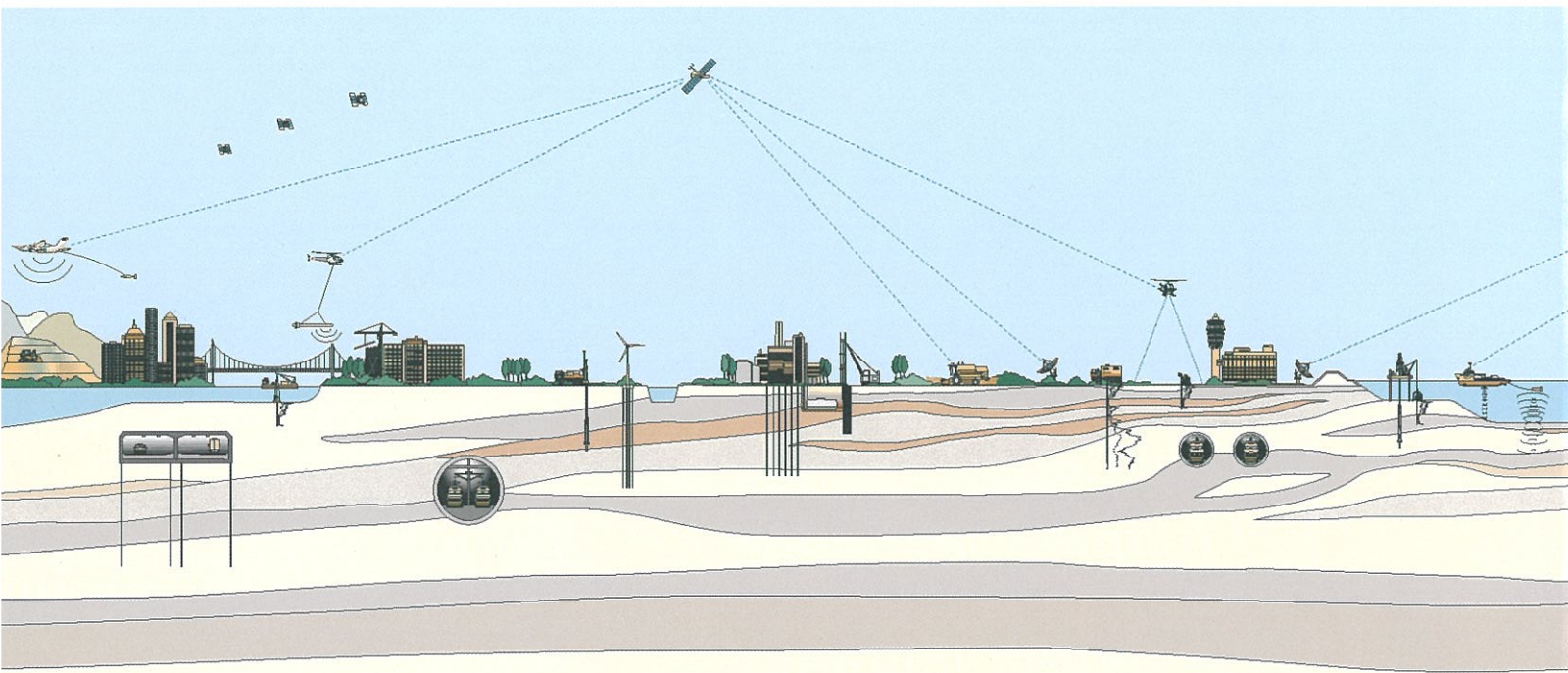


# **GEOTECHNICAL STUDY, PATIENT SIMULATION CENTER, VETERANS AFFAIRS PALO ALTO HEALTH CARE SYSTEM, PALO ALTO, CALIFORNIA**

Prepared for:  
Cannon Design

MAY 2013  
Fugro Project No. 04.72130005





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May 8, 2013  
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Attention: Mr. Mark Herman

Subject: Geotechnical Study, Patient Simulation Center, Veterans Affairs Palo Alto Health  
Care System, Palo Alto, California

Dear Mr. Herman:


Fugro Consultants, Inc. (Fugro) is pleased to submit this geotechnical study for the proposed Patient Simulation Center (Sim Center) at the Veterans Affairs Palo Alto (VA Palo Alto) Health Care System campus, Palo Alto, California.

This study presents the results of our field exploration and laboratory testing program, engineering analyses, and geotechnical conclusions and recommendations for the proposed facility and associated improvements. We appreciate this opportunity to be of service to Cannon Design. If you have any questions regarding the information presented in this report, please contact us.



Sincerely,  
FUGRO CONSULTANTS, INC.

  
Victor A. Crosariol, P.E.  
Project Engineer

  
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VAC/EPW:afp

Copies Submitted: (6+PDF)

Addressee

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## 1.0 INTRODUCTION

This report presents the results of a geotechnical study conducted by Fugro Consultants, Inc. (Fugro) for the proposed Patient Simulation Center (Sim Center) at the Veterans Affairs Palo Alto (VA Palo Alto) Health Care System campus. VA Palo Alto is located at 3801 Miranda Avenue in Palo Alto, California. The site location is shown on Plate 1 – Vicinity Map, with the following horizontal coordinates:

Latitude: 37.4068° N

Longitude: 122.1400°W

Please note that all elevations (El.) in this document reference the North American Vertical Datum of 1988 (NAVD88).

### 1.1 PROJECT DESCRIPTION

The Sim Center will consist of a new, two-story building that will serve as a medical training facility. The building site is currently an asphalt paved parking lot with planter and walkway areas adjacent to the new VA Education Center building, in the northeastern portion of the VA Palo Alto campus.

Based on preliminary plans provided by Cannon Design, we understand that the Sim Center consists of a two-floor steel framed superstructure with a gross area of about 12,000 square feet. The level one finished floor elevation will be at 96.90 feet. The Sim Center will adjoin to the east side of the Education Center building to create a functionally joined but structurally separate complex. Preliminary plans also show a curtain wall and retaining walls to accommodate topography that slopes downward to the north. Additional site improvements include planting areas, walkways, new utilities, and a reconfigured parking area.

### 1.2 SCOPE OF WORK

The purpose of our geotechnical field exploration and laboratory-testing program was to obtain information on subsurface conditions to evaluate the geotechnical aspects of the site where the Sim Center will be located. The scope of our services included:

- Review of available geotechnical and geologic data pertinent to the project, including previous geotechnical studies at the project site;
- Field exploration consisting of three exploratory borings to evaluate the subsurface conditions at the site;
- Laboratory testing program to characterize the engineering properties and corrosion potential of soils encountered during our field investigation;
- Engineering analysis to evaluate site earthwork, foundations, pavement, and site infrastructure;

- Development of geotechnical recommendations for design and construction of the proposed site improvements; and
- Preparation of this report presenting the results of our geotechnical study, and recommendations for design and construction of the proposed project.

## **2.0 DATA REVIEW, FIELD EXPLORATION AND LABORATORY TESTING**

The exploration and laboratory testing program described herein was developed to provide a general geotechnical characterization of the subsurface materials.

### **2.1 REVIEW OF EXISTING DATA**

We reviewed the results of a June 2011 geotechnical study performed by Treadwell & Rollo (T&R) for the adjacent Education Center. T&R drilled two borings to a maximum depth of 26 feet below ground surface (bgs) and advanced one cone penetration test (CPT) sounding to a depth of 45 feet bgs. The T&R report also included a previous boring performed by Woodward-Clyde Consultants (WCC) in October 1992 for the adjacent Building 101 southwest of the Sim Center site. The WCC boring was drilled to a maximum depth of 81½ feet bgs. The approximate locations of the previous explorations by T&R and WCC are shown on Plate 2 – Site Plan.

We also reviewed relevant, available information relating to geotechnical, geologic, and seismic data within the vicinity of the site, including geologic and seismic hazard maps. Pertinent documents are summarized in Section 9.0 – References.

### **2.2 FIELD EXPLORATION**

Our field investigation consisted of drilling three exploratory borings on January 22, 2012. The approximate locations of these borings, and the previous borings and CPT sounding by others are shown on Plate 2.

Hew Drilling Company (Hew) of East Palo Alto, California advanced three exploratory test borings, designated B-01, B-02, and B-03, using truck-mounted, solid-stem auger drilling equipment. The borings were advanced to depths ranging from about 26½ to 27½ feet bgs. The borings were backfilled with neat cement grout in accordance with Santa Clara Valley Water District requirements.

Representative soil samples were obtained from the borings using a Modified California split-barrel drive sampler (outside diameter of 3.0 inches, inside diameter of 2.5 inches) and a Standard Penetration Test (SPT) split-barrel drive sampler (outside diameter of 2.0 inches, inside diameter of 1.375 inches). We collected samples at intervals ranging between 1½- to 5-foot to the bottom of the borings.

Logs of the borings and details regarding the field explorations are included in Appendix A – Field Explorations. The subsurface and groundwater conditions encountered in our field investigation are summarized in Sections 4.2 and 4.3, respectively.



## **2.3 GEOTECHNICAL LABORATORY TESTING**

The soil samples collected during our field investigation were transported to our laboratory in Oakland, California, for review and confirmation of field classifications. Our geotechnical laboratory testing program consisted of moisture content, dry density, fines content, sieve analyses, Atterberg Limits, and R-value testing. Details of our laboratory testing program are presented in Appendix B – Laboratory Testing Program. Laboratory test results are presented in Appendix B and/or on the boring logs (Appendix A) at the appropriate sample depths.

