

**REPORT OF GEOTECHNICAL  
INVESTIGATION  
PROPOSED EMERGENCY DEPARTMENT AND  
POLICE DEPARTMENT EXPANSION**

**JERRY L. PETTIS MEMORIAL VETERANS MEDICAL CENTER  
11201 BENTON STREET  
LOMA LINDA, CALIFORNIA**

**Prepared for:**

**EWINGCOLE**

**Irvine, California**

**January 11, 2013**

**Project 4953-12-1321**



January 11, 2013

Ms. Ruby Lynn Carr  
EwingCole  
15231 Laguna Canyon Road, Suite 200  
Irvine, California 92618

Subject: **LETTER OF TRANSMITTAL**  
**Report of Geotechnical Investigation**  
**Proposed Emergency Department and Police Department Expansion**  
**Jerry L. Pettis Memorial Veterans Medical Center**  
**11201 Benton Street**  
**Loma Linda, California**

Dear Ms. Carr:

We are pleased to submit this report presenting the results of our geotechnical investigation for the proposed Emergency Department and Police Department Expansion to be constructed on the campus of the Jerry L. Pettis Memorial Veterans Medical Center (VA Loma Linda Healthcare System) in Loma Linda, California. This investigation was conducted in general accordance with our proposal dated November 14, 2012, as authorized by your office on November 15, 2012.

The scope of our services was planned with Mr. Vahid Tavakoulania of your office. Mr. Tavakoulania has provided us with prior geotechnical reports near the site and site plans for the proposed project.


The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.


Ms. Ruby Lynn Carr  
January 11, 2013  
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It has been a pleasure to be of professional service to you. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,

AMEC Environment & Infrastructure, Inc.

  
Alek Harounian  
for Project Engineer  
with permission

  
Mark A. Murphy  
Associate Geotechnical Engineer  
Project Manager



Reviewed By:



Marshall Lew, Ph.D.  
Principal Engineer - Geotechnical



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## **EXECUTIVE SUMMARY**

We have completed our geotechnical investigation of the site of the proposed Emergency Department and Police Department Expansion to be constructed at the campus of the Jerry L. Pettis Memorial Veterans Medical Center (VA Loma Linda Healthcare System) in Loma Linda, California. Our subsurface explorations, engineering analyses, and foundation design recommendations are summarized below.

VA Loma Linda Medical Center is planning to construct a new addition immediately adjacent to the north side of the existing hospital building. The addition is currently planned to be a 1-level structure but might be expanded to a 4-story structure in the future. No subterranean level is planned. The addition will be approximately 4,200 square feet in plan area.

We explored the soil conditions by drilling two borings at the site; fill soils, up to 11 feet deep were encountered in the borings drilled at the site. The fill consists primarily of sandy silt. The fill is underlain by natural soils consisted primarily of loose silty sand extending to a depth of 30 feet below grade. Medium dense to dense layers of silty sand soils including some poorly and well graded sand and sandy silt deposits are located below a depth of 30 feet. Groundwater was not encountered within the 76-foot maximum depth explored by our exploratory borings at the site. The corrosion studies performed as part of the previous investigations indicate that the on-site soils are severely corrosive to ferrous metals and that the potential for sulfate attack on portland cement concrete is considered negligible.

Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

Based on the historic-high groundwater level and the measurements from prior explorations, the potential for liquefaction and liquefaction-induced settlement is considered low; however, we estimate that the seismically-induced settlement above the historic-high groundwater level could be up to 7½ inches. The site is relatively level and the absence of nearby slopes precludes slope stability hazards. The potential for other geologic hazards such as tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

The existing fill soils are not considered suitable for support of the proposed Emergency Department and Police Department Expansion foundations, floor slab, or of exterior concrete walks and slabs on grade. In addition, due to the loose to medium dense nature of the underlying natural soils, relatively large static and seismic settlements are anticipated, as discussed previously. Therefore, we recommend that the proposed additions be supported on drilled cast-in-place concrete piles extending through the existing fill and the upper loose and medium dense natural soils. The floor slab for the building supported in this manner should be structurally supported.

Ideally, all existing fill soils should be removed and replaced as properly compacted fill to provide support for exterior walks and slabs on grade; however, if a greater than normal risk of distress to the exterior walks and slabs on grade is acceptable, the removal could be limited to the upper 2 feet of the uncertified fill soils. This removal should extend at least 2 feet laterally beyond the edge of

the exterior concrete walks and slabs on grade. In any case, distress to exterior walks and slabs on grade should be anticipated as a result of the estimated seismic settlement occurring in the event of the design earthquake. All required fill should be uniformly well compacted and observed and tested during placement.



## **1.0 SCOPE**

This report provides geotechnical design information for the proposed Emergency Department and Police Department Expansion to be constructed on the campus of the Jerry L. Pettis Memorial Veterans Medical Center (VA Loma Linda Healthcare System) in Loma Linda, California. The location of the proposed project site is shown on Figure 1, Vicinity Map. The location of the proposed expansion with respect to existing buildings, along with our exploration locations, is shown on Figure 2, Plot Plan.

EwingCole has provided us with prior geotechnical investigation reports prepared by Geobase, Inc. for a proposed Parking Structure and a proposed Outpatient Pharmacy. The recommendations in this report were developed in part using geotechnical information and laboratory test data from the previous investigations. Prior applicable data including boring logs and laboratory test results are presented in Appendix B of this report.

This investigation was authorized to evaluate the static physical characteristics of the soils underlying the site of the proposed project, and to provide recommendations for design of foundations, for floor slab support, and for grading. More specifically, the scope of this investigation included the following:

- Evaluate the current and prior subsurface explorations and laboratory tests, and provide a description of the soil and groundwater conditions encountered.
- Perform a geologic-seismic hazards evaluation of the site.
- Provide recommendations for design of foundations to be used for support of the proposed project together with the necessary design parameters, including frictional resistance, passive resistance, and anticipated total and differential settlements.
- Determine the applicable seismic design parameters based on the current California Building Code (CBC).
- Provide recommendations for floor slab support.
- Provide recommendations for grading, including site preparation, excavation and, the placing of any required compacted fill.

The scope of this investigation did not include the assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater of the site.

Our recommendations are based on the results of the current and previous field explorations, laboratory tests and appropriate engineering analyses. The results of our current field explorations and laboratory tests are presented in Appendix A. The results of the previous field investigations and laboratory test data are presented in Appendix B.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report. This report has been prepared for EwingCole and its design consultants to be used solely in the design of the proposed Emergency Department and Police Department Expansion. This report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

## **2.0 PROJECT DESCRIPTION AND SITE CONDITIONS**

We understand that the VA Loma Linda Healthcare System is planning to design and construct a new addition immediately adjacent to the north side of the existing hospital building in Loma Linda, California. The addition is currently planned to be a 1-level structure but might be expanded to a 4-story structure in the future. The addition will be approximately 4,200 square feet in plan area. No subterranean level is currently planned.

Structural loading information is not available to us at this time.

The existing hospital building, which contains no basement level and is predominantly supported on 36-inch diameter, 60- to 80-foot long drilled pile foundations, is located south of the proposed expansion. The proposed project site is currently being used as an asphalt-paved parking area with a small amount of landscaping. An asphalt-paved road crosses the site on the north. Some landscaped areas and water ponds are located adjacent and on the north and west sides of the proposed project site. Various underground utilities cross the site. The ground surface at the site is generally flat.

### **3.0 FIELD EXPLORATIONS AND LABORATORY TEST RESULTS**

The soil conditions beneath the project site were explored by drilling two borings to a depth of 76 feet below the existing grade. Data were also available from previous investigations performed by Geobase, Inc. for adjacent proposed projects. The locations of the current and relevant prior explorations are shown on Figure 2. Details of the current explorations and logs of the borings are presented in Appendix A. The results of the relevant prior explorations are presented in Appendix B.

Laboratory tests were performed on selected samples obtained from the current borings to aid in the classification of the soils and to evaluate the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations.
- Direct shear.
- Passing No. 200 sieve.

All testing was performed in general accordance with applicable ASTM specifications at the time of testing. Details of the current laboratory testing program and test results are presented in Appendix A. The results of the relevant prior laboratory tests by others are presented in Appendix B.

#### **4.0 SOIL AND GROUNDWATER CONDITIONS**

Fill soils, up to 11 feet deep, were encountered in our borings drilled at the site. The fill materials consist primarily of sandy silt and are not uniformly well compacted. Deeper fill could be present elsewhere at the site.

The fill is underlain by natural soils consisting primarily of loose silty sand extending to a depth of 30 feet below grade. Medium dense to dense layers of silty sand soils including some poorly and well graded sand and sandy silt deposits are located below a depth of 30 feet.

Groundwater was not encountered within the 76-foot maximum depth explored by our exploratory borings at the site.

The corrosion studies performed as part of the previous investigations indicate that the on-site soils are severely corrosive to ferrous metals and that the potential for sulfate attack on portland cement concrete is considered negligible. The previous corrosivity test results are presented in Appendix B.

## **5.0 LIMITED GEOLOGIC-SEISMIC HAZARDS EVALUATION**

Regionally, the site is located in the Peninsular Range geomorphic province. The Peninsular Range province is characterized by northwest/southeast trending alignments of mountains, hills and intervening basins (known as badlands), reflecting the influence of northwest trending major faults and folds, such as the nearby San Jacinto and Elsinore fault zones. The proposed site is located in the San Bernardino Valley on a broad alluvial fan bordering the San Timoteo Badlands to the south. The San Bernardino Mountains border the north side of the valley.

The geology of the region is shown in Figure 3, Regional Geologic Map. Figure 4, Regional Faults and Seismicity Map, shows major faults and earthquake epicenters in Southern California with respect to the project site.

### **Geologic Materials**

The proposed site is primarily underlain by unconsolidated alluvial material overlain by shallow artificial fill associated with previous grading at the site. The alluvial deposits are Holocene detrital material shed from the San Bernardino Mountains to the north.

Fill soils, up to 11 feet thick, were encountered in our borings. The fill soils consist predominantly of sandy silt. Deeper fill may be encountered at the project site from prior grading. Holocene age alluvial deposits below the fill consist of loose silty sand to 30 feet bgs; and medium dense to dense silty sand including some poorly and well graded sand and sandy silt to total depth. Beneath the younger alluvial deposits are older alluvial deposits consisting primarily of indurated clay-bearing deposits with gravelly, pebbly, and cobbly zones (USGS, 1963 and 1991).

### **Groundwater**

The site is in the southern boundary of the Bunker Hill Groundwater Subbasin of the Upper Santa Ana Valley Groundwater Basin. The alluvial materials beneath the site are part of the water-bearing deposits of the basin.

Borings for this investigation did not encounter groundwater within the maximum depth explored of 76 feet bgs.

State Groundwater Well No. 01S03W31A003S (California Department of Water Resources, 2012), located approximately 1.1 miles southeast of the site, recorded a high groundwater level of 64.9 feet bgs in 1945.

Based on the data discussed above, the historical maximum high groundwater level for the site is conservatively estimated to be deeper than 65 feet below the existing ground surface.

## **Faults**

Based on the available geologic data, active or potentially active faults with the potential for surface rupture are not known to be located beneath or projecting toward the site. The closest active fault to the site with the potential for surface rupture is the San Bernardino Loma Linda Section of the San Jacinto fault zone located approximately 0.3 miles to the southwest (USGS, 2012). Tables 1 and 2 list major active and potentially active faults, respectively.

According to the Loma Linda Seismic Safety Element (2009), the County of San Bernardino (2010), and the California Division of Mines and Geology (CDMG, 1977), the site is not either within a currently established Alquist-Priolo Earthquake Fault Zone (AP zone). The closest AP zone to the site is the San Jacinto fault zone located approximately one mile southwest of the site.

In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

## **Slope Stability**

According to the County of San Bernardino (2010) and the City of Loma Linda Seismic Safety Element (2009), the site is not located within an area of steep slopes and slope instability. The site of the proposed expansion is located on gently sloping ground with no slope stability problems. There is no potential for lurching (movement at right angles to a steep slope during strong ground shaking). Additionally, the property is not known to be on or in the path of any existing or potential landslide.

## **Liquefaction and Seismically-Induced Settlement**

Liquefaction potential is greatest where the groundwater level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the City of Loma Linda Seismic Safety Element (2009) and the County of San Bernardino (2010), the site is located within an area that has a potential for liquefaction (the County of San Bernardino indicates that the site is within a zone with "low" liquefaction potential). In addition, as previously stated, the historic-high groundwater level is conservatively estimated to be deeper than 65 feet beneath the site and groundwater was not encountered within the maximum 76-foot depth explored by our current borings. Accordingly, the potential for liquefaction adversely impacting the proposed project is considered to be low.

Seismic settlement is often caused by loose to medium-dense granular soils densified during ground shaking. Uniform settlement beneath a given structure would cause minimal damage. Dry and partially saturated soils as well as saturated granular soils are subject to seismically-induced settlement. Generally, differential settlements induced by ground failures such as liquefaction, flow slides, and surface ruptures would be much more severe than those caused by densification alone.

To evaluate the site-specific potential for seismically-induced settlement above the historic-high groundwater level, we have computed the peak ground acceleration (PGA) for the ground motion with a 2% probability of being exceeded in 50 years. This ground motion, which has a return period of 2,475 years, was corrected to be compatible with a Magnitude 7.5 earthquake. The Magnitude-7.5 compatible PGA was computed probabilistically using the average ground motions obtained from the Next Generation Attenuation (NGA) relationships of Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008). For all four NGA relationships, we have used an average shear wave velocity in the upper 30 meters equal to 310 meters per second based on our review of current exploratory borings and the available geologic data. We have used a depth to a shear wave velocity of 1,000 meters per second beneath the site ( $Z_{1.0}$ ) of 450 meters and a depth to a shear wave velocity of 2,500 meters per second ( $Z_{2.5}$ ) of 1 kilometer based on the available geologic data. To account for the uncertainty



in the ground motion attenuation relationships, each relationship was integrated to three standard deviations beyond the median.

We have used a PGA for our seismic settlement analyses that is  $\frac{2}{3}$  of the Magnitude-7.5 compatible PGA computed using EZ-FRISK for equivalence with the design level earthquake as defined in the 2010 California Building Code and ASCE 7-05. The Magnitude 7.5-compatible PGA computed in this manner for the subject site is 0.67g.

