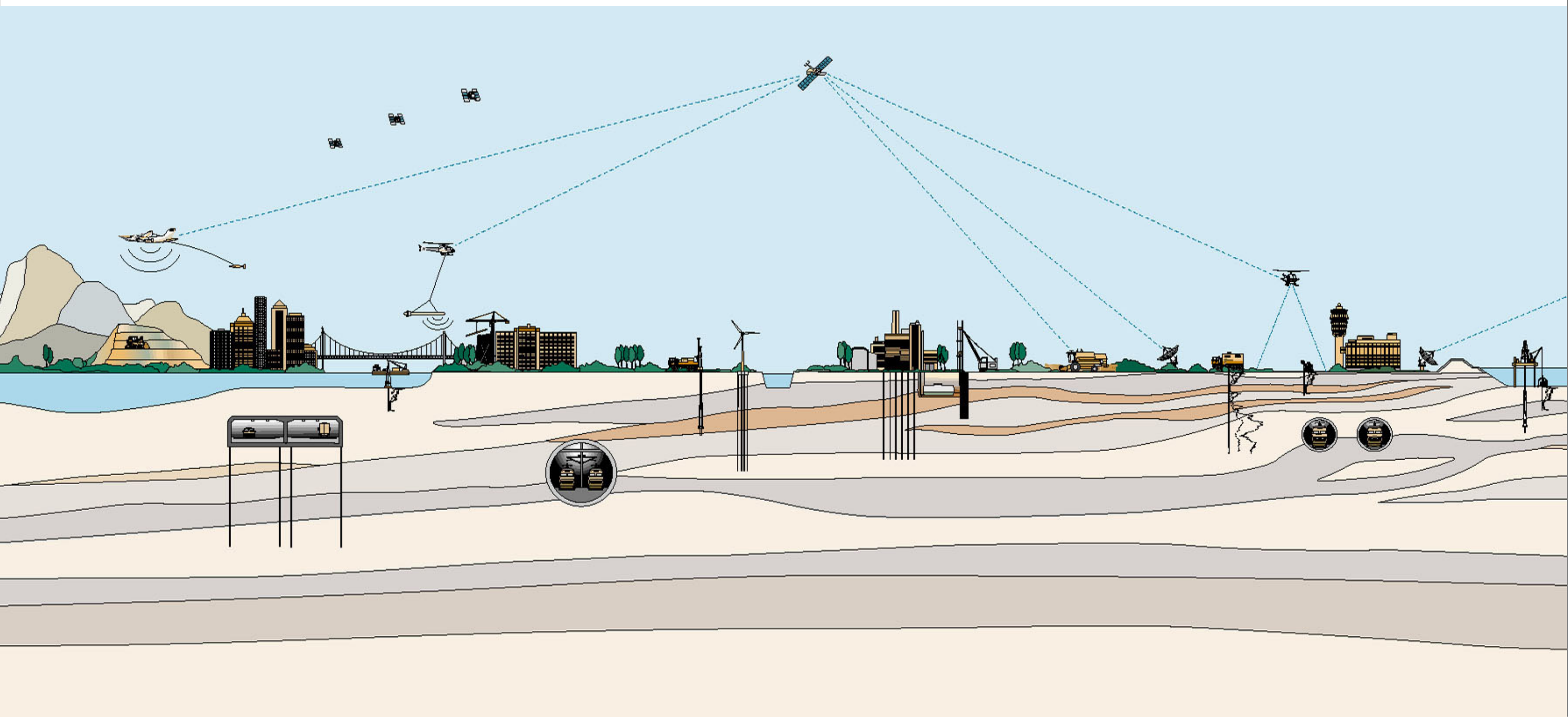
FUGRO CONSULTANTS, INC.

GEOTECHNICAL ENGINEERING REPORT
SAN JOAQUIN RESIDENCE APARTMENTS AND PRECINCT IMPROVEMENTS
UNIVERSITY OF CALIFORNIA, SANTA BARBARA
SANTA BARBARA, CALIFORNIA

Prepared for:
UNIVERSITY OF CALIFORNIA, SANTA BARBARA

July 2013
Fugro Project No. 04.62130070



FUGRO CONSULTANTS, INC.

July 29, 2013
Project No. 04.62130070

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University of California at Santa Barbara
Office of Planning and Construction, Building 439
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Attention: Ms. Deedee Ciancola

Subject: Geotechnical Engineering Report, San Joaquin Residence Apartments and Precinct Improvements, University of California, Santa Barbara, California

Dear Ms. Ciancola:

Fugro is pleased to present this Geotechnical Engineering Report for the design of the proposed San Joaquin Residence Apartments and Precinct Improvements project located on the West Campus of the University of California, Santa Barbara (UCSB). Authorization for our services was provided by the University of California Fund No.130586/986470, Authorization No. 01 dated June 24, 2013.

This report presents the findings of our subsurface exploration and laboratory testing programs and provides ground motion/seismic data and geotechnical recommendations for site development and grading, foundation design, pavements and the feasibility of using infiltration Best Management Practices (BMPs) in managing storm water runoff. Field and laboratory data collected for the project are included in this report.

We appreciate the opportunity to work on this interesting project and to continue our professional relationship with the University of California, Santa Barbara. Please call our office if you have any questions regarding the findings, conclusions, or preliminary recommendations provided in this report.

Sincerely,

FUGRO CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORY S. DENLINGER
No. 2249
GEOTECHNICAL
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1.0 INTRODUCTION

This report presents the findings of our subsurface exploration and laboratory testing programs and provides ground motion/seismic data and geotechnical recommendations for site development and grading, foundation design, and pavements. The study also provides information regarding the feasibility of using infiltration Best Management Practices (BMPs) in managing storm water runoff. Field and laboratory data collected for the project are included in this report.

1.1 PROJECT DESCRIPTION

We understand that the University of California, Santa Barbara (UCSB) is planning to expand the student housing at the existing Santa Catalina Residence Hall facility (Santa Catalina) located at the northeast corner of Storke Road and El Colegio Road in the West Campus area. UCSB refers to the proposed housing expansion project as the San Joaquin Residence Apartments and Precinct Improvements. The general location of the existing Santa Catalina facility is shown on Plate 1 - Vicinity Map. On a conceptual basis, we understand the proposed expansion project will consist of constructing a new 3-story dining commons and residence building, two 6-story residence towers, and several 2- to 3-story apartment-type student housing structures. The project will also involve redesigned parking lots and infrastructure. The areas of new construction are currently occupied by surface parking and open space. The layout of the site with the proposed improvements is shown on Plate 2 - Subsurface Exploration Plan.

1.2 SITE DESCRIPTION

The site is bordered to the south by El Colegio Road, to the west by Storke Road, on the north by the existing Storke Ranch housing development (private development) and on the east by an existing open space/wetlands area. As discussed above, the site is generally developed and is occupied by two, 11-story student housing residence towers connected by a central dining commons, large asphalt-paved parking areas, and grass-covered open space. Other existing infrastructure at the site consists of an outdoor pool, concrete sidewalks, and a paved bike parking area. The existing site terrain is relatively flat with grades that range from approximately elevation (El.) 15 to 40 feet.

2.0 WORK PERFORMED

2.1 PURPOSE

The purpose of this geotechnical engineering report is to evaluate the geotechnical conditions on the basis of the available existing data and our project-specific subsurface exploration and laboratory testing programs, and develop recommendations for the design of structure foundations, pavements, and site development and grading.

2.2 SCOPE OF WORK

Our services for this project were performed in general accordance with our proposal for geotechnical services dated June 3, 2013. A summary of the work performed is provided below:

- Prepared a health and safety plan for our work and coordinate site access with the University;
- Visited the site to observe the general site conditions, coordinate the field exploration program, mark exploration locations and clear the locations for utilities with Underground Services Alert and the University;
- Compiled and reviewed selected geotechnical reports and documents from our in-house files and previously obtained from UCSB;
- Explored the subsurface conditions at the site by drilling 20 hollow-stem-auger drill holes to depths ranging from approximately 15 to 50 feet below the existing ground surface;
- Performed 2 seismic cone penetration test soundings (seismic CPTs) at the site near locations recommended by others. We note that a third seismic CPT was performed because of shallow refusal of one of the two planned soundings;
- Performed 11 percolation tests across the site to evaluate the ability of the near surface soils to infiltrate surface water runoff;
- Performed laboratory testing on selected samples obtained from the field exploration to help classify the materials encountered and characterize their geotechnical properties;
- Evaluated data collected from the literature review, field explorations, and laboratory tests; and
- Prepared this report summarizing the findings of the study and providing our conclusions and recommendations regarding:
    - Soil and groundwater conditions encountered at the site;
    - Seismic setting, geotechnical parameters and ground motion for seismic design in accordance with the 2010 California Building Code;

- Liquefaction potential and estimated seismic settlement;
- Site preparation and grading, subgrade stabilization, and soil material and compaction requirements for on-site and imported soil materials;
- Suitable foundation support for the building such as from drilled shafts, driven piles, or shallow foundations;
- Shallow foundation design: bearing pressures, foundation embedment depths, and anticipated settlement;
- Passive pressure and friction coefficient for shallow foundations resisting lateral loads;
- Drilled shaft design: estimated axial capacity vs. depth, and lateral load versus pier head deflection for drilled piers (if shallow foundations are not feasible);
- Lateral earth pressures and sliding resistance for the design of retaining walls, and recommended backfill, compaction, and drainage of those walls;
- Design of concrete slabs-on-grade and flexible pavements;
- Considerations for the design of site drainage in building and pavement areas;
- Corrosion and swell potential of on-site soils; and
- Considerations for the contractor to design temporary slopes and shoring systems for excavations, adjacent structure, and adjacent utilities.

2.3 FIELD EXPLORATION

The field exploration program consisted of excavating 20 drill holes at the site to depths ranging from approximately 15 to 50 feet below the ground surface and advancing two seismic cone penetration test soundings (SCPTs) at the site. A third SCPT was added to the project due to shallow refusal of one of the planned soundings. The exploration program also included drilling of shallow drill holes for percolation testing. We performed the field exploration program for the project between June 17 and June 21, 2013. The drilling subcontractor for the project was Martini Drilling and the SCPT subcontractor was Kehoe Testing. Both subcontractors are from Huntington Beach, California.

Martini Drilling excavated the drill holes using a truck-mounted CME 75 drill rig equipped with 8-inch diameter hollow stem augers. The drill holes were sampled using a 2-inch-outside-diameter standard penetration test (SPT) split-spoon sampler and a 3-inch-outside-diameter modified California split-spoon sampler. The modified California sampler was equipped with 1-inch-high brass ring liners. The SPT sampler was used without liners. The samplers were driven into the materials at the bottom of the drill hole using a 140-pound automatic trip hammer with a 30-inch drop. The blow count (N-value) shown on the drill hole logs is the number of blows from the hammer that were needed to drive the sampler 1 foot, after the sampler had been seated at least 6 inches into the material at the bottom of the hole. Bulk samples were collected from the drill cuttings retrieved from the auger flights. The drill holes were backfilled with soil cuttings.

Holes excavated in paved areas were surface patched using cold mix asphalt or quick set concrete.

Kehoe Testing performed the SCPTs using a standard 30-ton truck, and performed shear wave velocity measurements at 5-foot intervals to a depth of 50 feet and at 10-foot intervals below that depth.

The approximate locations of the drill holes and SCPTs performed for this study are shown on Plate 2 - Subsurface Exploration Plan. The locations of previous subsurface explorations performed by Fugro and others are also shown on Plate 2. Drill hole logs for explorations performed for this work and logs of the seismic cone penetration test soundings are provided in Appendix A - Field Exploration. Data from the seismic cone penetration test soundings provided to Fugro by Kehoe Testing is provided in Appendix C - Seismic Cone Penetration Test Data From Kehoe Testing.

2.4 LABORATORY TESTING

Laboratory tests were performed on selected soil samples retrieved during the exploration program. The laboratory testing program for this project included moisture and density relationships, grain size analyses, Atterberg limits, direct shear, consolidation-swell tests, corrosion, unconsolidated undrained triaxial compression, compaction, R-value, and expansion index tests. The tests were performed in general accordance with the applicable standards of ASTM. The results of the geotechnical laboratory testing performed for this study are provided in Appendix B - Laboratory Testing.

2.5 PREVIOUS GEOTECHNICAL STUDIES

Previous geotechnical studies have been performed at or adjacent to the proposed site. Pertinent information from the previous studies was reviewed as part of our work. A summary of the reports reviewed are summarized below:

- Fugro (2002): Fugro prepared a Preliminary Fault Study report as part of a pre-acquisition geologic site assessment for USCB. The study provided preliminary information regarding the potential location of the south branch of the More Ranch fault extending through the site. Field explorations for that study consisted of drilling 7 hollow stem auger drill holes and advancing 9 cone penetrometer test (CPT) soundings. The locations of the explorations performed as part of the Fugro (2002) study are shown on Plate 2.
- Fugro (2012): Fugro prepared a Fault Study for the site and provided recommendations for structure setbacks and defined a zone where potential ground warping could occur in response to fault movement. The findings were based on field data obtained from a large number of CPT soundings, continuously cored drill holes, and fault trench excavations. Detailed geologic cross sections were developed from the data. The locations of the explorations performed as part of the Fugro (2012) study are provided on Plate 2.

- AMEC (2013a): AMEC prepared a Report of Ground Motion Studies for the San Joaquin site and provided horizontal and vertical response spectra for design and maximum considered earthquake ground motions. AMEC performed ReMi surveys along three profile lines at the site to develop generalized shear wave velocity profiles.
- AMEC (2013b): At the request of UCSB, AMEC prepared a Fault Displacement Hazard Evaluation for the San Joaquin Apartments and Precinct Improvement project. The report provides estimates of displacement on the faults at the site reported in Fugro (2012) and provides an estimate of potential tilting or ground warping that could occur at the site in areas adjacent to the faults. This report has currently been submitted to UCSB in draft form.

2.6 GENERAL CONDITIONS

Fugro prepared the conclusions and professional opinions presented in this report in accordance with generally accepted geotechnical engineering principles and practices at the time and location this report was prepared. This statement is in lieu of all warranties, expressed or implied.

This report has been prepared for the University of California Santa Barbara and their authorized agents only. It may not contain sufficient information for the purposes of other parties or for other uses. If any changes are made in the project or site conditions as described in this report, the conclusions and recommendations contained in this report should not be considered valid unless Fugro reviews the changes and modifies and approves, in writing, the conclusions and recommendations of this report. The report and drawings contained in this report are not intended to act as construction drawings or specifications.

Soil and rock deposits will vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions can vary seasonally or for other reasons. Therefore, we do not and cannot have complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed during construction.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements or absence of statements in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

3.0 SUBSURFACE CONDITIONS

3.1 GEOLOGIC SETTING

The UCSB campus is situated on the coastal plain south of the Santa Ynez Mountains. The Santa Ynez Mountains are part of the western Transverse Ranges, a predominantly east-west trending mountain block extending from Point Arguello eastward for 75 miles into Ventura County. The Santa Ynez Mountains and adjacent piedmont alluvial plain are composed almost entirely of sedimentary rocks ranging in age from late Jurassic to Recent.

In the Santa Barbara and Goleta area, the structure of the Santa Ynez Mountains consists of a south-dipping homocline with east-west striking faults and related folds preserved on the coastal plain (Dibblee, 1966). Late Pleistocene uplift has created the elevated UCSB-Isla Vista-Devereaux marine terrace. The More Ranch/Mission Ridge/Arroyo Parida faults are the principal fault system on the coastal plain, and form the northern boundary of this marine terrace.

The project site is located near the northern boundary of the marine terrace. The marine terrace is a wave-abraded surface that is typically covered with a thin veneer of marine sands and overlying undifferentiated alluvium. The base of the marine terrace is typically gently sloping to the south to generally flat-lying. Stream erosion has subsequently dissected the terrace to produce the present isolated mesa surfaces with intervening drainages. The marine terrace is composed of late Pleistocene age marine sand, with discontinuous, basal, fossiliferous sand and is overlain locally by undifferentiated alluvium of estuarine and non-marine deposition. These units are undifferentiated and included with the classification of Quaternary Marine Terrace deposits as mapped by Minor et al. (2009) and Gurrola (2004).

In the vicinity of the San Joaquin site, marine terrace sediments unconformably overlie a Quaternary/Tertiary age marine siltstone unit believed to be the Pico Formation bedrock (QTp) shown on Gurrola (2004) or the siltstone unit (QTst) shown on Minor et al. (2009). That marine siltstone is referred to as Pico Formation (in quotes) by Dibblee (1966) because it is not directly traceable to the Pico Formation in Ventura County. We understand that microfaunal analyses of similar siltstone samples performed by the USGS and obtained from borings excavated for the San Clemente housing project east of the site confirm that the fossil assemblage in the bedrock is slightly older than the Santa Barbara Formation (email correspondence from Rick Stanley of the USGS to Roger Slayman of CFS in 2002). The Pico Formation rests unconformably on the Sisquoc Formation in the project area.

On the basis of data in Fugro (2002) and data from this study, the Pico Formation appears to consist of two distinct units consisting of 1) a fine-grained siltstone to clayey siltstone unit and 2) a coarse-grained silty sandstone to sandstone unit. On the basis of discussions with Larry Gurrola (and USGS microfaunal data from the San Clemente housing project), we have assumed the two units can be assigned to the Pico Formation. However, the siltstone and sandstone units from the site have not been directly correlated by others. Using sediment samples collected from the San Joaquin site, we have obtained radiocarbon ages of >43,000 ybp from the Pico Formation, which indicates the deposits are older than the useful range of radiocarbon techniques.

4.0 LOCAL GEOLOGIC CONDITIONS

As encountered in the drill holes performed for this project and previous borings and CPT soundings performed at the site by Fugro, the site is underlain by artificial fill, undifferentiated alluvium, marine terrace deposits, and Pico Formation claystone and sandstone. Although USGS classifies the fine grained Pico Formation in this area as siltstone, our laboratory testing and field observations indicate that the material is classified as claystone. The following describes the geologic units encountered in the drill holes and CPT soundings. The locations of all the CPT soundings, drill holes, and fault trench excavations performed for at the site by Fugro are shown on Plate 2 - Subsurface Exploration Plan.

Preliminary logs for the drill holes performed for this study are provided in Appendix A. Logs of CPT soundings and drill holes performed previously by Fugro are provided in Fugro (2002) and Fugro (2012), respectively. Logs of the fault trench excavations and age dating information are also provided in Fugro (2012) and are not reproduced herein.

Descriptions of the predominant soil and bedrock units encountered in our explorations are presented below.

4.1 ARTIFICIAL FILL (AF)

We encountered up to about 3 feet of artificial fill material in the drill holes excavated for Fugro (2002) and a thin veneer was encountered in the (pre-punch) hand-auger excavations at the CPT locations performed for this study. The fill encountered consists of firm to stiff silty clay (CL-ML) to sandy clay (CL). The artificial fill materials encountered appear to be associated with previous site grading performed for utilities and parking areas, and driveways. In the parking lot areas, the fill appears to be covered by 2 to 4 inches of asphalt pavement. However, pavement thicknesses up to about 8 inches were encountered in drill holes along the primary east-west access road. The artificial fill materials are underlain by undifferentiated alluvial deposits.

Artificial fill was placed in the fault trenches excavated as part of our 2012 study. Fill materials were placed and compacted using a sheepsfoot compactor attached to an excavator. A Fugro technician was on-site periodically onsite to observe the fill placement and perform compaction tests on the fault trench fill. The fill in these areas extends to depths of about 12 to 15 feet below the ground surface.

4.2 UNDIFFERENTIATED ALLUVIUM (QAL)

Undifferentiated alluvial deposits of non-marine and estuarine depositional origin are present at the site. In general, the undifferentiated alluvium consists of medium stiff to hard lean to fat clay, and sandy lean to fat clay and clayey sand. On the basis of our data, the undifferentiated alluvium extends from near the ground surface to depths of up to about 15 to 25 feet. On the basis of the CPT and drill hole data acquired for the project, alluvial channel deposits are present in the southern portion of the site and consist of interlayered/interbedded medium stiff to stiff silty sand, silt, and lean to sandy lean clay. The undifferentiated channel deposits are present over a zone about 150 feet wide and extend from near the ground surface to a depth of about 20 to 25 feet. Channel deposits were also encountered in the northeast portion of the site. The channel deposits encountered in that location are present as a remnant narrow and incised channel that is most pronounced slightly northeast of the north tower.

4.3 MARINE TERRACE DEPOSITS (QT)

Marine terrace deposits consisting of dense to very dense silty sand to sand were encountered in our CPT soundings, borings, and trenches, and in previous explorations performed by Fugro. The marine terrace deposits were encountered in the CPT soundings performed in the northern portion of the site as a continuous, relatively uniformly thick (about 15 feet thick) stratum of silty sand to sand. Similar marine terrace deposits are also present in the southern portion of the site but appear to have been locally eroded and replaced by undifferentiated alluvial deposits. The sandy marine terrace deposits are underlain by Pico Formation bedrock.

4.4 PICO FORMATION (QTP)

Pico Formation was encountered below the marine terrace deposits. The Pico Formation is composed of weakly to non-cemented units of dark greenish gray claystone and yellowish brown sandstone. The claystone unit was generally encountered in the northern half of the site and within 50 to 200 feet of the southern boundary of the site. Sandstone bedrock units were encountered in the south-central portion of the site and are bounded by claystone bedrock to the north and south.

Bedding observed in the soil cores obtained from the drill holes indicate that layers within the Pico Formation range in dip from flat-lying up to about 58 degrees.

4.5 GROUNDWATER CONDITIONS

Groundwater was encountered in drill holes excavated for this study at depths of about 15 to 20 feet below the ground surface. However, locally groundwater was measured in drill holes DH-3 DH-6, and DH-7 excavated for this study at depths of 8 to 11 feet. The depth to groundwater was measured during drilling and the recorded values may not represent stabilized water levels.

Four vibrating wire piezometers were installed at the site in March 2012 as part of the Fugro (2012) study. The piezometers were installed in a vertically stacked arrangement with two piezometers grouted in BH-02 (at depths of 27 feet and 45 feet) and two grouted in BH-07 (at depths of 20 feet and 40 feet). Initial piezometer readings were obtained on March 21, 2012, and follow-up readings were obtained on May 15, 2012. The piezometers in BH-02 indicated that the groundwater was at about an El. of 12.4 to 13.7 feet and the piezometers in BH-07 indicated that the groundwater was at about an El. of 7.6 to 9.6 feet. Groundwater was also encountered at an El. of about +12 feet in fault trenches T-1 and T-2. Groundwater was not encountered in fault trench T-3, excavated to a depth of about 15 feet below the ground surface (about El. 18 feet).

Groundwater depths and soil moisture conditions will vary seasonally depending on rainfall, irrigation, storm runoff and other factors.

4.6 PERCOLATION TESTING AND EVALUATION OF SURFACE WATER INFILTRATION

4.6.1 Bore Hole Infiltration Testing

We conducted a total of thirteen percolation tests at the San Joaquin Student Housing site to evaluate the ability of the on-site, near-surface soils to infiltrate storm water. We performed these tests at percolation test hole locations A through M. The approximate locations of the planned percolation tests are shown on Plate 2. The tests were carried out utilizing a falling-head procedure in general accordance with the provisions outlined in the City of Santa Barbara's Storm Water BMP Guidance Manual (City of Santa Barbara, 2008). We understand the County of Santa Barbara has not developed internal guidelines for storm water management but considers the City of Santa Barbara's manual as an acceptable guidance document.

Percolation test holes were constructed in conjunction with the subsurface exploration tasks by drilling a 12-inch diameter hole or hand dug excavation. Excavations were subsequently pre-soaked and constructed by inserting a perforated plastic casing, and backfilling the annular space with gravel. The test holes were pre-soaked overnight on the day before the test by filling with water to the ground surface. The falling head tests were performed in the prepared bore holes. Readings were recorded at 30-minute intervals until a stabilized percolation rate was attained. The water level in the hole was adjusted or reset during the test as needed.

4.6.2 Percolation Test Results

The results of the individual percolation tests are provided in Table 1 - Percolation Test Data. On the basis of these tests, the stabilized percolation rates ($P_{measured}$) measured at the site range from less than 0.03 up to about 1.4 inches per hour (in/hr). The presented measured rates do not include correction factors as outlined by the City of Santa Barbara (2008). We assume that required safety factors will be selected and applied by the storm water infiltration system designer based on design considerations

Table 1. Percolation Test Data

Test Location	Test Depth (feet)	Cumulative Test Run Time (hr)	Estimated Stabilized Percolation Rate (in/hr)
A	4.8	3.5	0.14
B	2.6	4	1.4
C	3.0	N/A[1]	N/A
D	2.5	2	1.4
E	2.0	1	<0.12[2]
F	2.5	1	<0.24[2]
G	2.5	0.75	<0.16[2]
H	2.5	1	<0.12[2]
I	2.5	3.5	<0.03[2]
J	4.3	4	<0.03[2]
K	2.5	1	0.24
L	3.0	1	<0.12[2]
M	3.3	1	<0.12[2]

1) Percolation pre-soak volume not absorbed, test hole caved in, unable to clear hole and run test.

2) No actual drop in water level recorded. Estimated infiltration rate represents an estimated maximum, best case scenario assuming 0.01 ft (1/8 in) measurement error over total test duration. Actual percolation rates could be lower based on longer duration testing.

Infiltration Best Management Practices (BMPs) relying upon some infiltration component to manage stormwater flow should be set back from any structural foundation for buildings or other site structures (e.g., retaining walls) by 10 feet to reduce the potential for expansive soil effects and/or moisture intrusion. In addition, measures to maintain subgrade stability will be required if infiltration is incorporated into the project design.

5.0 SEISMIC CONDITIONS

5.1 LOCAL SEISMICITY

Regional north-south directed compressive strain in the Santa Barbara coastal area has resulted in generally east-west trending folds and faults. Gurrola and Keller (1998) describe the coastal plain region as the Santa Barbara Fold Belt (SBFB) characterized by active folding and buried reverse faulting.

The project is located in a seismically active area of southern California. As a result, significant ground shaking in response to local and/or regional earthquakes should be expected at the project site in the future. To aid in evaluating the fault and seismic conditions at the site, we used the computer program EZ-FRISK (Risk, 2012) to assess potential local and regional fault sources and fault source-site distances. The maximum characteristic earthquake magnitudes are defined as the maximum earthquake that appears to be capable of occurring within the known tectonic framework.

EZ-FRISK uses the 2008 USGS fault model (Peterson et al., 2008). The 2008 model is an update of the previous model developed by California Geological Survey (Cao, et al., 2003). The 2008 model incorporates some significant changes to the fault models in the project area. The changes include adjustment of parameters (i.e., dip angle, magnitude) of some faults that were included in the 2003 CGS model, as well as reassessment of fault locations and components. In summary, the new model recognizes the higher seismic hazard present in the project areas compared to the modeling done in the past, particularly some of the local faults in the Santa Barbara area that were not included in the previous model.

The summary of selected nearby and significant faults, the distance to selected faults, and the estimated maximum characteristic earthquake magnitudes are summarized in Table 2 - Selected Faults. The listed faults were selected on the basis of their proximity to the site, and their potential to generate strong ground motion at the site.

The proposed project is not located within and does not cross an Alquist-Priolo fault rupture hazard zone. However, the More Ranch Fault is known to pass through the site on a generally east/west strike. Our previous fault study (Fugro, 2012) provided structure setback zones around the areas found to exhibit faulting, but ground warping and minor shearing is still a concern in such close proximity to a fault trace. Surface fault rupture potential and consequences are discussed in Section 5.2 below. Table 2 provides a list of potentially significant faults within about 20 miles from the project site

We used the program EZ-FRISK (Risk, 2012) to perform a search of the active or potentially active faults mapped within a 24-mile (38-km) radius of the site. The site location was estimated as -119.8680 degrees longitude and 34.4186 degrees latitude. Summarized below are 10 faults and fault segments that were found to contribute most to the seismic hazard at an amplitude of 1.0g based upon a deaggregation of the seismic hazard on-site from EZ-FRISK. We note that the closest distance from the site to the Mission Ridge-Arroyo Parida-More Ranch fault determined using the 2008 USGS fault model is 0.7km. The USGS (2008) fault model maps the south branch of the More ranch fault at the site, so the closest distance of 0.7 km is likely the result of dipping fault geometry the depth to the rupture zone. Additional information is provided for these sources in the USGS (2008) fault database.

Table 2. Selected Faults

Fault	Closest Distance (km)	Maximum Moment Magnitude (Mw)
Red Mountain	3.1	7.4
Pitas Point (Lower, West)	4.9	7.3
North Channel	4.0	6.8
Pitas Point (Upper)	6.9	6.9
Mission Ridge-Arroyo Parida-Santa Ana-More Ranch	0.7	6.9
Pitas Point (Connected)	7.9	7.3
Pitas Point (Lower) - Montalvo	12.3	7.3
Santa Ynez Connected	14.0	7.4
Santa Ynez (West)	14.0	7.0
Channel Island Thrust	35.0	7.3

Notes:

[1] Distance and maximum magnitude values per fault location based on 2008 USGS fault model as coded into computer program EZ-FRISK.

Other faults have been mapped in the vicinity of the site (less than 5 to 10 km from the site) by others (Dibblee 1987, Olson 1982, Gurrola 2004). In general, those faults consist of Coal Oil Point and the Goleta Point faults located offshore of the main campus and the Dos Pueblos, Glen Annie, Carneros, Goleta, and the San Jose faults generally located in the foothills of the Goleta Valley. The Briggs Lineation/Campus fault has been mapped by Gurrola and Alex (1997) trending northeast-southwest through the north-central portion of the UCSB campus. The Briggs Lineation/Campus fault and the other Goleta Valley foothill faults are not included in the 2008 fault model; therefore, they were not considered as potential significant seismic sources in our evaluation of ground motion parameters for the project.

5.2 FAULT RUPTURE HAZARDS

Faults associated with the South Branch of the More Ranch fault are present on the San Joaquin site. Fault locations and structure setbacks from the mapped faults are provided in Fugro (2012). The structure setbacks are proposed to avoid ground rupture damage to the proposed new structures and facilities related to displacements on the mapped faults. The potential for primary fault rupture outside the structure setbacks is likely to be low to very low.

Fugro (2012) identified two zones of Pleistocene faulting or warping where measures should be incorporated into the design of the foundation and structural systems to accommodate some potential fault-related ground deformation. The zones are located in the southeast corner of the site and in the north-central portion of the site north of the existing north residential building tower.

At the request of UCSB, AMEC (2013b) quantitatively evaluated ground deformation hazards at the site related to movement on the three traces of the South Branch of the More Ranch fault identified in Fugro (2012). Their analyses included both deterministic and probabilistic fault displacement hazard analyses. AMEC (2013b) concluded that it is likely that surface deformation of about 2.6 to 3.0 feet due to fault rupture will occur by discrete displacement (predominantly reverse with a subordinate lateral slip component) on one or more of the mapped fault traces, with associated tilting or warping of the ground surface between the faults.

AMEC (2013b) indicated that ground tilting on the order of about 3 to 13-½ inches over a distance of 100 feet, may occur in areas between and adjacent to faults (specific amounts and directions of differential displacement are provided in the AMEC report). AMEC (2013b) noted that there is considerable uncertainty in their estimates of fault displacement, tilting, and differential displacement, and that the actual displacements may be significantly different than the ranges they estimate. Figure 5 of AMEC (2013b) identifies five zones or areas of the site and provides estimates ground warping or tilting that could occur in those zones. Estimates of ground deformation or tilting for the five zones are summarized below:

Zone	General Site Region	Estimated Range of Ground Deformation/Tilting Over a Distance of 100 feet
Zone 1	Approximately northern half of the site	3 to 5 inches
Zone 2	Approximately the area occupied by the existing north residence tower	3 to 5 inches
Zone 3	Eastward extension of Zone 2 to the eastern site boundary	7 to 13-1/2 inches
Zone 4	Approximately the southwest quarter of the site	6-1/2 to 12 inches
Zone 5	Approximately the southeast quarter of the site	7 to 12 inches

We note that the estimates of ground deformation or tilting reported in AMEC (2013b) may have a significant impact on the design of the structure foundation systems selected for the project. Methodology for considering the potential warping or tilting reported in AMEC (2013b) in the design of the proposed structures and foundations was developed by UCSB and is described in a letter dated July 22, 2013 (UCSB, 2013). The methodology consists of consider the warping and tilting as normal differential settlement with collapse prevention as the appropriate limit state.

Due to the importance and potential impacts of the findings reported in AMEC (2013b) we strongly suggest an independent peer review of the AMEC report and findings be performed. Fugro can assist UCSB is facilitating a team to perform the peer review if requested.

5.3 SHEAR WAVE VELOCITY PROFILE

As described in Section 2.3, seismic CPT shear wave velocity profile measurements were performed at three locations on-site to obtain shear wave velocity profile data for the onsite soils. The shear wave velocity profile in the upper 100 feet of soil strata is an important parameter that can be used to categorize the site class per ASCE 7-10 or to modify the site response based upon near surface soil conditions in a site-specific analysis. The seismic CPT's were performed by Kehoe Testing and Engineering of Huntington Beach, California.

The previous geophysical ReMi surveys conducted by AMEC (2013a) provided three additional generalized shear wave velocity profiles down to 100 foot depths for comparison with our seismic CPT results. Based upon these sources of data we assumed a V_{s30} (shear wave velocity in the upper 30 m or 100 ft) equal to 1000 ft/s across the site. This indicates that the site falls under Class "D" conditions. The design response spectrum (generated as discussed in Section 5.5 below) was therefore developed for Site Class "D" conditions and a V_{s30} value equal to 1000 ft/s or 305 m/s.

Shear wave velocity data provided to us by Kehoe Testing for SCPT-1, SCPT-2, and SCPT-3 are provided in Appendix A and Appendix C. In general, the shear wave velocity data obtained from the SCPTs generally support the shear wave velocity data interpreted by AMEC from the ReMi data. However, we recommend the shear wave velocity data be reviewed by AMEC and the seismic design criteria developed for the project (AMEC 2013a) be updated as necessary to consider the shear wave velocity data obtained from the SCPTs.

5.4 STRONG GROUND MOTION AND SITE RESPONSE SPECTRA

On the basis of discussions with UCSB and the project team, we understand that the proposed structures will be designed for the site-specific response spectra developed by AMEC and provided in their report dated March 29, 2013 (AMEC 2013a).

We used the site-specific peak ground acceleration of 0.73g for the site as reported in AMEC (2013a) and a mean earthquake magnitude of 7.0. We selected the mean magnitude of 7.0 using the USGS probabilistic seismic hazard analysis deaggregations web application (USGS, 2008)

5.5 LIQUEFACTION AND SEISMIC SETTLEMENT

Liquefaction is the phenomenon in which saturated sandy and silty sediments temporarily lose their shear strength due to increased pore pressures during periods of dynamic loading. The susceptibility of soils to liquefaction is a function of the distribution of grain sizes (gradation), soil density, cementation, total fines content, and plasticity characteristics of the fines. The resistance to liquefaction increases with increasing: a) soil density, b) age/cementation, c) fines content, and e) plasticity characteristics of the fines.

According to Seed (1979), at most of the sites where some surface evidence of liquefaction has been observed in the field, the critical layer in which liquefaction is believed to have occurred has been located at depths of less than 45 feet and the depth of the groundwater table has been less than 15 feet. However, liquefaction has been known to occur during earthquakes at depths deeper than 50 feet given the proper conditions such as low-density granular soils, presence of groundwater, and sufficient cycles of earthquake ground motion

(Martin and Lew, 1999). Settlement, lateral spreading, sand boils, and loss of bearing support are common manifestations of liquefaction.

The framework we selected for performing the liquefaction analyses for the proposed San Joaquin project is principally based on an evaluation of liquefaction triggering using the analytical procedures described in Youd et al. (2001) and primarily involved using the CPT data acquired as part of our fault study report (Fugro, 2012). Seismic parameters used in our analyses consisted of a peak ground acceleration of 0.73g and an estimated earthquake magnitude of 7.0. As noted, the peak ground acceleration and earthquake magnitude for liquefaction analysis were selected based on our review of the probabilistic seismic hazard analyses (PSHA) provided in AMEC (2013a) and our analyses using the USGS deaggregations web application (USGS, 2008). For the purpose of our analyses, we assumed the groundwater level to be at a depth of 15 feet below the ground surface based on the groundwater data available from our past and project-specific subsurface explorations at the site.

Our evaluation of liquefaction for the project was limited to the empirical liquefaction analysis procedures described in Youd et al. (2001) developed from research of case histories of liquefaction. This procedure is widely used and accepted for evaluating liquefaction potential. However, the empirical procedure (and other similar empirical procedures developed from case histories) has a number of limitations including:

- Limited case histories used to develop the analytical procedures;
- Uncertainty associated with the soil correlations, soil parameters, liquefaction susceptibility of silts and clayey soils, etc., and;
- Free field conditions are modeled and soil structure effects are not accounted for.

Considering that liquefaction occurs from an increase in pore water pressure in the soil in response to seismic shaking, the soils above the groundwater level can be considered to be non-liquefiable. In addition, in our opinion, the stiff to hard fine-grained clayey soils associated with the undifferentiated alluvial stratum and the claystone bedrock are also not susceptible to liquefaction due to the plasticity characteristics and stiff to hard consistency. We believe these soils behave in a "clay-like" manner (as described in Idriss and Boulanger 2008) and fall within the region described as "Not Susceptible" to liquefaction in Bray and Sancio (2006).

Soils present at the site that in our opinion could potentially be susceptible to liquefaction consist of the interbedded granular soils within the undifferentiated alluvial stratum and located below the groundwater level, the granular marine terrace deposits below the groundwater level, and the sandstone bedrock materials that are local present in the southern portion of the site.

The results of our analyses indicate that these units are generally not susceptible to liquefaction assuming an average groundwater depth of 15 feet below the existing ground surface. However, in our opinion there is a possibility for some limited localized liquefaction to occur in a few isolated soil layers if the groundwater level is assumed to be 10 feet below existing grade. The consequences of limited and localized liquefaction occurring at the site could consist of local settlements of about 1/2 inch. Because the potentially liquefiable deposits are isolated and highly localized we do not believe that the project warrants a full non-linear or equivalent linear site response analysis.

Loose granular deposits located above the water table can also be susceptible to some dry seismic shake down. During cyclic loading such deposits may be afforded the opportunity to realign themselves structurally into a more dense state. When this phenomenon occurs, the layer may volumetrically contract resulting in displacements at the ground surface. Based upon our review of the CPT tip resistance and SPT blow count data obtained during the current and previous explorations, we believe that the potential for dry seismic settlement on-site is negligible.

5.6 LANDSLIDING/SLOPE INSTABILITY

There are no slopes on or adjacent to the project site. Therefore, in our opinion there is no hazard to the project associated with landsliding or slope stability.

5.7 TSUNAMIS, SEICHES, AND FLOODING

The project site is at an elevation of about 15 to 40 feet and not located within a tsunami inundation area as mapped by OES-CGS-USC (2009). Therefore, in our opinion, the potential for tsunami inundation at the site is considered to be very low. Also, the site is not located adjacent to any rivers, creeks, or drainage channels and there are no lakes or water reservoirs on or adjacent to the site. As a result, the potential for flooding and seiches to impact the site is considered to be very low.

5.8 EXPANSIVE SOILS

On the basis of visual observations, past experience in the project area, and geotechnical testing performed for the project, in our opinion, the stiff to hard clayey alluvial and artificial fill soils have a moderate to high potential for expansion. We performed two expansion index (EI) tests on samples recovered from the San Joaquin site. In addition, we have performed three EI tests on samples from the proposed UCSB Sierra Madre Housing project (Fugro, 2004) and for the proposed private Ocean Meadows development project (Fugro, 2003). Both of those sites are located across Storke Road northwest of the site.

The California Building Code considers soils to be expansive if:

- The weighted Plasticity Index is greater than 5, and the weighted percent particle size less than 0.075 mm is greater than 10 percent, and the weighted percent particle size less than 0.005mm is greater than 10 percent; or

- The weighted EI in the upper 15 feet is greater than 20

A summary of the site-specific EI tests and tests from those nearby projects is provided below in Table 3.

Table 3. Summary of Expansion Index Data

Reference	Drill Hole No, Depth, Soil Type	Expansion Index (EI)	Liquid Limit Plasticity Index
This Study	DH-5, 2ft, Sandy Fat Clay	29	--
	DH-16, 2ft, Sandy Lean Clay	102	--
	DH-1, 15ft; DH-8, 6ft; DH-12, 10.5ft Sandy Lean Clay and Fat Clay	--	55,48,26, 37,37,8
Fugro (2004)	DH-2, 2ft, Sandy Lean Clay	61	36 20
	DH-4, 0-3ft, Sandy Lean Clay	112	42 25
	DH-6, 2ft, Sandy Lean Clay	65	35 19
Fugro (2003)	DH-5, 5ft, Sandy Lean Clay	111	30 12
	DH-10, 5ft, Clay	94	44 24
	DH-11, 4.5ft, Sandy Lean Clay	114	34 17

In addition to performing EI tests, we also performed consolidation swell tests to evaluate the swell potential of the clayey alluvial soils. The results of the tests performed for the project are summarized below in Table 4.

Table 4. Summary of Consolidation-Swell Test Data

Reference	Drill Hole No, Depth, Soil Type	Inundation Pressure (ksf)	Swell
This Study	DH-9, 8ft, Sandy Fat Clay	0.1	3%
	DH-12, 8ft, Sandy Fat Clay	0.1	12%
	DH-12, 8ft, Sandy Fat Clay	1	6%
	DH-15, 6ft, Sandy Fat Clay	0.1	2%

The above data support our opinion that the on-site clayey alluvial soils are expansive and have a moderate to high expansion potential. Based on the consolidation-swells test data, and assuming a 10-foot thickness of potentially expansive soil (determined assuming a 15-foot depth to groundwater and 5 feet of capillary rise), we estimate the potential total vertical heave at the site could range from less than an inch to about 9 inches.

Grading measures to help reduce the potential for expansive soils to impact on-grade slabs and foundations commonly used in the local area consist of:

- Overexcavate and Recompact the on-site soils at moisture contents above optimum, commonly used in conjunction with Pre-Saturation of the soils prior to slab construction, overexcavation depth could range from a few feet to several feet,
- Overexcavate and Replace expansive soils with low-expansion potential soil, overexcavation depth could consist of a few to several feet, replacement soils typically limited to the upper 2 to 4 feet,
- Overexcavate, Treat, and Recompact (treatment with hydrated lime is probably most common), this approach is similar to the above method and onsite soils are treated to achieve low expansion potential.

Those methods can be used in conjunction with foundation design measures such as:

- Conventional spread footings and on-grade slab, inclusion of a basement level would likely increase foundation performance.
- Conventionally-reinforced stiffened or ribbed foundations and on-grade slab.
- Post-tensioned slab and ribbed foundations.
- Drilled pier and grade beam foundations and raised floor system (void forms placed below grade beams and on-grade slab).
- Inclusion of a basement level with the above systems would likely increase foundation performance and reduce the potential foundation heave.

For the San Joaquin Residence Apartments and Precinct Improvements project, we believe that conventionally-reinforced slabs and foundations, post-tensioned slabs and ribbed foundations, and drilled pier-grade beam foundations with structural slabs are likely applicable to the lightly loaded wood-framed on-grade structures proposed for the North Village and possibly for the Portola Dining Commons. However, those systems may not be appropriate for the North and South Storke Tower structures. Design recommendations for conventionally-reinforced slabs on grade, post-tensioned slabs on grade, and drilled piers are described herein. Recommendations for conventionally-reinforced foundations provided herein assume that the design will incorporate remedial foundation grading and non-expansive fill. If soil treatment with lime is considered, additional testing will be required.

We note while the foundation systems described above may be applicable to the project, the foundation performance potential for expansive soil risk will not be uniform. Some level of risk will remain after construction unless soil replacement/treatment is performed to a significant depth (likely about 8 to 10 feet) or a drilled pier-grade beam-structural slab system is used.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY OF FINDINGS

A summary of the main findings of our geotechnical evaluation follows.

- The earth materials at the site generally consist of a relatively thin veneer of artificial fill overlying primarily stiff to hard undifferentiated fine-grained alluvial deposits and a relatively uniform thickness of granular marine terrace deposits. The marine terrace deposits are underlain by Pico Formation claystone and sandstone at depths of about 20 to 35 feet below grade. Groundwater is present at the site at depths generally ranging from about 15 to 20 feet below the ground surface, with groundwater levels measured locally at depths of about 8 to 11 feet. On-site clayey soils are moderately to highly expansive.
- The site is within a seismically active area and the project could experience strong ground shaking from earthquakes on local or regional faults. AMEC (2013a) reports a PGA of 1.09g corresponding to a 2 percent probability of exceedance in 50 years (2475-year return period or maximum considered earthquake [MCE]).
- The potential for liquefaction at the site is generally considered to be low to very low. However, there is a potential liquefaction to occur on a localized basis resulting in relatively nominal settlement potential of about 1/2 inch.
- Fugro (2012) identified three splays of the South Branch of the More Ranch fault that cross the site and provides structure setbacks from the mapped fault traces. Fugro (2012) also identified zones of potential warping or tilting that could occur at the site. AMEC (2013b) provides estimates of ground deformation, warping or tilting that occur across the site. The zones of potential ground deformation reported AMEC (2013b) are significantly more extensive that considered or interpreted in Fugro (2012). AMEC suggests that structures may need to be designed to accommodate ground deformations of 3 to 5 inches over a distance of about 100 feet in the northern portion of the site (Zone 1) and about 6-1/2 to 12 inches in the southern portion of the site. Higher deformations (up to about 13-1/2 inches in 100 feet) are estimated for an area in the east-central portion of the site and located within the fault setback zone provided in Fugro (2012).
- In our opinion, foundation systems considered applicable to the proposed structures generally consist of: 1) conventionally-reinforced shallow foundations and on-grade slabs, 2) post-tensioned ribbed or mat slabs, 3) thickened conventionally-reinforced structural mat, and 4) cast-in-drilled-hole (CIDH) pile and grade beam/pile cap with on-grade or structural floor slab. Geotechnical input for the design of those systems is provided herein.

6.2 SITE DEVELOPMENT AND GRADING

6.2.1 General Site Clearing and Grubbing

Existing fills, soil containing debris, organics, trees and root systems, and other unsuitable materials should be excavated and removed from improvement areas prior to commencing grading operations. Areas should be cleared of old foundations, slabs, pavement, abandoned utilities, and soils disturbed during the demolition process. Depressions or disturbed areas left from the removal of such material should be replaced with compacted fill.

The project specifications should provide for variations in the actual thickness and aerial extent of the existing fill materials. The limits and depths for removal of existing fills materials and infrastructure (pavement sections) is anticipated to be less than about 2 feet but should be evaluated during grading. We recommend that Fugro be contacted if extensive zones or thicknesses of existing fill material are encountered during grading.

6.2.2 Fill Placement

Fill should be placed and compacted to at least the minimum relative compaction recommended in this report. The moisture content of the fill should be 2 to 3 percent above the optimum. Each layer should be spread evenly and should be thoroughly blade-mixed during the spreading to provide relative uniformity of material within each layer. Jetting and ponding of water to assist with compaction should not be permitted for fill placement. Soft or yielding materials should be removed and be replaced with properly compacted fill material prior to placing the next layer.

Rock, gravel and other oversized material, greater than 3 inches in diameter, should be removed from the fill material being placed. Rocks should not be nested and voids should be filled with compacted material.

When the moisture content of the fill material is below that sufficient to achieve the recommended compaction, water should be added to the fill. While water is being added, the fill should be bladed and mixed to provide relatively uniform moisture conditions throughout the material. When the moisture content of the fill material is excessive, the fill material should be aerated by blading or other methods.

Fill and backfill materials should be placed in layers that can be compacted with the equipment being used. Fill should be spread in lifts no thicker than approximately 8 inches prior to being compacted. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction depending on the equipment being used.

6.2.3 Compaction

Fill placement and grading operations should be performed according to the grading recommendations of this report. Relative compaction should be assessed based on the latest approved edition of ASTM D1557. We recommend the minimum relative compaction for the locations described in Table 5.

Table 5. Recommended Relative Compaction for Site Development

Location	Recommended Minimum Relative Compaction
General	90 % U.O.N.
Utility trench bedding and pipe zone, and backfill materials	90 % U.O.N.
Backfill in landscape areas	85 % U.O.N.
Asphalt concrete, aggregate base or subbase	95 %
Building and foundation areas and areas within 5 feet horizontally of the building or structure footprint	90 %
Retaining wall, buried tank or basin, or basement backfill placed above the foundation level	90% U.O.N.

U.O.N. = unless otherwise noted

6.3 GRADING FOR FOUNDATIONS AND PAVEMENTS

The following are grading recommendations for building and foundation areas. The geotechnical engineer should review the bottom of excavations prior to placing fill materials to evaluate whether or not the artificial fill materials and other loose or unsuitable materials have been removed, and that the base of the excavation is suitable for placing compacted fill and for support of foundations. The project specifications should provide for review of the excavation by the geotechnical engineer, and for increasing the depth of the excavation to remove additional loose soil or other unsuitable materials if needed.