## **3.0 GEOLOGY AND SEISMIC SETTING**

### **3.1 REGIONAL GEOLOGY**

The project site is located along the western margin of the San Francisco Bay in the Coast Ranges geomorphic province of California, which is characterized by northwest-southeast trending valleys and ridges. These valleys and ridges are controlled by folds and faults that resulted from the collision of the Pacific and North American plates, subduction of the Pacific Plate beneath the North American Plate, and subsequent strike-slip faulting along the San Andreas Fault zone and the plate boundary fault systems. Bedrock underlying the region is primarily of the Franciscan Complex, characterized by a diverse assemblage of sandstone, shale, chert, greenstone and mélangé.

Geologic formations in the San Francisco Bay Region range in age from Jurassic (190 to 135 million years ago) to recent Holocene (less than 11 thousand years ago). The Franciscan Complex is the oldest, and underlies younger surficial deposits throughout the San Francisco Bay Region. The Franciscan Complex consists mainly of marine-deposited sedimentary and volcanic rocks in close association with bodies of serpentine. Following deposition, the Franciscan rocks were regionally uplifted and, in the process, extensively faulted and folded.

The Bay Area has experienced several episodes of uplift and faulting during late Tertiary time (about 25 to 2 million years ago). This produced a series of northwest-trending valleys and mountain ranges, including the Berkeley Hills, the San Francisco Peninsula and the intervening San Francisco Bay. Uplifted areas were eroded, and as a result, Pleistocene and recent marine sediments were deposited in the San Francisco Bay and stream and marshland sediments were deposited in low-lying areas adjacent to the Bay.

### **3.2 SITE GEOLOGY**

VA Palo Alto is located near northeastern foot of Cayote Hill. The geology of the northeastern flank of Cayote Hill is mapped as the Santa Clara Formation (Qsc); characterized as Pleistocene to late Pliocene age, gray to red-brown conglomerate, sandstone, and mudstone (Dibblee and Minch 2007). Dibblee and Minch mapped the geology of the VA Palo Alto campus as being underlain by Holocene age alluvial deposits (Qa). Pampayan (1993) maps the geology of the site as being underlain by the Santa Clara formation. Explorations at the Sim Center site

encountered alluvial deposits (beneath artificial fill), probably underlain by the Santa Clara formation. Subsurface conditions at the Sim Center site are discussed in Section 4.2.

Structurally, the Sim Center site is located on the northeast flank of a northeast trending anticline fold (Dibblee and Minch 2007), implying that bedrock dips to the northeast. Both maps discussed above show bedrock in the area dipping 20 to 35 degrees to the northeast.

### **3.3 REGIONAL SEISMICITY**

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. Dominated by the San Andreas Fault system, the Bay Area is comprised of mostly northwest-trending strike-slip faults derived by the interaction of the Pacific and North American Tectonic Plates. Movement between these two plates is predominantly accommodated on the San Andreas, Hayward-Rogers Creek, Calaveras, San Gregorio, and Concord-Green Valley faults. The major fault in the system is the San Andreas Fault, a major rift in the earth's crust that extends for at least 750 miles.

In 2008, the United States Geological Survey (USGS), in conjunction with Southern California Earthquake Center and the California Geological Survey, published the Uniform California Earthquake Rupture Forecast (UCERF). UCERF updated the forecast made in 2003 by the Working Group for California Earthquake Probabilities (WGCEP). The UCERF report evaluated the probabilities of significant earthquakes occurring in the Bay Area over the next three decades (2007-2036). UCERF found a 63 percent probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2036. This probability is an aggregate value that considers eight principal Bay Area fault systems and unknown faults (background values). The San Francisco Bay region continues to be seismically active. The principal active faults in the Bay Area include the San Andreas, Hayward, Calaveras, and the San Gregorio faults. Earthquakes occurring along these faults are capable of generating strong ground shaking at the project site.

The approximate distances of the site to the eight (8) closest known mapped active faults, based on the EQFAULT program by Thomas F. Blake (Blake, 2000), are summarized in Table 1. We ran the EQFAULT program using the 2002 CGS fault model (Blake, 2002).



**Table 1. Regional Active Faults**

<b>Fault</b>	<b>Approximate Distance from Site</b>	<b>Direction from Site</b>	<b>Maximum Moment Magnitude</b>
Monte Vista - Shannon	2.5 miles (4.0 km)	Southwest	6.7
San Andreas	4.7 miles (7.6 km)	Southwest	7.9
Hayward	14.4 miles (23.1 km)	Northeast	7.3
San Gregorio	15.8 miles (25.5 km)	West	7.4
San Andreas - Santa Cruz	17.6 miles (28.3 km)	South	7.0
Calaveras	18.2 miles (29.3 km)	Northeast	6.9
Zayante-Vergales	23.8 miles (38.3 km)	South	7.0
Mount Diablo	28.8 miles (46.3 km)	Northeast	6.7

Earthquakes on these or other active faults (including unmapped faults) could cause strong ground shaking at the site. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

## **4.0 SITE CONDITIONS**

### **4.1 SURFACE CONDITIONS**

The Sim Center site is located in the northeastern portion of the VA Palo Alto campus. The site is bounded by the perimeter access road (Loop Road) to the north, an existing parking area (Lot 101) to the west, Building 101 to the south, and the new Education Center to the east. The proposed building footprint is currently occupied by asphalt concrete pavement, planter areas, walkways, and existing utilities associated with adjacent buildings. The east side of the site features concrete hardscape and planter areas for the Education Center entrance. The existing grade is about El. 99 feet at the south end of the Sim Center building. Topography slopes gradually down across Lot 101 to about El. 94 feet at the northwest corner of the building before steepening to an embankment down to Loop Road with slope inclinations as steep as 3:1 (horizontal to vertical). An existing driveway with an approximately 9 percent slope occupies the northeast portion of the building footprint.

### **4.2 SUBSURFACE CONDITIONS**

Fugro's subsurface explorations encountered artificial fill overlying alternating layers of alluvial deposits consisting of mixed sand and clay with variable gravel content to the maximum depth explored of 27½ feet. Borings B-01 and B-03 were drilled within the planter areas and Boring B-01 was drilled within the existing asphalt concrete parking lot.

In Borings B-01 and B-03, we encountered artificial fill consisting primarily of medium dense clayey sand with gravel to depths of approximately 9 feet (El. 84 feet) and 3½ feet (El. 92½ feet), respectively. Laboratory tests indicate indicated fines content of up to 45% with a

plasticity index of 28 to 29. Therefore, although some of the clay is classified as a sandy material, it has a relatively high clay content and exhibits behavior similar to a high plasticity clay. At Boring B-02, we encountered a pavement section consisting of about 3 inches of asphalt concrete over about 9 inches of aggregate base. Below the pavement section, we encountered artificial fill consisting of medium stiff to very stiff clay with variable sand and gravel content extending to a depth of approximately 6 feet (El. 93½ feet).

Below the artificial fill, we encountered native, stiff to very stiff sandy lean clay to medium dense clayey sand with variable gravel content extending to depths ranging from about 9 feet (El. 91 feet) in Boring B-02 to about 14 feet (El. 84 feet) in Boring B-01. Underlying the upper native soils, all three borings encountered a medium stiff to hard, very dark brown lean clay with sand. This clay layer is about 9½ thick in Boring B-01 where it extends to about 23½ feet bgs (El. 69½ feet). The layer appears to thin towards the north where it extends to about 16 ½ feet bgs (El. 79½ feet) at Boring B-03 and 10 feet bgs (El. 89½ feet) at Boring B-02. Beneath the very dark brown clay, we encountered layers of yellowish, reddish, and olive brown dense clayey sand and stiff to hard sandy lean clay to the maximum depth explored of 27½ feet.

T&R's (2011) explorations at the adjacent Education Center encountered fill up to 5½ feet thick over interlayered dense to very dense sand with variable clay, gravel, and silt content and medium stiff to hard clay with variable sand and gravel content. The WCC boring (WCC-92-2) for Building 101 classified soils as interlayered of stiff to very stiff clay and silt and dense to very dense sand and gravel to the maximum depth explored of 81½ feet.

The boring logs and related information depict the depth at which specific subsurface conditions were encountered during our field investigation. The approximate locations of the borings were determined by using a measuring tape or pacing and should be considered accurate only to the degree implied by the method used.

#### **4.3 GROUNDWATER**

Free groundwater was not encountered in our borings during drilling. The borings were backfilled with neat cement grout in shortly after drilling. We note that the borings may not have been left open for a sufficient period of time to establish equilibrium ground water conditions. Fluctuation of groundwater level can occur due to change in seasons, variations in rainfall, and other factors.

T&R (2011) did not encounter groundwater in the explorations for the adjacent Education Center, and the WCC boring for Building 101 does not note the groundwater level. The T&R concluded that a design groundwater elevation of 76 feet is appropriate for the Education Center site based on stabilized groundwater levels measured by others. For consistency with the adjoining Education Center, we recommend using a design groundwater level of 76 feet.

## **5.0 DISCUSSION AND CONCLUSIONS**

We believe that the project is feasible from a geotechnical and engineering geologic standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections.

### **5.1 SEISMIC HAZARDS**

#### **5.1.1 Ground Shaking**

Strong ground shaking at the Sim Center site will likely occur during a moderate to severe earthquake on the nearby San Andreas Fault or one of the other active Bay Area faults. The intensity of shaking is dependent on the magnitude of the event, the distance to its zone of rupture, and local geologic conditions.

Strong ground shaking not only can cause structures to shake, but it also has the potential capability of inducing other phenomena that can indirectly cause damage to structures. These phenomena include soil liquefaction, seismically induced waves such as tsunamis and seiches, inundation due to dam or embankment failure, flooding, landsliding, and other shaking hazards such as lateral spreading, ground cracking, lurching, and water movement. Detailed discussions of some of these phenomena with respect to the Sim Center site are presented in the following sections.