We have computed the potential for seismically-induced settlement above the postulated historic-high ground-water level in accordance with the methodology of Tokimatsu and Seed (1987) using the results of the Standard Penetration Tests (SPTs) performed in our borings at the site. Based on the results of our analyses, we estimate that the seismically-induced settlement could be up to 7½ inches beneath the site. Differential seismically-induced settlement could be up to 3 inches across the width of the proposed expansion. Summary of dry settlement evaluation for Borings 1 and 2 are presented on Figures 5 and 6, respectively.

### **Subsidence**

Extensive subsidence has occurred in the western portion of the Bunker Hill Groundwater Subbasin as a result of extraction of groundwater and reduction in artesian head. This subsidence extends to the project site as determined by the United States Geological Survey (Lofgren, 1971). Total subsidence in the vicinity of Loma Linda was measured at 1.3 feet from 1943 to 1968-1969 (Lofgren, 1971). Large scale subsidence is not expected to cause significant damage to individual structures. Recent measures by water authorizes have mitigated the over-extraction of groundwater to minimize broad areal subsidence. The potential for significant additional subsidence occurring beneath the site is considered low.

### **Tsunamis, Inundation, Seiches, and Flooding**

The site is not within a potential tsunami inundation hazard zone and the risk of tsunami affecting the site is low. The site is not located downslope of any large bodies of water that could adversely affect the site in the event of earthquake-induced seiches (wave oscillations in an enclosed or semi-enclosed body of water). The site is not in an area of flooding potential as defined by the County of San Bernardino Seismic Safety Element (2005) and the Federal Emergency Management Agency map #06071C8692H (FEMA, 2012).

## **Conclusions**

Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

Based on the historic-high groundwater level and the measurements from prior explorations, the potential for liquefaction and liquefaction-induced settlement is considered low; however, we estimate that the seismically-induced settlement above the historic-high groundwater level could be up to 7½ inches beneath the project site. The site is relatively level and the absence of nearby slopes precludes slope stability hazards. The potential for other geologic hazards such as inundation, tsunamis, seiches, flooding, and subsidence affecting the site is considered low. The site could be susceptible to subsidence due to nearby groundwater withdrawal based on USGS data. Such subsidence would be expected to be distributed over a wide region. The thickness of the sediments beneath the site which could be subject to subsidence is anticipated to be relatively thin so the potential for subsidence to impact structures at the site is considered relatively low. The potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

## **6.0 RECOMMENDATIONS**

### **6.1 GENERAL**

The existing fill soils are not considered suitable for support of the proposed Emergency Department and Police Department Expansion foundations, floor slab, or of exterior concrete walks and slabs on grade. In addition, due to the loose to medium dense nature of the underlying natural soils, relatively large static and seismic settlements are anticipated, as discussed previously. Therefore, we recommend that the proposed additions be supported on drilled cast-in-place concrete piles extending through the existing fill and the upper loose and medium dense natural soils. The floor slab for the building supported in this manner should be structurally supported.

Ideally, all existing fill soils should be removed and replaced as properly compacted fill to provide support for exterior walks and slabs on grade; however, if a greater than normal risk of distress to the exterior walks and slabs on grade is acceptable, the removal could be limited to the upper 2 feet of the uncertified fill soils. This removal should extend at least 2 feet laterally beyond the edge of the exterior concrete walks and slabs on grade. In any case, distress to exterior walks and slabs on grade should be anticipated as a result of the estimated seismic settlement occurring in the event of the design earthquake.

The area surrounding the building should be graded so that surface water is directed away from the building and conveyed to storm drains. Landscaping should be kept to a minimum and, if necessary, drought resistant planting requiring little irrigation should be used near the building.

All required fill should be uniformly well compacted and observed and tested during placement.

### **6.2 FOUNDATIONS**

#### General

As a result of the estimated seismic settlement, downdrag forces will be imposed on pile foundations. To mitigate the potential for downdrag, we recommend that the upper 30 feet of the piles be isolated from the surrounding soils using a sonotube (or equivalent). Axial pile capacities are presented below which neglect the capacity within the upper 30 feet. As an alternative to the

use of a sonotube (or equivalent), downdrag loads could be added to the structural loads imposed on the piles. Downdrag loads are also presented for use with this alternative.

### Axial Capacities

The downward and upward capacities of 24-, 30- and 36-inch-diameter drilled cast-in-place concrete piles are presented on Figure 7, Allowable Drilled Pile Capacities. The pile capacities shown on Figure 7 are dead-plus-live load capacities; a one-third increase may be used for wind or seismic loads. The capacities presented are based on the strength of the soil materials; the compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least 3 diameters on centers. If the piles are so spaced, no reduction in the axial capacities need be considered due to group action.

### Downdrag Loads

As previously discussed, if sonotubes (or equivalent) are not installed within the upper 30 feet, downdrag loads will be imposed on the piles. For a 24-inch diameter pile, a downdrag load of 190 kips should be added to the structural loads imposed on the pile when determining the required lengths from Figure 7. The downdrag loads on piles of different diameters may be assumed to be proportional to the diameter.

### Settlement

We estimate the static settlement of the proposed structures supported on deep foundations in the manner recommended to be about ¼ inch or less.

### Lateral Loads

Lateral loads may be resisted by the piles and by the passive resistance of the soils against grade beams and pile caps. We have computed the lateral capacities of the piles using the computer program LPILE by ENSOFT, Inc. Resistance of the soils adjacent to 24-, 30-, and 36-inch-diameter drilled piles are shown in the following tables for top of pile deflection of ¼ and ½ inch. An additional ¼-inch should be added to the pile head deflections in the following tables if

sonotubes (or equivalent) are installed. These resistances have been calculated assuming both fixed and free-head pile conditions. The lateral resistance of other sizes of piles may be assumed to be proportional to the pile diameter. All piles should be at least 40 feet long.

**Lateral Load Design Data  
 24-inch Drilled Concrete Pile**

	Pile Head Deflection (inches)			
	$\frac{1}{4}$		$\frac{1}{2}$	
Pile Head Condition	Free	Fixed	Free	Fixed
Lateral Load (kips)	30	85	64	160
Maximum Moment (in-kips)	1,500	4,600	2,900	8,900
Depth to Maximum Moment (ft)	7	0	7	0
Depth to Negligible Moment (ft)	19	22	19	22

**Lateral Load Design Data  
 30-inch Drilled Concrete Pile**

	Pile Head Deflection (inches)			
	$\frac{1}{4}$		$\frac{1}{2}$	
Pile Head Condition	Free	Fixed	Free	Fixed
Lateral Load (kips)	45	120	90	230
Maximum Moment (in-kips)	2,500	7,800	5,000	15,300
Depth to Maximum Moment (ft)	7½	0	7½	0
Depth to Negligible Moment (ft)	23	26	23	26

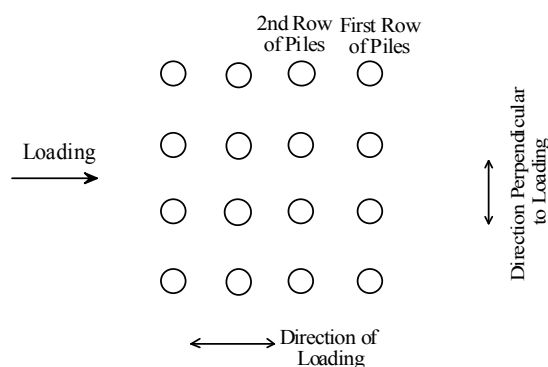
**Lateral Load Design Data  
 36-inch Drilled Concrete Pile**

	Pile Head Deflection (inches)			
	$\frac{1}{4}$		$\frac{1}{2}$	
Pile Head Condition	Free	Fixed	Free	Fixed
Lateral Load (kips)	60	160	120	320
Maximum Moment (in-kips)	3,900	12,500	7,700	23,900
Depth to Maximum Moment (ft)	8	0	8	0
Depth to Negligible Moment (ft)	26	30	26	30

By: LT 1/8/13 Checked: AH 1/9/13

For piles in groups spaced as shown below and at least 3 pile diameters on centers, no reduction in the lateral capacities need be considered for the first (leading) row of piles in the direction

perpendicular to loading. For subsequent rows in the direction of loading, piles in groups spaced closer than 8 pile diameters on centers will have a reduction in lateral capacity due to group effects. Therefore, the lateral capacity of piles in groups, except for the first row of piles, spaced at 3 pile diameters on centers, may be assumed to be reduced by half. The reduction of lateral capacity in the direction of loading for other pile spacing may be interpolated.



The passive resistance of properly compacted fill soils may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads.

No frictional resistances may be used between the building floor slab and the underlying fill soils.

The resistance of the piles and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

### Ultimate Values

The recommended values for drilled cast-in-place concrete pile design 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 following factors:

Design Item	Ultimate Design Factor
Axial Capacity of Piles	2.0
Lateral Capacity of Piles	1.0
Passive Pressure	1.5

In no event, however, should pile lengths be less than those required to support dead-plus-live loads when using the working stress design method.

### Pile Installation

It was not possible to observe caving during the drilling of our borings due to the use of a hollow stem auger. Based on the soil types encountered, potential for caving should be anticipated, particularly within the more sandy layers containing gravel. Therefore, precautions should be taken to reduce caving and raveling of the shaft side walls during installation. Piles spaced less than 8 feet on centers should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight. The concrete should be placed with special equipment so that the concrete is not allowed to fall freely more than 5 feet and to prevent concrete from striking the walls of the excavations.

Only competent drilling contractors with experience in the installation of drilled cast-in-place piles in similar soil conditions should be considered for the pile construction. We suggest requesting the piling contractor to submit a list of similar projects along with references for each project.

The drilling of the pile excavations and the placing of the concrete should be observed continuously by our personnel to verify that the desired diameter and depth of piles are achieved.

## **6.3 SEISMIC DESIGN PARAMETERS**

We have determined the seismic design parameters in accordance with the provisions of the 2010 California Building Code and ASCE 7-05 Standard (ASCE, 2005) using the United States Geological Survey (USGS, 2012) Web Application. The site location used was Latitude 34.0509° and Longitude -117.2505° with Site Class "D." The seismic parameters are presented in the following table:

### Seismic Parameters

Parameter	Value
$S_S$ (0.2 second period, Site Class B)	1.75g
$S_1$ (1.0 second period, Site Class B)	0.61g
Site Class	D
$F_a$	1.0
$F_v$	1.5
$S_{MS} = F_a S_S$ (0.2 second period)	1.75g
$S_{M1} = F_v S_1$ (1.0 second period)	0.91g
$S_{DS} = 2/3 \times S_{MS}$ (0.2 second period)	1.16g
$S_{D1} = 2/3 \times S_{M1}$ (1.0 second period)	0.61g

By: AH 1/8/13 Checked: MM 1/10/13

## 6.4 FLOOR SLAB SUPPORT

We recommend that the floor slab for the proposed building be structurally supported. However, it is anticipated that the exposed subgrade will be used as the bottom form for the structurally supported floor. If this is the case and vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4-inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
$\frac{3}{4}$ "	90–100
No. 4	0–10
No. 100	0–3

A low-slump concrete should be used to minimize possible curling of the slab. A 2-inch-thick layer of coarse sand should be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.



## **6.5 GRADING**

### **General**

The existing fill soils are not considered suitable for support of the exterior concrete walks and slabs on grade. Therefore, ideally, all existing fill soils should be removed and replaced as properly compacted fill to provide support for exterior walks and slabs on grade; however, if a greater than normal risk of distress to the exterior walks and slabs on grade is acceptable, the removal could be limited to the upper 2 feet of the uncertified fill soils. This removal should extend at least 2 feet laterally beyond the edge of the exterior concrete walks and slabs on grade. In any case, distress to exterior walks and slabs on grade should be anticipated as a result of the estimated seismic settlement occurring in the event of the design earthquake.

All required fill should be uniformly well compacted and observed and tested during placement.

### **Site Preparation**

After the site is cleared and the existing fill is excavated as recommended above, the exposed material should be carefully observed for the removal of all unsuitable and loose deposits. Next, the exposed material should be scarified to a depth of 6 inches, brought to near-optimum moisture content, and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

Good drainage of surface water away from the proposed addition should be provided by providing adequate slopes to all graded and paved surfaces. Such drainage will be important to minimize infiltration of water beneath floor slabs and pavement.

### **Compaction**

Any required fill should be placed in loose lifts not more than 8 inches thick and compacted. The fill should be compacted to at least 90% of the maximum density obtainable by the ASTM

Designation D1557 method of compaction. The moisture content of non-clayey soils at the time of compaction should vary no more than 2% below or above optimum moisture content.

### **Backfill**

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to minimize settlement of the backfill and to reduce settlement of overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction. The exterior grades should be sloped to drain away from the foundations to prevent ponding of water.

Some settlement of the backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

### **Material for Fill**

The fill soils at the site are not suitable for support of exterior concrete walks and slabs on grade, and should not be used within 2 feet of the subgrade for exterior walks, and other slabs on grade. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (at least 15% passing the No. 200 sieve) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by our personnel prior to being placed at the site.

## **6.6 GEOTECHNICAL OBSERVATION**

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of our firm. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The

representative should also observe proofrolling and delineation of areas requiring overexcavation.

- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe the installation of drilled cast-in-place concrete pile foundations.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

## **7.0 BASIS FOR RECOMMENDATIONS**

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our subsurface explorations and applicable data from the previous investigations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

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## **TABLES**



**Table 1**  
**Major Named Faults Considered to be Active**  
**in Southern California**

Fault (in increasing distance)	Maximum Magnitude	Slip Rate (mm/yr.)	Distance From Site (miles)	Direction From Site
San Jacinto (SB Section)	6.7 (a) SS	6.0	0.3	SW
San Andreas (SB N.Section)	7.5 (a) SS	22.0	7.1	NNE
Cucamonga	6.9 (a) RO	5.0	14	NW
San Andreas (Mojave S.Section)	7.4 (a) SS	29.0	22	NW
Elsinore (Glen Ivy Section)	6.8 (a) SS	5.0	23	SW
Chino-Central Avenue	6.7 (a) RO	1.0	24	WSW
San Jose	6.4 (a) RO	0.5	25	WNW
San Gabriel	7.2 (a) SS	1.0	27	NW
Sierra Madre	7.2 (a) RO	2.0	27	WNW
Whittier	6.8 (a) RO	2.5	27	WSW

By:PER 12/14/14 Chk: MAE 01/03/13

- |     |                 |    |                 |
|-----|-----------------|----|-----------------|
| (a) | CGS, 2003, 2008 | SS | Strike Slip     |
| (b) | Mark, 1977      | NO | Normal Oblique  |
| (c) | Slemmons, 1979  | RO | Reverse Oblique |
| (d) | Wesnousky, 1986 | BT | Blind Thrust    |

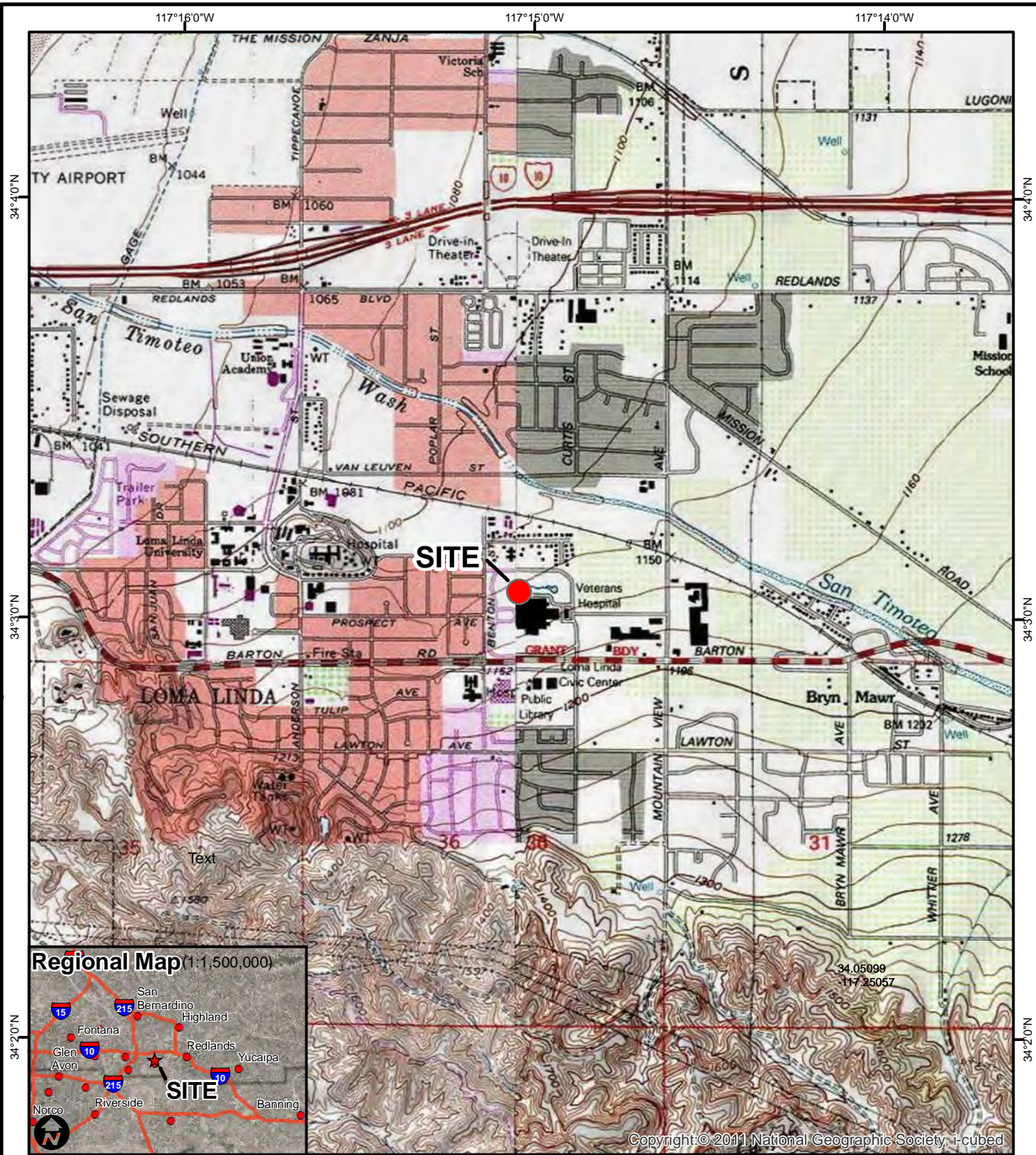
**Table 2**  
**Major Named Faults Considered to be Potentially Active**  
**in Southern California**

Fault (in increasing distance)	Maximum Magnitude	Slip Rate (mm/yr.)	Distance From Site (miles)	Direction From Site
Indian Hill	6.6 (b) RO	0.1	26	WNW
Peralta Hills	6.5 (b) RO	0.1	34	WSW
El Modeno	6.5 (b) NO	0.1	35	WSW
Duarte	6.7 (c) RO	0.1	37	WNW
Norwalk	6.7 (c) RO	0.1	41	W
Pelican Hill	6.3 (b) SS	0.1	46	WSW

By: PER 10/03/12 Chk: MAE 01/03/13

- |     |                 |    |                 |
|-----|-----------------|----|-----------------|
| (a) | CGS, 2003, 2008 | SS | Strike Slip     |
| (b) | Mark, 1977      | NO | Normal Oblique  |
| (c) | Slemmons, 1979  | RO | Reverse Oblique |
| (d) | Wesnousky, 1986 | BT | Blind Thrust    |

## **FIGURES**



Base: USGS 7.5 minute topographic maps, Redlands and San Bernardino South Quadrangles



0 500 1,000 2,000 3,000  
Feet

**amec**

AMEC  
Environment & Infrastructure, Inc.  
6001 Rickenbacker Road  
Los Angeles, California 90040  
Tel: 323.889.5300  
Fax: 323.721.6700

Proposed Emergency Department and Police Department Expansion  
Jerry L. Pettis Memorial Veterans Medical Center  
Loma Linda, California

LAT: 34.05099  
LON: -117.25057  
SCALE: 1:24,000  
DRAWN: PER  
CHECK: AH  
DATE: 01-08-13

VICINITY MAP

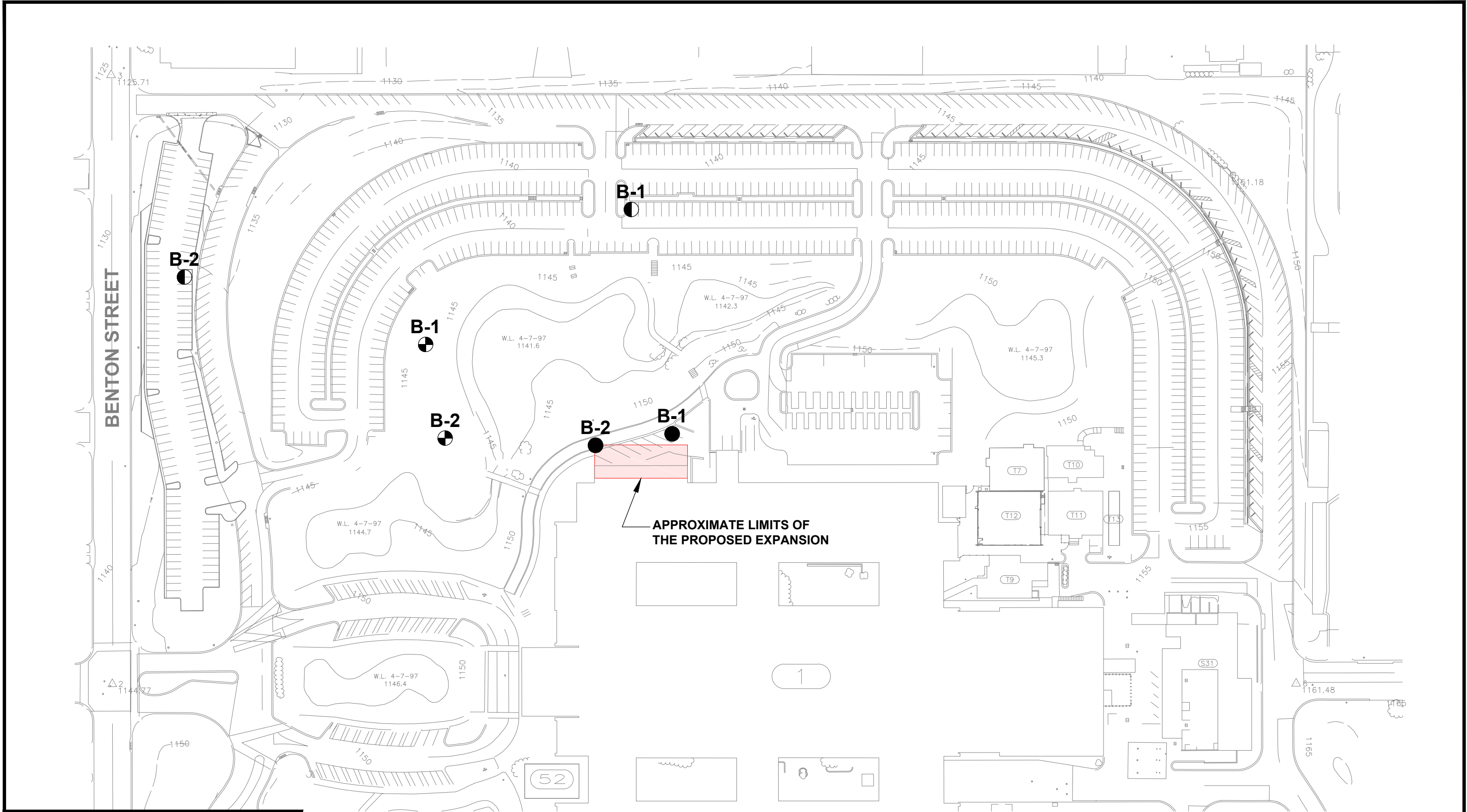
FIGURE:

**1**

PROJECT:  
4953-12-1321



Path: P:\953 Geotech\2012-projects\121321 VA Loma Linda Med. Ctr-ED Exp\CAD\DWG\4953-12-1321\_Fig-1\_PlotPlan.dwg [Fig-2]  
Date: January 08, 2013 - 10:40am By: vo.nguyen



## LEGEND

- B-2 ● CURRENT INVESTIGATION
- B-2 ● PRIOR INVESTIGATION (GEOBASE INC., 2012)
- B-2 ● PRIOR INVESTIGATION (GEOBASE INC., 2006)

0 50' 100' 200'  
SCALE: 1" = 100'



**amec**

AMEC  
Environment & Infrastructure, Inc.  
6001 Rickenbacker Road, Los Angeles, CA 90040  
Phone (323) 889-5300 Fax (323) 721-6700

## PLOT PLAN / EXPLORATION LOCATIONS

PROJECT: 4953-12-1321  
LT/LNG:  
SCALE: 1" = 100'  
DRAWN: V. Nguyen  
CHKD: A. Harounian  
PM: M. Murphy  
DATE: 12/06/2012

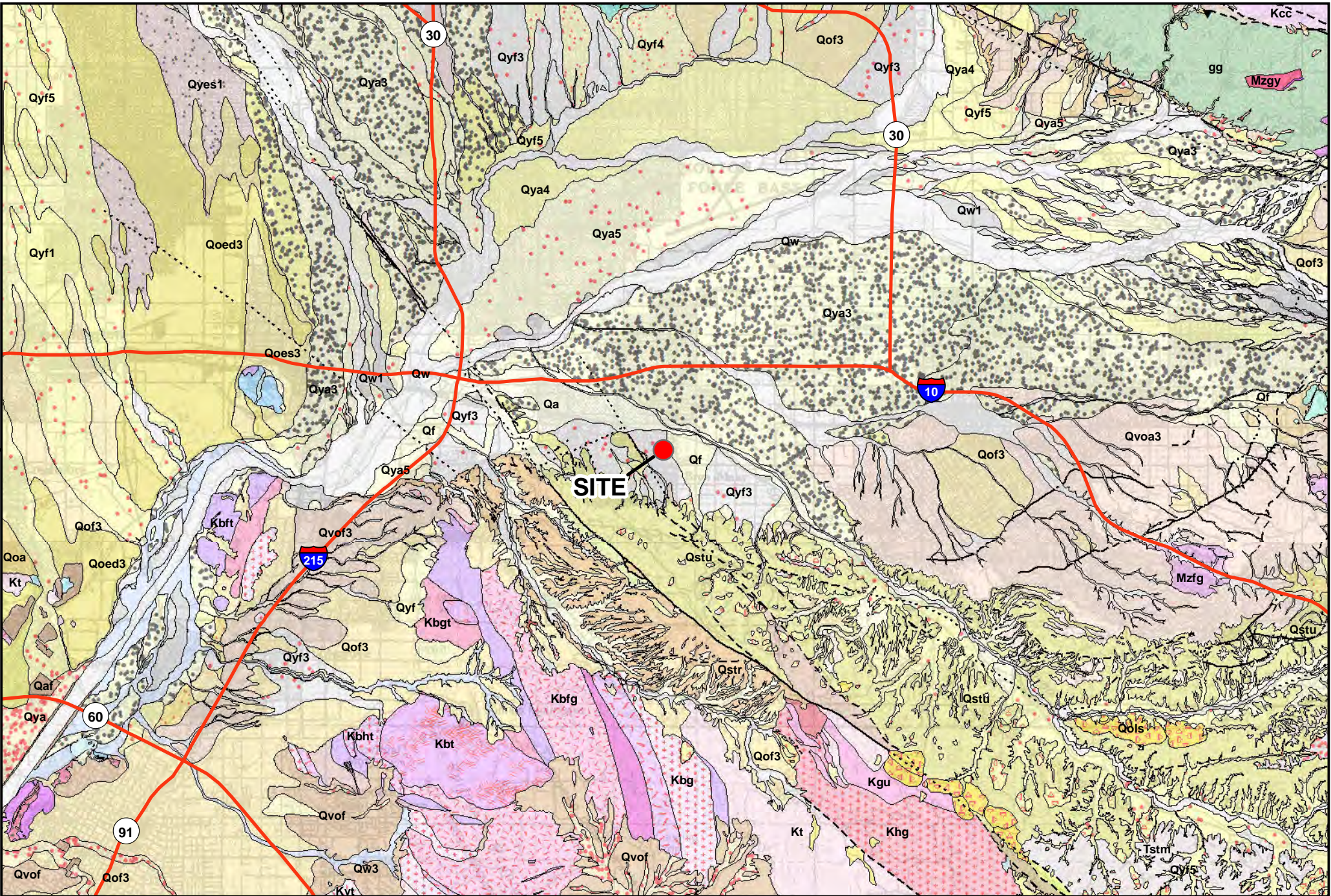
PROPOSED EMERGENCY DEPARTMENT &  
POLICE DEPARTMENT EXPANSION  
JERRY L. PETTIS MEMORIAL  
MEDICAL CENTER  
LOMA LINDA, CALIFORNIA

