PRELIMINARY FOUNDATION REPORT
PROPOSED PARKING GARAGE
SABRE SPRINGS/PENESQUITOS
TRANSIT STATION
SAN DIEGO, CALIFORNIA

July 10, 2009
July 10, 2009
Project No. 102174

Mr. Siegfried Fassmann
David Evans and Associates, Inc.
110 West A Street, Suite 1700
San Diego, California 92101

Subject: Preliminary Foundation Report
Proposed Parking Garage
Sabre Springs/Penasquitos Transit Station
13538 Sabre Springs Parkway
San Diego County, California

Dear Mr. Fassmann:

Enclosed is our Preliminary Foundation Report for the Sabre Springs/Penasquitos Transit Station parking garage project in San Diego County, California. This preliminary report is intended to provide preliminary geotechnical information for preliminary design and planning of the Sabre Springs/Penasquitos Transit Station parking garage.

We appreciate the opportunity to provide geotechnical engineering services for this project. If you have any questions regarding this report, please contact the undersigned.

Very truly yours,

KLEINFELDER WEST, INC.

Eren Koprulu
Geotechnical Engineer

Barry R. Bevier, GE 143
Principal Engineer

EK:BRB:rp
TABLE OF CONTENTS

1.0 INTRODUCTION .................................................................................................................. 1
  1.1 BACKGROUND ................................................................................................................ 1
  1.2 EXISTING FACILITIES AND PROPOSED IMPROVEMENTS .............................................. 1
    1.2.1 Documents Reviewed ............................................................................................... 1
    1.2.2 Existing Facilities and Proposed Improvements ..................................................... 2

2.0 SITE AND SUBSURFACE CONDITIONS .............................................................................. 3
  2.1 GEOLOGY AND SUBSURFACE CONDITIONS ................................................................ 3
    2.1.1 Regional Geology .................................................................................................... 3
    2.1.2 Subsurface Soil Conditions .................................................................................... 3

3.0 DISCUSSION AND RECOMMENDATIONS ......................................................................... 5
  3.1 SEISMIC DESIGN CONSIDERATIONS ............................................................................. 5
    3.1.1 Faulting and Siesmicity ............................................................................................ 5
    3.1.2 Ground Surface Rupture ......................................................................................... 6
    3.1.3 Seismic Shaking ....................................................................................................... 6
    3.1.4 Liquefaction ........................................................................................................... 7
    3.1.5 Seismic Compaction ............................................................................................... 8
    3.1.6 Expansive Soil ........................................................................................................ 8
    3.1.7 Tsunami, Seiche and Flood .................................................................................... 8
  3.2 SOIL CORROSIIVITY ...................................................................................................... 9
  3.3 FOUNDATIONS ............................................................................................................... 9
    3.3.1 Shallow Foundations .............................................................................................. 10

4.0 ADDITIONAL GEOTECHNICAL STUDY ............................................................................ 11

5.0 LIMITATIONS .................................................................................................................. 12

6.0 REFERENCES .................................................................................................................. 13

TABLES
Table 1 CBC Seismic Design Parameters .............................................................................. 7
Table 2 Soil Corrosion Test Summary .................................................................................. 9

PLATES
Plate 1 Site Vicinity Map
Plate 2 Current Site Plan
Plate 3 Regional Geology Map
Plate 4 Fault Map and Epicenters of Earthquakes

APPENDICES
Appendix A Previous Boring Location Plan and Previous Log of Test Borings
Appendix B Proposed Improvements Drawings
Appendix C ASFE Insert

102174/SD19R081 Page iii of iii July 10, 2009
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1.0 INTRODUCTION

1.1 BACKGROUND

As part of the I-15 Managed Lanes Project in San Diego County, a new Sabre Springs / Peñasquitos Transit Station project has been constructed with Direct Access Ramp (DAR) connection to I-15. The project consists of the construction of a multi-story parking garage to accommodate approximately 500 parking spaces. The project also consists of a new BRT staging area, permanent station amenities, and a permanent Park & Ride surface parking lot, which are not included within our scope of work. The project location is shown on the Site Vicinity Map, Plate 1.