Grading and foundation recommendations provided herein generally follow the standard of practice for addressing expansive soils and are intended to reduce, but not eliminate, the risks of expansive soils to the proposed structures and improvements. More substantial grading and foundation design details would be required to significantly reduce the risk of expansive soils to the proposed structures, slabs, and pavements. Design recommendations for conventionally-reinforced slabs-on-grade provided herein involve limited removal and recompaction/replacement of soil material beneath the building floor slabs. Additional reduction in the impacts to conventionally reinforced slabs from expansive soil can be achieved if the expansive soils are excavated and replaced to a larger depth than provided herein. Design recommendations for post-tensioned slabs were developed without the inclusion of a layer of non-expansive soil beneath the slab.

Slabs-on-grade, sidewalks, and pavements constructed on expansive subgrade soils could be susceptible to heave and shrinkage effects, potentially resulting in cracks and uneven surfaces, particularly at slab edges. Good site drainage to preclude standing water, avoidance of poor irrigation practices, and provisions for rapid removal of surface runoff should be implemented to reduce potential impacts from expansive soils.

6.3.1 Building Areas

We anticipate the proposed structures will be founded at or near the existing site grades and that cuts and fills for site development will be less than a few feet. Grading for foundation support will require some level of remedial grading consisting of excavating or removing the near-surface soils beneath the proposed foundation system and replacing the excavated soils as compacted fill. The excavated surface should extend beneath the proposed building footprint at a relatively uniform elevation. As noted above, the excavation may need to be locally deepened as needed to remove soft, wet or compressible native soils or undocumented fill material.

In general, we recommend that remedial grading for the two- to three-story apartment-type structures planned in the northern portion of the site should consist of excavating the existing soils to a depth of at least 3-1/2 feet below the existing ground surface or to a depth of 1 foot below the proposed foundation level whichever results in the greater excavation depth. Grading for foundations for the proposed six-story residence towers and the three-story dining commons and residence building planned in the southeast corner of the site should consist of removing the existing soils to a depth of 3-1/2 feet below existing grade or 2 feet below the proposed foundations, whichever is deeper. The limits of the remedial grading and excavation should extend at least 5 feet beyond the exterior or perimeter foundations.

6.3.2 Site Retaining Walls and Other Minor Structures

Site preparation and remedial foundation grading for minor structures (such as exterior building entrances, minor retaining walls, trash enclosures, etc.) should consist of excavating and removing the existing near surface soils in the foundation area to a depth of at least 2-1/2 feet below the existing ground surface or 1 foot below the foundation level, whichever is deeper. The limits of the remedial grading and excavation should extend at least 5 feet beyond the exterior or perimeter foundations. As noted above, the excavation should be deepened to remove unsuitable soils present below the excavation subgrade.

6.3.3 Pavements and Exterior Slabs

Clearing and grubbing should be performed according to the recommendations of this report prior to beginning grading for pavement and hardscape areas. As a minimum, we recommend that the existing soil be removed to a depth of at least 1 foot below the existing ground surface or to the bottom of the proposed structural section, whichever is deeper. The excavation should extend at least 3 feet beyond the proposed limits of the paving or exterior hardscape.

6.3.4 General Subgrade Preparation

The subgrade soils exposed in the excavations should cut as neat as possible and should be observed by a representative of Fugro prior to scarifying or placing fill materials. If loose, compressible, or otherwise unsuitable soils are present at the subgrade level, the excavation should be deepened as needed to remove those soils. The presence of loose or compressible materials can be evaluated using a hand probe, by proof rolling, or other methods. Provisions for deepening the overexcavation should be included in the project plans and specifications.

Following approval of the subgrade by Fugro personnel, the overexcavated subgrade should be scarified and cross-scarified to a depth of 8 inches, moisture conditioned as required, and compacted to at least 90 percent relative compaction. Roots or organics observed during the scarifying work should be removed prior to compaction. Compacted fill can be placed to finished grade after the subgrade preparation work has been completed.

6.3.5 Suggested Material Specifications

The following materials are referenced in various sections of this report.

Aggregate base shall consist of imported material conforming to Caltrans Standard Specifications for Class 2 aggregate base, Section 26-1.02A. Class 3 material that incorporates reclaimed or recycled materials can also be used as aggregate base, provided the Class 3 material complies with the gradation and quality requirements for Class 2 material. Class 3 material shall not be placed below building areas.

Asphalt concrete shall conform to Caltrans Standard Specifications for Type A hot mix asphalt, Section 39 with PG 64-10 asphalt binder.

Capillary break beneath the vapor barrier shall consist of clean, angular, crushed gravel conforming to ASTM C33, Grade 67.

Coarse sand to be placed below Floor Slabs shall consist of imported granular material conforming to ASTM C-33, and shall have no more than 5 percent material passing the passing the No. 100 sieve.

Compacted fill material shall consist of imported or on-site material free of organics, oversize rock (greater than 3 inches), trash, debris, corrosive, and other deleterious materials. Imported fill shall be reviewed by the geotechnical engineer prior to being brought to the site; however, imported fill materials shall comply with all specifications for the material as placed at the site. Fill materials shall comply with all specified material requirements for the area where the material is being placed. Fill materials placed in building areas shall have an Expansion Index of less than 50. Fill materials placed within 3 feet of finished grade in pavement areas shall have an R-value of at least 15 as determined by California Test 301.

Low Expansion Potential Soils to be used below on-grade slabs shall consist of imported or on-site soil materials that are free of organics and have at least 40 percent passing the No. 200 sieve, a plasticity index between 5 and 15, and an expansion index of less than 20 and.

Drainage material shall conform to Caltrans Standard Specifications for Class 1 permeable material or ASTM C-33 No. 8 coarse aggregate (pea gravel) provided the materials are enclosed in a filter fabric. As an alternative, prefabricated geocomposite drainage panels can be placed behind retaining walls as recommended in this report.

Float Rock. Float rock for subgrade stabilization shall consist of 4-inch minus quarry-run rock having 100 percent of the material passing the 4-inch sieve, 0 to 30 percent passing the 2-inch sieve, 0 to 10 percent passing the 3/4-inch sieve, and less than 5 percent passing the No. 4 sieve. Float rock aggregates shall have 100 percent fractured faces. The contractor shall submit the material source, gradation and quality specifications for review by the Engineer prior to importing the material to the site.

Geotextile for separation (**filter fabric**) shall consist of geotextile that conforms to the requirements outlined in the Caltrans Standard Specifications for Filter Fabric-underdrains, Section 88-1.03.

Geotextile for subgrade stabilization shall conform to the requirements outlined in Caltrans Standard Specifications for Rock Slope Protection Fabric, Section 88-1.04.

Geocomposite drain shall consist of a manufactured plastic core not less than ¼-inch thick with both sides covered with a layer of filter fabric that will provide a continuous drainage void in the horizontal and vertical directions. Geocomposite drain placed behind retaining walls shall have an impermeable backing. Geocomposite drain to be embedded in the ground shall be double-sided with filter fabric covering both sides of the drainage void.

The drain shall produce a flow rate through the drainage void of at least 10 gallons per minute per foot of width at a hydraulic gradient of 1.0 under a maximum externally applied pressure of 2,000 psf. The core materials and filter fabric shall be capable of maintaining the drainage void for the entire height of the geocomposite drain. Filter fabric shall be integrally bonded to the core materials with the drainage void. Core material manufactured from impermeable plastic sheets having non-connecting corrugations shall not be permitted.

The fabric shall overlap a minimum of 6 inches at all joints and wrap around the exterior edges of the drain a minimum of 6 inches beyond the edge. If additional fabric is needed to provide overlaps at joints and to wrap around the edges of core material, the added fabric shall overlap the fabric on the geocomposite drain at least 6 inches and be attached thereto.

Should the fabric on the geocomposite drain be torn or punctured: 1) the damaged section shall be replaced completely if damage is done to the core material, or 2) if the core material is not damaged than the repair can be performed by placing a piece of fabric that is large enough to cover the damaged area and provide a 1-foot overlap.

Pipe zone material shall consist of imported soil having a sand equivalent (SE per ASTM 2419) of at least 30 and conforming to Section 19-3.025B, Sand Bedding, of the Caltrans Standard Specifications.

Pipe bedding material - sand shall consist of imported material having a sand equivalent of at least 30, and conforming to Section 19-3.025B, Sand Bedding, of the Caltrans Standard Specifications.

Pipe bedding material - gravel for trench bottom stabilization shall consist of material conforming to Caltrans Section 90-3.02, Coarse Aggregate Grading or ASTM C-33 No. 8 coarse aggregate (pea gravel) and be enclosed in filter fabric.

Retaining wall backfill material shall consist of imported or on-site sand and silty sand having an expansion index of 0, or imported or on-site material conforming to Caltrans Standard Specifications for Structure Backfill, Section 19-3.06.

Trench backfill shall consist of imported or onsite material that is free of organics, debris, oversized material greater than 3 inches, and other deleterious materials. Trench backfill material shall have at least 75 percent of the material passing the U.S. Standard No. 4 sieve, and/or comply with the applicable requirements for the area where the trench backfill is being placed (such as below the pavement structural section).

Vapor Barrier installation procedures, including over-laps, seams, and sealing at penetrations or service openings, shall conform to ASTM E 1643-98, modified as appropriate based on written recommendations from the vapor barrier manufacturer. Vapor barrier shall be used below slabs on grade and shall conform to Class A material per Table 1 of ASTM E 1745-97 with the following modifications:

- The permeability rating per ASTM E 96 shall be no greater than 0.01 perms; and
- The puncture resistance per ASTM 1709 shall be no less than 2,400 grams.

A sample vapor barrier specification is available from:

http://www.stegoindustries.com/specifications.

6.4 FOUNDATION SUPPORT CONSIDERATIONS

In our opinion, the proposed buildings can be supported on spread footings provided the structures can tolerate the estimated settlements from static loads. If the estimated settlements are not tolerable, alternate foundation systems, possibly consisting of a thickened mat foundation or deep foundations, will be needed. In addition, the proposed foundations and structural systems for the proposed buildings should consider the potential for movement on the nearby faults to cause local ground tilting or warping as described in AMEC (2013b) and satisfy the performance requirements described in UCSB (2013).

6.5 SHALLOW FOUNDATION DESIGN

6.5.1 Allowable Bearing Pressure

Continuous and isolated spread footings should be supported by a nominal thickness of fill that is underlain by firm in-place undifferentiated alluvial soils can be designed using a maximum allowable bearing pressure of 3,500 pounds per square foot (psf). The allowable value incorporates a factor of safety of at least 3. The toe pressure below retaining wall or eccentrically loaded footings can exceed the recommended bearing pressure, provided the resultant pressure is within the middle third of the footing. A one-third increase in the allowable bearing pressure may be used for transient loads such as seismic or wind forces.

6.5.2 Minimum Embedment Depth and Width

In general, footings should be embedded to at least 2 feet below the adjacent grade. Continuous footings provided along the perimeter of the foundation should be at least 30 inches below the adjacent grade. For conventionally reinforced foundations, continuous footings should be at least 1.5 feet wide for two-story structures and 2 feet wide for three-story structures. Isolated pad footing should be at least 3 feet in least dimension. Minimum embedment depths and widths for ribs or grade beams used with stiffened slabs or post-tension slabs are provided in Section 6.8.

6.5.3 Sliding and Passive Resistance

Ultimate sliding resistance (friction) generated at the interface of concrete foundations and compacted soils can be computed by multiplying the total dead weight structural load by a coefficient of 0.4. The ultimate net passive resistance developed from lateral bearing of on foundations bearing against compacted backfill or undisturbed native soil can be estimated using an equivalent fluid weight of 350 pcf for soils above the groundwater level. The passive resistance for the upper 1 foot of soil should be neglected unless the soils are confined at the ground surface by slab-on-grade or pavement. Sliding resistance and passive pressure may be used together without reduction, when used with the recommended minimum factors of safety. For static conditions, minimum factors of safety of 1.5 and 2.0 are recommended for foundation overturning and sliding, respectively. The factor of safety for sliding can be reduced to 1.5, if passive resistance is neglected. The factor of safety for transient (seismic, wind) conditions should be at least 1.1.

6.5.4 Settlements

Static Settlements. Static settlements will generally occur in response to foundation loads on the foundation support material. On the basis of our recommended 3,500 psf allowable bearing capacity and the subgrade preparation recommendations provided herein, total static settlement of the shallow foundations are estimated to range from about 1 to 1-3/4 inches for isolated footings ranging from 25 to 100 square feet and 1-1/2 to 2-1/4 inches for footings ranging from 100 to 225 square feet. We recommend the structure be designed to accommodate static differential settlements to of at least 1/2 inch over a distance of 30 feet (i.e., a distortion ratio of approximately 1/720) for similarly sized and loaded footings. We note that larger differential settlements (about 3/4 inch to possibly 1 inch over a distance of 30 feet) could potentially occur where small lightly loaded foundations are located adjacent to larger, heavily loaded foundations.

Seismic Settlements. Seismically induced settlements are discussed in Section 5.4 of this report. Based on our analyses, settlement from liquefaction or seismic shaking is generally not anticipated at the project site for an average assumed groundwater depth of 15 feet. However, it is possible for some limited liquefaction to locally occur at the site if the depth to groundwater is assumed at 10 feet. In that case, shallow foundations and on-grade slabs could experience localized seismic settlement of about 1/2 inch.

Ground Deformation, Warping and Tilting. As discussed previously AMEC (2013b) provides an estimate of ground deformation, warping or tilting that could occur across the site from earthquakes on local faults. AMEC suggests that ground deformations of 3 to 5 inches cold occur in the northern portion of the site (Zone 1) over a distance of about 100 feet and about 6-1/2 to 12 inches could occur in the southern portion of the site (Zones 4 and 5) over a distance of 100 feet. Higher deformations (up to about 13-1/2 inches in 100 feet) are estimated for an area in the east-central portion of the site (Zone 3) located within the fault setback zone provided in Fugro (2012).

6.6 MAT FOUNDATION DESIGN

6.6.1 Subgrade Modulus

Mat foundations and structural slabs supported by a nominal thickness of fill that is underlain by firm in-place undifferentiated alluvial soils can be designed using a Winkler model (beam on elastic foundation) using a modulus of subgrade reaction (Kv1) of 100 tons per cubic foot. For preliminary design, we recommend that mat foundations for the six-story residential tower structures be at least 2 feet thick. Recommendations for post-tensioned slabs and post-tensioned mat slabs for the low-rise structures are provided in Section 6.8.

The modulus of subgrade reaction value (Kv1) represents a presumptive value based on soil classification data and is for a 1-foot-square plate assuming the bearing pressure below the mat will not exceed 2,000 psf. Depending on how the subgrade modulus value is used in design, the value may need to be scaled for size effects. Because the underlying soils are expansive, the mat should be designed for uplift pressures potentially on the order of 2,000 to 3,000 psf.

Equation 1 can be used to estimate the modulus of subgrade reaction for mat foundations bearing on firm cohesive subgrade.

$$K_{\mathrm{B}} = \frac{\mathrm{K}_{\mathrm{v1}}}{\mathrm{B}} \tag{1}$$

where:

K_b is the subgrade modulus for a mat foundation of width "B"

K_{v1} is the subgrade modulus for a 1-foot x 1-foot square plate.

For a rectangular slab having dimensions B (width) and L (length):

$$K_{\mathrm{BL}} = \frac{\mathrm{K}_{\mathrm{B}}(1 + B/2L)}{1.5} \tag{2}$$

6.6.2 Settlement

Total static settlements of mat foundations (with a maximum average applied pressure on the soil of 2,000 psf) are estimated to range from about 1-1/2 to 2-1/2 inches.

6.7 DEEP FOUNDATION DESIGN

As discussed previously, deep foundations consisting of driven piles or cast-in-drilled hole piles can be used for structure support in lieu of spread footings of a mat foundation. Because driven piles have not been used on past projects on the UCSB campus and we anticipate that noise and vibration impacts from pile driving operations would not be acceptable to UCSB, recommendations provided herein are limited to concrete cast-in-drilled hole (CIDH) piles.

Geotechnical conditions north and south of the faults referenced in Fugro (2012) generally consists of alluvial soils overlying marine terrace deposits and Pico Formation claystone. In the area between the faults shown in Fugro (2012), in general, the subsurface conditions generally consist of alluvial channel deposits overlying marine terrace deposits and Pico Formation sandstone. We developed assumed idealized soil profiles for use in the design of CIDH piles considering the different subsurface conditions as noted above. The soil profiles and general parameters used for the design of CIDH piles are summarized in Table 6a - Idealized Subsurface Conditions - Pico Claystone Profile and Table 6b - Idealized Subsurface Conditions - Pico Sandstone Profile .

Table 6a. Idealized Subsurface Conditions – Pico Claystone Profile

Depth Interval (feet) Below Ground Surface	Generalized Soil Material	L-Pile Soil Type	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (degrees)	ε_{50} (in/in)	k (pci)
0 to 5	Stiff to Hard Lean Clay	Stiff Clay without Free Water (Reese)	130	2,000	--	0.010	--
5 to 15	Stiff to Hard Lean Clay	Stiff Clay without Free Water (Reese)	130	4,000	--	0.005	--
15 to 35	Dense to Very Dense Silty Sand	Sand (Reese)	130	--	35	--	90
35+	Pico Formation Claystone "Very Stiff to Hard Clay"	Stiff Clay without Free Water (Reese)	125	8,000	--	0.004	--

Table 6b. Idealized Subsurface Conditions – Pico Sandstone Profile

Elevation Interval (feet) Assumed	Generalized Soil Material	L-Pile Soil Type	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (degrees)	ε_{50} (in/in)	k (pci)
0 to 5	Stiff to Hard Lean Clay	Stiff Clay without Free Water (Reese)	130	2,000	--	0.015	--
5 to 15	Stiff to Hard Lean Clay	Stiff Clay without Free Water (Reese)	130	4,000	--	0.015	--
15 to 30	Dense to Very Dense Silty Sand	Sand (Reese)	130	--	35	--	90
30+	Pico Formation Sandstone "Very Dense Silty Sand"	Sand (Reese)	130	--	37	--	125

6.7.1 Axial Capacity

We estimated the axial capacity of CIDH piles for the project using the computer program SHAFT5 (Ensoft, 2001). We used the FHWA SHAFT uses the analysis procedures described in FHWA (1999). We evaluated the axial capacity of 24, 30, and 36-inch diameter drilled shaft foundations for the two idealized soil profiles discussed above. The recommended allowable axial capacities of the CIDH piles as a function of pile embedment depth are presented on Plate 5 - CIDH Pile Axial Capacity. The axial capacities provided incorporate a factor of safety of 2.5. Because the CIDH piles will be installed below the water table, we anticipate that the contractor will likely have difficulty preventing loose material from collecting at the bottoms of the drilled holes. For this reason our analysis neglects the axial capacity contributed by end bearing.

Drilled shaft foundations should be embedded at least 10 feet into relatively hard/dense Pico Formation bedrock, have a diameter of at least 2 feet, and be spaced no closer than 3 shaft diameters on-center. The top of the Pico Formation varies across the site from about El. +10 feet to -10 feet and can be estimated from the geologic cross section data provided in Fugro (2012). The actual depth of the piers should be evaluated based on the depth to rock and subsurface conditions encountered at the time the pier is drilled.

6.7.2 Uplift Resistance

Drilled shaft foundations can be designed to support vertical loads acting in compression or tension. The uplift capacity of drilled shaft foundations can be estimated as 2/3 of the maximum allowable downward frictional resistance. The frictional capacity can be increased by 1/3 when considering seismic or other transient loads.

6.7.3 Settlement

Settlement of drilled shaft foundations will likely consist of elastic compression of the pile itself plus the settlement of the soil bearing materials. We estimate that settlements of drilled shaft foundations should be less than approximately 1/2 inch total and approximately 1/4 inch differential between adjacent foundation elements. Drilled piers bearing in Pico Formation bedrock are not expected to be impacted by significant seismic related settlement.

6.7.4 Resistance to Lateral Loads

Lateral pile load carrying capacity was estimated using the computer program LPILE Plus 5.0 (Ensoft 2008) with a soil resistance-pile deflection model (p-y analysis). LPILE was used to estimate lateral load deflection and maximum moment for the piles for a range of lateral loads at the pile head. Both fixed- and free-head conditions were evaluated. Our analysis used a minimum compressive strength for concrete of 4,000 pounds per square inch. We estimated the pier's lateral load capacity and maximum moment for an estimated approximately 1/4-inch horizontal movement at the top of the pier assuming. The moment of inertia was reduced by 50 percent to model a potential cracked section.

Our estimates are based on deflections at the top of the pier (ground surface) and no factor of safety has been applied to the estimated loads. Our preliminary estimated lateral capacities and maximum moment for drilled cast-in-place piers of various diameters are provided below. Plate 6 - Lateral Capacity Results summarizes the deflection, shear and moment experienced by the various diameter CIDH piles as a function of depth with 1/4-inch deflection at the pile head. Our recommendation to embed the pile head a minimum of 10 feet into the Pico Formation bedrock was chosen to allow a 36-inch-diameter pile tip to rest at a depth approximately 1.3 times the critical length. The critical length is defined herein as the pile tip embedment depth after which the pile head receives no added displacement stability (Ensoft 2004). We have estimated the critical length as the approximate depth at which the moment and shear forces within the pile element have dissipated.

Table 7. Estimated Lateral Capacity of Drilled Piers at ¼-inch Deflection

Pier Diameter	Head Conditions	Estimated Maximum Lateral Load (kips)	Estimated Maximum Moment (kip-inch)
24-inch	Fixed-Head Free-Head	65 30	3,000 1,200
30-inch	Fixed-Head Free-Head	95 45	5,300 2,100
36-inch	Fixed-Head Free-Head	130 60	8,400 3,300

Resistance to lateral loads can also be provided by passive pressure acting on the sides of piers caps or grade beams if the existing soils within 5 feet of the pier caps and grade beams are replaced with compacted fill. Passive resistance can be provided according to our recommendations presented in this report.

6.7.5 Pile Group Effects for Lateral Loading

Group effects generally result from shadowing of piles when the direction of loading is coincident with the alignment of the piles within the group. The lateral capacity of the pile group can be estimated by multiplying the individual pile capacity obtained from the table in Section 5.73 by a P-reduction factor that accounts for shadowing effects that occur transverse and longitudinal to the loading direction. The P-reduction factors recommended below are based on research and current practices being used by Caltrans.

The lateral capacity of a pile group can be estimated by summing the lateral capacity for each individual pile after having applied the P-reduction factors. Individual piles should be designed to tolerate the maximum bending moments corresponding to the applied lateral load without considering group effects. Group effects for longitudinal loads need not be considered when the center-to-center (CTC) pile spacing between rows is greater than 8 diameters.

Table 8 summarizes P-reduction factors for groups of piles subject to longitudinal loads:

Table 8. P-Reduction Factors for Laterally Loaded Pile Groups

Pile CTC Spacing in Direction of Loading	P-Reduction Factor[1]		
	Row 1	Row 2	Row 3 and Higher
3.0 to <5 diameters	0.75	0.55	0.4
5 to <7 diameters	1.0	0.85	0.7
7 to 8 diameters	1.0	1.0	0.9

1. P-reduction factors are selected consistent with the loading direction being considered.

6.8 SLAB-ON-GRADE

6.8.1 Conventionally-Reinforced Slabs

Design Basis. The 2010 CBC provides guidance for the design of conventionally-reinforced slab-on-grade foundations. The CBC recommends the procedures described in WRI/CRSI. The soil plasticity index is the main geotechnical engineering input parameter used in the WRI/CRSI design method. Based on the data from the project site and adjacent sites, we recommend that a weighted plasticity index (PI) of 27 be used for design. We note that that assumes that 2 feet of low to very low expansion potential soils will be placed below the on-grade slab (that is below the vapor barrier layer). We recommend that interior slab reinforcement be tied to footings or stem walls.

At-grade floor slab thickness should be designed by the structural engineer, but should not be less than 5 inches. Control joints should be spaced at a maximum spacing of 12 feet in both directions. The structural engineer should determine reinforcement requirements, but, at a minimum, reinforcement of on-grade floor slabs should consist of No. 4 bars at 18 inches each way, placed above slab mid-height with preferably about 1-1/2 to 2 inches of clear cover. Means should be provided to maintain reinforcement location during construction and concrete placement.

Proper concrete placement in accordance with applicable specifications and curing of concrete slabs inhibits moisture migration. The concrete slab water cement ratio should be maintained during concrete mixing and placement. ACI 302.2R-306 (2006, pg. 37) indicates that water cement ratios in the range of 0.4 to 0.5 with a compressive strength not less than 4,000 psi may provide a reasonable drying time; however, the architect and design engineer should select the desired concrete properties based on the concrete slab-on-grade performance requirements.

Allowable Bearing Pressure for Foundation Ribs and Grade Beams. Ribs and grade beams for conventionally reinforced stiffened slabs (and post tensioned slabs, if used) should bear on undisturbed soil and can be designed assuming an allowable bearing pressure of 3,500 pounds per square foot (psf). Footings should be sized using the recommended bearing pressures for dead plus probable maximum live loads. Recommended allowable bearing pressures incorporate a factor of safety of at least 3.0. Footing widths should not be less than 24 inches for two-story structures and not less than 30 inches for three-story structures.

A one-third increase in the allowable bearing pressures may be allowed for transient loads, such as wind or seismic loads.

Beam Depth and Presaturation. Recommended beam depths and pre-saturation requirements are presented in the following table. The pre-saturation moisture content should be obtained and maintained at least 2 days prior to casting the concrete. The pre-saturated subgrade soil should be observed by the project geotechnical consultant prior to placing vapor barrier material and casting concrete. Soils silted into the footing excavations during the pre-saturation operations should be removed prior to casting the concrete.

Table 9. Recommended Exterior Beam Depths and Slab Pre-saturation Requirements

EI Range	Exterior Beam Depth[1] (inches)	Interior Beam Depth[1] (inches)	Pre-saturation Depth (inches)	Pre-saturation Moisture Content (percent)
90-130	24 (2-stories) 30 (3-stories)	18 (2-stories) 24 (3-stories)	24[2]	120% of optimum moisture content

Notes: 1) Below lowest adjacent grade.

2) Pre-saturation should be performed for all slab and footing areas and should be initiated well before concrete placement. Testing should be performed to verify that water has penetrated into the soil.

As noted in Table 9, pre-saturation of subgrade soils in the moderate to high (EI range of 90-130) expansion category should be performed for all slab and footing areas and should be initiated well before concrete placement. Testing should be performed to verify that water has penetrated into the soil to the minimum depth below lowest adjacent grade noted in Table 9 (where applicable).

Settlement of Foundations Ribs and Grade Beams. Foundation ribs and grade beams for post-tensioned foundations should accommodate the static settlements estimated in Section 6.5.4.

6.8.2 Post-Tensioned Slabs

The 2010 CBC recommends that the design procedure for post-tensioned slabs-on-grade be based on recommendations developed by the Post-tensioning Institute (PTI) and provided in PTI (2012). Recommendations presented herein are intended for post-tensioned concrete slabs-on-grade for residential structures ranging up to three stories. Design parameters accommodate soils in the medium to high expansion categories in anticipation of possible variation in the expansion characteristics of pad subgrade materials. Design parameters needed for the PTI design procedure include the edge moisture distance variation, e_m, in feet, and the differential swell, y_m, in inches. The basis for selection of design values are based on input parameters described below. The parameters were estimated assuming standard foundation and remedial grading described in Section 6.3.

- Assume the following soil parameters: weighted EI of 100, weighted Liquid Limit of 45, weighted Plastic Limit of 30, 100 percent passing the No. 10 sieve, 65 percent passing No. 200 sieve, soil unit weight γ_{moist} pf 130 pcf (total unit weight at soil wet limit) and $\gamma_{in\text{-}situ}$ of 115 pcf (in-situ dry unit weight), and Soil Fabric Factor (F_f) of 1.0

- Estimated values of γ_{hswell} and $\gamma_{hshrink}$ range from about 0.04 to 0.07 computed using PTI (2012) methods 1, 2, 3, and 4. Assume γ_{hswell} and $\gamma_{hshrink}$ equal to 0.06,

- Modified Unsaturated Diffusion Coefficient (α') for shrink and swell conditions of 0.004,

- A Thornthwaite Moisture Index (Im) of -20 inches/year was estimated from Figure A3 in PTI (2012),

- Based on Figure 5.10 of PTI (2012), Edge Moisture Variation Distances, e_m, of 4 and 8 feet were estimated for edge and center lift conditions, respectively (α' controls e_m values). Those values do not consider the use of vertical or horizontal moisture barriers.

- Based on Figure 5.11 in the 2012 PTI Manual, an equilibrium soil suction of -3.9pF was estimated for a TMI of -20 inches/year.

- Estimate y_m as $y_m = \gamma_{hswell}$ (SCF) and $y_m = \gamma_{hshrink}$ (SCF); SCF (swell/wet) and SCF (shrink/dry) from PTI (2012) Table 5.2a.

Table 10. Summary of Design Parameters for PTI Slabs

EI	γ_{hswell} (edge) (Sec.5.1.2.1 through Sec.5.1.2.4)	$\gamma_{hshrink}$ (center) (Fig. 5.9)	Soil Suction at Depth Z_m	Soil Suction at Surface (pF) (Table 3.2)		Stress Change Factor (SCF) (Table 5.2a)		Differential Swell, y_m (inches)	
				Wet	Dry	Wet	Dry	Edge Lift	Center Lift
100	0.06	0.06	-3.9	3	4.5	20.7	-9.4	1.24	0.56

Notes: 1) Equations, figures, and tables, refer to PTI Manual (2012)

2) ym = SCF * γ_h, rounded to the nearest 0.01 inch

3) Edge moisture distance values of 4 feet and 8 feet should be used for edge lift and center lift conditions, respectively.

We recommend that post-tensioned ribbed slabs be at least 5 inches thick. Recommendations for beam depths listed in Table 9 are considered applicable to the design of ribbed post-tensioned slabs.

If thickened post-tensioned mat slab is used, we recommend the slab be at least 12 inches thick. Exterior beams should be provided for this case and the information provided in Table 9 is applicable, however, the minimum embedment depth of the exterior beams used with post-tensioned mat slabs can be reduced by 10 inches from the values listed in Table 9.

The final slab thickness, rib/beam depth and spacing, and reinforcing requirements should be determined by the structural engineer.

6.8.3 Vapor Barrier

Recommendations for slab-on-grade or slab-on-ground construction are presented below, and are based on ACI 302.2R-06, "Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials," published in 2006. The architect and design engineer should review that reference for background on moisture vapor penetration through concrete slabs and issues regarding protection from delamination of flooring, blistering, staining, mold growth, and other problems related to performance of moisture-sensitive flooring. Since 1999, water-based flooring adhesives have replaced solvent-based adhesives because of restrictions by the EPA, which has lead to an increase in moisture-related problems.

The performance of flooring is complicated as described in ACI 302.2R-06 and depends on many factors including sub-slab relative humidity, concrete materials and water-cement ratio, internal relative humidity, and construction aspects, such as curing, length of drying, environmental conditions, pH, etc. As noted above, the architect and design engineer should review pertinent background materials and decide what measures are needed depending on the type of flooring to be used. Recommendations presented below are not intended to resolve every issue regarding moisture vapor penetration through on-grade concrete slabs. If additional concerns need to be addressed, then additional information needs to be provided and reviewed by the geotechnical engineer and probably by an expert in vapor moisture transmission through concrete slabs. Where tile is used, we recommend that measures be implemented to reduce possible cracking of the tile and that a vinyl crack isolation membrane be used beneath the tile and the concrete slab.

A vapor barrier should be provided below slabs, especially those with floor coverings, to reduce the potential for vapor moisture migration from the subgrade up through the slab. Preferably, the vapor barrier should extend beneath footings and grade beams; however, because of design and construction difficulties, placement of the vapor barrier beneath footings and grade beams is left to the discretion of the design engineer. The vapor barrier should conform to a Class A per Table 1 of ASTM E 1745-97 with the following modifications:

- The perm rating per ASTM E 96 should be no greater than 0.01 perms.
- The puncture resistance per ASTM 1709 should be no less than 2400 grams.

The recommended vapor barrier characteristics and the associated puncture resistance and tensile strength should allow placement of the vapor barrier material directly on the capillary break, described below. Vapor barrier installation procedures, including over-laps, seams, and sealing at penetrations or service openings, should conform to ASTM E 1643-98, modified as appropriate based on written recommendations from the vapor barrier manufacturer.

A sample specification is available from http://www.stegoindustries.com/specifications, for the Option 3 Vapor Barrier (non-proprietary) case. Stego Industry products or equivalent can be used as vapor barriers.

6.8.4 Granular Fill above Vapor Barrier

ACI 302.2R-06 (2006) presents advantages and disadvantages for placement of a granular fill cushion/protection layer above the vapor barrier. Issues include cushion disturbance during reinforcement placement and construction activities, concrete slab performance during curing (e.g., curling and shrinkage), and the cushion layer providing a source of moisture, as well as other factors described in ACI 302.2R-06. The architect and design engineer (structural engineer) should decide whether a granular fill cushion beneath the vapor barrier is advantageous based on their experience with on-grade concrete slab performance and information in ACI 302.2R-06 (2006, see Figure 7.1 for guidance).

If used, granular fill placed above the vapor barrier should be between 1 and 2 inches thick and consist of fine concrete aggregate conforming to ASTM C33 or, more preferably, manufactured sand or crusher-run sand materials conforming to No. 10 size material per ASTM D448 with no more than 5 percent passing the No. 200 sieve. The material should have enough moisture to be compatible and easy to trim, but still dry enough at the time of concrete placement to act as a blotter. The material should be proof-rolled such that construction equipment can pass over the material without undue disturbance.

6.8.5 Capillary Break below Vapor Barrier

The capillary break, beneath the vapor barrier, should consist of 4 inches of clean, angular, crushed gravel conforming to ASTM C33 Grade 67 placed on the select fill subgrade.

The gravel should be lightly vibrated with three to four passes of a base-plate compactor or smooth-wheel vibratory roller.

6.9 MOLD

Mold in buildings is a growing concern for owners and occupants of buildings. The growth and development of mold in buildings can be nurtured by moisture either from sources inside or outside the structure. We note that the geotechnical engineering services for this report did not consider issues related to mold and recommendations provided herein do not eliminate the risk of future mold problems. To aid in reducing the risk of mold problems, we recommend the owner retain an experienced mold professional to review building and landscape plans and specifications, evaluate grading and earthwork, assess sources of fill and landscape materials, and other aspects of the project design and construction.

6.10 CORROSIVE SUBGRADE

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. As a general rule, Caltrans (2012) indicates that a minimum resistivity value of 1,000 ohm-cm is an indicator of high soluble salt content and a general indicator of corrosion potential. Caltrans considers soils to be corrosive or to represent a corrosive environment if one of the following criteria is met:

- Chloride content of 500 ppm or greater,
- Sulfate concentration of 2,000 ppm or greater, or
- pH is 5.5 or less.

We evaluated the potential for the on-site soils to be corrosive to ferrous metals and aggressive to concrete. For this effort, we tested one near-surface soil sample to assess the resistivity, pH, chloride content, and soluble sulfate content. The results are presented below and in Appendix B. We also reviewed and considered test data acquired for the UCSB Sierra Madre Housing (Fugro 2004) project and the private Ocean Meadows development (Fugro 2003) located northwest of the San Joaquin site. The test data for the sample tested from the project site and those reported in Fugro (2003, 2004) are summarized below:

Table 11. Corrosion Test Results

Drill Hole	Sample Depth (feet)	Material Description	Resistivity (ohms/cm)	PH (units)	Chloride (ppm)	Water Soluble Sulfate in Soil (wt % / ppm)
DH-5 (This Study)	2 to 4	Sandy Lean Clay (Qal)	1826	7.8	35	0.027 / 273
DH-1 (Fugro 2004)	0 to 5	Sandy Lean Clay (Qal)	2,200	5.6	170	0.018 / 180
DH-10 (Fugro 2003)	1 to 1.5	Silty Clay to Sandy Lean Clay (Qal)	1,997	6.8	243	0.011 / 110

The results suggest that the soils present in the project area are generally not corrosive or aggressive to concrete. Based on those limited data for preliminary design purposes, the soils at the site can likely be assumed as non-corrosive to ferrous metals and non-aggressive to concrete. Normal Type II cement can likely be used for concrete that will be in contact with on-site soils. We suggest additional testing should be performed during rough grading to confirm or modify that recommendation, if necessary.

6.11 LATERAL EARTH PRESSURES FOR EARTH RETAINING STRUCTURES

6.11.1 Site Retaining Walls and Basement Walls

Retaining walls should be designed to resist lateral earth pressures. Backfill material for the retaining wall should consist of well-drained granular soils conforming to the suggested material specifications of this report. Retaining wall foundations should be designed according to the recommendations for shallow foundations provided herein. We recommend the following equivalent fluid weights for use in estimating the lateral earth pressures that will act on the retaining wall for level backslope conditions behind the wall.

Table 12. Lateral Earth Pressures - Retaining Walls

Wall Loading Condition	Lateral Earth Pressure Condition	Equivalent Fluid Weight (pcf)
Unbraced/Cantilevered - Drained with level backslope	Active	40
Braced /Basement- Drained with level backslope	At-rest	65

6.11.2 Surcharges

The recommended equivalent fluid weights for retaining walls and buried structures do not account for surcharge loads acting on the backfill. The surcharge from foundation loads can be neglected, provided adjacent footings are setback behind a 1:1 line projected upward from the base of the wall. The lateral earth pressure from uniform surcharge loads can be estimated as 0.3 times the stress being applied at the ground surface. Traffic surcharges can be estimated as an additional 2 feet of soil cover, equal to a uniform pressure of 72 pounds per square foot. Fugro should provide additional recommendations if foundation loads act within the 1:1 line or other surcharges to retaining walls are anticipated.

6.11.3 Dynamic Considerations

Based on the 2010 California Building Code retaining walls need to be designed to resist dynamic earth pressures. Generally, retaining walls that are relatively free to deform or rotate in response to seismic loads can be designed using the Mononobe Okabe approach with a value of kh typically assumed as some percentage of the design horizontal ground acceleration. Using the Mononobe Okabe method and assuming a horizontal seismic coefficient kh of 0.36g (about 50% of the peak horizontal ground acceleration of 0.73g), the additional force on the wall from earthquake loading is estimated to be about $20H^2$ (pounds per foot of wall) [or 40H pcf equivalent fluid pressure] where H is the wall height in feet. The distribution of seismic pressure can be assumed to be an inverted triangle (zero seismic pressure at the base of the wall) and is considered as an additional pressure above the resulting static earth pressure.

6.11.4 Drainage

For drained backfill conditions, drainage should be provided behind retaining walls to reduce the potential for the buildup of hydrostatic pressures. Retaining Wall Backfill Material should be placed between the wall and a 1h:1v backslope projected up from the heel of the retaining wall footing or edge of buried structures.

Walls designed for drained loading conditions should be designed with collector pipes to assist in the removal of water from the backfill, and to prevent the buildup of hydrostatic pressures behind the wall. A continuous layer of granular drainage material consisting of either 1-foot of free draining soils or geocomposite drain panels should be provided along the backside of walls. The drainage material should be terminated 2 feet below the finished grade of the wall backfill, and be topped with on-site fine-grained soil or topsoil.

6.12 EXTERIOR SLABS

From a geotechnical standpoint, we suggest the exterior slabs-on-grade, walkways, and patio areas have a minimum thickness of 4 inches and be reinforced. Exterior slabs should also include control joints and expansion joints for crack control. Joints should be sealed and maintained. Grading for exterior slabs should conform to the requirements described in Section 6 and be underlain by at least 4 inches of aggregate base. Positive drainage should be provided to prevent ponding of water adjacent to exterior slabs and hardscape.

6.13 PAVEMENT DESIGN

Structural sections were estimated for asphalt concrete and Portland cement concrete pavements based on an R-value of 13 for the on-site soils and a range of traffic indices (TI) from 6 to 8.

6.13.1 Asphalt Concrete Pavements

Structural sections for asphalt concrete pavements were estimated based on methods presented in the Caltrans Highway Design Manual. Structural section recommendations for flexible 2-layer pavements, asphalt concrete (AC) over aggregate base (AB), are provided in the table below. Final pavement design sections should be evaluated on the basis of additional R-value tests performed during rough grading in the pavement areas. If, in general, the results of the additional testing suggest a higher design R-value is justified for final design, revised (less conservative) pavement sections could be used.

Table 13. Asphalt Concrete Pavement Recommendations

Traffic Index	Structural Section Thickness (inches)
6	3" AC over 12" AB
7	3.5" AC over 14" AB
8	4.5" AC over 15" AB

Compacted fill should be placed to the proposed subgrade level as described herein. Pavement materials should conform to Sections 26 and 39 of the Caltrans Standard Specifications (or equivalent) for aggregate base (AB) and asphalt concrete (AC), respectively. Base materials placed in the pavement areas should be compacted to at least 95 percent relative compaction.

Maintenance of asphalt concrete pavements should consist of periodic fog or slurry seals to reduce the potential for weathering.

6.13.2 Portland Cement Concrete Pavements

Structural sections for Portland cement concrete pavements were estimated using the Portland Cement Association (PCA) design method for 3,000-psi strength Portland cement concrete for various estimates of the average daily truck traffic (ADTT). Portland cement concrete pavements should be designed with control joints, expansion joints, and load transfer provisions in according to PCA or other applicable guidelines.

Table 14. Portland Cement Concrete Pavement Recommendations

ADTT	Structural Section Recommendation (inches)
$\leq$ 6	6" PCC over 4" AB
$\leq$ 60	6.5" PCC over 4" AB
$\leq$ 400	7" PCC over 4" AB

6.14 SURFACE DRAINAGE CONSIDERATIONS

Drainage should be provided such that surface water does not run over slopes or pond on pavements, slabs, or adjacent to foundations. Downspouts should be provided to collect roof drainage and direct the water to drainage pipes or area away from the building. The top of slopes should be graded to direct drainage away from the slopes, or be provided with dikes and ditches that will direct surface water to controlled drainage structures. Concentrated flows and runoff should not be permitted to discharge onto slopes. Down drains, solid pipes, or lined ditches should be provided to carry water to the base of the slope. Energy dissipation and erosion control devices should be provided at the outlet of drainage pipes and in areas of concentrated flow and runoff to reduce the potential for erosion.

6.15 CONSTRUCTION CONSIDERATIONS

6.15.1 Existing Utilities

We reviewed the utility atlas sheets with respect to existing utilities. Existing utilities in the anticipated building envelopes generally consist of storm drain, water and irrigation lines, gas, sewer, and street lighting. On the basis of our past experience on the UCSB campus, we anticipate the invert depths to those utilities (other than the storm drains and sewer) to be less than about 5 feet. We recommend that those utilities be removed from the building footprint and relocated as part of the proposed construction. The removal should consist of the excavation of the existing trench backfill and subsequent placement of new compacted fill. Excavation work required for the abandonment of those utilities is anticipated to be relatively nominal but should be considered in the construction documents. Invert depths for the sewer and storm drain lines should be reviewed or determined by potholing and abandonment plans should be developed accordingly.

Foundation construction and site preparation may also involve the construction of temporary slopes and shoring systems. The design of temporary slopes and shoring should also consider support of adjacent utilities and pipelines. Particular attention should be paid to pressurized lines that may rely on the lateral support of the ground to constrain the pipeline against movement.

6.15.2 Excavation Conditions and Groundwater

Subsurface materials encountered in our drill holes and CPT soundings consist primarily of a variable thickness of stiff to very stiff sandy lean and fat clay with minor interbedded medium dense to dense silty sand layers. Those soils overlie medium dense to dense granular terrace deposits and Pico Formation bedrock. We expect that excavations in those soils and rock materials can be made using conventional heavy-duty equipment that is in good working order.

Groundwater was encountered in our explorations at depths ranging from about 15 to 20 feet below the ground surface. However, locally groundwater was measured in drill holes DH-3 DH-6, and DH-7 excavated for this study at depths of 8 to 11 feet. If groundwater seepage is encountered in the proposed excavations, we recommend that an appropriate dewatering system be installed to lower the groundwater to below the lowest excavation level. The contractor should be responsible for the design, installation, and operation of the dewatering system. Depending on the conditions, the dewatering system could range from trenches and sumps to dewatering wells.

If the groundwater level has been lowered as recommended and pumping conditions are encountered, we recommend that the subgrade be stabilized prior to placing fill or construction of foundation elements. A discussion of potential subgrade stabilization measures is provided in this report. Measures to stabilize the subgrade may be required and provisions for stabilization work should be provided in the contract documents.

6.15.3 Temporary Excavations

The contractor should be responsible for the design of temporary slopes. Within the anticipated depths of excavation, the soil is anticipated to consist primarily of stiff to very stiff sandy lean and fat clay and local interbedded medium dense to dense silty sand layers. Soils below the groundwater level should not be considered capable of maintaining a stable vertical slope. Temporary slopes should be braced or sloped according to the requirements of OSHA.

As input to design, excavations without shoring that are shallower than 20 feet will likely be classified as Type B and should be sloped to 1:1 or flatter per OSHA guidelines. OSHA requires excavations greater than 20 feet deep be designed by a qualified professional for the subsurface conditions encountered. We recommend that temporary excavations be monitored for signs of instability and appropriate actions (such as flattening the slope, providing shoring, and controlling groundwater) are undertaken if evidence of potential instability is observed.

Excavation, dewatering, and shoring plans should be submitted by the contractor and reviewed by the geotechnical professional in advance of the excavation. Slopes should not be considered stable if seepage can daylight on the slope or groundwater is expected within the planned depths of excavation.

6.15.4 Cast-in-Place Drilled Shafts

Drilled piers will be excavated through the alluvial soils and terrace deposits into the underlying Pico Formation bedrock. Because the CIDH piles will be constructed below the groundwater level and will encounter uncemented silty sand materials, we expect that excavations for drilled shaft foundations will require the use of drilling slurry and/or casing to prevent caving of the drilled hole. Drilling and concrete placement for CIDH piles should conform to the requirements of Section 49 of Caltrans Standard Specifications and related Standard Special Provisions.

Prior to placing rebar and concrete, the sides of the excavated piers should be reamed to remove smeared material, and loose or disturbed materials should be removed from the bottom of the piers. Groundwater should be removed from the drilled shafts prior to placing concrete, or tremie pumping methods should be used to place concrete from the bottom of the drilled shafts and to displace groundwater or slurry during concrete placement. The piers should be overfilled with concrete until fresh, non-contaminated concrete surfaces at the top of the drilled shaft.

Concrete used for drilled pier construction should have a high level of workability with a slump in the range of 6 to 9 inches. Concrete aggregates should be sized small enough to be suitable for placement by pumping and with consideration for the spacing between reinforcing bars to ensure that concrete can move through the rebar cage and adhere to the sidewalls of the shaft without honeycombs or voids. Concrete should be placed the day the drilling is completed. A pier excavation should not be allowed to stand open overnight. In general, a minimum of 24 hours should be allowed between placing concrete in one pier shaft and the drilling of nearby pier shafts within four pier diameters, center to center.

The use of casing and wet placement methods can make observation of drilled shaft construction relatively difficult. Improper retrieval of the casing, and caving below the surface of the wet concrete, can result in necking or contamination of the completed shaft. We therefore recommend that integrity testing of the completed shafts be provided as part of the construction such as using cross-hole analyzers, nuclear density measurements, and/or low strain measurements.

7.0 CONTINUATION OF SERVICES

The geotechnical evaluation consists of an ongoing process involving the planning, design, and construction phases of the project. To provide this continued service, we recommend that the geotechnical engineer be provided the opportunity to review the project plans and specifications, and observe portions of the construction.

7.1 REVIEW OF PLANS AND SPECIFICATIONS

The geotechnical engineer should review the foundation and grading plans for the project. The purpose of the review is to evaluate if the plans and specifications were prepared in general accordance with the recommendations of this report.

7.2 GEOTECHNICAL OBSERVATION AND TESTING

Field exploration and site reconnaissance provides only a limited view of the geotechnical conditions of the site. Substantially more information will be revealed during the excavation and grading phases of the construction. Subsurface conditions, excavations and fill placement should be observed by the geotechnical professional during construction to evaluate if the materials encountered during construction are consistent with those assumed for this report.

