

**GEOTECHNICAL INVESTIGATION  
PROPOSED OUTPATIENT PHARMACY  
JERRY L. PETTIS MEMORIAL VETERANS MEDICAL CENTER  
11201 BENTON STREET  
LOMA LINDA, CALIFORNIA**

GEOTECHNICAL INVESTIGATION  
PROPOSED OUTPATIENT PHARMACY  
JERRY L. PETTIS MEMORIAL VETERANS MEDICAL CENTER  
11201 BENTON STREET  
LOMA LINDA, CALIFORNIA

Prepared for:

EwingCole  
Irvine, California

By:

GEOBASE, INC.  
23362 Peralta Drive, Unit 4  
Laguna Hills, California 92653  
(949) 588-3744

June 2012  
Project Number: C.329.03.00

# TABLE OF CONTENTS

	Page
Cover Page	i
Table of Contents	ii
I. INTRODUCTION.....	1
II. SITE AND PROJECT DESCRIPTIONS.....	1
2.1 Site Description. ....	1
2.2 Project Description.....	1
III. SITE INVESTIGATION.....	2
3.1 Field Program.....	2
3.2 Laboratory Testing.....	2
IV. SUBSURFACE CONDITIONS.....	3
4.1 Subsoils Conditions. ....	3
4.2 Groundwater.....	4
V. SEISMICITY.....	4
5.1 Site Coordinates.....	4
5.2 Site Classification.....	4
5.3 Site-Specific Ground Motion Procedures - Ground Motion Hazard Analysis.....	5
5.3.1 Mapped Accelerations Response Spectra (Mapped Parameters).....	5
5.3.2 Probabilistic MCE Spectra.....	6
5.3.3 Deterministic MCE Spectra.....	8
5.3.4 Site-Specific MCE Response Spectra.....	9
5.3.5 Design Response Spectra Based on Mapped Parameters.....	9
5.3.6 Site-Specific Design Spectra.....	10
5.3.7 Design Acceleration Parameters.....	10
5.4 Seismic Hazard Deaggregation.....	11
5.5 Seismic Design Category.....	11
5.6 Earthquake Effects.....	11
5.6.1 Liquefaction.....	11
5.6.2 Seismically Induced Settlements.....	12
5.6.3 Tsunami/Seiche, Inundation and Flooding.....	13
5.6.4 Surface Rupture.....	13
5.6.5 Landsliding.....	13
5.6.6 Lateral Spreading.....	13
VI. LIQUEFACTION/SEISMIC SETTLEMENTS MITIGATION.....	13
6.1 Introduction.....	13
6.2 Mitigation Alternatives.....	14
6.3 Evaluation of Mitigation Alternatives.....	14
6.3.1 Discussions.....	14
6.3.2 Conclusions.....	15
6.4 Wet Soil Mixing.....	15

# TABLE OF CONTENTS continued...

Page

VII.	SITE DEVELOPMENT RECOMMENDATIONS. ....	16
7.1	General. ....	16
7.2	Clearing. ....	16
7.3	Subgrade Preparation. ....	17
7.4	Fill Placement. ....	18
7.4.1	Preparation of Surface Soils. ....	18
7.4.2	Compaction. ....	18
7.4.3	Fill Material. ....	18
7.5	Drainage. ....	18
7.6	Temporary Excavations. ....	19
7.7	Trench Backfill. ....	19
VIII.	FOUNDATION RECOMMENDATIONS. ....	20
8.1	General. ....	20
8.2	Footings. ....	20
8.2.1	Soil Bearing Pressures. ....	20
8.2.2	Footings Adjacent to Trenches or Existing Footings. ....	21
8.2.3	Settlement. ....	21
8.2.4	Lateral Load Resistance. ....	21
8.2.5	Footing Observations. ....	21
8.3	Load Factors for Ultimate Design. ....	22
8.4	Floor Slabs. ....	22
IX.	CORROSION POTENTIAL. ....	23
X.	PLAN REVIEW, OBSERVATIONS AND TESTING. ....	23
XI.	LIMITATIONS. ....	23
	REFERENCES. ....	25
	LIST OF TABLES	
TABLE I	MCE MAPPED ACCELERATIONS. ....	6
TABLE II	FACTORS USED TO CONVERT SPECTRAL ACCELERATIONS OBTAINED FROM THE NGA RELATIONSHIPS TO THOSE CORRESPONDING TO MAXIMUM DIRECTION SPECTRA. ....	7
TABLE III	FAULT PARAMETERS USED FOR THE DETERMINISTIC ANALYSIS	8
TABLE IV	SITE-SPECIFIC DESIGN SPECTRAL RESPONSE ACCELERATIONS AS A FRACTION OF GRAVITATIONAL ACCELERATION (G). ....	10
TABLE V	Load Factors for Ultimate Design . ....	22

## LIST OF APPENDICES

### APPENDIX A

Figure A-1	Location Map
Figure A-2	Site and Boring Locations Plan
Figure A-3	Seismic Hazard Zones Map
Figure A-4	Probabilistic MCE Response Spectra
Figure A-5	Deterministic MCE Response Spectra
Figure A-6	Site-Specific MCE Response Spectra
Figure A-7	Site-Specific Design Response Spectra
Figure A-8	Flood Map

### APPENDIX B

Figure B-1	Explanation of Terms and Symbols
Figure B-2	Log of Boring B-1
Figure B-3	Log of Boring B-2

*GEOBASE, INC., November 2006 –*

Figure B-4	Log of Boring B-1
Figure B-5	Log of Boring B-2

### APPENDIX C

Figure C-1	Summary of Laboratory Test Results
Figure C-2	Consolidation Test Results
Figure C-3	Consolidation Test Results
Figure C-4	Consolidation Test Results
Figure C-5	Direct Shear Test
Figure C-6	Expansion Potential and Corrosivity Test Results
Figure C-7	Corrosivity Series Test Results by Anaheim Test Laboratory

*GEOBASE, INC., November 2006 --*

Figure C-8	Summary of Laboratory Tests
Figure C-9	Direct Shear Test Results
Figure C-10	Direct Shear Test Results
Figure C-11	Direct Shear Test Results
Figure C-12	Direct Shear Test Results
Figure C-13	Direct Shear Test Results
Figure C-14	Expansion Potential, Water Soluble, and Corrosivity Series Test Results
Figure C-15	Corrosivity Series Test Results by Anaheim Testing Laboratory

### APPENDIX D

Liquefaction and Seismic Settlement Analyses

## **I. INTRODUCTION**

GEOBASE, INC. (GEOBASE) was authorized by Ewing Cole to undertake a geotechnical investigation for the proposed Outpatient Pharmacy located at the Jerry L. Pettis Memorial Veterans Medical Center, 11201 Benton Street, in the City of Loma Linda, California. The site location is shown on the Site Location Map, Figure A-1, Appendix A.

For this geotechnical investigation, we were provided with a site plan prepared by EwingCole. This investigation was directed towards this plan which is reproduced herein as the Site and Boring Locations Plan, Figure A-2, Appendix A.

This report describes the site investigation and summarizes the results of both field and laboratory testing. The results of the field and laboratory tests are discussed with reference to the proposed development. Both general and specific recommendations pertinent to suitable site development and foundation design, respectively, are given. Construction guidelines related to the geotechnical aspects of the project are also addressed.

## **II. SITE AND PROJECT DESCRIPTIONS**

### **2.1 Site Description**

The Jerry L. Pettis Memorial Veterans Medical Center is bounded by Barton Road to the south, Benton Street to the west, Loma Linda Drive to the east and apartment buildings to the north, in the City of Loma Linda, California.

The proposed Outpatient Pharmacy is located near the northwest corner of the main hospital building. The site is currently occupied by landscape grass, occasional trees and is up to approximately five (5) feet above the existing parking lot to the west. In addition, nearby ponds were observed to the south and east of the proposed building site. Existing utilities including but not limited to electrical and irrigation lines are located within or adjacent to the proposed site.

### **2.2 Project Description**

The proposed construction is planned to consist of an at-grade structure, approximately 8000 square feet in plan area. This structure will be designed to accommodate four (4) storeys.

Column loads were not available at the time of writing this report.

### **III. SITE INVESTIGATION**

#### **3.1 Field Program**

The field investigation was carried out on April 28, 2012 and consisted of advancing two (2) borings at the site, at the approximate locations shown on the Site and Boring Locations Plan, Figure A-2, Appendix A. The borings were located in the field utilizing cloth tape. Therefore, the boring locations should be considered accurate only to the degree implied by the method used.

The borings were advanced to a maximum exploratory depth of seventy-one and one-half (71.5) feet utilizing a truck mounted CME-75 drill rig fitted with hollow stem augers. The Log of Borings together with an Explanation of Terms and Symbols used are given in Appendix B, Figures B-1 thru B-3, inclusive. The log of borings from a previous investigation at the site (GEOBASE, 2006) are also given in Appendix B, Figures B-4 and B-5.

Field testing consisted of the Standard Penetration Test (SPT). The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, two (2) inches outside diameter and one and three-eighths (1-3/8) inches inside diameter, is driven eighteen (18) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

Sampling consisted of:

- Collection of disturbed samples at selected locations retrieved from the auger;
- Collection of samples from the Standard Penetration Test (SPT) split spoon sampler; and,
- Collection of relatively undisturbed soil samples at selected locations using a California Modified Sampler. The soil samples were retained in a series of brass rings, each having an inside diameter of 2.41 inches and a height of one (1) inch. These ring samples were placed in close-fitting, moisture-tight containers for shipment to the laboratory.

#### **3.2 Laboratory Testing**

The samples obtained during the field program were returned to the laboratory for visual examination and

testing. The soils were classified in accordance with ASTM D 2487 and D 2488.

The laboratory testing program consisted of the following:

- Laboratory determination of water (moisture) content of soils, rock and soil-aggregate mixtures (ASTM D 2216) and dry density;
- Consolidation testing (ASTM D 2435);
- Direct Shear test of soils (ASTM D 3080);
- Test method for amount of materials in soils finer than No. 200 sieve (ASTM D 1140);
- Expansion potential of soils (ASTM D 4829); and,
- Corrosivity series tests (soluble sulfates, soluble chlorides, pH, and electrical resistivity) on soils (California Test 417 A).

The field and laboratory test results are presented on the Log of Borings, Figures B-2 thru B-5, inclusive, Appendix B, where applicable, and in Appendix C. In this respect, test results from the previous investigation (GEOBASE, 2006) are also included on the Log of Borings and in Appendix C.

#### **IV. SUBSURFACE CONDITIONS**

##### **4.1 Subsoils Conditions**

The site is currently occupied by landscape lawn with occasional trees and walkways adjacent to the site.

The generalized stratigraphic profile at the boring locations consist of up to eight (8) feet of fill soils (silty sands) overlying native silty sands with varying amount of fine gravels and sandy silts. The fill may be thicker at other locations, particularly at utility trench locations.

Based on the SPT test results at both boring locations, the upper twenty (20) feet of silty sands are inferred to be in a loose to very loose state. The underlying silty sands are inferred to be medium dense to a depth of approximately forty (40) feet below existing grade. Below forty (40) feet these soils are inferred to be in a medium dense to dense state and the sandy silts are inferred to have a firm to stiff consistency.



## 4.2 Groundwater

Groundwater was not observed within the seventy-one and one-half (71.5) foot depth of exploration, at both boring locations, during this site investigation and the previous site investigation (GEOBASE, 2006); however, groundwater conditions may be altered by geologic detail between borings, by seasonal and meteorological variations, and by construction activity.

The historical highest groundwater level in the general site area was reviewed based on well records by the California Department of Water Resources and USGS. The published groundwater data (CDWR, 2012) from groundwater Well No. 01S004W2SE007S, for a limited period (between 2003 and 2012), indicates that the highest groundwater level ranges from approximately thirty-seven (37) to forty-four (44) feet below existing ground surface. Due to limited and only recent groundwater data, the site being mapped to be in a liquefiable zone (City of Loma Linda General Plan, Public Health and Safety Element, 10.1, reproduced herein as Figure A-3, Appendix A) and the water ponds, immediately adjacent to the site, a historic highest groundwater level of ten (10) feet below ground surface is used for seismic design purposes.

## V. **SEISMICITY**

### 5.1 Site Coordinates

The site latitude and longitude are 34.0510 degrees north and 117.2510 degrees west, respectively.

### 5.2 Site Classification

Standard penetration testing was carried out at the boring locations and the results are presented on the log of borings, Appendix B. Based on the review and evaluation of these results, the average SPT blow counts within the top 100 feet are averaged to be greater than fifteen (15); however, liquefiable soils and/or soils subject to dry seismic settlements exist within the upper forty (40) feet below subgrade.

Based on discussions with the project architect, liquefaction mitigation within the upper forty (40) feet is the preferred alternative and appropriate recommendations are provided in Section VI.

Therefore, the subsoils at the site are judged to be Site Class D based on the average SPT blow counts within the top one hundred (100) feet and the fact that liquefaction will be mitigated.

### 5.3 Site-Specific Ground Motion Procedures - Ground Motion Hazard Analysis

Based on CBC 2010, subsection 1615.10.2, since the project site is located in a seismic hazard zone (Figure A-3, Appendix A), site-specific ground motion hazard analysis is required.

As part of ground motion hazard analysis (GMHA), probabilistic and deterministic spectral response accelerations corresponding to the Maximum Considered Earthquake (MCE) are determined. The MCE ground motions are defined as the maximum level of earthquake ground shaking that is considered as reasonable to design normal structures against collapse.

The site-specific MCE spectral response acceleration at any period is taken as the lesser of the spectral response accelerations obtained using the probabilistic and deterministic methods of GMHA. The design spectral response acceleration at any period is then determined as two-thirds (2/3) of the site-specific MCE spectral response acceleration. However, the design spectral response acceleration at any period should not be taken less than eighty (80) percent of the design spectral response acceleration determined from the general procedure, which is based on the mapped spectral response accelerations.

The CBC 2010 procedure includes:

- Determination of mapped parameters.
- Use of the Next Generation Attenuation (NGA) relationships in the calculation of the probabilistic and deterministic response spectra.
- Use of the 2008 USGS fault model in the seismic hazard evaluations.
- Use of the maximum rotated component of earthquake loading in the calculation of both the probabilistic and deterministic response spectra.
- Use of the eighty four (84) percentile values in the determination of characteristic earthquakes corresponding to the faults in the calculation of deterministic response spectra.

#### 5.3.1 *Mapped Accelerations Response Spectra (Mapped Parameters)*

Mapped MCE spectral response accelerations for 0.2 and 1.0 second periods are provided in maps published in the ASCE 7-05 and CBC 2010. These maps are prepared by the USGS and the California portion of the map was prepared jointly with the CGS. These maps use results of seismic hazard analyses from both probabilistic and deterministic procedures, and are applicable to site Class B and five (5) percent of critical

damping. The mapped site accelerations are adjusted for site class effects using parameters  $F_a$  and  $F_v$ , which are functions of site class and mapped site spectral accelerations.

Mapped spectral response parameters may also be obtained using computer programs that can determine these parameters for selected site coordinates. The computer program Seismic Hazard Curves and Uniform Hazard Response Spectra version 5.1.0 dated February 10, 2011 was used to obtain mapped parameters for the project site. This program is available on the USGS website.

The project site is Site Class D and therefore, coefficient values  $F_a$  and  $F_v$ , of 1.0 and 1.5, respectively, are obtained for the site. Mapped MCE accelerations obtained for the project site are summarized in Table I.

**TABLE I**  
**MCE MAPPED ACCELERATIONS**

PERIOD (SECONDS)	MCE MAPPED ACCELERATION PARAMETERS (g)	SITE CLASS D
		MCE ACCELERATIONS ADJUSTED FOR SITE CLASS EFFECTS (g)
PGA	0.699	0.699
0.2	$S_s$ : 1.748	1.748
1	$S_1$ : 0.607	0.911

Based on the above, the mapped spectral response accelerations, adjusted for site Class D,  $S_{MS}$  and  $S_{M1}$  are 1.748 g and 0.911 g, respectively.

### 5.3.2 Probabilistic MCE Spectra

The probabilistic MCE horizontal spectral response accelerations are taken as the spectral response accelerations represented by five (5) percent damped accelerations response spectra having two (2) percent probability of exceedence within a fifty (50) year period or, equivalently, a return period of 2,475 years.

The probabilistic seismic risk analysis is based on the premise that moderate to large earthquakes occur on mappable Quaternary faults and that the occurrence rate of earthquakes on each fault is proportional to the Quaternary fault-slip-rate. This analysis assumes that earthquakes are distributed uniformly and therefore does not consider when the last earthquake occurred on the fault. The length of rupture of the fault as a function of earthquake magnitude is accounted for, and ground motion estimates at a site are made using the magnitude of the earthquake and the closest distance from the site to the rupture zone. The probabilistic risk analysis has explicitly taken into account uncertainties associated with:

- The earthquake magnitude;
- The rupture length given magnitude;
- The location of rupture zone on the fault;
- The maximum possible magnitude of earthquakes; and,
- The acceleration at the site given magnitude of earthquake and distance from the rupture zone to the site.

Probabilistic seismic hazard analyses were performed using the computer program "2008 Interactive Deaggregations (Beta)" available on the USGS website. The 2008-update source and attenuation models of the NSHMP (Petersen and others, 2008) are used for the determination of the response spectra in this program. The program provides seismic-hazard deaggregations for the spectral periods: 0.0 s (PGA), 0.1 s, 0.2 s, 0.3 s, 0.5 s, 1.0 s, 2.0 s, 3.0 s, 4.0 s and 5.0 s.

For each of these periods, the program provides the average of response spectra obtained from the three NGA attenuation relationships recommended to be used by the CBC 2010 to evaluate the attenuation of earthquake energy with distance from the source. These NGA attenuation relationships are proposed by Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008). Since the CBC 2010 requires use of the maximum rotated horizontal component to be used in the analysis, the result obtained for each period from the aforementioned software is multiplied by the appropriate factor to convert it to that corresponding to the maximum rotated component. Table II presents the conversion factors used for the various periods as suggested by proposal SDPRG-1R4 (2009), Table I, page 35.

**TABLE II**  
**FACTORS USED TO CONVERT SPECTRAL ACCELERATIONS OBTAINED FROM THE NGA RELATIONSHIPS TO THOSE CORRESPONDING TO MAXIMUM DIRECTION SPECTRA**

Period (Seconds)	Factor
PGA	1.1
0.1	1.1
0.2	1.1
0.3	1.1
0.5	1.2
1.0	1.3
2.0	1.3
4.0+	1.4

The probabilistic spectral accelerations corresponding to the average spectra obtained from the aforementioned three attenuation relationships, and used for the determination of the site-specific MCE response spectra at the project site are shown in Figure A-4, Appendix A. In this respect, the site coordinates shown in Figure A-1, Appendix A were used.

Based on the results of standard penetration tests carried out at the site, and considering implementation of the recommended liquefaction mitigation, Section VI, a shear wave velocity of 250 m/s was used in the probabilistic seismic hazard analyses.

### 5.3.3 *Deterministic MCE Spectra*

The CBC 2010 specifies the deterministic MCE response acceleration at each period as the eighty fourth (84) percentile of the largest five (5) percent damped spectral response acceleration computed at that period for characteristic earthquakes on all known active faults within the region. The spectral accelerations should correspond to the maximum rotated component of ground motion; however, the ordinate of the deterministic MCE ground motion response spectrum should not be taken less than the corresponding ordinate of a lower limit response spectrum curve determined as a function of the coefficients  $F_a$  and  $F_v$ , assuming that the values of  $S_s$  and  $S_1$  are 1.5 and 0.6, respectively.

The faults that have the largest influence on the site seismicity are the San Jacinto - SBV+SJV+A+CC+B+SM, San Jacinto - SJV+A+CC+B+SM and San Andreas - PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO . For the project site coordinates, provided in Figure A-1, Appendix A, a search was carried out using the USGS/CGS 2008 Fault Model data base, and the faults with characteristic that produce the strongest earthquakes at the project site were selected. Names of these faults and their corresponding parameters are provided in Table III.

**TABLE III**  
**FAULT PARAMETERS USED FOR THE DETERMINISTIC ANALYSIS**

FAULT NAME	DISTANCE FROM SITE (KM)	HANKS MAGNITUDE (Mw)	FAULT TYPE	PREFERRED DIP (DEGREE)	RUPTURE TOP (KM)
San Jacinto	1.54	7.88	SS	90	0.1
San Jacinto	3.96	7.76	SS	90	0.1
San Andreas	11.12	8.18	SS	86	0.1

Peak ground accelerations and response spectra corresponding to the characteristic earthquake for each of the faults were determined using the average of the three (3) attenuation relationships discussed in subsection

5.3.2 and recommended for use by the CBC 2010. The Microsoft Excel spreadsheet prepared by L. Atiq, available at the website: [http://peer.berkeley.edu/ngawest/rep\\_nqa\\_models.htm](http://peer.berkeley.edu/ngawest/rep_nqa_models.htm), was used to obtain the response spectra corresponding to the characteristic earthquakes. Using this spreadsheet, the eighty four percentile (sigma plus one standard deviation) values were selected. These values were then multiplied by the factors shown in Table II to convert them to values corresponding to the maximum rotated component. As noted previously, a shear wave velocity of 250 m/s was used in the determination of characteristic earthquakes for each of the faults.

Figure A-5 Appendix A, shows spectral response accelerations of the characteristic earthquakes, which correspond to the specified MCE accelerations. This figure also shows the specified lower limits of the MCE spectral accelerations, obtained as described in the ASCE 7-05 standard.

By comparing the ordinates of the specified MCE spectral response accelerations from the faults governing maximum ground motions at the site with the corresponding ordinates from the specified lower limits of the acceleration response spectra curve, the response spectra from the deterministic method were obtained and are also shown in Figure A-5, Appendix A.

#### 5.3.4 *Site-Specific MCE Response Spectra*

The site-specific MCE spectral response acceleration at any period is taken as the lesser of the spectral response accelerations obtained from the probabilistic and deterministic methods. The MCE probabilistic and deterministic spectra obtained as described in subsections 5.3.2 and 5.3.3, respectively, are presented in Figure A-6, Appendix A. The site-specific MCE spectra defined as the lesser of the probabilistic and deterministic spectra is also shown in Figure A-6, Appendix A.

#### 5.3.5 *Design Response Spectra Based on Mapped Parameters*

Section 11.4.5 of ASCE 7-05 describes a procedure to obtain a design response spectra curve for use in cases where a design response spectrum is required by the ASCE 7-05 standard, and site-specific ground motion procedures are not used. This procedure is based on the use of the mapped spectral response accelerations adjusted for site class effects, in the determination of the design response spectrum curve. Site-specific design spectra for the site should not be taken lower than eighty (80) percent of the response spectra obtained from the mapped parameters.

### 5.3.6 Site-Specific Design Spectra

The ASCE 7-05 specifies the design spectral response acceleration at any period as two-thirds (2/3) of the site specific MCE spectral response acceleration. However, the design spectral response acceleration at any period should not be taken less than eighty (80) percent of the design spectral response acceleration determined using the mapped parameters for the site.

The site-specific design response spectra based on two-thirds (2/3) of site-specific MCE spectral response accelerations, together with the response spectra curve obtained as eighty (80) percent of the spectra based on mapped parameters for the project site are shown in Figure A-7, Appendix A. The site-specific design response spectra curve for the project site is also shown in Figure A-7, Appendix A, as the greater of the two spectra curves. Numerical values of the site-specific design spectral response accelerations for the project site are provided in Table IV.

**TABLE IV**  
**SITE-SPECIFIC DESIGN SPECTRAL RESPONSE ACCELERATIONS AS A FRACTION OF GRAVITATIONAL**  
**ACCELERATION (G)**

Period (Seconds)	Site-specific Design Spectral Response Acceleration (g)
PGA	0.74
0.02	0.83
0.05	0.97
0.075	1.00
0.1	1.00
0.20	1.10
0.30	1.24
0.40	1.31
0.50	1.39
0.75	1.41
1.00	1.30
1.50	1.11
2.00	0.93
3.00	0.73
4.00	0.54
5.00	0.44

### 5.3.7 Design Acceleration Parameters

The CBC 2010 specifies the design response spectrum at short period,  $S_{DS}$ , as the design spectrum at the

period of 0.2 second; however, this value should not be less than ninety (90) percent of the design spectra obtained at any period more than 0.2 second. Also, the CBC 2010 specifies  $S_{D1}$  as the greater of the design response spectrum at one second and twice the spectrum at two seconds.

Based on the above, and the values of site-specific design response spectra provided in Table IV, the design acceleration parameters are obtained as follows:

$$S_{DS} = 1.27g$$
$$S_{D1} = 1.86g$$
$$PGA = 0.74g$$

Based on the CBC 2010, a  $PGA = 0.74g$  is used for the liquefaction and seismic settlement analyses.

#### 5.4 Seismic Hazard Deaggregation

Relative contributions of various combinations of earthquake magnitudes and distances to a particular seismic hazard at a site are determined using deaggregation of the seismic hazards. Magnitude-distance deaggregation obtained from the computer program "2008 Interactive Deaggregations (Beta)" available on the USGS website, indicates that the deaggregated modal magnitude and distance for the peak ground acceleration at the project site are 7.00 and 2.3 kilometers, respectively. A magnitude of 7.00 was therefore used in the liquefaction and seismic settlement analyses for the project site.

#### 5.5 Seismic Design Category

The mapped spectral response acceleration parameter at one (1) second period ( $S_1$ ) is 0.607g which is less than 0.75g. The design spectral response acceleration coefficients  $S_{DS}$  and  $S_{D1}$  are 1.27 and 1.86, respectively. Therefore, a seismic design category D should be used for the design of the Outpatient Pharmacy per Section 1613.5.6 of CBC 2010.