Our understanding of the proposed project is based on discussions with Mr. Siegfried Fassman and our review of the available plans. Kleinfelder’s scope of services for this phase of the project is limited to providing preliminary geotechnical input for design of the four- to five-story Parking Structure. Design loads for the proposed structure were not available at the time of this report.

1.2 EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

1.2.1 Documents Reviewed

We reviewed a geotechnical investigation report and a set of drawings noted below that were provided to Kleinfelder by David Evans and Associates. Portions of the geotechnical report relevant to subsurface soil conditions and proposed improvement drawings are included in Appendices A and B, respectively.

- "Sabre Springs/Penasquitos Transit Station Interim and Ultimate Improvements" (2 sheets), prepared by Berryman & Henigar, dated December 6, 2006.

In addition, we reviewed the Poway Quadrangle regional geologic map by Kennedy and Peterson (1975).
1.2.2 Existing Facilities and Proposed Improvements

The Sabre Springs/Penasquitos Transit Station is located on 13538 Sabre Springs Parkway as shown on Plate 2 Site Plan. The site is bounded to the north by Ted Williams Parkway, to the east by Sabre Springs Parkway, and to the south by Evening Creek Drive North. The site is bounded to the west by a natural creek channel. The western-half of the site is currently used as a temporary, approximately 96 parking asphalt concrete surface Park and Ride parking lot with bus staging areas. The Ultimate Improvements Plan shows this portion of the site as the future BRT staging area, with associated permanent station amenities (such as shelters, TVM’s etc). Caltrans construction trailers, Caltrans lab trailer, and temporary BTM facilities currently occupy the eastern-half of the site. Ultimate Improvements Plan shows this portion of the site as the future Parking Structure and a permanent park and ride asphalt concrete surface parking lot (approximately 88 parking spots). Parking Structure will be constructed south of the SDG&E easement and the surface parking lot on the north. Proposed improvements are shown in Appendix B.
2.0 SITE AND SUBSURFACE CONDITIONS

2.1 GEOLOGY AND SUBSURFACE CONDITIONS

2.1.1 Regional Geology

The project site is located in the western San Diego County section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Traverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Mexico), and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous, Tertiary, and Quaternary age sedimentary rock. Specifically, the site is underlain at depth (below fill and alluvium) by geologic material consisting Jurassic age Santiago Peak Volcanics (Kennedy and Peterson, 1975). This unit as well as other soil units below the site are described in more detail in the Subsurface Soil Conditions section below. Review of site plans indicate that the original topographic relief ranged from approximately 527 to 550 feet MSL (above mean sea level). A Regional Geology Map is provided on Plate 3.

2.1.2 Subsurface Soil Conditions

The subsurface soil conditions described in this section are based on log of test borings from the February 27, 2004 Geotechnical Investigation Report (see Appendix A) prepared by Ninyo & Moore and review of a regional geologic map of the project area (Kennedy and Peterson, 1975). As described above the site is underlain by basement rock consisting of the Santiago Peak Volcanics. Most of this material is overlain by either alluvial soils deposited within the drainage feature along the western side of the site or artificial fill soils placed during previous grading operations. A general discussion of these units is provided below.

2.1.2.1 Artificial Fill

Artificial Fill was encountered in all of the borings performed by Ninyo & Moore throughout the project area. Within the proposed parking structure site, fill was encountered to the maximum depth explored of 5 feet at three boring locations (B-5, B-6 and B-7). In general, the fill consists of damp to moist, medium dense, silty and
clayey fine to coarse sand with some gravel and cobble layers. We anticipate that minor grading was performed at the site for the existing temporary Park & Ride after the preparation of the above referenced subsurface investigation report. However, since the previous borings within the proposed structure footprint were terminated without penetrating the fill, the actual fill depth across this area is unknown.

2.1.2.2 Alluvium

Alluvium was not encountered within any of the boring on the project area and appears to be limited to the drainage feature west of the site. We therefore anticipate that alluvium does not underlie the site.