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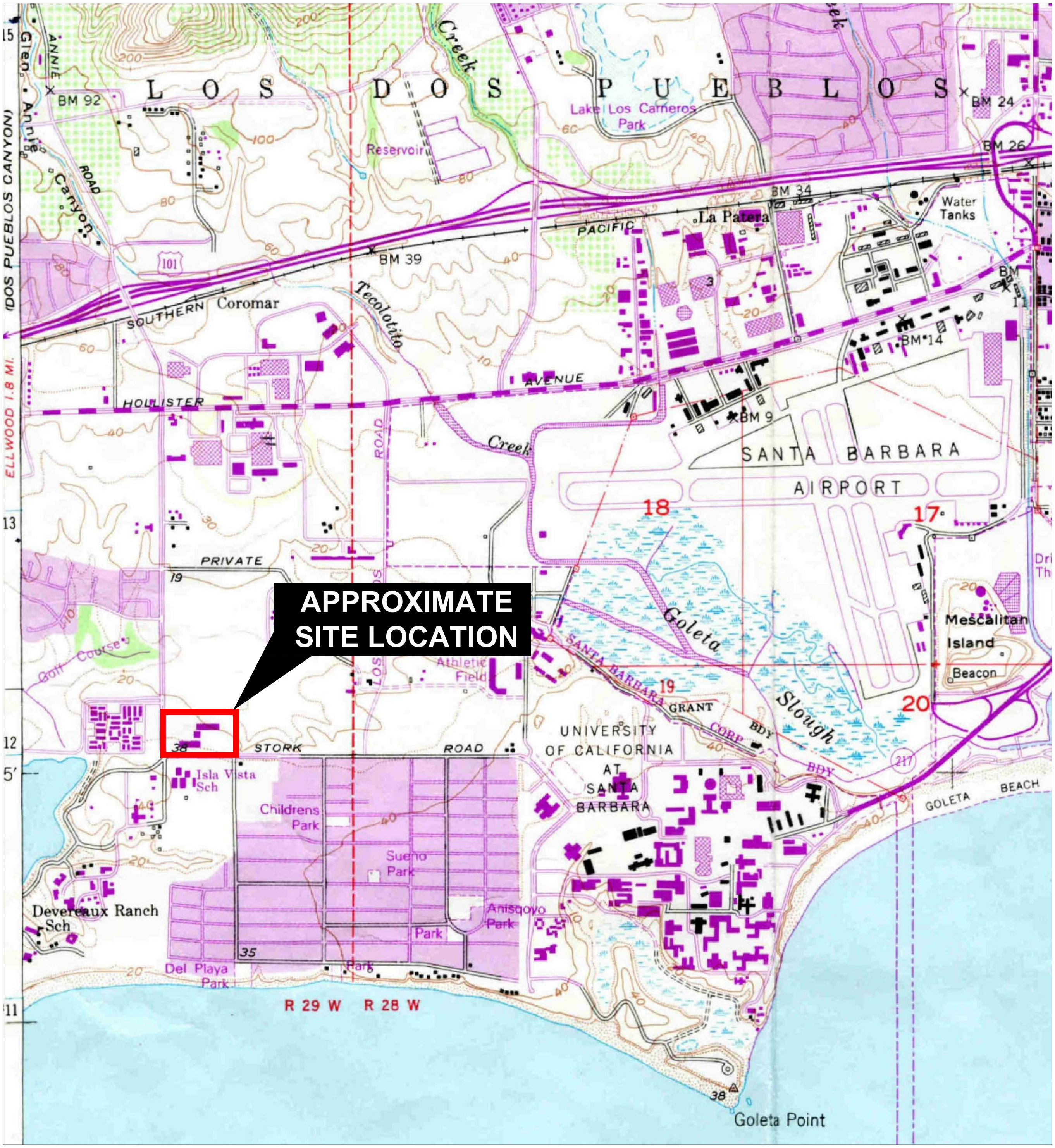
PLATES

University of California Santa Barbara
Project No. 04.62130070



BASE MAP SOURCE: USGS Topographic Map, 7.5 Minute Series, Goleta Quadrangle (1950, revised 1982).

NORTH

0 2000

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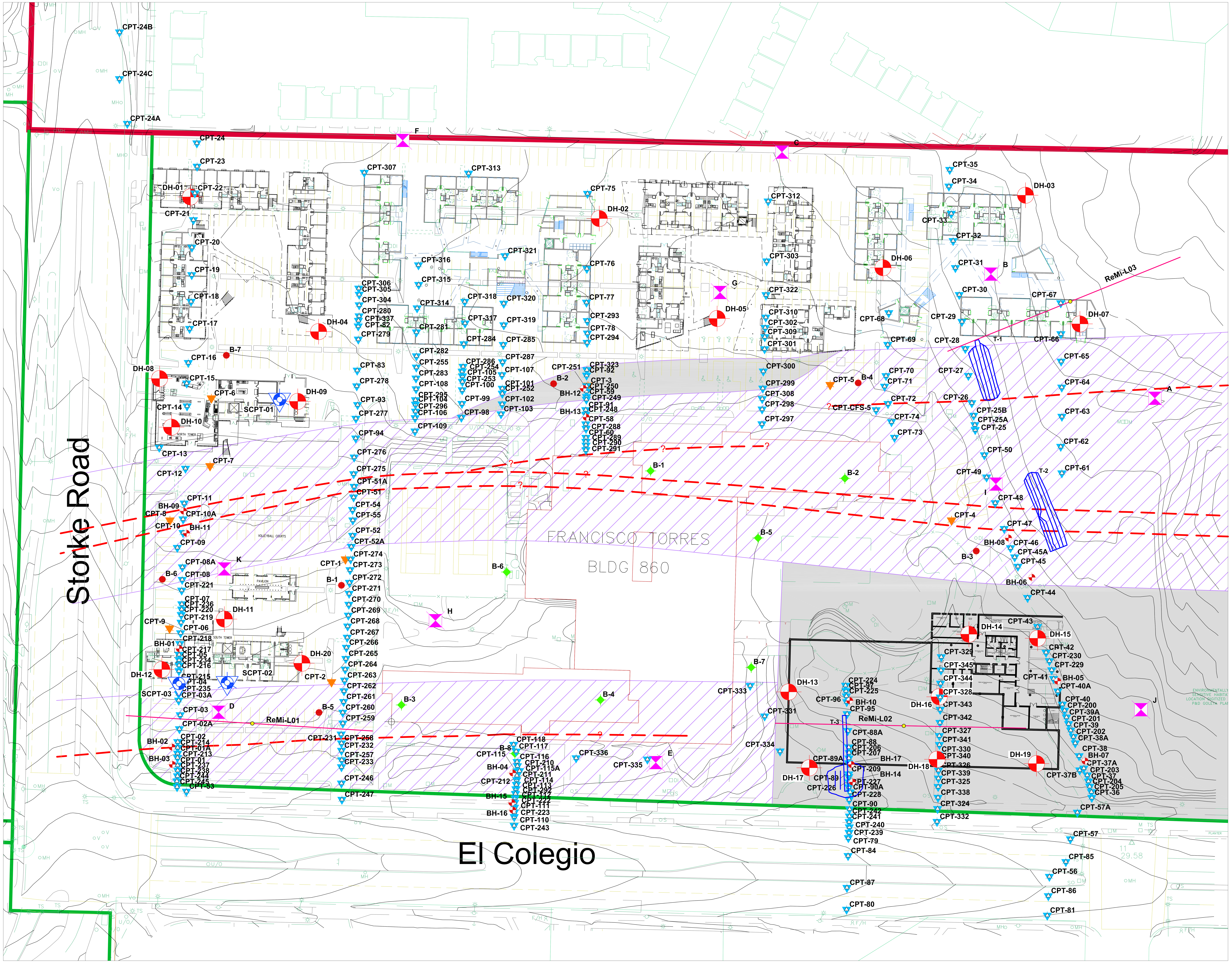
VICINITY MAP

San Joaquin Apartments and Precinct Improvements
University of California Santa Barbara
Santa Barbara, California

PLATE 1



LEGEND

DH-20 Approximate boring location (this study)

SCPT-03 Approximate Seismic CPT (this study)

K Approximate percolation test location

BH-17 Approximate boring location (Fugro, 2012)

CPT-39 Approximate CPT location (Fugro, 2012)

B-7 Approximate boring location (Fugro, 2002)

CPT-9 Approximate CPT location (Fugro, 2002)

B-8 Approximate boring location (LeCroy Crandall, 2002)

ReMi-L03 Refraction Micrometer (ReMi) Survey Line (AMEC, 2012)
Center of survey line

Estimated fault location (queried where uncertain)

Recommended fault setback

Setback Zone

Zone of Pleistocene faulting or warping, where foundation mitigation measures should be used

T-1 Fault Trench

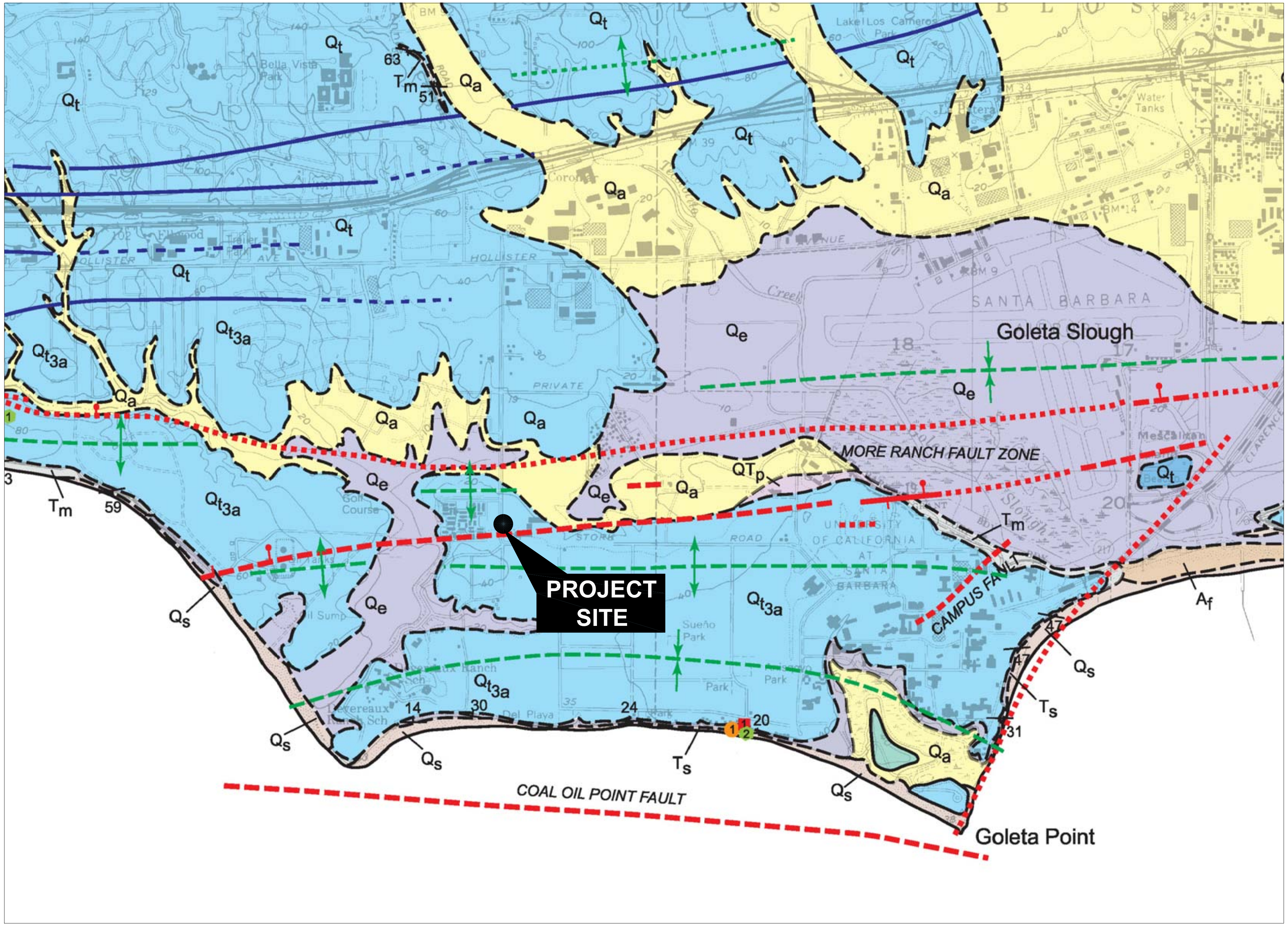
BASE MAP SOURCE: * Interpreted Fault Conditions Map from Fugro 2011 report,
* Site Plan from Fugro 2002 report.

FUGRO CONSULTANTS, INC
4820 McGrath St., Suite 100, Ventura, California 93003
Tel.: (805) 650-7000, FAX: (805) 650-7010

SUBSURFACE EXPLORATION PLAN
San Juaquin Residence Apartments
Santa Barbara, California

University of California Santa Barbara

04.62130070 July 2013 Plate 2



LEGEND

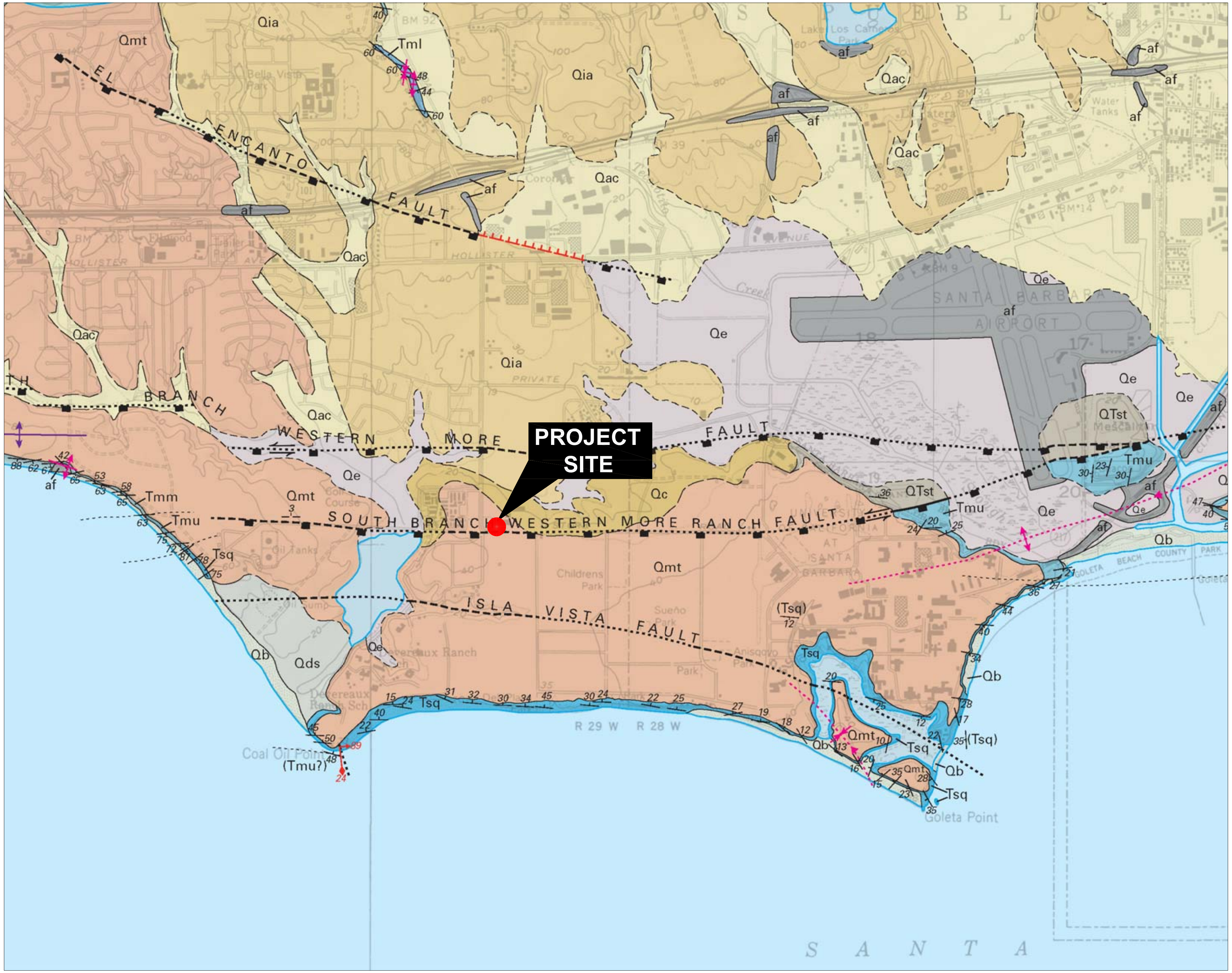
- Af — Artificial fill
- Qs — Beach Sand - Unconsolidated marine and aeolian sand
- Qa — Alluvium - Undifferentiated alluvial, stream channel, and flood plain deposits composed of silty sands to sandy gravels
- Qe — Estuarine Deposits - Commonly estuarine silts and clays with sand interbeds and lenses
- Qt3a — Marine Terrace Deposits - Second marine terrace associated with oxygen isotope substage 5a with Qt5a1 indicating youngest terrace
- Qt — Marine Terrace Deposits (unknown age) - medial to nearshore marine sands with aeolian silts
- QTp — Pico Formation - Marine conglomerates, sandstones, and siltstone with minor fossils
- Ts — Sespe Formation - Undifferentiated gravel conglomerate, sandstone, and shale deposits
- Geologic contact, location approximate, concealed or inferred
- 57 — Fault, location approximate where dashed, location concealed (blind) or inferred where dotted; ball and bar on downthrown side, tic indicates dip of fault; single arrow indicates minor component of strike-slip, double arrows indicate strike-slip fault
- Anticline, location approximate where dashed, inferred where dotted; plunge direction indicated
- Syncline, location approximate where dashed, inferred where dotted; plunge direction indicated
- Marine terrace shoreline or strandline, location approximate
- 35 — Strike and dip of inclined beds
- Vertical beds
- 85 — Strike and dip of overturned beds
- 2 — Optical stimulated luminescence sample site number 5
- 2 — Uranium-series sample site number 2
- 3 — Radiocarbon sample site number 3

NORTH

0 2000 4000
FEET

BASE MAP SOURCE: Geologic Map of the Western Santa Barbara Fold Belt, Santa Barbara, California (Gurrola, 2004).

LOCAL GEOLOGIC MAP - GURROLA (2004)
San Joaquin Apartments and Precinct Improvements
University of California Santa Barbara
Santa Barbara, California



LEGEND

af	Artificial Fill	Qmt	Marine terrace deposits
Qb	Beach deposits	QTst	Siltstone unit
Qds	Dune sand	Tsq	Sisquoc Formation
Qe	Estuarine deposits	Tmu	Upper siliceous unit
Qac	Alluvium and Colluvium	Tmm	Middle shale unit
Qc	Colluvium	Tml	Lower calcareous unit
Qia	Intermediate alluvial deposits		

Contact - Long-dashed where approximately located; short-dashed where inferred; dotted where concealed; tic shows direction and angle of dip

Fault - Long-dashed where approximately located; short dashed where inferred; dotted where concealed; queried where uncertain; bi-directional arrows indicate superposed dextral and sinistral slip on same fault

Fault-line scrap - Inferred from aerial photographs; hachures point downscrap

Reverse Fault - Rectangles on apparent upthrown side

Anticline - Large arrow indicates plunge direction

Syncline - Large arrow indicates plunge direction

Strike and dip of beds:

40 Inclined

35 Inclined (approximate)

NORTH

0 2000 4000
FEET

BASE MAP SOURCE: Geologic Map ot the Santa Barbara Coastal Plain Area, Santa Barbara County, California, (Minor, Kellogg, Stanley, Gurrola, Keller and Brandt, 2009).

LOCAL GEOLOGIC MAP - MINOR et al. (2009)
San Joaquin Apartments and Precinct Improvements
University of California Santa Barbara
Santa Barbara, California

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APPENDIX A – FIELD EXPLORATION

LOCATION: The drill hole location referencing local landmarks or coordinates

SURFACE EL: Using local, MSL, MLLW or other datum

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLES	BLOW COUNT / REC"/DRIVE"	MATERIAL DESCRIPTION	
-12	2		1		25	Well graded GRAVEL (GW)	COARSE GRAINED
-14	4		2		(25)	Poorly graded GRAVEL (GP)	COARSE GRAINED
-16	6					Well graded SAND (SW)	COARSE GRAINED
-18	8		3		(25)	Poorly graded SAND (SP)	COARSE GRAINED
-20	10		4		(25)	Silty SAND (SM)	COARSE GRAINED
-22	12		5		18"/ 30"	Clayey SAND (SC)	COARSE GRAINED
-24	14		6			Silty, Clayey SAND (SC-SM)	COARSE GRAINED
-26	16					Elastic SILT (MH)	FINE GRAINED
-28	18		7			SILT (ML)	FINE GRAINED
-30	20		8		20"/ 24"	Silty CLAY (CL-ML)	FINE GRAINED
-32	22		9		(25)	Fat CLAY (CH)	FINE GRAINED
-34	24		10		30"/ 30"	Lean CLAY (CL)	FINE GRAINED
-36	26					CONGLOMERATE	ROCK
-38	28		11		20"/ 24"	SANDSTONE	ROCK
-40	30		12			SILTSTONE	ROCK
-42	32		13			MUDSTONE	ROCK
-44	34					CLAYSTONE	ROCK
-46	36					BASALT	ROCK
-48	38					ANDESITE BRECCIA	ROCK
						Paving and/or Base Materials	

General Notes

Soil Texture Symbol

Sloped line in symbol column indicates transitional boundary

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

Symbol for:

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

Sampler Driving Resistance

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

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
86/11"	After driving sampler the initial 6" of seating, 36 blows drove sampler through the second 6" interval, and 50 blows drove the sampler 5" into the third interval
50/6"	50 blows drove sampler 6" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

Classification of Soils per ASTM D2487 or D2488

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

Strength Legend

Q = Unconfined Compression
u = Unconsolidated Undrained Triaxial
t = Torvane
p = Pocket Penetrometer
m = Miniature Vane

Water Level Symbols

▽ Initial or perched water level
▼ Final ground water level
Seepages encountered

Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.

KEY TO TERMS & SYMBOLS USED ON LOGS

PLATE A-1

LOCATION: See Plate 2

SURFACE EL: 25 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
	0			**ASPHALT CONCRETE (3")**							
				ARTIFICIAL FILL (af) Silty SAND (SM): very dark grayish brown, moist							
				Poorly graded GRAVEL with sand (GP): very dark grayish brown, moist, coarse, subangular to rounded							
				UNDIFFERENTIATED ALLUVIUM (Qal) Fat CLAY with sand (CH): very dark gray, moist							
	5	1	9	- stiff - brown							p 3.8 p 2.0
	10	2a 2b	(35)	Sandy fat CLAY (CH): hard, reddish brown, moist, fine	125	104	20	71			
	15	3	16	Fat CLAY with sand (CH): stiff to very stiff, mottled reddish and grayish brown, moist, orange staining					55	37	p 3.5
	19			**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND with silt (SP-SM): dense to very dense, yellowish brown, wet, fine							
	20	4	(51)		123	103	19				

Elevation scale: 24, 22, 20, 18, 16, 14, 12, 10, 8, 6, 4, 2, 0. Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26.

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 21.5 ft
DEPTH TO WATER: 16.0 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-1

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-2

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 22 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (3.5")							
						ARTIFICIAL FILL (af) Poorly graded SAND with clay (SP-SC): yellowish brown, moist, fine to medium							
20	2		A			**UNDIFFERENTIATED ALLUVIUM (Qal)** Lean CLAY with sand (CL): dark grayish brown, moist							
18	4												
					(36)								
16	6		1			Sandy fat CLAY (CH): very stiff, mottled grayish brown with reddish brown, moist, fine, orange staining	136	119	14				p 4.0
14	8												
12	10		2		20								p 4.5+
						- lenses of clayey sand below 11 feet							
10	12												
8	14												
					(49)								
6	16		3				137	120	14	48			
4	18					**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND with clay (SP-SC): medium dense, yellowish brown, wet, fine							
2	20		4		20								
0	22												
-2	24												
-4	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 21.5 ft
DEPTH TO WATER: 11.8 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-2

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-3

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 16.5 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
16			A			**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): light brown, moist, black flecks, lenses of clayey SAND (SC) throughout layer							
14	2												
12	4				(28)								
10	6		1			- very stiff, mottled grayish brown and reddish brown, moist, fine to medium sand	131	112	17				
8	8		2		(27)					47			p 3.3
6	10		3		18								p 2.3
4	12												
2	14												
0	16		4		(28)		132	113	18				
-2	18												
-4	20		5		17								p 3.5
-6	22					**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): light brownish gray, wet, mostly fine, sample disturbed due to flowing sands							
-8	24												
-10	26		6		(Push)								

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 26.5 ft
DEPTH TO WATER: 7.8 ft
BACKFILLED WITH: Cuttings
DRILLING DATE: June 17, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-3

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

LOCATION: See Plate 2

SURFACE EL: 25 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
				ASPHALT CONCRETE (4")							
24				**ARTIFICIAL FILL (af)** Silty SAND (SM): yellowish brown, dry to moist, fine to medium							
	2			Lean CLAY with sand (CL): brown, moist							
22				**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): very dark gray, moist to wet							
	4										
20		1	6	- medium stiff, reddish brown							
	6										
18											
	8										
16											
	10		(32)								p 4.5+
14		2		- very stiff, mottled reddish brown and grayish brown, orange staining	134	117	14				
	12										
12		3	(16)								
	14										
10		3	16								p 3.0
	16										
8				**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND with silt (SP-SM): dense to very dense, yellowish brown, moist to wet, fine, orange staining							
	18										
6											
	20		(88/11")								
4		4									
	22										
2											
	24										
0											
	26										

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 21.5 ft
DEPTH TO WATER: 16.9 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-4

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-5

LOCATION: See Plate 2

SURFACE EL: 21.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (2.5")							
20						**ARTIFICIAL FILL (af)** Silty SAND with gravel (SM): very dark gray, moist, fine gravel							
	2		A			Sandy lean CLAY (CL): very dark gray, moist, fine sand, rootlets							
18						**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): mottled reddish brown with grayish brown, moist							
	4												
16					(37)	- very stiff, pockets of very dark gray at 5 feet							p 3.5
	6		1			- mottled reddish brown, grayish brown, and pale brown with orange staining and brown veins below 5.5 feet	134	116	15				
14					(33)								p 4.5+
	8		2a				133	114	17	59			
			2b										
12													
	10		3		18								p 4.5+
10													
	12												
8													
	14												
6					(32)	Fat CLAY with sand (CH): very stiff to hard, mottled reddish brown with grayish brown, moist, orange staining							p 2.8
	16		4				124	100	24				
4													
	18												
2													
	20				(24)								
0			5				124	97	28				
	22												
-2													
	24					**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, yellowish brown, wet, fine, grades to gray with shell fragments and increased clay content (SP-SC)							
-4					(39)								
	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 41.5 ft
DEPTH TO WATER: 15.5 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-5

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-6a

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 21.5 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
-6	28												
-8	30												
-10	32												
-12	34												
-14	36		6a 6b		(50/5.5")	**PICO FORMATION (Qtp)** SEDIMENTARY ROCK (CLAYSTONE): fresh, very soft to soft, dark greenish gray, moist, massive							
-16	38												
-18	40		7a 7b		(84/11")		124	100	24		54	32	u 9.0
-20	42												
-22	44												
-24	46												
-26	48												
-28	50												
-30	52												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 41.5 ft
DEPTH TO WATER: 15.5 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-5

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-6b

LOCATION: See Plate 2

SURFACE EL: 20 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
				ASPHALT CONCRETE (2")							
				ARTIFICIAL FILL (af) Silty SAND with gravel (SM): very dark gray, moist, fine gravel							
18	2			**POSSIBLE ARTIFICIAL FILL (af)** Sandy SILT (ML)/Silty SAND (SM): very dark gray, moist, fine sand - reddish brown							
16	4										
			(26)	**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): stiff, very dark gray, moist, fine sand							p 2.5
14	6	1		- mottled grayish brown and reddish brown with orange staining	131	111	18				
12	8										
10	10	2	13								p 3.3
8	12										
6	14										
			(31)	- very stiff below 15 feet, lense of clayey SAND (SC)							p 1.5
4	16	3			130	108	20				
2	18										
0	20	4	17								p 3.8
-2	22			**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, light yellowish brown, wet, fine							
-4	24										
			(46)								
-6	26	5									

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 26.5 ft
DEPTH TO WATER: 9.9 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-6

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-7

LOCATION: See Plate 2

SURFACE EL: 16 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
		A		**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): dark grayish brown, moist, fine sand							
14	2										
12	4										
		1	(28)	very stiff, mottled reddish brown and dark grayish brown, rootlets	131	112	17				p 2.0
10	6										
		2a	(30)		132	111	19				
8	8	2b		- increased sand content							
6	10										
		3	(34)	- with black flecks	135	116	17				p 3.8
4	12										
2	14										
		4	15								
0	16										
-2	18										
-4	20										
		5	(40)		132	112	18	72			
-6	22										
-8	24										
		6	32	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND with silt (SP-SM): dense, light brownish gray, wet, fine to medium							
-10	26										

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 26.5 ft
DEPTH TO WATER: 10.8 ft
BACKFILLED WITH: Cuttings
DRILLING DATE: June 17, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-7

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

LOCATION: See Plate 2

SURFACE EL: 26.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
26	0			**ASPHALT CONCRETE (5")**							
				ARTIFICIAL FILL (af) Clayey SAND (SC): yellowish brown, moist, pockets of pale brown, fine sand							
24	2			**POSSIBLE ARTIFICIAL FILL (af)** Sandy lean CLAY (CL): very dark brownish gray, moist, fine sand							
22	4		(18)								
20	6	1		**UNDIFFERENTIATED ALLUVIUM (Qal)** Lean CLAY with sand (CL): stiff, very dark gray to black, moist, fine sand	129	109	18		48	37	p 3.3
18	8										
16	10	2	16								p 4.5
14	12			Fat CLAY with sand (CH): stiff, mottled reddish brown, grayish brown, and brown, moist, orange staining							
12	14		(39)	- very stiff, black laminants							p 2.8
10	16	3			130	109	20				
8	18			**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, light yellowish brown, wet, fine							
6	20	4	31								
4	22										
2	24										
0	26										

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 21.5 ft
DEPTH TO WATER: 16.8 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 19, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-8

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-9

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 26.5 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
26						**ASPHALT CONCRETE (8")**							
						ARTIFICIAL FILL (af) Clayey SAND (SC): yellowish brown, moist, pockets of pale brown							
24	2		A			**UNDIFFERENTIATED ALLUVIUM (Qal)** Fat CLAY with sand (CH): dark brownish gray, moist							
22	4												
					(6)								p 1.5
20	6		1			- soft, 1" lense of clayey SAND (SC): dark yellowish brown, moist, fine sand at approximately 5.5 feet	131	109	20				
					(39)	- very stiff, grades to mottled reddish brown and grayish brown with orange staining at approximately 8 feet							
18	8		2a				133	115	16				
			2b				135	117	16				u 5.6
16	10		3		23								p 4.5+
14	12												
12	14				(33)								p 4.0
10	16		4			- laminated, with gray lenses	135	117	16				
8	18												
6	20		5		28								
						MARINE TERRACE DEPOSITS (Qt) Poorly graded SAND with silt (SP-SM): medium dense, mottled pale and reddish brown, moist to wet, fine sand							
4	22												
2	24		6		(n/a)	- sand heave before sampling at 25 feet							
0	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 41.5 ft
DEPTH TO WATER: 19.5 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 19, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-9

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-10a

LOCATION: See Plate 2

SURFACE EL: 26.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
-2	28												
-4	30		7		40	- dense, pale brown and brown with orange flecks, sand heave before sampling at 30 feet							
-6	32												
-8	34					**PICO FORMATION (Qtp)** SEDIMENTARY ROCK (CLAYSTONE): fresh, very soft to soft, dark greenish gray, moist, massive, shell fragments							
-10	36		8a		(46)		124	100	24				u 7.9
			8b				124	100	24				
-12	38												
-14	40		9a		(86/11")								
			9b										
-16	42												
-18	44												
-20	46												
-22	48												
-24	50												
-26	52												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 41.5 ft
DEPTH TO WATER: 19.5 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 19, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-9

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-10b

LOCATION: See Plate 2

SURFACE EL: 27 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
26				**POSSIBLE ARTIFICIAL FILL (af)** Sandy lean CLAY (CL): brown, moist, rootlets - grades to pockets of brown and reddish brown with orange flecks, moist							
	2	A		SILT with sand (ML): very dark gray, moist, fine sand							
24											
	4			moist to wet at approximately 4 feet							
22			(20)	**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): stiff, mottled pale brown, reddish brown and grayish brown, moist, orange staining							p 4.5+ p 1.3
	6	1			125	102	22				
20				- dark gray, decreased sand content in seam at 5.5 feet							
			(29)	- mottled pale brown, reddish brown and grayish brown							p 4.3
	8										
		2			135	117	16	59			
18											
	10	3	15	- stiff to very stiff							
16											
	12										
14											
	14										
12			(39)	- very stiff							
	16	4			135	117	16				
10											
	18										
8											
	20	5	40	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, pale brown, moist to wet, fine sand							
6											
	22										
4											
	24										
2											
	26										

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 21.5 ft
DEPTH TO WATER: 17.2 ft
BACKFILLED WITH: Cuttings
DRILLING DATE: June 19, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-10

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-11

LOCATION: See Plate 2

SURFACE EL: 31.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
				ASPHALT CONCRETE (4")							
				ARTIFICIAL FILL (af) Silty SAND with gravel (SM): pale brown, dry to moist							
30				Silty SAND (SM): brown, moist							
	2		(27)	**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): very stiff, mottled reddish brown and grayish brown, moist, fine sand, orange staining							
28	4	1			134	120	12				
26			(66)	- hard, decrease in sand content							p 4.5+
	6	2			133	120	10				
24	8		(79/11")	Fat CLAY with sand (CH): hard, mottled reddish brown and grayish brown, moist, fine sand, orange staining							p 4.5+
		3			131	112	17				u 12.3
22	10	4	(60)								
20	12			Sandy fat CLAY (CH): hard, mottled reddish brown and grayish brown, moist, black flecks, interbedded with fat CLAY with sand				79			
18	14										
16	16	5	51	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): very dense, pale brown, moist, fine							
14	18										
12	20										
10	22										
8	24										
6	26										

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 16.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 19, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-11

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 33 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (3")							
32						**ARTIFICIAL FILL (af)** Silty SAND with gravel (SM): light brown, dry to moist							
	2		A			**POSSIBLE ARTIFICIAL FILL (af)** Sandy lean CLAY (CL): brown, moist, fine sand							
30													
	4												
28					(61)	**UNDIFFERENTIATED ALLUVIUM (Qal)**							p 4.5+
	6		1a 1b			Sandy fat CLAY (CH): hard, mottled brown and reddish brown, moist, fine sand	136	121	12				
26													
	8		2		(55)	- reduced sand content at approximately 7.5 feet	129	104	23				
24													
	10		3		(38)	Sandy lean CLAY (CL): very stiff, mottled gray and reddish brown, moist, fine sand	118	108	9		26	8	
22													
	12												
20													
	14												
18			4		16	- stiff to very stiff							p 4.5+
	16												
16													
	18												
14													
	20		5a 5b		(75/9.5")	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND with silt (SP-SM): very dense, mottled gray and yellowish brown with gray, moist to wet, fine to medium	127	111	15				
12													
	22												
10													
	24												
8			6		42	- dense, gray, wet, with abundant shells, lenses of black possibly organic material, reduced fines							
	26					**PICO FORMATION (Tp)** SEDIMENTARY ROCK (SANDSTONE): fresh, soft, yellowish brown, wet, silty, fine to medium grained, sand heave in augers before sampling							
6													
	28												
4													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 51.5 ft
DEPTH TO WATER: 24.2 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 17, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-12

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-13a

LOCATION: See Plate 2

SURFACE EL: 33 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
			7		(50/5.5")								
2													
	32												
0													
	34												
-2			8		24	- fine sand, heave before sampling							
	36												
-4													
	38												
-6													
	40					- unable to sample due to flowing sands							
-8													
	42												
-10													
	44												
-12			9		(50/4")								
	46												
-14													
	48												
-16													
	50				50/5"								
-18			10										
	52												
-20													
	54												
-22													
	56												
-24													
	58												
-26													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 51.5 ft
DEPTH TO WATER: 24.2 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 17, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-12

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-13b

FUGRO

LOCATION: See Plate 2

SURFACE EL: 32 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (3")							
						BASE (4")							
30	2					**POSSIBLE ARTIFICIAL FILL (af)** Lean CLAY with sand (CL): brown, moist							
28	4												
					(48)								
26	6		1			**UNDIFFERENTIATED ALLUVIUM (Qal)** Fat CLAY with sand (CH): very stiff, mottled brown, grayish brown, and yellowish brown, orange staining	135	114	19				
24	8												
22	10				21								
			2			Poorly graded SAND (SP): medium dense, pale brown and dark yellowish brown laminants, moist, fine - 2 to 4 foot thick seam of sandy fat CLAY (CH) at approximately 12.5 feet							
20	12				(41)								
			3				130	108	20				
18	14												
			4		35	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, pale brown, moist, fine							
16	16												
14	18												
12	20												
10	22												
8	24												
6	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 16.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cuttings and Quick Set Concrete Patch
DRILLING DATE: June 18, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-13

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-14

LOCATION: See Plate 2

SURFACE EL: 24.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
24.5	0			**ASPHALT CONCRETE (4")**							
				ARTIFICIAL FILL (af) Silty SAND (SM): brown, moist							
				UNDIFFERENTIATED ALLUVIUM (Qal) Lean CLAY with sand (CL): reddish brown, moist							
	5.5	1	(28)	Interbedded Sandy fat CLAY (CH) and Clayey SAND (SC): very stiff/medium dense, mottled reddish and grayish brown, moist, fine to medium sand, orange staining	131	112	17				
	10	2	15	- stiff/medium dense				34			
	15	3	(54)	- hard/dense							
	20	4	15								p 2.5
	25	5	(39)	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, light yellowish brown, wet, fine							

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 26.5 ft
DEPTH TO WATER: 13.8 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 20, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-14

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 23.5 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
	0			**ASPHALT CONCRETE (5")**							
22	2			**POSSIBLE ARTIFICIAL FILL (af)** Sandy SILT (ML): very dark gray, moist							
20	4										
18	6	1	(21)	**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): stiff, mottled reddish brown and grayish brown, moist, fine sand, orange staining	129	107	21				
16	8										
14	10	2	11	Clayey SAND (SC): medium dense, mottled reddish brown and grayish brown, moist, fine sand, orange staining				33			
12	12		(57)	- increase in grain size at approximately 12.5 feet, decreased fines content							
10	14	3			129	114	13				
8	16	4	19	Sandy fat CLAY (CH): very stiff, mottled reddish brown and grayish brown, moist, fine sand							
6	18										
4	20	5	54	**MARINE TERRACE DEPOSITS (Qt)** Clayey SAND (SC): dense, mottled reddish brown and grayish brown, wet, fine to medium, black flecks of possible organic material, blowcounts unreliable due to sampler overpacking							
2	22										
0	24										
-2	26	6	(76)								

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 26.5 ft
DEPTH TO WATER: 14.3 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 18, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-15

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

LOCATION: See Plate 2

SURFACE EL: 27.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (4") **POSSIBLE ARTIFICIAL FILL (af)** Sandy lean CLAY (CL): brown, moist, fine sand - very dark gray at approximately 1 foot							
26	2		A										
24	4												
22	6		1		(24)	**UNDIFFERENTIATED ALLUVIUM (Qal)** Sandy fat CLAY (CH): very stiff, mottled grayish brown and reddish brown, moist, fine sand	133	116	15	55			p 3.8
20	8		2a 2b		(29)								p 3.8
18	10		3		(42)		134	116	15				
16	12												
14	14												
12	16		4		32	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND with clay (SP-SC): dense, pale brown grading to very light gray, moist, fine sand, wet at 15.5 feet							
10	18												
8	20		5		(76/9")	- mottled dark yellowish brown and light brown at 20 feet							
6	22												
4	24												
2	26		6		31	- medium dense, orange staining in horizontal layers at approximately 26 feet							
0	28												
-2													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 41.5 ft
DEPTH TO WATER: 20.2 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 18, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-16

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-17a

LOCATION: See Plate 2

SURFACE EL: 27.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
			(74)								p 4.5+
		7a 7b		**PICO FORMATION (Qtp)** SEDIMENTARY ROCK (CLAYSTONE): fresh, very soft to soft, dark greenish gray, moist, massive							
-4	32										
-6	34										
-8	36	8	48				20				
-10	38										
-12	40	9a 9b	(50/6")		131	111	18				u 8.5
-14	42										
-16	44										
-18	46										
-20	48										
-22	50										
-24	52										
-26	54										
-28	56										
-30	58										
-32											

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 41.5 ft
DEPTH TO WATER: 20.2 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 18, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-16

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-17b

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	LOCATION: See Plate 2 SURFACE EL: 33 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
32	0–5		A			**UNDIFFERENTIATED ALLUVIUM (Qal)** Lean CLAY with sand (CL): dark grayish brown and reddish brown, moist							
28	5–11.5		1		(37)	Sandy fat CLAY (CH): very stiff, grayish brown mottled with reddish brown, moist, rootlets, pockets of sand							
			2		(33)		131	113	16	69			p 3.8
			3		(36)	- increased sand content at approximately 10 feet							p 4.5+
22	11.5–16.5		4		35	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): dense, pale brown and dark yellowish brown laminants, moist, fine							

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 16.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cuttings
DRILLING DATE: June 18, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-17

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-18

LOCATION: See Plate 2

SURFACE EL: 30 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (4.5")							
						UNDIFFERENTIATED ALLUVIUM (Qal) Sandy fat CLAY (CH): very dark gray, dry to moist							
28	2		A										
26	4												
					(39)	- very stiff, mottled brownish gray and reddish brown, moist							p 4.5+
24	6		1				134	116	16				
					(31)								
22	8		2a 2b										
20	10				(43)								p 4.5+
			3										
						MARINE TERRACE DEPOSITS (Qt) Poorly graded SAND (SP): medium dense, pale brown, moist, fine sand							
18	12												
16	14												
			4		26								
14	16												
12	18												
10	20		5		(78/11")	Poorly graded SAND with clay (SP-SC): dense, dark yellowish brown, moist to wet, fine sand							
8	22												
6	24												
4	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 21.5 ft
DEPTH TO WATER: 20.0 ft
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 18, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-18

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

FUGRO

LOCATION: See Plate 2

SURFACE EL: 28 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLE NO.	SAMPLERS	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ASPHALT CONCRETE (5")							
						POSSIBLE ARTIFICIAL FILL (af) Clayey SAND (SC): reddish brown and very dark gray intermixed, moist to wet, pockets of clay, very fine sand							
26	2												
24	4												
			1		(16)	**UNDIFFERENTIATED ALLUVIUM (Qal)** Fat CLAY with sand (CH): stiff, mottled reddish brown and pale brown, moist, orange staining	127	104	22				
22	6												
20	8												
18	10		2		20	- very stiff at 10 feet							p 4.5+
16	12				(40)	- increased sand content below 12.5 feet							p 4.5+
			3				132	113	17				
14	14					- possible organic black flecks and veins, grades to increased sand below 14 feet							
			4		30	**MARINE TERRACE DEPOSITS (Qt)** Poorly graded SAND (SP): medium dense, dark yellowish brown, moist, fine sand							
12	16												
10	18												
8	20												
6	22												
4	24												
2	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 16.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 19, 2013

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-19

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-20

University of California at Santa Barbara
Project No. 04.62130070

LOCATION: See Plate 2

SURFACE EL: 34 ft +/- (rel. NAVD88 datum)

SAMPLE NO.	SAMPLER BLOW COUNT	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
		ASPHALT CONCRETE (4")							
		ARTIFICIAL FILL (af) Silty SAND with gravel (SM): pale brown, dry to moist							
		POSSIBLE ARTIFICIAL FILL (af) Sandy SILT (ML): brown, moist							
A		Lean CLAY with sand (CL): very dark grayish brown with brown pockets, moist - mottled reddish brown and grayish brown with orange staining at 1.9 feet							
1	(20)	**UNDIFFERENTIATED ALLUVIUM (Qal)** SILT with sand (ML): stiff, mottled pale brown and grayish brown, dry to moist, fine sand	108	101	6		17	2	
2	(46)	Sandy fat CLAY (CH): hard, mottled grayish brown and reddish brown, moist, fine sand, orange staining	131	116	13				
3	30								
4	(74/5")	**MARINE TERRACE DEPOSTIS (Qt)** Poorly graded SAND with clay (SP-SC): very dense, mottled brown and light grayish brown, moist, fine sand, orange staining							

ELEVATION, ft	DEPTH, ft
32	2
30	4
28	6
26	8
24	10
22	12
20	14
18	16
16	18
14	20
12	22
10	24
8	26

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 16.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cuttings and Asphalt Concrete Patch
DRILLING DATE: June 19, 2013

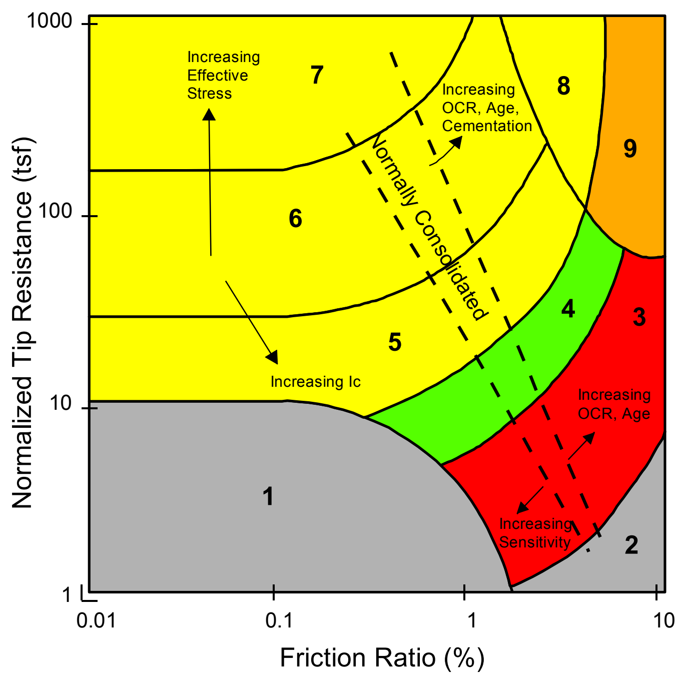
DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: 140 lb Automatic Trip
DRILLED BY: Martini Drilling Company
LOGGED BY: J Martos
CHECKED BY: G S Denlinger

LOG OF DRILL HOLE NO. DH-20

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE A-21

CPT CORRELATION CHART
(Robertson and Wride, 1990)

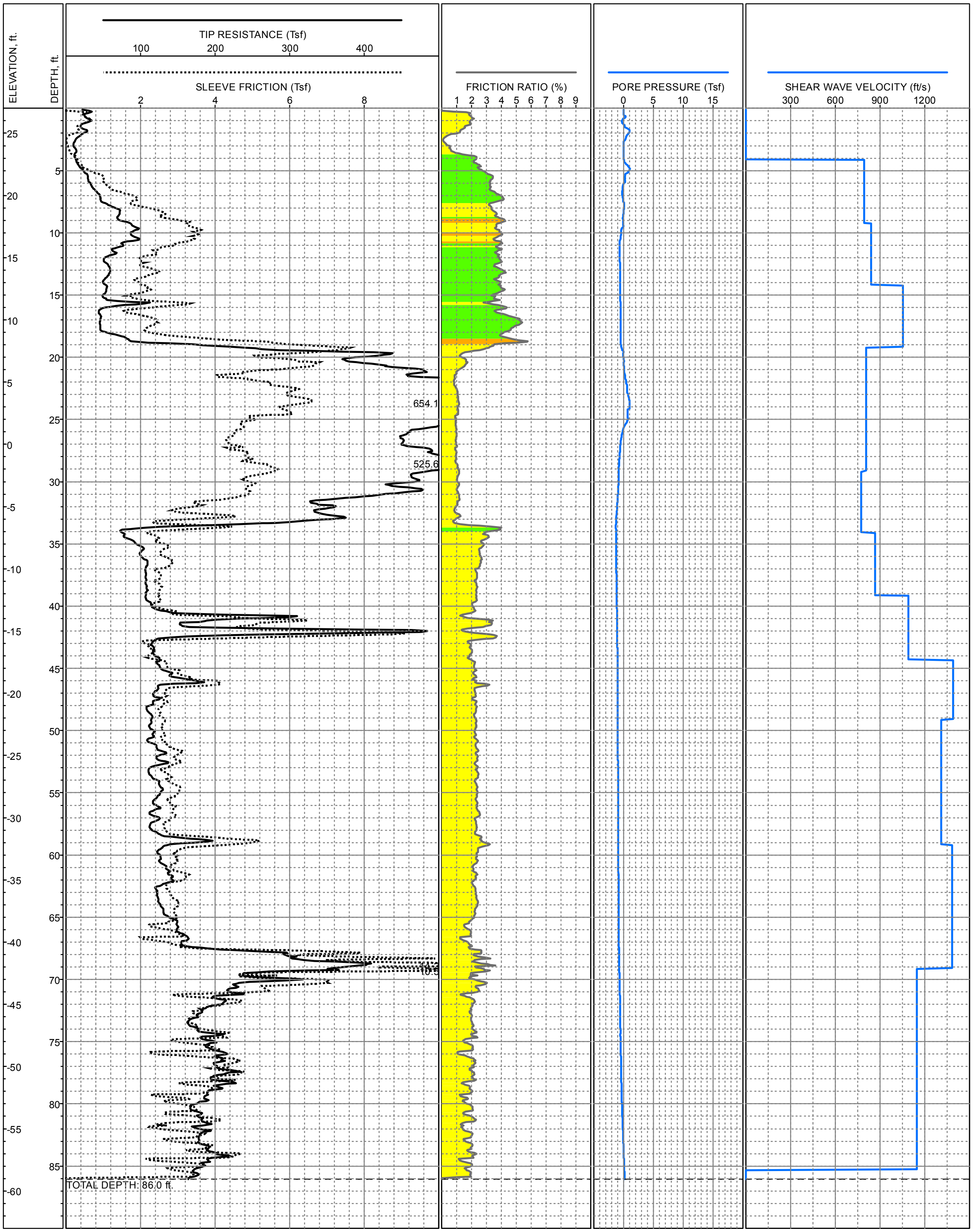


Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Peats
3	Silty Clay to Clay
4	Clayey Silt to Silty Clay
5	Silty Sand to Sandy Silt
6	Clean Sand to Silty Sand
7	Gravelly Sand to Dense Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine-Grained*