FIGURE NO.  
**2**



Geologic Units

- Qaf - Artificial fill
- Qc - Very young colluvial deposits
- Qa - Very young axial-channel deposits
- Qls - Very young landslide deposits
- Qf - Very young alluvial-fan deposits
- Qw - Very young wash deposits
- Qyw - Young wash deposits
- Qyls - Young landslide deposits
- Qya - Young axial-channel deposits
- Qyf - Young alluvial-fan deposits
- Qyed - Young eolian deposits (dune sand)
- Qyes - Young eolian deposits (sheet sand)
- Qoa - Old axial-channel deposits
- Qoed - Old eolian deposits (dune sand)
- Qof - Old alluvial-fan deposits
- Qols - Old landslide deposits
- Qvoa - Very old axial-channel deposits
- Qvof - Very old alluvial-fan deposits
- Qvols - Very old landslide deposits
- Qvor - Very old regolith
- Qvos - Very old surficial deposits
- Qstcq - San Timoteo Beds
- Qstr - San Timoteo Beds
- Qsts - San Timoteo Beds
- Qstu - San Timoteo Beds
- Tgh - Hypabyssal granitic rocks
- Tsg - Conglomerate, sandstone, and arkose
- Tstd - San Timoteo Beds
- Tstl - San Timoteo Beds
- Tstm - San Timoteo Beds
- gg - Gneissic granitoid rocks and gneiss
- Kba - Box Springs plutonic complex
- Kcc - Monzogranite of City Creek
- Kg - Granitic dikes
- Khg - Heterogeneous granitic rocks
- Kmgt - Monzogranite and tonalite, undifferentiated
- Kps - Pelona Schist
- Krg - Granite of Riverside area
- Kt - Tonalite, undifferentiated
- Kvt - Val Verde Pluton
- Mzfg - Foliated granitoid rocks
- Pzmp - Marble, Peninsular Ranges
- Pzmp - Marble, Peninsular Ranges
- Pzmp - Marble, Peninsular Ranges
- Pzms - Marble and schist, undifferentiated
- Pzsgp - Biotite schist and gneiss
- Pzsgp - Biotite schist and gneiss
- Pzsgp - Biotite schist and gneiss

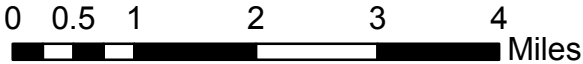
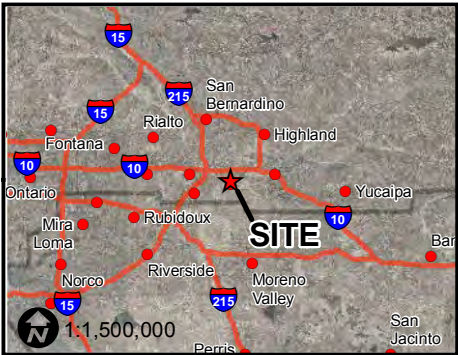



Morton, D.M., Miller, F.K., 2006, "Geologic Map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California", U.S. Geological Survey Open-File Report 2006-1217 Version 1.0.

Geologic Contacts

- contact, identity and existence certain, location accurate
- - contact, identity and existence certain, location approximate
- ..... contact, identity and existence certain, location concealed
- - - contact, identity and existence certain, location inferred
- fault, identity and existence certain, location accurate
- - fault, identity and existence certain, location approximate
- ..... fault, identity and existence certain, location concealed
- - - fault, identity and existence certain, location inferred
- fault, identity or existence questionable, location accurate
- ..... fault, identity or existence questionable, location concealed (Queried where contacts are questionable)

Regional Map



 <small>AMEC Environment &amp; Infrastructure, Inc. 6001 Rickenbacker Road Los Angeles, California 90040 Tel: 323.889.6300 Fax: 323.721.6700</small>	Proposed Emergency Department and Police Department Expansion Jerry L. Pettis Memorial Veterans Medical Center Loma Linda, California		
	LAT: 34.05099	FIGURE: <b>3</b>	
	LON: -117.25057	PROJECT: 4953-12-1321	
	SCALE: 1:100,000	REGIONAL GEOLOGIC MAP	
	DRAWN: PER		
CHECK: MAE			
DATE: 12-13-12			

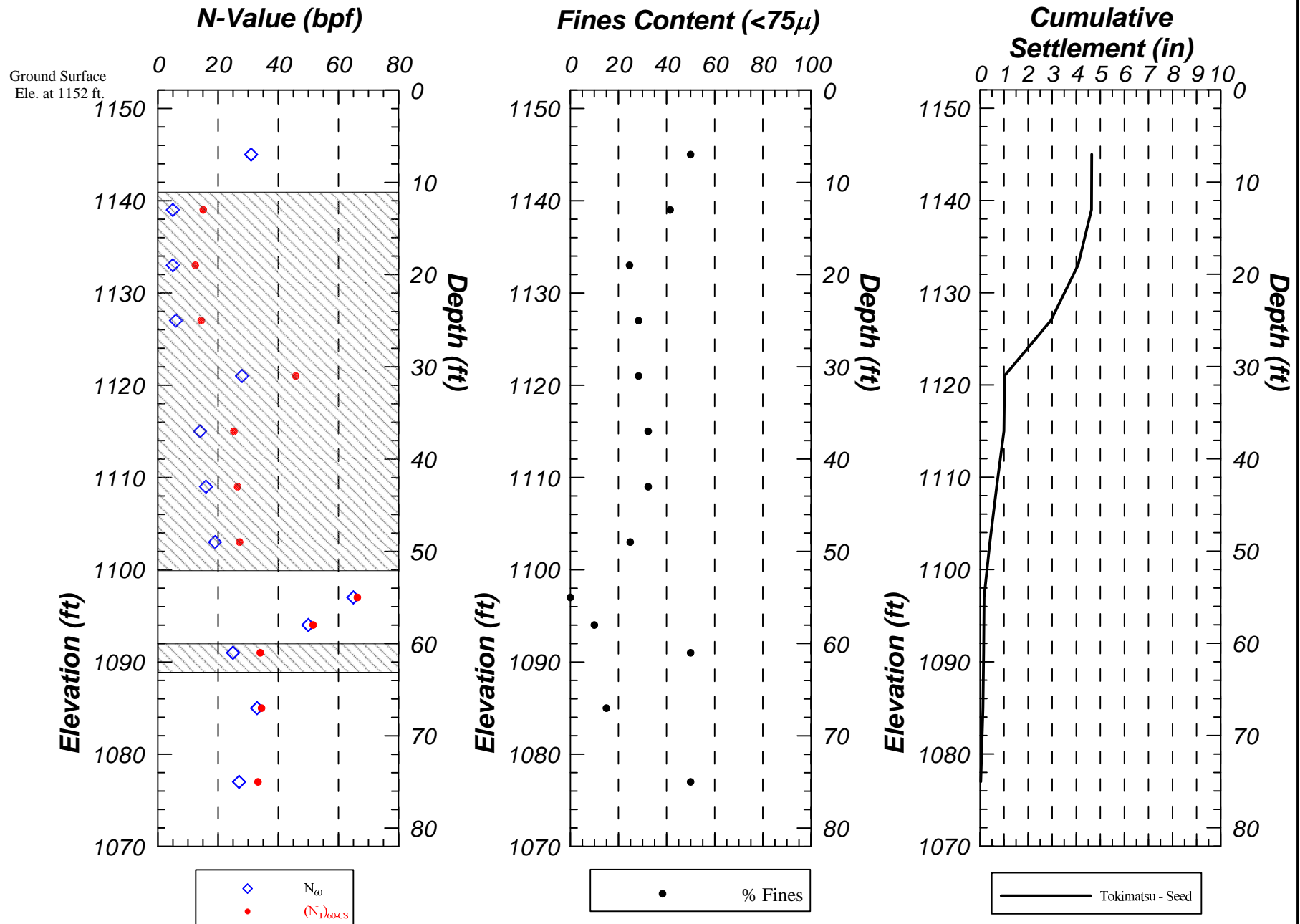


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Soil Layers Susceptible to Dry Settlement



**Notes:**

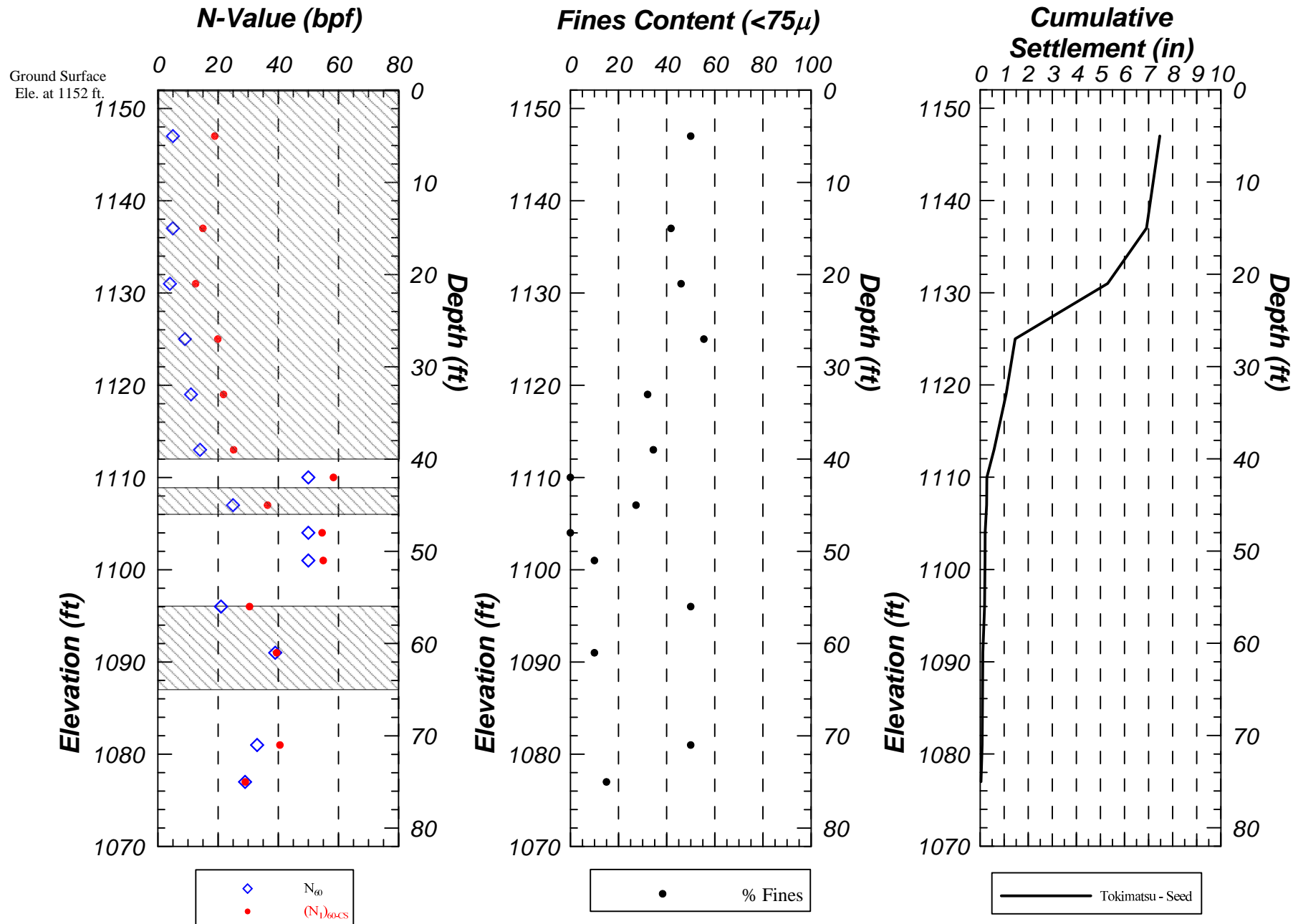
- Design level earthquake (DLE) parameters:  $a_{max}=0.67g$ ;  $M=7.5$
- Current groundwater depth = not encountered within the 76-foot maximum depth explored
- Historic high groundwater depth = 65 feet
- Cumulative settlements shown are for un-remediated existing soil conditions

**Summary of Dry Settlement Evaluation in Boring B-1**

**Figure 5**

Project No.: 4953-12-1321	
Proposed Emergency Department and Police Department Expansion	
Jerry L. Pettis VA Memorial Medical Center	

Soil Layers Susceptible to Dry Settlement



## Notes:

- Design level earthquake (DLE) parameters:  $a_{max}=0.67g$ ;  $M=7.5$
- Current groundwater depth = not encountered within the 76-foot maximum depth explored
- Historic high groundwater depth = 65 feet
- Cumulative settlements shown are for un-remediated existing soil conditions