2.1.2.3 Santiago Peak Volcanics

Metavolcanic rock of the Santiago Peak Volcanics was encountered underlying the fill at a depth of approximately 5 feet in boring B-8, located north of the proposed structure and at depth of approximately 1 foot at B-3 just to the northwest. In general, the Santiago Peak Volcanics consisted of damp, intensely weathered metavolcanic rock.

2.1.2.4 Groundwater

Groundwater was not encountered in any of the exploratory borings conducted by Ninyo & Moore. The regional groundwater table is expected to be at depths greater than 35 feet. However, it is possible that some isolated perched groundwater zones may exist within the upper few feet at the site. Fluctuations in the groundwater level may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors.
3.0 DISCUSSION AND RECOMMENDATIONS

The following sections present our discussions and recommendations for design of the project.

3.1 SEISMIC DESIGN CONSIDERATIONS

3.1.1 Faulting and Siesmicity

Faulting in the southern California region is controlled by strain release within the San Andreas Fault System. The San Andreas Fault delineates the boundary between two global tectonic plates consisting of the North American Plate on the east and the Pacific Plate on the west. The San Andreas Fault stretches from the Gulf of California in Mexico along a northwest alignment through the desert region of Southern California up to Northern California, where it eventually trends offshore north of San Francisco. Right lateral slip movement along the plate boundary of the San Andreas Fault is by far the most dominant factor controlling the seismicity throughout northern and southern California (Wallace, 1990; Weldon and Sieh, 1985). Within Southern California, the strain associated with the plate boundary movement extends well westward for up to 150 miles (241 kilometers) from the main San Andreas Fault strand in the Imperial Valley to well offshore of San Diego.

The major faults east of San Diego (from east to west) include the San Andreas Fault, the San Jacinto fault and the Elsinore fault. Major faults west of San Diego include the Palos Verdes-Coronado Bank fault, the San Diego Trough fault, and the San Clemente fault. The most dominant zone of faulting within the western portion of San Diego County is several faults associated with the Rose Canyon Fault Zone (RCFZ). Refer to Plate 4 for a Fault Map and Epicenters of Earthquakes.
3.1.2 Ground Surface Rupture

The site does not lie within an Alquist-Priolo Special Studies Zone (DMG, 2000). Based on our geologic literature review, no active or potentially active faults are known to transect the project site. Therefore, the possibility of primary surface fault rupture at the site is considered low.

3.1.3 Seismic Shaking

The site is located in a seismically active region of southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking due to nearby and distant earthquakes should be anticipated during the life of the structure.

We understand that the proposed improvements will be designed in accordance with the requirements of the latest 2007 edition of the California Building Code (CBC). This section presents our recommendations for seismic design parameters in accordance with the 2007 California Building Code (CBC) (CBSC 2007) and ASCE/SEI 7-05 (2006) standard. Based on the limited available field information and using the 2007 CBC Table 1613.5.2, we classify the site as Site Class D. This site is defined as stiff soil profile with average shear wave velocities within the upper 100 feet between 600 ft/s (183 m/s) and 1,200 ft/s (366 m/s), average SPT 15 ≤ N ≤ 50, or average undrained shear strength 1,000 psf ≤ s_u ≤ 2,000 psf.

Based on the Site Class D designation and on the site location (Latitude = 32.9634° and Longitude = -117.0915°) with respect to mapped spectral acceleration parameters S_S and S_1, Kleinfelder developed 2007 CBC seismic design parameters. The recommended seismic design parameters developed from our analyses are presented in Table 1.
### Table 1
Recommended 2007 CBC Seismic Design Parameters