*heavily overconsolidated or cemented

KEY TO CPT INTERPRETATION

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



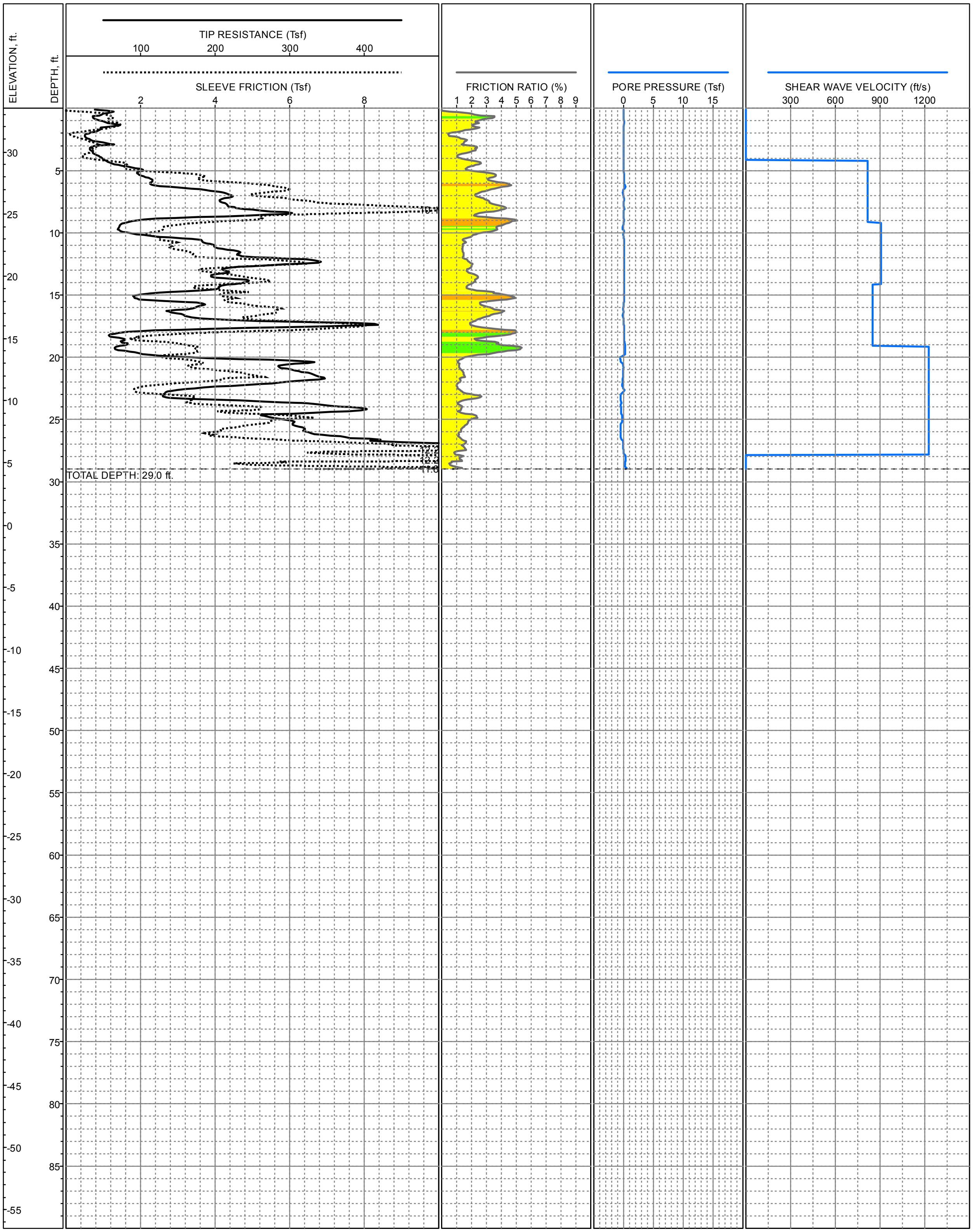
LOCATION: E5,997,998, N 1,979,946, CA State Plane, Zone 5, NAD83, Feet
SURFACE EL: 27.0ft +/- (NAVD88)
COMPLETION DEPTH: 86.0ft
TESTDATE: 6/21/2013

EXPLORATION METHOD: CPT
PERFORMED BY: Kehoe Testing and Engineering
REVIEWED BY: G. Denlinger
CONE AREA RATIO: 0.85

LOG OF CPT NO: SCPT-1

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

N:\Projects\04_2013\04_6213_0070_UCSB_SJApts\Explorations\CPT\2013\Logs\2013_07_02_Logs_Vs\MXD\CPT_Logs_VQ13_Vs.mxd,07/29/2013,CDean

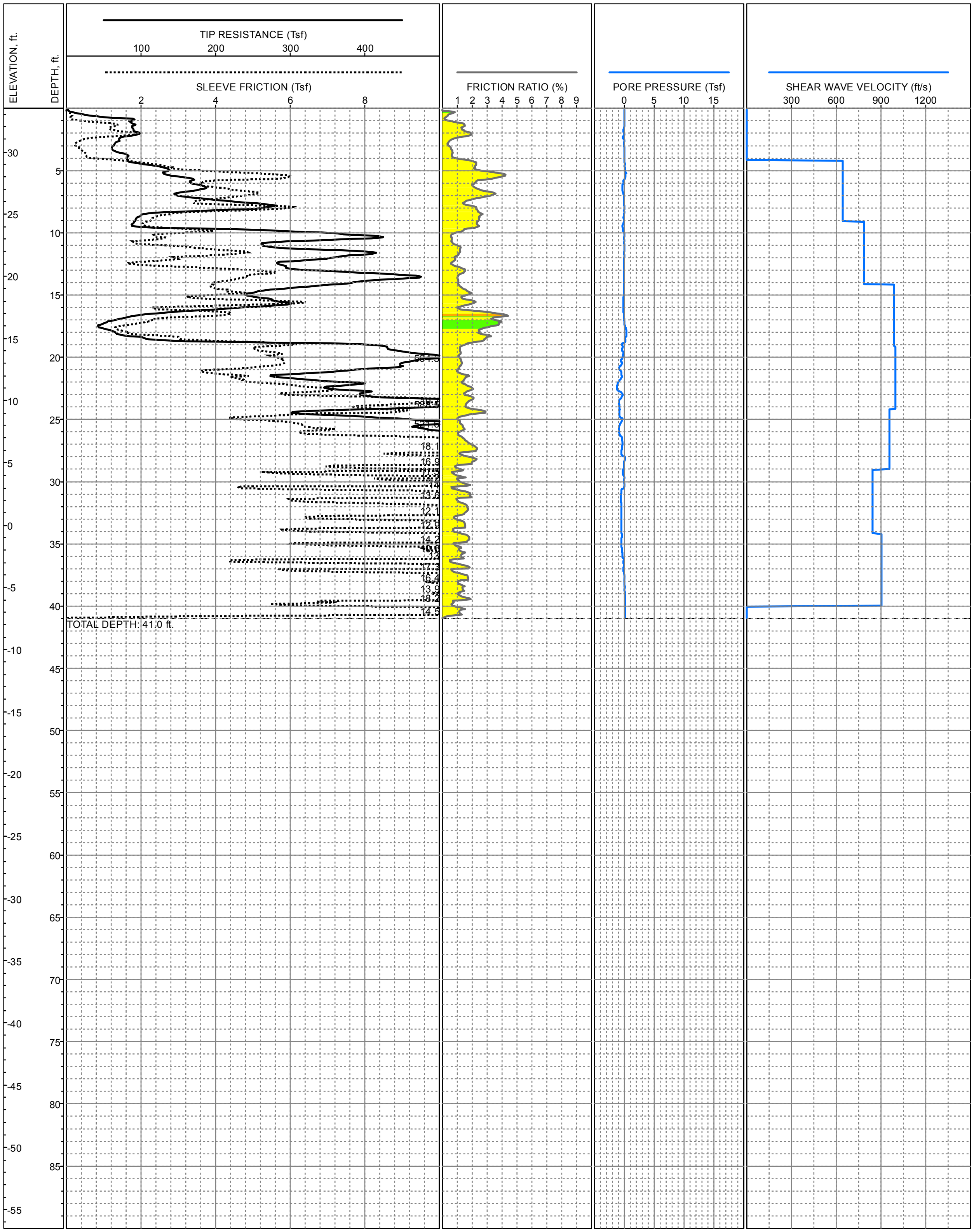


LOCATION: E5,997,950, N 1,979,685, CA State Plane, Zone 5, NAD83, Feet
SURFACE EL: 33.5ft +/- (NAVD88)
COMPLETION DEPTH: 29.0ft
TESTDATE: 6/21/2013

EXPLORATION METHOD: CPT
PERFORMED BY: Kehoe Testing and Engineering
REVIEWED BY: G. Denlinger
CONE AREA RATIO: 0.85

LOG OF CPT NO: SCPT-2

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



LOCATION: E5,997,905, N 1,979,685, CA State Plane, Zone 5, NAD83, Feet
SURFACE EL: 33.5ft +/- (NAVD88)
COMPLETION DEPTH: 41.0ft
TESTDATE: 6/21/2013

EXPLORATION METHOD: CPT
PERFORMED BY: Kehoe Testing and Engineering
REVIEWED BY: G. Denlinger
CONE AREA RATIO: 0.85

LOG OF CPT NO: SCPT-3

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

N:\Projects\04_2013\04_6213_0070_UCSB_SJApts\Explorations\CPT\2013\Logs\2013_07_02_Logs_Vs\MXD\CPT_Logs_VQ13_Vs.mxd,07/29/2013,CDean

APPENDIX B – LABORATORY TESTING

University of California at Santa Barbara
Project No. 04.62130070

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	Specific Gravity
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (%)				
DH-1	10.5	2a	Fat CLAY with sand (CH)	125	104	20	71																
DH-1	15.0	3	Fat CLAY with sand (CH)					55	37														
DH-1	20.5	4	Poorly graded SAND with silt (SP-SM)	123	103	19																	
DH-2	2.0	A	Lean CLAY with sand (CL)							120.9	11.4												
DH-2	6.0	1	Sandy Fat CLAY (CH)	136	119	14																	
DH-2	16.0	3	Clayey SAND (SC)	137	120	14	48																
DH-3	6.0	1	Sandy Fat CLAY (CH)	131	112	17																	
DH-3	8.0	2	Clayey SAND (SC)				47																
DH-3	15.5	4	Sandy Fat CLAY (CH)	132	113	18																	
DH-4	11.0	2	Sandy Fat CLAY (CH)	134	117	14																	
DH-5	2.0	A	Sandy Fat CLAY (CH)													1826	7.80	35	0.0273	19	29.0		
DH-5	6.0	1	Sandy Fat CLAY (CH)	134	116	15																	
DH-5	8.0	2a	Sandy Fat CLAY (CH)	133	114	17	59																
DH-5	16.0	4	Fat CLAY with sand (CH)	124	100	24																	
DH-5	21.0	5	Fat CLAY with sand (CH)	124	97	28																	
DH-5	40.5	7a	CLAYSTONE (Rx)	124	100	24		54	32						9(3.2)								
DH-6	6.0	1	Sandy Fat CLAY (CH)	131	111	18																	
DH-6	16.0	3	Sandy Fat CLAY (CH)	130	108	20																	
DH-7	5.5	1	Sandy Fat CLAY (CH)	131	112	17																	
DH-7	7.5	2a	Sandy Fat CLAY (CH)	132	111	19																	
DH-7	10.5	3	Sandy Fat CLAY (CH)	135	116	17																	
DH-7	20.5	5	Fat CLAY with sand (CH)	132	112	18	72																
DH-8	6.0	1	Lean CLAY with sand (CL)	129	109	18		48	37														
DH-8	16.0	3	Fat CLAY with sand (CH)	130	109	20																	
DH-9	5.5	1	Fat CLAY with sand (CH)	131	109	20																	
DH-9	8.0	2a	Fat CLAY with sand (CH)	133	115	16																	
DH-9	8.5	2b	Fat CLAY with sand (CH)	135	117	16									5.6(0.7)								
DH-9	16.0	4	Fat CLAY with sand (CH)	135	117	16																	
DH-9	35.5	8a	CLAYSTONE (Rx)	124	100	24									7.9(2.8)								
DH-9	36.0	8b	CLAYSTONE (Rx)	124	100	24																	

SUMMARY OF LABORATORY TEST RESULTS

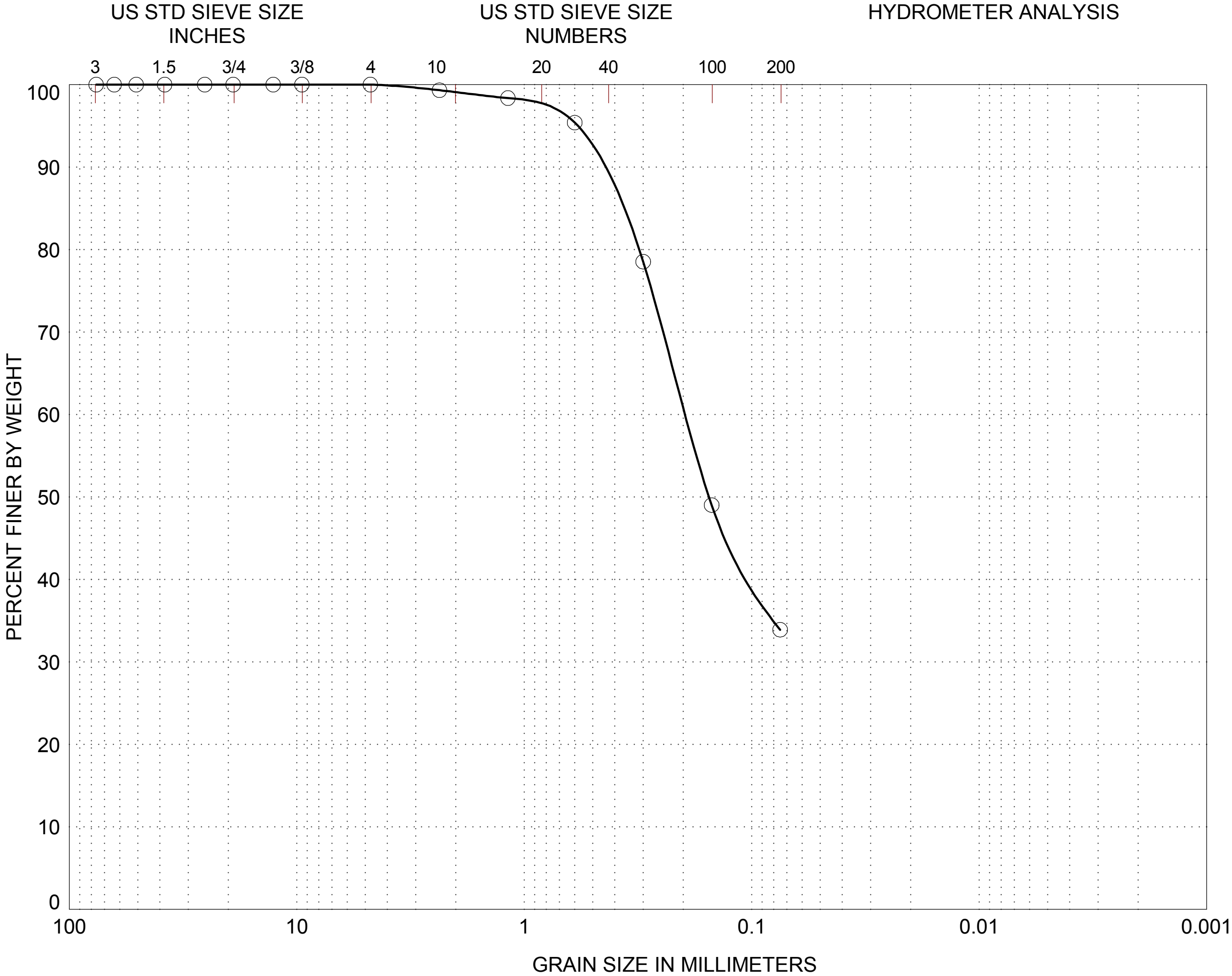
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE B-1a

LAB SUMMARY TABLE VENTURA _N:\PROJECTS\04_2013\04_6213_0070_UCSB_SJAPTS\EXPLORATIONS\GINT\201304_6213_0070_VQ138.GPJ_ 7/28/13 06:03 PM-cab

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	Specific Gravity
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (%)				
DH-10	6.0	1	Fat CLAY with sand (CH)	125	102	22																	
DH-10	8.5	2	Sandy Fat CLAY (CH)	135	117	16	59																
DH-10	16.0	4	Sandy Fat CLAY (CH)	135	117	16																	
DH-11	3.5	1	Sandy Fat CLAY (CH)	134	120	12																	
DH-11	6.0	2	Sandy Fat CLAY (CH)	133	120	10																	
DH-11	8.5	3	Fat CLAY with sand (CH)	131	112	17									12.3(0.7)								
DH-11	11.0	4	Fat CLAY with sand (CH)				79																
DH-12	5.5	1a	Sandy Fat CLAY (CH)	136	121	12																	
DH-12	8.0	2	Fat CLAY with sand (CH)	129	104	23																	
DH-12	10.5	3	Sandy Lean CLAY (CL)	118	108	9		26	8			0.1	32										
DH-12	20.0	5a	Poorly graded SAND with silt (SP-SM)	127	111	15																	
DH-13	5.5	1	Fat CLAY with sand (CH)	135	114	19																	
DH-13	13.5	3	Sandy Fat CLAY (CH)	130	108	20																	
DH-14	6.0	1	Sandy Fat CLAY (CH)	131	112	17						0.5	25										
DH-14	10.0	2	Clayey SAND (SC)				34																
DH-15	6.0	1	Sandy Fat CLAY (CH)	129	107	21																	
DH-15	10.0	2	Clayey SAND (SC)				33																
DH-15	13.5	3	Poorly graded SAND with clay (SP-SC)	129	114	13																	
DH-16	2.0	A	Sandy Lean CLAY (CL)																		102.0		
DH-16	6.0	1	Sandy Fat CLAY (CH)	133	116	15	55																
DH-16	10.5	3	Sandy Fat CLAY (CH)	134	116	15																	
DH-16	35.0	8	CLAYSTONE (Rx)			20																	
DH-16	40.0	9a	CLAYSTONE (Rx)	131	111	18									8.5(3.2)								
DH-17	8.5	2	Sandy Fat CLAY (CH)	131	113	16	69																
DH-18	6.0	1	Sandy Fat CLAY (CH)	134	116	16																	
DH-19	5.5	1	Fat CLAY with sand (CH)	127	104	22																	
DH-19	13.5	3	Sandy Fat CLAY (CH)	132	113	17																	
DH-20	5.5	1	SILT with sand (ML)	108	101	6		17	2														
DH-20	11.0	2	Sandy Fat CLAY (CH)	131	116	13																	

SUMMARY OF LABORATORY TEST RESULTS
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

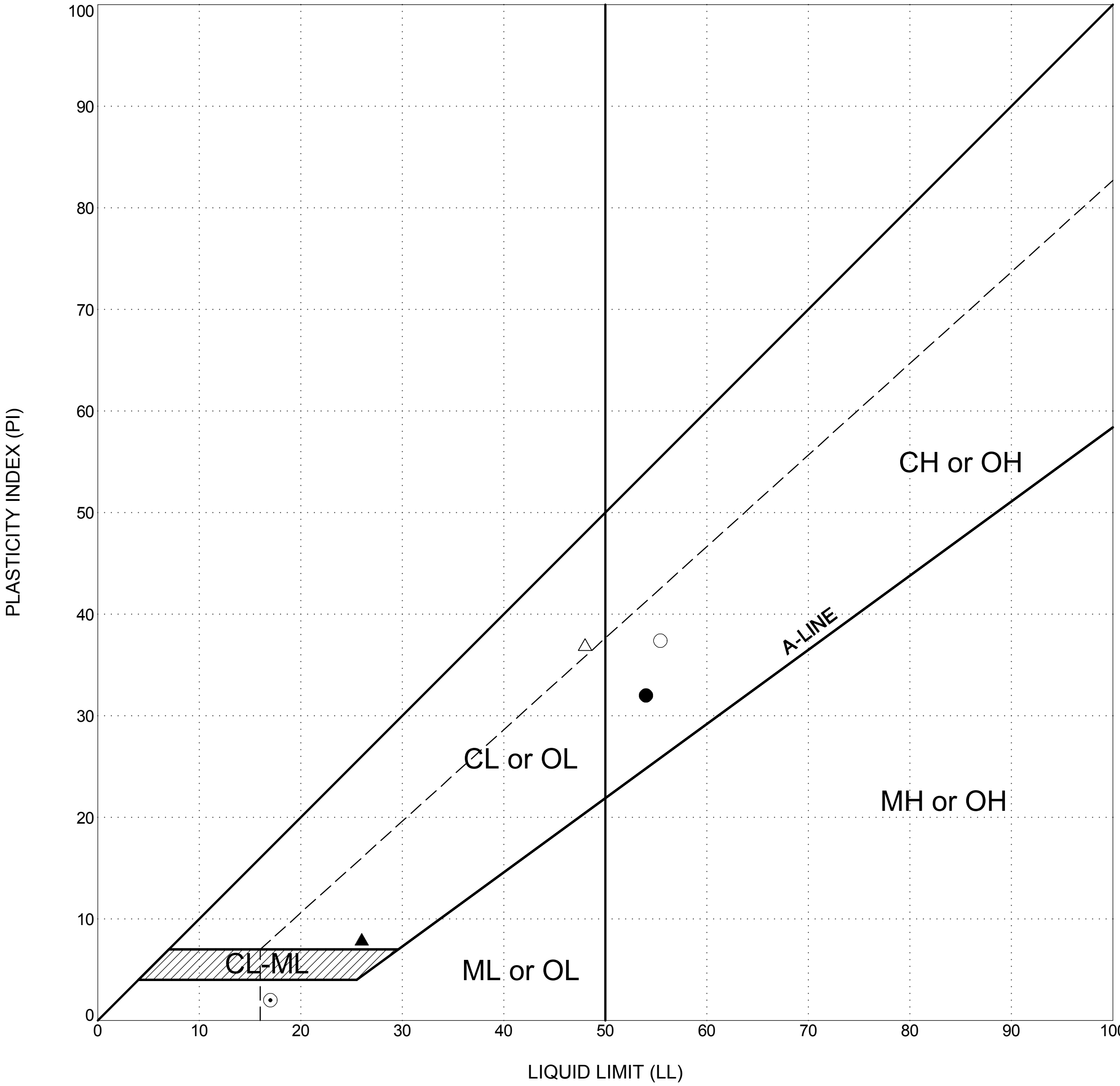


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LEGEND (location)	(depth,ft)	CLASSIFICATION	Cc	Cu
○ DH-14	10.0	Clayey SAND (SC)		

GRAIN SIZE CURVES
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE B-2

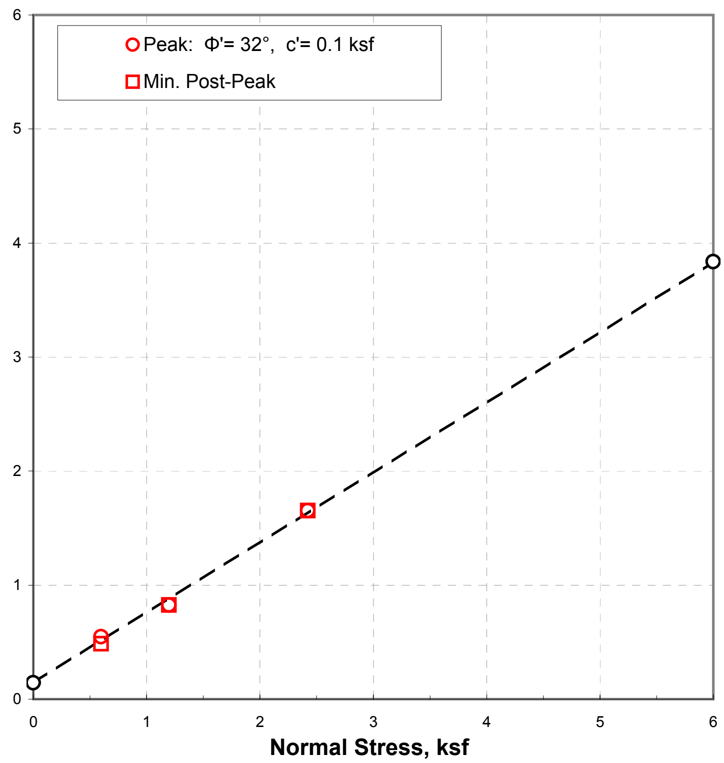


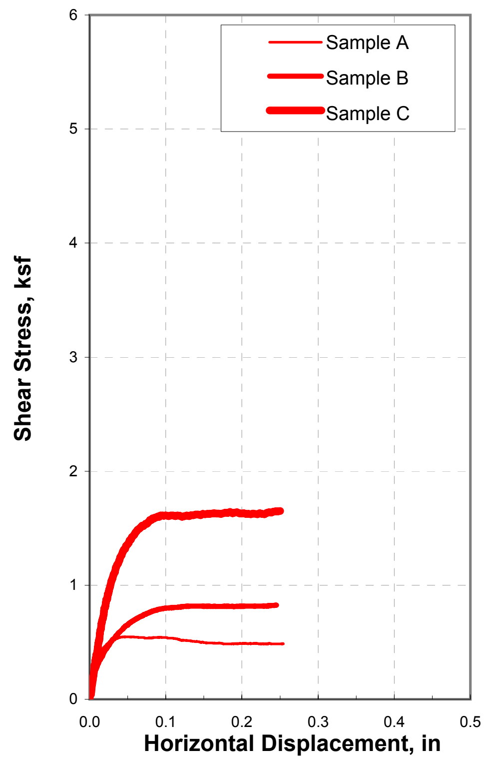
ATTERBERG LIMITS TEST RESULTS

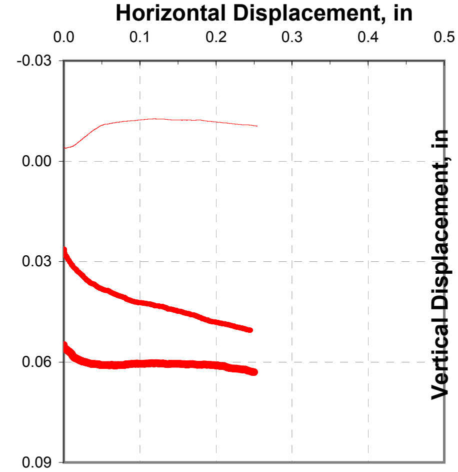
LEGEND	location	depth, ft	CLASSIFICATION	LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
○	DH-1	15.0	Fat CLAY with sand (CH)	55	18	37
●	DH-5	40.5	CLAYSTONE (Rx)	54	22	32
△	DH-8	6.0	Lean CLAY with sand (CL)	48	11	37
▲	DH-12	10.5	Sandy Lean CLAY (CL)	26	18	8
⊙	DH-20	5.5	SILT with sand (ML)	17	15	2

PLASTICITY CHART
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE B-3







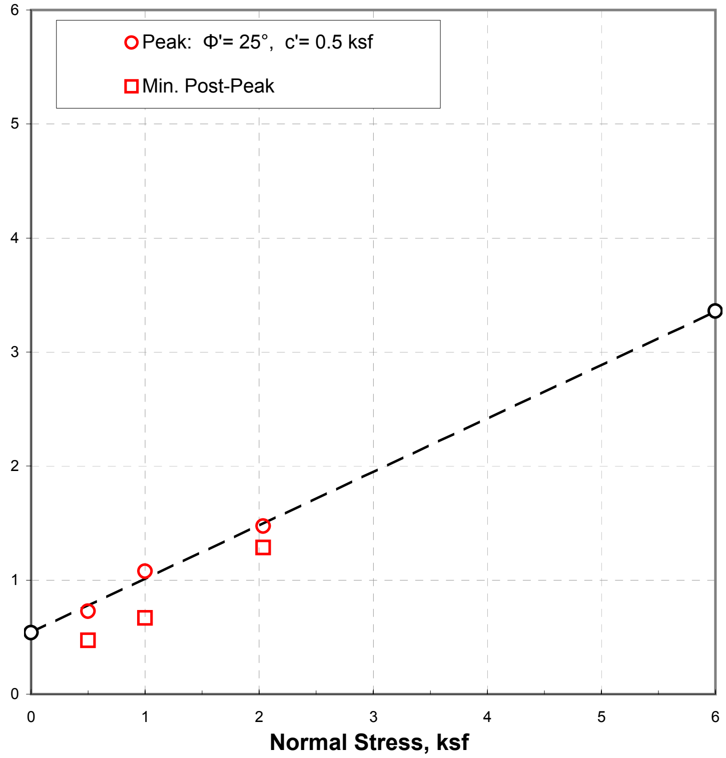
SAMPLE ID	Boring Number:	DH-12			
	Sample Number:	3			
	Sample Depth:	10.5 ft			
	USCS Classification:	Sandy lean CLAY (CL): brown mottled w/black & FeO2 staining, moist			
INITIAL	Specimen	A	B	C	D
	Water Content, %	9.3%	9.3%	9.3%	
	Dry Unit Weight, pcf	110.4	102.4	107.8	
	Saturation, %	49%	40%	46%	
	Void Ratio	0.50	0.61	0.53	
	Diameter, in	2.42	2.42	2.42	
	Height, in	1.00	1.00	0.98	
FINAL	Water Content, %	20.8%	20.5%	19.2%	
	Dry Unit Weight, pcf	103.4	102.1	106.3	
	Void Ratio	0.60	0.62	0.56	
TEST SUMMARY	Displacement at Peak, in	0.04	0.24	0.25	
	Displacement Rate, in/min	0.0004	0.0004	0.0004	
	Normal Stress, ksf	0.6	1.2	2.4	
	Peak Shear Stress, ksf	0.55	0.83	1.65	
	Min. Post-Peak Stress, ksf	0.49	0.82	1.65	
	Swell Pressure, ksf	1.02	0.32	0.37	
REMARKS	Test Method: ASTM D3080				

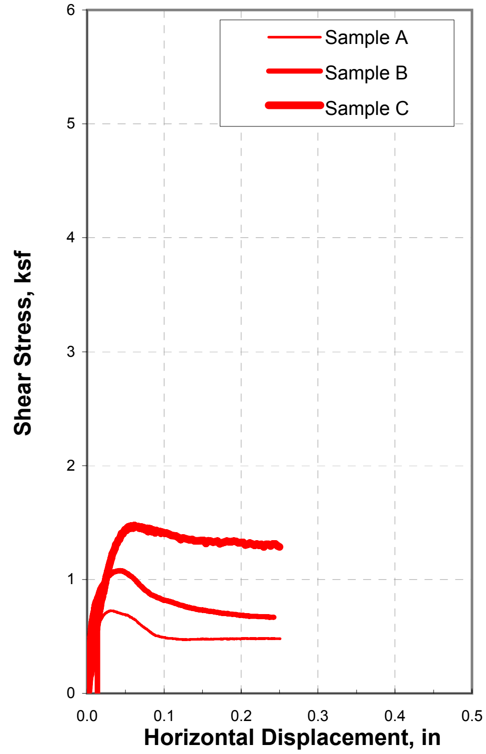
CLASSIFICATION		
	Sieve Size	% Passing
	3/8-in. (9.5mm)	---
	#4 (4.75mm)	---
	#16 (1.18mm)	---
	#30 (0.6mm)	---
	#100 (0.150mm)	---
	#200 (0.075mm)	---
	Atterberg Limits	
	Liquid Limit, %	26
	Plastic Limit, %	18
	Plasticity Index, %	8
	Estimated Gs	2.65
	k_{avg} 20°C, cm/sec	---

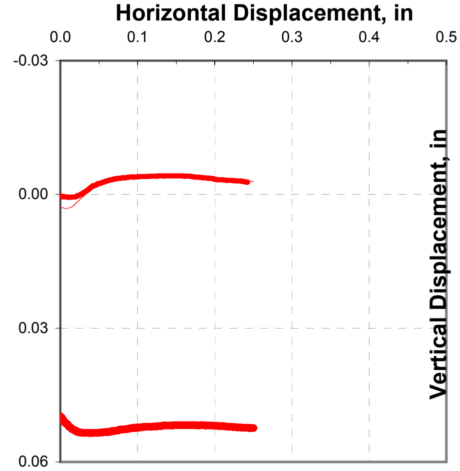
DIRECT SHEAR

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE B-4a







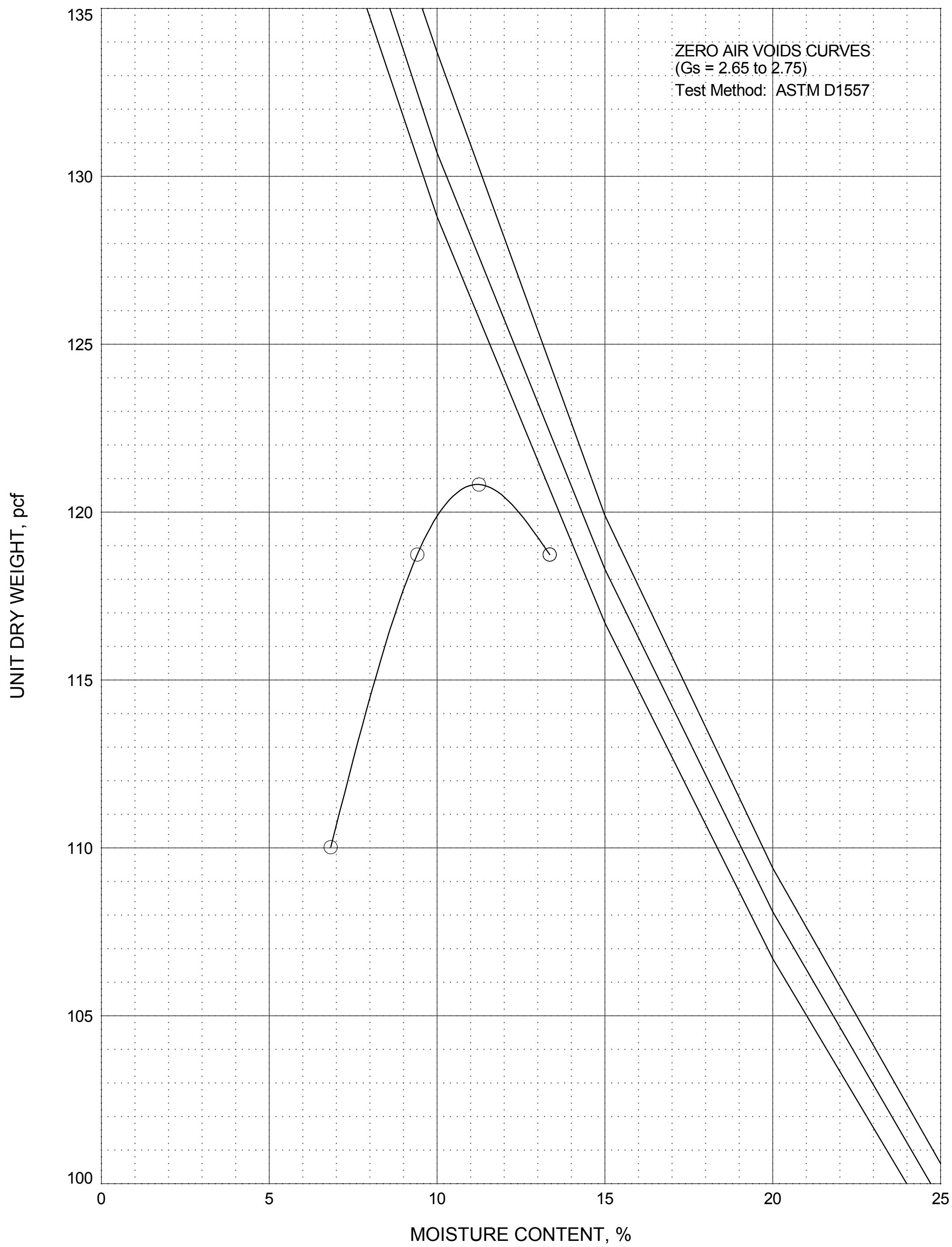
SAMPLE ID		
	Boring Number:	DH-14
	Sample Number:	1
	Sample Depth:	6.0 ft
	USCS Classification:	Sandy fat CLAY (CH): brown mottled w/dark gray, moist

	Specimen	A	B	C	D
INITIAL	Water Content, %	17.4%	17.4%	17.4%	
	Dry Unit Weight, pcf	111.7	112.0	112.4	
	Saturation, %	96%	97%	98%	
	Void Ratio	0.48	0.48	0.47	
	Diameter, in	2.42	2.42	2.42	
	Height, in	1.00	1.00	0.99	
FINAL	Water Content, %	21.2%	20.8%	19.8%	
	Dry Unit Weight, pcf	106.5	107.2	109.1	
	Void Ratio	0.55	0.54	0.52	
TEST SUMMARY	Displacement at Peak, in	0.03	0.04	0.06	
	Displacement Rate, in/min	0.0004	0.0004	0.0004	
	Normal Stress, ksf	0.5	1.0	2.0	
	Peak Shear Stress, ksf	0.73	1.08	1.47	
	Min. Post-Peak Stress, ksf	0.47	0.67	1.29	
	Swell Pressure, ksf	0.54	0.98	---	
REMARKS	Test Method: ASTM D3080				

CLASSIFICATION	Sieve Size	% Passing
	3/8-in. (9.5mm)	---
	#4 (4.75mm)	---
	#16 (1.18mm)	---
	#30 (0.6mm)	---
	#100 (0.150mm)	---
	#200 (0.075mm)	---
	Atterberg Limits	
	Liquid Limit, %	---
	Plastic Limit, %	---
	Plasticity Index, %	---
	Estimated Gs	2.65
	k_{avg} 20°C, cm/sec	---

DIRECT SHEAR

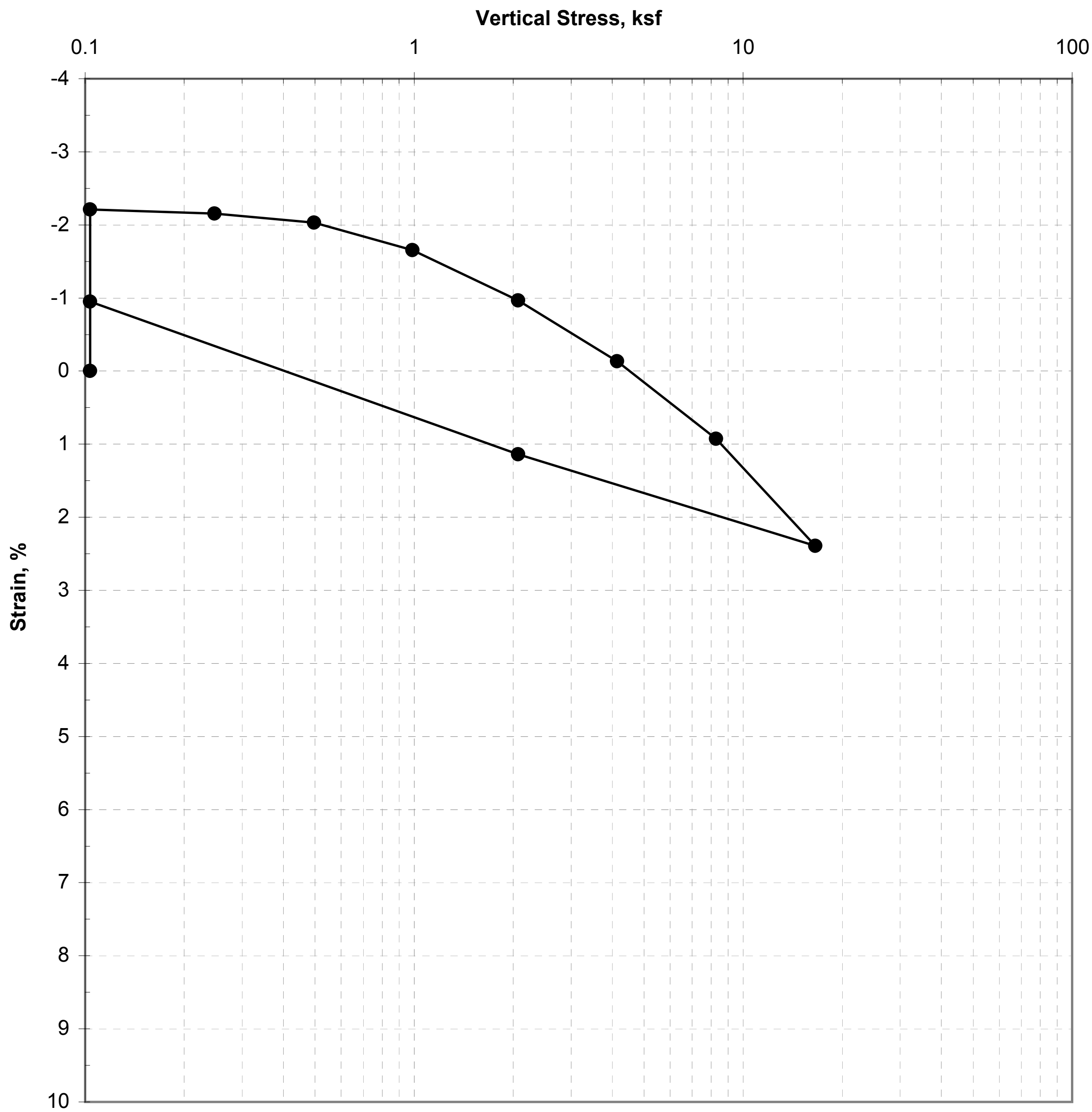
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



LEGEND (location)	depth, ft	CLASSIFICATION	MAXIMUM UNIT DRY WEIGHT, pcf	OPTIMUM WATER CONTENT, %
○ DH-2	2.0	Lean CLAY with sand (CL)	120.9	11.4

COMPACTION TEST RESULTS
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

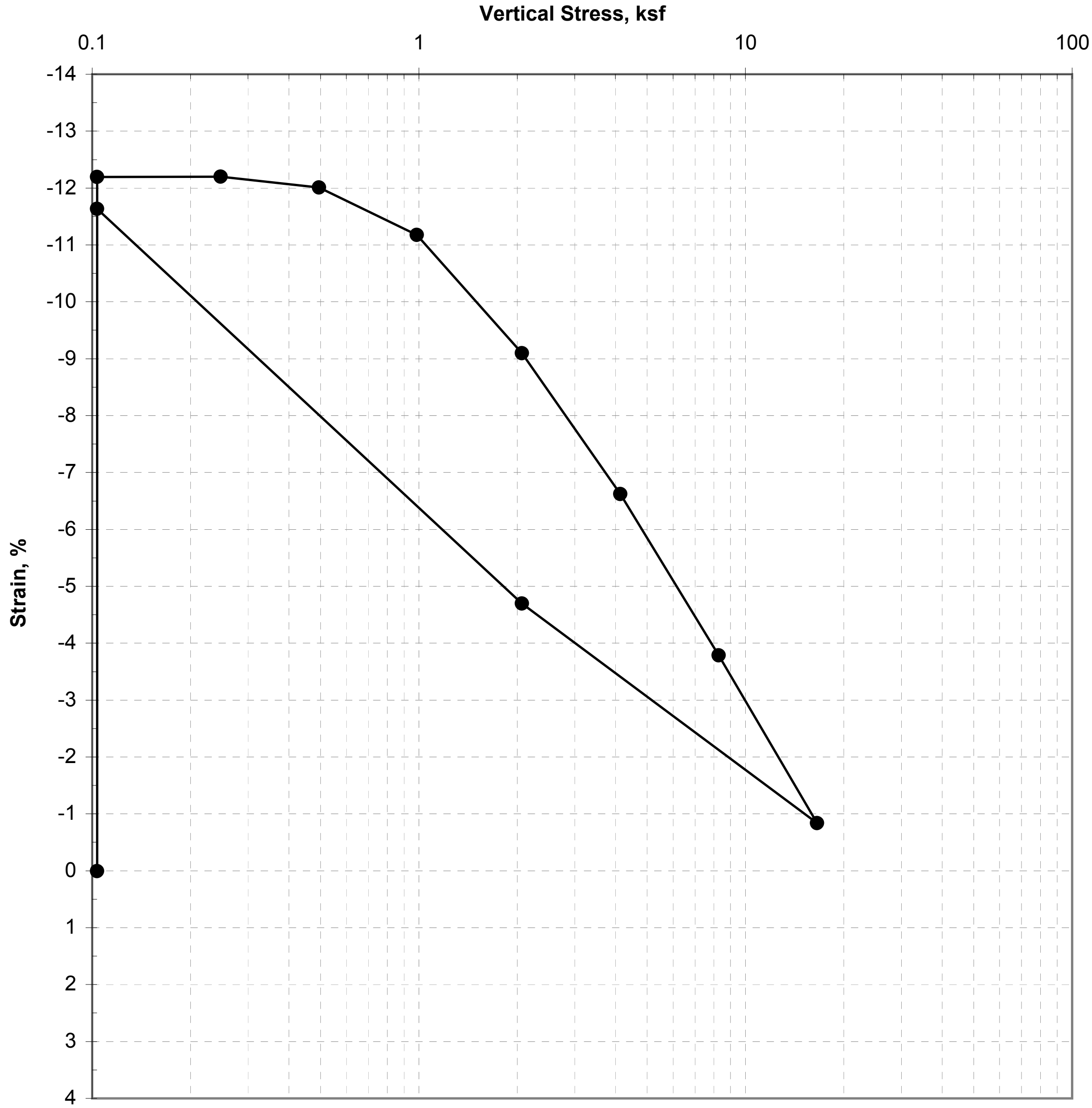
PLATE B-5



SAMPLE ID	Boring, Sample #, Depth	DH-9 , #2A , 8.0 ft		SUMMARY	Preconsolidation Pressure, ksf	---
	USCS Classification:	Fat CLAY with sand (CH): dark grayish brown mottled w/brown & black, moist			Inundation Increment, ksf	0.10
PROPERTIES		Initial	Final		Liquid Limit	---
	Water Content, %	15.7%	17.7%		Plastic Limit	---
	Dry Unit Weight, pcf	114.8	113.7		Plasticity Index	---
	Saturation, %	91%	100%		Passing #200	---
	Void Ratio	0.46	0.48		Estimated Gs	2.69
	Diameter, in	2.42	2.42	REMARKS	Test Method: D2435	
	Height, in	0.82	0.83			

CONSOLIDATION

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID		
Boring, Sample #, Depth	DH-12 , #2 , 8.0 ft	
USCS Classification:	Fat CLAY with sand (CH): brown mottled w/pale brown, black, & FeO2 staining, moist	

PROPERTIES	Initial	Final
Water Content, %	23.1%	30.2%
Dry Unit Weight, pcf	104.4	93.5
Saturation, %	100%	100%
Void Ratio	0.63	0.82
Diameter, in	2.42	2.42
Height, in	0.82	0.91