#### **5.1.2 Liquefaction and Dynamic Densification**

Seismic liquefaction is a phenomenon in which saturated (submerged), cohesionless soil experiences a temporary loss of strength due to buildup of excess pore water pressure during cyclic loading induced by an earthquake. The susceptibility of a soil to liquefaction is a function of the gradation, density, aging/cementation, and fines content of the soil. The resistance to liquefaction increases with respective increases in a) distribution of grain size, b) soil density, c) aging, d) cementation, e) fines content, and f) plasticity characteristics of the fines. The soil most susceptible to liquefaction is loose, clean, saturated, poorly (uniformly) graded, fine-grained sand.

The liquefaction susceptibility of the site is considered to be low, based on the Association of Bay Area Governments (ABAG) Liquefaction Susceptibility Map (2000). Also, the site is not located in an area designated as a zone of required investigation for liquefaction by California Geological Survey (CGS), as shown on the Seismic Hazard Zones Maps, Palo Alto Quadrangle (CGS, 2006).

Based on the soils encountered in our borings and previous explorations by others, potentially liquefiable soil was not encountered below the anticipated groundwater level. For this evaluation, we consider the liquefaction hazard to be low, supported by the results of site subsurface explorations.

During a major earthquake the potential for seismically-induced settlement or dynamic densification of non-saturated soil has been observed. Soils generally susceptible to seismically-induced settlement are relatively clean, loose to medium dense, cohesionless soils. The soils encountered in our borings do not appear to be susceptible to seismically-induced settlement. Based on the densities and soil types encountered our explorations, the potential for seismically-induced settlement is considered low.

### **5.1.3 Lateral Spreading**

Lateral Spreading occurs where the contact between a layer of liquefiable material and the material below is sloped. Saturated sands lose their strength during an earthquake and become fluid-like and mobile. As a result, the ground may undergo large permanent displacements that can damage underground utilities and well-built surface structures. Lateral spreading involves displacement of large blocks of ground down gentle slopes or towards the open face of slopes such as a stream channel. In areas where there is no open face, buckling of the overburden is often observed. Historic earthquakes in the Bay Area have produced lateral spreading features. However, lateral spreading is considered unlikely at this site, because the probability for liquefaction at the site is considered low (discussed in Section 5.1.2).

## **5.2 SOIL EXPANSION**

Based on the available subsurface data and laboratory test results, the near surface clayey fill soils observed at the Sim Center site has a high expansion potential. Expansive soils are subject to volume change with fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs, sidewalks, and pavements should be anticipated. This movement could result in damage to slabs, foundations, sidewalks, and pavements that might require periodic maintenance or replacement. In order to reduce the potential impact on foundations resulting from swelling and shrinkage of expansive materials, we conclude that the perimeter of the foundations should be deepened to bear below the depth subject to swelling and shrinking. In addition, a layer of non-expansive engineered fill should be provided beneath the interior slabs-on-grade. Note that special design considerations will also apply for the design of exterior slabs and sidewalks. Recommendations related to expansive soils are presented in Section 6.0.

## **5.3 FOUNDATION SUPPORT**

The primary consideration for foundation design is the presence of potentially expansive undocumented fill to depths ranging from 3½ feet to 9 feet bgs. In our opinion, structures supported by such soils may be susceptible to future differential settlement resulting from building loads. We consider the alluvial deposits beneath the fill to be suitable for foundation support of the proposed Sim Center building. Therefore, the Sim Center building may be supported on conventional continuous and isolated spread footings deepened to bear directly on competent native soil beneath the undocumented fill. As an alternative to deepening footings, the foundation excavations may be over-excavated into competent native soil and backfilled with re-worked engineered fill, controlled low-strength material (CLSM), or concrete (lean or structural) to the planned bottom of footing elevation.

The long-term total and differential static settlement of shallow foundations constructed as recommended in this report should be taken into account in the design of the foundations. Section 6.3 presents more detailed foundation recommendations that we judge appropriate for the soils present at the Sim Center site.

## 5.4 CONSTRUCTION CONSIDERATIONS

Excavations will be required to construct building foundations, install utilities, remove existing utilities and foundations, and remove unsuitable soils. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

If earthwork is performed during the dry season, moisture conditioning will be required to raise the in situ moisture contents to near optimum moisture content (per ASTM D1557). If earthwork is performed during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

## 6.0 RECOMMENDATIONS

### 6.1 SEISMIC DESIGN

The proposed structure should be designed to resist the lateral forces generated by earthquake shaking in accordance with Chapter 16 of the 2010 California Building Code (CBC). Based on the site geology and available subsurface information, the site may be characterized as Site Class D, described as a “stiff soil” profile. Seismic design parameters are presented in Table 2.

**Table 2. Seismic Design Parameters**

LATITUDE: 37.4068 LONGITUDE: -122.1400	ASCE 7-05 TABLE/FIGURE	FACTOR/COEFFICIENT	VALUE
Short-Period MCE at 0.2s	Figure 22-3	$S_s$	1.959g
1.0s Period MCE	Figure 22-4	$S_1$	0.795g
Soil Profile Type	Table 20.3-1	Site Class	D
Site Coefficient	Table 11.4-1	$F_a$	1.00
Site Coefficient	Table 11.4-2	$F_v$	1.50
Adjusted MC Spectral Response Parameters	Equation 11.4-1	$S_{MS}$	1.959
	Equation 11.4-2	$S_{M1}$	1.192
Design Spectral Acceleration Parameters	Equation 11.4-3	$S_{DS}$	1.31
	Equation 11.4-4	$S_{D1}$	0.80

Based on the above seismic design parameters and per 2010 CBC § 1613.5.6, structures of Occupancy Category I, II, and III (defined in 2010 CBC Table 1604.5) shall be assigned a Seismic Design Category "E."

## **6.2 SITE PREPARATION AND GRADING**

### **6.2.1 Site Preparation**

Where necessary, the site should be cleared of all obstructions, including vegetation, root systems and organic materials, miscellaneous fills, top soils, asphalt concrete, concrete, and utility lines. Concrete and pavements may be reused as fill, provided it is broken up to meet the requirements in Section 6.2.3. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material compacted to the requirements in Section 6.2.4. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the geotechnical engineer.

After clearing, the portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the geotechnical engineer at the time of construction. Stripped materials should be removed from the site, or stockpiled for later use in landscaping, if approved by the owner.

### **6.2.2 Subgrade Preparation**

Following excavation to the required grades, soil subgrades in areas to receive engineered fill, as defined in Section 6.2.3, or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned to slightly above optimum moisture content, and compacted to at least 90 percent of the soil's maximum dry density. The top 6 inches of subgrade in areas to receive pavements should be moisture conditioned and compacted to at least 95 percent of the soil's maximum dry density. Locally weak soils, if encountered, should be excavated and replaced, or otherwise stabilized as recommended by the geotechnical engineer at the time of construction. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. If the subgrade is allowed to become dry, it should be moisture conditioned to eliminate shrinkage cracks.

In order to achieve satisfactory compaction of the subgrade and fill materials, it may be necessary to adjust the water content at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet.

### **6.2.3 Engineered Fill Materials**

All fill placed at the site should consist of engineered fill meeting the requirements presented in this report, except for landscaping materials which are placed on level ground.



Onsite soil below the stripped layer and having an organic content of less than 3 percent by volume can be reused as fill. All engineered fill placed at the site, including onsite soils, should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

“Non-expansive” fill should be predominantly granular, have an organic content of less than 3 percent by volume, should have a liquid limit less than 40 percent, have a plasticity index not exceeding 12, and should contain no environmental contaminants or debris. Imported fill should consist of “non-expansive” fill.

#### **6.2.4 Fill Placement and Compaction**

Engineered fill less than 5 feet thick should be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM Designation D1557 (latest edition). The upper 6 inches of subgrade soils beneath slabs and pavements should be compacted to at least 95 percent of the soil's maximum dry density. Engineered fill greater than 5 feet in thickness should be compacted to at least 95 percent of the soil's maximum dry density. Soils should be compacted at moisture content at or close to laboratory optimum. Fill material should be spread and compacted in lifts not exceeding 8 inches in pre-compacted thickness. In order to achieve satisfactory compaction of the subgrade and fill materials, it may be necessary to adjust the water content at the time of construction. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet.

#### **6.2.5 Trench Backfill**

Pipeline trenches should be backfilled with materials satisfying the criteria described above for fill, placed in lifts of approximately 8 inches in pre-compacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the project geotechnical engineer and the required minimum degree of compaction is achieved. Onsite soil used for trench backfill should be compacted to the requirements of the previous section. Alternatively, sand can be used for trench backfill if the sand is compacted to at least 95 percent of the soil's maximum dry density and sufficient water is added during backfilling operations to prevent the soil from “bulking” during compaction. The upper 3 feet of trench backfill below slabs and pavements should be compacted to at least 95 percent relative compaction. Compaction should be achieved by mechanical means only (i.e., jetting should not be permitted).

Where utility trenches backfilled with sand enter the building pad, the trenches should be backfilled by an impermeable plug at the exterior wall foundation. The plugs can be composed of compacted clayey soil, compacted bentonite, or a bentonite-cement or sand-cement slurry mixture. The plugs should be at least 2 feet thick and should extend at least 2 feet beyond the edges and bottom of the trench to ‘key in’ the plug. The plug should also extend to within 1 foot of the lowest adjacent grade.

### 6.2.6 Surface Drainage

Positive surface gradients should be provided adjacent to structures to direct surface water away from foundations and slabs toward suitable discharge facilities. Similarly, roof downspouts should be connected to solid collector pipes that discharge to appropriate facilities. Ponding of surface water should not be allowed adjacent to foundations or on pavements.