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Symbol</th>
<th>Recommended Value</th>
<th>2007 CBC (ASCE 7) Reference(s)</th>
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<td>Site Class</td>
<td>--</td>
<td>D</td>
<td>Section 1613A.5.2</td>
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<td>Mapped spectral acceleration for short periods</td>
<td>$S_s$</td>
<td>1.01g</td>
<td>Section 1613A.5.1</td>
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<td>Mapped spectral acceleration for a 1-second period</td>
<td>$S_1$</td>
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<td>Section 1613A.5.1</td>
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<tr>
<td>Site Coefficient</td>
<td>$F_a$</td>
<td>1.10</td>
<td>Table 1613A.5.3(1)</td>
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<tr>
<td>Site Coefficient</td>
<td>$F_v$</td>
<td>1.67</td>
<td>Table 1613A.5.3(2)</td>
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<td>MCE* Peak Ground Acceleration (SM at T=0)</td>
<td>$PGA_M$</td>
<td>0.44 g</td>
<td>N/A</td>
</tr>
<tr>
<td>MCE* spectral response acceleration for short periods</td>
<td>$S_{MS}$</td>
<td>1.11 g</td>
<td>Section 1614A.1.1 (Section 21.4)</td>
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<tr>
<td>MCE* spectral response acceleration at 1-second period</td>
<td>$S_{M1}$</td>
<td>0.61 g</td>
<td>Section 1613A.5.3 (Section 21.4)</td>
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<td>$PGA_D$</td>
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<td>Design spectral response acceleration (5% damped) at short periods</td>
<td>$S_{DS}$</td>
<td>0.74 g</td>
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<td>Design spectral response acceleration (5% damped) at 1-second period</td>
<td>$S_{D1}$</td>
<td>0.41 g</td>
<td>Section 1613A.5.4 (Section 21.4)</td>
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</table>

*MCE - Maximum Considered Earthquake

### 3.1.4 Liquefaction

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, silty, sandy and gravelly soils below the groundwater table. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or “cyclic mobility”, increased lateral earth pressures on retaining walls, post liquefaction settlement, lateral spreading and “flow failures” in slopes.
Based on our review of available boring logs information, the potential for soil liquefaction at the site is low due to the presence of dense to very dense soils and the lack of groundwater at the site.

3.1.5 Seismic Compaction

Seismic compaction is a phenomenon in which loose, unsaturated sands tend to densify and settle during strong earthquake shaking. Based on the typically dense soils at the site and the underlying metavolcanic rock, we anticipate that seismically induced settlement due to the design earthquake would be negligible. This should be evaluated in the Draft/Final Foundation Report using suitable analytical techniques such as Tokimatsu and Seed (1987).

3.1.6 Expansive Soil

Expansive soils shrink upon drying and expand upon wetting, which can lead to damage to pavements and foundations. We did not observe soils with moderate or high expansive characteristics during our review of readily available reports. Therefore, the hazard posed to the project by expansive soils is considered to be low.

3.1.7 Tsunami, Seiche and Flood

A tsunami is a sea wave generated by submarine earthquakes, landslides, or volcanic activity that displaces a relatively large volume of water in a very short period of time. Considering that the site lies approximately 16.5 km (10.2 miles) from the ocean shoreline and that the site lies at least 530 feet above sea level, the potential for significant tsunami effects at the site is considered low.

Seiches are defined as oscillations in a body of water due to earthquake shaking or earthquake rupture. The hazard to the project posed by seiches is considered low due to the lack of nearby surface water bodies.

The Federal Emergency and Management Administration (FEMA) maintains a collection of Flood Insurance Rate Maps (FIRM), which covers the entire United States. These maps identify those areas that may be subjected to 100-year and 500-year cycle floods. A set of these maps for the County of San Diego are available for viewing on the SANGIS website (www.sangis.org). Based on our review, the site is within
FEMA flood panel map 1354F and is not mapped within either a 100-year or 500-year floodplain.

3.2 SOIL CORROSiVITY

Caltrans (2003) considers a site to be corrosive if one or more of the following conditions exist for the representative soil samples taken at the site: Chloride concentration is 500 ppm or greater, sulfate concentration 0.2 % or greater, or the pH is 5.5 or less. Corrosivity of the site soils was investigated in Ninyo & Moore’s Geotechnical Evaluation report by collecting a representative sample from the Fill material during field investigation and performing laboratory testing for pH, minimum resistivity, chloride and sulfate content. Corrosion test results are summarized in Table 2.