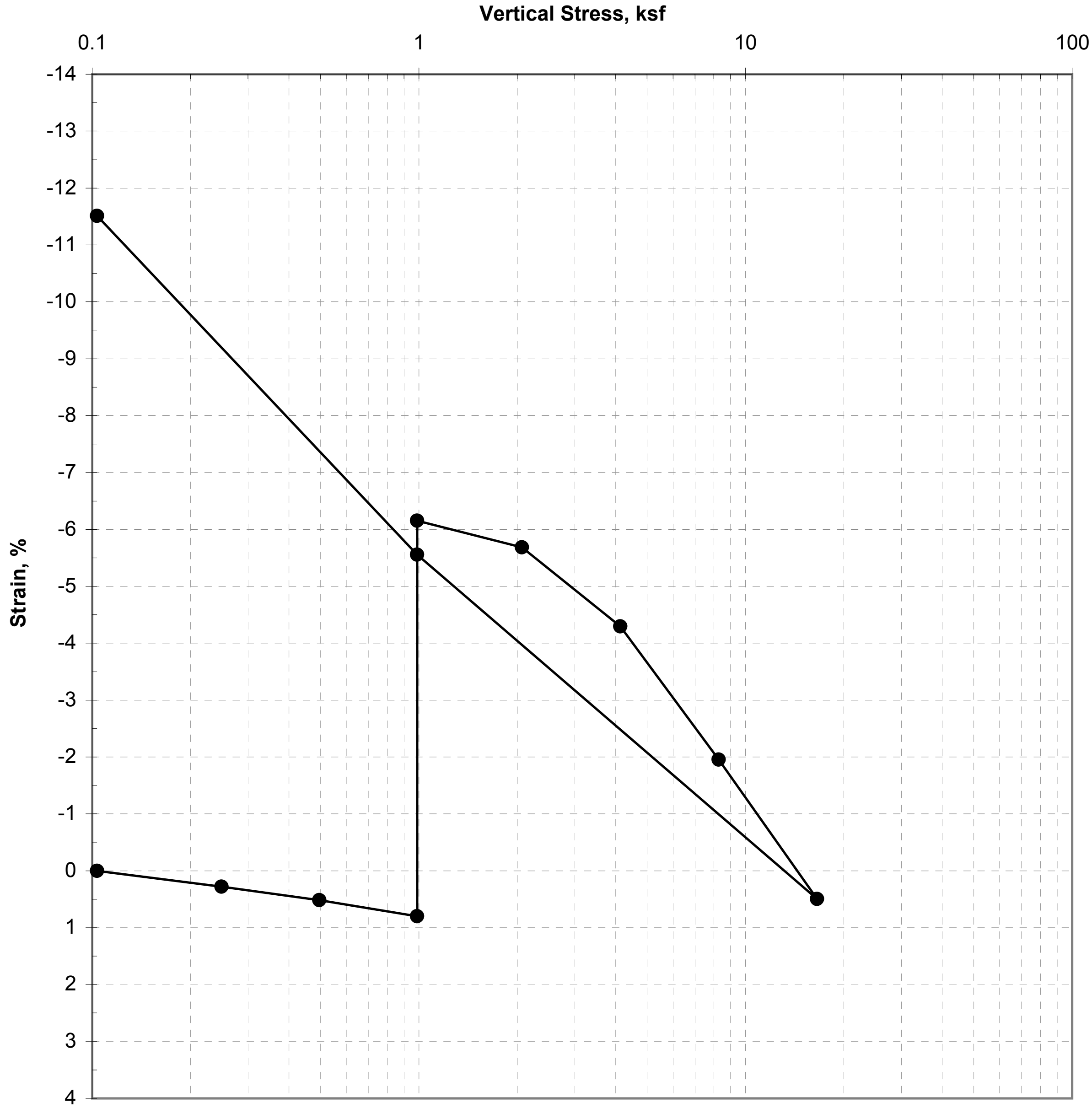
SUMMARY	
Preconsolidation Pressure, ksf	---
Inundation Increment, ksf	0.10
Liquid Limit	---
Plastic Limit	---
Plasticity Index	---
Passing #200	---
Estimated Gs	2.73

REMARKS: Test Method: D2435

CONSOLIDATION

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

PLATE B-6b



SAMPLE ID			
Boring, Sample #, Depth	DH-12 , #2 , 8.0 ft		
USCS Classification:	Fat CLAY with sand (CH): brown mottled w/pale brown, black, & FeO2 staining, moist		

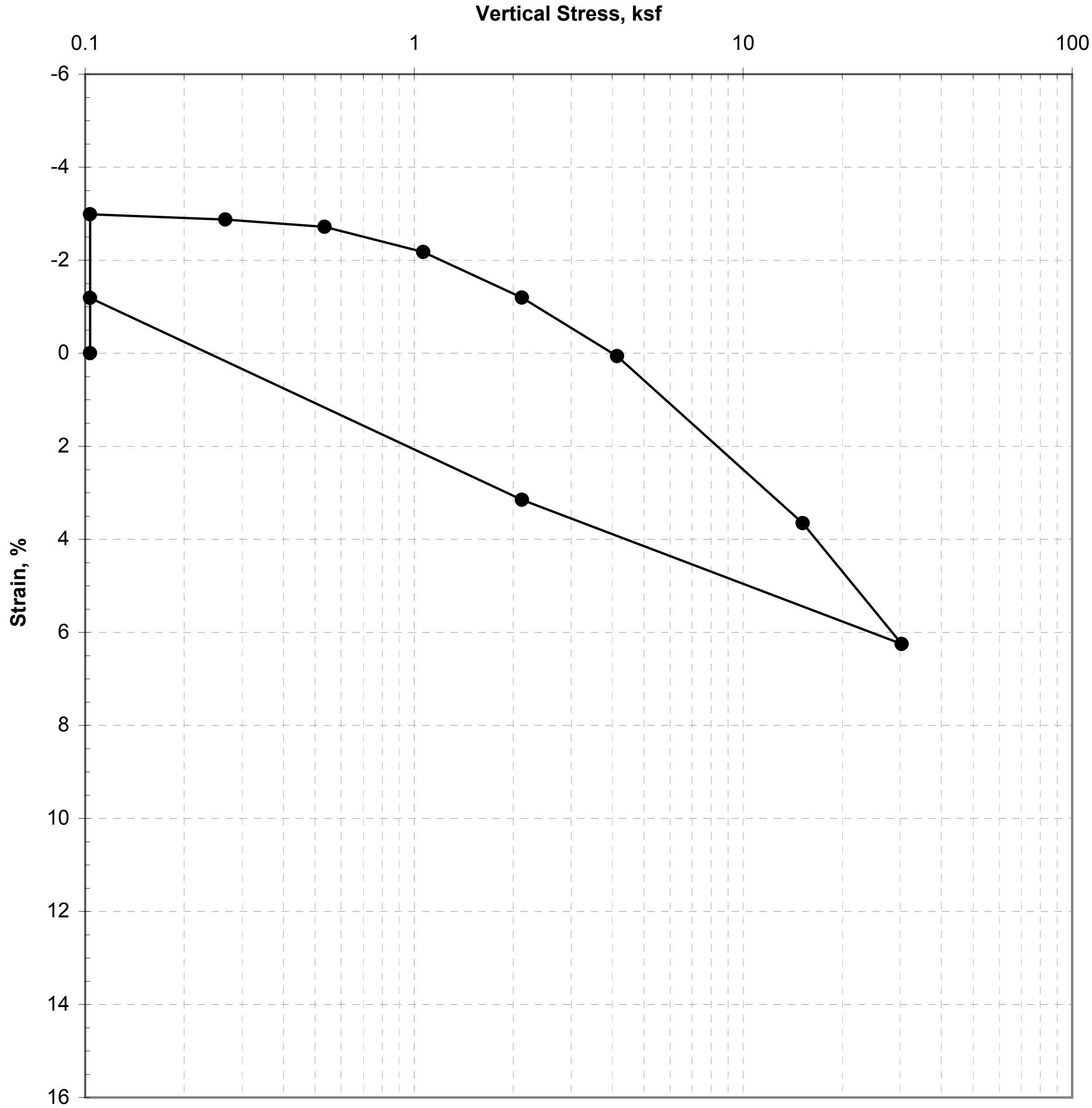
PROPERTIES	Initial	Final
Water Content, %	25.5%	33.8%
Dry Unit Weight, pcf	99.5	89.3
Saturation, %	96%	100%
Void Ratio	0.73	0.93
Diameter, in	2.42	2.42
Height, in	0.82	0.91

SUMMARY	
Preconsolidation Pressure, ksf	---
Inundation Increment, ksf	0.99
Liquid Limit	---
Plastic Limit	---
Plasticity Index	---
Passing #200	---
Estimated Gs	2.76

REMARKS: Test Method: D2435

CONSOLIDATION

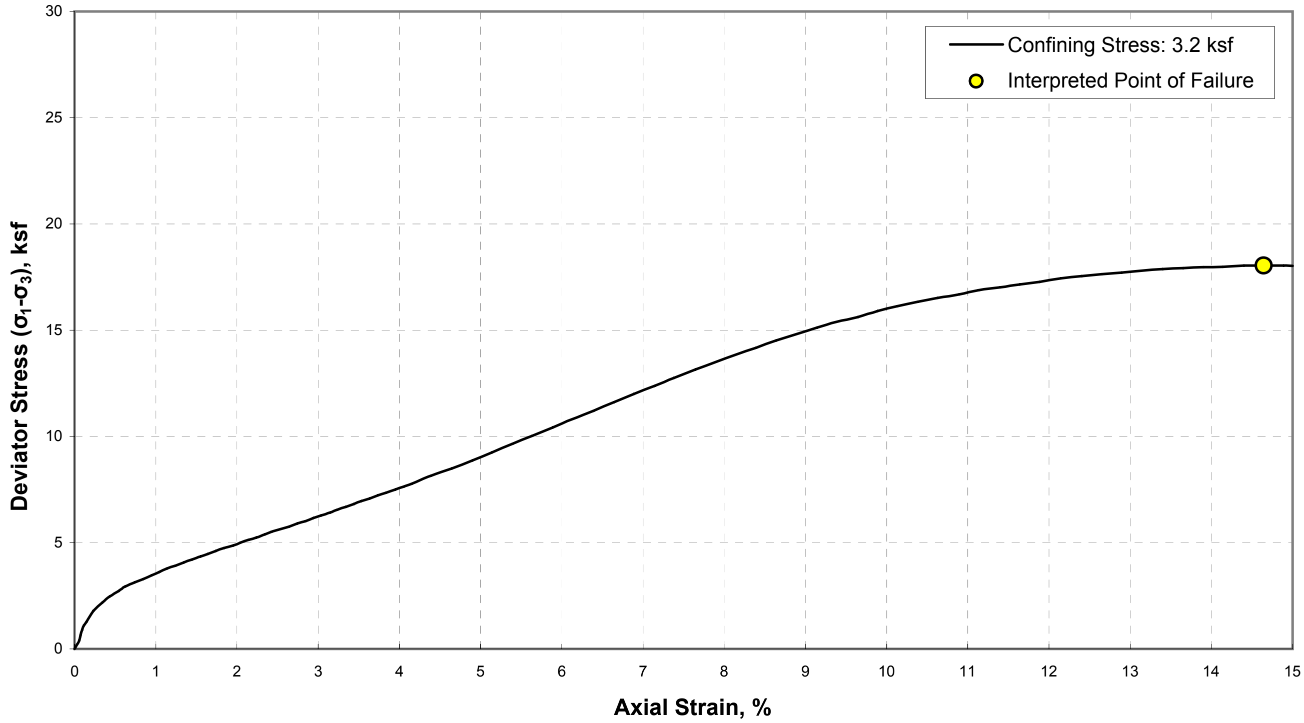
San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID	Boring, Sample #, Depth	DH-15 , #1 , 6.0 ft		SUMMARY	Preconsolidation Pressure, ksf	---
	USCS Classification:	Sandy fat CLAY (CH): brown mottled w/dark gray & yellowish brown, moist, w/pale brown fine sand vaining			Inundation Increment, ksf	0.10
PROPERTIES		Initial	Final		Liquid Limit	---
	Water Content, %	20.6%	22.0%		Plastic Limit	---
	Dry Unit Weight, pcf	107.3	106.1		Plasticity Index	---
	Saturation, %	97%	100%		Passing #200	---
	Void Ratio	0.58	0.59		Estimated Gs	2.71
	Diameter, in	2.42	2.42	REMARKS	Test Method: D2435	
	Height, in	0.82	0.83			

CONSOLIDATION

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-5
Sample Number:	7A
Sample Depth:	40.5 ft
USCS Classification:	CLAYSTONE (Rx): very dark greenish gray, moist

SAMPLE PROPERTIES

Water Content, %	23.9%
Dry Unit Weight, pcf	100.2
Saturation, %	97%
Void Ratio	0.65
Diameter, in	2.42
Height, in	4.88

SAMPLE IMAGES


CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	54
#4 (4.75mm)	---	Plastic Limit	22
#16 (1.18mm)	---	Plasticity Index	32
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

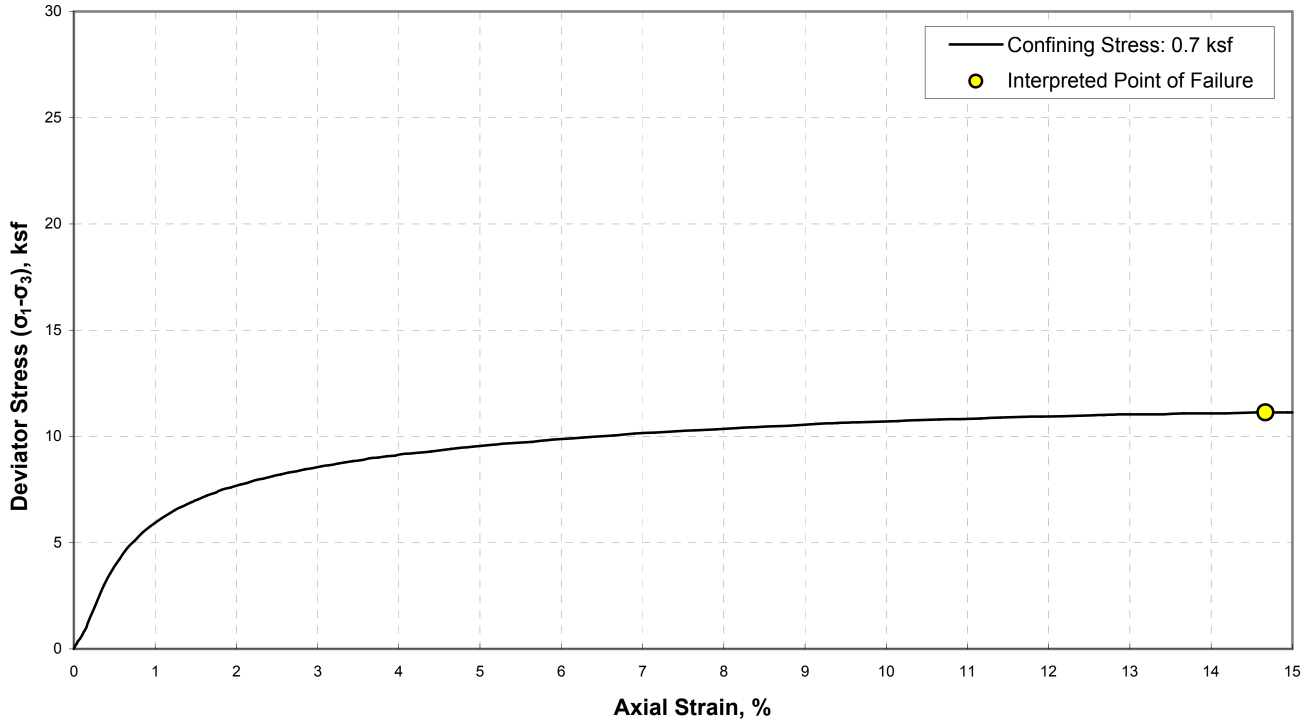
Maximum Deviator Stress, ksf	18.0
Undrained Shear Strength, ksf	9.0
Axial Strain at Failure, %	14.6
Strain Rate, %/min	0.3
Cell Pressure, ksf	3.2
Tested By:	JC
Date Tested:	7/1/13

REMARKS

Test Method: ASTM 2850

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-9
Sample Number:	2B
Sample Depth:	8.5 ft
USCS Classification:	Fat CLAY with sand (CH): olive brown mottled w/dark gray, pale brown, & FeO2 staining, moist

SAMPLE PROPERTIES

Water Content, %	15.6%
Dry Unit Weight, pcf	117.2
Saturation, %	100%
Void Ratio	0.41
Diameter, in	2.42
Height, in	4.88

SAMPLE IMAGES

CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

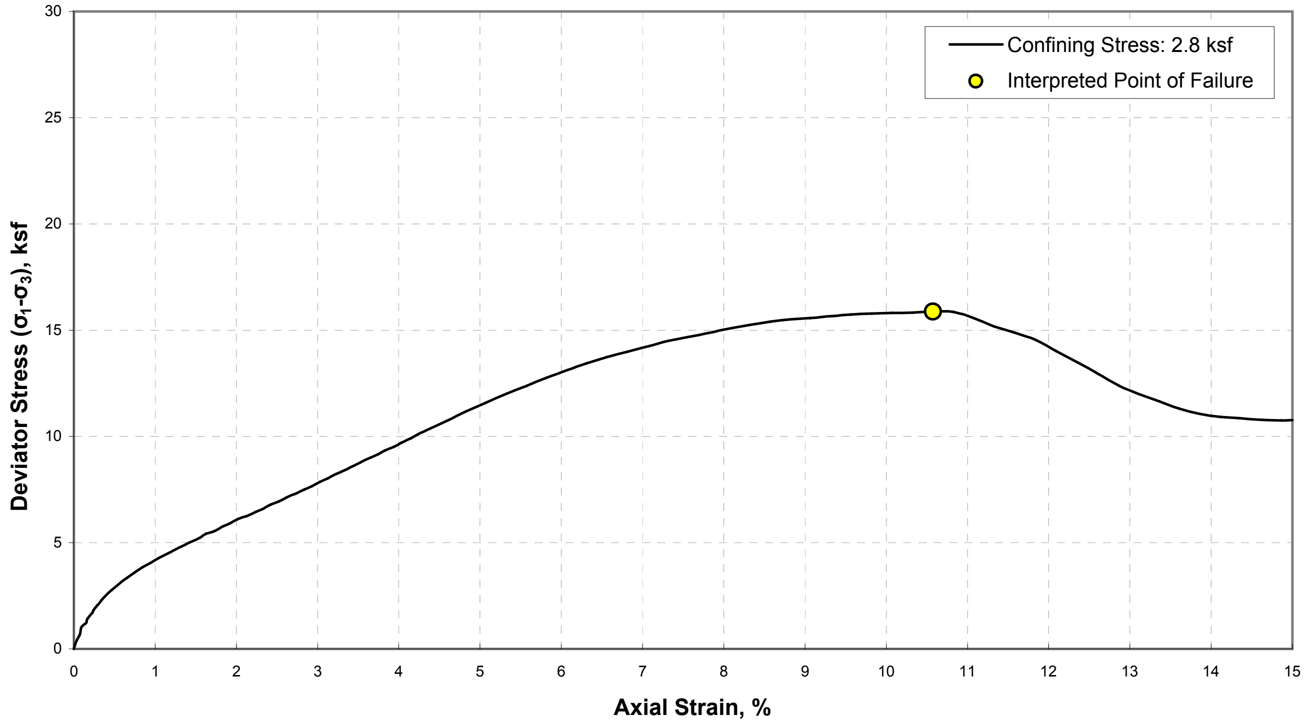
Maximum Deviator Stress, ksf	11.1
Undrained Shear Strength, ksf	5.6
Axial Strain at Failure, %	14.7
Strain Rate, %/min	0.5
Cell Pressure, ksf	0.7
Tested By:	JC
Date Tested:	7/2/13

REMARKS

Test Method: ASTM 2850

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-9
Sample Number:	8A
Sample Depth:	35.5 ft
USCS Classification:	CLAYSTONE (Rx): very dark greenish gray, moist

SAMPLE PROPERTIES

Water Content, %	24.4%
Dry Unit Weight, pcf	99.8
Saturation, %	98%
Void Ratio	0.66
Diameter, in	2.42
Height, in	4.88

SAMPLE IMAGES


CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

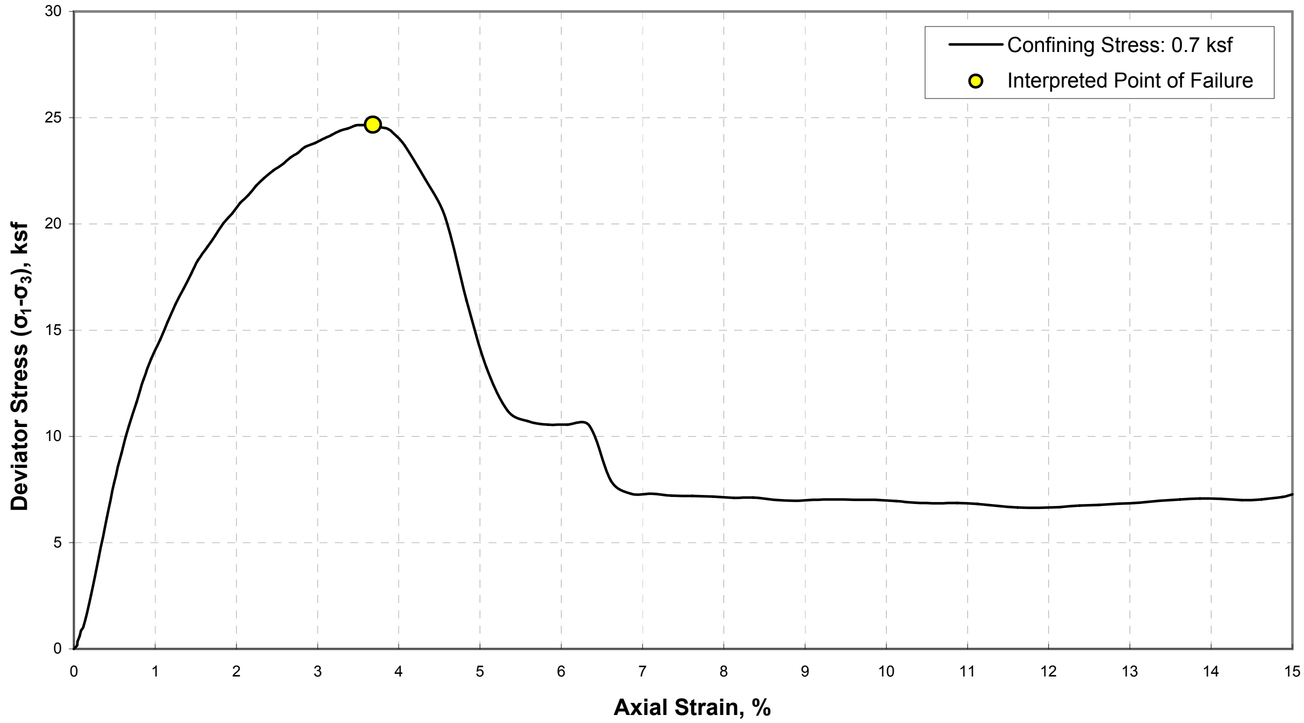
Maximum Deviator Stress, ksf	15.9
Undrained Shear Strength, ksf	7.9
Axial Strain at Failure, %	10.6
Strain Rate, %/min	0.5
Cell Pressure, ksf	2.8
Tested By:	JC
Date Tested:	7/2/13

REMARKS

Test Method: ASTM 2850

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-11
Sample Number:	2
Sample Depth:	8.5 ft
USCS Classification:	Sandy fat CLAY (CH): brown mottled w/dark gray, black, & FeO2 staining, moist

SAMPLE PROPERTIES

Water Content, %	17.1%
Dry Unit Weight, pcf	112.1
Saturation, %	95%
Void Ratio	0.48
Diameter, in	2.42
Height, in	4.88

SAMPLE IMAGES

CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

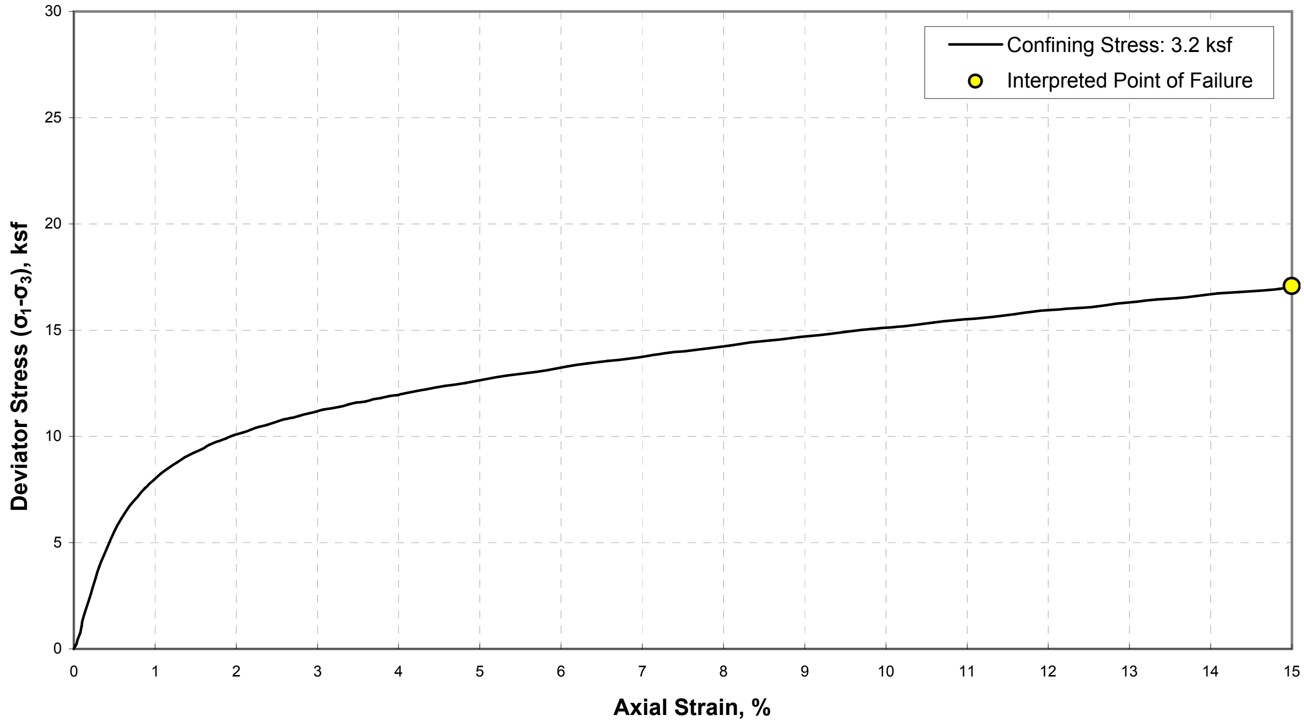
Maximum Deviator Stress, ksf	24.7
Undrained Shear Strength, ksf	12.3
Axial Strain at Failure, %	3.7
Strain Rate, %/min	0.5
Cell Pressure, ksf	0.7
Tested By:	JC
Date Tested:	7/2/13

REMARKS

Test Method: ASTM 2850

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California



SAMPLE ID

Boring Number:	DH-16
Sample Number:	9A
Sample Depth:	40.0 ft
USCS Classification:	CLAYSTONE (Rx): very dark greenish gray, moist

SAMPLE PROPERTIES

Water Content, %	18.1%
Dry Unit Weight, pcf	111.1
Saturation, %	99%
Void Ratio	0.49
Diameter, in	2.42
Height, in	4.88

SAMPLE IMAGES

CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

Maximum Deviator Stress, ksf	17.1
Undrained Shear Strength, ksf	8.5
Axial Strain at Failure, %	15.0
Strain Rate, %/min	0.5
Cell Pressure, ksf	3.2
Tested By:	JC
Date Tested:	7/3/13

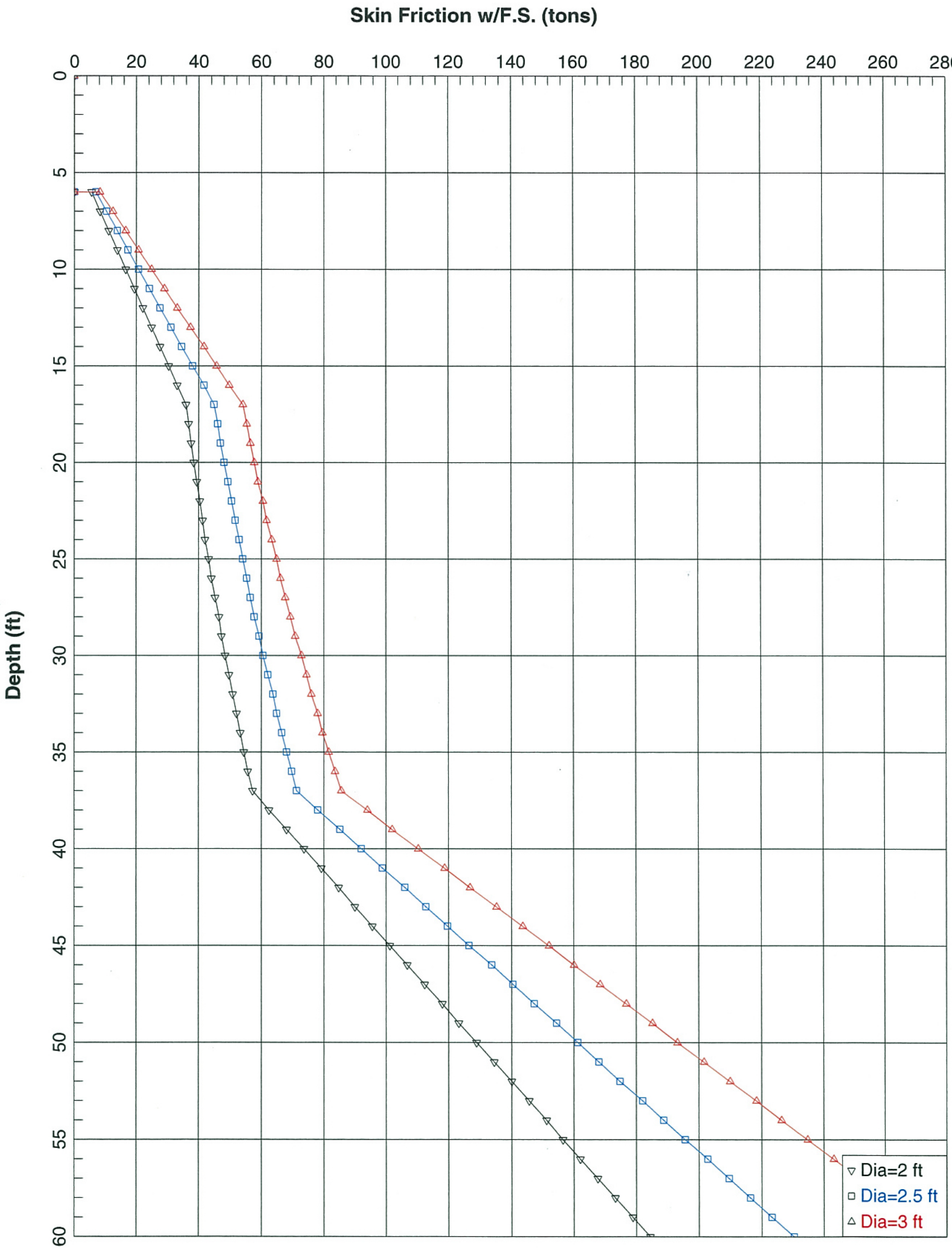
REMARKS

Test Method: ASTM 2850

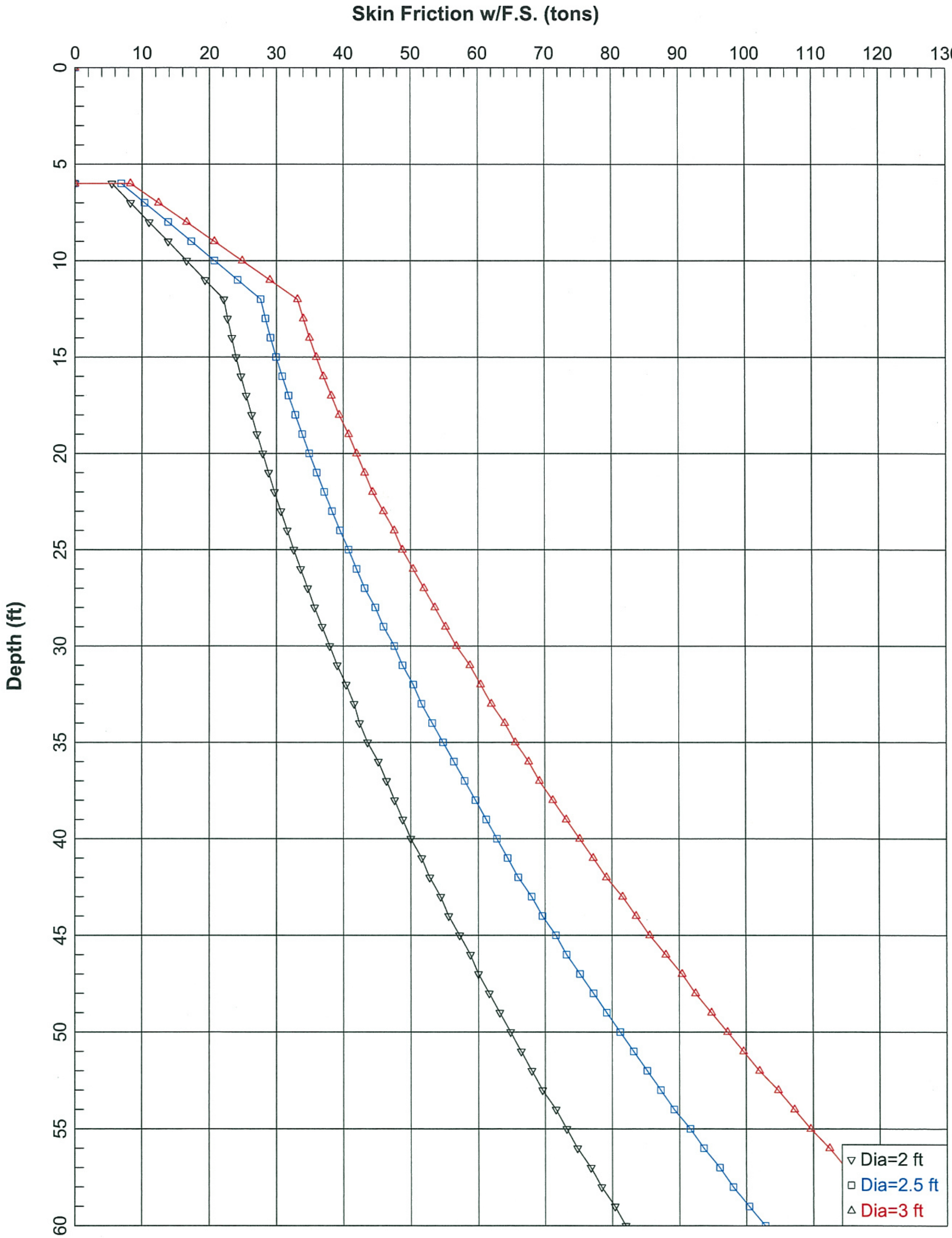
UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

San Joaquin Residence Apartments & Precinct Improvements
Santa Barbara, California

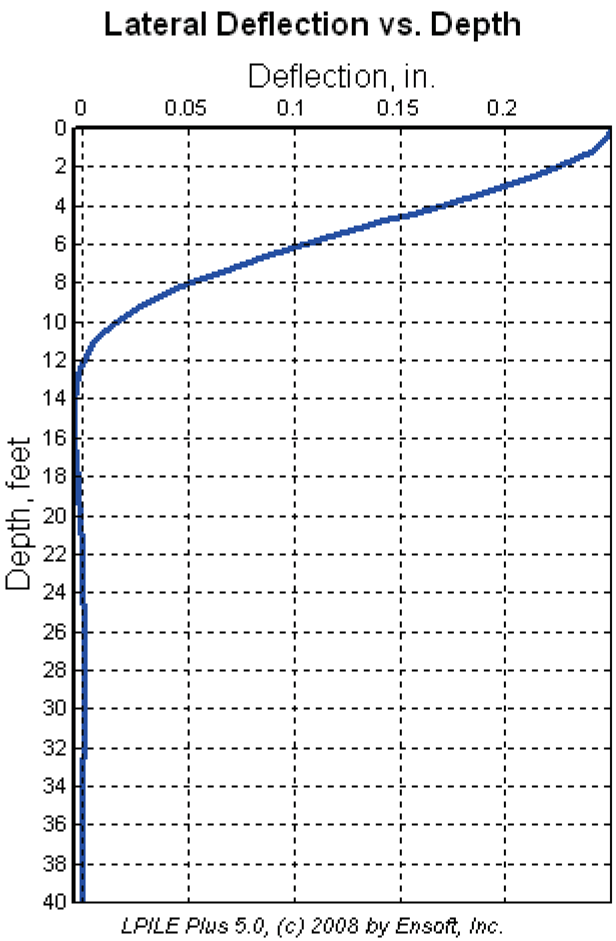
APPENDIX C – AXIAL AND LATERAL PILE CAPACITY RESULTS

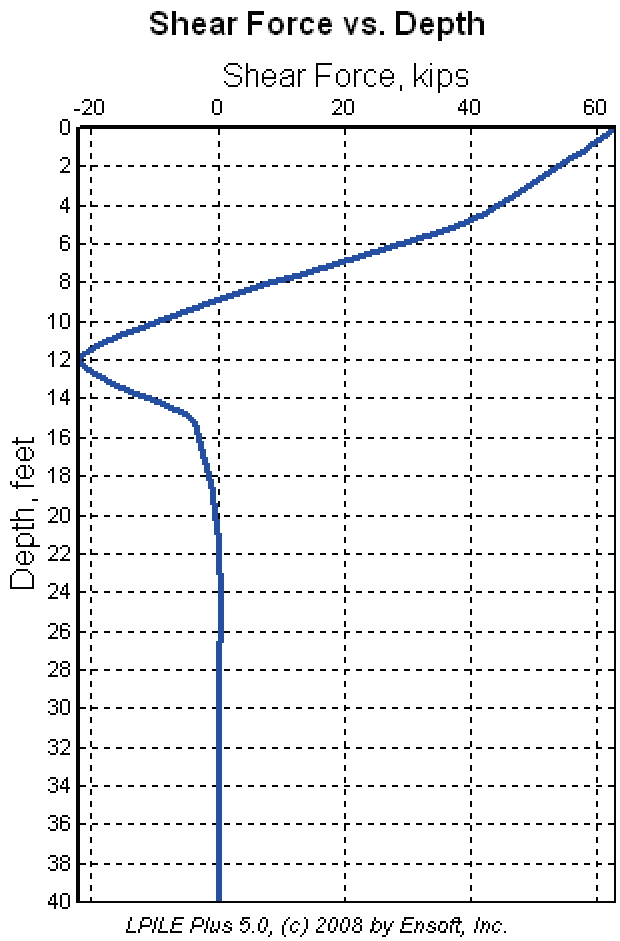


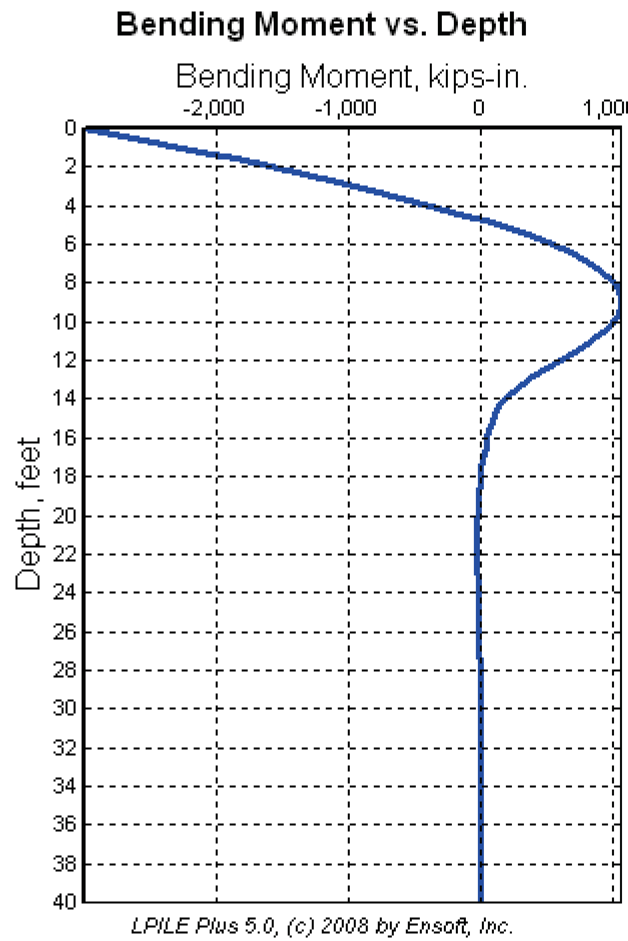
CIDH Pile Axial Capacity - Pico Claystone



CIDH Pile Axial Capacity - Pico Sandstone







Loading Conditions:

Axial Load = 350 Kips

Pile Head Deflection = 0.25”

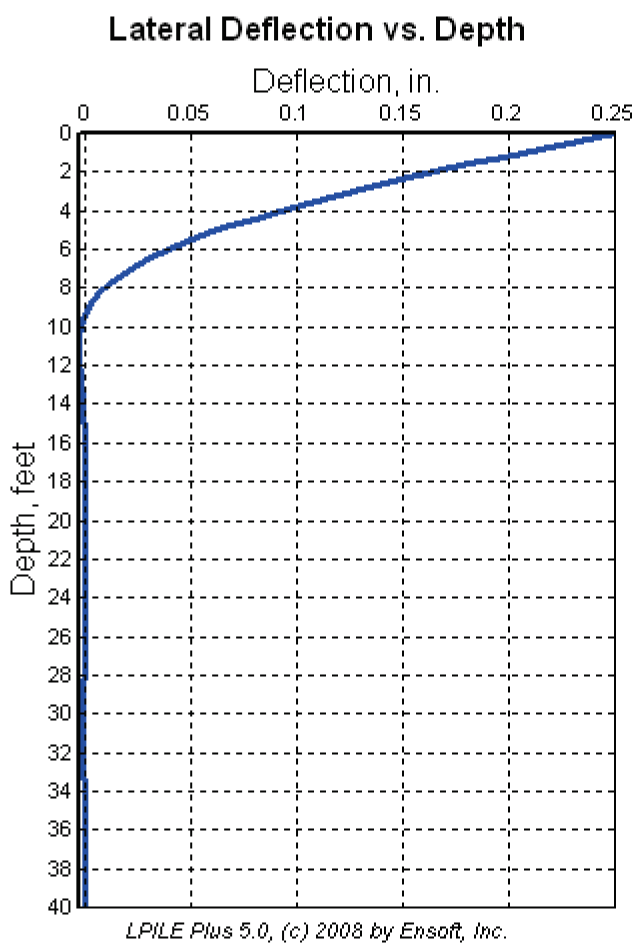
Pile Head Slope = 0

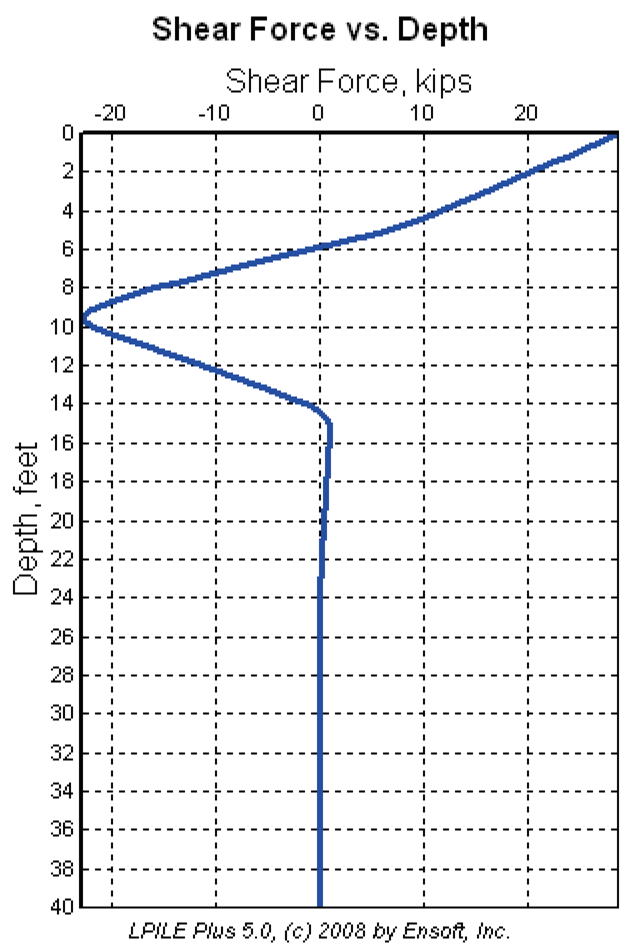
Lateral Capacity Results

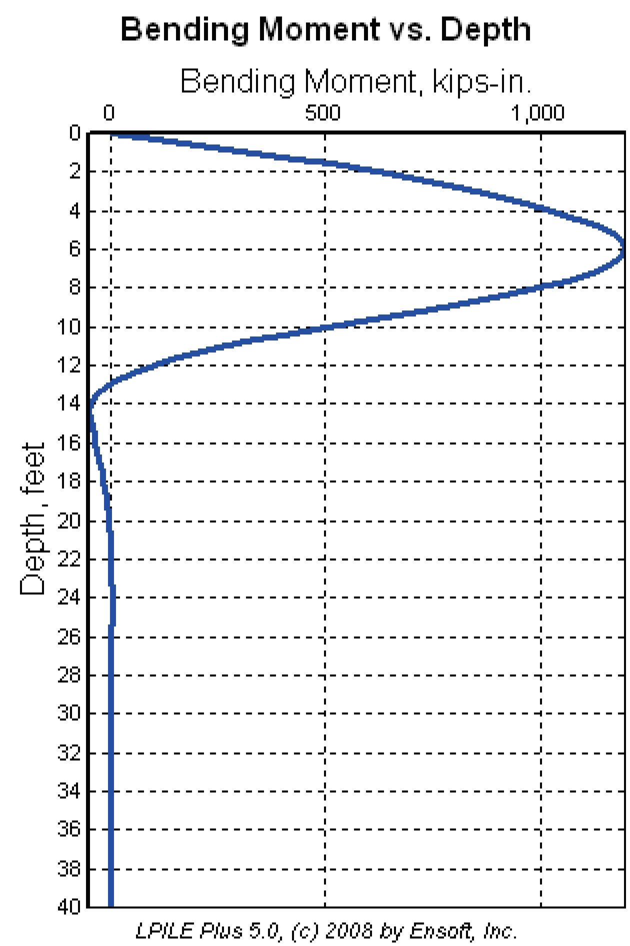
24-inch Fixed Head CIDH Pile

San Joaquin Residence Apartments & Precinct Improvements

Santa Barbara, California







Loading Conditions:

Axial Load = 350 Kips

Pile Head Deflection = 0.25”

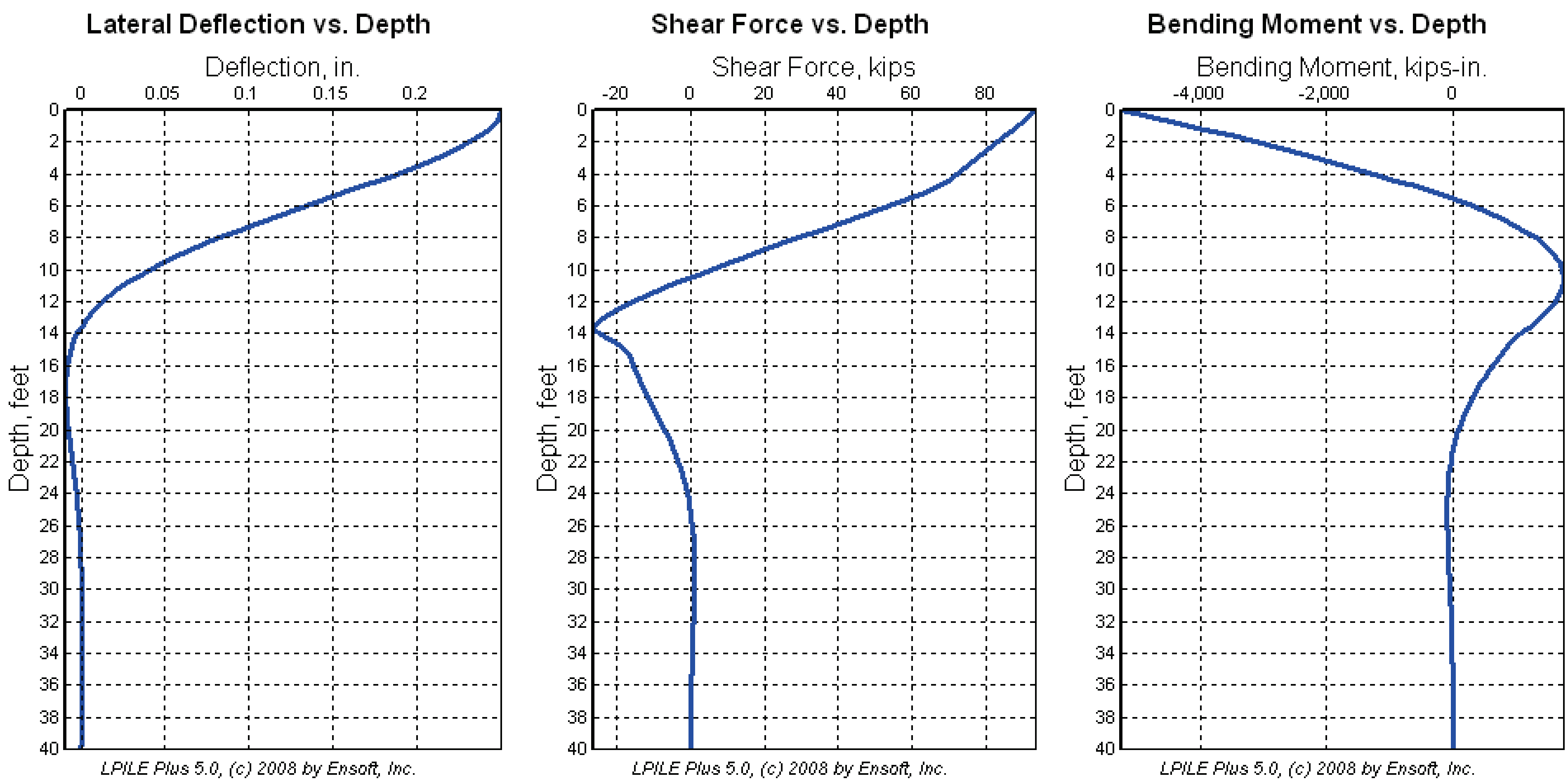
Pile Head Moment = 0 Kip-inch

Lateral Capacity Results

24-inch Free Head CIDH Pile

San Joaquin Residence Apartments & Precinct Improvements

Santa Barbara, California



Loading Conditions:

Axial Load = 350 Kips

Pile Head Deflection = 0.25”

Pile Head Slope = 0

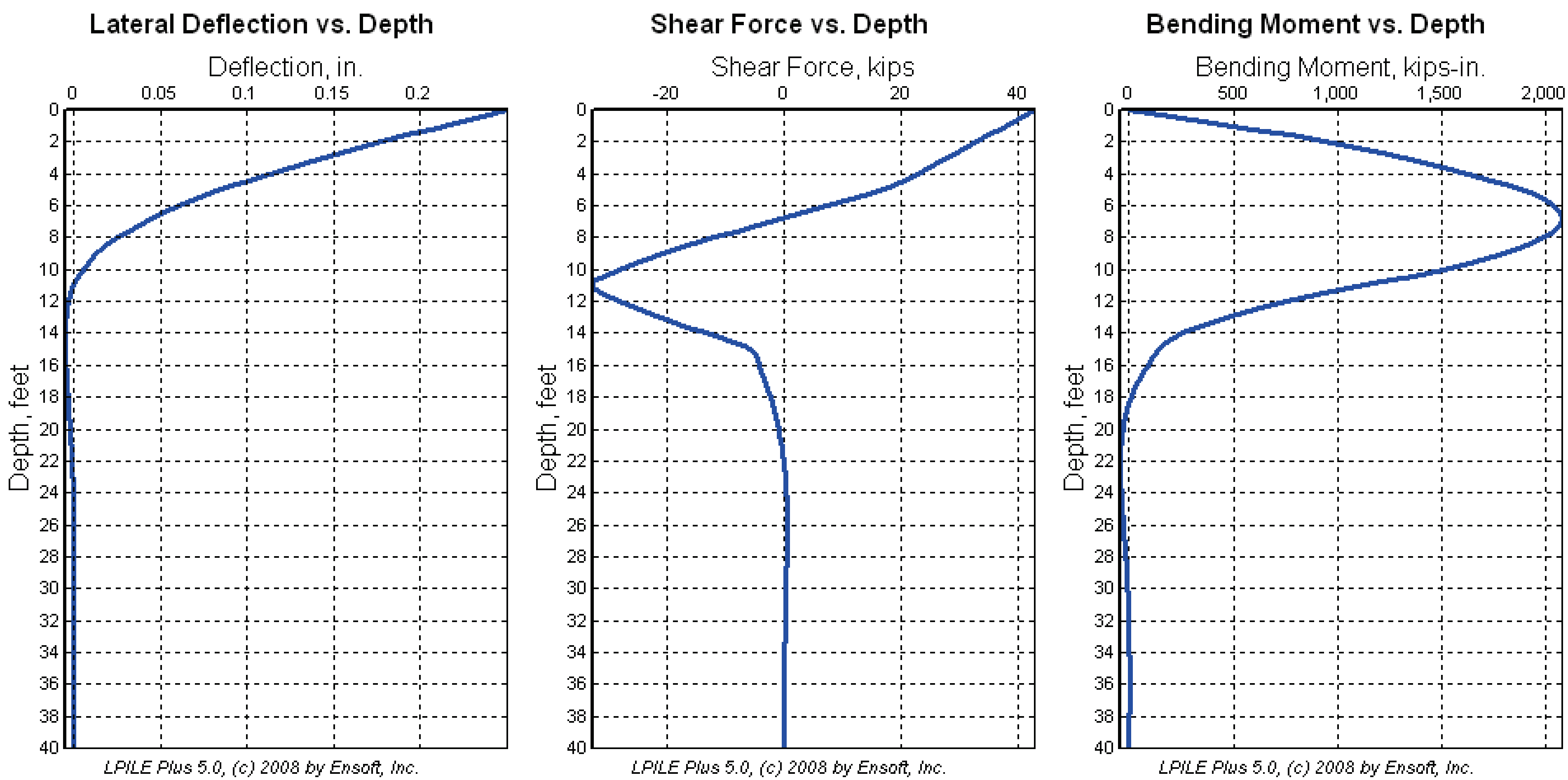
Lateral Capacity Results

30-inch Fixed Head CIDH Pile

San Joaquin Residence Apartments & Precinct Improvements

Santa Barbara, California

PLATE C-2c



Loading Conditions:

Axial Load = 350 Kips

Pile Head Deflection = 0.25”

Pile Head Moment = 0 Kip-inch

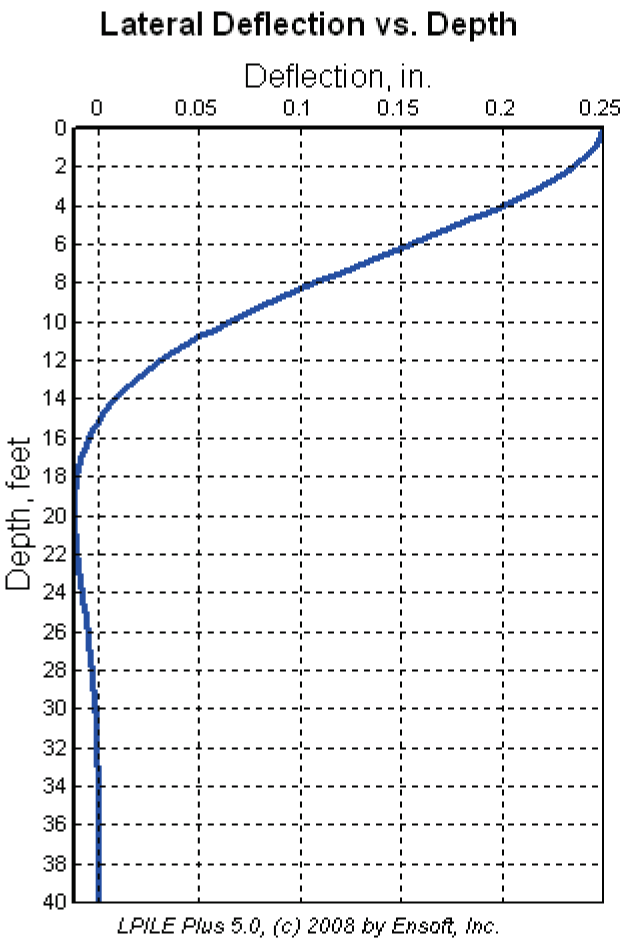
Lateral Capacity Results

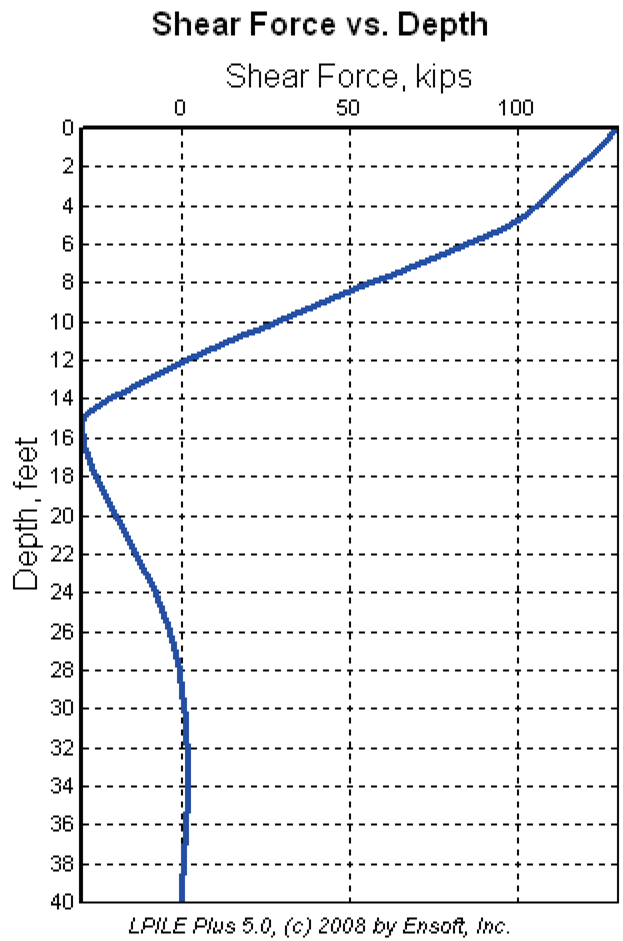
30-inch Free Head CIDH Pile

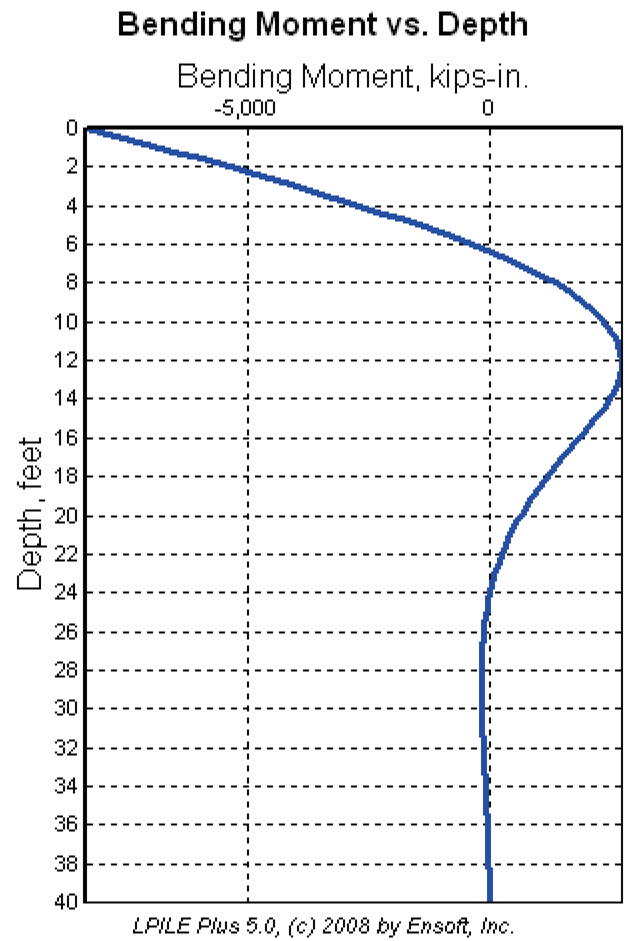
San Joaquin Residence Apartments & Precinct Improvements

Santa Barbara, California

PLATE C-2d







Loading Conditions:

Axial Load = 350 Kips

Pile Head Deflection = 0.25”

Pile Head Slope = 0

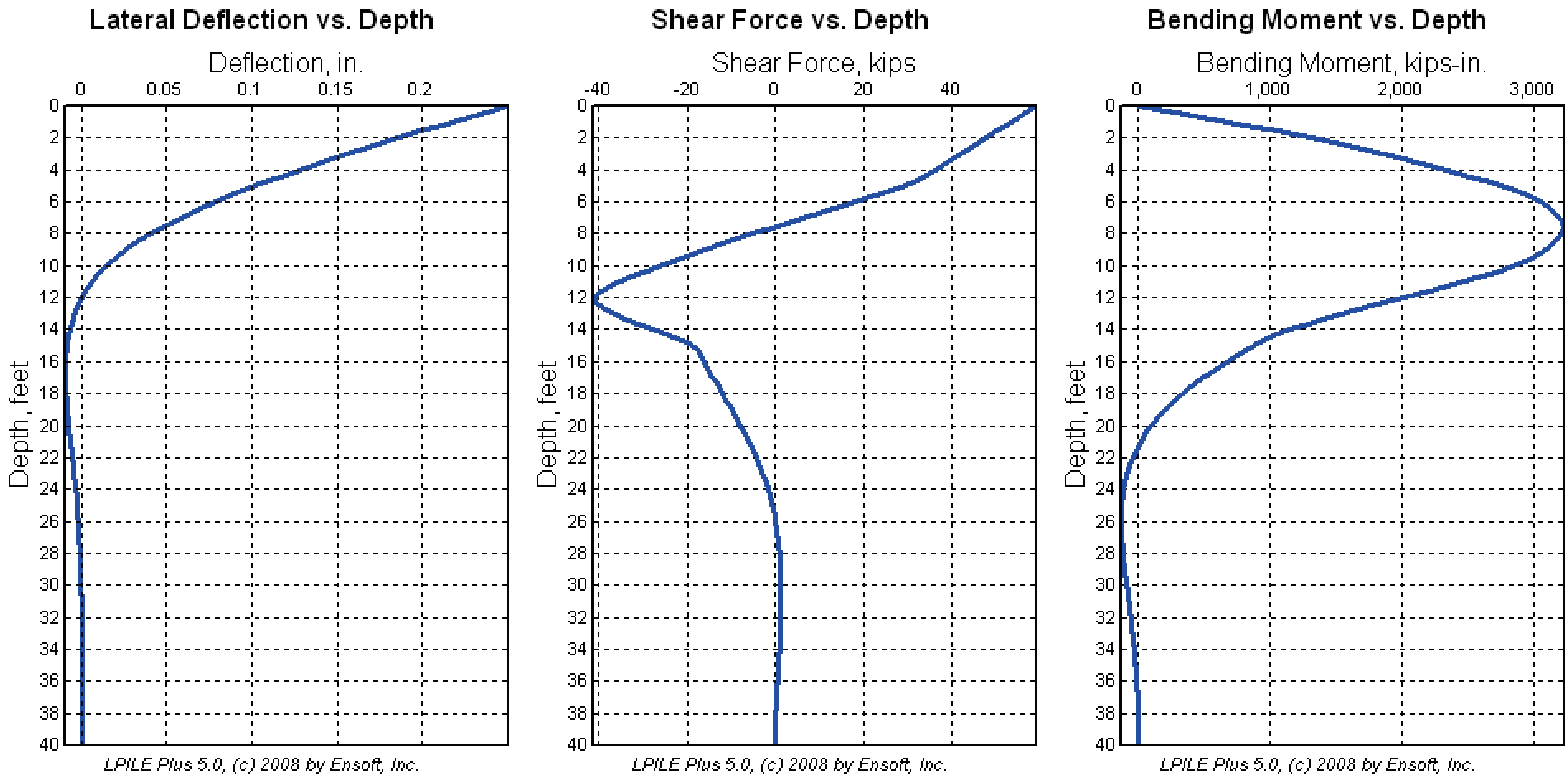
Lateral Capacity Results

36-inch Fixed Head CIDH Pile

San Joaquin Residence Apartments & Precinct Improvements

Santa Barbara, California

PLATE C-2e



Loading Conditions:

Axial Load = 350 Kips

Pile Head Deflection = 0.25"

Pile Head Moment = 0 Kip-inch

Lateral Capacity Results

36-inch Free Head CIDH Pile

San Joaquin Residence Apartments & Precinct Improvements

Santa Barbara, California

APPENDIX D – SEISMIC CONE PENETRATION TEST DATA FROM KEHOE TESTING

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

UCSB Santa Catalina Student Housing Facility
Storke Road & El Colegio Road
Goleta, CA
June 21, 2013

Prepared for:

Mr. Greg Denlinger
Fugro West, Inc.
4820 McGrath Street, Ste 100
Ventura, CA 93003-7778
Office (805) 650-7000 / Fax (805) 650-7010

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the UCSB Santa Catalina Student Housing Facility project located at Storke Road & El Colegio Road in Goleta, California. The work was performed by Kehoe Testing & Engineering (KTE) on June 21, 2013. The scope of work was performed as directed by Fugro West, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at three locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warrant the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	86	Refusal, hole open to 9 ft (dry)
CPT-2	29	Refusal, hole open to 4 ft (dry)
CPT-3	41	Refusal, no cave depth taken

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At location CPT-1, CPT-2 & CPT-3, shear wave measurements were obtained at approximately 5-foot intervals. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the attached CPT Classification Chart (Robertson) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Tables of basic CPT output from the interpretation program CPeT-IT are provided for CPT data averaged over one foot intervals in the Appendix. Spreadsheet files of the averaged basic CPT output and averaged estimated geotechnical parameters are also included for use in further geotechnical analysis. We recommend a geotechnical engineer review the assumed input parameters and the calculated output from the CPeT-IT program. A summary of the equations used for the tabulated parameters is provided in the Appendix.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

06/28/13-ds-1874-7

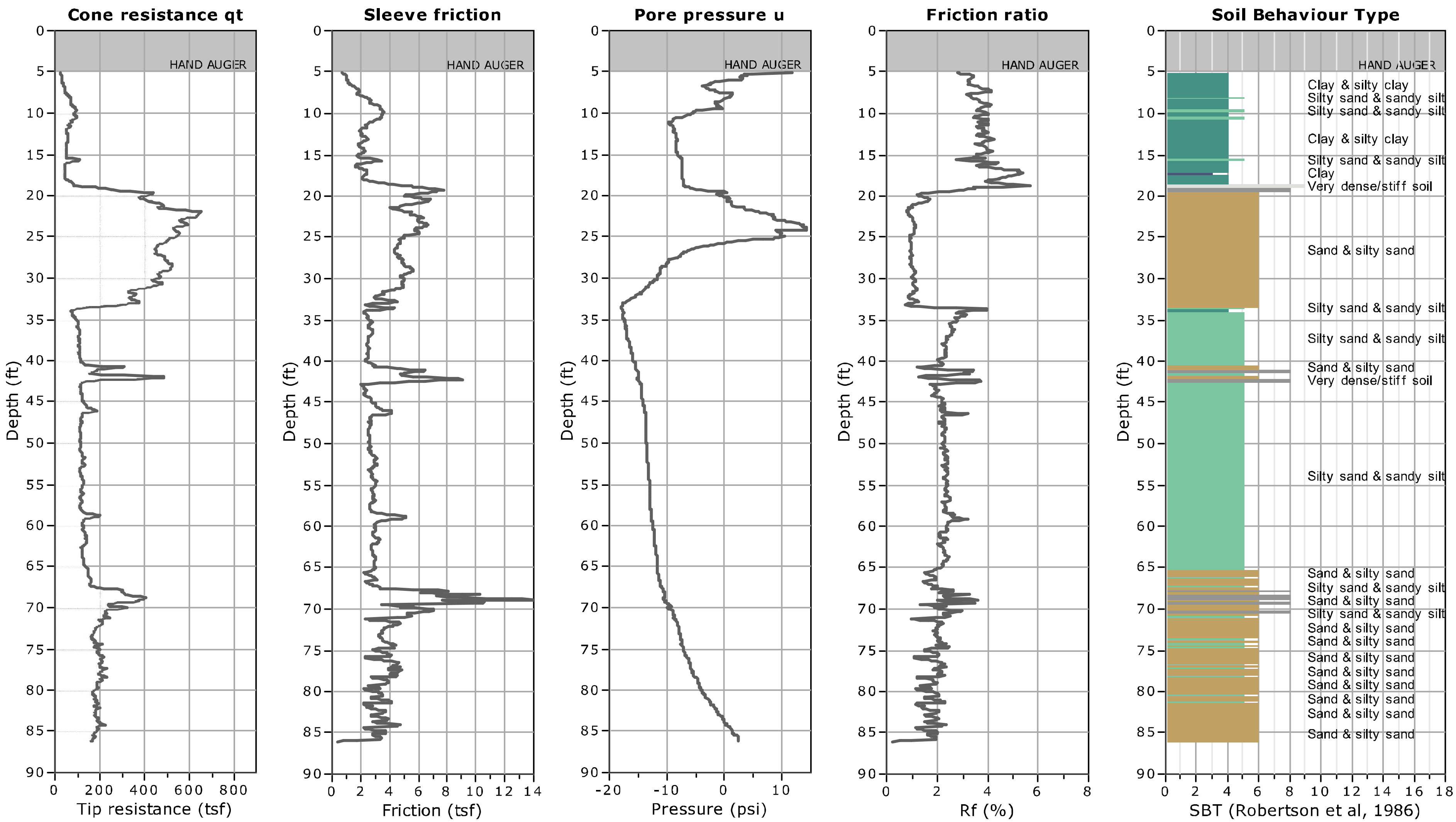
APPENDIX

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: CPT-1

Total depth: 86.23 ft, Date: 6/21/2013
Cone Type: Vertek

Project: **Fugro West/UCSB Santa Caltalina Student Housing Facility**
Location: **Storke Rd. & El Colegio Rd. Goleta, CA**



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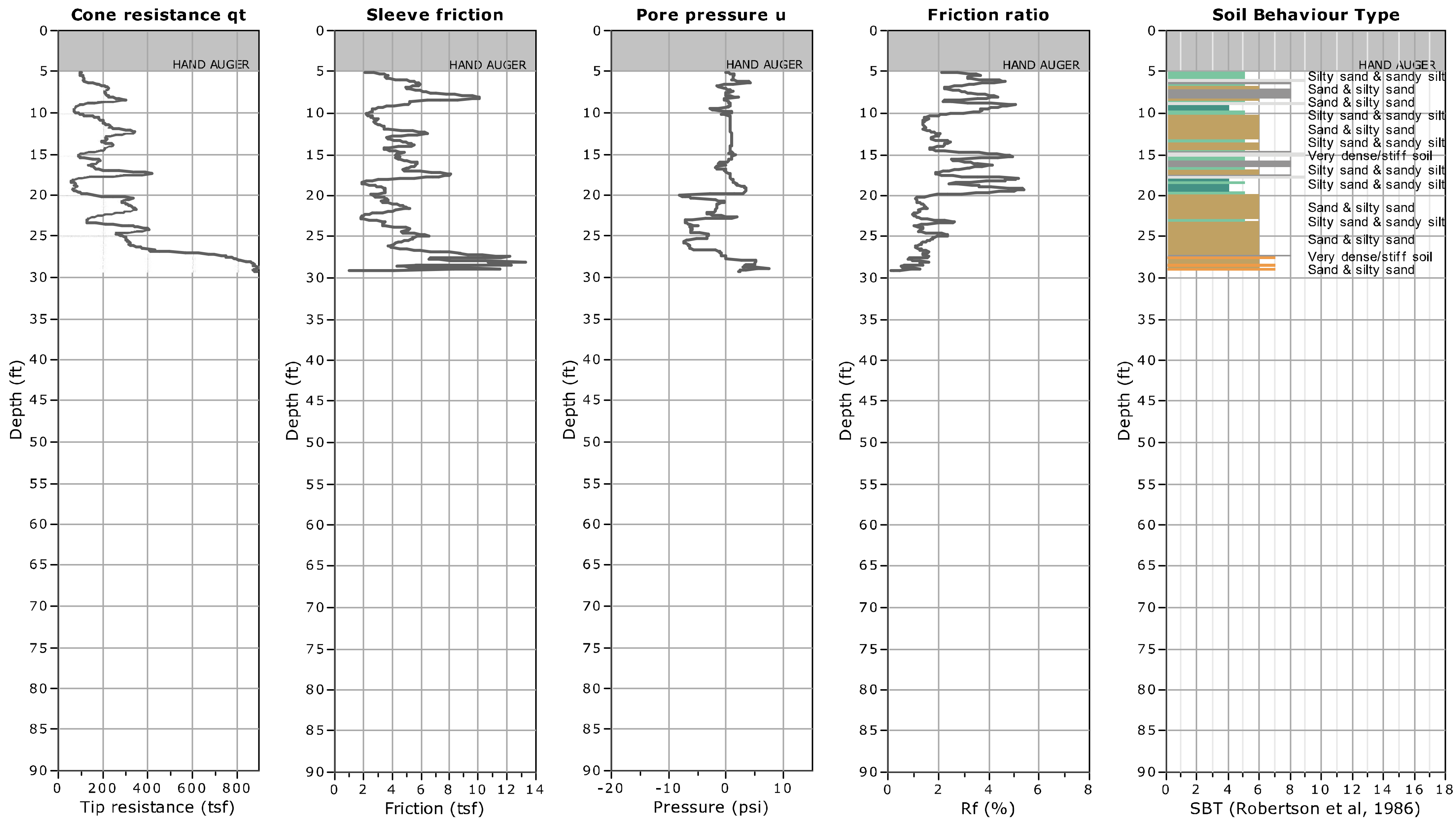
CPT: CPT-2

Total depth: 29.35 ft, Date: 6/21/2013
Cone Type: Vertek

Project: **Fugro West/UCSB Santa Caltalina Student Housing Facility**
Location: **Storke Rd. & El Colegio Rd. Goleta, CA**



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rich@kehoetesting.com
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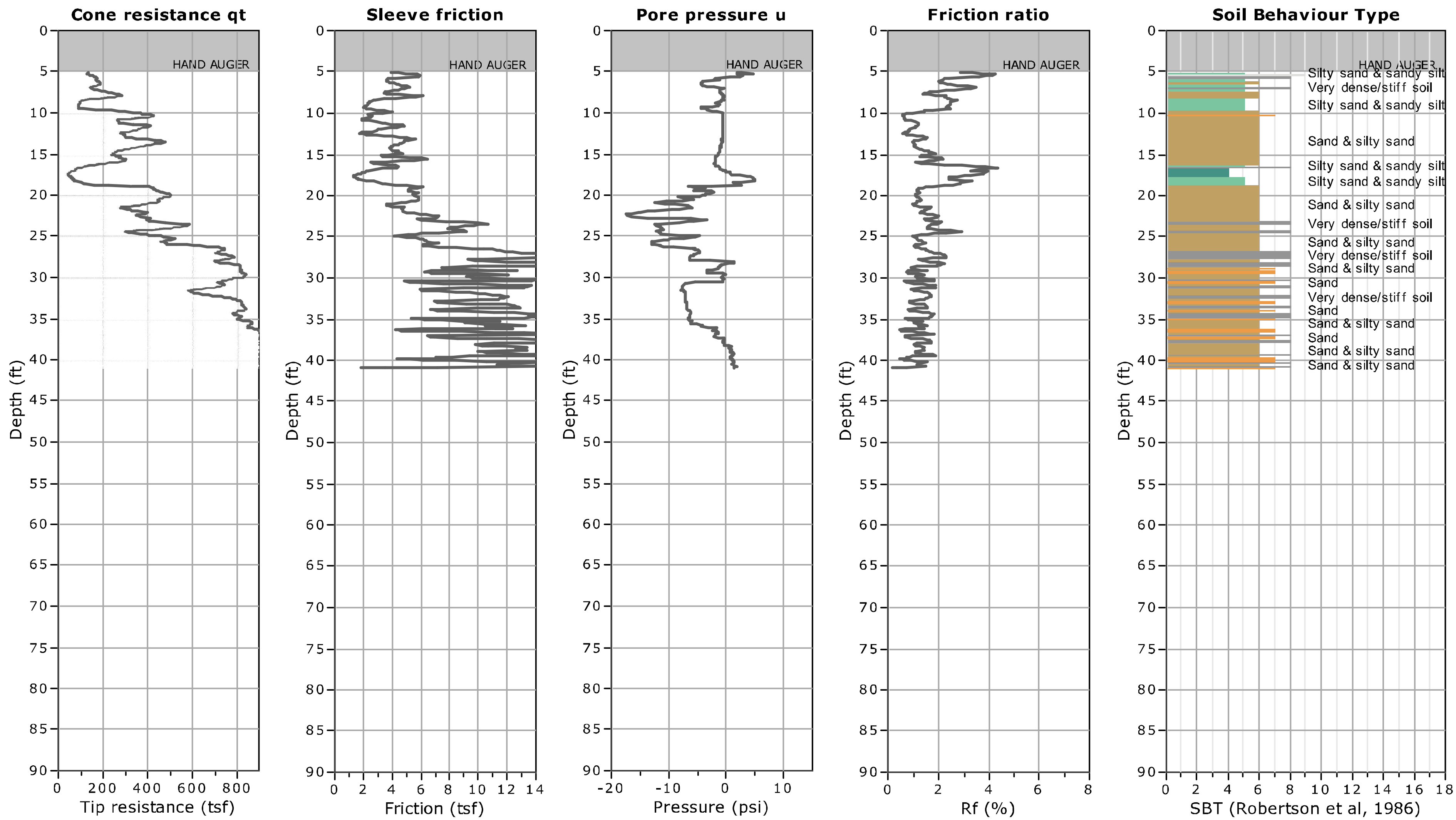
CPT: CPT-3
Total depth: 41.09 ft, Date: 6/21/2013
Cone Type: Vertek

Project: **Fugro West/UCSB Santa Caltalina Student Housing Facility**
Location: **Storke Rd. & El Colegio Rd. Goleta, CA**



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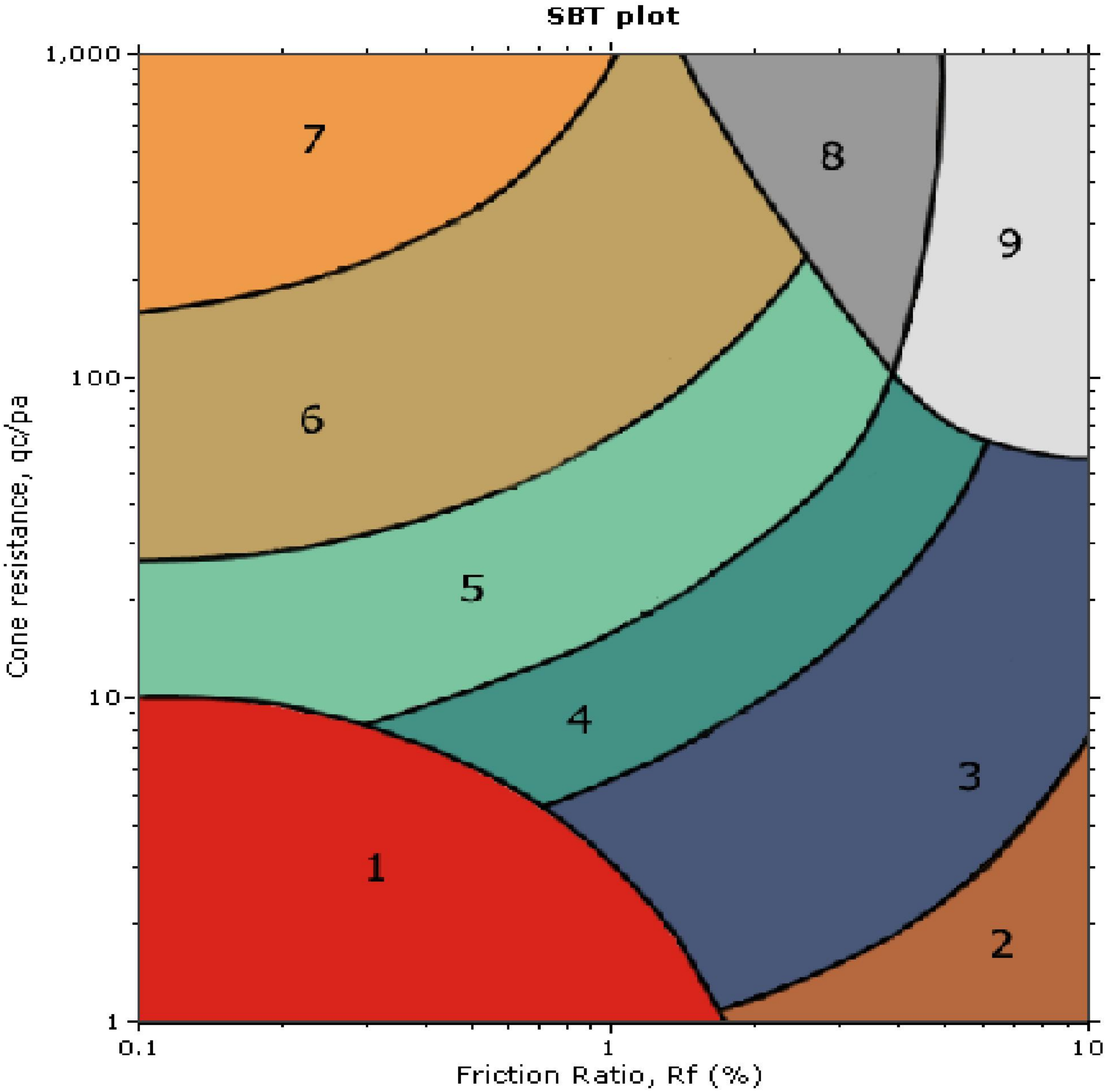

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

	CPT-1	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	34.2	0.65	-3.84	1.91	34.153	1.9032	5	2.46868	115.7311	0.05787	0	0.0579	589.21	1.9064	-0.008	5	0.7054	2	2.2381	64.44547
2	23.8	0.27	11.72	1.11	23.9435	1.1277	5	2.46842	108.4365	0.11208	0	0.1121	212.62	1.133	0.0354	5	0.7006	2	2.2187	45.04525
3	9.7	0.06	0	0.57	9.7	0.6186	4	2.70402	95.22736	0.1597	0	0.1597	59.74	0.6289	0	5	0.7861	2	2.437	18.03276
4	12.5	0.29	0	2.3	12.5	2.32	4	2.87443	107.3741	0.21338	0	0.2134	57.58	2.3603	0	4	0.8656	2	2.6391	23.22375
5	23.4	0.65	11.88	2.78	23.5454	2.7606	4	2.69527	114.824	0.2708	0	0.2708	85.949	2.7927	0.0368	4	0.8047	2	2.4723	43.99289
6	32.8	1.06	1.08	3.23	32.8132	3.2304	4	2.62768	119.2118	0.3304	0	0.3304	98.313	3.2633	0.0024	4	0.7859	2	2.4154	61.39792
7	45.3	1.8	-2.84	3.99	45.2652	3.9766	4	2.58683	123.871	0.39234	0	0.3923	114.37	4.0113	-0.005	4	0.7782	2	2.3876	84.81724
8	66.5	2.23	0.9	3.35	66.511	3.3528	4	2.41678	126.377	0.45553	0	0.4555	145.01	3.376	0.001	5	0.7277	1.8465	2.245	115.2705
9	67.5	3.08	-1.08	4.57	67.4868	4.5639	4	2.51101	128.7754	0.51991	0	0.5199	128.8	4.5993	-0.001	9	0.7751	1.7346	2.3639	109.7825
10	86.6	3.39	-5.44	3.92	86.5334	3.9176	4	2.39005	130.0834	0.58496	0.0312	0.5538	155.21	3.9442	-0.005	8	0.7388	1.6135	2.2643	131.0587
11	76.8	2.91	-9.13	3.79	76.6883	3.7946	4	2.41422	128.6717	0.64929	0.0624	0.5869	129.56	3.827	-0.009	8	0.7523	1.558	2.2955	111.9605
12	51.8	2	-8.75	3.87	51.6929	3.869	4	2.53766	124.9657	0.71177	0.0936	0.6182	82.47	3.923	-0.014	4	0.8007	1.5378	2.4187	74.09236
13	59.5	2.42	-8.29	4.08	59.3985	4.0742	4	2.51198	126.6994	0.77512	0.1248	0.6503	90.145	4.128	-0.012	4	0.798	1.4747	2.4076	81.70305
14	49.5	1.96	-8.29	3.96	49.3985	3.9677	4	2.5593	124.7072	0.83748	0.156	0.6815	71.258	4.0362	-0.016	4	0.8207	1.4349	2.462	65.85212
15	49.2	1.63	-8.15	3.32	49.1002	3.3197	4	2.50676	123.3434	0.89915	0.1872	0.712	67.703	3.3817	-0.016	4	0.8062	1.3763	2.4202	62.69788
16	52.9	2.32	-7.52	4.4	52.808	4.3933	4	2.57087	126.1038	0.9622	0.2184	0.7438	69.704	4.4748	-0.015	4	0.8362	1.3428	2.4957	65.7945
17	46.1	2.38	-7.37	5.19	46.0098	5.1728	4	2.66352	125.9545	1.02518	0.2496	0.7756	58.001	5.2907	-0.017	4	0.8764	1.3129	2.5975	55.81717
18	52.9	2.22	-7.21	4.2	52.8118	4.2036	4	2.55699	125.7816	1.08807	0.2808	0.8073	64.072	4.292	-0.015	4	0.8417	1.2558	2.5023	61.38515
19	178.2	6.16	-6.34	3.46	178.122	3.4583	8	2.15422	136.2143	1.15618	0.312	0.8442	209.63	3.4809	-0.004	8	0.6975	1.1706	2.1194	195.7868
20	390.5	5.06	-0.47	1.3	390.494	1.2958	6	1.60963	136.6894	1.22452	0.3432	0.8813	441.69	1.2999	-1E-03	6	0.4975	1.0952	1.5897	402.918
21	479.8	5.62	1.06	1.17	479.813	1.1713	6	1.52393	137.28	1.29316	0.3744	0.9188	520.83	1.1745	-6E-04	6	0.469	1.0685	1.5104	483.2028
22	652.9	5.48	5.99	0.84	652.973	0.8392	6	1.33055	137.28	1.3618	0.4056	0.9562	681.46	0.841	4E-05	6	0.3993	1.0413	1.3229	641.2366
23	551.4	5.87	10.27	1.06	551.526	1.0643	6	1.45676	137.28	1.43044	0.4368	0.9936	553.62	1.0671	0.0006	6	0.4502	1.0287	1.452	534.8081
24	532.9	5.76	14.09	1.08	533.072	1.0805	6	1.47005	137.28	1.49908	0.468	1.0311	515.55	1.0836	0.001	6	0.4584	1.0119	1.469	508.3756
25	535	5.1	10.06	0.95	535.123	0.9531	6	1.42294	137.28	1.56772	0.4992	1.0685	499.34	0.9559	0.0004	6	0.4437	0.9957	1.4257	502.0679
26	461.2	4.5	-2.45	0.98	461.17	0.9758	6	1.46715	136.2369	1.63584	0.5304	1.1054	415.7	0.9793	-0.002	6	0.464	0.9799	1.4743	425.5693
27	449.4	4.38	-7.2	0.97	449.312	0.9748	6	1.47324	135.9756	1.70383	0.5616	1.1422	391.87	0.9785	-0.002	6	0.4696	0.9647	1.4845	408.0999
28	505.1	4.82	-9.67	0.95	504.982	0.9545	6	1.4372	136.9609	1.77231	0.5928	1.1795	426.63	0.9579	-0.003	6	0.4588	0.9514	1.4513	452.4586
29	512.3	5.69	-11.12	1.11	512.164	1.111	6	1.48944	137.28	1.84095	0.624	1.217	419.35	1.115	-0.003	6	0.4818	0.9348	1.5072	450.8655
30	465.3	4.95	-11.82	1.06	465.155	1.0642	6	1.49628	136.9553	1.90943	0.6552	1.2542	369.35	1.0686	-0.003	6	0.4881	0.9204	1.519	402.9353
31	424.5	4.9	-13.25	1.16	424.338	1.1547	6	1.54795	136.657	1.97775	0.6864	1.2914	327.07	1.1602	-0.004	6	0.5117	0.9031	1.576	360.4849
32	361.2	3.39	-15.04	0.94	361.016	0.939	6	1.51672	133.5672	2.04454	0.7176	1.3269	270.53	0.9444	-0.005	6	0.5038	0.8922	1.5513	302.692
33	357.2	3.34	-17.65	0.93	356.984	0.9356	6	1.5185	133.4311	2.11125	0.7488	1.3625	260.47	0.9412	-0.006	6	0.5077	0.8795	1.5573	294.9836
34	71.1	2.22	-17.77	3.13	70.8825	3.1319	5	2.37628	126.4993	2.1745	0.78	1.3945	49.271	3.2311	-0.03	4	0.8562	0.7895	2.4676	51.26606
35	94.3	2.63	-17.53	2.8	94.0854	2.7953	5	2.25621	128.4301	2.23872	0.8112	1.4275	64.34	2.8635	-0.023	5	0.8105	0.7845	2.3434	68.09692
36	101.2	2.65	-17.17	2.62	100.99	2.624	5	2.21519	128.6582	2.30305	0.8424	1.4607	67.564	2.6853	-0.021	5	0.7979	0.7732	2.306	72.11297
37	106.4	2.42	-16.94	2.28	106.193	2.2789	5	2.15534	128.1164	2.36711	0.8736	1.4935	69.518	2.3308	-0.02	5	0.7782	0.7647	2.2502	75.03944
38	106.9	2.24	-16.58	2.1	106.697	2.0994	5	2.12799	127.5624	2.43089	0.9048	1.5261	68.323	2.1484	-0.02	5	0.7716	0.7538	2.2285	74.2837
39	107	2.49	-16.21	2.33	106.802	2.3314	5	2.16092	128.339	2.49506	0.936	1.5591	66.904	2.3872	-0.02	5	0.788	0.7368	2.2675	72.63269
40	115.3	2.38	-15.96	2.07	115.105	2.0677	5	2.10058	128.191	2.55915	0.9672	1.592	70.697	2.1147	-0.019	5	0.7671	0.731	2.2095	77.7534
41	237.2	5.1	-15.5	2.15	237.01	2.1518	6	1.91444	135.5292	2.62692	0.9984	1.6285	143.92	2.1759	-0.009	5	0.6888	0.7431	1.9994	164.5953
42	484.5	7.12	-15.11	1.47	484.315	1.4701	6	1.6062	137.28	2.69556	1.0296	1.666	289.09	1.4784	-0.004	6	0.5644	0.774	1.6684	352.2967
43	115.1	2.12	-14.58	1.85	114.922	1.8447	5	2.06512	127.3407	2.75923	1.0608	1.6984	66.039	1.8901	-0.019	5	0.7655	0.6961	2.1922	73.78784
44	119.2	2.16	-14.43	1.82	119.023	1.8148	5	2.04944	127.563	2.82301	1.092	1.731	67.129	1.8589	-0.018	5	0.7625	0.6871	2.1802	75.45309
45	126.8	2.63	-14.27	2.08	126.625	2.077	5	2.07398	129.1545	2.88759	1.1232	1.7644	70.131	2.1255	-0.017	5	0.7744	0.673	2.2073	78.70599
46	173.6	4.06	-13.97	2.34	173.429	2.341	5	2.0254	133.0987	2.95413	1.1544	1.7997	94.722	2.3816	-0.013	5	0.7535	0.6702	2.148	107.9732
47	117.5	2.65	-13.81	2.26	117.331	2.2586	5	2.12318	129.024	3.01865	1.1856	1.8331	62.362	2.3182	-0.019	5	0.8024	0.6435	2.2722	69.51554
48	112.1	2.59	-13.81	2.31	111.931	2.3139	5	2.14473	128.7415	3.08302	1.2168	1.8662	58.325	2.3795	-0.02	5	0.8154	0.6296	2.3021	64.76804
49	115.5	2.6	-13.66	2.25	115.333	2.2544	5	2.1276	128.8428	3.14744	1.248	1.8994	59.062	2.3176	-0.02	5	0.8118	0.6219	2.2886	65.93843
50	117.6	2.61	-13.66	2.22	117.433	2.2226	5	2.11776	128.9149	3.2119	1.2792	1.9327	59.099	2.2851	-0.02	5	0.8112	0.6134	2.283	66.21731
51	114.5	2.51	-13.51	2.2	114.335	2.1953	5	2.12164	128.5638	3.27618	1.3104	1.9658	56.496	2.2601	-0.021	5	0.817	0.6029	2.294	63.2773
52	125	2.97	-13.35	2.38	124.837	2.3791	5	2.12214	130.0094	3.34118	1.3416	1.9996	60.76	2.4445	-0.019	5	0.8187	0.5939	2.2943	68.19313
53	113	2.71	-13.2	2.4	112.838	2.4017	5	2.15435	129.0926	3.40573	1.3728	2.0329	53.83	2.4764	-0.021	5	0.8372	0.5789	2.3388	59.86693
54	122.6	2.67	-13.2	2.18	122.438	2.1807	5	2.09947	129.183	3.47032	1.404	2.0663	57.575	2.2443	-0.02	5	0.8176	0.5786	2.2832	65.04869
55	126.3	3.04	-13.05	2.41	126.14	2.41	5	2.12339	130.2052	3.53542	1.4352	2.1002	58.377	2.4795	-0.019	5	0.8297	0.5662	2.3106	65.6085
56	121.4	2.89	-13.05	2.38	121.24	2.3837	5	2.13115	129.7383	3.60029	1.4664	2.1339	55.129	2.4567	-0.02	5	0.8373	0.5558	2.3265	61.79396
57	125.6	2.78	-13.03	2.22	125.441	2.2162	5	2.09767	129.5375	3.66506	1.4976	2.1675	56.183	2.2829	-0.02	5	0.8271	0.5526	2.2954	63.60033
58	115.4	2.7	-12.89	2.34	115.242	2.3429	5	2.14023	129.117	3.72962	1.5288	2.2008	50.669	2.4213	-0.022	5	0.8494	0.5368	2.35	56.57566
59	168.9	4.93	-12.69	2.92	168.745	2.9216	5	2.10887	134.4526	3.79685	1.56	2.2369	73.741	2.9888	-0.015	5	0.8303	0.5371	2.2953	83.73051
60	126.1	2.98	-12.43	2.37	125.948	2.3661	5	2.11781	130.0556	3.86187	1.5912	2.2707	53.766	2.4409	-0.02	5	0.8456	0.5243	2.3312	60.49437
61	134.7	2.76	-12.28	2.05	134.55	2.0513	5	2.05234	129.6556	3.9267	1.6224	2.3043	56.687	2.113	-0.019	5	0.822	0.5274	2.265	65.11053
62	142.8	3.12	-12.12	2.19	142.652	2.1872	5	2.05657	130.6953	3.99205	1.6536	2.3385	59.296	2.2501	-0.018	5	0.8254	0.5197	2.2696	68.10374
63	121.2	2.79	-11.97	2.3	121.053	2.3048	5	2.12066	129.4769	4.05679	1.6848	2.372	49.324	2.3847	-0.022	5	0.8586	0.5	2.3528	55.2875
64	126.5	3.02	-11.82	2.39	126.355	2.3901	5	2.12019	130.1611	4.12187	1.716	2.4059	50.806	2.4707	-0.021	5	0.8606	0.4932	2.3538	56.97065
65	138.4	2.99	-11.66	2.16	138.257	2.1626	5	2.06176	130.3076	4.18702	1.7472	2.4398	54.951	2.2302	-0.019	5	0.8387	0.4962	2.2921	62.87669
66	148.3	2.76	-11.36	1.86	148.161	1.8628	6	1.99325	129.8906	4.25197	1.7784	2.4736	58.179	1.9179	-0.018	5	0.8136	0.5011	2.2218	68.15682
67	151.1	2.72	-11.05	1.8	150.965	1.8018	6	1.97702	129.8295	4.31688	1.8096	2.5073	58.489	1.8548	-0.018	5	0.8102	0.4971	2.2086	68.89792
68	289.4	3.83	-10.78	1.32	289.268	1.324	6	1.69359	133.9198	4.38384	1.8408	2.543	112.02	1.3444	-0.009	6	0.6842	0.5488	1.8734	147.7628
69	372.2	5.14	-10.69	1.38	372.069	1.3815	6	1.64442	136.6863	4.45218	1.872	2.5802	142.48	1.3982	-0.007	6	0.6611	0.5547	1.8081	192.7282
70	319	6.92	-9.98	2.17	318.878	2.1701	6	1.84514	137.28	4.52082	1.9032	2.6176	120.09	2.2013	-0.008	5	0.7461	0.5088	2.0267	151.1477
71	218.6	5.19	-8.75	2.37	218.493	2.3754	6	1.96992	135.4588	4.58855	1.9344	2.6542	80.592	2.4263	-0.012	5	0.8094	0.4751	2.1883	96.03617
72	206.9	4.18	-8.13	2.02	206.8	2.0213	6	1.92809	133.7411	4.65542	1.9656	2.6898	75.152	2.0678	-0.013	5	0.7985	0.4747	2.1553	90.69586
73	177.6	3.57	-7.98	2.01	177.502	2.0112	6	1.96768	132.2142	4.72153	1.9968	2.7247	63.412	2.0662	-0.015	5	0.8224	0.4594	2.2137	75.01202
74	179.2	3.67	-7.52	2.05	179.108	2.049	6	1.97149	132.4383	4.78775	2.028	2.7598	63.165	2.1053	-0.015	5	0.8269	0.4526	2.2212	74.56578
75	199.8	2.79	-7.21	1.39	199.712	1.397	6	1.81376	130.698	4.8531	2.0592	2.7939	69.744	1.4318	-0.013	5	0.7657	0.4755	2.0559	87.5611
76	216.7	2.27	-6.6	1.05	216.619	1.0479	6	1.69744	129.387	4.91779	2.0904	2.8274	74.875	1.0723	-0.012	6	0.7213	0.4922	1.9349	98.47135
77	203.2	4.17	-5.99	2.05	203.127	2.0529	6	1.93819	133.6798	4.98463	2.1216	2.863	69.207	2.1046	-0.013	5	0.8193	0.4424	2.1884	82.84129
78	193.9	3.94	-5.37	2.03	193.834	2.0327	6	1.94732	133.1505	5.05121	2.1528	2.8984	65.133	2.0871	-0.013	5	0.828	0.4342	2.2067	77.46207
79	189.7	3.89	-4.6	2.05	189.644	2.0512	6	1.95631	133.0037	5.11771	2.184	2.9337	62.899	2.1081	-0.014	5	0.8356	0.4265	2.2224	74.37757
80	176	2.99	-4.14	1.7	175.949	1.6994	6	1.91407	130.8956	5.18316	2.2152	2.968	57.537	1.7509	-0.015	5	0.8257	0.4267	2.1919	68.8691
81	167.1	3.41	-2.92	2.04	167.064	2.0411	6	1.98937	131.7309	5.24902	2.2464	3.0026	53.891	2.1073	-0.015	5	0.86	0.4078	2.2779	62.36181
82	178.7	2.52	-1.84	1.41	178.677	1.4104	6	1.84899	129.6818	5.31386	2.2776	3.0363	57.098	1.4536	-0.014	5	0.8067	0.4273	2.1334	70.00228
83	178.3	2.83	-1.07	1.59	178.287	1.5873	6	1.88795	130.5253	5.37913	2.3088	3.0703	56.316	1.6367	-0.014	5	0.825	0.4153	2.1773	67.85717
84	194	4.69	0.23	2.42	194.003	2.4175	5	2.00673	134.4276	5.44634	2.34	3.1063	60.701	2.4873	-0.012	5	0.8704	0.3916	2.2924	69.79056
85	184.6	3.72	1.38	2.02	184.617	2.015	6	1.95756	132.6112	5.51265	2.3712	3.1415	57.013	2.077	-0.013	5	0.857	0.3935	2.2527	66.61307
86	163.9	0	2.46	0	163.93	0	0	0	769.6	5.89745	2.4024	3.4951	45.216	0	-0.014	0	1	0.3028	0	0