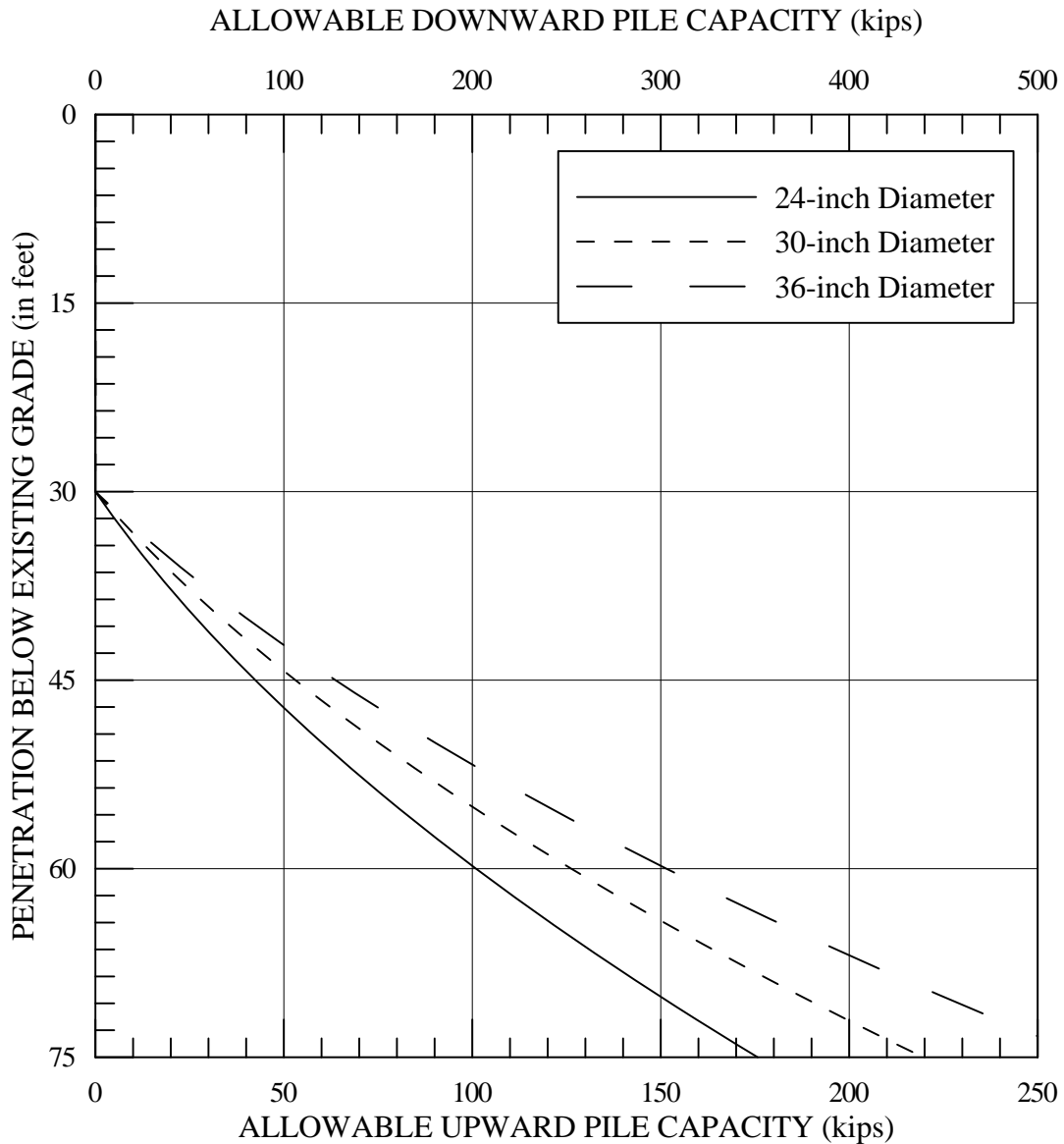
## Summary of Dry Settlement Evaluation in Boring B-2

Figure 6

Project No.: 4953-12-1321  
 Proposed Emergency Department and  
 Police Department Expansion  
 Jerry L. Pettis VA Memorial Medical Center





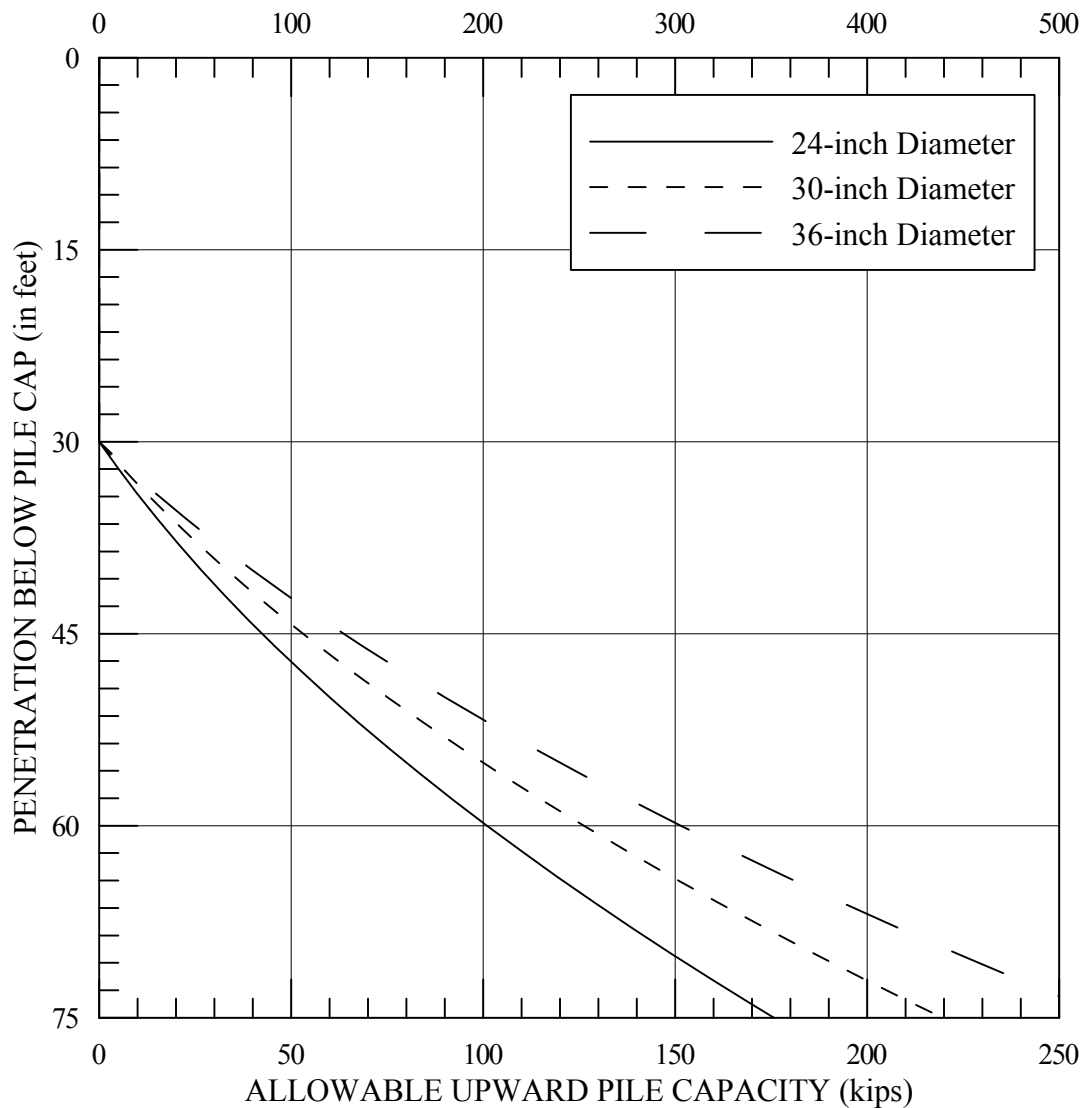


- NOTES:
- (1) The indicated values refer to the total of dead plus live loads.
  - (2) Piles in groups should be spaced a minimum of 3 pile diameters on centers.
  - (3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.
  - (4) Portions of pile to a depth of 30 feet below existing grade should have sonotube or equivalent materials installed to break the axial bond with the surrounding soils. Alternatively, downdrag loads should be added to the structural loads imposed on the piles when determining the required length.

Prepared/Date: LT 1/8/13  
 Checked/Date: AH 1/9/13

REVISED

## ALLOWABLE DOWNWARD PILE CAPACITY (kips)



- NOTES:
- (1) The indicated values refer to the total of dead plus live loads.
  - (2) Piles in groups should be spaced a minimum of 3 pile diameters on centers.
  - (3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.
  - (4) Portions of pile to a depth of 30 feet below pile cap should have sonotube or equivalent materials installed to break the axial bond with the surrounding soils. Alternatively, downdrag loads should be added to the structural loads imposed on the piles when determining the required length.

REVISED

Prepared/Date: LT 1/8/13  
Checked/Date: AH 1/9/13

## **APPENDIX A**

### **FIELD EXPLORATIONS AND LABORATORY TEST RESULTS**

## **APPENDIX A**

### **FIELD EXPLORATIONS AND LABORATORY TEST RESULTS**

#### **FIELD EXPLORATIONS**

The soils conditions beneath the site were explored by drilling two borings to depth of 76 feet below the existing grade at the locations shown on Figure 2. The borings were drilled using truck-mounted 8-inch-diameter hollow stem auger-type drilling equipment.

The soils encountered were logged by our field technician and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the borings are presented on Figures A-1.1 through A-1.2; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches using a 140 pound automatic hammer falling 30 inches are indicated on the logs. In addition to obtaining undisturbed samples, standard penetration tests (SPT) were performed in both of the borings; the results of the tests are indicated on the logs. The soils are classified in the accordance with the Unified Soil Classification System described on Figure A-2.

#### **LABORATORY TESTS**

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left on the boring logs.

To aid in classifying the soils, tests to determine the percentage of fines (material passing through a -200 sieve) in selected samples were performed. The results of these tests are presented on the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and after soaking to near-saturated

moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented on Figure A-3, Direct Shear Test Data.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE. REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

							<b>BORING 1</b>	
							DATE DRILLED:	December 1, 2012
							EQUIPMENT USED:	Hollow Stem Auger
							HOLE DIAMETER (in.):	8
							ELEVATION:	1,152 **
ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA(ppm)***	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
1150								4-inch thick Asphalt-Concrete over 4-inch thick Base FILL - SANDY SILT - moist, brown, fine sand, some small gravel
	5							Becomes more silty, dark gray Becomes brown Change to olive green, fine to medium sand
1145		31		9.8 10.5	124 -	31		Becomes sandier, fine grained, few gravel
	10			15.3	123	65		
1140		5		9.6	-			SILTY SAND - loose, moist, brown, fine to medium grained (42% Passing No. 200 Sieve)
	15			10.5	106	9		
1135		5		5.8	-			(25% Passing No. 200 Sieve)
	20							
1130				13.8	102	9		Few to little gravel
	25	6		5.4	-			Becomes finer, light greenish brown, few fine gravel, (28% Passing No. 200 Sieve)
1125				7.3	109	12		Becomes brown, fine to medium grained, trace gravel
	30	28		5.1	-			Interbedded with thin layer of Poorly Graded Sand with Silt, grayish-brown, fine to medium grained, trace fine gravel Becomes medium dense, grayish-brown, fine grained
1120				9.1	106	16		
	35	14		8.6	-			Becomes olive gray, some black angular gravel (up to 1-inch in size) increasing with depth, (32% Passing No. 200 Sieve)
1115								
	40			13.5	108	17		

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: WL  
Prepared By: WL  
Checked By: AH

**Proposed Emergency Department and  
Police Department Expansion  
Jerry L. Pettis VA Memorial Medical Center**



**LOG OF BORING**

Project: 4953-12-1321

Figure: A-1.1a

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	OVA(ppm)***	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
1110		16		4.4	-		
45				10.1	108	23	
1105		19		5.8	-		
50				6.9	114	44	
1100		65		2.7	-		
55				2.2	-	80/11"	
1095							
60		25		12.5	-		
1090				10.2	97	62	
65							
1085		33		10.1	-		
70				7.6	-	85/9"	
1080							
75		27		14.4	-		
1075							
80							

## BORING 1 (Continued)

DATE DRILLED: December 1, 2012  
EQUIPMENT USED: Hollow Stem Auger  
HOLE DIAMETER (in.): 8  
ELEVATION: 1,152 \*\*

Thin layer of fine grained with abundant fine gravel, gravel (up to 1-inch in size) increasing with depth

Interbedded thin layers of Well Graded Sand with fine gravel

Becomes olive gray, fine grained, some medium

Becomes more fine grained, (25% Passing No. 200 Sieve)

Fine to coarse grained

POORLY GRADED SAND - moist, olive gray, fine to coarse grained, increasing coarser material with depth

Becomes Poorly Graded Sand with Silt, gray, abundant fine gravel (rounded and angular shape, up to 1/4-inch in size)

WELL GRADED SAND with SILT - moist, gray, fine to coarse grained, abundant gravel (up to 2 1/2-inch in size)

SANDY SILT - very stiff, moist, olive gray, fine sand

SILTY SAND - very dense, moist, olive gray, fine to medium grained, some fine gravel

Becomes dense

Becomes gray, fine to coarse grained, few to little gravel

SILT - very stiff, moist, brownish gray, some fine sand

END OF BORING AT 76 FEET

Borehole backfilled with soil cuttings, tamped, and patched. Groundwater not encountered.

"N" Value Standard Penetration Test: Number of blows required to drive SPT sampler 18 inches using 140 pound automatic hammer falling 30 inches.

\*Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

\*\*Elevation obtained from Conceptual Site Plan by Ewing Cole, dated September 21, 2012.