### 6.3 FOUNDATION SUPPORT

As discussed in Section 5.4, the proposed building may be supported on conventional continuous and isolated spread footings. Footings should be deepened bear directly on competent soils beneath the undocumented fill. Alternatively, the foundation excavations may be over-excavated into competent native soil and backfilled with re-worked engineered fill, controlled low-strength material (CLSM), or concrete (lean or structural) to the planned bottom of footing elevation.

Because of the expansive nature of the onsite soils, we recommend that the entirety of the perimeter foundation be continuous and extend to a minimum of 24 inches below the lowest adjacent finished grade. Interior footings should extend a minimum of 24 inches below the lowest adjacent soil subgrade elevation. Continuous footings should be at least 18 inches wide, and isolated footings should be at least 24 inches wide. Interior slabs-on-grade should be supported on a minimum of 12 inches of select, predominantly granular “non-expansive” engineered fill. Section 6.4.1 provides detailed recommendations for interior slabs-on-grade.

Footings located adjacent to other footings or buried utilities should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench. We understand that a portion of the eastern edge of the Sim Center will functionally adjoin the existing Education Center building. Footing and grade beam subgrade elevations along the adjoining building edge should be designed to bear at similar elevations as the adjacent Education Center. The geotechnical engineer should be consulted for special construction recommendations if foundation excavations along the adjoining edge need to extend below the as-built Education Center foundation bearing elevations.

Spread footings should be designed for allowable bearing pressures as shown in the following table:

**Table 3. Allowable Bearing Pressures for Spread Footings**

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	2,000
Dead plus Live Loads	3,000
Total Loads (including wind or seismic)	4,000

Based on these bearing pressures and the conditions encountered in our test borings, we estimate the post construction total and differential settlements to be less than about 1/2 inch and 1/4 inch, respectively.

We recommend that we observe the footing excavations prior to placing reinforcing steel or concrete, to check that footings are founded on appropriate material. All foundation excavations should be cleaned of loose material and should be free of water. The footings should be kept moist prior to concrete placement.

### **6.3.1 Lateral Load Resistance**

Resistance to lateral loads may be provided by friction along the base of foundations and by passive pressures acting on the sides of foundations and grade beams. A friction coefficient of 0.35 times the dead load may be used to evaluate the frictional resistance along the bottom of foundations constructed on undisturbed native soil or engineered fill. A friction coefficient of 0.40 times the dead load may be used to evaluate the allowable frictional resistance along the bottom of foundations on concrete or CLSM. A passive pressure equal to an equivalent fluid pressure (EFP) of 250 pounds per cubic foot (pcf) in existing fill, 300 pcf in engineered fill, or 350 pcf in native soil can be used for lateral load resistance against the face of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material. The upper one-foot of soil should be ignored, unless it is confined by a slab. Note that relatively large deflections are required to mobilize the passive resistance. The above allowable passive pressures assume a deflection of approximately 1/2 inch in order to fully mobilize the passive resistance.

## **6.4 CONCRETE SLABS-ON-GRADE**

### **6.4.1 Interior Slabs-on-Grade**

We recommend that interior slabs-on-grade be supported on a minimum of 12 inches of select, predominantly granular "non-expansive" engineered fill meeting the requirements discussed in Section 6.2.3. In addition, slabs should be reinforced with a minimum of #4 bars on 18-inch centers, both ways. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. Slab subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

If migration of moisture through the floor slab is undesirable, a moisture retarder system should be provided between the slab and subgrade. We recommend that the system consist of 4 inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than 3 percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder that is placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class A, B, or C Underslab Vapor Retarders.

If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or

equal to 0.006 gr/ft<sup>2</sup>/hr (i.e., .012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A") may be used in place of the retarder.

During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing. The vapor retarder or barrier should be placed directly under the slab, or at the structural engineer's option, the retarder may be covered with 2 inches of sand. Sand, if used, should be lightly moistened just prior to placing the concrete.

#### **6.4.2 Concrete Exterior Slabs-on-Grade**

As discussed in Section 5.2, the onsite soils have a high expansion potential if subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs, sidewalks, and pavements should be anticipated. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

To reduce the impact of expansion pressures, consideration should be given to reinforcing exterior slabs such as sidewalks with steel reinforcing bars in lieu of wire mesh. Dowels should be provided at all expansion and cold joints. Although adequately reinforced sidewalks and other exterior slabs may still crack, trip hazards requiring replacement of the slabs will be reduced. To reduce the formation of large cracks and offsets in the slab, #4 bars spaced at approximately 18 inches on center in both directions could be used. All cold joints and expansion joints could be constructed with #4 bar dowels. The dowels should be at least 24 inches long and should be spaced at a maximum lateral spacing of 12 inches.

Walkways and pavement curbs and gutters should be supported directly on properly prepared native soils. Eliminating rock base beneath slabs will reduce the potential for migration of landscape irrigation water into pavement and walkway subgrade. Curbs should extend to the bottom of the pavement and baserock layer. One to two days prior to placing concrete, subgrade soils should be soaked to increase their moisture content to at least 3 to 5 percent above laboratory optimum moisture (ASTM D1557). The water content of subgrade soils should be checked by field testing by the Geotechnical Engineer prior to placing concrete.

To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and/or a "drip" irrigation watering system be used. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the tree from slabs or pavements to reduce the effects of tree roots on these improvements.

#### **6.5 RETAINING WALLS**

Unrestrained walls (walls that are free to rotate) should be designed for active earth pressure. Restrained walls (walls that are prevented from rotating) should be designed for at-rest earth pressures. Walls must also be designed to resist additional loads caused by

surcharging and seismic pressures. If drainage is not provided behind retaining walls, as discussed below, the walls should be designed for the additional equivalent fluid pressure (EFP) below to account for hydrostatic pressures. Walls should be designed for the following lateral loads:

*Retained Earth Pressure:* We recommend that restrained retaining walls be designed to resist an equivalent fluid pressure of 45 pcf. Unrestrained retaining walls (walls that are free to rotate) should be designed to resist an equivalent fluid pressure of 35 pcf. These values assume level backfill.

*Surcharge Loads:* Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained and restrained retaining walls, respectively. The design surcharge should include the anticipated surcharge caused by equipment during construction. Surcharge loads from adjacent structures need to be considered if the proposed walls extend below the zone of influence of adjacent foundations. The zone of influence of adjacent foundations can be defined as the area below an imaginary 1.5:1 (horizontal to vertical) line extending downward from the bottom of footings near the wall.

*Sloping Backfill:* Retaining walls with backfill inclined upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for every 2 degrees of slope inclination.

*Seismic Pressure:* For seismic loading conditions, a seismic increment of equal to a uniform pressure of  $15H$  psf, where  $H$  equals the height of the wall in feet, should be applied to the entire wall height. For restrained walls under seismic loading, the retained earth pressure (above) may be reduced to the active (unrestrained) condition (i.e., from 45 to 35 pcf, plus the seismic increment).

The recommended lateral pressures assume walls are fully back-drained to prevent the build-up of hydrostatic pressures. Additional drainage could be provided by means of either weep holes with permeable material installed behind the walls or by means of a system of subdrains. If adequate drainage is not provided behind walls, the walls should be designed for an additional equivalent fluid pressure of 45 pcf over the full retained height of the wall to account for hydrostatic pressures.

The back-drainage system should consist of a drain rock layer at least 12 inches thick extending to within 1-foot of the ground surface. Four-inch-diameter perforated plastic pipe should be installed (with perforations down) along the base of the walls on a 2-inch thick bed of drain rock. The pipe should be sloped to drain by gravity to a suitable drainage facility. Drain rock should conform to Caltrans specifications for Class 2 permeable material. A more open-graded material, such as 3/4-inch crushed rock, could be used, provided the rock is surrounded by a geotextile (such as Mirafi 140N or equivalent) to reduce the migration of fine-grained soils into the drain rock. Pavement, concrete, or a one-foot-thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration. Alternatively, wall back-drainage can be provided by a prefabricated drainage material (such as Miradrain 6000 or an approved

alternative). The drainage material can be installed on the back (soil) face of the retaining walls and should terminate at a 4-inch diameter perforated plastic pipe surrounded by at least 6 inches of drain rock as defined above.

Retaining wall backfill less than 5 feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

## 6.6 PAVEMENTS

### 6.6.1 Flexible Pavement Design

One R-value (resistance) test was conducted on a bulk sample from the upper 5 feet of Boring B-01 of the onsite near-surface materials. The result of this test indicated an R-value of 19 and is presented in Appendix B. We developed the following pavement sections based on Topic 630 of the State of California Department of Transportation Highway Design Manual. In our design, an R-value of 19 was assumed for the following assumed traffic indices. Pavement designs for pavement lives of 11 to 20 years are presented below.

**Table 4. Recommended Pavement Design Alternatives**

Location	Anticipated Pavement Life (years)	Pavement Components		Total Thickness (inches)
		Asphalt Concrete (inches)	Caltrans Class 2 Aggregate Base (inches)	
Automobile Parking & Access Areas (T.I. = 5.0)	11-20	3.0	8.0	11
Heavy Truck Access (T.I. = 6.5)	11-20	3.5	11.5	15

The traffic indices used in our design were established assuming typical automobile traffic for a T.I. = 5.0, and heavy truck access for a T.I. = 6.5 once construction has been completed. However, if the lighter pavements (T.I. = 5.0) are planned to be placed prior to, or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. Therefore, if the pavement sections will be used for construction access, our firm should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use. If requested, we could provide recommendations for a phased placement of the asphalt concrete to minimize the potential for mechanical scars caused by construction traffic on the finished grade.



The traffic indices should provide the indicated pavement lives with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on geotechnical engineering judgment, and not on an equivalent wheel load analysis either developed from a traffic study or furnished to us. Therefore, the traffic indices provided herein should be confirmed by the project civil engineer as appropriate for the intended use.