	CPT-2	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	46.2	1.1	0.68	2.38	46.2083	2.3805	5	2.42849	120.3178	0.06016	0	0.0602	767.1	2.3836	0.0011	5	0.6954	2	2.2114	87.2277
2	32.4	0.17	0.48	0.54	32.4059	0.5246	5	2.19526	105.7896	0.11305	0	0.1131	285.64	0.5264	0.0011	6	0.5905	2	1.9296	61.0388
3	39.9	0.83	0.16	2.09	39.902	2.0801	5	2.4395	117.8992	0.172	0	0.172	230.98	2.0891	0.0003	5	0.7027	2	2.2167	75.09622
4	48.7	0.6	0.16	1.24	48.702	1.232	5	2.23224	116.0109	0.23001	0	0.23	210.74	1.2378	0.0002	6	0.6226	2	1.9993	91.62004
5	102.7	2.04	0	1.99	102.7	1.9864	5	2.12219	126.785	0.2934	0	0.2934	349.03	1.9921	0	6	0.5975	2	1.9257	193.5655
6	111.7	4.85	0.96	4.34	111.712	4.3415	9	2.35436	133.3269	0.36006	0	0.3601	309.25	4.3556	0.0006	8	0.6992	2	2.1843	210.4732
7	223.6	4.93	-0.25	2.21	223.597	2.2049	6	1.93787	135.139	0.42763	0	0.4276	521.87	2.2091	-8E-05	8	0.5624	1.6645	1.8171	351.0659
8	228.1	9.93	1.29	4.35	228.116	4.3531	8	2.17864	137.28	0.49627	0	0.4963	458.66	4.3625	0.0004	8	0.6638	1.6529	2.0744	355.5762
9	93.8	4.7	0.64	5.01	93.8078	5.0102	9	2.45015	132.671	0.56261	0	0.5626	165.74	5.0405	0.0005	9	0.764	1.6203	2.3294	142.786
10	84.7	2.23	0.22	2.64	84.7027	2.6327	5	2.26832	126.9667	0.62609	0.0312	0.5949	141.33	2.6523	-2E-04	5	0.6994	1.4959	2.1558	118.8656
11	198.5	2.83	0.64	1.43	198.508	1.4256	6	1.82212	130.7874	0.69149	0.0624	0.6291	314.45	1.4306	-8E-05	6	0.5449	1.3275	1.7459	248.1815
12	274.9	3.82	0.64	1.39	274.908	1.3896	6	1.72371	133.7764	0.75837	0.0936	0.6648	412.39	1.3934	-2E-04	6	0.5153	1.2706	1.6637	329.2036
13	208.6	3.49	0.8	1.67	208.61	1.673	6	1.86138	132.4423	0.8246	0.1248	0.6998	296.92	1.6796	-3E-04	6	0.5692	1.2653	1.801	248.4811
14	238.7	5.11	0.8	2.14	238.71	2.1407	6	1.91079	135.561	0.89238	0.156	0.7364	322.96	2.1487	-4E-04	6	0.5934	1.24	1.86	278.7014
15	95.2	4.1	-0.06	4.31	95.1993	4.3068	9	2.39496	131.7075	0.95823	0.1872	0.771	122.23	4.3506	-0.002	9	0.7763	1.2785	2.335	113.8731
16	174.1	5.65	-0.77	3.24	174.091	3.2454	8	2.13757	135.5261	1.02599	0.2184	0.8076	214.3	3.2647	-0.002	8	0.6865	1.2038	2.0949	196.8941
17	218.2	5.12	-0.32	2.35	218.196	2.3465	6	1.96597	135.3561	1.09367	0.2496	0.8441	257.21	2.3583	-0.001	6	0.6267	1.1522	1.9335	236.3999
18	78.1	3.92	0.75	5.01	78.1092	5.0186	9	2.5009	130.8965	1.15912	0.2808	0.8783	87.611	5.0942	-0.003	9	0.8317	1.1675	2.4675	84.90692
19	80.6	3.21	2.98	3.97	80.6365	3.9808	4	2.41538	129.512	1.22388	0.312	0.9119	87.087	4.0422	-0.001	4	0.8041	1.127	2.3909	84.58596
20	163.2	2.43	-8.16	1.49	163.1	1.4899	6	1.8933	129.1932	1.28847	0.3432	0.9453	171.18	1.5018	-0.006	6	0.6103	1.0712	1.8781	163.8199
21	303	3.91	-1.12	1.29	302.986	1.2905	6	1.67251	134.184	1.35556	0.3744	0.9812	307.42	1.2963	-0.002	6	0.5307	1.0409	1.6648	296.7188
22	291.8	3.52	-2.03	1.21	291.775	1.2064	6	1.65943	133.3232	1.42223	0.4056	1.0166	285.6	1.2123	-0.002	6	0.5293	1.0214	1.6567	280.2779
23	130.5	2.66	-6.51	2.04	130.42	2.0396	5	2.05952	129.3095	1.48688	0.4368	1.0501	122.78	2.0631	-0.007	5	0.6864	1.0052	2.065	122.4914
24	374.5	5.23	-6.53	1.4	374.42	1.3968	6	1.64687	136.8287	1.55529	0.468	1.0873	342.93	1.4027	-0.003	6	0.5311	0.9857	1.6529	347.332
25	289.8	5.67	-3.39	1.96	289.759	1.9568	6	1.83052	136.7945	1.62369	0.4992	1.1245	256.24	1.9678	-0.003	6	0.6053	0.9638	1.843	262.4636
26	318.9	4.11	-6.69	1.29	318.818	1.2891	6	1.65884	134.6733	1.69103	0.5304	1.1606	273.24	1.296	-0.003	6	0.5431	0.951	1.6752	285.0291
27	580.9	8.95	-1.08	1.54	580.887	1.5408	6	1.58595	137.28	1.75967	0.5616	1.1981	483.38	1.5454	-0.001	6	0.5166	0.9379	1.6008	513.3045
28	842.9	12.87	5.24	1.53	842.964	1.5268	8	1.5146	137.28	1.82831	0.5928	1.2355	680.8	1.5301	-3E-04	6	0.4908	0.9268	1.5286	736.7101
29	876.2	6.81	5.06	0.78	876.262	0.7772	7	1.24009	137.28	1.89695	0.624	1.273	686.88	0.7789	-3E-04	7	0.3887	0.9307	1.2556	769.0629

	CPT-3	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	87.9	0.35	0	0.4	87.9	0.3982	6	1.75405	113.5073	0.05675	0	0.0568	1547.8	0.3984	0	6	0.4223	2	1.495	166.0383
2	98.2	1.87	-1.21	1.91	98.1852	1.9046	5	2.12293	126.0387	0.11977	0	0.1198	818.76	1.9069	-9E-04	6	0.5881	2	1.9224	185.3599
3	62.6	0.28	-0.88	0.44	62.5892	0.4474	6	1.90823	111.0462	0.1753	0	0.1753	356.05	0.4486	-0.001	6	0.4861	2	1.6478	117.9727
4	80.3	0.76	-0.34	0.95	80.2958	0.9465	6	1.98954	118.9601	0.23478	0	0.2348	341.01	0.9493	-3E-04	6	0.5317	2	1.7601	151.3287
5	136.2	3.74	1.82	2.75	136.222	2.7455	5	2.14509	131.909	0.30073	0	0.3007	451.97	2.7516	0.001	8	0.6145	2	1.9693	256.9143
6	165.3	3.6	-1.21	2.18	165.285	2.1781	5	2.01408	132.1015	0.36678	0	0.3668	449.64	2.1829	-5E-04	6	0.5763	1.8415	1.8587	287.0139
7	148.6	4.76	-0.56	3.21	148.593	3.2034	5	2.17412	133.8856	0.43372	0	0.4337	341.6	3.2128	-3E-04	8	0.6452	1.7779	2.0336	248.944
8	262.2	5.96	-0.35	2.27	262.196	2.2731	6	1.90874	136.9157	0.50218	0	0.5022	521.11	2.2775	-1E-04	8	0.5645	1.523	1.8132	376.6736
9	92.7	2.26	-1.25	2.44	92.6847	2.4384	5	2.21723	127.2841	0.56582	0	0.5658	162.8	2.4534	-1E-03	5	0.6762	1.5269	2.0986	132.9356
10	350.4	3.03	-0.62	0.86	350.392	0.8647	6	1.49694	132.6729	0.63216	0.0312	0.601	582	0.8663	-2E-04	6	0.4247	1.2716	1.4341	420.3204
11	263.4	2.66	-0.52	1.01	263.394	1.0099	6	1.62855	131.0239	0.69767	0.0624	0.6353	413.52	1.0126	-4E-04	6	0.4753	1.2744	1.5627	316.4058
12	355.4	2.76	-0.58	0.78	355.393	0.7766	6	1.45717	132.0246	0.76368	0.0936	0.6701	529.23	0.7783	-4E-04	6	0.4174	1.2101	1.4063	405.56
13	326.3	5.01	-0.7	1.53	326.291	1.5354	6	1.71456	136.1786	0.83177	0.1248	0.707	460.36	1.5394	-5E-04	6	0.5181	1.2323	1.6658	379.0489
14	383.8	3.94	-1.04	1.03	383.787	1.0266	6	1.53142	134.8165	0.89918	0.156	0.7432	515.2	1.029	-6E-04	6	0.4537	1.1739	1.4925	424.7776
15	240.7	3.91	-1.22	1.62	240.685	1.6245	6	1.81271	133.6226	0.96599	0.1872	0.7788	307.81	1.6311	-0.001	6	0.5613	1.1877	1.7703	269.0794
16	237.3	2.39	-1.92	1.01	237.277	1.0073	6	1.65797	129.986	1.03099	0.2184	0.8126	290.73	1.0117	-0.002	6	0.5064	1.143	1.622	255.2079
17	65.6	2.31	-0.62	3.53	65.5924	3.5218	4	2.4364	126.6009	1.09429	0.2496	0.8447	76.357	3.5815	-0.005	4	0.8019	1.198	2.3934	73.02506
18	65.1	1.61	4.62	2.46	65.1566	2.471	5	2.32931	123.9432	1.15626	0.2808	0.8755	73.105	2.5156	0.0008	5	0.766	1.1562	2.2952	69.93381
19	410.2	6.11	-8.18	1.49	410.1	1.4899	6	1.64868	137.28	1.2249	0.312	0.9129	447.89	1.4943	-0.002	6	0.5154	1.0791	1.633	416.9671
20	504.5	5.58	-4.03	1.11	504.451	1.1062	6	1.49132	137.28	1.29354	0.3432	0.9503	529.45	1.109	-0.001	6	0.4594	1.0506	1.4816	499.5824
21	386.8	4.05	-12.91	1.05	386.642	1.0475	6	1.53656	135.0361	1.36106	0.3744	0.9867	390.49	1.0512	-0.003	6	0.4796	1.0341	1.5301	376.5399
22	379.3	4.91	-14.48	1.29	379.123	1.2951	6	1.61667	136.3971	1.42926	0.4056	1.0237	368.97	1.3	-0.004	6	0.5136	1.0172	1.6148	363.0724
23	392.5	5.52	-4.05	1.41	392.45	1.4066	6	1.63804	137.28	1.4979	0.4368	1.0611	368.44	1.4119	-0.002	6	0.5253	0.9985	1.6407	368.9359
24	493.9	7.5	-11.37	1.52	493.761	1.519	6	1.6143	137.28	1.56654	0.468	1.0985	448.05	1.5238	-0.003	6	0.5193	0.9807	1.6204	456.195
25	461.1	4.57	-4.8	0.99	461.041	0.9912	6	1.47285	136.3492	1.63471	0.4992	1.1355	404.58	0.9948	-0.002	6	0.4688	0.9675	1.4832	420.0437
26	547.1	6	-12.6	1.1	546.946	1.097	6	1.46988	137.28	1.70335	0.5304	1.173	464.85	1.1004	-0.003	6	0.4704	0.9527	1.4827	490.92
27	702.8	13.55	-4.78	1.93	702.741	1.9282	8	1.6386	137.28	1.77199	0.5616	1.2104	579.13	1.933	-0.001	8	0.5369	0.9304	1.6527	616.3337
28	680.8	11.88	0.53	1.74	680.806	1.745	8	1.6045	137.28	1.84063	0.5928	1.2478	544.12	1.7497	-8E-04	8	0.527	0.9168	1.622	588.2615
29	815.2	9.39	-2.13	1.15	815.174	1.1519	6	1.40792	137.28	1.90927	0.624	1.2853	632.76	1.1546	-1E-03	6	0.454	0.9155	1.4256	703.648
30	824.1	9.99	-1.02	1.21	824.088	1.2123	6	1.42621	137.28	1.97791	0.6552	1.3227	621.53	1.2152	-9E-04	6	0.4637	0.9017	1.4464	700.5648
31	723.8	13.44	-7.63	1.86	723.707	1.8571	8	1.61859	137.28	2.04655	0.6864	1.3602	530.57	1.8624	-0.002	8	0.5409	0.873	1.6443	595.4102
32	636.7	10.69	-7.21	1.68	636.612	1.6792	8	1.60166	137.28	2.11519	0.7176	1.3976	453.99	1.6848	-0.002	6	0.5383	0.8609	1.6327	516.2299
33	825.5	7.36	-7.14	0.89	825.413	0.8917	7	1.30511	137.28	2.18383	0.7488	1.435	573.67	0.894	-0.002	6	0.4255	0.8784	1.3326	683.4089
34	830.4	6.65	-6.79	0.8	830.317	0.8009	7	1.26229	137.28	2.25247	0.78	1.4725	562.36	0.8031	-0.002	7	0.4118	0.8728	1.2919	683.019
35	794.5	5.36	-6.62	0.68	794.419	0.6747	7	1.20584	137.28	2.32111	0.8112	1.5099	524.6	0.6767	-0.002	7	0.3933	0.8695	1.2387	650.9016
36	847	10.52	-4.5	1.24	846.945	1.2421	6	1.43113	137.28	2.38975	0.8424	1.5474	545.81	1.2456	-0.001	6	0.4817	0.8327	1.4661	664.6428
37	955.1	6.77	-1.92	0.71	955.077	0.7088	7	1.18776	137.28	2.45839	0.8736	1.5848	601.1	0.7107	-0.001	7	0.3899	0.8543	1.2205	769.0997
38	940.8	8.26	-0.36	0.88	940.796	0.878	7	1.27482	137.28	2.52703	0.9048	1.6222	578.38	0.8803	-1E-03	7	0.4261	0.8336	1.3107	739.1435
39	944.3	8.96	0.7	0.95	944.309	0.9488	7	1.30502	137.28	2.59567	0.936	1.6597	567.41	0.9515	-9E-04	6	0.4402	0.8202	1.3432	730.0148
40	939	7.5	0.94	0.8	939.012	0.7987	7	1.23778	137.28	2.66431	0.9672	1.6971	551.73	0.801	-1E-03	7	0.4171	0.8211	1.2779	726.6502
41	994.4	0	1.57	0	994.419	0	0	0	769.6	3.04911	0.9984	2.0507	483.43	0	-9E-04	0	1	0.516	0	0

Storke Rd & El Colegio Rd
Goleta, CA

CPT Shear Wave Measurements

CPT-1	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
	5.13	4.13	6.49	8.08	802.61	
	10.19	9.19	10.46	13.08	799.86	795.40
	15.22	14.22	15.07	18.56	812.15	841.48
	20.17	19.17	19.81	23.06	859.12	1052.87
	25.04	24.04	24.55	28.93	848.75	808.03
	30.14	29.14	29.57	35.01	844.50	824.24
	35.07	34.07	34.43	41.29	833.98	775.33
	40.11	39.11	39.43	47.05	838.01	866.91
	45.30	44.30	44.58	51.78	860.97	1089.42
	50.14	49.14	49.39	55.24	894.17	1390.88
	60.16	59.16	59.37	62.86	944.49	1309.34
	70.10	69.10	69.28	70.02	989.44	1384.04
	80.06	79.06	79.22	78.69	1006.71	1146.17
	86.24	85.24	85.39	83.76	1019.42	1216.68

CPT-2						
	5.15	4.15	6.50	5.45	1192.27	
	10.16	9.16	10.44	10.27	1016.14	816.99
	15.14	14.14	15.00	15.30	980.26	907.00
	20.16	19.16	19.80	20.96	944.74	848.70
	25.24	24.24	24.75	24.99	990.41	1227.95
	28.86	27.86	28.31	27.38	1033.79	1487.37

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

CPT-3

5.19	4.19	6.52	5.03	1296.92	
10.10	9.10	10.38	11.03	941.36	643.28
15.11	14.11	14.97	16.86	887.88	786.71
20.13	19.13	19.77	21.72	910.34	988.26
25.15	24.15	24.66	26.62	926.45	997.86
30.06	29.06	29.49	31.66	931.36	957.31
35.14	34.14	34.50	37.61	917.42	843.23
40.08	39.08	39.40	43.03	915.61	903.02
41.00	40.00	40.31	43.92	917.83	1025.54

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \le 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Peak drained friction angle, φ (°) ::

$$\varphi = 17.60 + 11 \cdot \log(Q_{tn})$$

(applicable only to SBT_n: 5, 6, 7 and 8)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$a = 14$ for $Q_{tn} > 14$

$a = Q_{tn}$ for $Q_{tn} \le 14$

$M_{CPT} = a \cdot (q_t - \sigma_v)$

If $I_c \le 2.20$

$M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1,25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = 0.1 \cdot \left(\frac{q_t - \sigma_v}{\sigma'_{vo}}\right)$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Effective Stress Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot \left(0.256 + 0.336 \cdot B_q + \log Q_t\right)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 4th Edition, July 2010
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

APPENDIX E – AMEC (2013A) REPORT OF GROUND MOTION STUDIES

REPORT OF GROUND MOTION STUDIES

UNIVERSITY OF CALIFORNIA, SANTA BARBARA – WEST CAMPUS
SANTA BARBARA, CALIFORNIA

Prepared for:

THE REGENTS OF THE UNVERSITY OF CALIFORNIA

March 29, 2013

Project 4953-12-0132

amec

March 29, 2013

Ms. Deedee Ciancola
Project Manager, Design and Construction Services
Building 439, Office of Design and Construction
University of California, Santa Barbara
Santa Barbara, California 93106-1030

Subject: **LETTER OF TRANSMITTAL**
Report of Ground Motion Studies
University of California, Santa Barbara-West Campus
El Colegio and Storke Roads
Santa Barbara, California
AMEC Project 4953-12-0132

Dear Ms. Ciancola:

We are pleased to submit the results of our ground motion studies performed for the University of California, Santa Barbara (UCSB) west campus. This study was conducted in general accordance with our proposal dated April 3, 3012 and the professional agreement dated January 31, 2012 between the Regents of the University of California and AMEC Environment and Infrastructure, Inc.

The scope of our services was planned with Mr. Ronald Strahl of your office. The locations of the field refraction microtremor (ReMi) surveys were discussed with you prior to performing the actual field surveys. Preliminary (draft) subsurface stratigraphic profiles developed by Fugro Consultants based on their recent fault rupture hazard investigations were utilized by us in preparing this report.

The results of the field ReMi surveys and the site-specific ground motion studies are presented in the attached report.

Correspondence:
AMEC Environment and Infrastructure, Inc.
6001 Rickenbacker Road
Los Angeles, California 90040, USA
Tel +1 (323) 889-5300
Fax +1 (323) 721-6700

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 and Infrastructure, Inc.

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Principal Engineer

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(5 copies submitted)

Attachments

REPORT OF GROUND MOTION STUDIES

UNIVERSITY OF CALIFORNIA, SANTA BARBARA – WEST CAMPUS
SANTA BARBARA, CALIFORNIA

Prepared for:

THE REGENTS OF THE UNIVERSITY OF CALIFORNIA

AMEC ENVIRONMENT AND INFRASTRUCTURE, INC.

Los Angeles, California

March 29, 2013

Project 4953-12-0132

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1.0 SCOPE

This report provides the results of the ground motion studies performed for the west campus of the University of California, Santa Barbara. The UCSB west campus is located at the northeast corner of El Colegio and Storke Roads in Santa Barbara, California. The location of the west campus is shown on Figure A-1, Geophysical Survey Map, included in Appendix A.

These consultation services were authorized to perform refraction microtremor (ReMi) surveys at the UCSB west campus to estimate shear-wave velocities of the subsurface materials and to perform site-specific ground motion studies for seismic design of future buildings at the west campus. The report of the ground motion studies was to include the following:

- Results of the ReMi surveys along with the generalized shear wave velocity profiles of the subsurface materials.
- Site-specific horizontal response spectra for Design and Maximum Considered Earthquake (MCE) ground motions developed in accordance with guidelines presented in Chapter 21 of ASCE 7-05 and for damping ratios of 2, 5 and 10%.
- Site-specific vertical response spectra for damping ratios of 2, 5 and 10%.
- Digitized data of the spectral ordinates of acceleration and velocity response spectra for the Design and MCE horizontal ground motions and vertical ground motions for damping ratios of 2, 5 and 10%.

Other than the ReMi surveys, field explorations consisting of borings or cone penetration test (CPT) soundings were not performed for the ground motion study. However, subsurface information at the site was available from the following prior geotechnical investigations performed by our legacy company LeRoy Crandall and Associates (LC&A) at the west campus.

- Geotechnical Investigation, Residential Apartment Complex, El Colegio and La Carneros Roads, University of California, Santa Barbara, California, report dated April 26, 1979 (LC&A Job No. AE-79034).
- Fault Hazard Investigation, Residential Apartment Complex Site 2, El Colegio and La Carneros Roads, University of California, Santa Barbara, California, report dated June 13, 1979 (LC&A Job No. E-79152).

In addition, preliminary (draft) subsurface stratigraphic profiles developed based on the cone penetrations test (CPTs) and continuous core boring data by Fugro Consultants (Fugro) was also made available to us. The investigations performed by Fugro are primarily to evaluate fault rupture hazard at the UCSB west campus. Seismic shear-wave velocity measurements were not obtained in the Fugro CPTs. The fault rupture hazard investigation by Fugro is in progress and the final report has not been prepared at the time of this report.

2.0 PROJECT DESCRIPTION AND BACKGROUND

We were requested to perform ground motion studies for the west campus of the University of California, Santa Barbara (UCSB). The proposed development at the west campus will include the expansion of the existing Santa Catalina Residence Hall Facility which is located in the northeast of El Colegio and Storke Roads. The proposed buildings are anticipated to be five-story or less and may or may not include subterranean levels. The ground surface slopes down from west to east (Elevation 32 to 22) and from south to north (Elevation 32 to 16).

As stated previously, Fugro Consultants is currently performing a fault rupture investigation of the UCSB west campus and the results of the investigations are not available at this time. However, preliminary (draft) subsurface stratigraphic profiles at the west campus developed by Fugro based on the recent cone penetration tests and continuous core borings performed for the fault investigation were provided to us.

The UCSB main campus is underlain by shallow (10 to 15 feet thick) marine terrace deposits over Sisquoc bedrock formation, however, the west campus is underlain by variable thicknesses of artificial fill, alluvial deposits, and marine terrace deposits overlying siltstone and sandstone bedrock of the Pico formation based on the ongoing investigations being performed by Fugro and the current interpretations of the results thus far.

Shear-wave velocity measurements were not obtained in the CPTs performed by Fugro for the fault rupture hazard investigation. Therefore, we performed three refraction microtremor (ReMi) surveys at the site to obtain shear-wave velocity measurements of the soil and bedrock.

3.0 RESULTS OF FIELD REMI SURVEYS

Shear wave velocity data of the underlying soil and bedrock are required to at least a depth of about 100 feet or 30 meters below the ground surface ($V_{s,30}$) for ground motion studies. Therefore, three refraction microtremor (ReMi) surveys were performed to determine the shear wave velocity of the subsurface materials and to evaluate the variability of the shear wave velocity across the site.

The field ReMi survey procedures, the locations of the ReMi surveys and the survey results are presented in Appendix A. The generalized shear wave velocity profiles are also presented in Appendix A. The estimated $V_{s,30}$ values for the three ReMi surveys are presented below.

Survey Location*	$V_{s,30}$ (feet/sec)
ReMi-L01	1070
ReMi-L02	960
ReMi-L03	990
*see Figure A-1 for survey locations	

Using the $V_{s,30}$ values from the three survey lines, the average $V_{s,30}$ is estimated to about 1,005 feet per sec (or 305 meters per second) with a standard deviation of 46 feet per second, corresponding to a coefficient of variation (COV) of 0.046. Based on the $V_{s,30}$ values presented above, the site may be classified as "Site Class D" in accordance with current California Building Code (CBC).

The shear wave velocity measurements indicate that the velocity of the overlying soil and bedrock are similar and that the variability of the shear wave velocity across the site in the upper 30 meters is low. It is noted, however, that shear wave velocities can be more reliably obtained using either (a) p- and s-wave suspension logging in a drilled borehole and/or (b) advancing seismic CPTs. Seismic CPTs are more cost effective compared to suspension logging. It is suggested that seismic CPTs be performed when geotechnical investigations are performed for the future buildings at the west campus, especially if the site is located farther away from the area of this study.

4.0 GROUND MOTION STUDY

We have performed a Probabilistic Seismic Hazard Analyses (PSHA) and Deterministic Seismic Hazard Analyses (DSHA) using the computer program EZ-FRISK, Version 7.62 (Risk Engineering, 2012) in order to develop site-specific response spectra in accordance with the 2007 CBC and Chapter 21 of ASCE 7-05. For the DSHA, a composite deterministic response spectrum was compiled from the maximum spectral ordinates computed for all known faults.

All known fault sources and background seismicity within 200 kilometers of the site were included in the PSHA (USGS, 2008). The fault sources, fault mechanism and the closest distance from the fault to the site are listed in Table 1. Because of close proximity of several earthquake faults, directivity effects were also considered in the analysis using the procedures recommended by Somerville (1997) and Abrahamson (2000) for fault normal and fault parallel components.

The site-specific probabilistic and deterministic response spectra were developed using the average of the median 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 the Abrahamson and Silva attenuation relationship, the hanging wall correction term was used (Abrahamson, 2009).

For all four NGA relationships, we have used an average shear wave velocity in the upper 30 meters of the profile equal to 1,000 feet per second (about 305 meters per second), as determined from ReMi surveys performed at the site. For the Abrahamson and Silva and the Chiou and Youngs attenuation equations, we have estimated the depth to a shear wave velocity of 1,000 meters per second beneath the site ($Z_{1.0}$) to be approximately 400 meters using the Abramson and Silva (2008) soil depth model (relationship between $V_{s,30}$ and $Z_{1.0}$). For the Campbell and Bozorgnia attenuation relationship, we have used a depth to a shear wave velocity of 2,500 meters per second ($Z_{2.5}$) of 2,000 meters (2 kilometers) using the Campbell and Bozorgnia (2007) relationship between $Z_{1.0}$ and $Z_{2.5}$. To account for the uncertainty in the ground motion attenuation relationships, each relationship was integrated to three standard deviations beyond the median.

In accordance with Chapter 21 of ASCE 7-05, the probabilistic Maximum Considered Earthquake (MCE) response spectrum was taken as the response spectrum with a 2% probability of being exceeded in 50 years. The deterministic MCE response spectrum was taken as the maximum of the

84th percentile deterministic response spectrum and the deterministic lower limit, as defined on Figure 21.2-1 of ASCE 7-05. The site-specific MCE response spectrum was taken as a composite of the probabilistic and deterministic MCE response spectra, determined as described above, which consisted of the lesser of the spectral ordinates between the two spectra. The site-specific MCE response spectra for fault normal and fault parallel components are presented on Figures 1.1 and 2.1, respectively, for 2%, 5%, and 10% of critical structural damping. The components of the 5% damped site-specific MCE response spectrum for fault normal and fault parallel components are shown on Figures 1.2 and 2.2 respectively. The site-specific design response spectrum was determined as the lesser of ⅔ of the site-specific MCE response spectrum and 80% of the code-based design response spectrum at each period. The site-specific design response spectra for fault normal and fault parallel components are presented on Figures 1.3 and 2.3, respectively, for 2%, 5%, and 10% of critical structural damping. The components of the 5% damped site-specific design response spectrum for fault normal and fault parallel components are shown on Figures 1.4 and 2.4 respectively.

Digitized data of the response spectra are also provided in tabular form. The site-specific MCE and site-specific design response spectra in digitized form for fault normal component are presented in Tables 2.1 and 2.2 respectively. The digitized data for fault parallel component are presented in Tables 3.1 and 3.2 respectively.

Based on the results of our analyses, the site-specific design acceleration parameters (S_{MS} and S_{M1}, S_{DS} and S_{D1}), as defined in Section 21.4 of ASCE 7-05, were computed for fault normal and fault parallel components as presented below.

Design Parameters	Fault Normal	Fault Parallel
S_{MS}	2.19g	2.19g
S_{M1}	1.64g	1.46g
S_{DS}	1.46g	1.46g
S_{D1}	1.09g	0.97g
S_{MS} and S_{M1} are spectral accelerations at 0.2 sec and 1 sec for MCE S_{DS} and S_{D1} are spectral accelerations at 0.2 sec and 1 sec for Design		

The three fault sources controlling the ground motions at the west campus: North Channel Fault, Pitas Point (lower, West) Fault and Red Mountain Fault are aligned roughly in the east-west direction. Therefore, the fault normal and fault parallel components for seismic design of buildings may be considered to be in the north-south and east-west directions, respectively.

The vertical response spectrum was computed using attenuation relationship of Campbell-Bozorgnia (2003). The site-specific vertical response spectra for 2% probability of being exceeded in 50 years and for 2%, 5%, and 10% of critical structural damping is presented in Figure 3.1 Digitized data of the response spectra are also provided in tabular form in Tables 4.1 and 4.2.

5.0 GENERAL LIMITATIONS AND BASIS FOR RECOMMENDATIONS

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, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for the Regents of the University of California and University of California-Santa Barbara (UCSB) and their design consultants to be used solely in the planning and design of the proposed buildings at UCSB west campus. 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.

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 current and previous subsurface explorations. 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.

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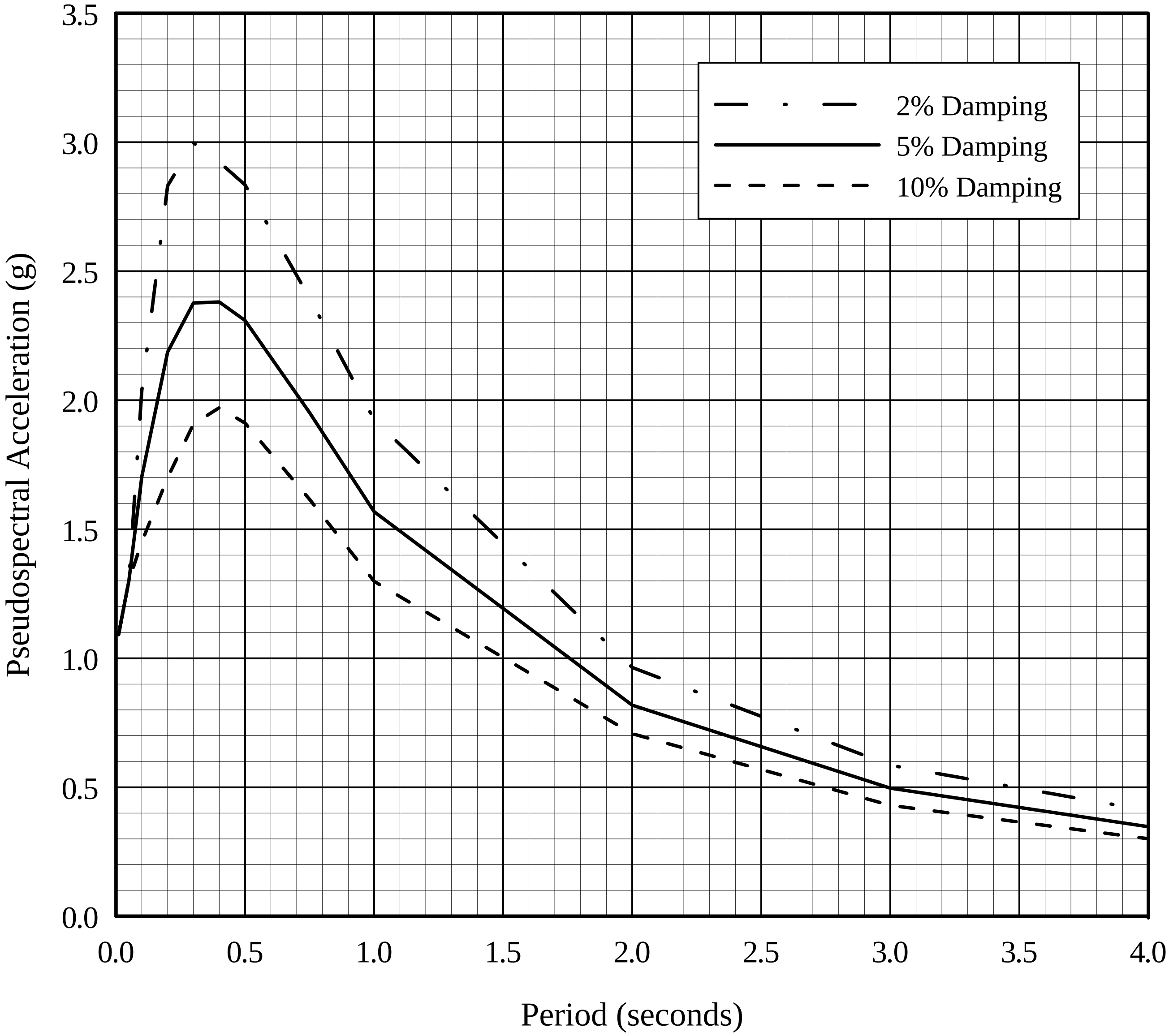
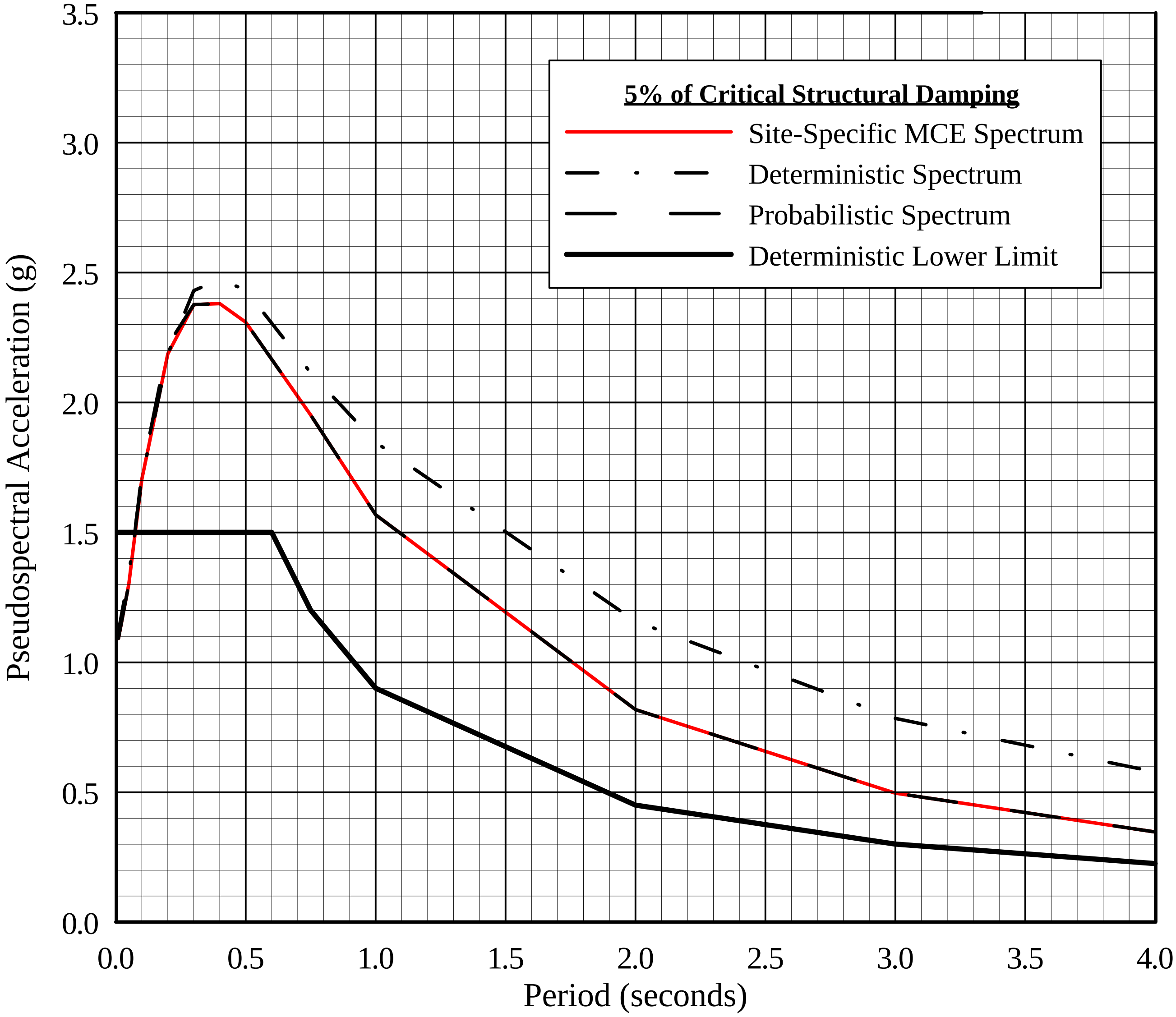
FIGURES



Prepared/Date: LT 5/21/12
Checked/Date: HP 5/22/12

University of California - Santa Barbara West Campus Santa Barbara, California	amec	Site Specific Maximum Considered Earthquake (MCE) Horizontal Response Spectra (Fault Normal) Project No. 4953-12-0132 Figure 1.1

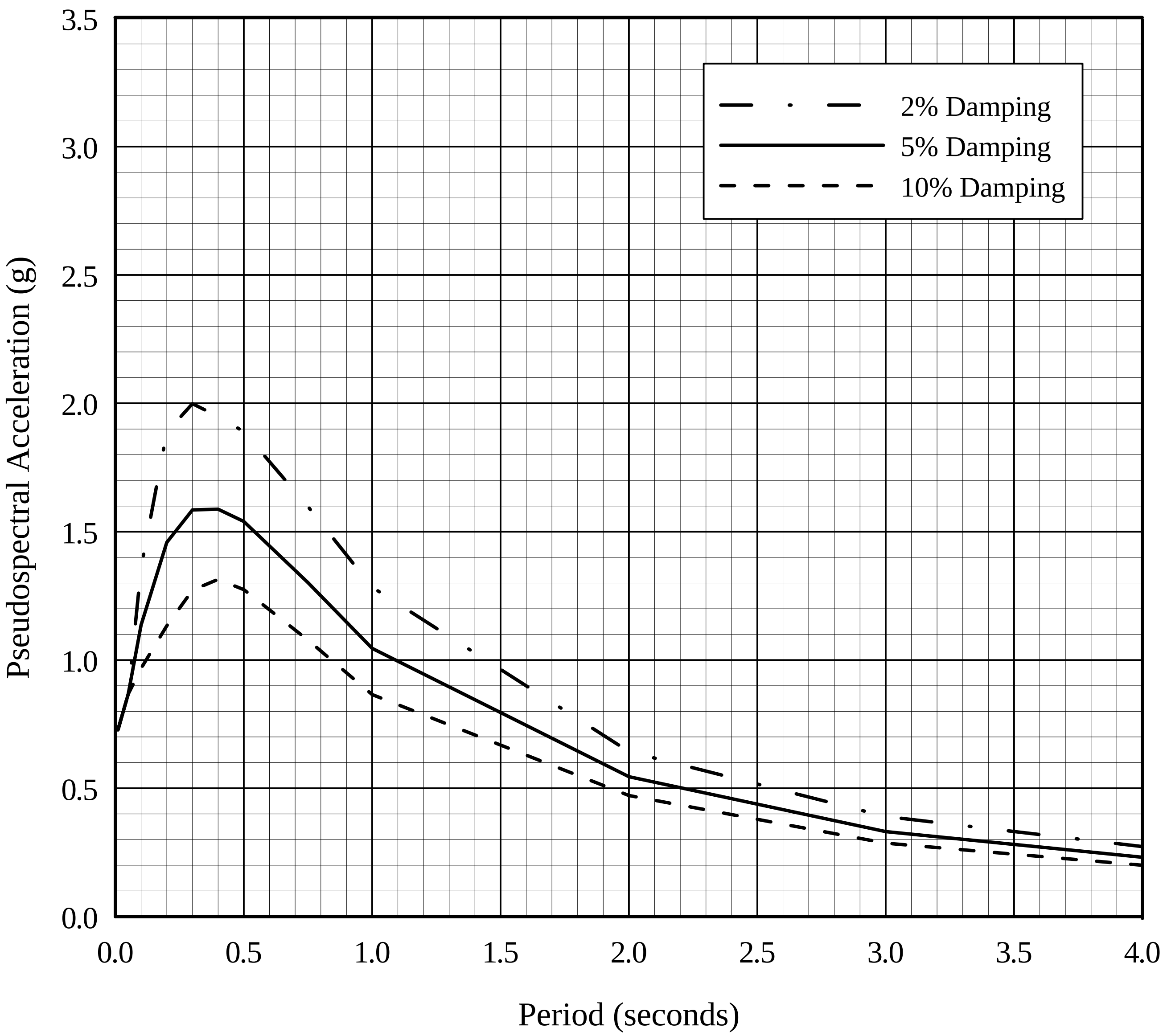


NOTES: Probabilistic spectrum was computed for a ground motion level with a 2% probability of being exceeded in 50 years.

Deterministic spectrum is governed by:
Magnitude-6.8 earthquake on the North Channel Fault from 0 to 0.2 seconds,
Magnitude-7.3 earthquake on the Pitas Point (Lower, West) Fault from 0.2 to 1.0 second, and
Magnitude-7.4 earthquake on the Red Mountain Fault beyond 1.0 second.