**Table 2**

**Soil Corrosion Test Summary**
(from Ninyo and Moore)

<table>
<thead>
<tr>
<th>Boring and Sample No's</th>
<th>Depth (m)</th>
<th>Minimum Resistivity (Ohm-cm)</th>
<th>pH</th>
<th>Chloride Content (ppm)</th>
<th>Sulfate Content (%)</th>
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</thead>
<tbody>
<tr>
<td>B-6</td>
<td>0.0 – 5.0</td>
<td>1480</td>
<td>7.7</td>
<td>95</td>
<td>&lt; 0.01</td>
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</tbody>
</table>

Corrosivity test results show minimum resistivity value of 1480 ohm-cm, pH value of 7.7, chloride content of 95 ppm, and sulfate content of less than 0.01 %. Based on Caltrans criteria, the on-site soils would not be classified as corrosive. Although the results of the sulfate test was not significantly high, due to the variability in the on-site soils, the fact that only one sample was tested for corrosivity, and the potential future use of reclaimed water at the site, we recommend additional corrosion sampling and testing as part of the Final Foundation Report.

3.3 FOUNDATIONS

Selection of the appropriate foundation type for support of the parking structure will depend on the magnitude of loads, soil conditions and available space for temporary excavations. The following sections provide recommendations for shallow foundation s.
3.3.1 Shallow Foundations

The allowable bearing capacity of footings is a function of soil strength, footing embedment, groundwater conditions, and slope of the ground adjacent to the footings. In general, footings may be founded in newly compacted fill or the underlying Santiago Peak Volcanics.

The existing fill is undocumented and considered unsuitable for structural support. All existing fill and other unsuitable soil should be removed and replaced with compacted granular structural backfill. The existing soil may be reused for compacted fill.

Spread footings are considered feasible for support of structures and can be founded in either new engineered fill or Santiago Peak Metavolcanics. For preliminary design purposes, we recommend an allowable bearing pressure of 4,000 psf for footings bearing in engineered fill. We recommend an allowable soil bearing pressure of 9,000 pounds per square foot (psf) for footings bearing in the Santiago Peak Metavolcanics. Resistance to horizontal loadings can be developed by passive earth pressure on the side of footings and frictional resistance developed along the footing bottoms. Passive resistance to lateral earth pressures may be calculated using an equivalent fluid unit weight of 330 pounds per cubic foot (pcf). A frictional coefficient of 0.45 may be applied to vertical dead loads supported on formational soils. The passive pressure and frictional resistance can be combined to resist lateral loads.

We recommend that all footing excavations be observed by a geotechnical engineer or engineering geologist prior to placing reinforcing steel or concrete. Any soils disturbed by construction activities or weather should be excavated and recompacted as directed by the Geotechnical Engineer.
4.0 ADDITIONAL GEOTECHNICAL STUDY

We recommend that 6 to 8 additional test borings be advanced within the footprint of the proposed Parking Structure. The test borings should extend to a depth to penetrate through any overburden soil and into the Santiago Peak Volcanics.

Laboratory testing should be performed including but not limited to soil corrosivity tests (pH, soluble sulfate, soluble chloride and resistivity), gradation tests, and shear tests. The exact type and number of tests will depend upon the conditions encountered, and the sample quality that can be achieved with the drilling and sampling methods used.

Final foundation recommendations should be developed based on the additional geotechnical information, structural loads and construction considerations.
5.0 LIMITATIONS

Our firm has prepared this report for the exclusive use of our client for preliminary planning and design purposes only. Additional geotechnical investigation is necessary for final design of the project. We have utilized only existing information and no new subsurface data was collected by Kleinfelder for this study. Kleinfelder is not responsible for the accuracy of the information reviewed for this report. Additional investigation is recommended to provide final engineering recommendations.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive evaluations yield more information, which may help the client understand and manage the level of risk. Since detailed evaluation and analysis involve greater expense, our clients participate in determining levels of service, which provides adequate information for their purposes as acceptable levels of risk. David Evans Associates has reviewed our scope of work and determined that it does not need or want a greater level of service than that being provided for this study. A brochure prepared by ASFE (Association of Firms Practicing in the Geosciences) has been included in this report. All individuals reading this report should also read the attached brochure.