#### 5.6 Earthquake Effects

##### 5.6.1 *Liquefaction*

Liquefaction occurs when the pore water pressures generated within a soil mass become near or equal to the overburden pressure. This results in a loss of strength and the soil then possesses a certain degree of mobility.



Factors considered to evaluate liquefaction potential include groundwater conditions, soil type, particle size distribution, earthquake magnitude and acceleration, and soil density obtained through the Standard Penetration Test (SPT). Soils subject to liquefaction comprise saturated fine grained sands to coarse silts. Coarser-grained soils are considered free-draining and therefore dissipate excess pore pressures, while fine-grained soils possess undrained shear strength.

Geologic Hazards Map from the City of Loma Linda General Plan, Public Health and Safety Element, adopted May 26, 2009 indicates that the project site is located within an area where historic occurrences of liquefaction, or local geological, geotechnical and groundwater conditions would indicate a potential for permanent ground displacement due to liquefaction (Figure A-3, Appendix A).

Liquefaction analyses were conducted using the recommendations of the NCEER, summarized by Youd, et al. (2001), and the soil stratigraphy as obtained from Borings B-1 and B-2 drilled during the current site investigation. For these analyses, a PGA of 0.74g based on the site-specific GMHA results described in subsection 5.3 and an earthquake magnitude of 7.00 was used based on the deaggregation results discussed in subsection 5.4. Liquefaction potential for fine-grained soils was determined using the criteria recommended by Seed et al. (2003). A historic high groundwater depth of ten (10) feet was used in the analyses. These results indicate that the factor of safety of the subsoils for the occurrence of liquefaction is less than 1.3 at the aforementioned boring locations; seismic settlements in the saturated and dry layers of the soils as a result of ground shaking will occur as described in subsection 5.6.2, below.

The results of the liquefaction analyses are presented in Appendix D.

#### 5.6.2 *Seismically Induced Settlements*

Based on an examination of the subsoils conditions, seismic settlement analyses were conducted for borings B-1 and B-2 at the proposed location of the Outpatient Pharmacy. For these analyses, a PGA of 0.74g based on the site-specific GMHA results described in subsection 5.3, and an earthquake magnitude of 7.00 was used based on the deaggregation results as discussed in subsection 5.4. Seismic settlements for the saturated sands were estimated using the Ishihara and Yoshimine (1992) Method and for the unsaturated sands using the Tokimatsu and Seed (1987) Method.

The results of the seismic settlement analyses are provided in Appendix D.

Based on our evaluation of the analyses results, seismic settlement at the site for borings B-1 and B-2 are in the order of nine (9) and thirteen (13) inches, respectively.

### 5.6.3 *Tsunami/Seiche, Inundation and Flooding*

The property is far and high enough from the coast or large inland body of water to preclude damage from a tsunami or seiche wave. The site is located in an area determined to be outside the 500 year flood plain, as defined by the City of Loma Linda General Plan, Public Health and Safety Element, adopted May 26, 2006, (Figure A-8, Appendix A).

### 5.6.4 *Surface Rupture*

The site is not located within any of the Alquist-Priolo Earthquake Fault Zone. The likelihood of direct surface fault rupture at the site is considered very low based on the presently known tectonic framework. Cracking due to shaking from distant events is not considered a significant hazard, although it is a possibility at any site.

### 5.6.5 *Landsliding*

The site lies far enough from the nearest significant upland slopes to preclude the hazards of induced landsliding.

### 5.6.6 *Lateral Spreading*

Seismically induced lateral spreading involves primarily movement of earth materials due to ground shaking. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography of the project site area is relatively flat and liquefaction will be mitigated. Therefore, the potential for lateral spreading at the subject site is considered low.

## **VI. LIQUEFACTION/SEISMIC SETTLEMENTS MITIGATION**

### 6.1 Introduction

The California Geological Survey (CGS) guidelines for evaluating and mitigating seismic hazards in California (CGS, 2008) were adhered to.

Based on the analyses presented in Appendix D and discussed in subsections 5.6.1 and 5.6.2, the subsoils at the site are potentially liquefiable to a depth of approximately forty (40) feet, and the associated seismic settlements are in the order of nine (9) to thirteen (13) inches. The aforementioned analyses assume a Site Class D since mitigation of the upper forty (40) feet of the subsoils will be implemented. In this respect, without

mitigation, analyses based on a Site Class F would result in greater settlements. Therefore, mitigation alternatives were considered and are discussed in the following subsections.

## 6.2 Mitigation Alternatives

Suitable mitigation alternatives may include:

- In-situ soil densification or other types of ground modifications;
- Excavation, and removal or recompaction of potentially liquefiable soils;
- Deep foundations designed to accommodate liquefaction effects; and,
- Reinforced shallow and improved structural design to withstand the predicted settlements.

## 6.3 Evaluation of Mitigation Alternatives

### 6.3.1 *Discussions*

The reinforced shallow foundations were not considered appropriate in light of the magnitude of the predicted settlements.

Most of the structures at the site are founded on piles in the order of seventy (70) feet long. Based on the SPT results at the boring locations, allowable downward capacity of a seventy (70) foot long, thirty-six (36) inch diameter drilled cast-in-place concrete pile would be in the order of 150 kips. This alternative, based on CBC 2010 would require the site to be classified Site Class F and a site response analysis would have to be completed. This alternative was discussed with the structural engineer and was not considered.

Excavation, and removal and replacement with compacted soils would extend to approximately forty (40) feet below existing grade. Due to the thickness of fill, settlement monitoring is recommended to verify that preliminary consolidation is complete prior to the start of construction of the foundations.

In-situ soil densification and other types of ground modifications were evaluated. Based on the soil types observed at the boring locations and the fines content of the subsoils determined in the laboratory, wet soil mixing is considered appropriate to mitigate liquefaction at the subject site. Vibro-stone columns may not be effective due to fines content of the subsoils, in excess of thirty (30) percent. Jet grouting may also be considered since it can be effective across a wide range of soil types.

### 6.3.2 *Conclusions*

Wet soil mixing is considered most appropriate for the subject site. Description of the procedure and acceptance criteria are provided in the following subsection.

Alternative methods of ground improvement are considered acceptable provided that: the acceptance criteria for wet soil mixing, presented in subsection 6.4 can be met; and, seismic settlement can be limited to one (1) inch using the Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) methods for settlement calculations.

### 6.4 Wet Soil Mixing

Wet soil mixing is a ground improvement procedure which consists of the mechanical blending of the in-situ soil with a cement-based slurry. This is achieved by pumping the slurry through the hollow stem of the shaft and injecting it into the soil through nozzles located on the backside of the leading rotating mixing blades. Injection is continuous until the target depth is reached, and also continues as the tool is withdrawn. Spoils (excess soil), in the order to twenty (20) to forty (40) percent of the volume treated, will have to be disposed of/exported.

It is the Soil Mixing Contractor's responsibility to determine and implement the systems required to achieve the performance criteria outlined below. In this respect, the Soil Mixing Contractor may require additional geotechnical investigation. Further, the Soil Mixing Contractor should submit with the bid documents: soil-mix design; a work procedures plan; and, a quality control plan. The aforementioned should be forwarded to the geotechnical engineer for review.

For the proposed Outpatient Pharmacy, the stabilized soil-mix columns should be clustered to provide ground improvement to a depth of approximately forty (40) feet over the entire building area. This area should extend a minimum of fifteen (15) feet beyond the building limits.

The soil mixed shall be considered acceptable if the following criteria are met:

- Demonstration that for an allowable dead-plus-live load soil bearing capacity of 4,000 pound per square foot, the total settlement will not exceed one (1) inch;
- For the soil mix, an average unconfined compressive strength greater than 150 psi with no more than ten (10) percent less than 50 psi.

- Core recovery greater than eighty-five (85) percent at the location of borings with continuous cores.

At areas not meeting the acceptable criteria listed above should be remediated at the contractor's own cost.

Prior to production of soil-mix columns installation, an indicator soil mixing program should be performed. We recommend that a minimum of three (3) soil mixing points be installed, with the same equipment used for production soil mixing, and establish/finalize the acceptance criteria. A minimum of one (1) boring with continuous cores should be performed within the treated depths in the center of the indicator area treated. Core samples should be obtained within the treated depths to evaluate unconfined compressive strengths.

During production, at least one (1) verification boring with continuous cores should be performed within the treated depths for every 3,000 square feet of production area to verify the unconfined compressive strength. The number and location of the borings should be selected by GEOBASE.

In addition to the above, four (4) sets of cylinders of the soil-mix, per day and per rig, should be obtained to verify the unconfined compressive strength.

Full time observation of both indicator and production soil mixing installation should be carried out by GEOBASE.

## **VII. SITE DEVELOPMENT RECOMMENDATIONS**

### **7.1 General**

The proposed development, described in subsection 2.2, is feasible from a geotechnical engineering standpoint. Project plans and specifications should take into account the appropriate geotechnical features of the site and conform to the geotechnical recommendations.

### **7.2 Clearing**

All surface vegetation, tree roots, asphaltic concrete, trash, debris and underground pipes, if encountered, should be cleared and removed from the proposed site. Topsoil, is not considered suitable for reuse as structural fill, but may be stockpiled for future use.

Underground facilities such as utilities, pipes or underground storage tanks may exist at the site. Removal of underground tanks is subject to state law as regulated by County or City Health and/or Fire Department

agencies. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects.

Septic tanks should be removed in their entirety. Cesspools or seepage pits should be pumped of their contents and backfilled with a two-sack sand-cement slurry.

Any water wells, if encountered during construction, should be exposed and capped in accordance with the requirements of the regulating agencies.

Depressions resulting from the removal of buried obstructions, building foundations and pipes should be backfilled with properly compacted material.

### 7.3 Subgrade Preparation

Prior to ground improvement as recommended in subsection 6.4, undocumented fill soils within the building pad, should be removed and replaced as properly compacted fill. The lateral extent of removal beyond the building limits should be at least equal to the depth of fill beneath the proposed floor slab level or a minimum of five (5) feet beyond the building limits. Undocumented fill soils of up to eight (8) feet in thickness were encountered at the boring locations, at existing grade elevation. Thicker fills may be encountered away from the boring locations, particularly at utility trenches. If undocumented fills are observed to extend deeper at other locations during construction, they should be removed and replaced as properly compacted fill.

Notwithstanding the above, in order to provide a uniform bearing surface for floor slabs, subsequent to ground improvement, the building pad areas should be overexcavated a minimum of three (3) feet below the slab subgrade elevation to provide a uniform compacted fill blanket.

The subsoils beneath the walkways, driveways and patios areas should be over-excavated a minimum of two (2) feet to facilitate construction of a two(2) foot thick compacted fill blanket. The lateral extent of over-excavation beyond the aforementioned areas should be at least equal to the depth of fill. The aforementioned partial removal and recompaction should only be considered if future maintenance as a result of possible excessive settlement of the underlying undocumented fills can be tolerated. Alternatively, one option to mitigate the potential adverse effects of the underlying undocumented fills is to remove and replace all the undocumented fills.

Construction activities and exposure to the environment can cause deterioration of the subgrade. Therefore, it is recommended that the condition of the subgrade soils be observed and/or tested by GEOBASE

immediately prior to slab-on-grade construction.

#### 7.4 Fill Placement

##### 7.4.1 *Preparation of Surface Soils*

Prior to placing any fill, the exposed surface soils should be scarified to a minimum depth of six (6) to eight (8) inches, moisture-conditioned (wetted or dried), and compacted to a minimum of ninety (90) percent relative compaction, based on ASTM D 1557.

##### 7.4.2 *Compaction*

Cohesive soils should be placed in loose lifts not exceeding six (6) inches, moisture-conditioned (wetted or dried) to approximately two (2) to four (4) percentage points above optimum, and compacted to a minimum of ninety (90) percent relative compaction (ASTM D1557).

Granular fill materials should be placed in loose lifts of six (6) to eight (8) inches, moisture-conditioned (wetted or dried) to near optimum, and compacted to a minimum of ninety (90) percent relative compaction (ASTM D1557).

##### 7.4.3 *Fill Material*

The on-site soils have a "very low" expansion potential (Expansion Index = 10). The soils may be reused as compacted fill provided they are free of organics, deleterious materials, debris and particles over six (6) inches in largest dimension.

Any soils imported to the site for use as fill for subgrade materials should be predominantly granular, non-expansive (Expansion Index less than twenty [20]) and should contain approximately twenty (20) percent fines. The imported soils should be approved by GEOBASE prior to importing. Laboratory testing required for approval of import sources may require forty eight (48) hours. GEOBASE should be notified of import locations a minimum of seventy two (72) hours prior to its proposed use.

#### 7.5 Drainage

To enhance future site performance, it is recommended that all pad drainage be collected and directed away from proposed structures to disposal areas off-site. For soils areas, we recommend that a minimum of five (5)

percent gradient away from foundation elements be maintained. All roof drains should be connected to solid pipes discharging to the curb or other suitable area drains. It is important that drainage be directed away from foundations and that proper drainage patterns be established at the time of construction and maintained throughout the life of the structures.

Landscape areas within ten (10) feet of the building perimeter should consist of planters that have sealed bottoms and bottom drains to prevent infiltration of water into the adjacent foundation soils. The surface of the ground in these areas should also be maintained at a minimum gradient of five (5) percent towards surface area drains.

Care should be exercised in controlling surface runoff onto slopes. The area back of the slope crest should be graded such that water will not be allowed to flow freely onto the slope face. If excavations of temporary slopes are carried out in the rainy season, appropriate erosion protection measures may be required to minimize erosion of the slope cuts.

#### 7.6 Temporary Excavations

Temporary construction excavations are anticipated for construction of utility trenches and footings.

Temporary construction excavations in soils may be made vertically without shoring to a depth of approximately four (4) feet below adjacent surrounding grade. For deeper cuts in soils, the slopes should be properly shored or sloped back at least 1H:1V (Horizontal:Vertical) or flatter. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the toe of excavation unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at forty-five (45) degrees below the edge of any nearby adjacent existing site facilities, including foundations of existing buildings and underground pipelines, should be properly shored to maintain foundation support of the adjacent structures and utilities.

All excavations and shoring systems should meet, as a minimum, the requirements given in the State of California Occupational Safety and Health Standards. Stability of temporary slopes are the responsibility of the contractor.

#### 7.7 Trench Backfill

It is our opinion that utility trench backfill could be placed and compacted by mechanical means. Jetting or flooding of backfill material is not recommended.



If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, other methods of utility trench compaction may also be appropriate, as approved by the geotechnical engineer at the time of construction.

## **VIII. FOUNDATION RECOMMENDATIONS**

### **8.1 General**

The following recommendations have been formulated from visual, physical and analytical considerations of existing site conditions and are believed to be applicable for the proposed development.

Based on the results of field and laboratory tests, and subsequent analyses, the proposed structure may be founded on footings subsequent to ground improvement by wet soil mixing, as described in subsection 6.4.

The on-site soils are considered to have a "very low" expansion potential. The recommendations presented in the following subsections are based on a "very low" expansion potential for the subgrade soils. The potential for favorable foundation performance can be further enhanced by maintaining uniform moisture conditions. Foundations and slab reinforcement configurations should meet, as a minimum, the requirements of the governing agencies or the California Building Code.

### **8.2 Footings**

Spread or continuous footings, based on the improved subsoils, may be used to support the proposed four (4) storey structure. Footing bottoms should be compacted to a minimum of ninety-five (95) percent relative compaction and the footings should be based a minimum of thirty-six (36) inches below the lowest adjacent grade.

Planters adjacent to footings should be avoided unless they are lined and landscape water is allowed to drain to a suitable discharge point.

#### **8.2.1 *Soil Bearing Pressures***

Spread or continuous footings may be designed for an allowable dead-plus-live load bearing pressure of 4,000 psf. This allowable bearing pressure may be increased by one-third (1/3) for short-term wind or seismic loads. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed the above-mentioned allowable bearing values.

Footings placed closer than one (1) width apart should be structurally tied.

#### 8.2.2 *Footings Adjacent to Trenches or Existing Footings*

Where footings are located adjacent to utility trenches, they should extend below a one-to-one plane projected upward from the inside bottom corner of the trench. Footing excavations adjacent to the footings of existing buildings should be carried out such that the existing footings are not undermined.

#### 8.2.3 *Settlement*

For an allowable dead-plus-live load bearing pressure of 4,000 psf, total and differential settlements of footings are not anticipated to exceed one (1) inch and one-half (½) inch, respectively. The aforementioned differential settlement may occur between adjacent, similarly loaded columns or over a distance of thirty (30) feet for continuous footings.

Seismic settlements at footing base elevation are estimated to be negligible subsequent to ground improvement by wet soil mixing.

#### 8.2.4 *Lateral Load Resistance*

Lateral loads (wind or seismic) against structures may be resisted by friction between the bottom of foundations and the supporting soils. An allowable friction coefficient of 0.3 is recommended. An allowable lateral bearing pressure equal to an equivalent fluid weight of 200 pounds per cubic foot acting against the foundations may also be used, provided the foundations are poured tight against improved soil or compacted fill.

The frictional resistance and lateral resistance of the soils can be combined without reduction in determining the total lateral resistance.

#### 8.2.5 *Footing Observations*

All foundation excavations should be observed by GEOBASE prior to the placement of forms, reinforcement, or concrete, for verification of conformance with the intent of these recommendations and confirmation of the bearing capacities. All loose or unsuitable material should be removed and footing bottoms should be compacted to a minimum of ninety-five (95) percent relative compaction prior to the placement of reinforcement and concrete. Materials from footing excavations should not be spread in slab-on-grade areas unless compacted.

### 8.3 Load Factors for Ultimate Design

The recommended design values presented in this report are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the factors given in the table below.

**TABLE V**  
**LOAD FACTORS FOR ULTIMATE DESIGN**

Bearing Value (without increase)	3
Passive Pressure	1.33
Coefficient of Friction	1.25

In no event, however, should the foundation sizes be reduced from those required for support of the dead-plus-live loads when using working stress values.

### 8.4 Floor Slabs

Concrete slab-on-grade floors may be used for the proposed structure. The subgrade for the floor slab should be prepared in accordance with the recommendations provided in subsections 7.3 and 7.4.

The slab should be underlain by a minimum of six (6) inches of gravel or sand base. In moisture-sensitive areas, the slab should be damp-proofed per Section 1805.2.1 of CBC 2010.

Thickness of floor slabs should be at least five (5) inches actual, and the slab should be designed and reinforced by the structural engineer for temperature and shrinkage stresses, and the project loading and service conditions. A subgrade modulus of 150 pound per cubic inch may be used for slab design.

The recommendations presented herein are intended to reduce the potential for cracking of slabs and foundations; however, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints, and proper concrete placement and curing. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction and curing practices, and should be incorporated into project construction. Further, to control the location and spread of concrete shrinkage cracks, crack control joints should be provided by the structural engineer, taking into consideration criteria of the ACI when establishing crack control patterns.

## **IX. CORROSION POTENTIAL**

Electrical resistivity, pH, chloride and water soluble sulfate tests were conducted on representative samples by Anaheim Test Laboratory and the results are provided in Figure C-7, Appendix C. The test results indicate that the subsoils have a "low" corrosive potential with respect to concrete and a "severe" corrosion potential with respect to steel (Figure C-14). Type I or II Portland cement may be used for the construction of concrete structures in contact with the subgrade soils. Protection of steel against corrosion will be required for steel structures placed in contact with the subsoils at the site.

## **X. PLAN REVIEW, OBSERVATIONS AND TESTING**

Post-investigation services are an important and integrated part of this investigation and should be carried out by GEOBASE. The project plans and specifications should be forwarded to GEOBASE for review for conformance with the intent of the soils recommendations.

Geotechnical observations of exposed excavation bases should be carried out prior to fill placement. Observations and testing of all fill placement should be carried out on a continuous basis to verify the design assumptions and conformance with the intent of the recommendations. Observations and testing of footing bases should be carried out prior to concrete pour to confirm the bearing capacities.

## **XI. LIMITATIONS**

This investigation was performed in accordance with generally accepted geotechnical engineering principles and practices. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is intended for use by the client and its representatives, and with regard to the specific project discussed herein. Any changes in the design or location of the proposed new structure, however slight, should be brought to our attention so that we may determine how they may affect our conclusions. The conclusions and recommendations contained in this report are based on the data relating only to the specific project and location discussed herein. This report does not relate any conclusions or recommendations about the potential for hazardous and/or contaminated materials existing at the site.

The analyses and recommendations submitted in this report are based upon the observations noted during drilling of the borings, interpretation of laboratory test results, and geological evidence. This report does not reflect any variations which may occur away from the borings and which may be encountered during

construction. If conditions observed during construction are at variance with the preliminary findings, we should be notified so that we may modify our conclusions and recommendations, or provide alternate recommendations, if necessary.

The recommendations presented herein assume that the plan review, observations and testing services, outlined in Section XI of the report, will be provided by GEOBASE. During execution of the aforementioned services, GEOBASE can finalize the report recommendations based on observations of actual subsurface conditions evident during construction. GEOBASE cannot assume liability for the adequacy of the recommendations if another party is retained to observe construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project, and incorporated into the plans and specifications. In this respect, it is recommended that we be allowed the opportunity to review the project plans and the specifications for conformance with the geotechnical recommendations.

This office does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for other than our own personnel on the site. Therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein to be unsafe.

This report may be subject to review by the appropriate regulating agencies.

Respectfully submitted  
GEOBASE, INC.



H. D. Nguyen, B.Sc.  
Project Engineer



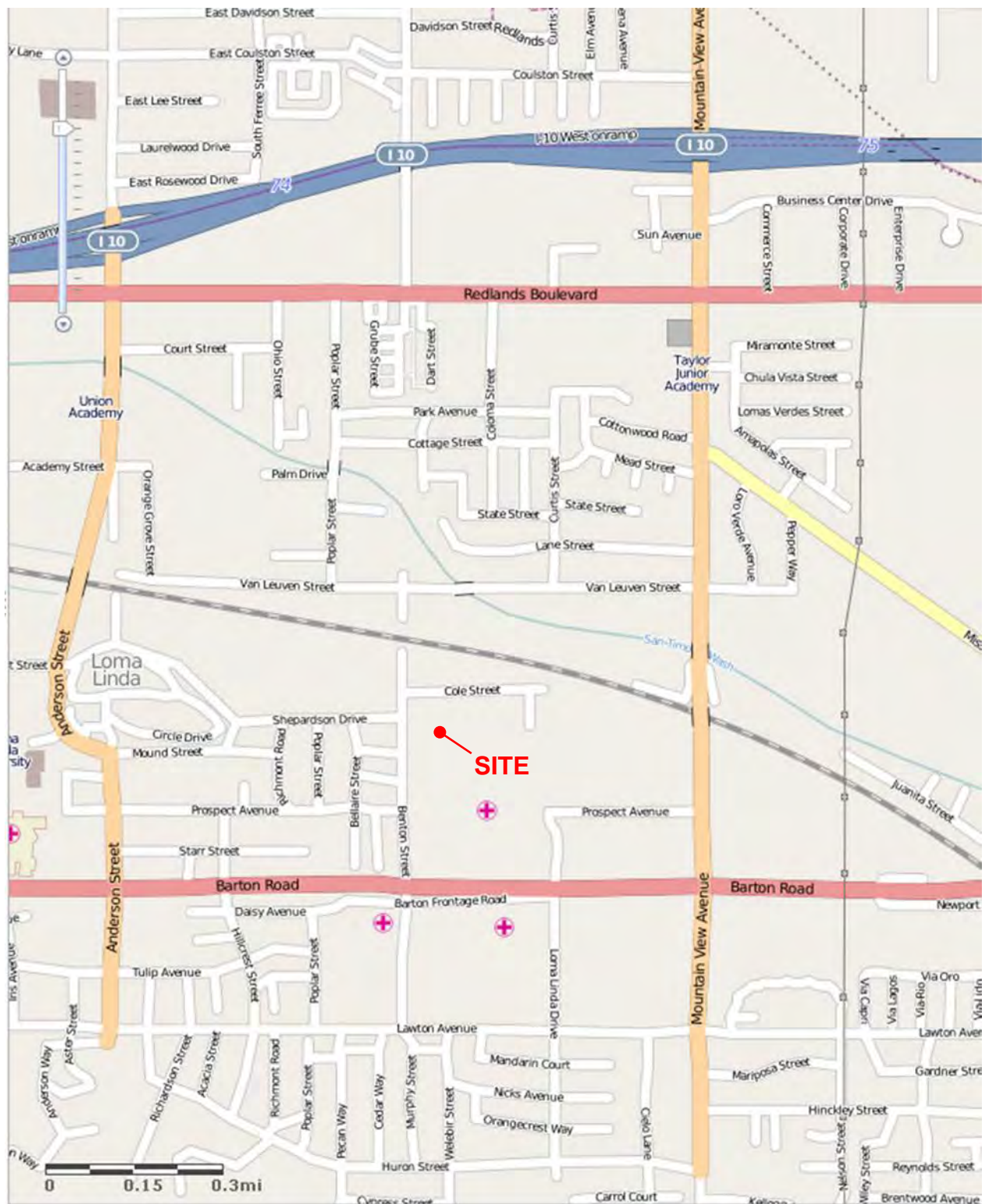
J-M. Chevallier, P.E., G.E.  
R.C.E. 39198; G.E. 2056  
Managing Principal

## REFERENCES

- American Society of Civil Engineers, 2005 "Minimum Design Loads for Buildings and Other Structures", ASCE Standard, ASCE/SEI 7-05.
- Boore, D.M. and Atkinson, G.M. (2007), Boore-Atkinson NGA Ground Motion Relations for the Geometric Mean Horizontal Component of Peak and Spectral Ground Motion Parameters, PEER 2007/01, Pacific Earthquake Engineering Research Center, Berkeley, California.
- California Division of Mines and Geology, January 17, 2001, Seismic Hazard Zone Map for the Tustin 7.5-MIN Quadrangle, Orange County, California.
- California Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the Tustin 7.5-MIN Quadrangle, Orange County, California: CDMG, Seismic Hazard Zone Report 012.
- Campbell, K. and Bozorgnia, J., 2008. NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD, and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s, Earthquake Spectra, v 24. pp. 139-172.
- CGS, 2008, "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California", Revised and Re-Adopted September 11, 2008 by the California Geologic Survey in Accordance with the Seismic Hazards Mapping Act of 1990.
- Chiou, B. and Youngs, R., 2008. An NGA model for the average horizontal component of peak ground motion and response spectra, Earthquake Spectra, v 24. pp. 173-216.
- Day, S., R. Graves, J. Bielak, D. Dreger, S. Larsen, K. Olsen, A. Pitarka, and L. Petersen, M. and others, 2008. Documentation for the 2008 update of the national seismic hazard maps, USGS OFR 08-1128. Available on the web at <http://pubs.usgs.gov/of/2008/1128/>.
- Federal Emergency Management Agency (FEMA), Orange County, California and Incorporated Areas, Flood Insurance Rate Map, Map Number 06059C0279J, effective date December 3, 2009.
- GEOBASE, INC., November 2006, "Geotechnical Evaluation for Parking Structure, VA Medical Center, Loma Linda, California", prepared for VA Loma Linda Healthcare System, Loma Linda, California, project number C.322.01.00.
- Ishihara, K. and Yoshimine, M., 1992. Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 32, No. 1, March 1992.
- Proposal SDPRG-1R4 (2009). Proposal for changes. 3rd Member Organization Ballot (December, 2008).
- Tokimatsu, K., and Seed, H.B., 1987 "Evaluation of Settlements in Sands due to Earthquake Shaking", J. Geotechnical Engineering Division, ASCE, Vol. 113, No. 8, pages 861 to 878
- Youd, T.L., et al., 2001. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 127, No. 10, Pages 817 to 833.