In areas where pavements will abut planted areas, the pavement aggregate base layer, pavement section subgrade soils, and trench backfill should be protected against saturation. Planned concrete curbs should extend at least to the bottom of the aggregate base layer, forming a concrete barrier between the landscaped areas and the pavement section. In addition, water should not be allowed to pond behind the curb and gutter during or after the completion of construction.

The Aggregate Base for use in flexible pavements should conform to Caltrans Standard Specification Section 26-1.02A for Class 2 Aggregate Base. The Aggregate Base used in the pavement sections should be compacted to 95 percent relative compaction (ASTM D1557) and should be firm and unyielding at the time of asphalt concrete placement.

## **6.7 ADDITIONAL GEOTECHNICAL SERVICES**

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations prior to construction. The analyses, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from the subsurface explorations conducted in the vicinity of the Sim Center site, and upon the conditions existing when services were conducted. Variations of subsurface conditions from those analyzed or characterized in the report are possible, as may become evident during construction. In that event, it may be advisable to revisit certain analyses or assumptions.

We recommend that Fugro be retained to provide geotechnical services during site grading and foundation installation to observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered. During construction, our field engineer should observe and/or test the following:

- Soil conditions exposed by site grading and foundation excavations, to check that they are consistent with those encountered during the field exploration;
- Pavement subgrade preparation; and
- Fill placement and compaction, including backfill of utilities and compaction of aggregate base.

## **7.0 LIMITATIONS**

Our services consisted of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted, local geotechnical engineering principles and

practices at the time our services were performed. This warranty is in lieu of all other warranties, either express or implied.

The analyses and recommendations contained in this report are based on the data obtained from the subsurface exploration conducted for this study and relevant previous explorations. These explorations indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analysis of the observed conditions. If conditions other than those described in this report are encountered, we should be notified so that we can provide additional recommendations, if warranted.

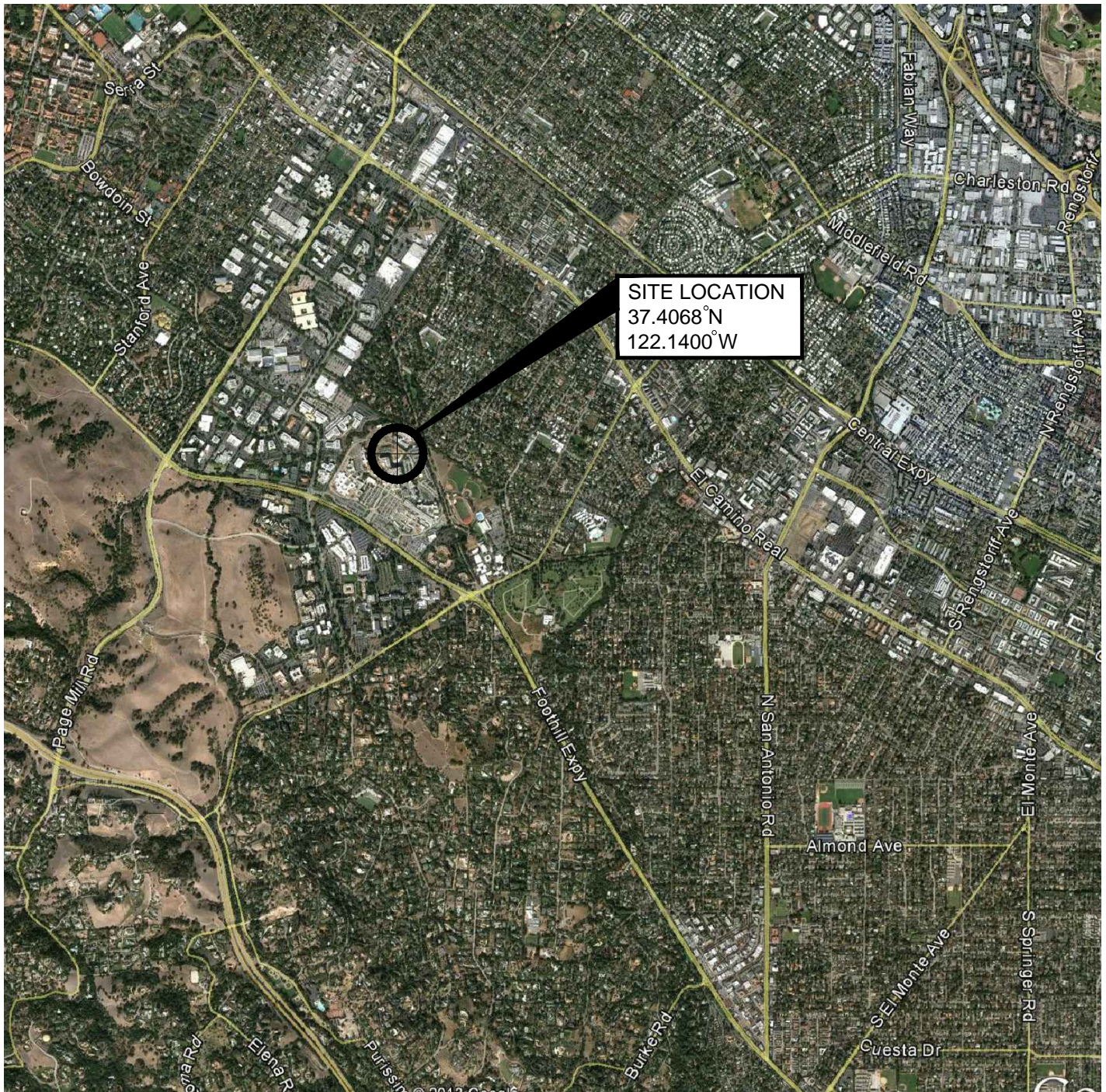
This report has been prepared for the exclusive use of Department of Veterans Affairs, Cannon Design, and their consultants for specific application to the proposed Patient Simulation Center, Veterans Affairs Palo Alto Health Care System in Palo Alto, California. In the event that there are any changes in the ownership, nature, design, or location of the proposed project, or if any future additions are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) the project changes are reviewed by Fugro, and 2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

## 8.0 REFERENCES

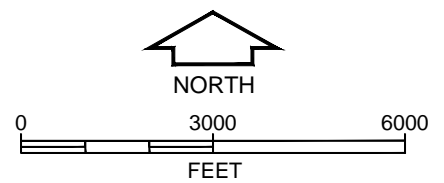
- Association of Bay Area Governments (ABAG), (2000), Map of Liquefaction Susceptibility in the Bay Area, Based on Work by William Lettis & Associates, Inc., and USGS Open File Rpt. 00-444, Knudsen et al., [www.abag.ca.gov/bayarea/eqmaps/liquefac/liquefac.html](http://www.abag.ca.gov/bayarea/eqmaps/liquefac/liquefac.html)
- American Society of Civil Engineers (ASCE), 2006, Minimum Design Loads for Buildings and other Structures, ASCE-7.05.
- Blake, T.F., 2000, *EQFAULT*, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults ,Version 3.00b,
- Blake, T.F., 2002, New Fault-Data Files for Use with EQFAULT Derived from CGS Data, filename: CGSFLTE.DAT, <http://thomasblake.com/>
- California Building Standards Commission, (2010) *California Building Code*, California Code of Regulations, Title 24, Part 2, Volume 2.
- California Department of Transportation (Caltrans), Highway Design Manual, Section 630, May 2012.
- California Geological Survey, 2006, "Seismic Hazard Zone Report for the Palo Alto 7.5-Minute Quadrangle, Santa Clara County, California," Seismic Hazard Zone Report 111, 65p
- Dibblee, T.W., Minch, J.A., 2007, *Geologic Map of the Palo Alto and Mountain View Quadrangles, Alameda, San Mateo, and Santa Clara Counties, California*, Dibblee Foundation Map DF-350, Dibblee Geological Foundation, scale 1:24,000.
- Graymer, R.W., Moring, B.C., Saucedo, G.J., Wentworth, C.M., Brabb, E.E., Knudsen, K.L., 2006, *Geologic Map of the San Francisco Bay Region*, USGS Scientific Investigations Map-2918.
- Pampeyen, E.H., 1993, *Geology of the Palo Alto and Part of the Redwood Point 7.5 Minute Quadrangles, San Mateo and Santa Clara Counties, California*, USGS IMAP: 2371.
- Treadwell & Rollo, Inc., 2011, *Geotechnical Investigation, Mental Health Center, Veterans Affairs Medical Center*, Palo Alto, California, Treadwell & Rollo. Project No. 750487606, June 6, 2011.
- United States Geological Survey (USGS) Working Group on California Earthquake Probabilities (2003), Earthquake probabilities in the SFBP:2002-2031: A Summary of Finding, U.S. Geological Survey Open File Report 03-214.