The services provided under this contract as described in this report include professional opinions and judgments based on the data collected. These services have been performed according to our agreed scope of services at the time the report was written. No warranty is expressed or implied. This report is issued with the understanding the owner chooses the risk he/she wishes to bear by the expenditures involved with the construction alternatives and scheduling that is chosen.

Regulations and professional standards applicable to Kleinfelder’s services are continually evolving. Techniques are, by necessity, often new and relatively untried. Different professionals may reasonably adopt different approaches to similar problems. The conclusions and recommendations presented in this report are based on information obtained from the review of documents, our field study, observations of our engineer and geologist, our laboratory testing program, and our experience. It is the client’s responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety.
6.0 REFERENCES


Kennedy, M.P., and Peterson, G.L. (1975), "Geology of the San Diego Metropolitan Area, California," California Division of Mines and Geology, Bulletin 200, Scale 1:24,000, Plate 1A and 2B.


PLATES
December 2007 aerial looking west

Bus staging area
MAGNITUDE : 4 5 6 7 8
SYMBOL : ☀ ☐ ☐ ☐ ☐

FAULT MAP AND EPICENTERS OF EARTHQUAKES

RADIUS OF LARGEST CIRCLE IS 100 KM
APPENDIX A

Proposed Drawings and Improvements
2012 CONSTRUCTED ITEM - ULTIMATE TRANSIT CENTER
(in conjunction with opening full BRT service)

A BRT staging area
B Permanent station amenities (shelters, next-bus, TVM's, etc.)
C Parking structure (~500 spaces)
D Permanent park 'n ride (~88)
APPENDIX B

Previous Log of Test Borings
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<th>DEPTH (feet)</th>
<th>DEPTH (meters)</th>
<th>Bulk Driven</th>
<th>SAMPLES</th>
<th>BLOWS/300 mms (blow/foot)</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WEIGHT kN/m³ (pcf)</th>
<th>SYMBOL</th>
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</table>

**FILL:**
Dark brown, damp to moist, medium dense, silty fine to medium SAND.

Gravel/cobbles encountered at approximately 0.9 m (3').

Brown.

Interlayered with gravel and cobbles.

Layer of yellowish brown; silty fine sand to fine sandy silt.

**SC**
Dark gray, damp to moist, medium dense, clayey fine-grained SAND; scattered pieces of grass (reworked topsoil).

**ML**
ALLUVIUM:
Dark reddish brown, damp to moist, hard, clayey sandy SILT.

TERRACE DEPOSITS: (See description at 6.09 m (20'))

---

**BORING LOG**

SABRE SPRINGS - PENASQUITOS TRANSIT STATION
SAN DIEGO, CALIFORNIA

<table>
<thead>
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<th>PROJECT NO.</th>
<th>DATE</th>
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<td>02/04</td>
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**DESCRIPTION/INTERPRETATION**

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<th>DEPTH (meters)</th>
<th>Bulk Density</th>
<th>Blows/300 mms (blows/foot)</th>
<th>Moisture (%)</th>
<th>Dry Unit Weight (kn/m³ (pcf))</th>
<th>Symbol</th>
<th>Classification U.S.C.S.</th>
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<td>7</td>
<td>100/50 mm (2&quot;)</td>
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</tbody>
</table>

**TERRACE DEPOSITS:**

Light brown, damp, weakly cemented, silty fine-grained SANDSTONE; interbedded with gravel and cobbles.

No recovery. Auger/Sampler Refusal.

Total Depth = 7.07 meters (23.2 feet) (Refusal).

Groundwater not encountered during drilling.