## **APPENDIX A**

Figure A-1	Location Map
Figure A-2	Site and Boring Locations Plan
Figure A-3	Seismic Hazard Zones Map
Figure A-4	Probabilistic MCE Response Spectra
Figure A-5	Deterministic MCE Response Spectra
Figure A-6	Site-Specific MCE Response Spectra
Figure A-7	Site-Specific Design Response Spectra
Figure A-8	Flood Map



**Site Coordinates:**

Lat: 34.051° N

Lon: 117.251° W

# GEOBASE

## LOCATION MAP

JERRY L. PETTIS MEMORIAL

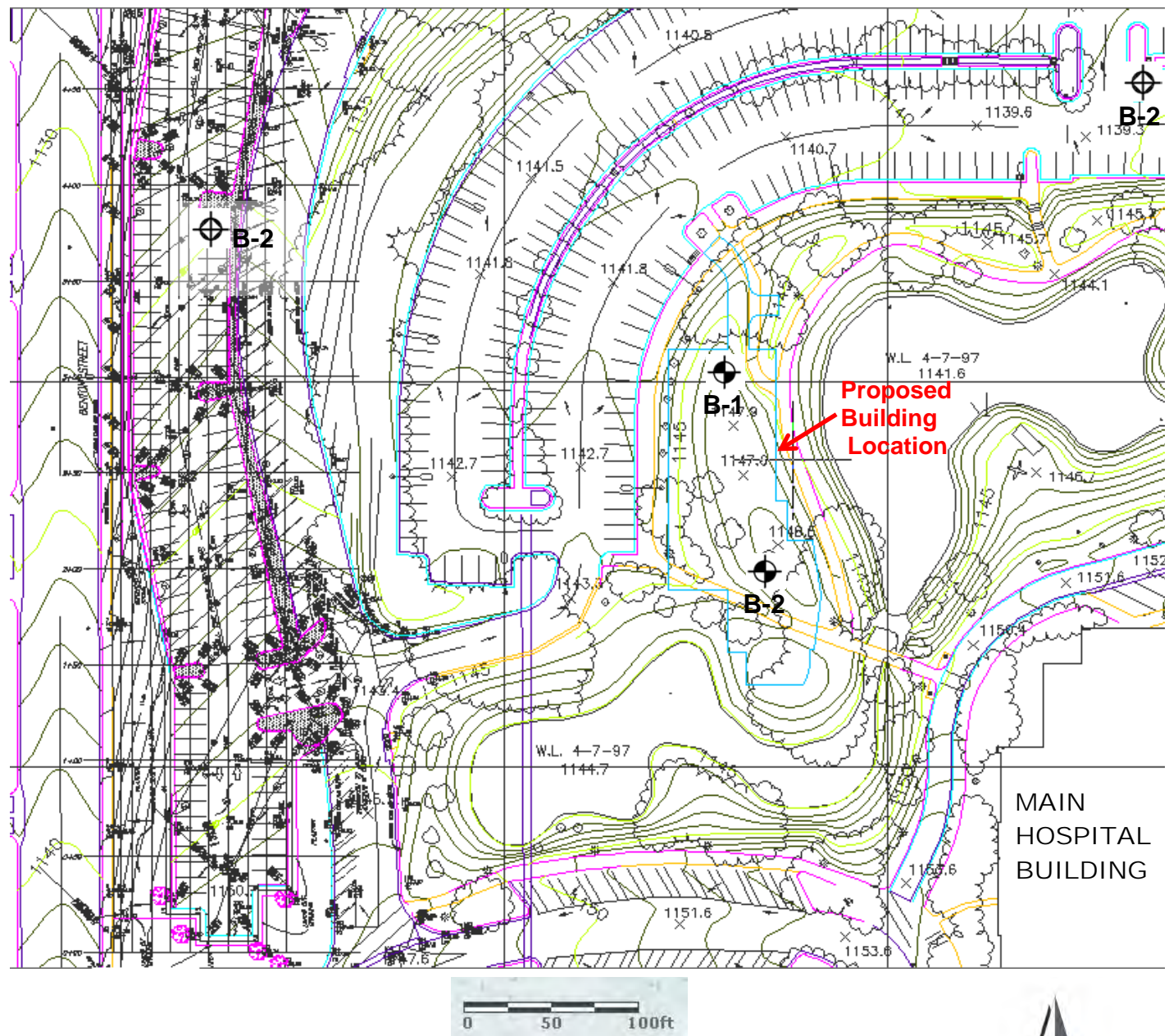
VETERANS MEDICAL CENTER – OP PHARMACY

LOMA LINDA, CALIFORNIA



C.329.03.00

FIGURE A-1





#### EXPLANATIONS:

-  Approximate Boring Locations
-  Approximate Boring Locations (GEOBASE, 2006)

#### NOTES:

1. GEOBASE INC. has added only Geotechnical data to this plan prepared by others. We have not checked any other information on this plan and give no assurance of its accuracy.
2. This Drawing is part of GEOBASE INC.'s report C.329.03.00 dated June 06, 2012 and should be read with the complete report for evaluation.

# GEOBASE

**SITE AND BORING LOCATIONS PLAN**  
**JERRY L. PETTIS MEMORIAL**  
**VETERANS MEDICAL CENTER – OP PHARMACY**  
**C.329.03.00 LOMA LINDA, CALIFORNIA FIGURE A-2**



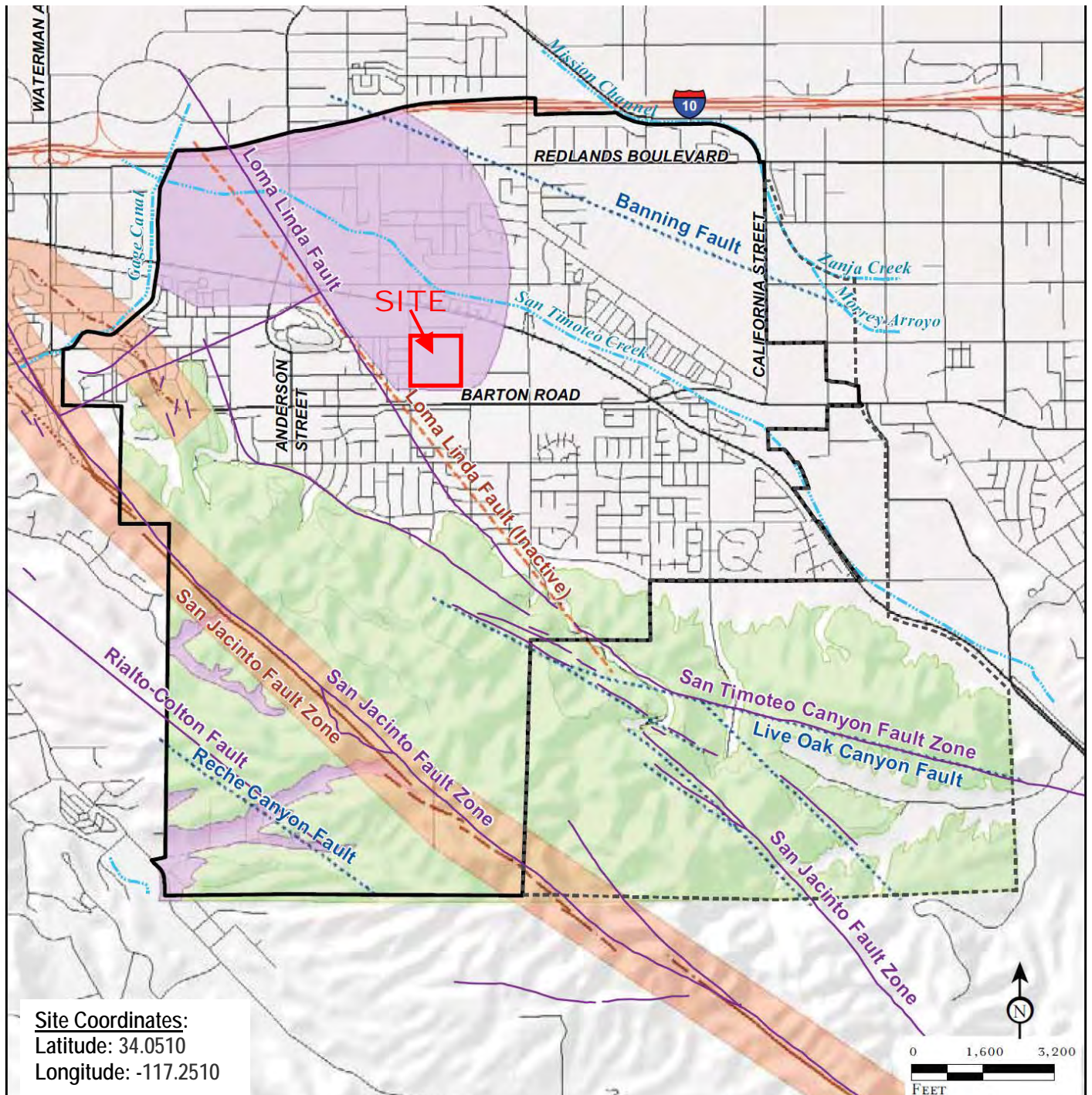


FIGURE 10.1

L S A

**Geologic Hazards**

- San Jacinto Alquist-Priolo Zone
- Fault, Certain
- Fault, Approximate
- Fault, Concealed
- Fault Lines
- Loma Linda Fault
- USGS Faults
- Liquefaction Zones
- Area of Steep Slopes and Slope Instability

**JURISDICTIONAL AND INFRASTRUCTURE**

- City Boundary
- City Sphere of Influence
- Railroad
- Water Ways

SOURCE: Safety Element of the April 1, 1991 General Plan, Rick Engineering Company; California Geological Survey, 2002; USGS 30x60 San Bernardino Geologic Map, 2003; Thomas Bros 2009  
\*Branch Fault Lines are not all named per the USGS Geologic Maps 2003