## PLATES



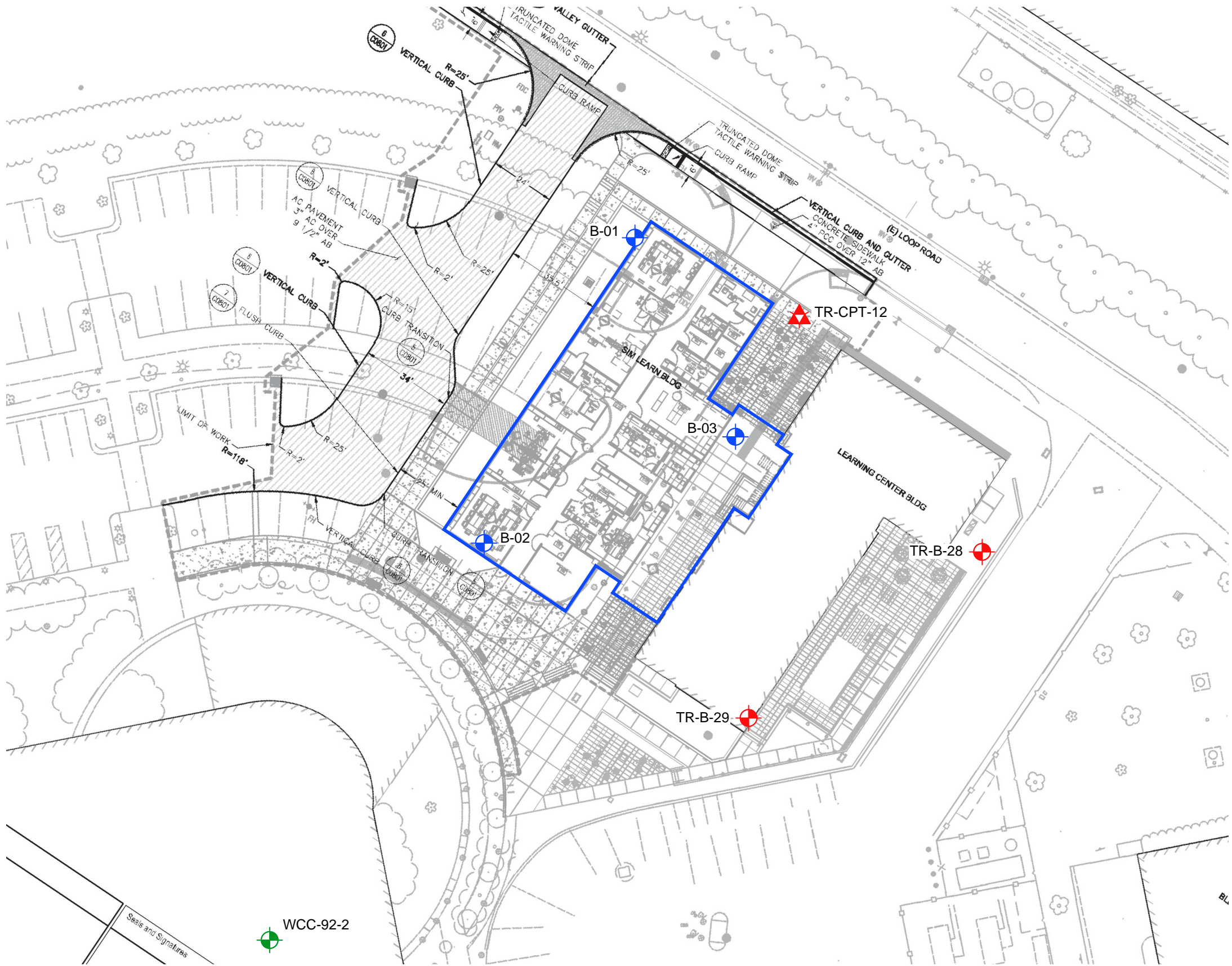


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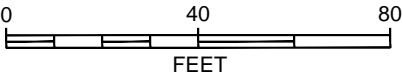
**VICINITY MAP**  
VA Palo Alto Simulation Learning Center  
Palo Alto, California





**LEGEND**

- Approximate VA Simulation Center Footprint
- B-01 Exploratory Boring Location
- TR-B-28 Previous Boring by Treadwell & Rollo for the Education Center (2011)
- WCC-92-2 Previous Boring by Woodward-Clyde Consultants for Building 101 (1992)
- TR-CPT-12 Previous CPT by Treadwell & Rollo for the Education Center (2011)



**SITE PLAN**  
VA Palo Alto Simulation Learning Center  
Palo Alto, California

**BASE MAP SOURCE:** Layout Plan, Drawing No. C0301 of the Development Set, Dated January 30, 2013. Sheet prepared by BKF Engineers.



**APPENDIX A**  
**FIELD EXPLORATION**

## **APPENDIX A FIELD EXPLORATIONS**

The field exploration consisted of a surface reconnaissance and a subsurface exploration program. Our subsurface exploration included drilling three conventional geotechnical borings. The approximate locations of our borings are shown on the Site Plan - Plate 2.

Hew Drilling Company, Inc., of East Palo Alto, California drilled three 6-inch diameter exploratory borings to a maximum depth of 27.5 feet below ground surface. The borings were drilled using a truck-mounted, CME-75 drill rig with solid-stem auger drilling equipment. Our field representative logged soils encountered in the borings, and described the soils in accordance with the Unified Soil Classification System (ASTM D-2487).


















Upon completion of our field explorations, the borings were backfilled with neat cement grout. The logs of the borings are included as part of this appendix.

Representative soil samples were obtained from the borings using a Modified California split-barrel drive sampler (outside diameter of 3.0 inches, inside diameter of 2.5 inches) and a Standard Penetration Test (SPT) split-barrel drive sampler (outside diameter of 2.0 inches, inside diameter of 1.375 inches). All samples were transmitted to our laboratory for evaluation and appropriate testing. Both sampler types are indicated in the "Sampler" column of the boring logs as designated in Plate A-1, Terms and Symbols Used on Boring Logs.

Resistance blow counts were obtained with the samplers by dropping a 140-pound hammer through a 30-inch fall using an automatic trip hammer system. The sampler was driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blow counts were recorded by our engineer in the field per six-inch interval and are presented on the boring logs. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches. Note that field blow counts from samples obtained using the Modified California Liner sampler are not considered standard blow counts because of the larger diameter of the Modified California sampler.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated; we do not warrant that they are representative of subsurface conditions at other locations and times.

## CLASSIFICATION AND MATERIAL SYMBOLS

MAJOR DIVISIONS PER ASTM D2488-06			MAJOR GROUP NAMES AND MATERIAL SYMBOLS		
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	
Notes: Classification of soils on the boring logs is in general accordance with ASTM D2488, or D2487 if appropriate laboratory data are available. The geologic formation is noted in bold font at the top of interpreted interval on the boring logs.			OTHER MATERIAL SYMBOLS		
				Debris or Mixed Fill	
				Pavement with Aggregate Base	

## SAMPLER TYPE

SPT (Driven) 1-3/8" ID 2" OD	Modified California (Driven) 2-3/8" ID 3" OD	Modified California (Driven) 1-7/8" ID 2-1/2" OD
Shelby Tube (Pushed) 2-7/8" ID 3" OD	Pitcher Barrel (Rotary-cut) 2-7/8" ID	Osterberg (Piston) 2-7/8" ID
101 Geobarrel (Rotary-cut) 2-7/8" ID	Rock Core (Rotary-cut) See log for size	Vibracore (Vibrated) See log for size
Push-core (Pushed) See log for size	Collected from Auger	Other See log for details

Note: Refer to text of report for additional details or other sampler types.

## BLOW COUNT

Number of blows required to drive sampler each of three 6-in. intervals, as measured in the field (uncorrected). An SPT hammer (140 lb., falling 30-in.) was used unless otherwise noted on the boring log. For example:

Blow Count	Description
5 7 8	5, 7, and 8 blows for first, second, and third interval, respectively.
35 50/3"	35 blows for the first interval. 50 blows for the first 3 inches of the second interval. Lack of third value implies that driving was stopped 3 inches into the second interval.
WOH WOH 5	"WOH" indicates that the weight of the hammer was sufficient to advance the sampler over the first two intervals. 5 blows were required to advance the sampler over the third interval.

## N-VALUE

The N-Value represents the blowcount for the last 12 inches of the sample drive if three 6-inch intervals were driven. N-value presented is independent of impact energy. If 50 hammer blows were insufficient to drive through either the second or the third interval, the total number of blows and total length driven are reported (excluding the first interval). "ref" (refusal) indicates that 50 blows were insufficient to drive through the first 6-inch interval.

Parenthesis indicate that an approximate correction has been applied for non-SPT drive samplers. For example, a factor of 0.63 is commonly used to adjust blow counts obtained using a 3-inch outside diameter modified California sampler to correspond to Standard Penetration Test.

## UNDRAINED SHEAR STRENGTH

A value of undrained shear strength is reported. The value is followed by a letter code indicating the type of test that was performed, as follows:

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

## OTHER TESTS

Field or laboratory tests without a dedicated column on the boring log are reported in the Other Tests column. A letter code is used to indicate the type of test. For certain tests, a value representing the test result is also provided. Typical letter codes are as follows. Additional codes may be used. Refer to the report text and the laboratory testing results for additional information.

k - Permeability (cm/s)  
Consol - Consolidation  
Gs - Specific Gravity  
MA - Particle Size Analysis  
EI - Expansion Index  
OVM - Organic Vapor Meter

## WATER LEVEL SYMBOLS

▽ Initial water level  
▼ Final water level  
~ Seepage encountered

## INCREASING MOISTURE CONTENT

↓ Dry  
Moist  
Wet

## CONSISTENCY OF COHESIVE SOIL

CONSISTENCY	UNDRAINED SHEAR STRENGTH (KIPS PER SQUARE FOOT)
Very Soft	< 0.25
Soft	0.25 to 0.50
Medium Stiff	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	> 4.0

Note: In absence of test data, consistency has been estimated based on manual observation.

