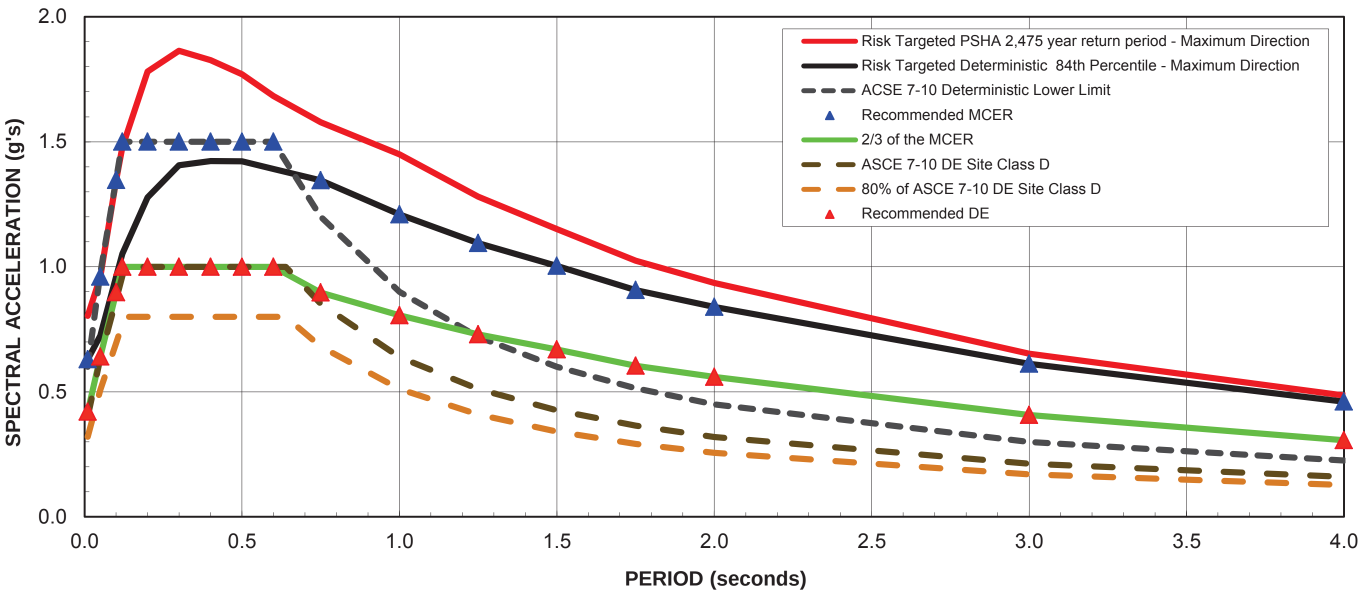

Damping Ratio = 5%

Note:
1. Estimated site avg. V_{s30} = 245 m/s

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COMPARISON OF DETERMINISTIC, PROBABILISTIC AND CODE SPECTRA

Date 11/06/14	Project No. 750612403	Figure D-4

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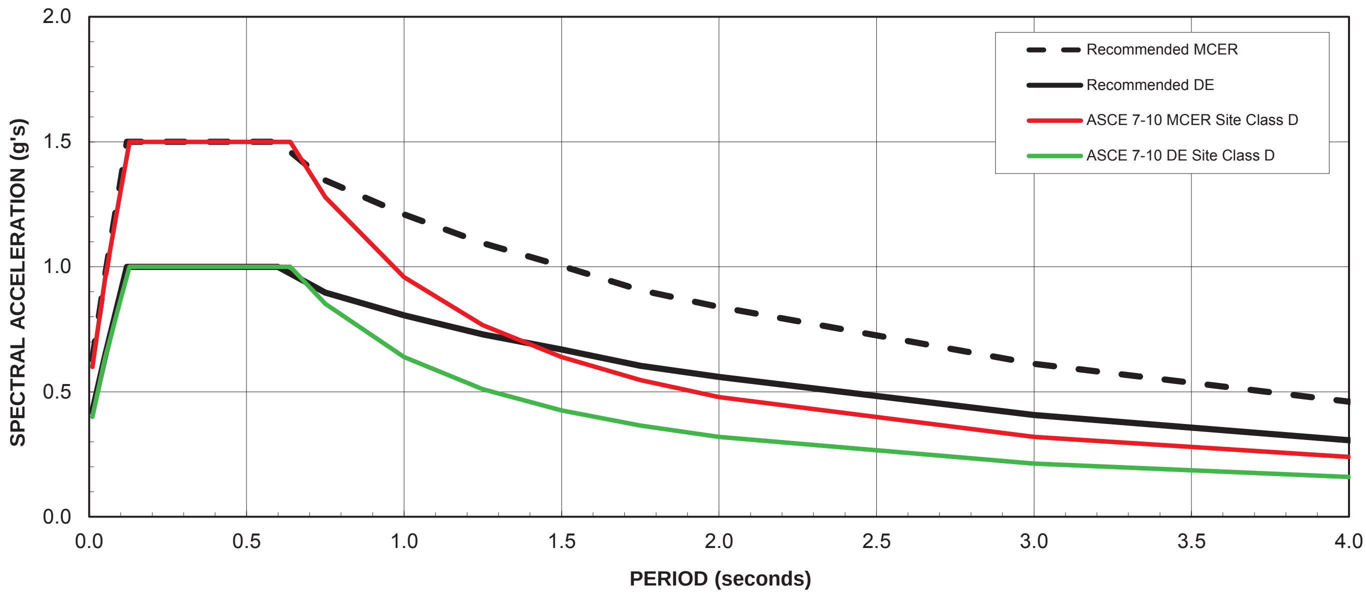