City of Loma Linda General Plan

**GEOLOGIC HAZARDS**

**GEOBASE**

**SEISMIC HAZARD ZONES MAP**

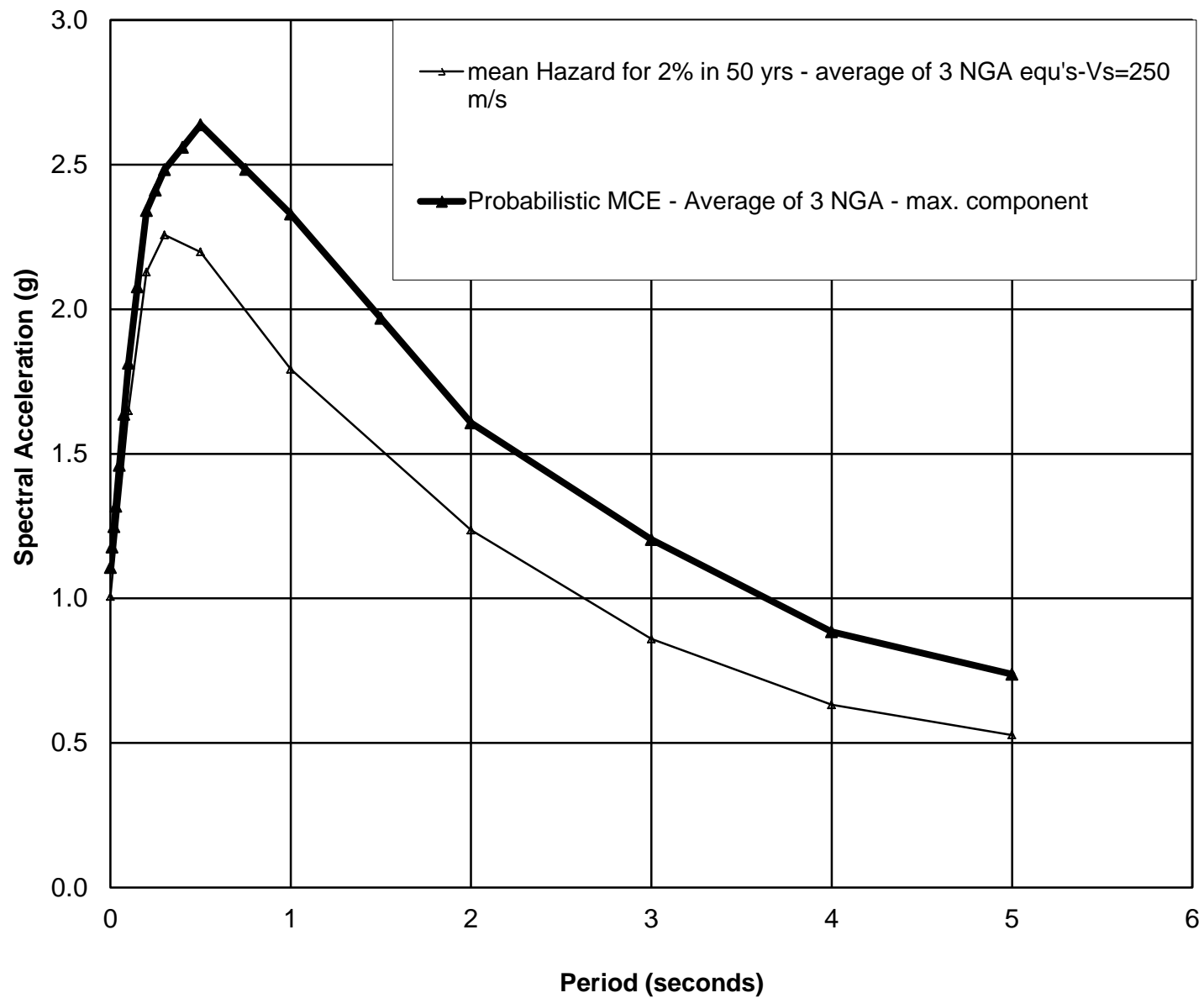
JERRY L. PETTIS MEMORIAL

VETERANS MEDICAL CENTER – OP PHARMACY

C.329.03.00

LOMA LINDA, CALIFORNIA

FIGURE A-3



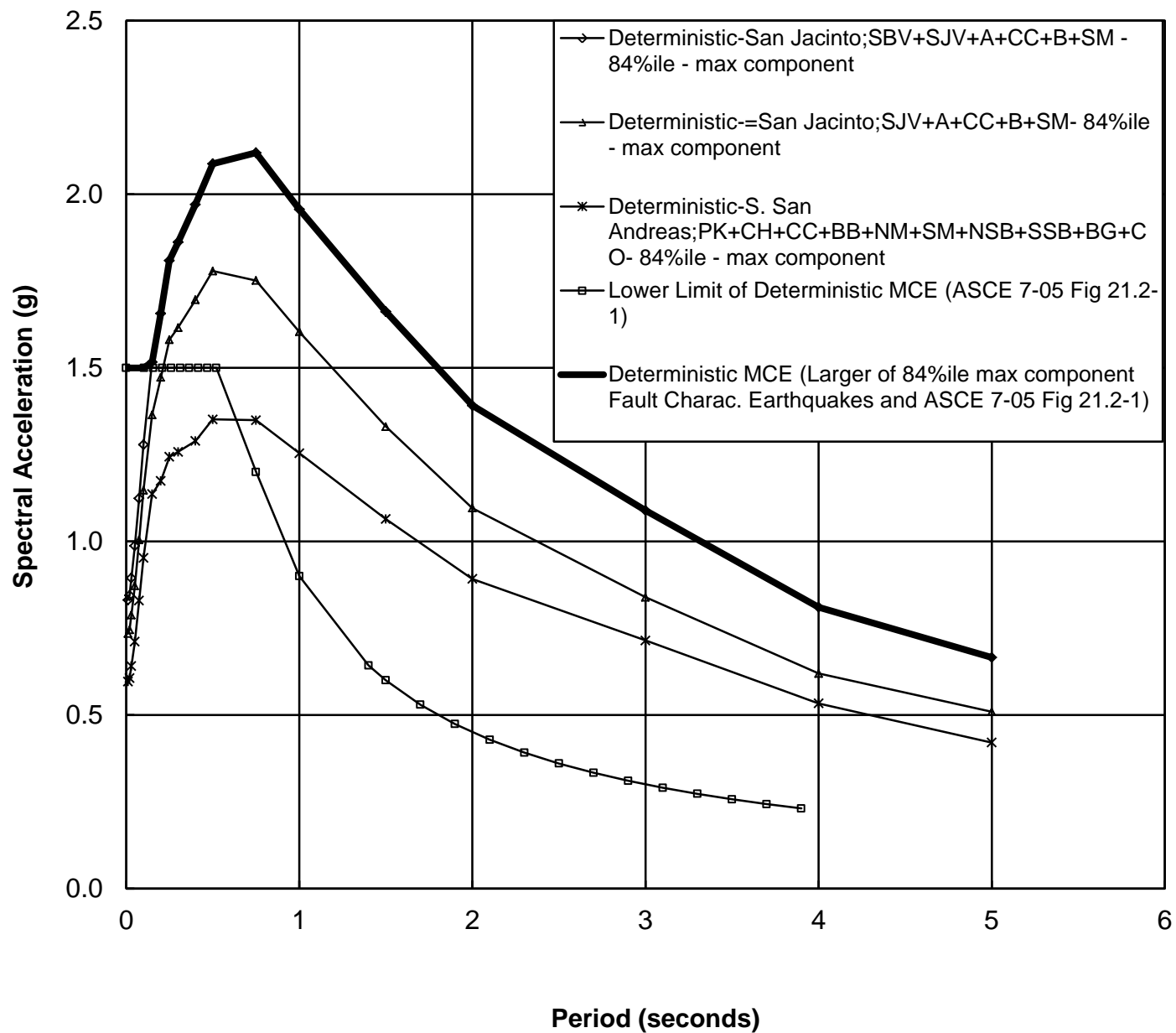
GEOBASE

# PROBABILISTIC MCE RESPONSE SPECTRA

JERRY L. PETTIS MEMORIAL  
VETERANS MEDICAL CENTER – OP PHARMACY  
LOMA LINDA, CALIFORNIA

C.329.03.00

FIGURE A-4



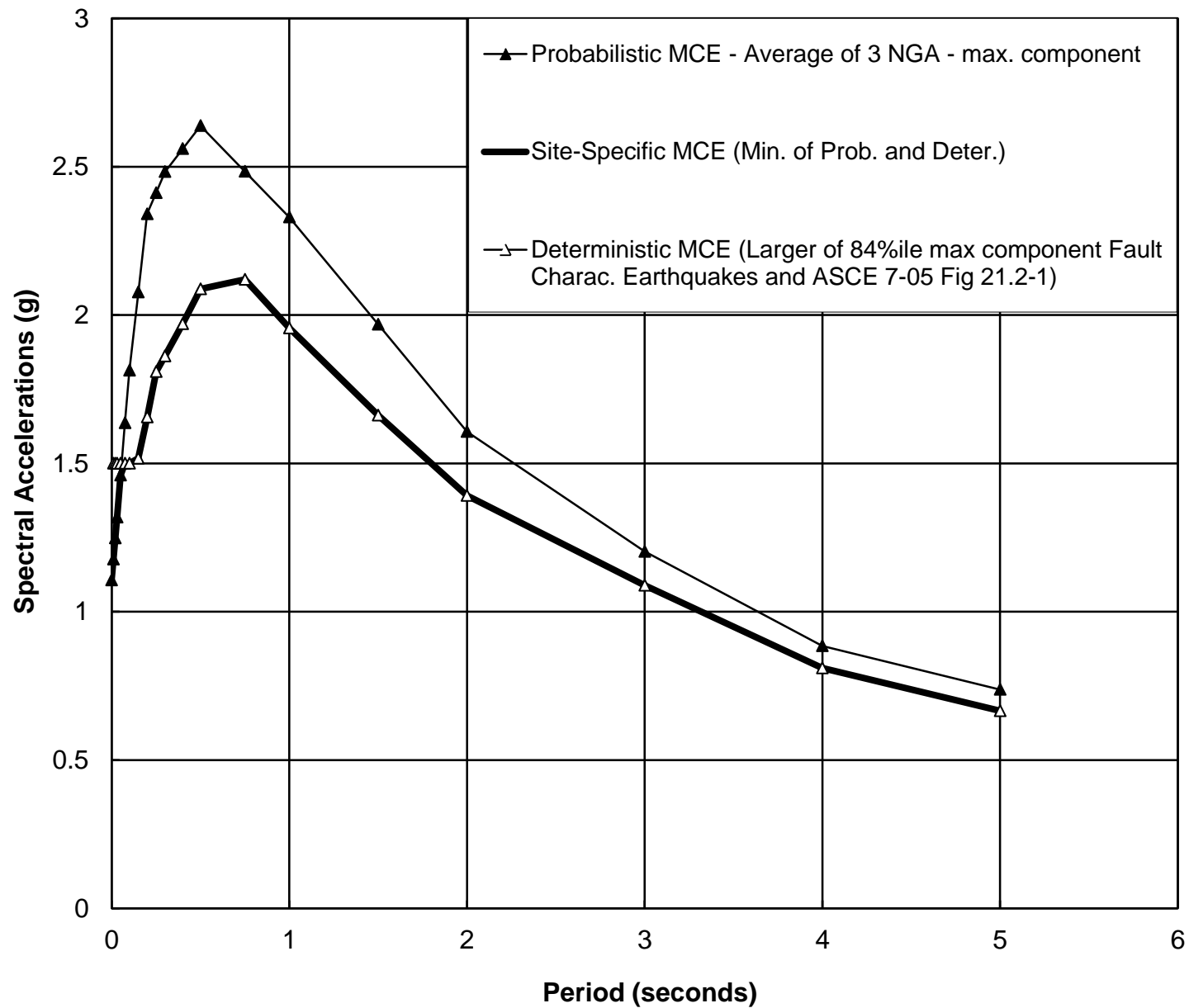
GEOBASE

# DETERMINISTIC MCE RESPONSE SPECTRA

JERRY L. PETTIS MEMORIAL  
VETERANS MEDICAL CENTER – OP PHARMACY  
LOMA LINDA, CALIFORNIA

C.329.03.00

FIGURE A-5



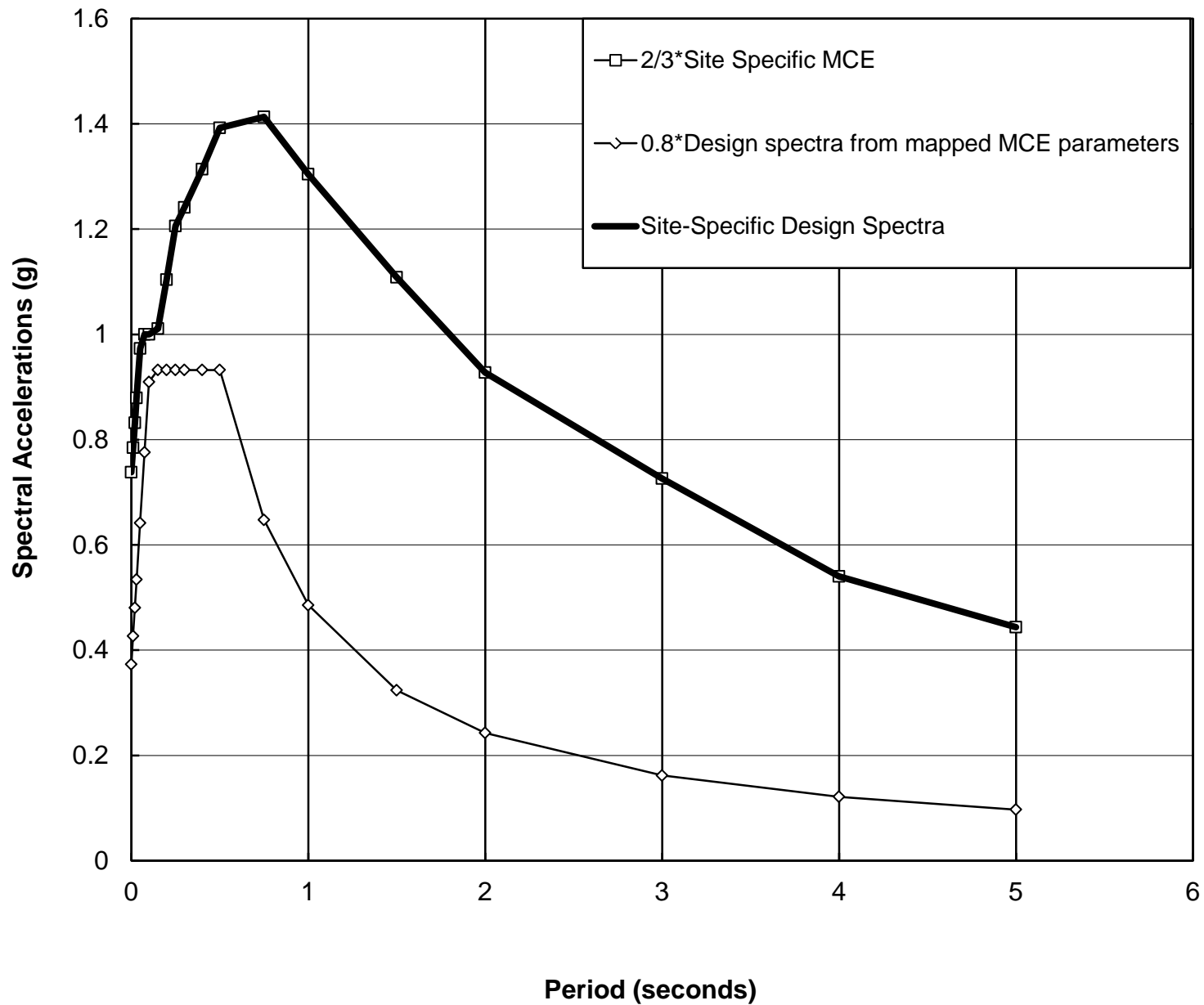
GEOBASE

# SITE-SPECIFIC MCE RESPONSE SPECTRA

JERRY L. PETTIS MEMORIAL  
VETERANS MEDICAL CENTER – OP PHARMACY  
LOMA LINDA, CALIFORNIA

C.329.03.00

FIGURE A-6



GEOBASE

SITE-SPECIFIC DESIGN RESPONSE SPECTRA

JERRY L. PETTIS MEMORIAL  
VETERANS MEDICAL CENTER – OP PHARMACY  
LOMA LINDA, CALIFORNIA

C.329.03.00

FIGURE A-7



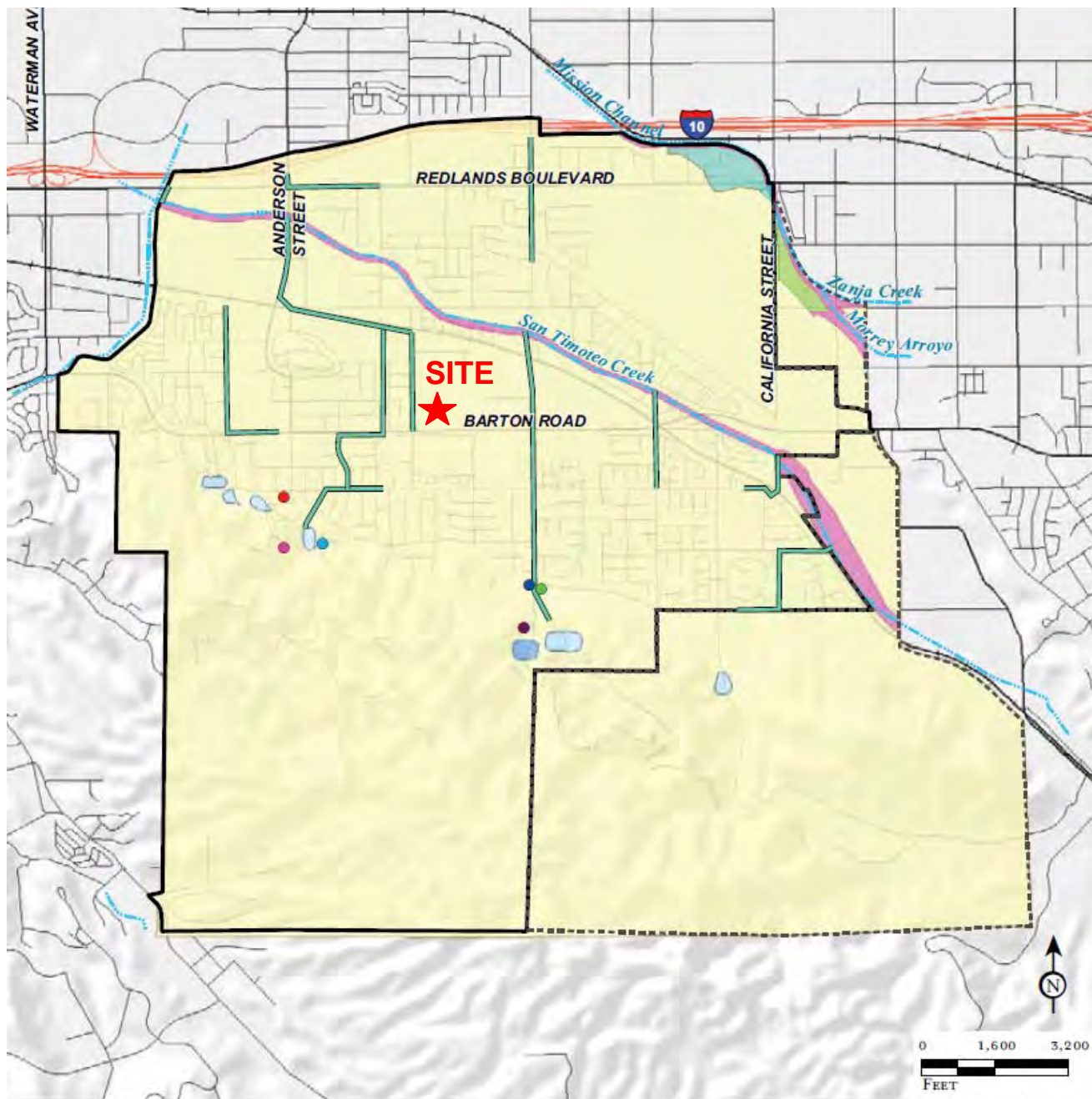


FIGURE 10.2

LSA

**Flood Zones\***

- Zone A  
No base flood elevations determined, contained within channel
- Zone AO - Flood Depths of 1-3 ft
- Zone X - 100 to 500 yr  
Areas of 500 yr flood; areas of 100 yr flood with average depths of less than 1 foot

- Zone X - 500 yr or greater  
Areas determined to be outside the 500 yr floodplain
- Existing Detention Basin
- Proposed Detention Basin
- Major Storm Drains

- Water Tanks**
- Water Tank - 3.2MG
  - Water Tank - 8.0MG
  - Water Tank - 2.0MG
  - Water Tank - 0.6MG
  - Water Tank - 1.0MG
  - Water Tank - 0.1MG

**Jurisdictional and Infrastructure**

- City Boundary
- City Sphere of Influence
- Railroad
- Water Ways

\*Flood Zone AE is located within the confines of the San Timoteo Creek Channel  
SOURCE: FEMA DFIRM 2008; Thomas Bros 2009

*City of Loma Linda General Plan*

**FLOOD HAZARD AREAS AND  
FLOOD CONTROL FACILITIES**

**GEOBASE**

**FLOOD MAP**

**JERRY L. PETTIS MEMORIAL  
VETERANS MEDICAL CENTER – OP PHARMACY  
LOMA LINDA, CALIFORNIA**

**C.329.03.00**

**FIGURE A-8**

## **APPENDIX B**

Figure B-1	Explanation of Terms and Symbols
Figure B-2	Log of Boring B-1
Figure B-3	Log of Boring B-2

*GEOBASE, INC., November 2006 –*

Figure B-4	Log of Boring B-1
Figure B-5	Log of Boring B-2



The terms and symbols used on the Log of Borings to summarize the results of the field investigation and subsequent laboratory testing are described in the following:

It should be noted that materials, boundaries, and conditions have been established only at the boring locations, and are not necessarily representative of subsurface conditions elsewhere across the site.

#### A. PARTICLE SIZE DEFINITION (ASTM D2487 AND D422)

Boulder	-- larger than 12-inches	Sand, medium	-- No.40 to No. 10 sieves
Cobble	-- 3-inches to 12-inches	Sand, fine	-- No.200 to No. 40 sieves
Gravel, coarse	-- 3/4-inch to 3-inches	Silt	-- 5µm to No. 200 sieves
Gravel, fine	-- No.4 sieve to 3/4 -inch	Clay	-- smaller than 5 µm
Sand, coarse	-- No.10 to No.4 sieve		

#### B. SOIL CLASSIFICATION

Soils and bedrock are classified and described according to their engineering properties and behavioral characteristics. The soil of each stratum is described using ASTM D2487 and D2488.

The following adjectives may be employed to define percentage ranges by weight of minor components:

trace	--	1-10%	some	--	20-35%
little	--	10-20%	"and" or "y"	--	35-50%

The following descriptive terms may be used for stratified soils:

parting	--	0 to 1/16-in. thickness;	layer	--	½-in. to 12-in. thickness;
seam	--	1/16 to ½-in. thickness;	stratum	--	greater than 12-in. thickness.

#### C. SOIL DENSITY AND CONSISTENCY

The density of coarse grained soils and the consistency of fine grained soils are described on the basis of the Standard Penetration Test:

COARSE GRAINED SOILS		FINE GRAINED SOILS		
DENSITY	SPT BLOWS PER FOOT	ESTIMATED CONSISTENCY	SPT BLOWS PER FOOT	ESTIMATED RANGE OF UNCONFINED COMPRESSIVE STRENGTH (TSF)
very loose	less than 4	very soft	less than 2	less than 0.25
loose	5 to 10	soft	2 to 4	0.25 to 0.50
medium	11 to 30	firm (medium)	5 to 8	0.50 to 1.0
dense	31 to 50	stiff	9 to 15	1.0 to 2.0
very dense	over 50	very stiff	16 to 30	2.0 to 4.0
		hard	over 30	over 4.0

**GEOBASE**

**EXPLANATION OF TERMS  
AND SYMBOLS USED**

#### D. STANDARD PENETRATION TEST (SPT) -- D1586

The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, 2-inches outside diameter and 1 3/8-inches inside diameter, is driven eighteen (18) inches. The sampler is seated in the first six (6) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

#### E. ABBREVIATION OF LABORATORY TEST DESIGNATIONS

C	Consolidation	pH	pH
CBR	California Bearing Ratio	pp	Pocket Penetrometer
Ch	Water Soluble Chlorides	PS	Particle Size
DS	Direct Shear	RV	R-Value
EI	Expansion Index	SE	Sand Equivalent
ER	Electrical Resistivity	SG	Specific Gravity
k	Permeability	SO <sub>4</sub>	Water Soluble Sulfates
MD	Moisture	TX	Triaxial Compression
MP	Modified Proctor Compaction Test	TV	Torvane Shear
O	Organic Content	U	Unconfined Compression

#### F. STRATIFICATION LINES

The stratification lines indicated on the boring logs and profiles represent the ***approximate*** boundary between material types and the transition may be gradual.

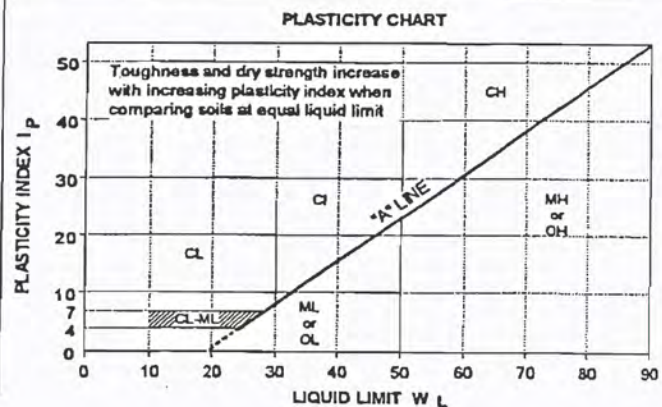
# SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISION			GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
HIGHLY ORGANIC SOILS			PI		Peat and other highly organic soils	Strong color or odor and often fibrous texture	
COARSE-GRAINED SOILS (More than half by weight larger than No. 200 sieve size)	GRAVELS (More than half coarse fraction larger than No. 4 sieve size)	CLEAN GRAVELS	GW		Well-graded Gravels, Gravel-Sand mixtures (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		Poorly-graded Gravels and Gravel-Sand mixtures (<5% fines)	Not meeting all above requirements	
		DIRTY GRAVELS	GM		Silty Gravels, Gravel-Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p < 4$	
			GC		Clayey Gravels, Gravel-Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p > 7$	
	SANDS (More than half coarse fraction smaller than No. 4 sieve size)	CLEAN SANDS	SW		Well-graded Sands, Gravelly Sands (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		Poorly-graded Sands or Gravelly Sands (<5% fines)	Not meeting all above requirements	
		DIRTY SANDS	SM		Silty Sands, Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p < 4$	
			SC		Clayey Sands, Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p > 7$	
FINE-GRAINED SOILS (More than half by weight passes No. 200 sieve size)	SILTS		ML		Inorganic Silts and very fine Sands, Rock Flour, Silty Sands of slight plasticity	$W_L < 50$	See chart below
	Below "A" line on plasticity chart: negligible organic content		MH		Inorganic Silts micaceous or diatomaceous, fine Sandy or Silty soils	$W_L > 50$	
	CLAYS		CL		Inorganic Clays of low plasticity, Gravelly, Sandy, or Silty Clays, lean Clays	$W_L < 30$	
	Above "A" line on plasticity chart: negligible organic content		CI		Inorganic Clays of medium plasticity, Silty Clays	$W_L > 30, < 50$	
			CH		Inorganic Clays of high plasticity, fat Clays	$W_L > 50$	
	ORGANIC SILTS & ORGANIC CLAYS		OL		Organic Silts and organic Silty Clays of low plasticity	$W_L < 50$	
	Below "A" line on plasticity chart		OH		Organic Clays of high plasticity	$W_L > 50$	

The soil of each stratum is described using ASTM D2487 and D2488 modified slightly so that an inorganic clay of "medium plasticity" is recognized.

## ADDITIONAL SOIL CLASSIFICATION

	Fill Soil
	Sa Sandstone
	Cs Claystone
	Ms Siltstone



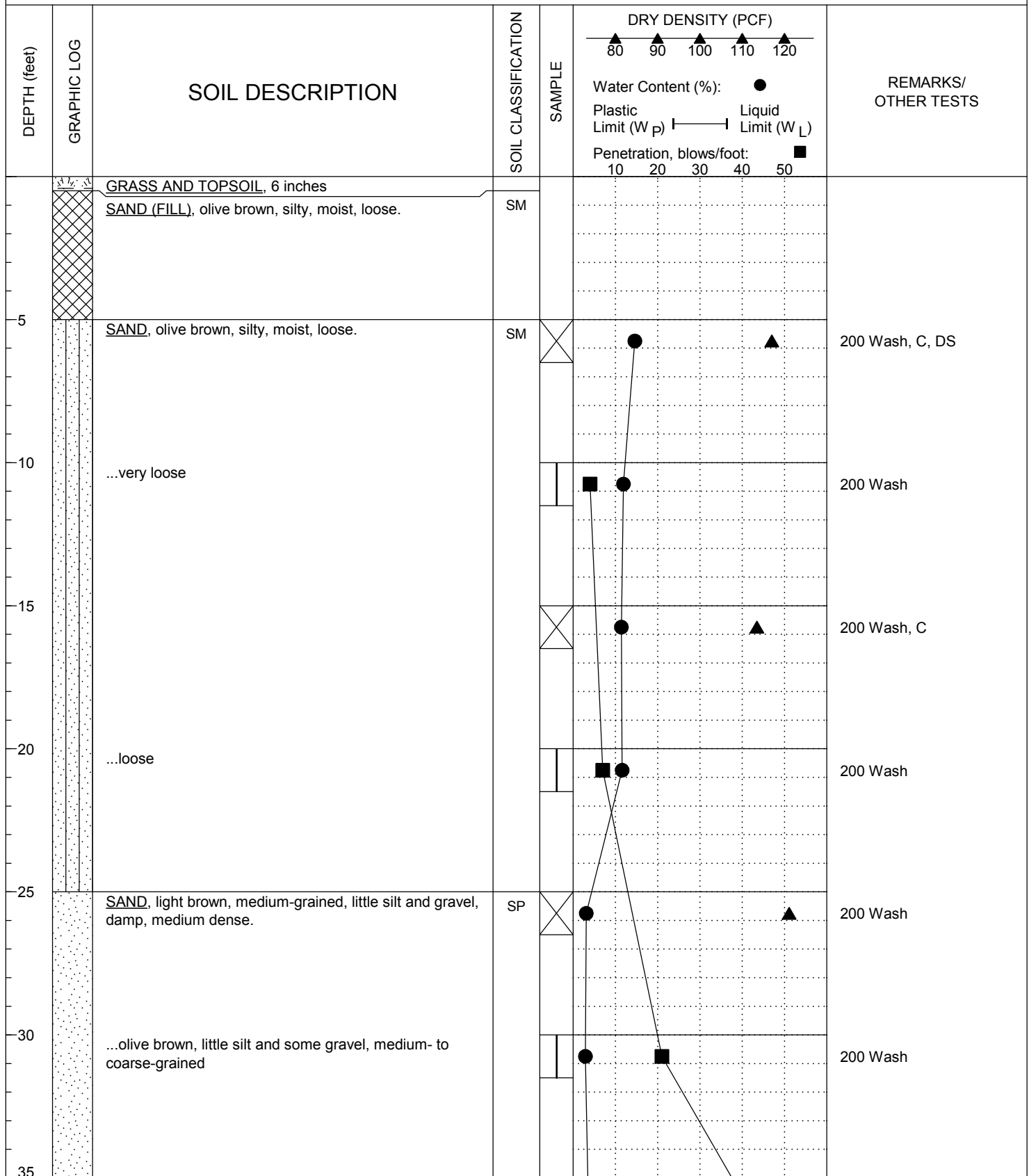
GEOBASE

EXPLANATION OF TERMS  
AND SYMBOLS USED

Figure B-1

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE

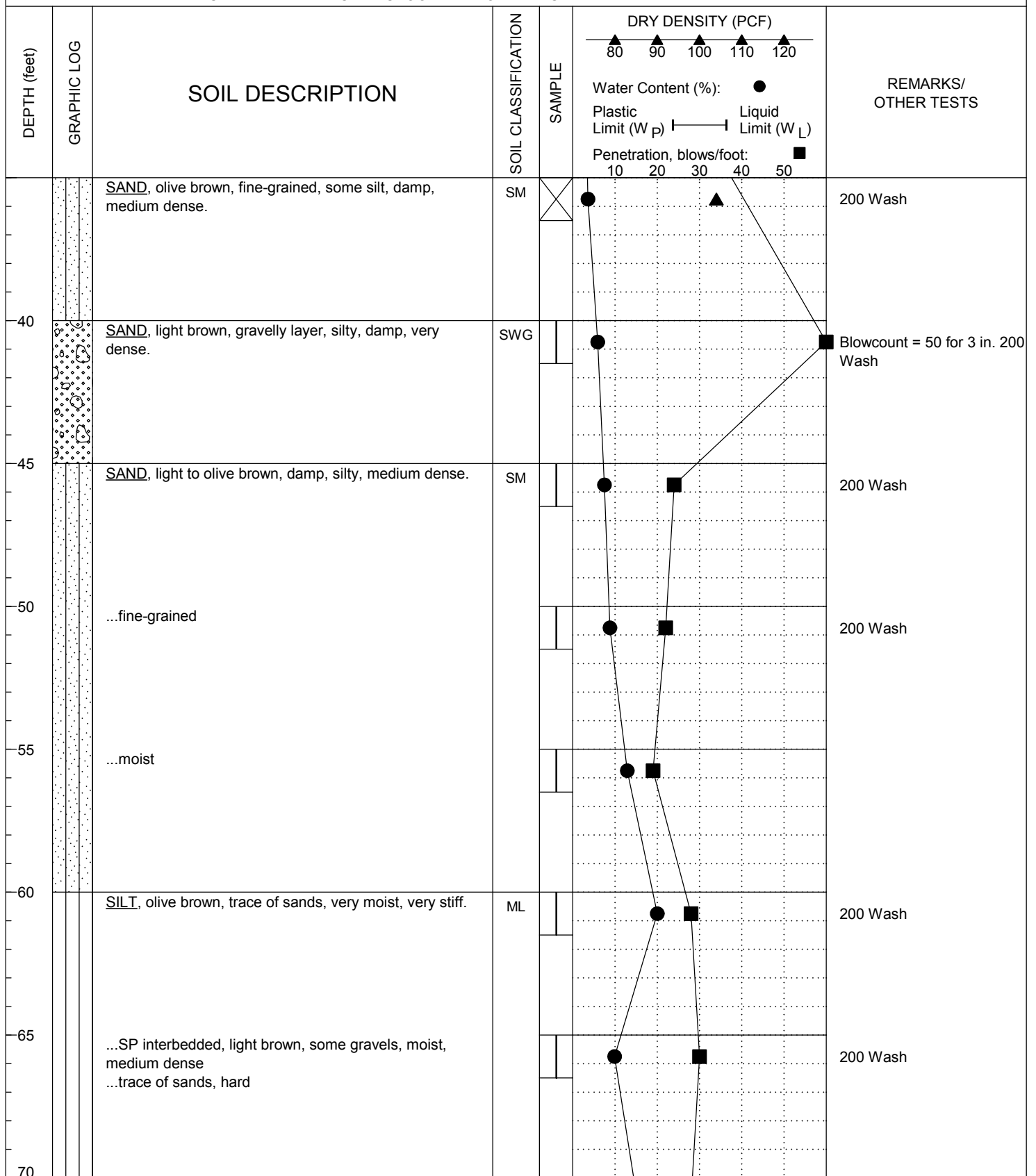


GEOBASE, INC.	PROJECT Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA			BORING NO. B-1
	DEPTH TO WATER feet ▼	SURFACE ELEV. 1146 feet	LOGGED BY HDN	PROJECT NO. C.329.03.00
	DEPTH TO SLOUGH ▲	DRILL RIG CME-75 HT DRILLER JDK	DATE LOGGED 04/28/2012	FIGURE NO. B-2

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE



<b>GEOBASE, INC.</b>	PROJECT <b>Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA</b>			<b>BORING NO. B-1</b>
	DEPTH TO WATER feet ▼	SURFACE ELEV. <b>1146 feet</b>	LOGGED BY <b>HDN</b>	PROJECT NO. <b>C.329.03.00</b>
	DEPTH TO SLOUGH ▲	DRILL RIG <b>CME-75 HT</b> DRILLER <b>JDK</b>	DATE LOGGED <b>04/28/2012</b>	FIGURE NO. <b>B-2</b>

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE

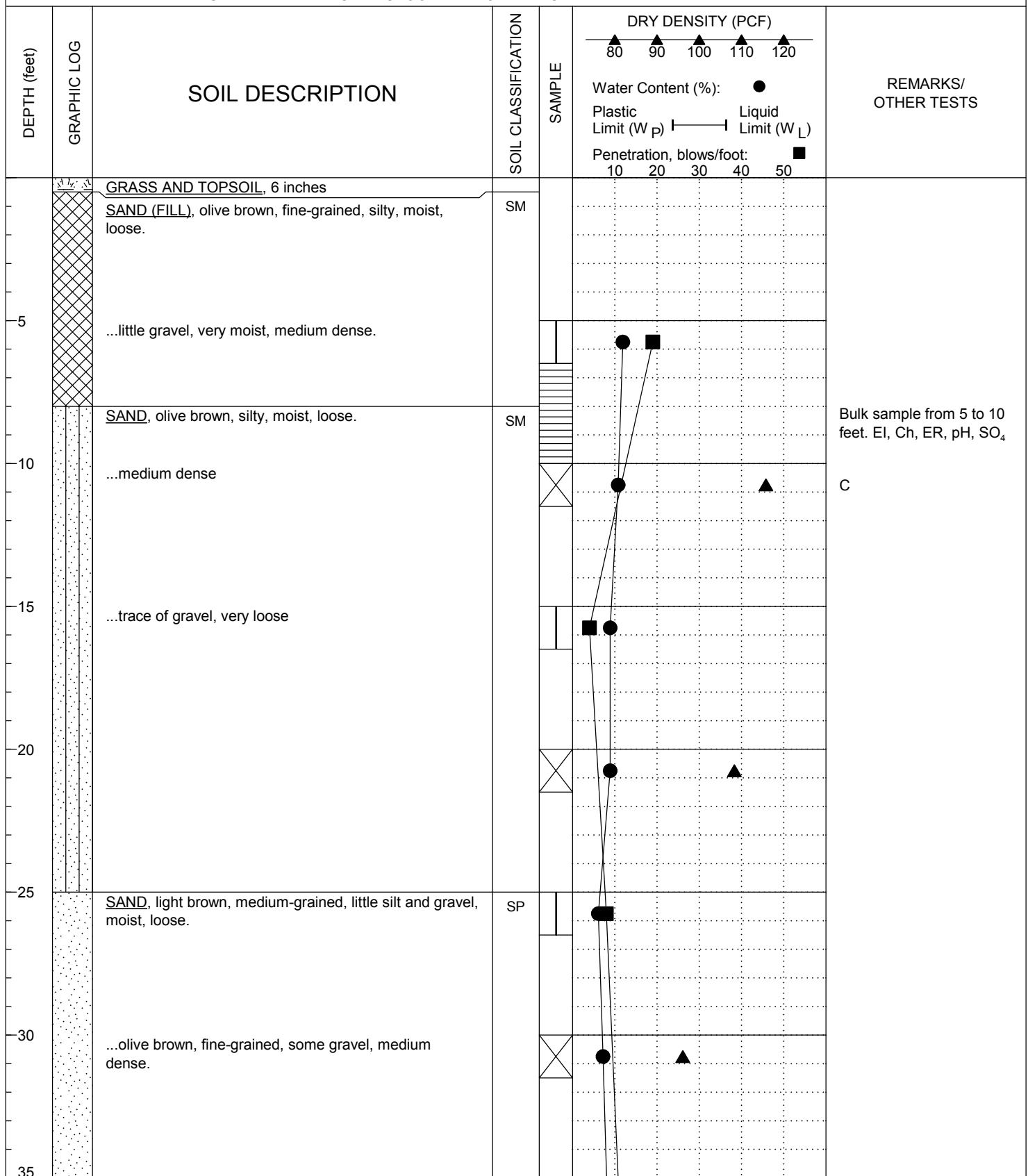
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF)		REMARKS/ OTHER TESTS
					80	90	
		<u>SILT</u> , olive brown, trace of sands, very moist, very stiff.	ML				
75		End of boring at 71.5 feet. Boring dry at completion of drilling. Backfilled with on-site soils.					
80							
85							
90							
95							
100							
105							