## APPARENT DENSITY OF COHESIONLESS SOIL

APPARENT DENSITY	N-VALUE
Very Loose	0 to 4
Loose	5 to 9
Medium Dense	10 to 29
Dense	30 to 49
Very Dense	> 49



ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: VA Sim Center SURFACE EL: 93.0 ft +/- (rel. NAVD88 datum)	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, $S_u$ ksf	OTHER TESTS
								<b>ARTIFICIAL FILL (af) to 9.0 feet depth</b>							
				9 000	11	12 18"		CLAYEY SAND with GRAVEL (SC): medium dense, dark yellowish brown mottled dark green and black, moist, medium plasticity, fine to coarse sand, trace fine gravel, concrete chunk at 2.0 ft		17	44	45	29		R Value = 19
				4 49	13	18 18"									
90				9 011	13	18 18"									
	5			5 67	13	18 18"									
				9 011	13	18 18"			105	17		44	29		
85				7 911	13	15 18"		CLAYEY SAND (SC): medium dense, dark yellowish brown, moist, fine to coarse sand, trace fine gravel							
	10			7 911	13	15 18"									
				7 911	18	18 18"									
80				6 1015	16	15 18"		Lean CLAY with SAND (CL): stiff, very dark brown mottled yellow, moist, medium plasticity, fine to medium sand						1.5 P	
	15			6 1015	25	18 18"		- very stiff						2.5 P	
				12 1821	24	18 18"		- trace fine gravel, increase sand						4.0 P	
75				8 1725	42	18 18"		CLAYEY SAND (SC): dense, reddish brown mottled olive, moist, fine to coarse sand						3.0 P	
	25														
								NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 26.5 ft  
BACKFILL: Grout  
DEPTH TO WATER: Not Encountered  
FIELDWORK DATE: January 22, 2013  
DRILLING METHOD: 6-in. dia. Solid Stem Auger

HAMMER TYPE: Automatic Trip  
RIG TYPE: CME-75  
DRILLED BY: Hew Drilling Company  
LOGGED BY: VAC  
CHECKED BY: EPW

**LOG OF BORING NO. B-01**  
VA Palo Alto Simulation Learning Center  
Palo Alto, California



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: VA Sim Center SURFACE EL: 99.5 ft +/- (rel. NAVD88 datum)	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, $S_u$ ksf	OTHER TESTS
0							3" Asphalt Concrete (AC)							
0							9" Aggregate Baserock (AB)							
1.3			13	19	16		<b>ARTIFICIAL FILL (af) to 6.0 feet depth</b>	114	18		45	29	3.5 P	
1.4			14		18		Lean CLAY with SAND (CL): very stiff, reddish brown, moist, medium plasticity, fine to coarse sand, trace gravel							
1.7			17		18		SANDY Lean CLAY (CL): medium stiff, dark yellowish brown, moist, medium plasticity, fine to coarse sand, fine gravel							
5			5	6	10			99	19					
5			5	6	10		SANDY Lean CLAY (CL): medium stiff, dark yellowish brown, moist, medium plasticity, fine to coarse sand, fine gravel							
5			5	6	10				22					
10			10	20	16		SANDY Lean CLAY (CL): very stiff, dark yellowish brown, moist, medium plasticity, fine to coarse sand, fine gravel							
10			10	20	16									
10			10	23	15		Lean CLAY (CL): very stiff, dark brown, moist, medium plasticity, trace fine to medium sand						4.3 P	
10			10	23	15									
10			10	10	12		CLAYEY SAND (SC): medium dense, dark yellowish brown, dark brown mottling, moist, fine to coarse sand, fine gravel, medium plasticity clay		21	45				MA
10			10	10	12									
15			15	13	14		- increased sand							
15			15	13	14									
15			15	13	14		SANDY Lean CLAY (CL): stiff, olive brown, moist, low to medium plasticity, fine to coarse sand							
15			15	13	14									
20			20	34	16		CLAYEY SAND (SC): dense, olive brown, moist, fine to coarse sand							
20			20	34	16									
20			20	34	16		SANDY Lean CLAY (CL): hard, dark yellow brown, olive and reddish mottling, low to medium plasticity, fine to medium sand							
20			20	34	16									
25			25	44	18									
25			25	44	18									
25			25	44	18		NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 26.5 ft  
BACKFILL: Grout  
DEPTH TO WATER: Not Encountered  
FIELDWORK DATE: January 22, 2013  
DRILLING METHOD: 6-in. dia. Solid Stem Auger

HAMMER TYPE: Automatic Trip  
RIG TYPE: CME-75  
DRILLED BY: Hew Drilling Company  
LOGGED BY: VAC  
CHECKED BY: EPW

**LOG OF BORING NO. B-02**  
VA Palo Alto Simulation Learning Center  
Palo Alto, California



ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: VA Sim Center SURFACE EL: 96.0 ft +/- (rel. NAVD88 datum)	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, $S_u$ ksf	OTHER TESTS
							<b>MATERIAL DESCRIPTION</b>							
							<b>ARTIFICIAL FILL (af) to 3.5 feet depth</b>							
							SANDY Lean CLAY with GRAVEL (CL): stiff to very stiff, moist, medium plasticity, fine to coarse sand, fine gravel	111	19					
							Well-graded SAND with GRAVEL (SW): stiff to very stiff, gray, fine to coarse sand, fine gravel							
							SANDY Lean CLAY with GRAVEL (CL): stiff, dark yellow brown mottled black, medium plasticity, fine to coarse sand, fine gravel							
							CLAYEY SAND with GRAVEL (SC): medium dense, dark yellowish brown, fine to coarse sand, trace fine gravel, medium plasticity clay	96	23					
									17	28				MA
							- increased clay							
							Lean CLAY (CL): medium stiff, very dark brown, moist, medium plasticity, trace fine to medium sand						0.8 P	
							CLAYEY SAND with GRAVEL (SC): very dense, dark yellow brown, moist, fine to coarse sand, fine gravel, subangular to angular						0.4 T	
							Lean CLAY with SAND (CL): hard, reddish yellow brown, medium plasticity, fine to coarse sand							
							- trace fine dark gray gravel						4.5 P	
							NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 27.5 ft  
BACKFILL: Grout  
DEPTH TO WATER: Not Encountered  
FIELDWORK DATE: January 22, 2013  
DRILLING METHOD: 6-in. dia. Solid Stem Auger

HAMMER TYPE: Automatic Trip  
RIG TYPE: CME-75  
DRILLED BY: Hew Drilling Company  
LOGGED BY: VAC  
CHECKED BY: EPW

**LOG OF BORING NO. B-03**  
VA Palo Alto Simulation Learning Center  
Palo Alto, California



**APPENDIX B**  
**LABORATORY TEST PROGRAM**

## **APPENDIX B LABORATORY TESTING PROGRAM**

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content was determined on nine samples of the materials recovered from the borings in accordance with ASTM Test Designation D2216. This water content is recorded on the boring logs at the appropriate sample depths.

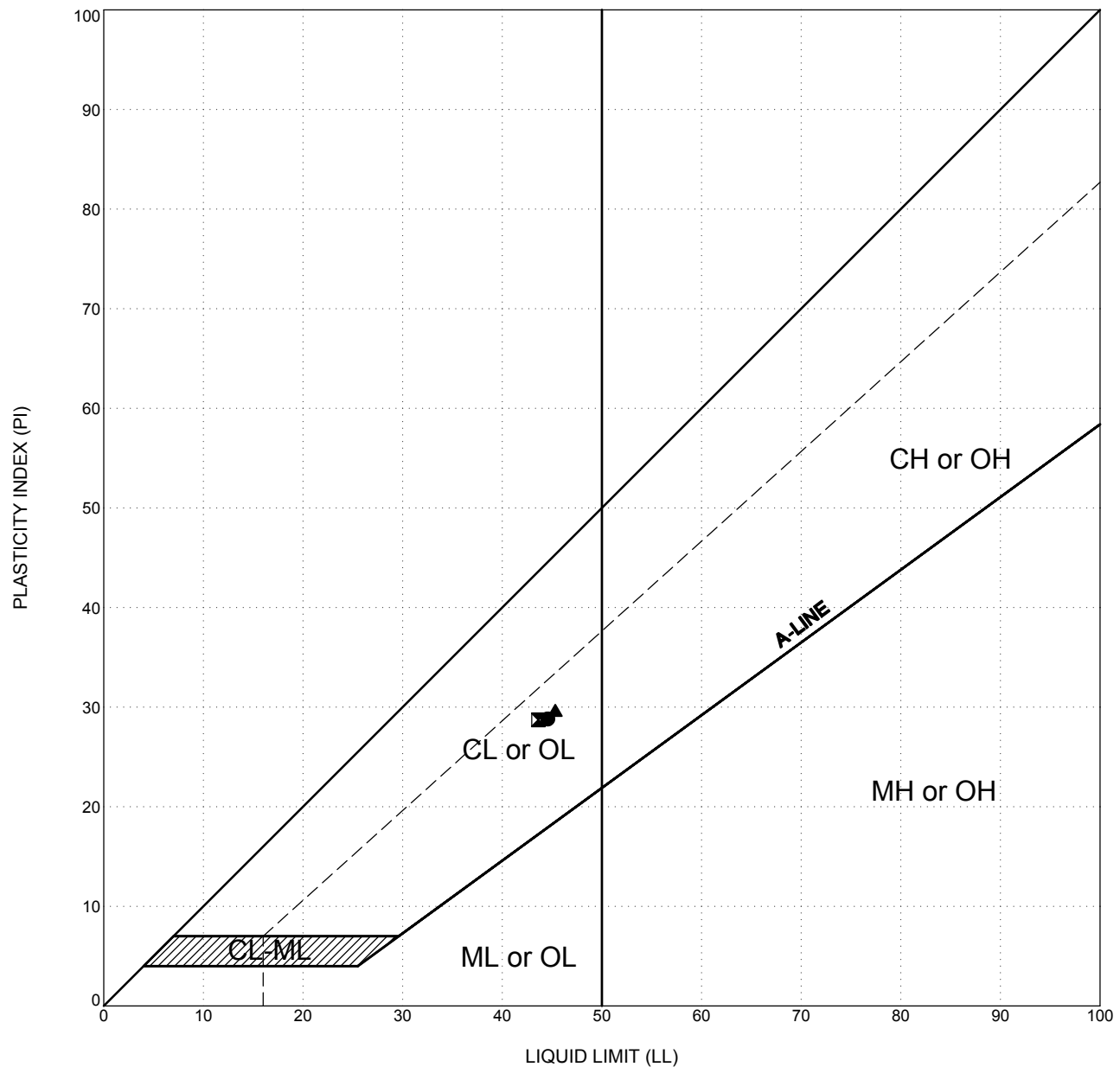
Total and Dry density determinations were performed on twelve (12) samples of the subsurface soils in accordance with ASTM Test Designation D2937. The results of these tests are shown on the boring logs at the appropriate sample depths.