Field Tech: WL  
Prepared By: WL  
Checked By: AH

Proposed Emergency Department and  
Police Department Expansion  
Jerry L. Pettis VA Memorial Medical Center



## LOG OF BORING

Project: 4953-12-1321

Figure: A-1.1b

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

							<b>BORING 2</b>	
							DATE DRILLED: December 1, 2012 EQUIPMENT USED: Hollow Stem Auger HOLE DIAMETER (in.): 8 ELEVATION: 1,152 **	
ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	OVA (ppm) ***	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
1150								4-inch thick Asphalt-Concrete over 4-inch thick Base FILL - SANDY SILT - moist, brown, fine sand
	5			16.3	116	18		Fine to medium grained, some fine gravel
1145				8.5	121	24		
	10							SILTY SAND - loose, moist, brown, fine to medium grained, few gravel
1140				15.6	104	4		
	15	5		11.2	-			(42% Passing No. 200 Sieve)
1135				3.8	105	12		Becomes less silty
	20	4		11.1	-			Trace gravel, (46% Passing No. 200 Sieve)
1130				4.3	101	11		Interbedded with thin layer of Poorly Graded Sand, moist, brownish-gray, fine to coarse grained
	25							SANDY SILT - stiff, moist, olive gray, fine sand
1125		9		12.6	-			(56% Passing No. 200 Sieve)
	30			8.4	108	16		SILTY SAND - moist, brownish gray, fine to medium grained, trace gravel
1120		11		7.5	-			Medium dense, (32% Passing No. 200 Sieve)
	35			11.8	103	13		
1115		14		7.8	-			Becomes olive gray, fine to medium grained, (35% Passing No. 200 Sieve)
	40							

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: WL  
 Prepared By: WL  
 Checked By: AH

**Proposed Emergency Department and  
 Police Department Expansion  
 Jerry L. Pettis VA Memorial Medical Center**



**LOG OF BORING**

Project: 4953-12-1321

Figure: A-1.2a



THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	OVA(ppm)***	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
1110				5.7	107	52	SP
	45	25		7.1	-		SM
1105				2.9	118	50/6"	SW
	50	50/6"		2.9	-		SW-SM
1100				4.4	114	33	ML
1095				14.7	-		SP-SM
	60	39		12.9	106	35	ML
1090				17.1	-		SM
	65	33		7.8	124	45	
1085							
	70						
1080							
	75						
1075							
	80						

## BORING 2 (Continued)

DATE DRILLED: December 1, 2012  
EQUIPMENT USED: Hollow Stem Auger  
HOLE DIAMETER (in.): 8  
ELEVATION: 1,152 \*\*

POORLY GRADED SAND - moist, gray, fine to medium grained

SILTY SAND - medium dense, moist, gray, fine to coarse grained, few gravel

(27% Passing No. 200 Sieve)

WELL GRADED SAND - medium dense, moist, gray, fine to coarse grained, abundant gravel (angular and rounded shape, up to 1½-inch in size)

WELL GRADED SAND with SILT - moist, gray, fine to coarse grained, some fine gravel (up to ¼-inch in size)

Interbedded with thin layer of Well Graded Sand with Gravel - moist, olive gray, few gravel (up to 1-inch in size), increasing gravel with depth

SANDY SILT - moist, olive gray, fine to medium sand

POORLY GRADED SAND with SILT - dense, moist, gray with some reddish pigmentation, some gravel (rounded, up to ½-inch in size)

Becomes more fine grained and siltier with depth  
SANDY SILT - hard, moist, olive gray, fine sand

SILTY SAND - moist, olive gray, fine to coarse grained

END OF BORING AT 76 FEET

Borehole backfilled with soil cuttings, tamped, and patched.  
Groundwater not encountered.

"N" Value Standard Penetration Test: Number of blows required to drive SPT sampler 18 inches using 140 pound automatic hammer falling 30 inches.

\*Number of blows required to drive Crandall Sampler 12 inches using 140 pound hammer falling 30 inches.

\*\*Elevation obtained from Conceptual Site Plan by Ewing Cole, dated September 21, 2012.

Field Tech: WL  
Prepared By: WL  
Checked By: AH

Proposed Emergency Department and  
Police Department Expansion  
Jerry L. Pettis VA Memorial Medical Center



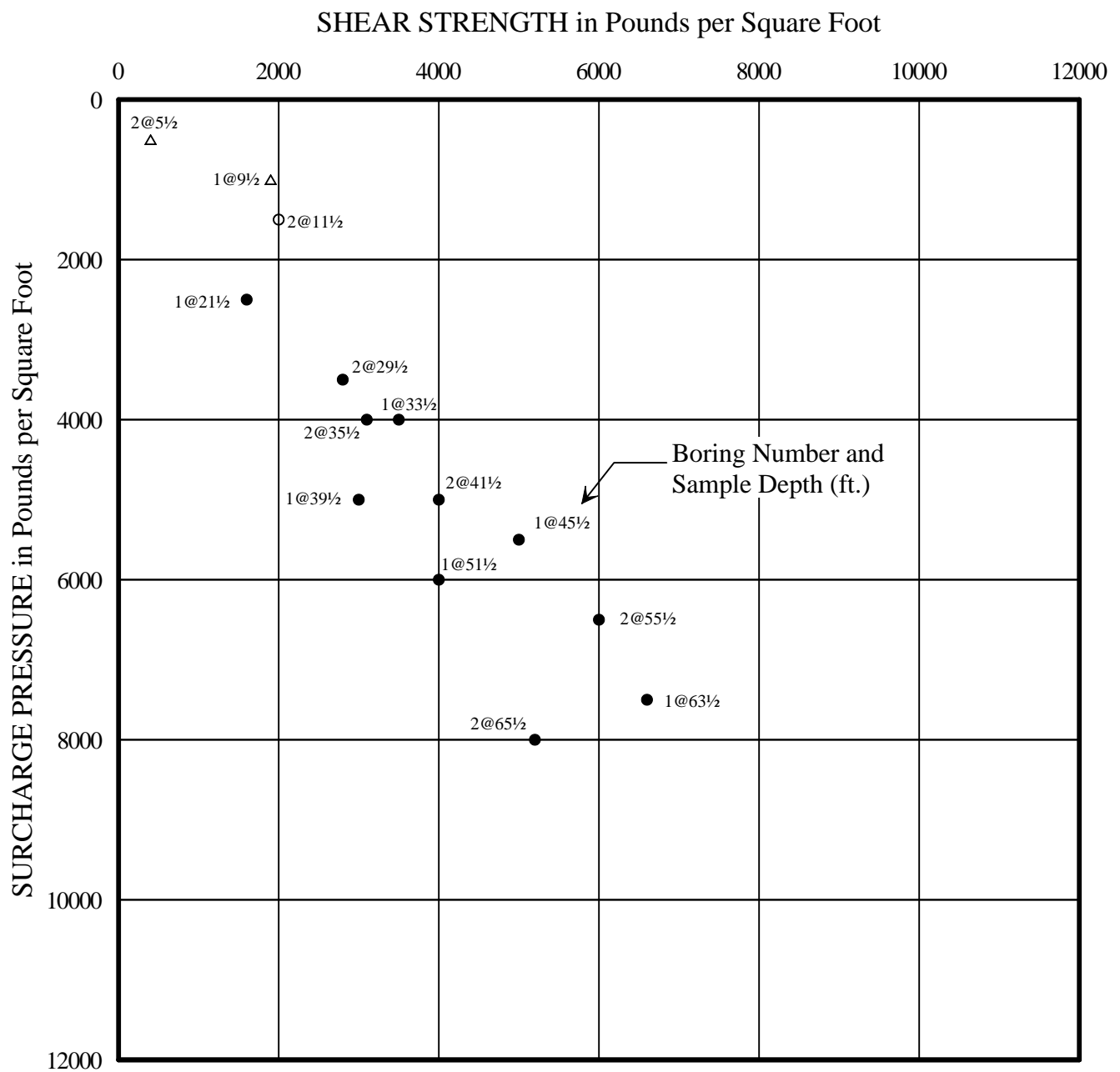
# LOG OF BORING

Project: 4953-12-1321

Figure: A-1.2b

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES		Undisturbed Sample		Auger Cuttings																																	
<b>COARSE GRAINED SOILS</b> (More than 50% of material is LARGER than No. 200 sieve size)	<b>GRAVELS</b> (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	<b>CLEAN GRAVELS</b> (Little or no fines)		GW	Well graded gravels, gravel - sand mixtures, little or no fines.		Split Spoon Sample			Bulk Sample																															
				GP	Poorly graded gravels or grave - sand mixtures, little or no fines.		Rock Core			Crandall Sampler																															
		<b>GRAVELS WITH FINES</b> (Appreciable amount of fines)		GM	Silty gravels, gravel - sand - silt mixtures.		Dilatometer			Pressure Meter																															
				GC	Clayey gravels, gravel - sand - clay mixtures.		Packer			No Recovery																															
	<b>SANDS</b> (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	<b>CLEAN SANDS</b> (Little or no fines)		SW	Well graded sands, gravelly sands, little or no fines.		Water Table at time of drilling			Water Table after drilling																															
				SP	Poorly graded sands or gravelly sands, little or no fines.																																				
		<b>SANDS WITH FINES</b> (Appreciable amount of fines)		SM	Silty sands, sand - silt mixtures																																				
				SC	Clayey sands, sand - clay mixtures.																																				
<b>FINE GRAINED SOILS</b> (More than 50% of material is SMALLER than No. 200 sieve size)	<b>SILTS AND CLAYS</b> (Liquid limit LESS than 50)			ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.	Correlation of Penetration Resistance with Relative Density and Consistency																																			
				CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.																																				
				OL	Organic silts and organic silty clays of low plasticity.																																				
	<b>SILTS AND CLAYS</b> (Liquid limit GREATER than 50)			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	<table><tr><th colspan="2">SAND &amp; GRAVEL</th><th colspan="2">SILT &amp; CLAY</th></tr><tr><th>No. of Blows</th><th>Relative Density</th><th>No. of Blows</th><th>Consistency</th></tr><tr><td>0 - 4</td><td>Very Loose</td><td>0 - 1</td><td>Very Soft</td></tr><tr><td>5 - 10</td><td>Loose</td><td>2 - 4</td><td>Soft</td></tr><tr><td>11 - 30</td><td>Medium Dense</td><td>5 - 8</td><td>Medium Stiff</td></tr><tr><td>31 - 50</td><td>Dense</td><td>9 - 15</td><td>Stiff</td></tr><tr><td>Over 50</td><td>Very Dense</td><td>16 - 30</td><td>Very Stiff</td></tr><tr><td></td><td></td><td>Over 30</td><td>Hard</td></tr></table>				SAND & GRAVEL		SILT & CLAY		No. of Blows	Relative Density	No. of Blows	Consistency	0 - 4	Very Loose	0 - 1	Very Soft	5 - 10	Loose	2 - 4	Soft	11 - 30	Medium Dense	5 - 8	Medium Stiff	31 - 50	Dense	9 - 15	Stiff	Over 50	Very Dense	16 - 30	Very Stiff			Over 30	Hard
			SAND & GRAVEL		SILT & CLAY																																				
			No. of Blows	Relative Density	No. of Blows					Consistency																															
	0 - 4	Very Loose	0 - 1	Very Soft																																					
	5 - 10	Loose	2 - 4	Soft																																					
11 - 30	Medium Dense	5 - 8	Medium Stiff																																						
31 - 50	Dense	9 - 15	Stiff																																						
Over 50	Very Dense	16 - 30	Very Stiff																																						
		Over 30	Hard																																						
	CH	Inorganic clays of high plasticity, fat clays																																							
Bedrock				CLAYEY SILTSTONE																																					
<b>BOUNDARY CLASSIFICATIONS:</b> Soils possessing characteristics of two groups are designated by combinations of group symbols.																																									
<table><tr><td rowspan="2">SILT OR CLAY</td><td colspan="3">SAND</td><td colspan="2">GRAVEL</td><td rowspan="2">Cobbles</td><td rowspan="2">Boulders</td></tr><tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr><tr><td></td><td>No.200</td><td>No.40</td><td>No.10</td><td>No.4</td><td>3/4"</td><td>3"</td><td>12"</td></tr></table> <p>U.S. STANDARD SIEVE SIZE</p>											SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders	Fine	Medium	Coarse	Fine	Coarse		No.200	No.40	No.10	No.4	3/4"	3"	12"										
SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders																																		
	Fine	Medium	Coarse	Fine	Coarse																																				
	No.200	No.40	No.10	No.4	3/4"	3"	12"																																		
<div>Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)</div> <div></div>																																									

Figure A-2



- Samples tested at field moisture content
  - └ Natural soils
- △ ○ Samples soaked to a moisture content near saturation
  - └ Natural soil
  - └ Fill soils

Prepared/Date: JF 12/21/12  
Checked/Date: AH 1/7/13

## **APPENDIX B**

### **PRIOR FIELD EXPLORATIONS AND LABORATORY TEST RESULTS**

## **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	--	1/2-in. to 12-in. thickness;
seam	--	1/16 to 1/2-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.

**GEOBASE**

**EXPLANATION OF TERMS  
AND SYMBOLS USED**

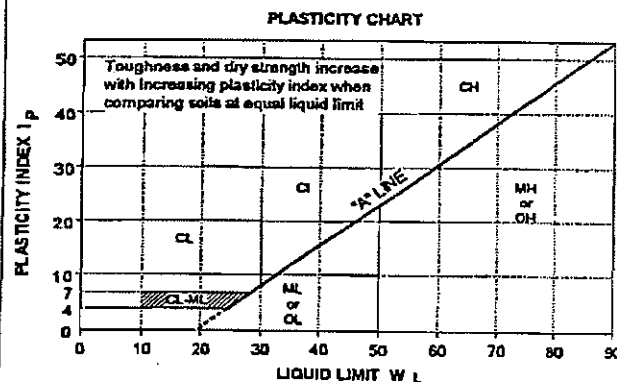
# SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

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 Below "A" line on plasticity chart: negligible organic content		ML		Inorganic Silts and very fine Sands, Rock Flour, Silty Sands of slight plasticity	$W_L < 50$	See chart below	
			MH		Inorganic Silts micaceous or diatomaceous, fine Sandy or Silty soils	$W_L > 50$		
	CLAYS Above "A" line on plasticity chart: negligible organic content		CL		Inorganic Clays of low plasticity, Gravelly, Sandy, or Silty Clays, lean Clays	$W_L < 30$		
			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 Below "A" line on plasticity chart		OL		Organic Silts and organic Silty Clays of low plasticity	$W_L < 50$		
						OH		

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
	Ca Claystone
	Ma 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