<b>GEOBASE, INC.</b>	PROJECT		Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA		BORING NO.	B-1
	DEPTH TO WATER	feet ▼	SURFACE ELEV.	1146 feet	LOGGED BY	HDN
	DEPTH TO SLOUGH	▲	DRILL RIG	CME-75 HT	DATE LOGGED	04/28/2012
			DRILLER	JDK		FIGURE NO. B-2

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE

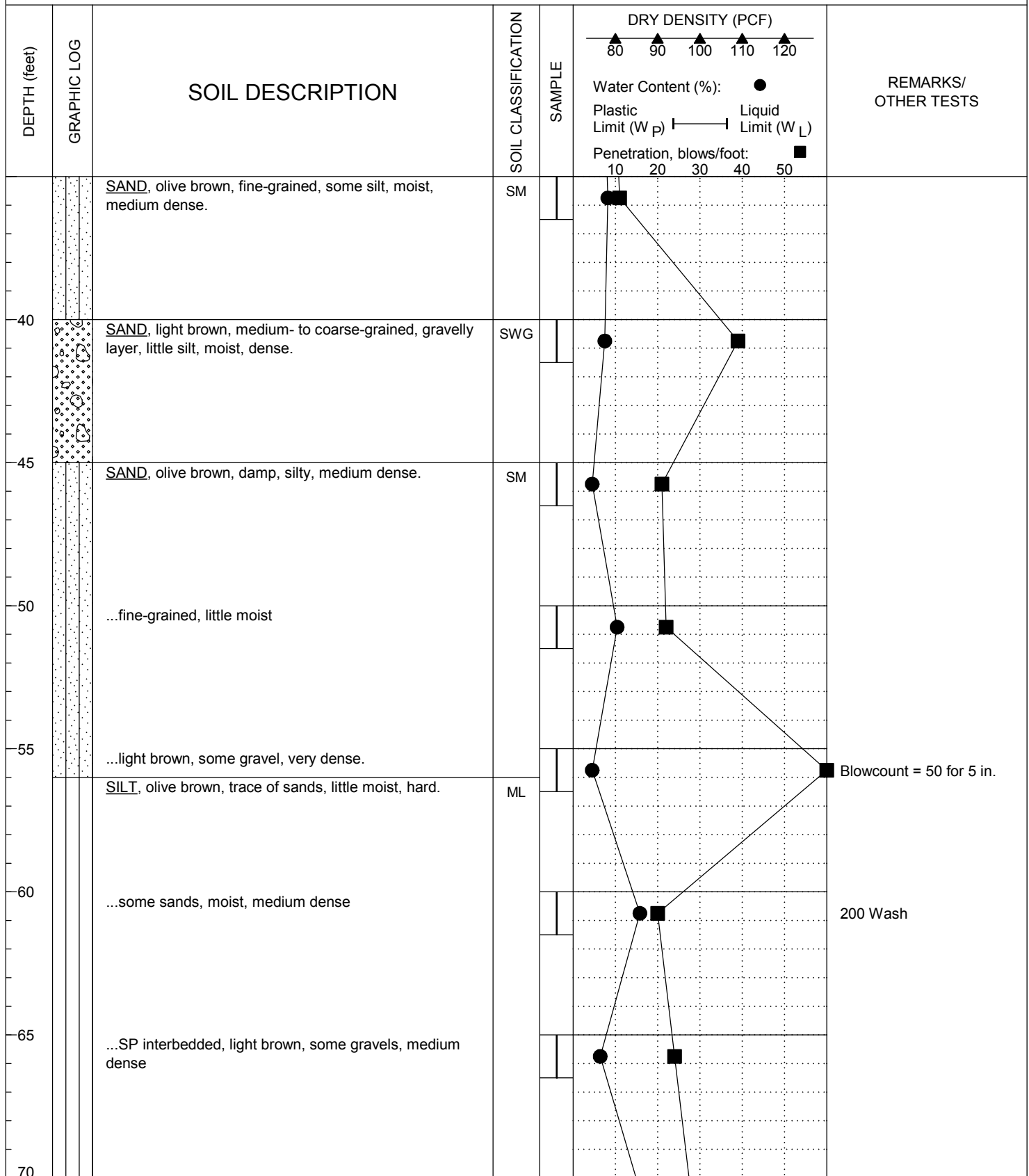


GEOBASE, INC.	PROJECT Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA			BORING NO. B-2
	DEPTH TO WATER feet ▼	SURFACE ELEV. 1148 feet	LOGGED BY HDN	PROJECT NO. C.329.03.00
	DEPTH TO SLOUGH ▲	DRILL RIG CME-75 HT DRILLER JDK	DATE LOGGED 04/28/2012	FIGURE NO. B-3

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE



<b>GEOBASE, INC.</b>	PROJECT <b>Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA</b>			<b>BORING NO. B-2</b>
	DEPTH TO WATER feet ▼	SURFACE ELEV. <b>1148 feet</b>	LOGGED BY <b>HDN</b>	PROJECT NO. <b>C.329.03.00</b>
	DEPTH TO SLOUGH ▲	DRILL RIG <b>CME-75 HT</b> DRILLER <b>JDK</b>	DATE LOGGED <b>04/28/2012</b>	FIGURE NO. <b>B-3</b>

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.



# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W <sub>P</sub> ) ——— Liquid Limit (W <sub>L</sub> ) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		<u>SILT</u> , olive brown, some fine-grained sands, moist, very stiff.	ML			
75		End of boring at 71.5 feet. Boring dry at completion of drilling. Backfilled with on-site soils.				
80						
85						
90						
95						
100						
105						

<b>GEOBASE, INC.</b>	PROJECT <b>Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA</b>		<b>BORING NO. B-2</b>
	DEPTH TO WATER feet ▼	SURFACE ELEV. <b>1148 feet</b>	LOGGED BY <b>HDN</b>
	DEPTH TO SLOUGH ▲	DRILL RIG <b>CME-75 HT</b> DRILLER <b>JDK</b>	DATE LOGGED <b>04/28/2012</b>
			PROJECT NO. <b>C.329.03.00</b>
			FIGURE NO. <b>B-3</b>

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

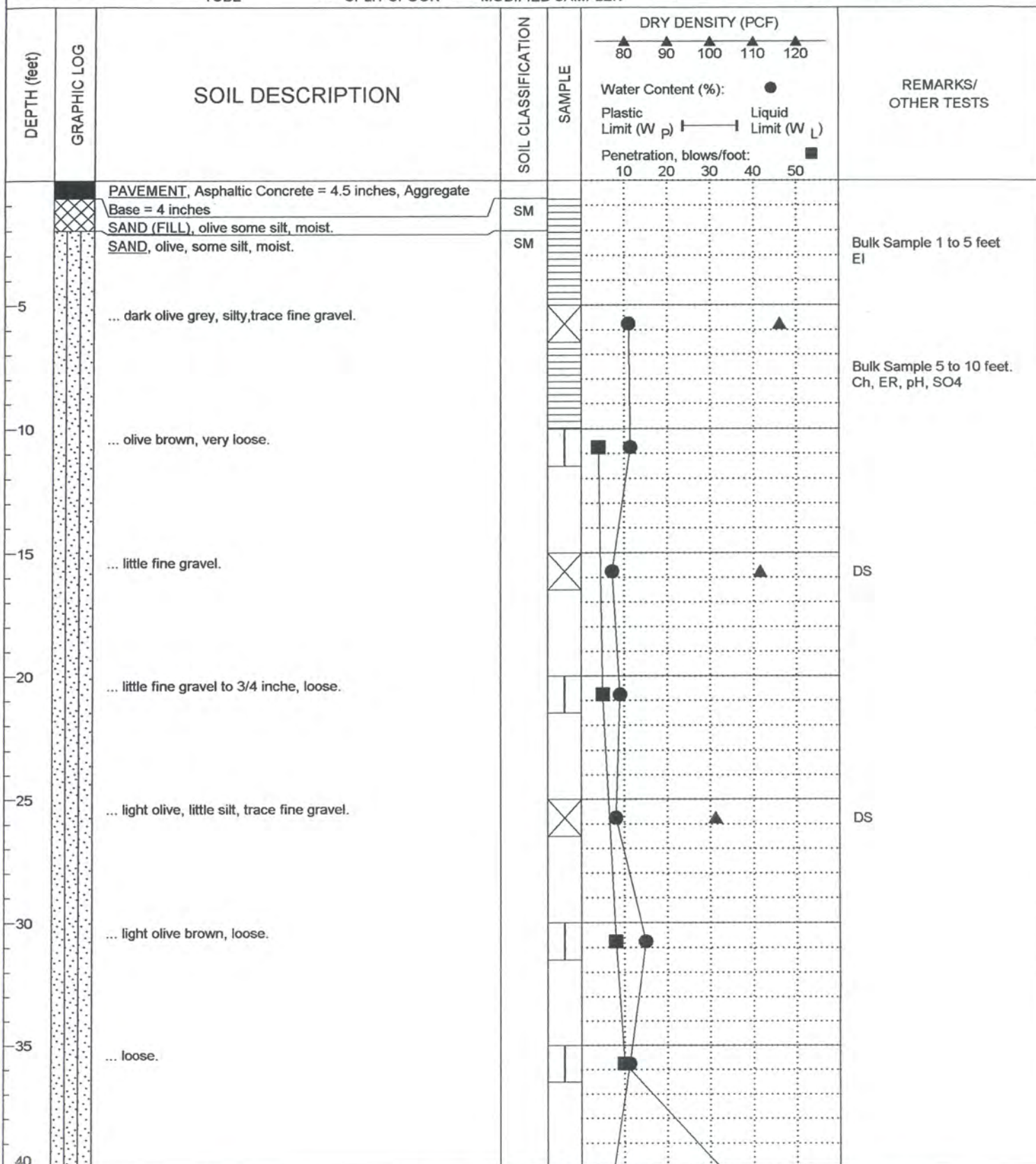
*GEOBASE, INC., November 2006 –*

Figure B-4      Log of Boring B-1

Figure B-5      Log of Boring B-2

# LOG OF BORING

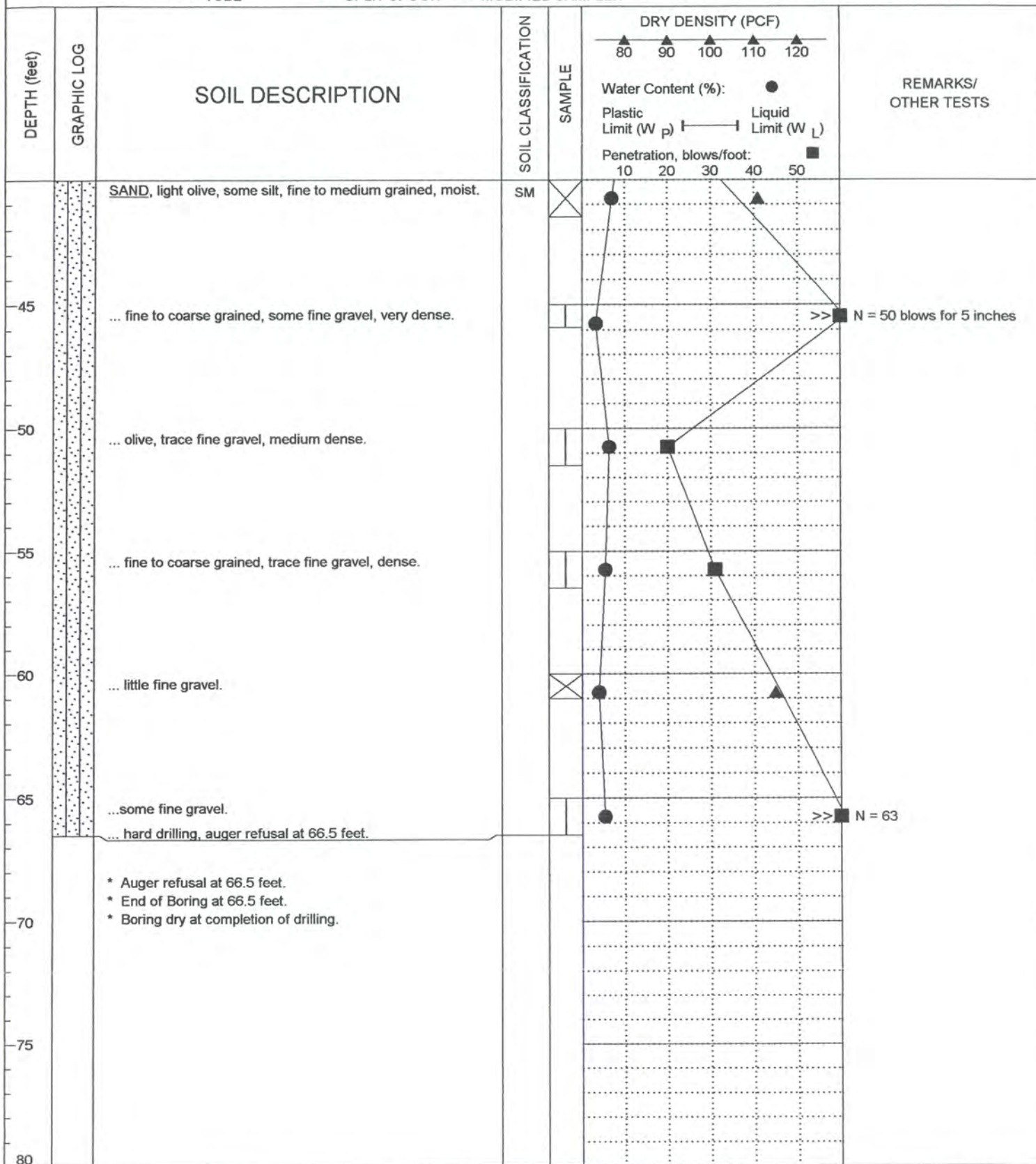
SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE



GEOBASE, INC.	PROJECT			PARKING STRUCTURE		BORING NO. B-1	
	V.A. MEDICAL CENTER, LOMA LINDA, CALIFORNIA						
	DEPTH TO WATER	feet	▼	SURFACE ELEV.	LOGGED BY RAP	PROJECT NO. P.322.01.00	
	DEPTH TO SLOUGH		▲	DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED 10/26/2006	FIGURE NO. B- 2	
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.							page 1 of 2

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE

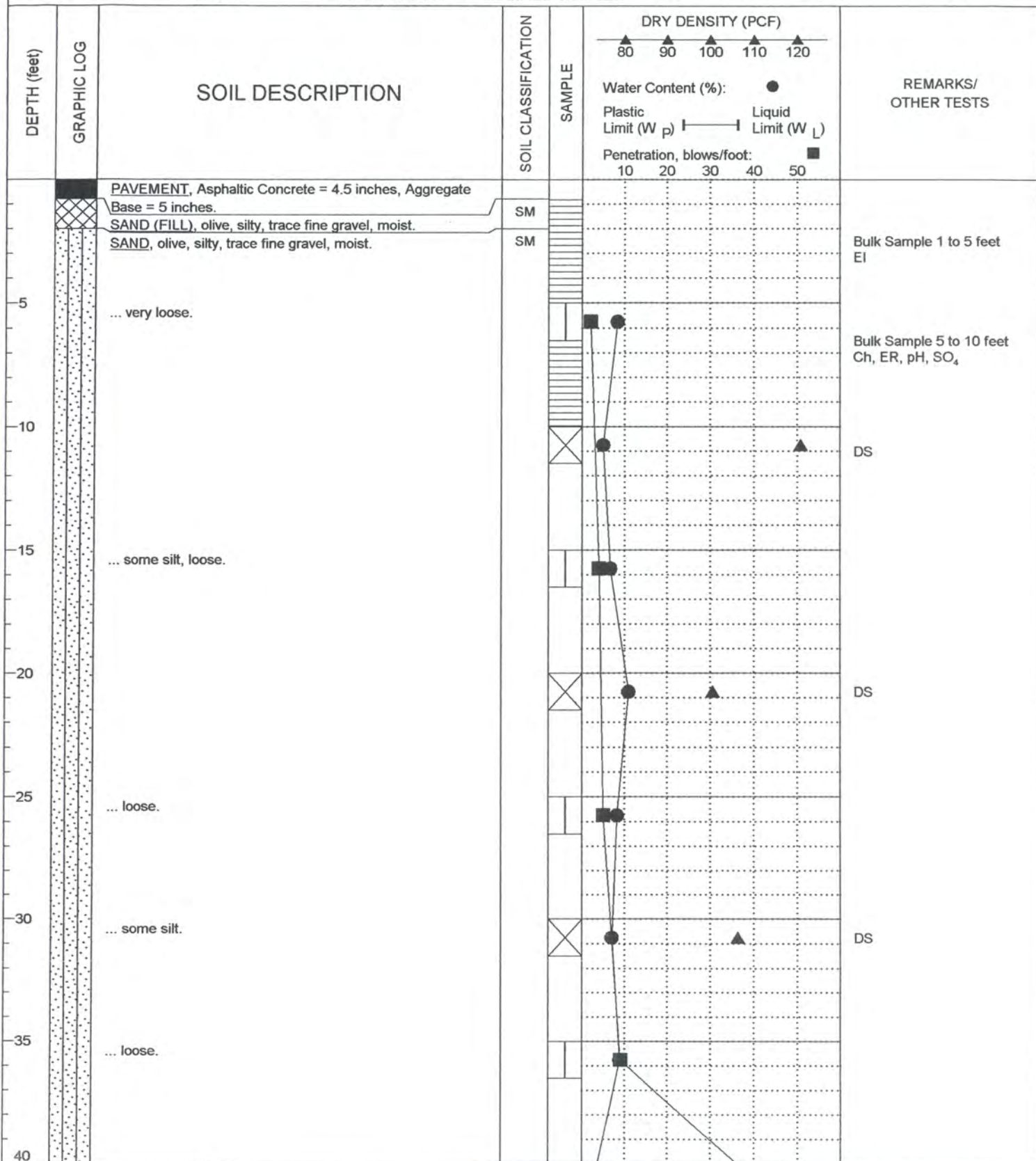


GEOBASE, INC.	PROJECT				PARKING STRUCTURE		BORING NO. B-1
	V.A. MEDICAL CENTER, LOMA LINDA, CALIFORNIA						
	DEPTH TO WATER		feet	▽	SURFACE ELEV.	LOGGED BY RAP	
	DEPTH TO SLOUGH			⌆	DRILL RIG CME-75 HT	DATE	FIGURE NO. B- 2
					DRILLER MARTINI	LOGGED 10/26/2006	
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.							page 2 of 2



# LOG OF BORING

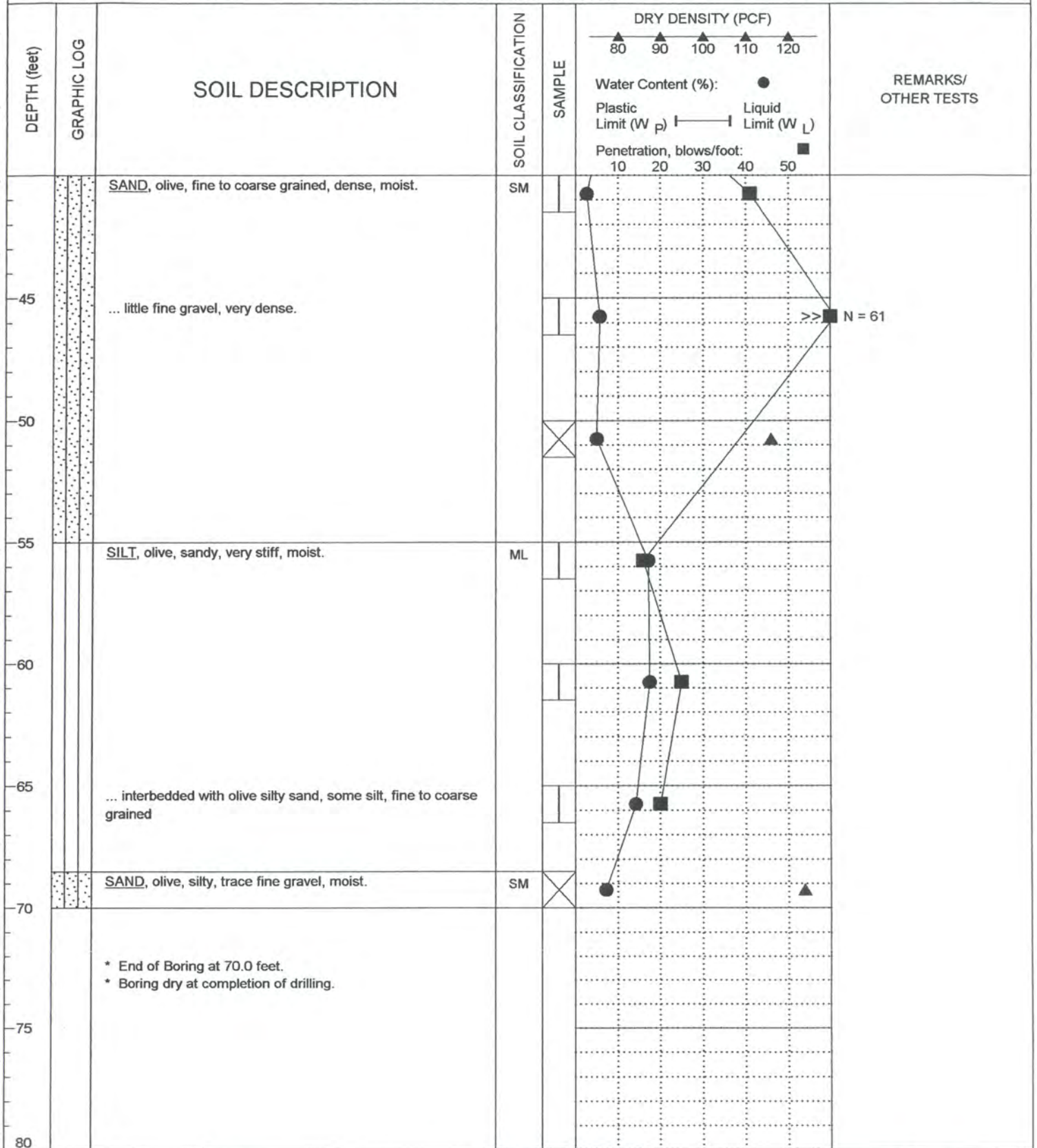
SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT ☐ SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE



GEOBASE, INC.	PROJECT		PARKING STRUCTURE		BORING NO.	B-2
	V.A. MEDICAL CENTER, LOMA LINDA, CALIFORNIA				PROJECT NO.	P.322.01.00
	DEPTH TO WATER	feet	SURFACE ELEV.	LOGGED BY	RAP	FIGURE NO.
	DEPTH TO SLOUGH		DRILL RIG	CME-75 HT	DATE	10/26/2006
			DRILLER	MARTINI	LOGGED	
Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.						page 1 of 2

# LOG OF BORING

SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE



**GEOBASE, INC.**

PROJECT **PARKING STRUCTURE**  
**V.A. MEDICAL CENTER, LOMA LINDA, CALIFORNIA**  
 DEPTH TO WATER feet ☒ SURFACE ELEV. ☐  
 DEPTH TO SLOUGH ☐ DRILL RIG **CME-75 HT**  
 DRILLER **MARTINI**  
 LOGGED BY **RAP**  
 DATE LOGGED **10/26/2006**

BORING NO. **B-2**  
 PROJECT NO. **P.322.01.00**  
 FIGURE NO. **B- 3**

Note: This log of boring should be evaluated in conjunction with the complete geotechnical report. This log of boring represents conditions observed at the specific boring location and at the date indicated.

page 2 of 2

## **APPENDIX C**

Figure C-1	Summary of Laboratory Test Results
Figure C-2	Consolidation Test Results
Figure C-3	Consolidation Test Results
Figure C-4	Consolidation Test Results
Figure C-5	Direct Shear Test
Figure C-6	Expansion Potential and Corrosivity Test Results
Figure C-7	Corrosivity Series Test Results by Anaheim Test Laboratory

*GEOBASE, INC., November 2006 --*

Figure C-8	Summary of Laboratory Tests
Figure C-9	Direct Shear Test Results
Figure C-10	Direct Shear Test Results
Figure C-11	Direct Shear Test Results
Figure C-12	Direct Shear Test Results
Figure C-13	Direct Shear Test Results
Figure C-14	Expansion Potential, Water Soluble, and Corrosivity Series Test Results
Figure C-15	Corrosivity Series Test Results by Anaheim Testing Laboratory