Note:
1. Estimated site avg. V_{s30} = 245 m/s

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COMPARISON OF RECOMMENDED AND ASCE 7-10 SPECTRA

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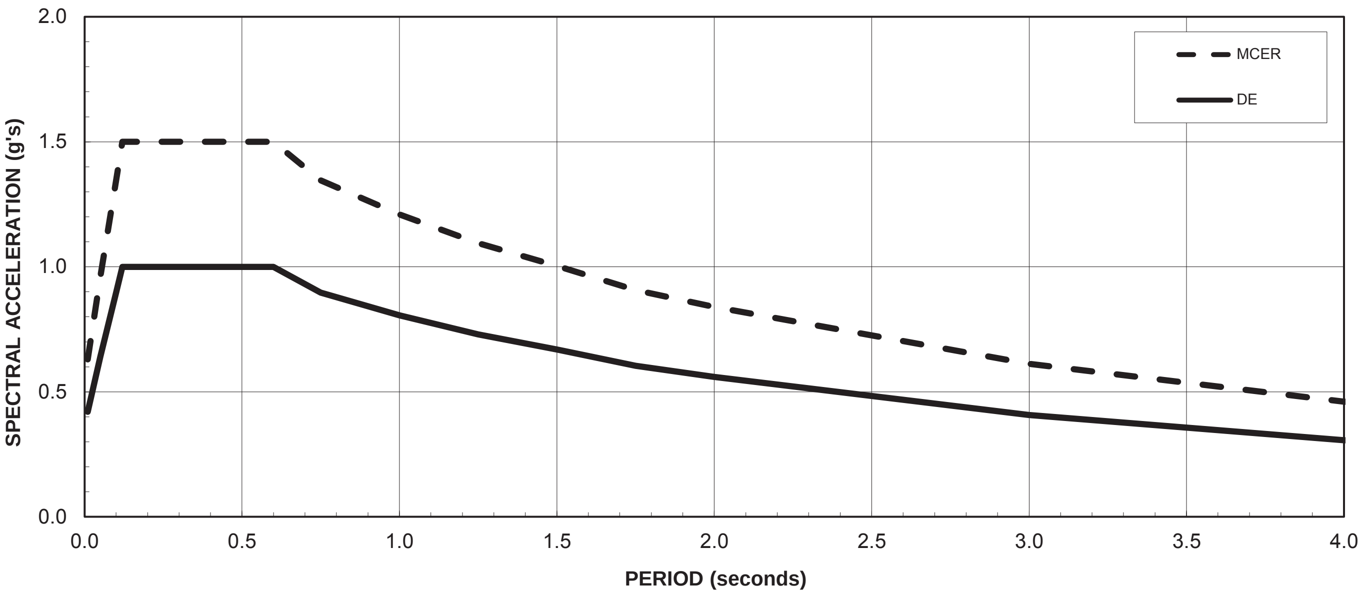

Damping Ratio = 5%

Note:
1. Estimated site avg. V_{s30} = 245 m/s

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RECOMMENDED MCE_R AND DE SPECTRA

Date 11/06/14	Project No. 750612403	Figure D-6

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