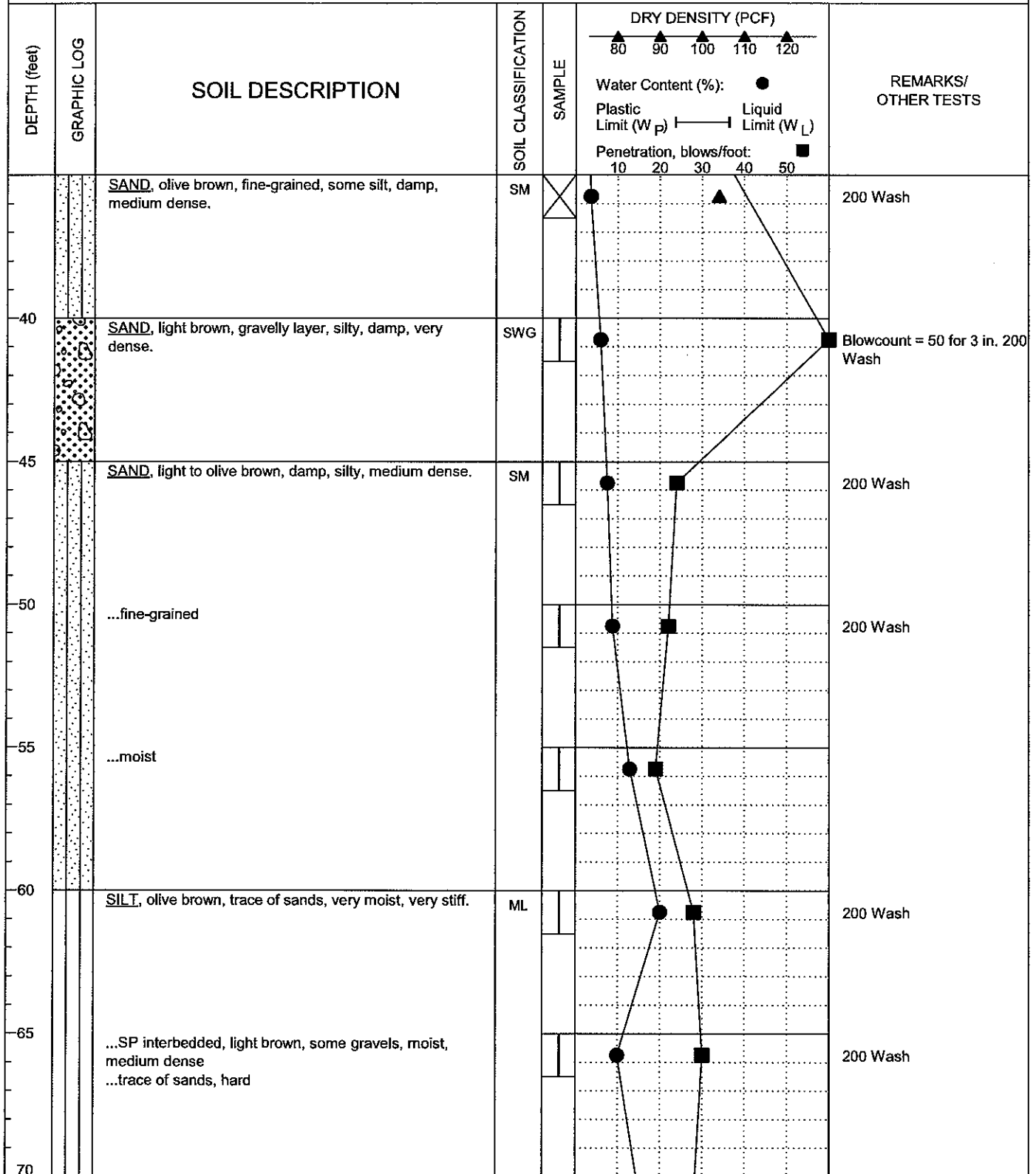
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): Plastic Limit (W <sub>p</sub> ) — Liquid Limit (W <sub>L</sub> ) Penetration, blows/foot: 10 20 30 40 50	REMARKS/ OTHER TESTS
		GRASS AND TOPSOIL, 6 inches				
		SAND (FILL), olive brown, silty, moist, loose.	SM			
5		SAND, olive brown, silty, moist, loose.	SM			200 Wash, C, DS
10		...very loose				200 Wash
15						200 Wash, C
20		...loose				200 Wash
25		SAND, light brown, medium-grained, little silt and gravel, damp, medium dense.	SP			200 Wash
30		...olive brown, little silt and some gravel, medium- to coarse-grained				200 Wash
35						

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	DATE LOGGED 04/28/2012	FIGURE NO. B-2
		DRILLER JDK		

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-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	DATE LOGGED 04/28/2012	FIGURE NO. B-2
			DRILLER JDK		

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

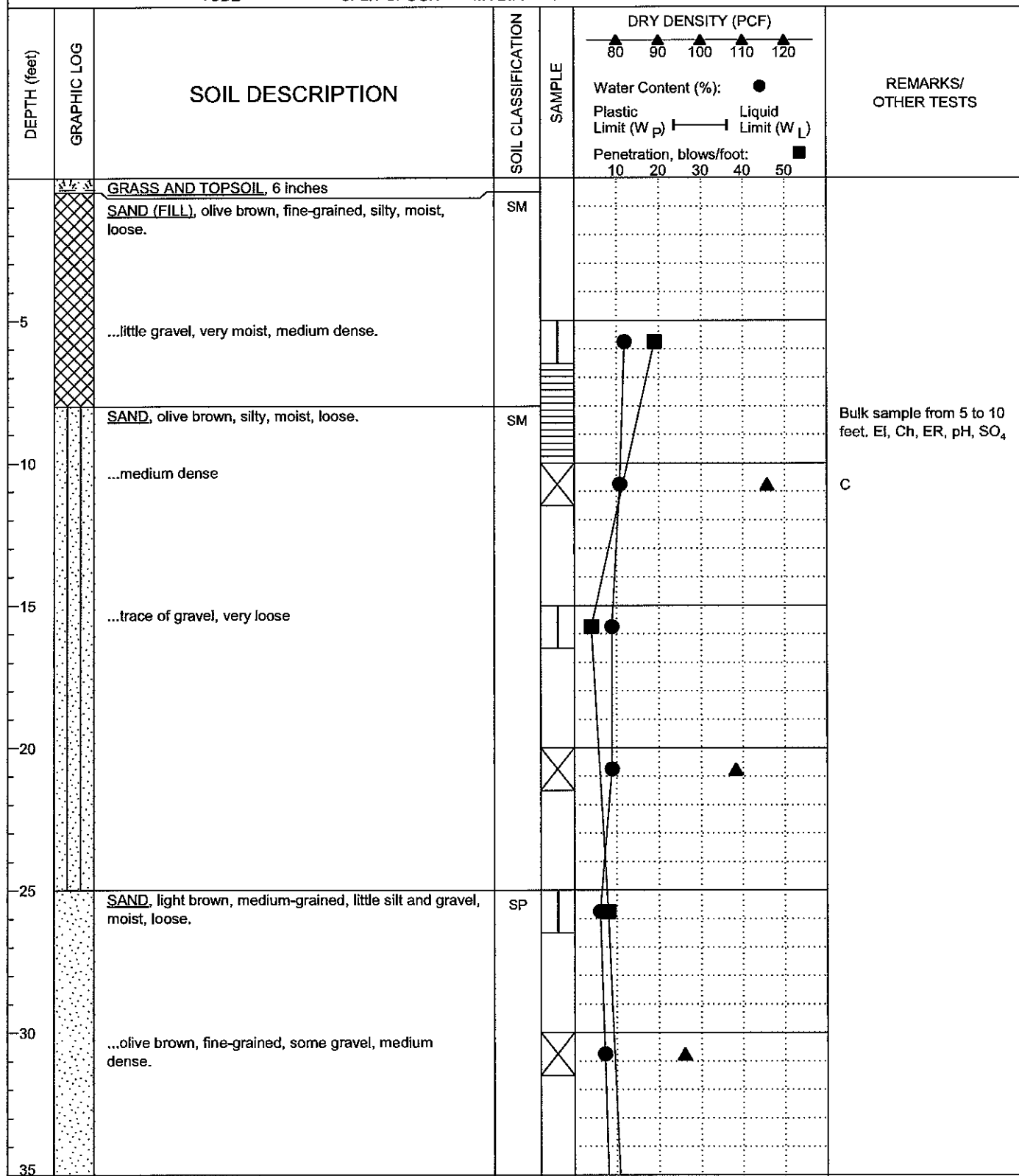
SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE

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, trace of sands, very moist, very stiff.	ML		● at 10 PCF ■ at 20 PCF	
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	04/28/2012	
			DRILLER	JDK	LOGGED	04/28/2012	
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.						FIGURE NO.	B-2

# LOG OF BORING

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

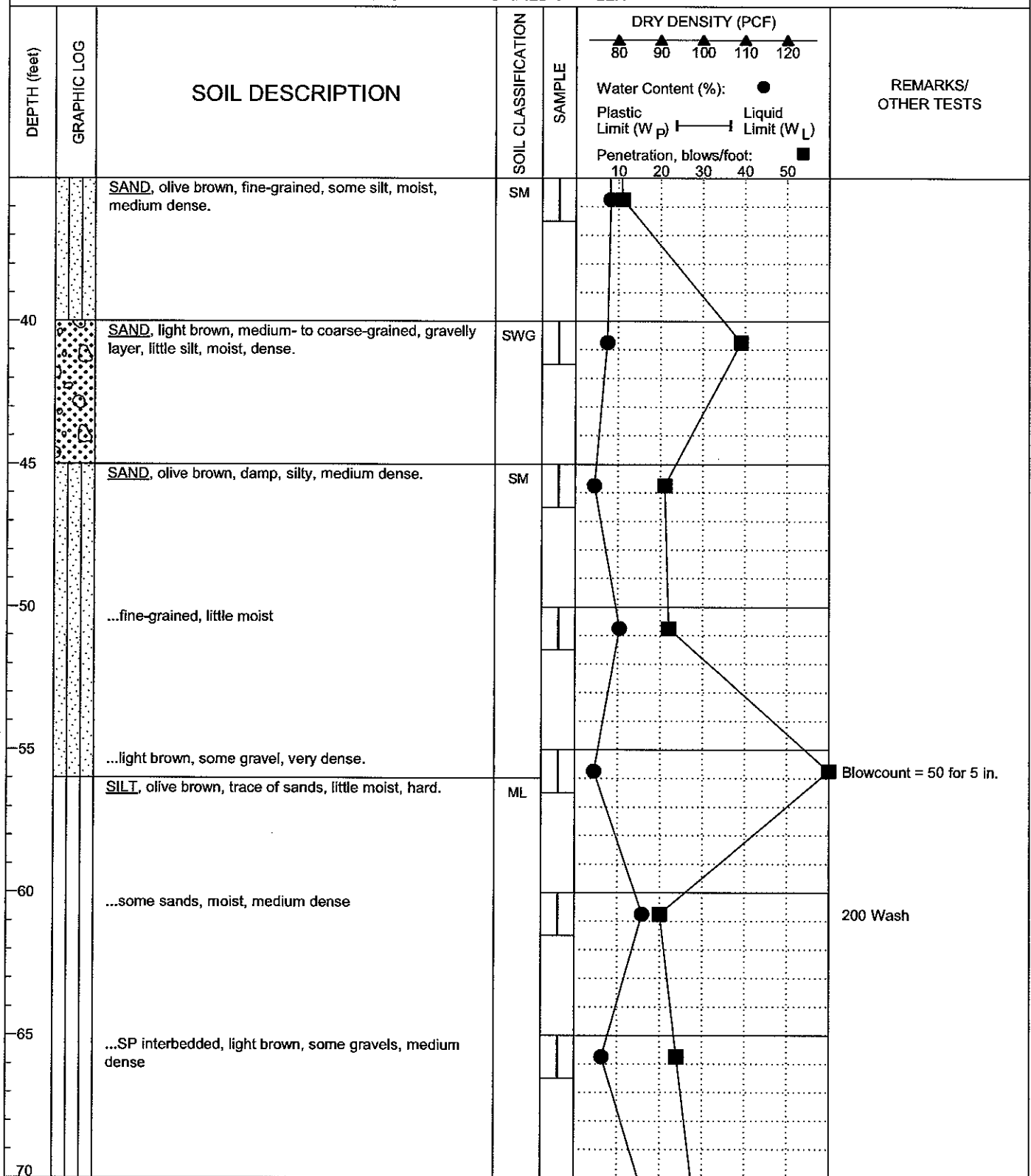


GEOBASE, INC.	PROJECT Jerry L. Pettis Memorial Veterans Medical Center – OP Pharmacy, Loma Linda, CA			BORING NO. B-2
	DEPTH TO WATER feet	SURFACE ELEV. 1148 feet	LOGGED BY HDN	PROJECT NO. C.329.03.00
	DEPTH TO SLOUGH	DRILL RIG CME-75 HT	DATE 04/28/2012	FIGURE NO. B-3
		DRILLER JDK		

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	DATE LOGGED 04/28/2012	FIGURE NO. B-3
			DRILLER JDK		

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.