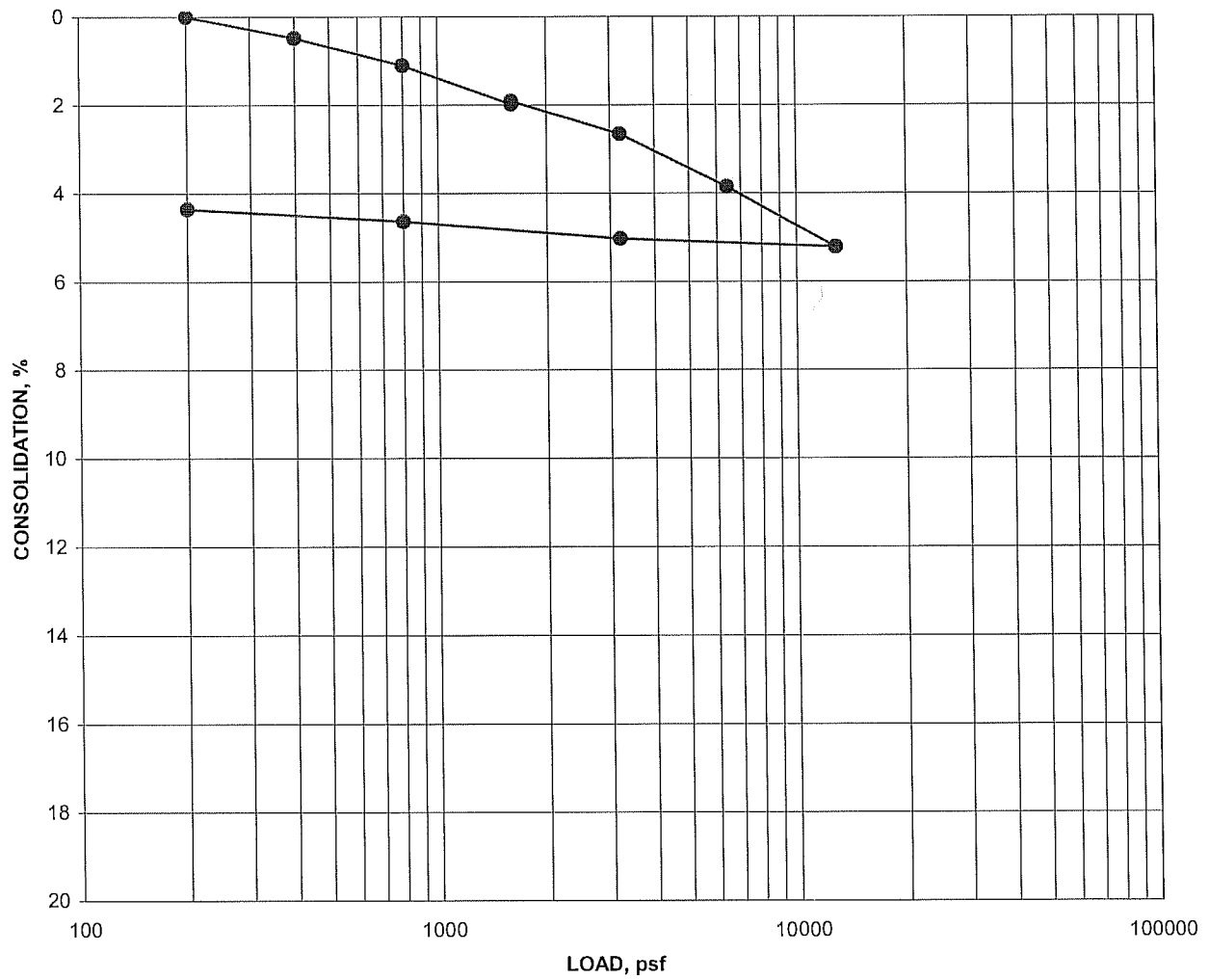
## CONSOLIDATION TEST

PROJECT: V. A Loma Linda

Boring No: B-1

Depth: 5'-6.5'

Sample Saturation @: 1600



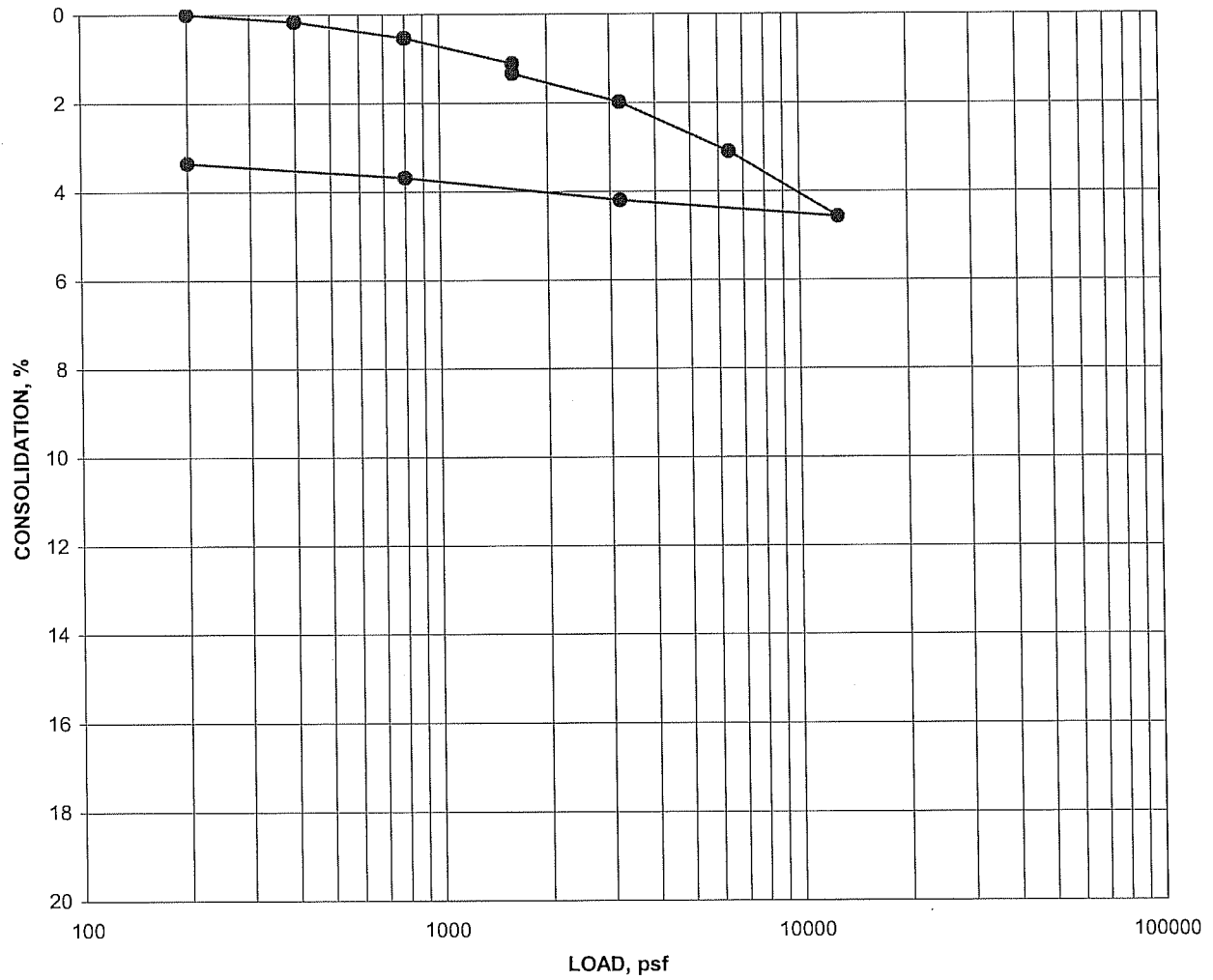
## CONSOLIDATION TEST

PROJECT: V. A Loma Linda

Boring No: B-1

Depth: 10'-11.5'

Sample Saturation @: 1600



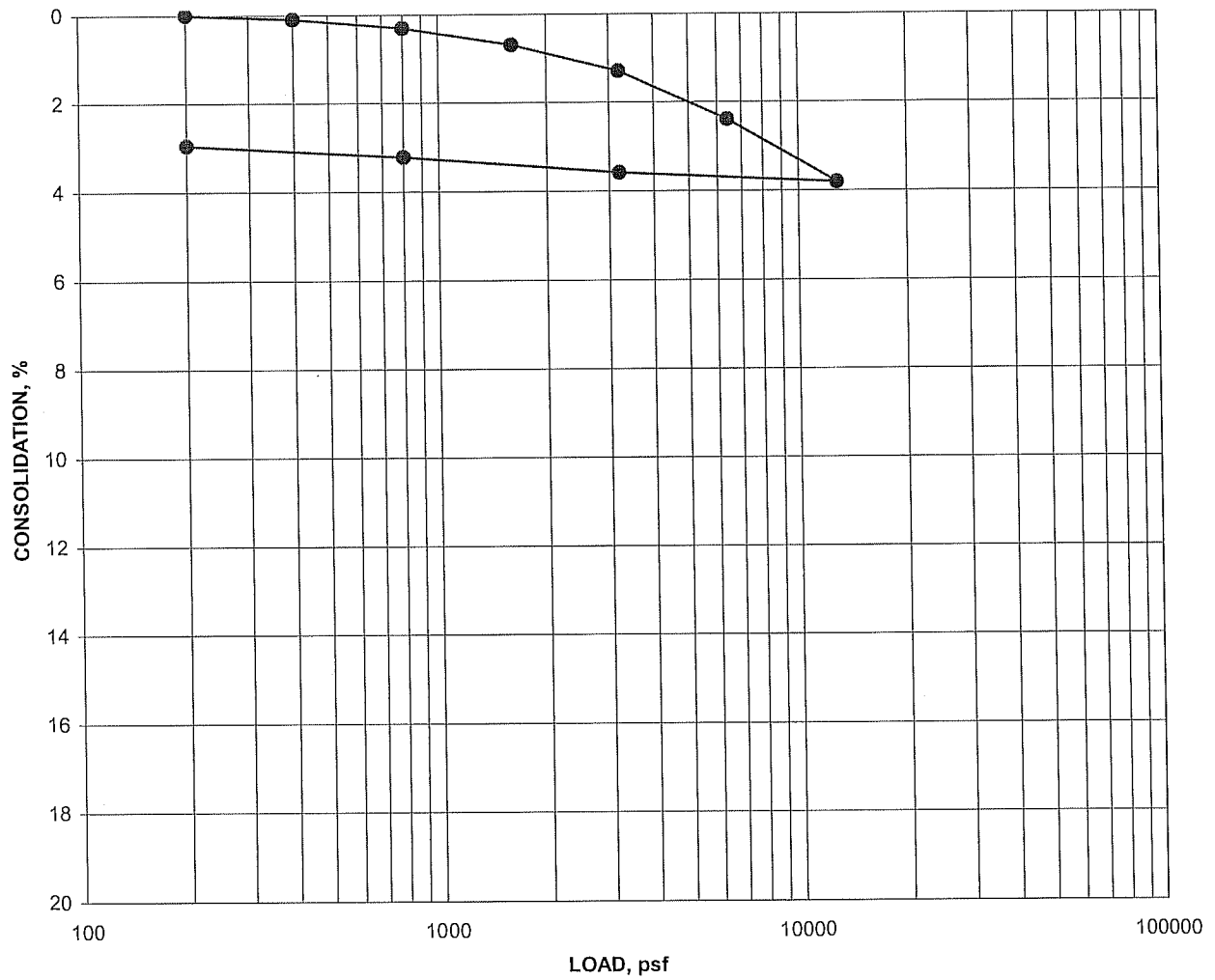
## CONSOLIDATION TEST

PROJECT: V. A Loma Linda

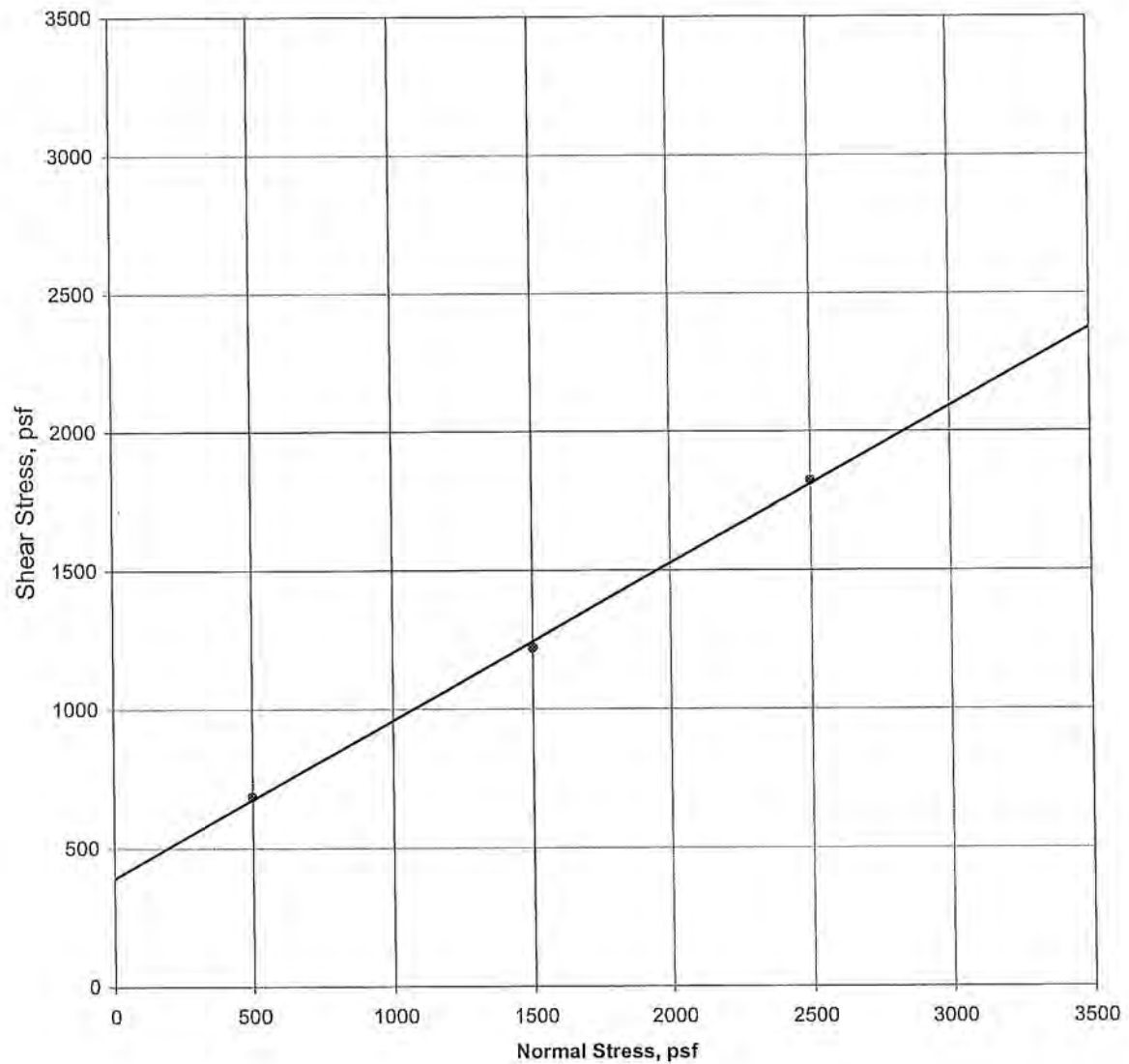
Boring No: B-2

Depth: 10'-11.5'

Sample Saturation @: 1600



## DIRECT SHEAR TEST



Project: V. A Loma Linda  
Project No.: 232E-400  
Boring No.: B-1  
Depth Elevation: 5'-6.5' ft.

Cohesion,  $C = 400$ psf  
Friction Angle =  $30^\circ$   
Light brown sandy SILT

### EXPANSION POTENTIAL

ASTM D4829

SOIL SAMPLE LOCATION (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-2 at 5.0-10.0	10	Very Low

### WATER-SOLUBLE SULFATES

CAL. 417-A

SOIL SAMPLE LOCATION (feet)	SOLUBLE SULFATES PPM	POTENTIAL FOR ATTACK ON CONCRETE
B-2 at 5.0-10.0	576	Low

### CORROSIVITY SERIES TEST

SOIL SAMPLE LOCATION (feet)	pH (CAL 747)	SOLUBLE CHLORIDE (CAL.422) (PPM)	ELEC. RESISTIVITY (CAL.643) (OHM-CM)	POTENTIAL FOR ATTACK ON STEEL (SENATOROFF)
B-2 at 5.0-10.0	6.9	114	2044	Moderate

# ANAHEIM TEST LABORATORY

3008 ORANGE AVENUE  
SANTA ANA, CALIFORNIA 92707  
PHONE (714) 549-7267

TO:

GEOBASE  
23362 PERALTA DRIVE, # 4&6  
LAGUNA HILLS, CA. 92653

DATE: 5/1/12

P.O. NO.: VERBAL

LAB NO. : B-5648

SPECIFICATION: CA-417/422/643

ATTN: BOB PEARSON  
JOHN

MATERIAL: SOIL

PROJECT #: C.329.03.00  
Loma Linda

## ANALYTICAL REPORT

### CORROSION SERIES SUMMARY OF DATA

	PH	SOLUBLE SULFATES per CA. 417 ppm	SOLUBLE CHLORIDES per CA. 422 ppm	MIN. RESISTIVITY per CA. 643 ohm-cm
B-2 @ 5-10'	6.9	576	114	2,044

RESPECTFULLY SUBMITTED



WES BRIDGER CHEMIST

*GEOBASE, INC., November 2006 --*

Figure C-8	Summary of Laboratory Tests
Figure C-9	Direct Shear Test Results
Figure C-10	Direct Shear Test Results
Figure C-11	Direct Shear Test Results
Figure C-12	Direct Shear Test Results
Figure C-13	Direct Shear Test Results
Figure C-14	Expansion Potential, Water Soluble, and Corrosivity Series Test Results
Figure C-15	Corrosivity Series Test Results by Anaheim Testing Laboratory

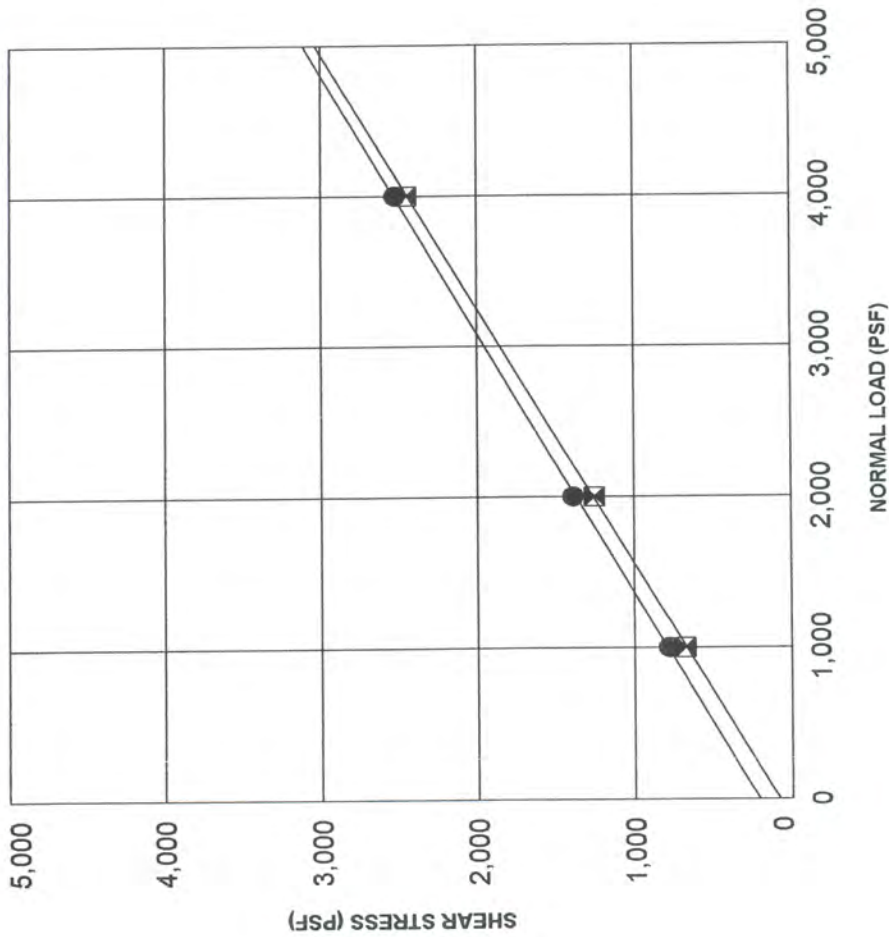
# GEOBASE, INC.

## SUMMARY OF LABORATORY TEST RESULTS

Figure C - 1  
Page 1 of 1

PROJECT: Parking Structure V.A. Medical Center, Loma Linda, California				PROJECT NO: P.322.01.00				DATE: November 2006						
BORING	DEPTH (feet)	MOISTURE CONTENT (Percent)	DRY DENSITY (pcf)	ATTERBERG LIMITS				PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS	
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)				
B-1	1.0-5.0	--										El	SM	
	5.0-6.5												SM	
	5.0-10.0	--										pH, Ch, ER,SO4	SM	
	10.0-11.5	11											SM	
	15.0-16.5	7	111.6									DS	SM	
	20.0-21.5	9											SM	
	25.0-26.5											DS	SM	
	30.0-31.5	15											SM	
	35.0-36.5	11											SM	
	40.0-41.5												SM	
	45.0-45.9	3											SM	
	50.0-51.5	6											SM	
	55.0-56.5	5											SM	
	60.0-61.0												SM	
	65.0-66.5	5											SM	
B-2	1.0-5.0	--										El	SM	
	5.0-6.5	8											SM	
	5.0-10.0	--										pH, Ch, ER,SO4	SM	
	10.0-11.5											DS	SM	
	15.0-16.5	7											SM	
	20.0-21.5											DS	SM	
	25.0-26.5	8											SM	
	30.0-31.5											DS	SM	
	35.0-36.5	9											SM	
	40.0-41.5	3											SM	
	45.0-46.5	6											SM	
	50.0-51.5												SM	
	55.0-56.5	17											ML	
	60.0-61.5	18											ML	
	65.0-66.5	14											ML	
	6850-70.0												SM	





SAMPLE DESCRIPTION: SM	HEIGHT (in): 1.0	DRY DENSITY (pcf): 109.2	PEAK	ULTIMATE
BORING NO.: B-1	AREA (sq in): 4.58	INITIAL MOISTURE (%): 9.8	●	⊠
DEPTH INTERVAL (ft): 15.0 - 16.5	SHEAR RATE (in/min): 0.002	FINAL MOISTURE (%): 18.7	212	82
			30	31
			COHESION (psf)	FRICTION ANGLE (deg)

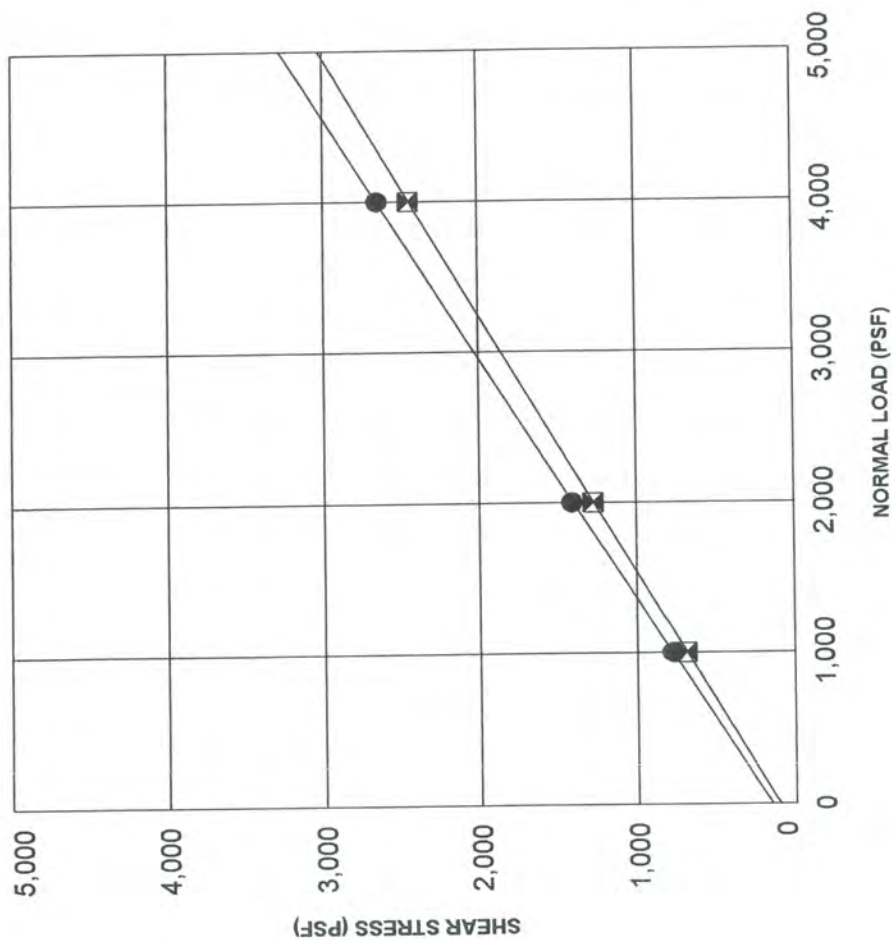
NOTES:

**Direct Shear Test Results**  
 PARKING STRUCTURE  
 V.A. MEDICAL CENTER  
 LOMA LINDA, CALIFORNIA

**GEOBASE**

P.322.01.00

Figure C-2



SAMPLE DESCRIPTION: SM BORING NO.: B-1 DEPTH INTERVAL (ft): 25.0 - 26.5	HEIGHT (in): 1.0	DRY DENSITY (pcf): 98.7	PEAK	ULTIMATE
	AREA (sq in): 4.58	INITIAL MOISTURE (%): 6.6	154	106
	SHEAR RATE (in/min): 0.002	FINAL MOISTURE (%): 23.0	32	30