Backfilled and grouted on 01/27/04.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SYMBOL</th>
<th>CLASSIFICATION</th>
<th>DESCRIPTION/INTERPRETATION</th>
</tr>
</thead>
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<td>0</td>
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<td>Fill: Brown, moist, medium dense, clayey fine to coarse SAND.</td>
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<tr>
<td>20</td>
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</table>

**TOTAL DEPTH = 1.52 meters (5 feet).**

Groundwater not encountered during drilling.

Backfilled on 01/27
Date Drilled: 01/27/04

Ground Elevation: 164.3 m (539'± MSL)

Method of Drilling: 200 mm (8") Diameter Hollow Stem Auger

Drive Weight: 63 kg (140 lbs (Auto Trip Hammer)) Drop: 760 mm (30")

Sampled by: RTW
Logged by: RTW
Reviewed by: RI

**Description/Interpretation**

**SM**
- **Fill:**
  - Light brown, damp, medium dense, silty fine to medium SAND.

**Santiago Peak Volcanics:**
- Dark yellowish brown, damp, intensely weathered METAVOLCANIC ROCK.

Total Depth = 1.52 meters (5 feet).
Groundwater not encountered during drilling.
Backfilled on 01/27/04.
**BORING LOG**

SABRE SPRINGS - PENASQUITOS TRANSIT STATION  
SAN DIEGO, CALIFORNIA

<table>
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<th>PROJECT NO.</th>
<th>DATE</th>
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**DATE DRILLED** 01/27/04  **BORING NO.** B-4

**GROUND ELEVATION** 164.6 m (540'± (MSL))  **SHEET** 1 OF 1

**METHOD OF DRILLING** 200 mm (8") Diameter Hollow Stem Auger

**DRIVE WEIGHT** 63 kg (140 lbs. (Auto Trip Hammer))  **DROP** 760 mm (30")

**SAMPLED BY** RTW  **LOGGED BY** RTW  **REVIEWS BY** RI

### DESCRIPTION/INTERPRETATION

**FILL:**
Reddish brown, damp, medium dense, silty fine to medium SAND; little clay; scattered gravel.

Total Depth 1.52 meters (5 feet).
Groundwater not encountered during drilling.
Backfilled on 01/27/04.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
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<th>Bulk密度</th>
<th>SAMPLES</th>
<th>BLOWS/300 mms</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WEIGHT kN/m³ (pcf)</th>
<th>SYMBOL</th>
<th>CLASSIFICATION</th>
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**DATE DRILLED:** 01/27/04  **BORING NO.:** B-5

**GROUND ELEVATION:** 164.6 m (540'± (MSL))  **SHEET:** 1 OF 1

**METHOD OF DRILLING:** 200 mm (8") Diameter Hollow Stem Auger

**DRIVE WEIGHT:** 63 kg (140 lbs. (Auto Trip Hammer))  **DROP:** 760 mm (30")

**SAMPLED BY:** RTW  **LOGGED BY:** RTW  **REVIEWED BY:** RJ

**DESCRIPTION/INTERPRETATION**

**FILL:**
Brown, grayish brown and reddish brown, damp, medium dense, silty fine to coarse SAND; few gravel and cobbles.

Total Depth 1.52 meters (5 feet).
Groundwater not encountered during drilling.
Backfilled on 01/27/04.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>DEPTH (meters)</th>
<th>SAMPLES</th>
<th>BLOWS/1000 ft (blows/foot)</th>
<th>MOISTURE (%)</th>
<th>DRY UNIT WEIGHT kN/m³ (pcf)</th>
<th>SYMBOL</th>
<th>CLASSIFICATION U.S.C.S.</th>
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**DATE DRILLED:** 01/27/04  
**BORING NO.:** B-6  
**GROUND ELEVATION:** 164.6 m (540' (MSL))  
**METHOD OF DRILLING:** 200 mm (8") Diameter Hollow Stem Auger  
**DRIVE WEIGHT:** 63 kg (140 lbs. (Auto Trip Hammer)) DROP 760 mm (30")