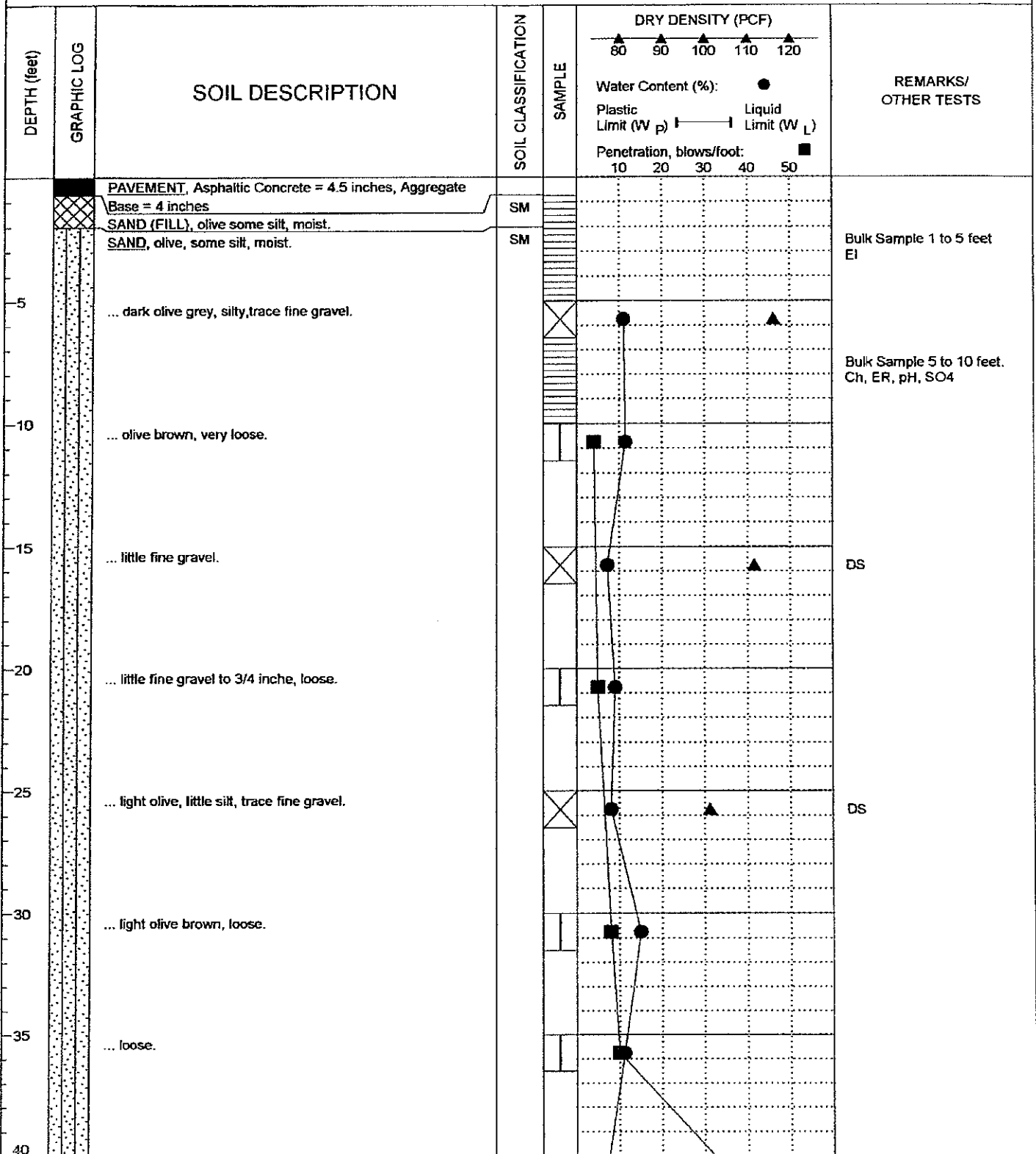
*GEOBASE, INC., November 2006 –*

Figure B-4      Log of Boring B-1

Figure B-5      Log of Boring B-2

# LOG OF BORING

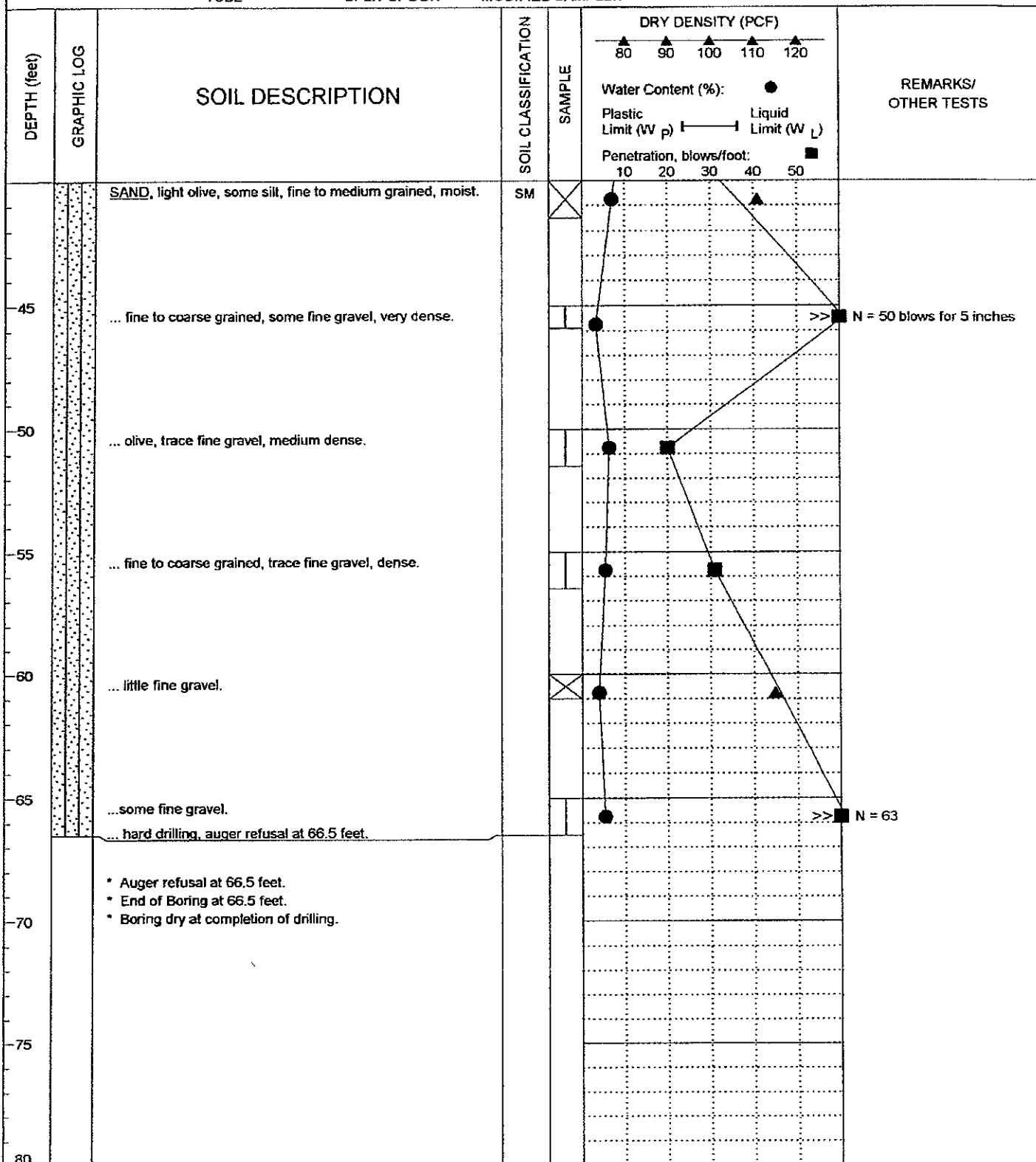
SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☒ NO RECOVERY ☐ CORE



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	
	DEPTH TO SLOUGH		DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED	10/26/2005	
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

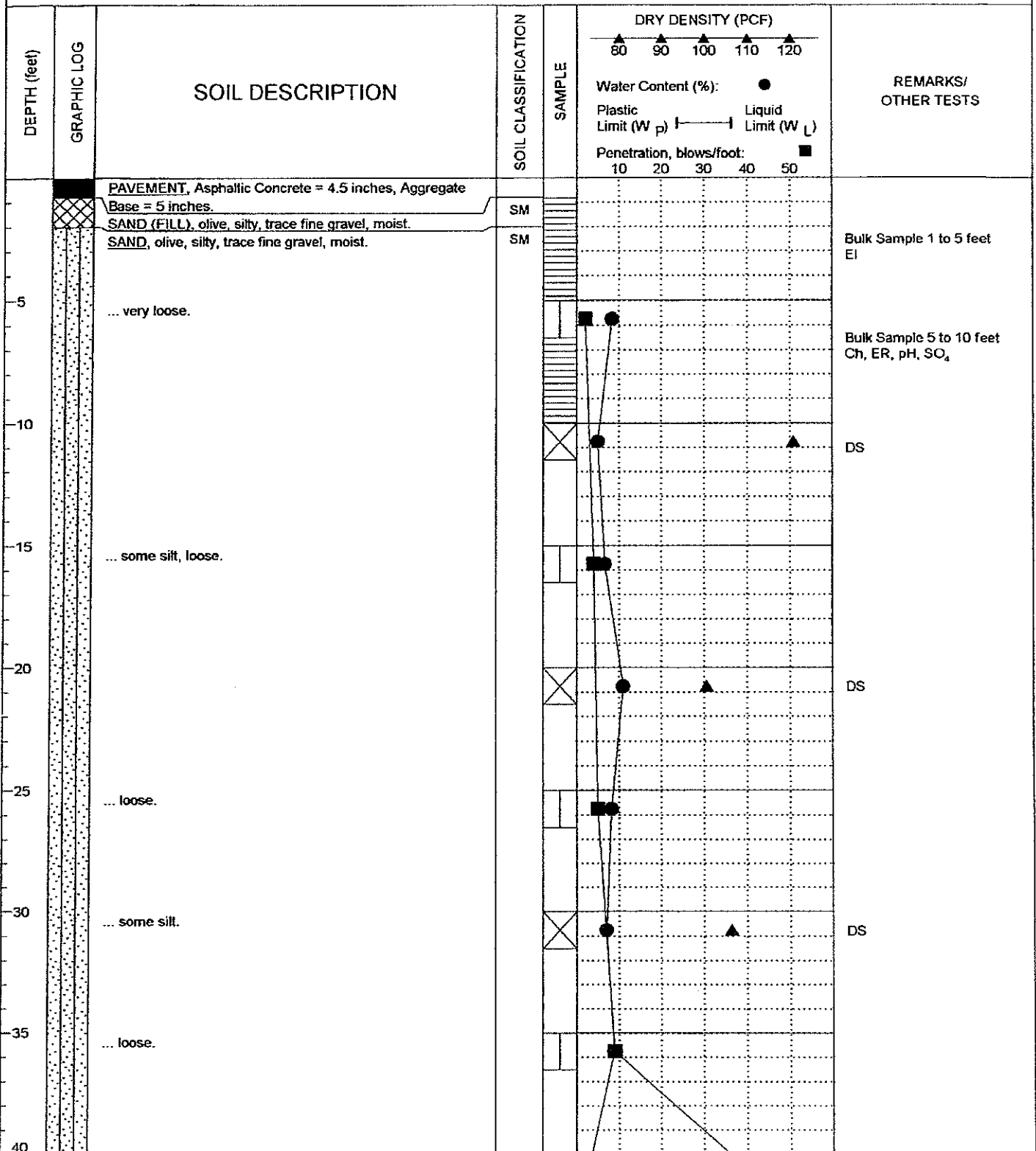


GEOBASE, INC.	PROJECT PARKING STRUCTURE			BORING NO.	B-1
	V.A. MEDICAL CENTER, LOMA LINDA, CALIFORNIA			PROJECT NO.	P.322.01.00
	DEPTH TO WATER	feet	SURFACE ELEV.	LOGGED BY	RAP
	DEPTH TO SLOUGH		DRILL RIG CME-75 HT DRILLER MARTINI	DATE LOGGED	10/26/2006
Note: This log of boring should be evaluated in 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

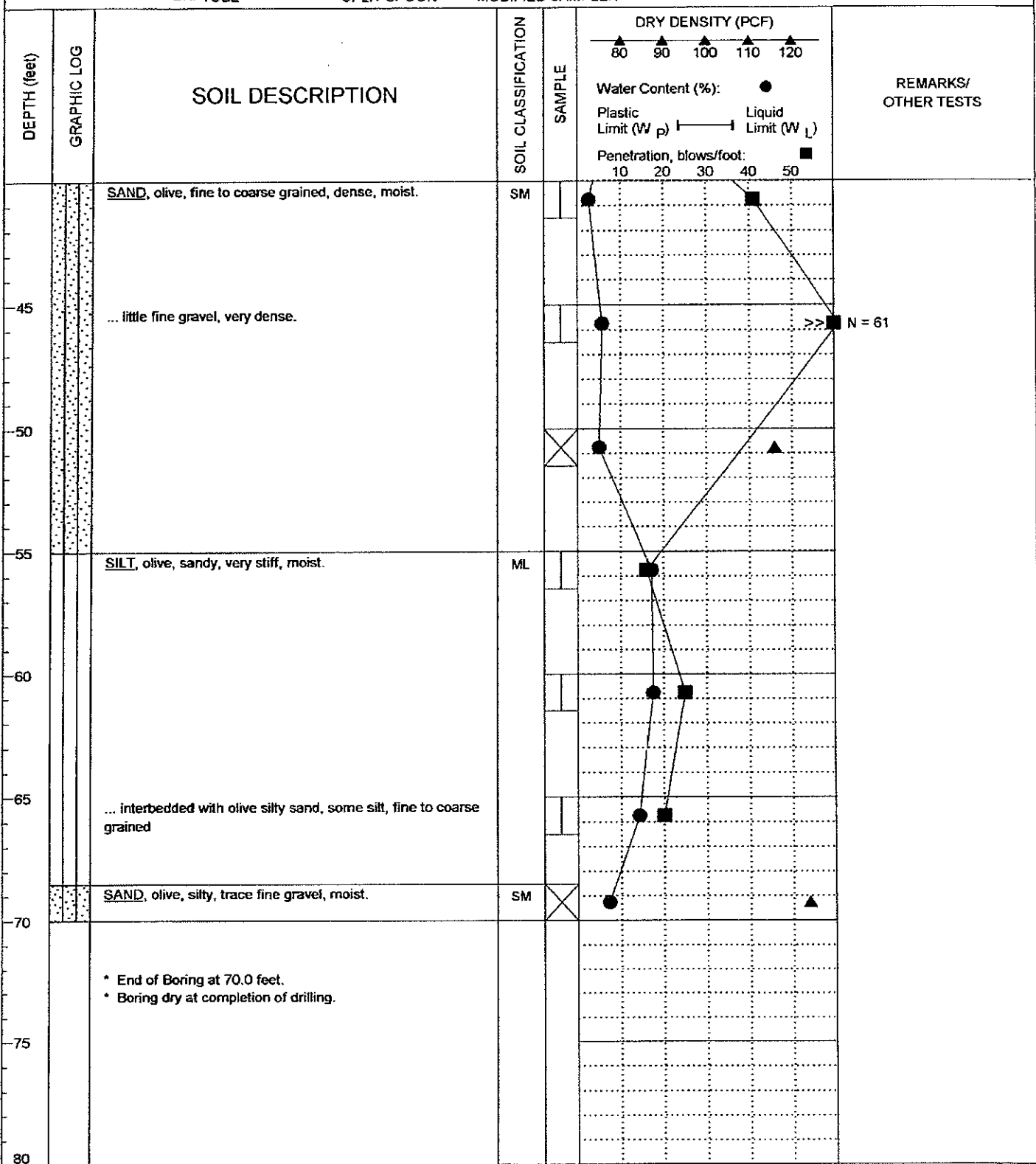
SAMPLE TYPE: ☒ THIN WALLED TUBE ☐ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☐ DISTURBED ☐ NO RECOVERY ☐ CORE



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.	PARKING STRUCTURE				BORING NO.	B-2	
	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- 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

# 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

  
ANAHEIM TEST LAB

WES BRIDGER CHEMIST

C.329.03.00

FIGURE C-7

# 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

DATE: 11/01/06

P.O. No. VERBAL

ATTN: BOB PEARSON

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 @ 5'-10'	8.2	33	35	2,803
#2 BOR. #2 @ 5'-10'	8.0	62	40	672

RESPECTFULLY SUBMITTED

  
POPPY BRIDGER Chief Chemist