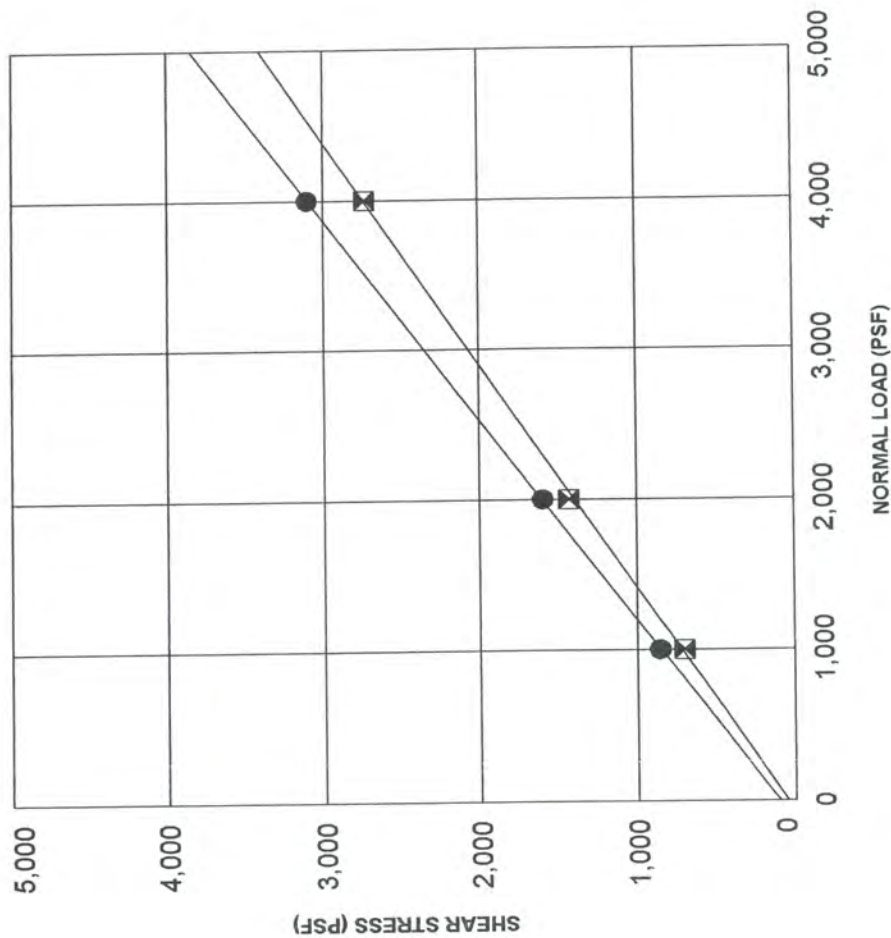
NOTES:

**Direct Shear Test Results**  
 PARKING STRUCTURE  
 V.A. MEDICAL CENTER  
 LOMA LINDA, CALIFORNIA

**GEOBASE**

Figure C-3

P.322.01.00

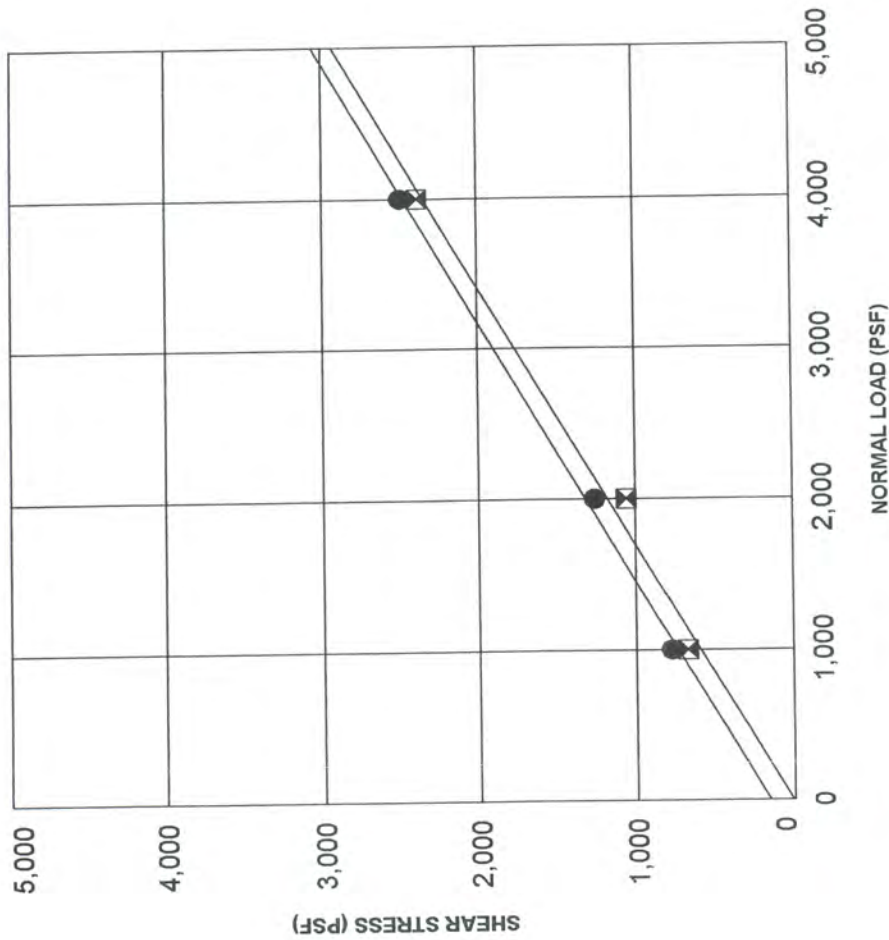


SAMPLE DESCRIPTION: SM  BORING NO.: B-2  DEPTH INTERVAL (ft): 10.0 - 11.5	HEIGHT (in): 1.0	DRY DENSITY (pcf): 113.0	PEAK ●	ULTIMATE ☒
	AREA (sq in): 4.58	INITIAL MOISTURE (%): 6.9	104	52
	SHEAR RATE (in/min): 0.002	FINAL MOISTURE (%): 16.2	37	34
	COHESION (psf)		FRICTION ANGLE (deg)	

NOTES:

# Direct Shear Test Results PARKING STRUCTURE V.A. MEDICAL CENTER LOMA LINDA, CALIFORNIA

## GEOBASE



PEAK ●  
ULTIMATE ✕

COHESION (psf) 157  
FRICTION ANGLE (deg) 30

DRY DENSITY (pcf): 98.4  
INITIAL MOISTURE (%): 7.2  
FINAL MOISTURE (%): 22.9

HEIGHT (in): 1.0  
AREA (sq in): 4.58  
SHEAR RATE (in/min): 0.002

SAMPLE DESCRIPTION: SM

BORING NO.: B-2

DEPTH INTERVAL (ft): 20.0 - 21.5

NOTES:

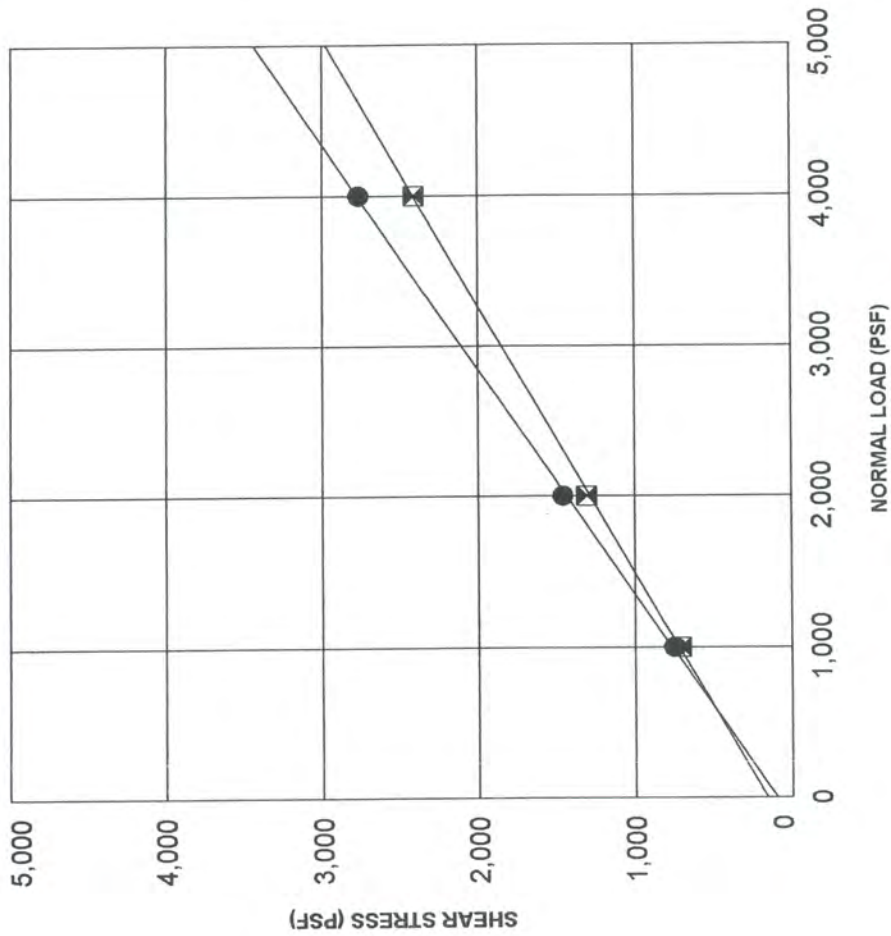
**Direct Shear Test Results**  
PARKING STRUCTURE  
V.A. MEDICAL CENTER  
LOMA LINDA, CALIFORNIA

**GEOBASE**

P.322.01.00

Figure C-5





SAMPLE DESCRIPTION: SM BORING NO.: B-2 DEPTH INTERVAL (ft): 30.0 - 31.5	HEIGHT (in): 1.0 AREA (sq in): 4.58 SHEAR RATE (in/min): 0.002	DRY DENSITY (pcf): 100.5 INITIAL MOISTURE (%): 9.2 FINAL MOISTURE (%): 22.6	PEAK ●	ULTIMATE ☒
			COHESION (psf) 98	161
			FRICTION ANGLE (deg) 34	29

NOTES:

# Direct Shear Test Results PARKING STRUCTURE V.A. MEDICAL CENTER LOMA LINDA, CALIFORNIA

# GEOBASE

P.322.01.00

Figure C-6

**EXPANSION POTENTIAL**  
ASTM D4820/U.B.C. No. 29-2

SOIL SAMPLE LOCATION (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-1 at 1.0 to 5.0	0	Very Low
B-2 at 1.0 to 5.0	7	Very Low

**WATER-SOLUBLE SULFATES**  
CAL. 417-A

SOIL SAMPLE LOCATION (feet)	SOLUBLE SULFATES PPM	POTENTIAL FOR ATTACK ON CONCRETE
B-1 at 1.0 to 5.0	33	Low
B-2 at 1.0 to 5.0	62	Low

**CORROSIVITY SERIES TEST**

SOIL SAMPLE LOCATION (feet)	pH (CAL 747)	SOLUBLE CHLORIDE (CAL.422) (PPM)	ELEC. RESISTIVITY (CAL.643) (OHM-CM)	POTENTIAL FOR ATTACK ON STEEL (SENATOROFF)
B-1 at 1.0 to 5.0	8.2	35	2,803	Moderate
B-2 at 1.0 to 5.0	8.0	40	672	Severe

# ANAHEIM TEST LABORATORY

3008 S. ORANGE AVENUE  
SANTA ANA, CALIFORNIA 92707  
PHONE (714) 549-7267

TO: GEOBASE:  
23362 PERALTA DR. #4&6  
LAGUNA HILLS, CA. 92653

ATTN: BOB PEARSON

DATE: 11/01/06

P.O. No. VERBAL

Shipper No.

Lab. No. B-0019

Specification:

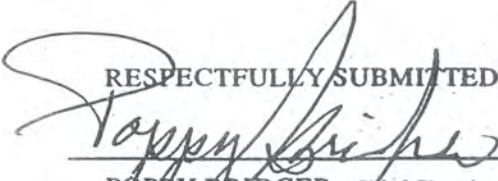
Material: SOIL

PROJECT: #P.322.01.00

## ANALYTICAL REPORT

### CORROSION SERIES SUMMARY OF DATA

pH	SOLUBLE SULFATES per CA. 417 ppm	SOLUBLE CHLORIDES per CA. 422 ppm	MIN. RESISTIVITY per CA. 643 ohm-cm
#1 BOR. #1 8.2 @ 5'-10'	33	35	2,803
#2 BOR, #2 8.0 @ 5'-10'	62	40	672

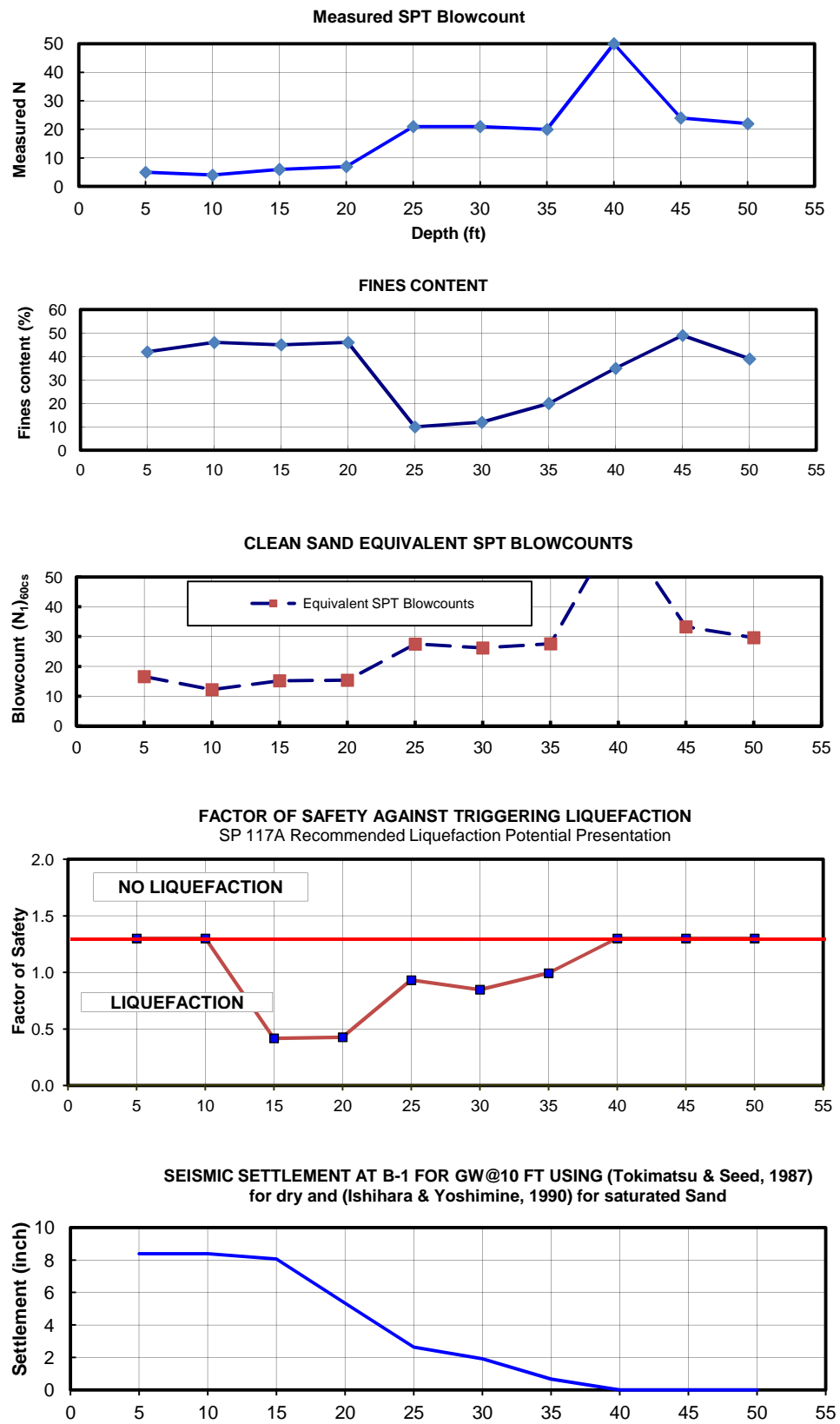
RESPECTFULLY SUBMITTED  
  
POPPY BRIDGER Chief Chemist

## APPENDIX D

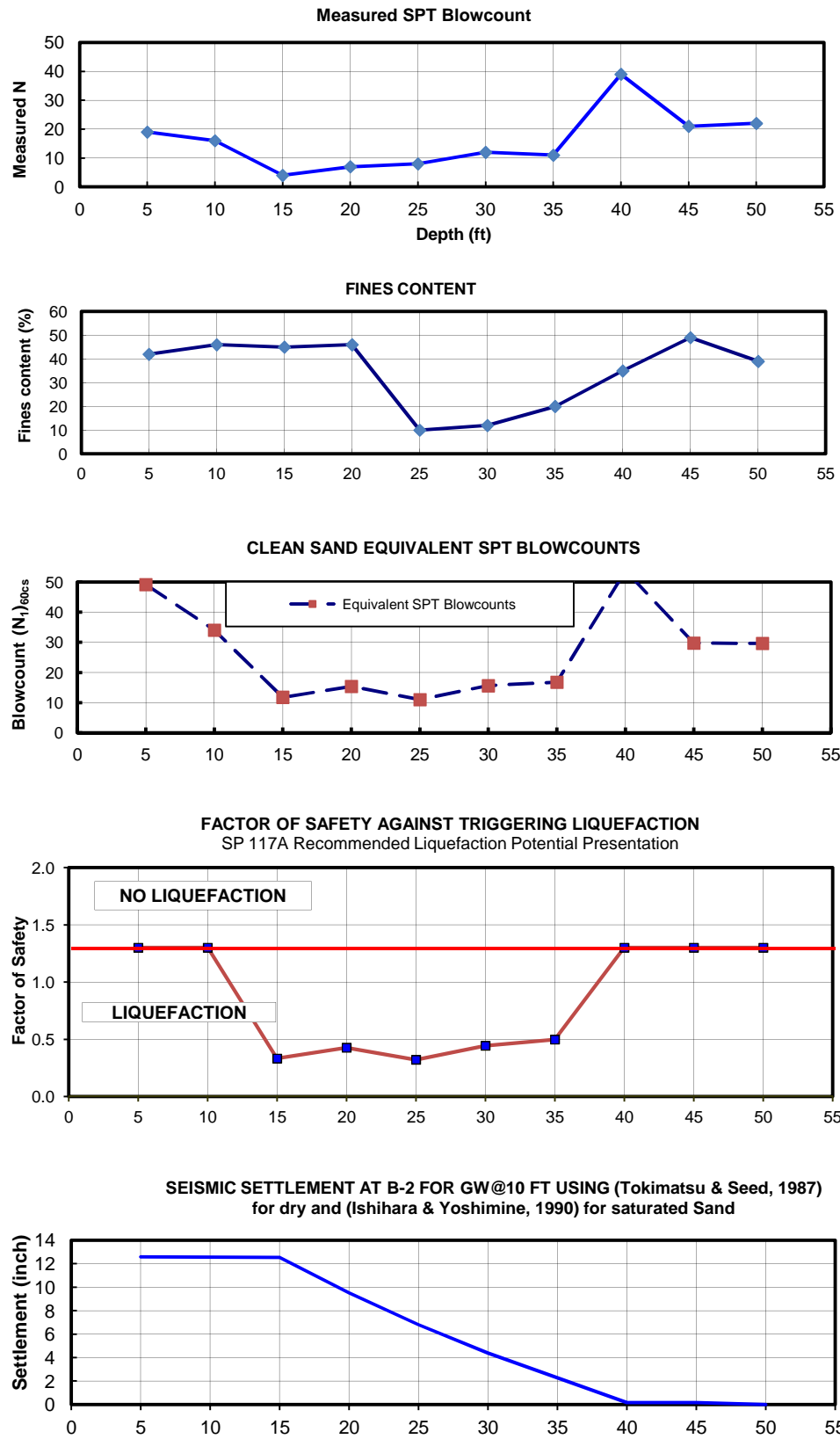
### Liquefaction and Seismic Settlement Analyses



VA Loma Linda BORING B-1 @ GW 10 FT



## VA Loma Linda BORING B-2@ GW 10 FT



Design GW Depth: 10.0 feet  
Measured GW Depth: 100.0 feet  
Design GW Elev.: 1138.0 feet  
Measured GW Elev.: 1048.0  
Ground Surface Elev.: 1148 feet  
M<sub>s</sub>: 7.00  
PHGA MSF: 0.74 g  
1.19

P<sub>min</sub>: 1.0442717  
Y<sub>center</sub>: 62.427961 pcf  
Y: 115 pcf

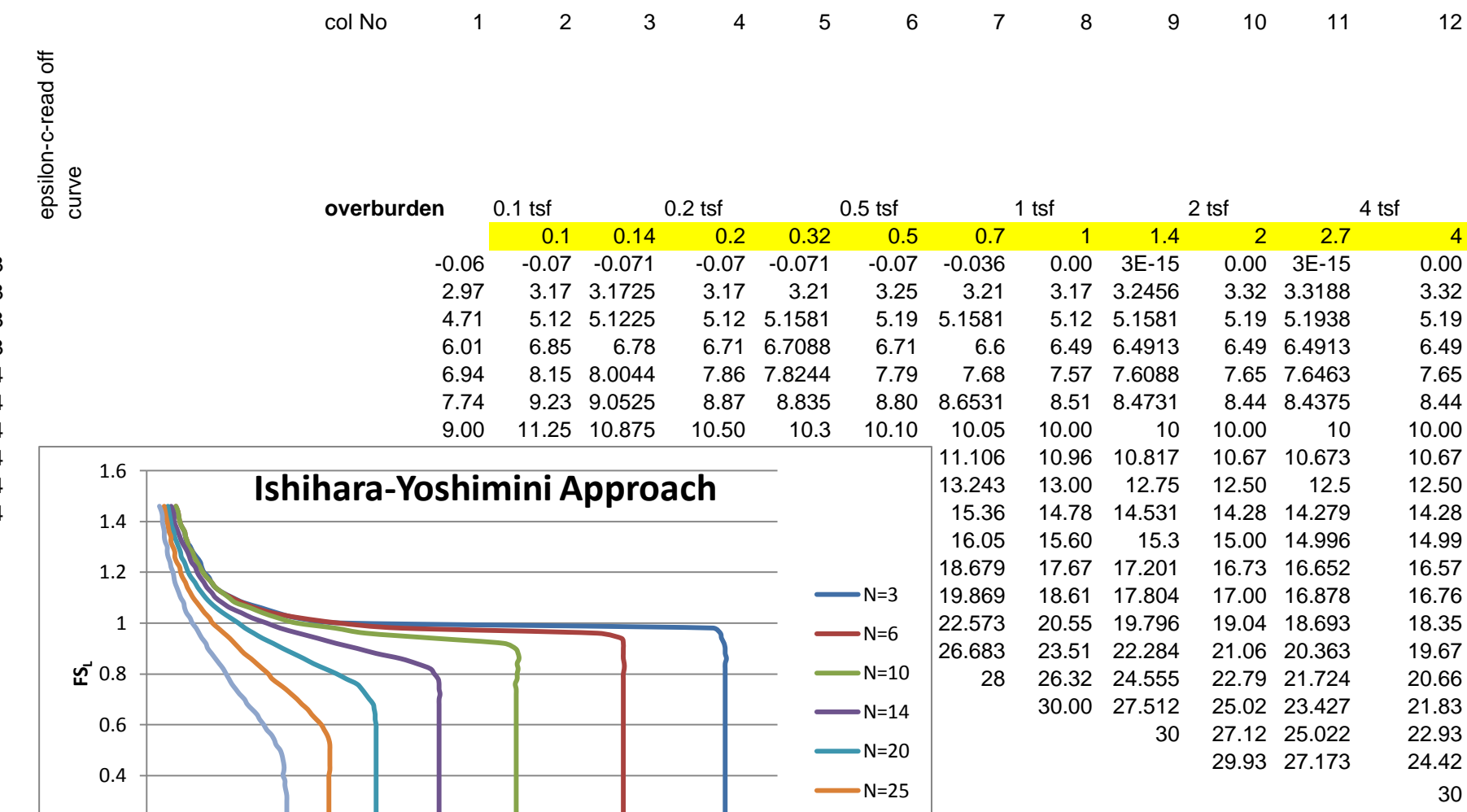
Dry Sand Seismic Settlement Calculations																								
Depth of layer base	Layer thickness (ft)	Average layer depth	Eff layer stress (tsf)	N - field	SPT depth	C <sub>u</sub>	C <sub>u</sub>	NI-60	FC(%)	alpha	beta	NI-60/sa	K2-max	G-max	r-d	gamma-e (Gae/G <sub>o</sub> )	gamma-e-calculated	epsilon-e (Max Z)-calculated	epsilon-c (M)	Z'epsilon-c (M)	settlement (in)	Cumulative settlement (inch)	Depth (ft) of top of layer	sig-v tsf
10	10	5	0.30	5	5	1.70	1	10	42	5.00	1.20	17	51	1E+06	0.990	2.27E-04	7.94E-04	1.06E-03	1.06E-03	2.12E-03	0.25	1.26	0.0	0.30
15	5	12.5	0.59	4	10	1.33	1	6	46	5.00	1.20	12	46	2E+06	0.974	3.49E-04	1.30E-03	2.72E-03	2.72E-03	5.45E-03	0.33	1.01	10.0	0.59
20	5	17.5	0.86	6	15	1.10	1	9	45	5.00	1.20	15	50	2E+06	0.963	3.88E-04	9.70E-04	1.49E-03	1.49E-03	2.39E-03	0.18	0.68	15.0	0.86
25	5	22.5	1.13	7	20	0.96	1	9	46	5.00	1.20	15	50	2E+06	0.960	4.38E-04	1.14E-03	1.71E-03	1.71E-03	3.42E-03	0.21	0.50	20.0	1.13
30	5	27.5	1.41	21	25	0.86	1	26	10	0.87	1.02	28	60	3E+06	0.932	3.94E-04	6.03E-04	3.38E-04	3.38E-04	6.76E-04	0.04	0.30	25.0	1.41
35	5	32.5	1.68	21	30	0.79	2	24	12	1.55	1.03	26	59	3E+06	0.907	4.26E-04	9.54E-04	5.83E-04	5.83E-04	1.17E-03	0.07	0.26	30.0	1.68
40	5	37.5	1.95	20	35	0.73	3	22	20	3.61	1.08	28	60	4E+06	0.872	4.34E-04	9.54E-04	5.33E-04	5.33E-04	1.07E-03	0.06	0.19	35.0	1.95
45	5	42.5	2.22	50	40	0.69	4	52	35	5.00	1.20	67	81	5E+06	0.828	3.26E-04	5.01E-04	2.95E-04	2.95E-04	5.91E-04	0.04	0.12	40.0	2.22
50	5	47.5	2.49	24	45	0.65	5	24	49	5.00	1.20	33	64	5E+06	0.778	4.11E-04	8.01E-04	3.31E-04	3.31E-04	6.62E-04	0.04	0.09	45.0	2.49
55	5	52.5	2.76	22	50	0.61	6	21	39	5.00	1.20	30	62	5E+06	0.727	4.21E-04	8.01E-04	3.97E-04	3.97E-04	7.95E-04	0.05	0.05	50.0	2.76
Total settlement (in)																					1.26			