**SAMPLED BY:** RTW  
**LOGGED BY:** RTW  
**REVIEWED BY:** RJ  

**DESCRIPTION/INTERPRETATION**

Total Depth 1.52 meters (5 feet).  
Groundwater not encountered during drilling.  
Backfilled on 01/27/04.
DATE DRILLED 01/27/04  BORING NO. B-7
GROUND ELEVATION 165.5 m (543' ± (MSL))  SHEET 1 OF 1
METHOD OF DRILLING 200 mm (8") Diameter Hollow Stem Auger
DRIVE WEIGHT 63 kg (140 lbs. (Auto Trip Hammer)) DROP 760 mm (30")
SAMPLED BY RTW LOGGED BY RTW REVIEWED BY RI

DESCRIPTION/INTERPRETATION

SM
FILL:
Dark reddish brown, damp, medium dense, silty fine to coarse SAND; few gravel.

Total Depth 1.52 meters (5 feet).
Groundwater not encountered during drilling.
Backfilled on 01/27/04.
<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>BLANKS/300 mm (p/300 in)</th>
<th>BLOWNS/300 mm (p/300 in)</th>
<th>DRY UNIT WEIGHT (KN/m3 (pcf))</th>
<th>SYMBOL</th>
<th>CLASSIFICATION U.S.C.S.</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>165 (101)</td>
<td>GM</td>
<td>FILL:</td>
</tr>
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<td></td>
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<td></td>
<td>Brown, damp, medium dense, silty fine to coarse sandy GRAVEL with cobbles.</td>
</tr>
<tr>
<td>2.5</td>
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<td>SANTIAGO PEAK VOLCANICS:</td>
</tr>
<tr>
<td>5</td>
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<td>Brown, damp, intensely weathered, METAVOLCANIC ROCK; some relic structure observed.</td>
</tr>
<tr>
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<td>41</td>
<td>Total Depth = 1.98 meters (6.5 feet).</td>
</tr>
<tr>
<td>2</td>
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<td>Groundwater not encountered during drilling.</td>
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</table>

**DATE DRILLED**: 01/27/04  **BORING NO.**: B-8

**GROUND ELEVATION**: 166.4 m (546± (MSL))  **SHEET**: 1 OF 1

**METHOD OF DRILLING**: 200 mm (8") Diameter Hollow Stem Auger

**DRIVE WEIGHT**: 63 kg (140 lbs. (Auto Trip Hammer))  **DROP**: 760 mm (30")

**SAMPLED BY**: RTW  **LOGGED BY**: RTW  **REVIEWED BY**: RJ

**DESCRIPTION/INTERPRETATION**

GM FILL:
Brown, damp, medium dense, silty fine to coarse sandy GRAVEL with cobbles.

SANTIAGO PEAK VOLCANICS:
Brown, damp, intensely weathered, METAVOLCANIC ROCK; some relic structure observed.

Total Depth = 1.98 meters (6.5 feet).
Groundwater not encountered during drilling.
Backfilled on 01/27/04.
APPENDIX C

ASFE Insert
Important Information About Your Geotechnical Engineering Report

**Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects**

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared solely for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. And no one—not even you—should apply the report for any purpose or project except the one originally contemplated.

**Read the Full Report**

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

**A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors**

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client’s goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it’s changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

**Subsurface Conditions Can Change**

A geotechnical engineering report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical engineering report whose adequacy may have been affected by: the passage of time; by man-made events, such as construction or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

**Most Geotechnical Findings Are Professional Opinions**

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

**A Report’s Recommendations Are Not Final**

Do not overrely on the construction recommendations included in your report. Those recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual
subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report’s recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members’ misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team’s plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer’s Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report’s accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled ‘limitations’ many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a geoenvironmental study differ significantly from those used to perform a geotechnical study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated environmental problems have led to numerous project failures. If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. Do not rely on an environmental report prepared for someone else.

Obtain Professional Assistance To deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant. None of the services performed in connection with the geotechnical engineer’s study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.

Rely, on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.