Atterberg Limit determinations were performed on three (3) samples of the subsurface soils to determine the range of water content over which these materials exhibit plasticity. The Atterberg Limits were determined in accordance with ASTM Test Designations D4318. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of these tests are presented on Plate B-1, and on the logs of the borings at the appropriate sample depths.

Gradation tests were performed on two samples of the subsurface soils in accordance with ASTM Test Designation D422. These tests were performed to assist in the classification of the soils and to determine their grain size distribution. The results of these tests are presented on Plate B-2.

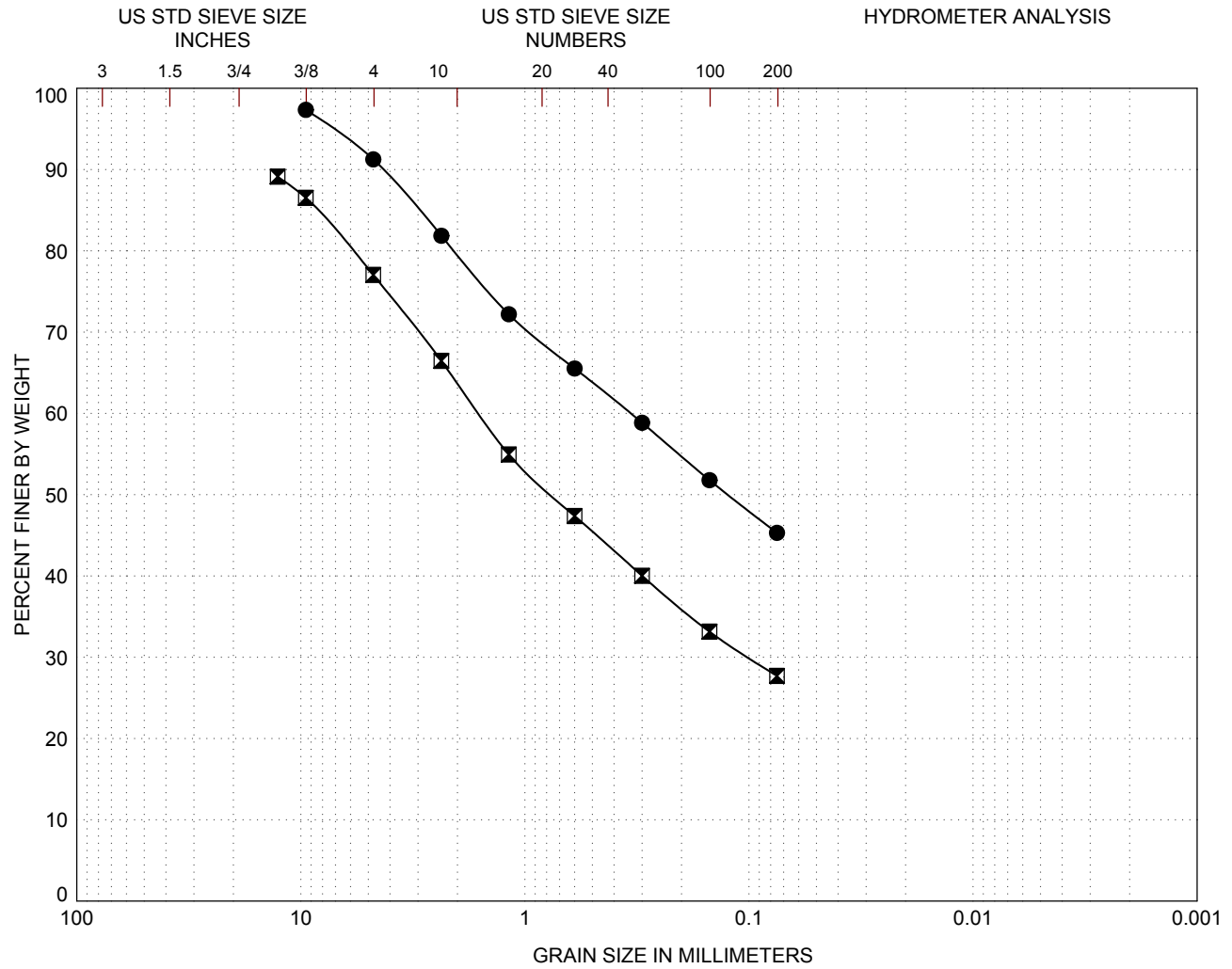
The percent passing the No. 200 sieve was measured on three samples of the subsurface soils to aid in the classification of the soil. These tests were performed in accordance with ASTM Designation D1140. The results of the tests are shown on the boring logs at the appropriate sample depths.

A resistance R-value test was performed on a representative sample of the surface soils onsite to provide data for pavement design. The test was performed in accordance with California Test Method 301-F and indicated an R-value of about 19 at an exudation pressure of 300 pounds per square inch. The result of this test is presented on Plate B-3, and on the boring log at the appropriate sample depth.



LEGEND			CLASSIFICATION			ATTERBERG LIMITS TEST RESULTS		
	location	depth, ft				LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
●	B-01	1.5	CLAYEY SAND with GRAVEL (SC)			45	16	29
⊠	B-01	5.5	CLAYEY SAND with GRAVEL (SC)			44	15	29
▲	B-02	1.5	Lean CLAY with SAND (CL)			45	16	29

**PLASTICITY CHART**  
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GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

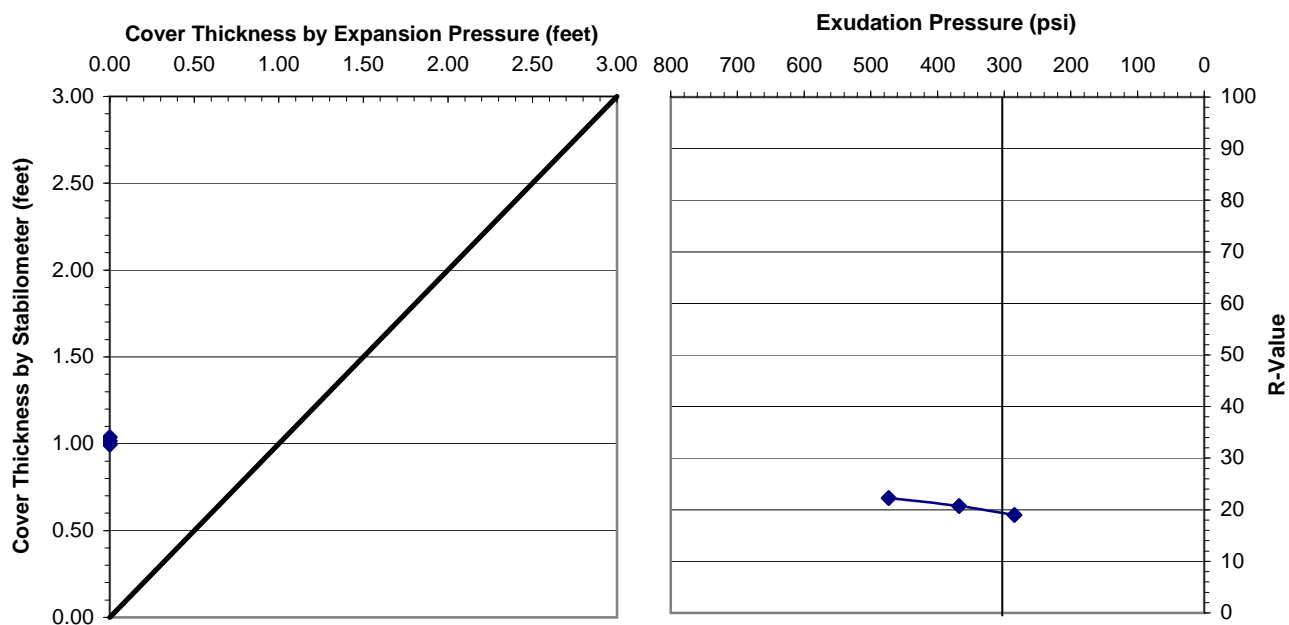
LEGEND	
(location)	(depth,ft)
●	B-02      10.0
⊠	B-03      6.5

CLASSIFICATION
CLAYEY SAND (SC)
CLAYEY SAND with GRAVEL (SC)

C <sub>c</sub>	C <sub>u</sub>	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>
				0.34
		0.10	1.60	

## GRAIN SIZE CURVES

VA Palo Alto Simulation Learning Center  
Palo Alto, California



**Sample No.** Bulk-1 **Depth:** 0.5' - 5.0'  
**Description:** Clayey SAND w/ Gravel (SC) 44% passing #200 Sieve & 79% passing #4  
**Date Tested:** 1/30/2013  
**Test Method:** ASTM D2844, CT301  
**Initial Moisture Content:** 12.9%

Dry Unit Weight (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
108.2	20.6	285	0	19
111.1	18.9	368	0	21
113.4	17.3	473	0	22

**R-value at Exudation Pressure of 300 psi:** 19  
**R-value by Expansion Pressure:** TI = 4 **N/A**

**Remarks:** R-value by stabilometer controls.

**R-VALUE TEST RESULTS**  
**VA Palo Alto Simulation Learning Center**  
**Palo Alto, California**