Total Dry and Saturated Sand Seismic Settlement Calculations

Inpt. SPT Depth	Ground Elev.	Effective Vertical Stress at test location, $\sigma'_{vc}$	Total Vertical Stress, $\sigma_{vo}$	Stress Reduction Coefficient	Input depth of layer base	Cyclic Stress Ratio (CSR)	Input: Too Plastic to liquefy=1	Location Relative to GWT	Input measured representative SPT ( $N_{60}$ )	$C_u$	$(N_{60})_{cs} = N_{60} \times C_{u1} \times C_{u2} \times C_{u3} \times C_{u4} \times C_{u5}$					Calculated ( $N_{160}$ )		CRR <sub>7.5</sub> based on CPT Correlated SPT Blow Counts	Input measured or estimated fines (%)	for plotting only SPT Analysis	FS for Seismic Settlement	Dry Sand Settlement (inch)	Layer thickness (ft)	column Number	Saturated sand vol strain (%)	Cum. Total seismic Settlement (inch)		
											$\alpha$	$\beta$	$(N_{160})_{cs}$															
meters	Input depth (ft)	(ft)	(tsf)	(tsf)	$f_{qd}$	CSR $N_{eq7.5}$				$C_{u1}$	$C_{u2}$	$C_{u3}$	$C_{u4}$	$C_{u5}$														
1.52	5.00	1143.0	0.30	0.30	0.988	10	0.475	0	ABOVE GW	5	1.7	1.20	0.75	1.15	1.1	10	5.00	1.20	17	0.177	42	1.30	2.00	0.25	10	5	0.00	8.65
3.05	10.00	1138.0	0.59	0.59	0.977	15	0.470	0	ABOVE GW	4	1.3	1.20	0.75	1.15	1.1	6	5.00	1.20	12	0.134	46	1.30	2.00	0.33	5	3	0.00	8.39
4.57	15.00	1133.0	0.86	0.86	0.965	20	0.464	0	BELOW GW	6	1.1	1.20	0.85	1.15	1.1	9	5.00	1.20	15	0.162	45	0.42	0.42	0.00	5	5	4.53	8.07
6.10	20.00	1128.0	1.13	1.13	0.953	25	0.459	0	BELOW GW	7	1.0	1.20	0.85	1.15	1.1	9	5.00	1.20	15	0.164	46	0.43	0.43	0.00	5	5	4.53	5.35
7.62	25.00	1123.0	1.41	1.41	0.942	30	0.453	0	BELOW GW	21	0.9	1.20	0.95	1.15	1.1	26	0.87	1.02	28	0.354	10	0.93	0.93	0.00	5	11	1.20	2.63
9.14	30.00	1118.0	1.68	1.68	0.930	35	0.447	0	BELOW GW	21	0.8	1.20	0.95	1.15	1.1	24	1.55	1.03	26	0.318	12	0.85	0.85	0.00	5	10	2.08	0.91
10.67	35.00	1113.0	1.95	1.95	0.899	40	0.428	0	BELOW GW	20	0.7	1.20	1.00	1.15	1.1	22	3.61	1.08	28	0.356	20	0.99	0.99	0.00	5	9	1.10	0.66
12.19	40.00	1108.0	2.22	2.22	0.848	45	0.406	0	BELOW GW	50	0.7	1.20	1.00	1.15	1.1	52			67	too dense or clay rich to liquefy	35	1.30	2.00	0.00	5	14	0.00	0.00
13.72	45.00	1103.0	2.49	2.49	0.808	50	0.389	0	BELOW GW	24	0.6	1.20	1.00	1.15	1.1	24	5.00	1.20	33	too dense or clay rich to liquefy	49	1.30	2.00	0.00	5	10	0.00	0.00
15.24	50.00	1098.0	2.76	2.76	0.767	55	0.369	1	BELOW GW	22	0.6	1.20	1.00	1.15	1.1	21	5.00	1.20	30	too dense or clay rich to liquefy	39	1.30	2.00	0.00	5	9	0.00	0.00

Table 2: Corrections to SPT (Modified from Skempton, 1986) as Listed by Robertson and Wride (this report)

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C <sub>u</sub>	(σ' <sub>vo</sub> /σ' <sub>atm</sub> ) <sup>0.5</sup> C <sub>u</sub> ≤ 2
Energy ratio	Donut Hammer Safety Hammer Automatic-Trip Donut-Type Hammer	C <sub>e</sub>	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole diameter	65 mm to 115 mm 150 mm 200 mm	C <sub>d</sub>	1.0 1.05 1.15
Rod length	3 m to 4 m 4 m to 6 m 6 m to 10 m 10 m to 30 m > 30 m	C <sub>r</sub>	0.75 0.85 0.95 1.0 ≥1.0
Sampling method	Standard sampler Sampler without liners	C <sub>s</sub>	1.0 1.1 to 1.3



N=3	N=6			N=10			N=14			N=20			N=25			N=30		
3.6	5.4	7.2	9.6	12	14.4	16.8	20.4	24	27	30	33	36						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33						
5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1								



Design GW Depth: 10.0 feet  
Measured GW Elev.: 100.0 feet  
Design GW Elev.: 1138.0 feet  
Measured GW Elev.: 1048.0 feet  
Ground Surface Elev.: 1148 feet  
M<sub>s</sub>: 7.00  
PHGA: 0.74 g  
MSF: 1.19

Dry Sand Seismic Settlement Calculations

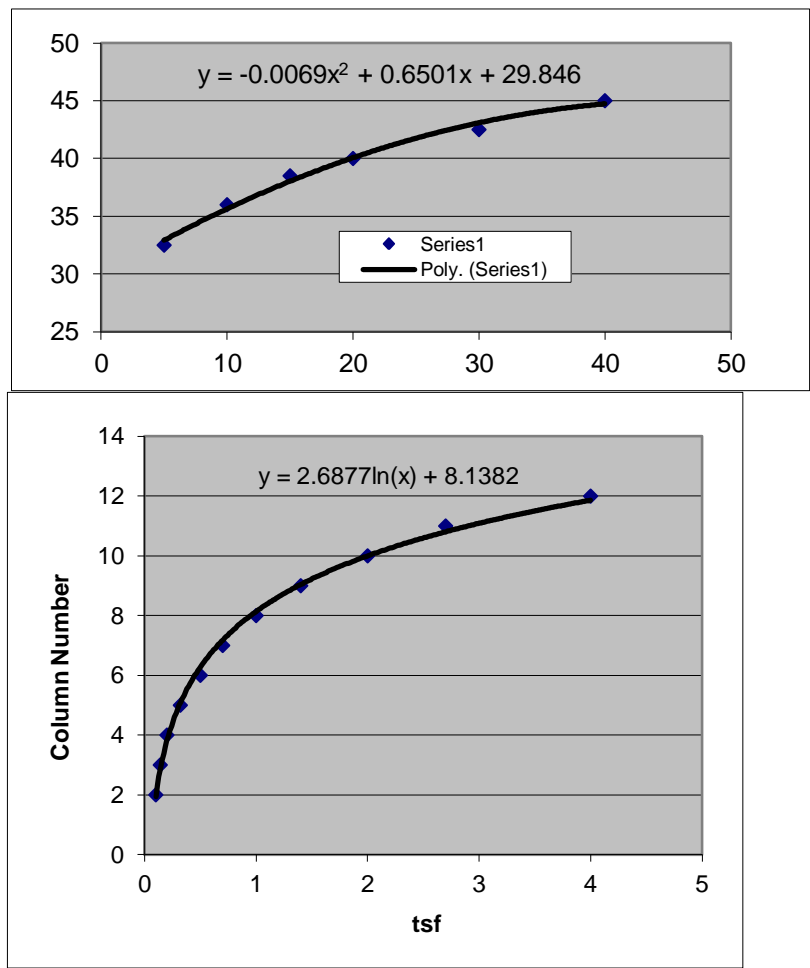
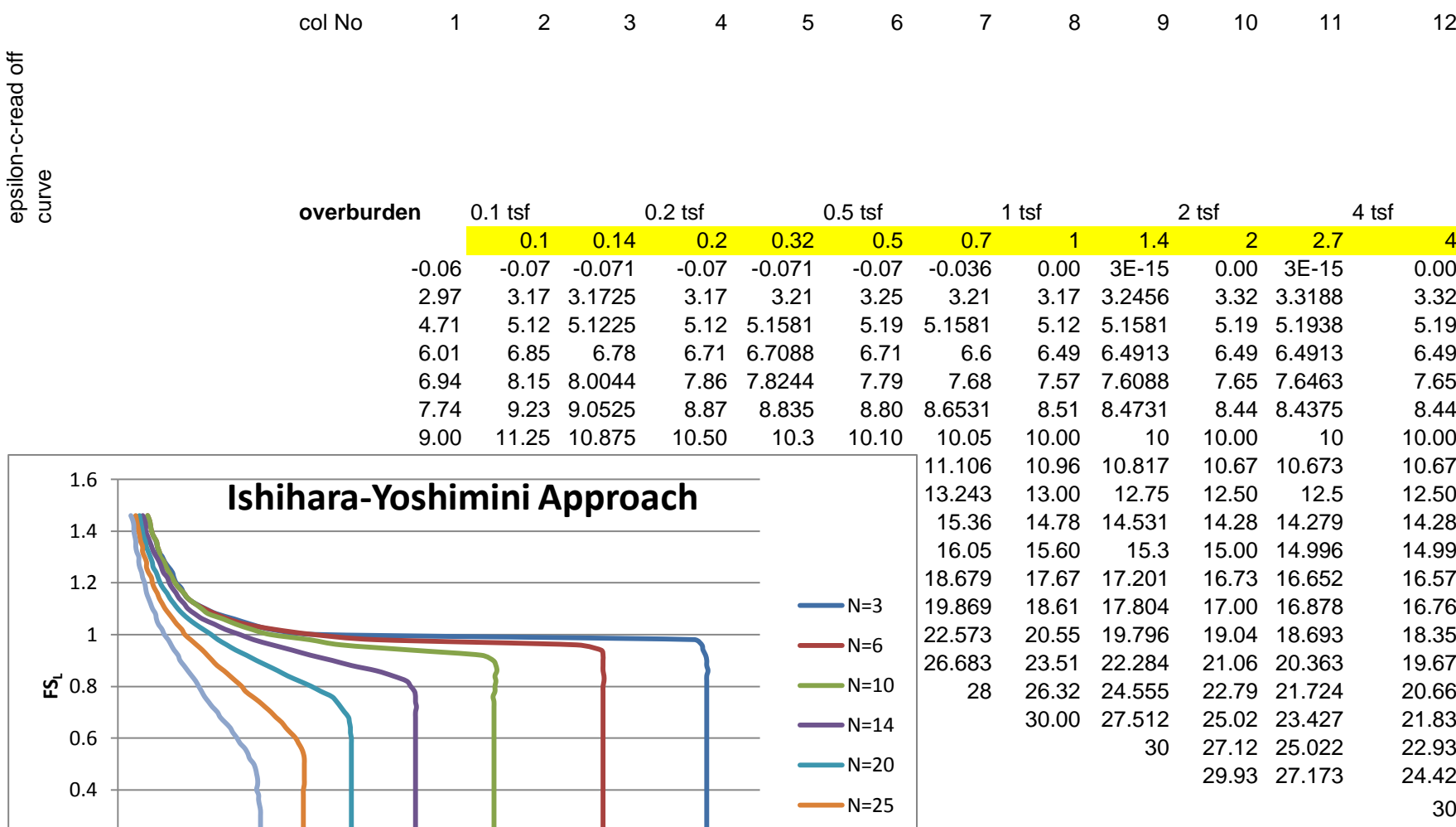
Depth of layer base	Layer thickness (ft)	Average layer depth	Eff layer stress (tsf)	N field	SPT depth	C <sub>u</sub>	C <sub>c</sub>	N1-60	FC(%)	alpha	beta	N1-60/ps	K2-max	Gmax	r-d	gamma-e (G-a-G, m)	gamma-e-calculated	epsilon-c (delta z / z)-calculated	epsilon-c (M)	Z epsilon-c (M)	settlement (in)	Cumulative settlement (inch)	Depth (ft) of top of layer	sig-v tsf	column Number	x	y	gamma-e-calculated	gamma-e-read from curve	epsilon-c-calculated	epsilon-c-read off curve				
10	10	5	0.30	19	5	1.70	1	37	42	5.00	1.20	49	73	2E+06	0.990	1.58E-04	3.02E-04	9.28E-05	9.28E-05	1.86E-04	0.02	1.90	0.0	0.30	5	12.0	14.8	3.0E-04	-1.0E+00	9.3E-05	15.36	14.78	14.531	14.28	14.279
15	5	12.5	0.59	16	10	1.33	1	24	46	5.00	1.20	34	65	2E+06	0.974	2.49E-04	4.47E-04	1.79E-04	1.79E-04	3.57E-04	0.02	1.88	10.0	0.59	7	14.0	16.5	4.5E-04	-1.0E+00	1.8E-04	18.679	17.67	17.201	16.73	16.652
20	5	17.5	0.86	4	15	1.10	1	6	45	5.00	1.20	12	46	2E+06	0.963	4.23E-04	1.81E-03	3.99E-03	3.99E-03	7.98E-03	0.48	1.85	15.0	0.86	8	16.3	22.6	1.8E-03	7.0E-04	4.0E-03	19.899	18.61	17.804	17.00	16.876
25	5	22.5	1.13	7	20	0.96	1	9	46	5.00	1.20	15	50	2E+06	0.950	4.38E-04	1.14E-03	1.71E-03	1.71E-03	3.42E-03	0.21	1.38	20.0	1.13	8	16.4	20.6	1.1E-03	1.9E-03	1.7E-03	22.573	20.55	19.796	19.04	18.693
30	5	27.5	1.41	8	25	0.86	1	10	10	0.87	1.02	11	44	2E+06	0.932	5.34E-04	1.69E-03	4.08E-03	4.08E-03	8.17E-03	0.49	1.17	25.0	1.41	9	17.3	22.3	1.7E-03	6.0E-03	4.1E-03	26.683	23.51	22.284	21.06	20.363
35	5	32.5	1.68	12	30	0.79	2	14	12	1.55	1.03	16	50	3E+06	0.907	5.06E-04	1.69E-03	2.49E-03	2.49E-03	4.98E-03	0.30	0.68	30.0	1.68	10	17.0	22.3	1.7E-03	1.0E+00	2.5E-03	28	26.32	24.555	22.79	21.724
40	5	37.5	1.95	11	35	0.73	3	12	20	3.61	1.08	17	51	3E+06	0.872	5.12E-04	1.69E-03	2.22E-03	2.22E-03	4.44E-03	0.27	0.38	35.0	1.95	10	17.1	22.3	1.7E-03	2.0E+00	2.2E-03	30.00	27.512	25.02	23.497	
45	5	42.5	2.22	39	40	0.69	4	41	35	5.00	1.20	54	75	5E+06	0.828	3.52E-04	5.01E-04	1.64E-04	1.64E-04	3.28E-04	0.02	0.11	40.0	2.22	10	15.5	17.0	5.0E-04	3.0E+00	1.6E-04	11.106	10.96	10.817	10.67	10.673
50	5	47.5	2.49	21	45	0.65	5	21	49	5.00	1.20	30	62	4E+06	0.778	4.27E-04	8.01E-04	3.94E-04	3.94E-04	7.89E-04	0.05	0.10	45.0	2.49	11	16.3	19.0	8.0E-04	4.0E+00	3.9E-04	13.243	13.00	12.75	12.50	12.5
55	5	52.5	2.76	22	50	0.61	6	21	39	5.00	1.20	30	62	5E+06	0.727	4.21E-04	8.01E-04	3.97E-04	3.97E-04	7.95E-04	0.05	0.05	50.0	2.76	11	16.2	19.0	8.0E-04	5.0E+00	4.0E-04	16.05	15.60	15.3	15.00	14.996
Total settlement (in)																					1.90														

Total Dry and Saturated Sand Seismic Settlement Calculations

Inpt SPT Depth	Ground Elev.	Effective Vertical Stress at test location, σ'vo	Total Vertical Stress, σvo	Stress Reduction Coefficient	Input depth of layer base	Cyclic Stress Ratio (CSR)	Input: Too Plastic to liquefy=1	Location Relative to GWT	Input measured representat ve SPT (N60s)	Calculated (N60s)					(N60s) = N'Cv-Cp-Cp-Cp-Cp-Cp					Calculated (N60s)	CRR75 based on CPT Correlated SPT Blow Counts		Input measured or estimated fines (%)	for plotting only SPT Analysis
Input depth	calc'd					CSR				Cu	Cc	Cu	Cc	Cc		α	β	(N60s)						
meters (ft)	(ft)	(tsf)	(tsf)	rd	(ft)																			
1.52	5.00	1143.0	0.30	0.30	0.988	10	0.475	0	ABOVE GW	19	1.7	1.20	0.75	1.15	1.1	37	5.00	1.20	49	too dense or clay rich to liquefy	42	1.30		
3.05	10.00	1138.0	0.59	0.59	0.977	15	0.470	0	ABOVE GW	16	1.3	1.20	0.75	1.15	1.1	24	5.00	1.20	34	too dense or clay rich to liquefy	46	1.30		
4.57	15.00	1133.0	0.86	0.86	0.965	20	0.464	0	BELOW GW	4	1.1	1.20	0.85	1.15	1.1	6	5.00	1.20	12	too dense or clay rich to liquefy	45	0.33		
6.10	20.00	1128.0	1.13	1.13	0.953	25	0.459	0	BELOW GW	7	1.0	1.20	0.85	1.15	1.1	9	5.00	1.20	15	too dense or clay rich to liquefy	45	0.43		
7.62	25.00	1123.0	1.41	1.41	0.942	30	0.453	0	BELOW GW	8	0.9	1.20	0.95	1.15	1.1	10	0.87	1.02	11	too dense or clay rich to liquefy	10	0.32		
9.14	30.00	1118.0	1.68	1.68	0.930	35	0.447	0	BELOW GW	12	0.8	1.20	0.95	1.15	1.1	14	1.55	1.03	16	too dense or clay rich to liquefy	12	0.44		
10.67	35.00	1113.0	1.95	1.95	0.899	40	0.428	0	BELOW GW	11	0.7	1.20	1.00	1.15	1.1	12	3.61	1.08	17	too dense or clay rich to liquefy	17	0.50		
12.19	40.00	1108.0	2.22	2.22	0.848	45	0.408	0	BELOW GW	39	0.7	1.20	1.00	1.15	1.1	41	5.00	1.20	54	too dense or clay rich to liquefy	35	1.30		
13.72	45.00	1103.0	2.49	2.49	0.808	50	0.389	0	BELOW GW	21	0.6	1.20	1.00	1.15	1.1	21	5.00	1.20	30	too dense or clay rich to liquefy	49	1.30		
15.24	50.00	1098.0	2.76	2.76	0.767	55	0.369	1	BELOW GW	22	0.6	1.20	1.00	1.15	1.1	21	5.00	1.20	30	too dense or clay rich to liquefy	39	1.30		

Table 2. Corrections to SPT (Modified from Skempton, 1986) as Listed by Robertson and Wride (this report)

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C <sub>u</sub>	(P <sub>u</sub> /σ' <sub>vo</sub> ) <sup>0.5</sup> C <sub>u</sub> ≤ 2
Energy ratio	Donut Hammer Safety Hammer Automatic-Trip Donut-Type Hammer	C <sub>e</sub>	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole diameter	65 mm to 115 mm 150 mm 200 mm	C <sub>d</sub>	1.0 1.05 1.15
Rod length	3 m to 4 m 4 m to 6 m 6 m to 10 m 10 m to 30 m > 30 m	C <sub>r</sub>	0.75 0.85 0.95 1.0 ~1.0
Sampling method	Standard sampler Sampler without liners	C <sub>s</sub>	1.0 1.1 to 1.3



N=3	N=6			N=10			N=14			N=20			N=25			N=30		
	3.6	5.4	7.2	9.6	12	14.4	16.8	20.4	24	27	30	33	36					
0	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.02	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.04	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.06	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.08	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.1	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.12	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.14	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.16	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.18	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.2	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.22	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.24	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.26	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.28	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.3	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.32	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.34	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.36	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.38	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.532	1.33					
0.4	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.514	1.294					
0.42	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.963	1.743	1.523	1.303					
0.44	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.963	1.743	1.523	1.303					
0.46	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.963	1.743	1.5185	1.294					
0.48	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.963	1.743	1.5135	1.284					
0.5	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.963	1.743	1.5045	1.266					
0.52	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.963	1.743	1.496	1.229					
0.54	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9585	1.734	1.4725	1.211					
0.56	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9495	1.716	1.4495	1.183					
0.58	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.9355	1.688	1.413	1.138					
0.6	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4815	2.183	1.922	1.661	1.3855	1.11					
0.62	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.477	2.174	1.8945	1.615	1.34	1.073					
0.64	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.477	2.174	1.8715	1.569	1.3075	1.046					
0.66	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4725	2.165	1.8485	1.532	1.266	1.1					
0.68	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4675	2.1565	1.8255	1.5005	1.2265	1.065					
0.7	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4625	2.1475	1.8025	1.4645	1.1865	1.025					
0.72	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4575	2.1385	1.7795	1.4275	1.1495	0.9875					
0.74	5.506	5.019	4.532	4.023	3.514	3.147	2.78	2.4525	2.1295	1.7565	1.3905	1.1125	0.9505					
0.76	5.506	5.019	4.532	4.0185	3.505	3.1425	2.78	2.39	2.12825	1.70325	1.32025	1.0807						
0.78	5.506	5.019	4.532	4.0275	3.523	3.147	2.771	2.335	1.899	1.548	1.193	0.9865	0.778					
0.8	5.506	5.019	4.532	4.0275	3.523	3.147	2.771	2.335	1.899	1.548	1.193	0.9865	0.778					
0.82	5.506	5.0235	4.541	4.0365	3.532	3.1415	2.697	2.197	1.697	1.399	1.101	0.913	0.725					
0.84	5.506	5.0235	4.541	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.86	5.506	5.0235	4.541	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.88	5.506	5.019	4.532	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.9	5.506	5.019	4.532	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.92	5.506	5.019	4.532	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.94	5.506	5.019	4.532	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.96	5.506	5.019	4.532	4.032	3.523	3.06	2.569	2.078	1.597	1.3165	1.046	0.867	0.688					
0.98	5.471	4.992	4.513	3.6235	2.734	2.179	1.624	1.3625	1.101	0.945	0.789	0.656	0.523					
1	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.02	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.04	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.06	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.08	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.1	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.12	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.14	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.16	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.18	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.2	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.22	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.24	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.26	5.471	4.9605	4.455	3.515	2.425	1.875	1.325	1.075	0.915	0.755	0.615	0.505	0.415					
1.28	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.3	5.459	4.8715	4.284	3.1835	2.085	1.575	1.144	1.229	1.018	0.876	0.734	0.614	0.495					
1.32	5.377	0.381	0.385	0.385	0.385	0.3575	0.33	0.307	0.284	0.2665	0.229	0.2015	0.174					
1.34	0.365	0.366	0.367	0.367	0.367	0.3395	0.312	0.289	0.266	0.2475	0.229	0.2017	0.165					
1.36	0.335	0.3558	0.358	0.358	0.358	0.326	0.294	0.2705	0.247	0.225	0.204	0.181	0.146					
1.38	0.333	0.333	0.333	0.333	0.333	0.3075	0.275	0.252	0.229	0.211	0.193	0.17	0.147					
1.4	0.311	0.311	0.312	0.312	0.312	0.2845	0.257	0.243	0.229	0.211	0.193	0.17	0.147					
1.42	0.282	0.2825	0.303	0.303	0.303	0.28	0.257	0.2385	0.22	0.205	0.183	0.165	0.147					
1.44	0.282	0.2825	0.303	0.303	0.303	0.28	0.257	0.2385	0.22	0.205	0.183	0.165	0.147					
1.46	0.271	0.273	0.275	0.275	0.275	0.252	0.229	0.2155	0.202	0.1835	0.165	0.142	0.119					