Prepared/Date: LT 5/21/12
Checked/Date: HP 5/22/12

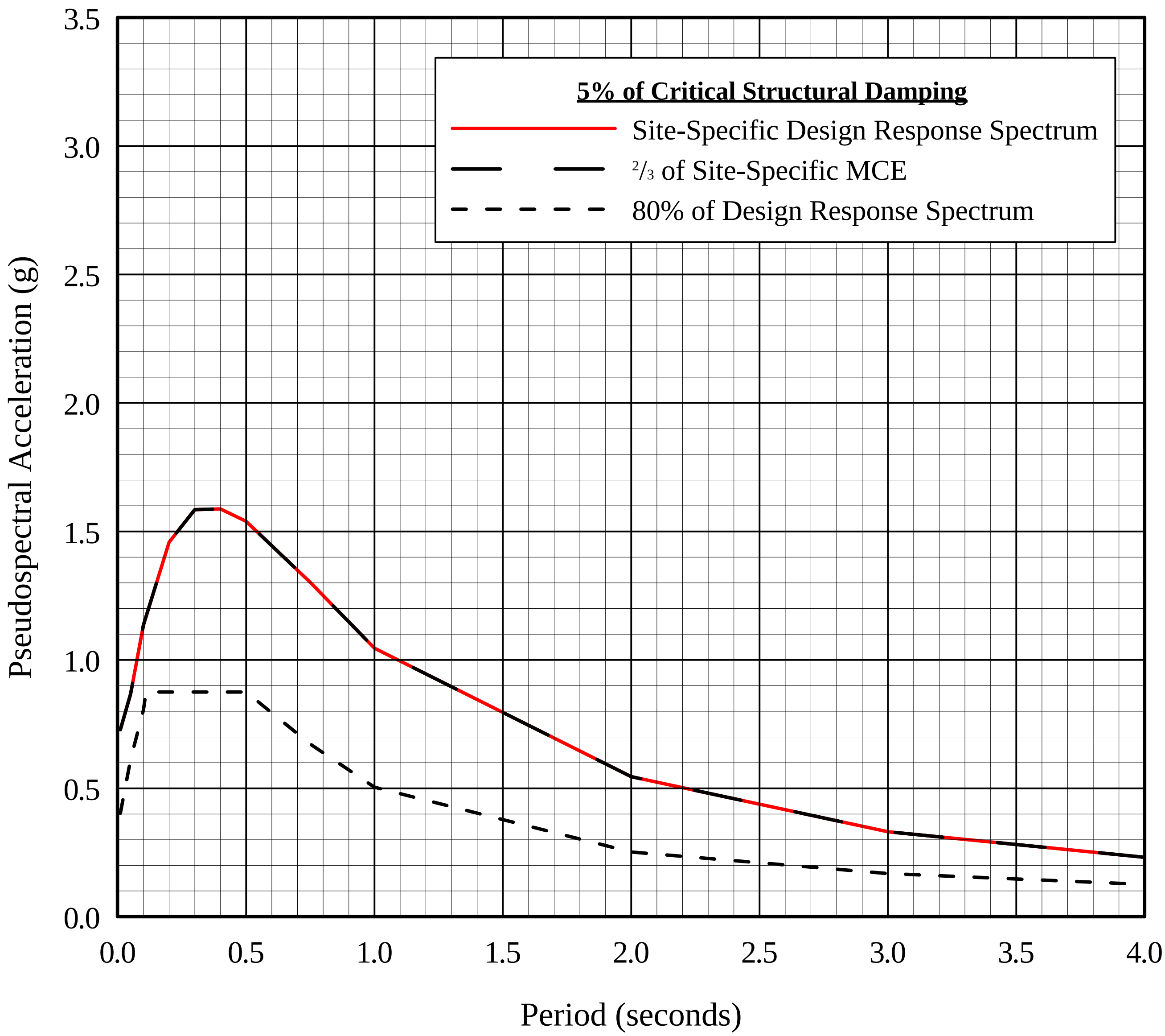
University of California - Santa Barbara West Campus Santa Barbara, California	amec	Components of the Site-Specific Maximum Considered Earthquake (MCE) Horizontal Response Spectrum (Fault Normal) Project No. 4953-12-0132 Figure 1.2



Prepared/Date: LT 5/21/12
Checked/Date:HP 5/22/12

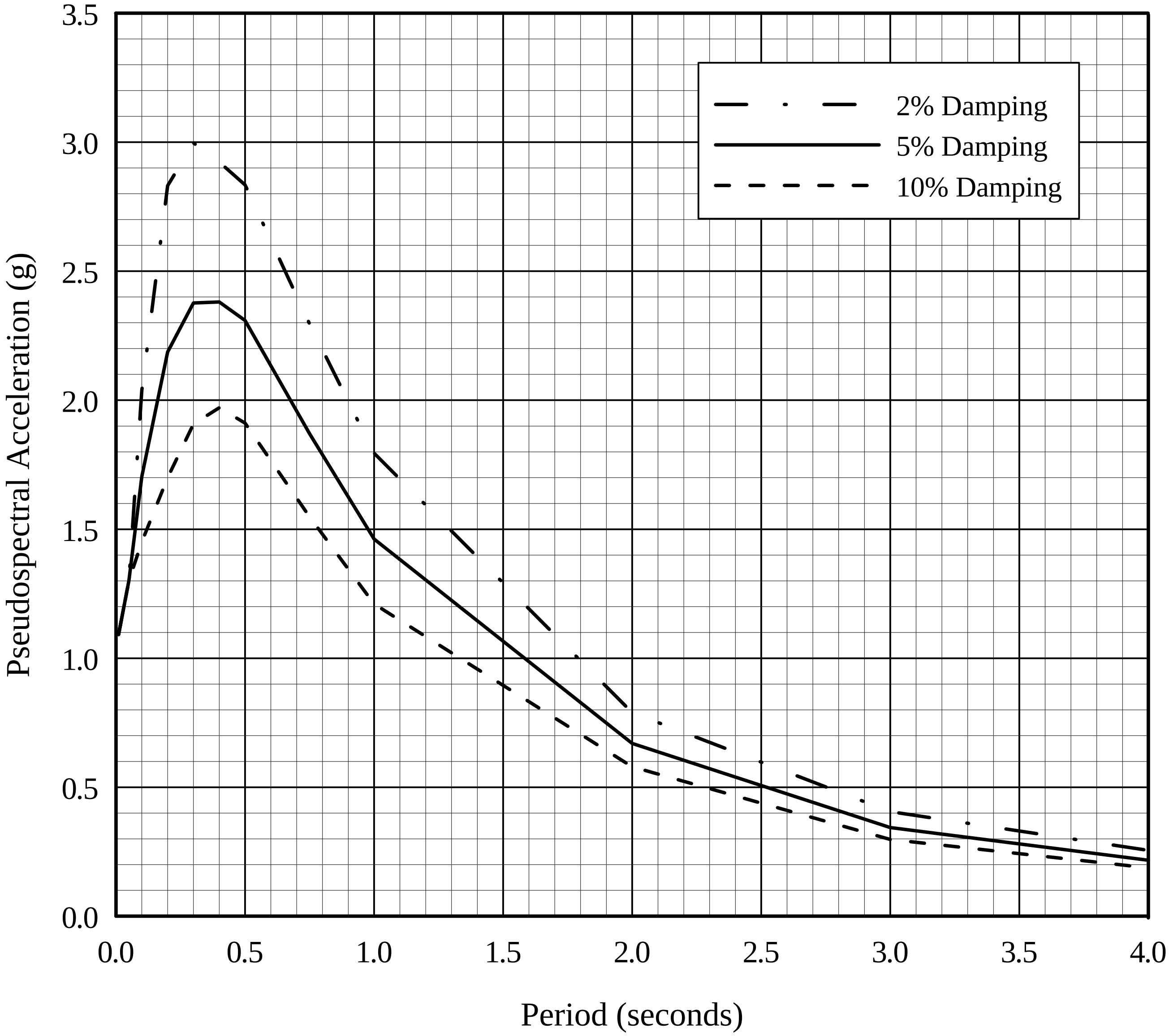
University of California -
Santa Barbara West Campus
Santa Barbara, California

Site Specific
Design Horizontal Response Spectra
(Fault Normal)
Project No. 4953-12-0132 Figure 1.3



Prepared/Date: LT 5/21/12
Checked/Date: HP 5/22/12

University of California - Santa Barbara West Campus Santa Barbara, California	amec	Components of the Site-Specific Design Horizontal Response Spectrum (Fault Normal) Project No. 4953-12-0132 Figure 1.4

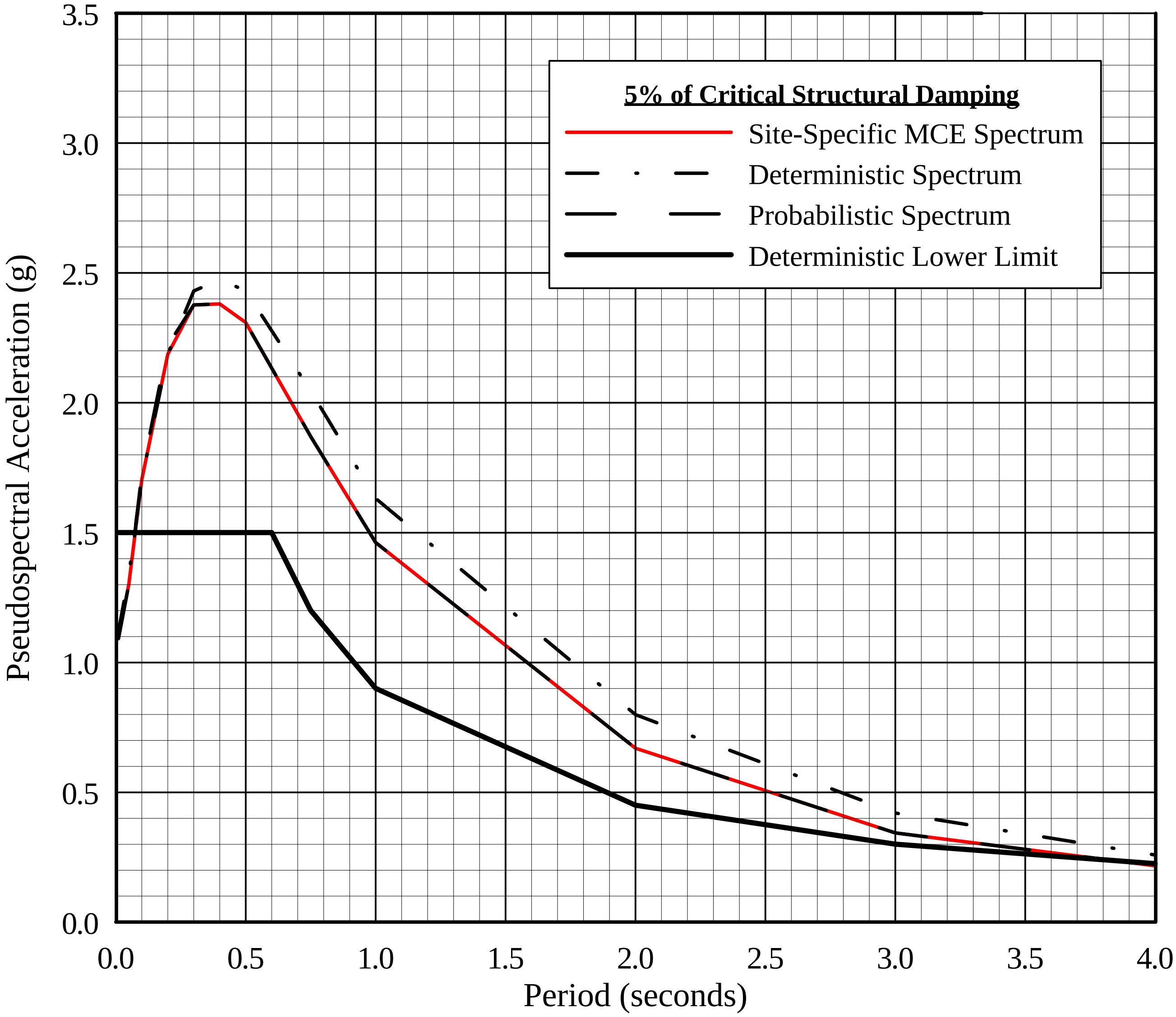


Prepared/Date: LT 5/21/12
Checked/Date: HP 5/22/12

University of California -
Santa Barbara West Campus
Santa Barbara, California

amec

Site Specific
Maximum Considered Earthquake (MCE)
Horizontal Response Spectra
(Fault Parallel)
Project No. 4953-12-0132 Figure 2.1

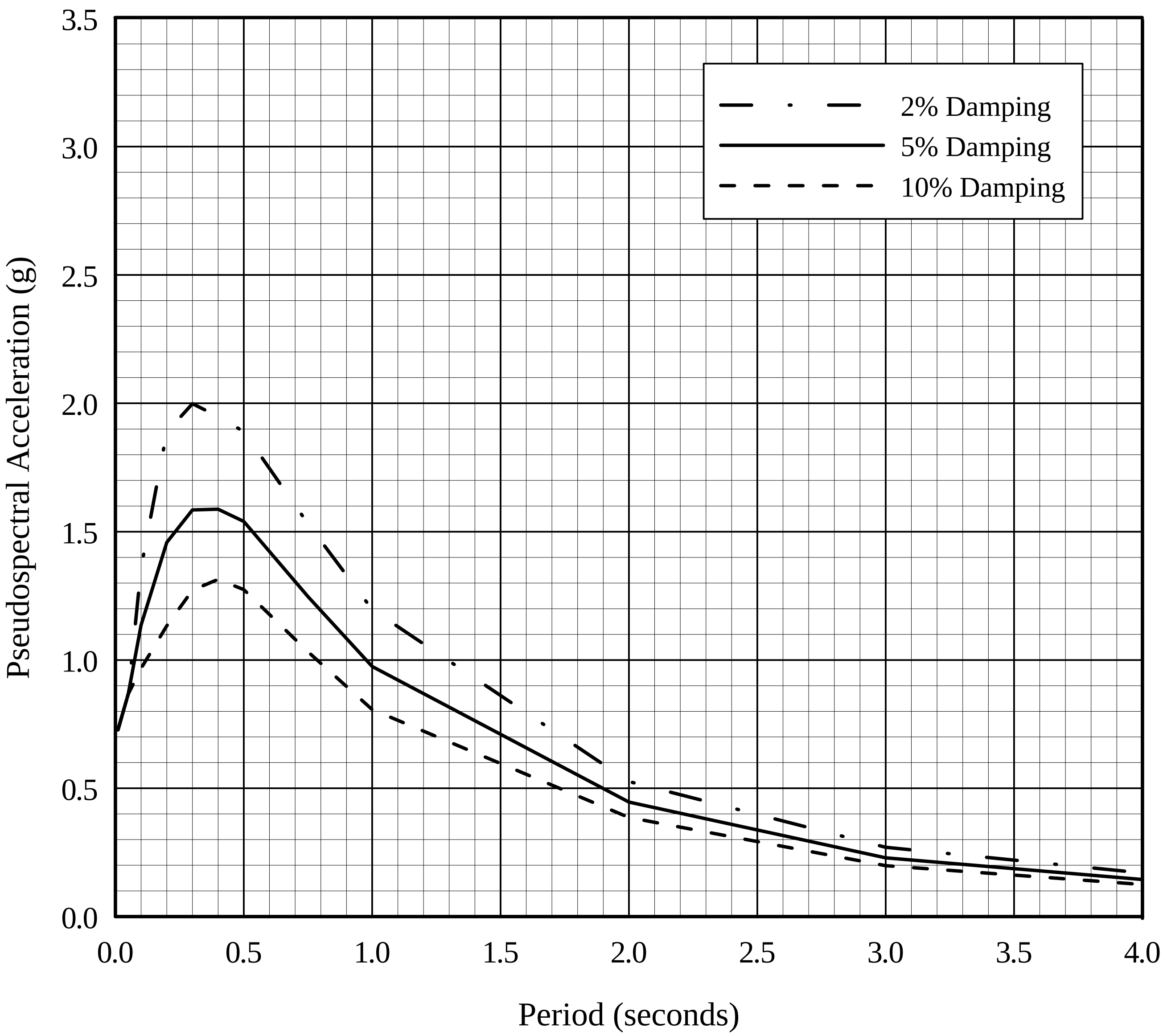


NOTES: Probabilistic spectrum was computed for a ground motion level with a 2% probability of being exceeded in 50 years.

Deterministic spectrum is governed by:
Magnitude-6.8 earthquake on the North Channel Fault from 0 to 0.2 seconds,
Magnitude-7.3 earthquake on the Pitas Point (Lower, West) Fault from 0.2 to 2.0 seconds, and
Magnitude-7.4 earthquake on the Red Mountain Fault beyond 2.0 seconds.

Prepared/Date: LT 5/21/12
Checked/Date: HP 5/22/12

University of California - Santa Barbara West Campus Santa Barbara, California	amec	Components of the Site-Specific Maximum Considered Earthquake (MCE) Horizontal Response Spectrum (Fault Parallel) Project No. 4953-12-0132 Figure 2.2

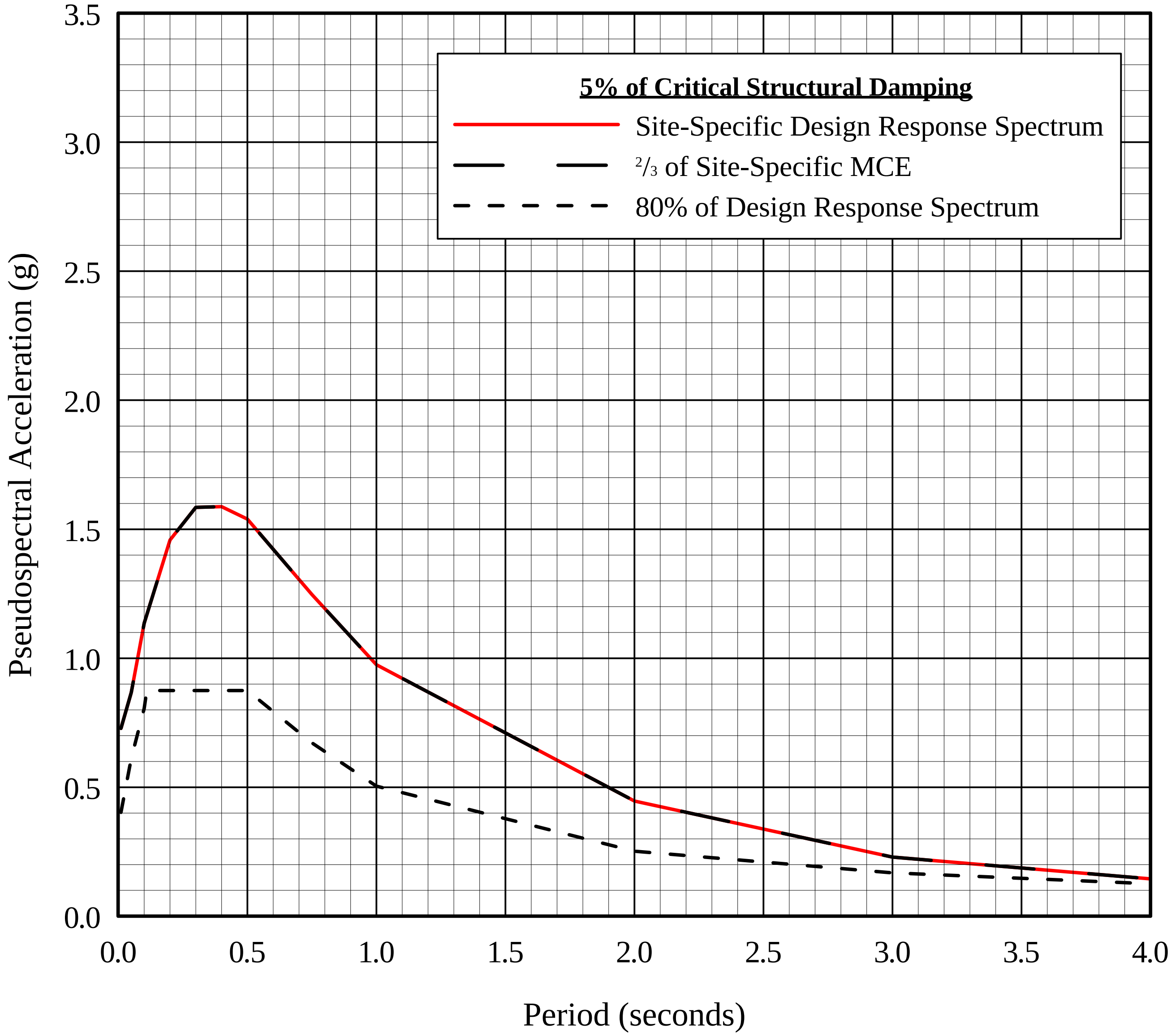


Prepared/Date: LT 5/21/12
Checked/Date:HP 5/22/12

University of California -
Santa Barbara West Campus
Santa Barbara, California

amec

Site Specific
Design Horizontal Response Spectra
(Fault Parallel)
Project No. 4953-12-0132 Figure 2.3



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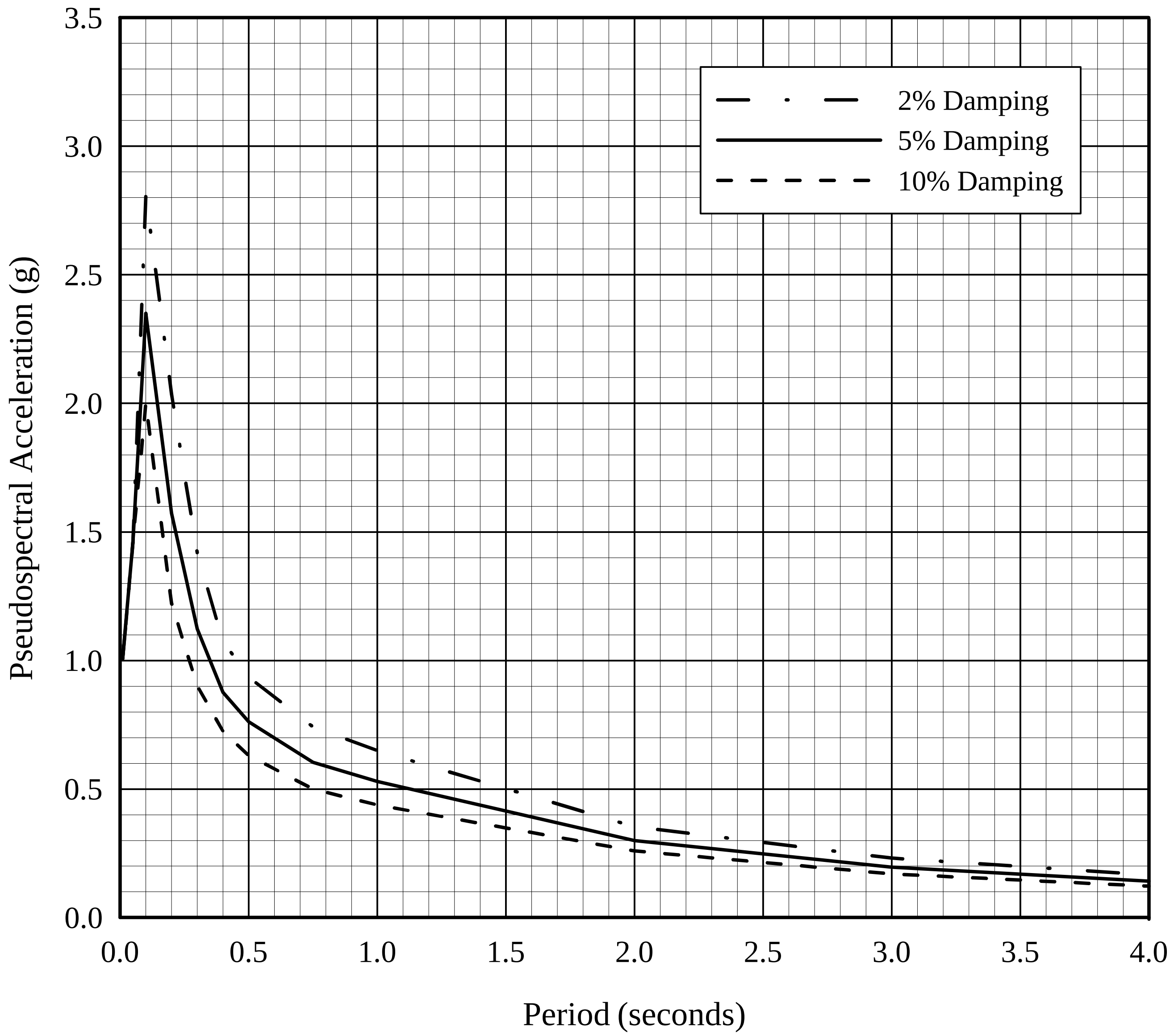
University of California - Santa Barbara West Campus Santa Barbara, California	amec	Components of the Site-Specific Design Horizontal Response Spectrum (Fault Parallel) Project No. 4953-12-0132 Figure 2.4



NOTES: Probabilistic spectrum was computed for a ground motion level with a 2% probability of being exceeded in 50 years.

Prepared/Date: JF 3/29/2013
Checked/Date: LT 3/29/2013

University of California - Santa Barbara West Campus Santa Barbara, California	amec	Site Specific Vertical Response Spectra (2% in 50 years) Project No. 4953-12-0132, Figure 3.1

TABLES

Table 1: Fault Sources within 200 Kilometers of the Site

Fault Source	Closest Distance (Kilometers)	Magnitude (M_w)	Fault Mechanism	Dip Angle (Degrees)	Dips to
Mission Ridge-Arroyo Parida-Santa Ana	0.6	6.9	Reverse	70	S
Red Mountain	3.1	7.4	Reverse	56	N
North Channel	4.0	6.8	Reverse	26	N
Pitas Point (Lower, West)	4.9	7.3	Reverse	13	N
Pitas Point (Upper)	6.9	6.9	Reverse	42	N
Pitas Point Connected	7.9	7.3	Reverse	55.3	N
Pitas Point (Lower)-Montalvo	12.3	7.3	Reverse	16	N
Santa Ynez (West)	14.0	7.0	Strike Slip	70	S
Santa Ynez Connected	14.0	7.4	Strike Slip	70	S
Santa Ynez (East)	22.7	7.2	Strike Slip	70	S
Oak Ridge (Offshore)	25.4	7.0	Reverse	32	S
Oak Ridge Connected	25.4	7.4	Reverse	53.1	S
Ventura-Pitas Point	28.1	7.0	Reverse	64	N
Los Alamos-West Baseline	28.3	6.9	Reverse	30	SW
Channel Islands Thrust	35.0	7.3	Reverse	20	N
Santa Cruz Island	40.2	7.2	Strike Slip	90	--
Santa Rosa Island	45.4	6.9	Strike Slip	90	--
Lions Head	45.7	6.8	Reverse	75	NE
San Luis Range (So Margin)	58.6	7.2	Reverse	45	NE
Oak Ridge (Onshore)	63.7	7.2	Reverse	65	S
Casmalia (Orcutt Frontal)	64.1	6.7	Reverse	75	SW
San Cayetano	65.1	7.2	Reverse	42	N
Anacapa-Dume	66.9	7.2	Reverse	41-45	N
Santa Monica	69.1	7.4	SS|R	44-75	N
Simi-Santa Rosa	70.5	6.9	Strike Slip	60	N
Southern San Andreas	72.0	8.2	Strike Slip	58-90	N,NE
Pleito	73.6	7.1	Reverse	46	S
San Juan	81.4	7.1	Strike Slip	90	--
Los Osos	88.7	7.0	Reverse	45	SW
Hosgri	93.3	7.3	Strike Slip	80	NE
Malibu Coast	95.0	7.0	Strike Slip	74-75	N
San Gabriel	96.6	7.3	Strike Slip	61	NE
Garlock	97.8	7.72	Strike Slip	90	--
Santa Susana, alt 1	101.2	6.9	Reverse	55	N
Holser, alt 1	102.2	6.8	Reverse	58	S
White Wolf	103.7	7.2	Reverse	75	SE
Northridge	105.5	6.9	Reverse	35	S
Rinconada	115.5	7.5	Strike Slip	90	--
Sierra Madre (San Fernando)	128.2	6.7	Reverse	45	N
Sierra Madre Connected	128.2	7.3	Reverse	51	N

Table 1 (Continued): Fault Sources within 200 Kilometers of the Site

Fault Source	**Closest Distance (Kilometers)**	**Magnitude (M_w)**	**Fault Mechanism**	**Dip Angle (Degrees)**	**Dips to**
Palos Verdes	130.4	7.3	Strike Slip	90	--
Palos Verdes Connected	130.4	7.7	Strike Slip	90	--
Verdugo	134.0	6.9	Reverse	55	NE
Hollywood	136.2	6.7	Strike Slip	70	N
Newport-Inglewood	142.1	7.5	Strike Slip	88-90	NE
Sierra Madre	145.7	7.2	Reverse	53	N
Puente Hills (LA)	147.3	7.0	Reverse	27	N
Elysian Park (Upper)	148.3	6.7	Reverse	50	N
Puente Hills	149.8	7.1	Reverse	25	N
Great Valley 14 (Kettleman Hills)	151.6	7.2	Reverse	22	SW
Raymond	154.6	6.8	Reverse	79	N
Puente Hills (Santa Fe Springs)	159.8	6.7	Reverse	29	N
Clamshell-Sawpit	166.4	6.7	Reverse	50	NW
Elsinore	173.8	7.85	Strike Slip	75-90	NE
Puente Hills (Coyote Hills)	177.5	6.9	Reverse	26	N
Great Valley 13 (Coalinga)	181.3	7.1	Reverse	15	SW
San Jose	185.1	6.7	Strike Slip	74	NW
San Andreas - creeping segment	186.9	6.7	Strike Slip	90	--
Chino	194.0	6.8	Strike Slip	50-65	SW
So Sierra Nevada	194.4	7.5	Normal	50	E
Cucamonga	195.2	6.7	Reverse	45	N
San Joaquin Hills	195.5	7.1	Reverse	23	SW

Notes:

Fault Sources and fault parameters from 2008 USGS Fault Database
SS – Strike Slip, R – Reverse

Table 2.1. Site-Specific Horizontal Response Spectra (Fault Normal)
Pseudospectral Acceleration in g

	2% Damping		5% Damping		10% Damping	
Period in Seconds	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake
0.01	0.73	1.09	0.73	1.09	0.73	1.09
0.05	0.87	1.30	0.87	1.30	0.87	1.30
0.10	1.36	2.04	1.14	1.70	0.97	1.45
0.20	1.89	2.83	1.46	2.19	1.13	1.70
0.30	2.00	3.00	1.58	2.38	1.27	1.91
0.40	1.95	2.92	1.59	2.38	1.31	1.97
0.50	1.89	2.83	1.54	2.31	1.27	1.91
0.75	1.60	2.40	1.30	1.95	1.08	1.62
1.00	1.28	1.92	1.05	1.57	0.87	1.30
2.00	0.64	0.96	0.55	0.82	0.47	0.71
3.00	0.39	0.59	0.33	0.50	0.29	0.43
4.00	0.27	0.41	0.23	0.35	0.20	0.30

By: LT 5/21/12, Checked: HP 5/23/12

Table 2.2. Site-Specific Horizontal Response Spectra (Fault Normal)
Pseudospectral Velocity in Inches/Second

	2% Damping		5% Damping		10% Damping	
Period in Seconds	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake
0.01	0.45	0.67	0.45	0.67	0.45	0.67
0.05	2.67	4.00	2.67	4.00	2.67	4.00
0.10	8.36	12.54	6.99	10.48	5.95	8.92
0.20	23.21	34.82	17.93	26.90	13.94	20.91
0.30	36.87	55.30	29.24	43.85	23.46	35.19
0.40	47.94	71.90	39.05	58.57	32.32	48.48
0.50	58.11	87.16	47.33	71.00	39.18	58.77
0.75	73.69	110.53	60.02	90.03	49.68	74.53
1.00	78.92	118.38	64.29	96.43	53.21	79.82
2.00	79.06	118.59	67.08	100.62	58.02	87.03
3.00	71.99	107.98	61.08	91.62	52.83	79.24
4.00	67.11	100.66	56.94	85.41	49.25	73.87

By: LT 5/21/12, Checked: HP 5/23/12

Table 3.1. Site-Specific Horizontal Response Spectra (Fault Parallel)
Pseudospectral Acceleration in g

	2% Damping		5% Damping		10% Damping	
Period in Seconds	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake
0.01	0.73	1.09	0.73	1.09	0.73	1.09
0.05	0.87	1.30	0.87	1.30	0.87	1.30
0.10	1.36	2.04	1.14	1.70	0.97	1.45
0.20	1.89	2.83	1.46	2.19	1.13	1.70
0.30	2.00	3.00	1.58	2.38	1.27	1.91
0.40	1.95	2.92	1.59	2.38	1.31	1.97
0.50	1.89	2.83	1.54	2.31	1.27	1.91
0.75	1.53	2.30	1.25	1.87	1.03	1.55
1.00	1.20	1.79	0.97	1.46	0.81	1.21
2.00	0.53	0.79	0.45	0.67	0.39	0.58
3.00	0.27	0.41	0.23	0.34	0.20	0.30
4.00	0.17	0.26	0.14	0.22	0.13	0.19

By: LT 5/21/12, Checked: HP 5/23/12

Table 3.2. Site-Specific Horizontal Response Spectra (Fault Parallel)
Pseudospectral Velocity in Inches/Second

	2% Damping		5% Damping		10% Damping	
Period in Seconds	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake	Design	Maximum Considered Earthquake
0.01	0.45	0.67	0.45	0.67	0.45	0.67
0.05	2.67	4.00	2.67	4.00	2.67	4.00
0.10	8.36	12.54	6.99	10.48	5.95	8.92
0.20	23.21	34.82	17.93	26.90	13.94	20.91
0.30	36.87	55.30	29.24	43.85	23.46	35.19
0.40	47.94	71.90	39.05	58.57	32.32	48.48
0.50	58.11	87.16	47.33	71.00	39.18	58.77
0.75	70.59	105.89	57.50	86.25	47.60	71.40
1.00	73.59	110.38	59.94	89.91	49.62	74.42
2.00	64.73	97.09	54.92	82.38	47.50	71.25
3.00	49.84	74.75	42.29	63.43	36.57	54.86
4.00	41.90	62.85	35.55	53.33	30.75	46.13

By: LT 5/21/12, Checked: HP 5/23/12

Table 4.1. Site-Specific Vertical Response Spectra
Pseudospectral Acceleration in g
(2% probability of exceedence in 50 years)

Period in Seconds	2% Damping	5% Damping	10% Damping
0.01	1.01	1.01	1.01
0.05	1.46	1.46	1.46
0.10	2.81	2.35	2.00
0.20	2.04	1.57	1.22
0.30	1.42	1.12	0.90
0.40	1.07	0.88	0.72
0.50	0.93	0.76	0.63
0.75	0.74	0.60	0.50
1.00	0.65	0.53	0.44
2.00	0.35	0.30	0.26
3.00	0.23	0.20	0.17
4.00	0.17	0.14	0.12

By: LT 3/29/13, Checked: HP: 3/29/13

Table 4.2. Site-Specific Vertical Response Spectra
Pseudospectral Velocity in Inches/Second
(2% probability of exceedence in 50 years)

Period in Seconds	2% Damping	5% Damping	10% Damping
0.01	0.62	0.62	0.62
0.05	4.50	4.50	4.50
0.10	17.29	14.45	12.31
0.20	25.05	19.35	15.04
0.30	26.13	20.72	16.63
0.40	26.44	21.54	17.83
0.50	28.75	23.42	19.39
0.75	34.20	27.86	23.06
1.00	39.95	32.54	26.93
2.00	43.46	36.87	31.89
3.00	42.66	36.20	31.31
4.00	40.97	34.76	30.06

By: LT 3/29/13, Checked: HP: 3/29/13

APPENDIX A

FIELD REMI SURVEYS

REFRACTION MICROTREMOR (ReMi) SURVEY

A total of three Refraction Microtremor (ReMi) surveys (L01, L02 and L03) were conducted on April 26, 2012, at the University of California, Santa Barbara west campus in Santa Barbara, California. The locations of the ReMi surveys are shown on Figure A-1, Geophysical Survey Plan. The purpose of these surveys was to determine the International Building Code (IBC) seismic site classification needed as part of site-response characterization for design earthquake ground motion estimation. ReMi surveys produce shear-wave velocity profiles at the site which are used to calculate the average shear wave velocity of the upper 100 feet or 30 meters of the subsurface which is the basis of the IBC site classification. The IBC site classifications are defined as follows:

Site Class	Site Class Description	Average Shear-Wave Velocity in upper 100 feet or 30 meters ($V_{s,30}$)	
		(feet/second)	(meters/second)
A	Hard Rock	$V_s > 5000$	$V_s > 1524$
B	Rock	$2500 < V_s \leq 5000$	$762 < V_s \leq 1524$
C	Very Dense Soil/Soft Rock	$1200 < V_s \leq 2500$	$366 < V_s \leq 762$
D	Stiff Soil	$600 < V_s \leq 1200$	$183 < V_s \leq 366$
E	Soft Soil	$V_s < 600$	$V_s < 183$

A ReMi survey uses ambient vibrations from all sources, including passing vehicles, trains and construction activity, as its energy source and measures particle-velocity time histories at a linear array of geophones. The measured time history records are used to produce a shear-wave velocity profile representative of a location at the center of the survey line. The equipment used during this investigation consisted of a Seismic Source DAQ Link II 24 seismograph and 24 4.5-Hz vertically oriented geophones connected by refraction cables. For each survey, the geophones were spaced 10 feet apart producing a total survey line length of 230 feet. A total of 40 sets of ReMi records were collected for each survey line and then reduced using the SeisOpt ReMi software package (Optim, 2003).

The data files were transformed into a spectral energy Rayleigh-wave frequency versus the inverse of Rayleigh-wave phase velocity (or “slowness”) presentation which captures Rayleigh-wave dispersion caused by the subsurface conditions along the survey line. Points were then picked along the lower bound of the spectral energy Rayleigh-wave velocity dispersion trend at a variety of frequencies. Several inverse modeling iterations were performed to generate the calculated dispersion curve with the least square-root mean square (RMS) error which best fits the selected dispersion curve points. The results of the modeling for survey lines L01, L02 and L03 are summarized in Figure A-2, Shear-Wave Velocity Model, which consist of an interpreted shear-wave velocity profile and two supportive illustrations showing the calculated dispersion curve with data points picked from the Rayleigh-wave phase velocity-frequency image and the image itself with dispersion picks used in the modeling process for each line. It must be understood that this type of geophysical measurement and modeling interpretation may not result in a unique solution. Therefore, the shear-wave velocity models were developed with an understanding of the subsurface conditions based on available geologic information at the site consisting of eight borings performed in 1965 by our legacy company LeRoy Crandall and Associates for the design of the existing facility and stratigraphic profiles interpreted from cone penetration tests (CPTs) performed in 2011 and 2012 by Fugro Consultants. The locations of the CPTs are not shown on Figure A-1 for clarity purposes.

The shear-wave velocity profile of L01 indicates fill materials and/or slightly stiff soils (alluvium) to a depth of about 10 feet (V_s = 770 ft/s). Moderately stiff soil (alluvium) is present from about 10 to 29 feet (Vs = 930 ft/s) and the soft rock (weathered sandstone) present below 29 feet (V_s = 1,030 ft/s). The CPT profile at this location indicates fined grained alluvium to a depth of 10 feet, sandy alluvium from 10 to 22 feet, marine terrace deposits from 22 to 27 feet, and sandstone bedrock below 27 feet. A higher shear-wave velocity layer (V_s = 2,400 ft/s) is also shown as a dashed line below a depth of 85 feet in the profile. However, geologic information at this depth is not available to identify this layer. Based on our geologic information and interpretation in the vicinity of the site, this layer is considered as a local variation in the stiffness of the bedrock and/or possibly an artificial result from the uncertainty of the modeling algorithm, which sometimes appears at the limits of the effective depth of the profile. The average shear wave velocity in the upper 100 feet or 30 meters ($V_{s,30}$) is approximately 1,069 ft/s corresponding to an IBC Site Class “D”.

The shear-wave velocity profile of L02 indicates fill materials and/or soft soils (alluvium) to a depth of about 9 feet (V_s = 560 ft/s). Moderately stiff soil (marine terrace deposits) is present from about 9 to 26 feet (V_s = 980 ft/s) and the soft rock (weathered siltstone) present below 26 feet (V_s =

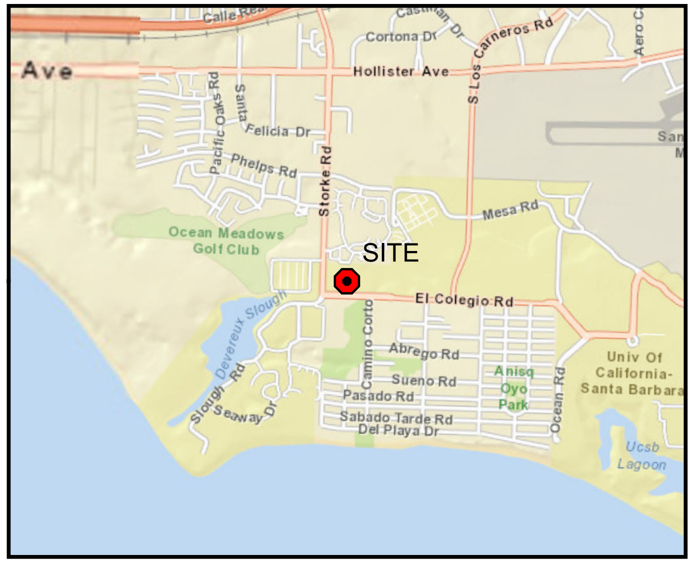
1,040 ft/s). The CPT profile at this location indicates alluvium to a depth of 12 feet, marine terrace deposits between 12 and 26 feet, and siltstone bedrock below 26 feet. The average shear wave velocity in the upper 100 feet or 30 meters ($V_{s,30}$) is approximately 959 ft/s corresponding to an IBC Site Class "D".

The shear-wave velocity profile of L03 indicates soft soils (alluvium) to a depth of about 25 feet (V_s = 640 ft/s). Moderately stiff soil (marine terrace deposits) is present from about 25 to 42 feet (V_s = 870 ft/s) and the soft rock (weathered siltstone) present below 42 feet (V_s = 1,010 ft/s). The CPT profile at this location indicates alluvium to a depth of 24 feet, marine terrace deposits between 24 and 40 feet, and siltstone bedrock below 40 feet. Similar to the profile of L01, a higher shear-wave velocity layer (Vs = 2,040 ft/s) is also shown as a dashed line at a depth of 70 feet in the profile, and it is considered same as above. The average shear wave velocity in the upper 100 feet or 30 meters ($V_{s,30}$) is approximately 992 ft/s corresponding to an IBC Site Class "D".

The mean value of the three Vs_{100} values is 1006.7 ft/s with a standard deviation of 46.1 ft/s, corresponding to a coefficient of variation (COV) of 0.046.

References

Optim L.L.C., 2003, SeisOpt ReMi® v2.0 for Windows 95/98/00/NT/Me/XP, Optim Software and Data Services, UNR-MS-174, 1664 N. Virginia St., Reno, Nevada, 89557.



Symbols

ReMi-L01

Refraction Micrometer (ReMi) survey line

Center of the survey line

Prior Boring Location
(AMEC Job No.: A-65293)

Geologic Units

Qsu - Undifferentiated Surficial Deposits; includes colluvium, slope wash, talus deposits, and other surface deposits of all ages

Ql - Lacustrine, Playa and Estuarine (Paralic) Deposits

Qoa - Old Alluvial Valley Deposits

Qot - Old Terrace Deposits

Contacts

contact, identity and existence certain, location approximate

contact, identity and existence certain, location inferred

Reference: California Geological Survey, 2010, "Geologic Compilation of Quaternary Surficial Deposits in Southern California", vector spatial data, Special Report 217, July 2010
Base: Aerial Photos, Bing, 2012

N

0 50 100 200 300 Feet

amec

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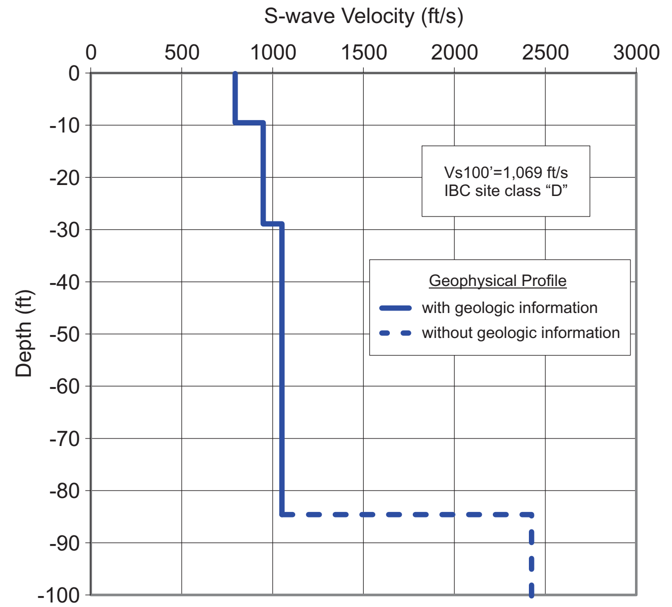
Figure A-1. Geophysical Survey Plan
University of California, Santa Barbara-West Campus
Santa Barbara, California

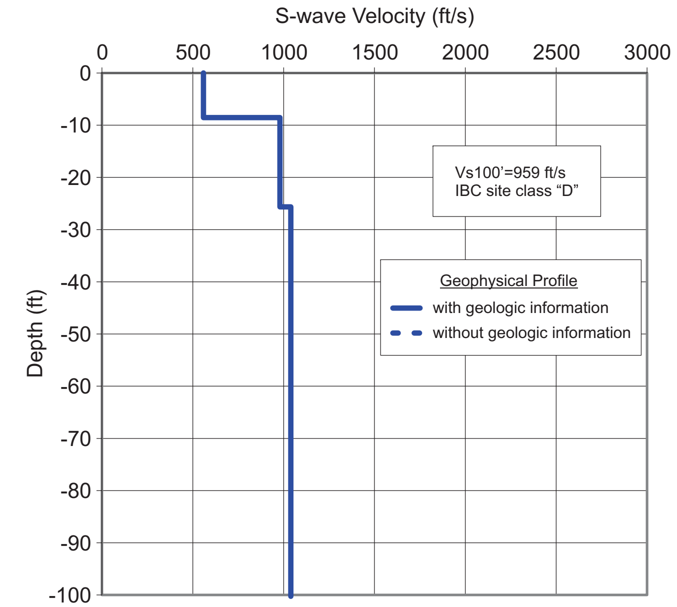
JOB NO.:	4953-12-0132	REVISIONS:
DATE:	05/08/2012	
SCALE:	1 inch = 100 feet	
DRAWN BY:	PWK	
CHECKED BY:	JRK	

Path: G:\4953_Geotech\2012\4953-12-0132_UCSB San Joaquin Apartments Ground Motion Study\GIS\Figure A-1_Site Location in Local Geologic Map.mxd

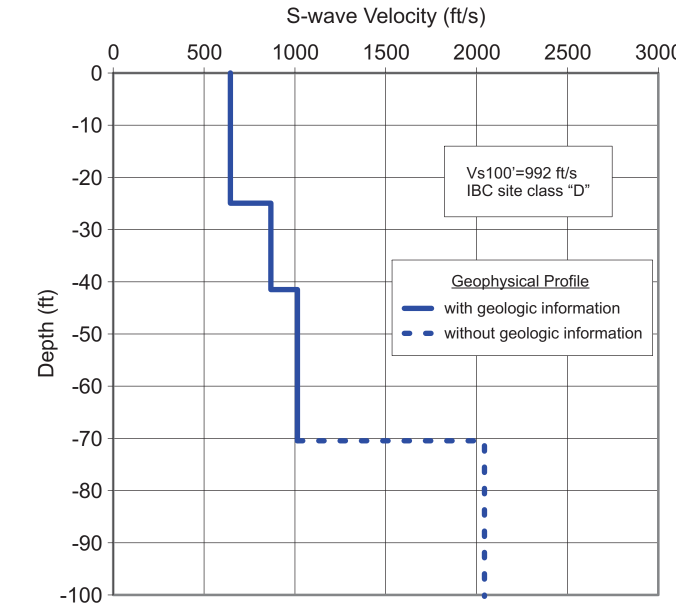
Shear-Wave Velocity Model (L01)



Shear-Wave Velocity Model (L02)

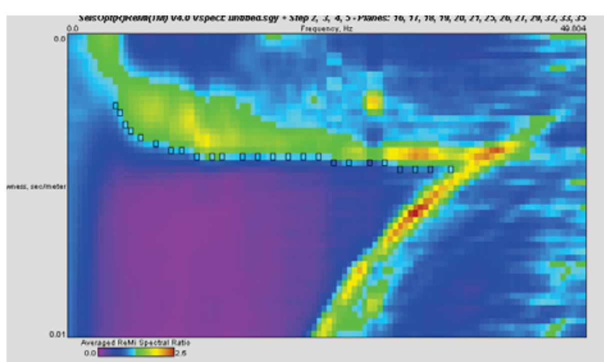


Shear-Wave Velocity Model (L03)

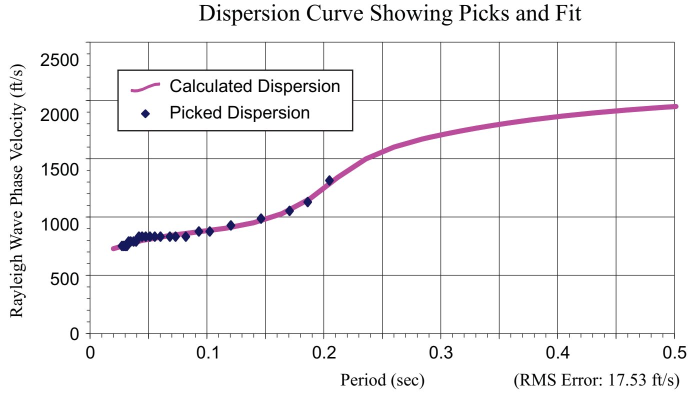


Supportive Illustrations (L01)

p-f Image with Dispersion Modeling Picks

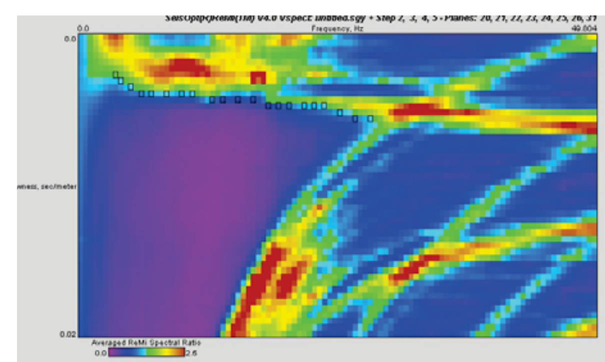


Dispersion Curve Showing Picks and Fit

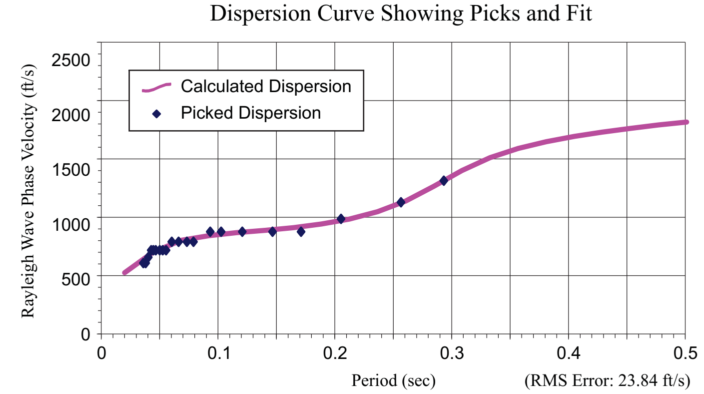


Supportive Illustrations (L02)

p-f Image with Dispersion Modeling Picks

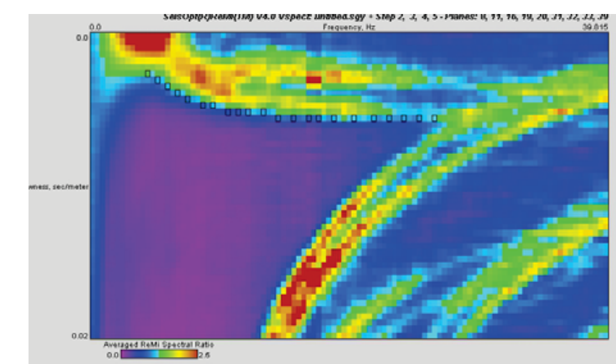


Dispersion Curve Showing Picks and Fit



Supportive Illustrations (L03)

p-f Image with Dispersion Modeling Picks



Dispersion Curve Showing Picks and Fit

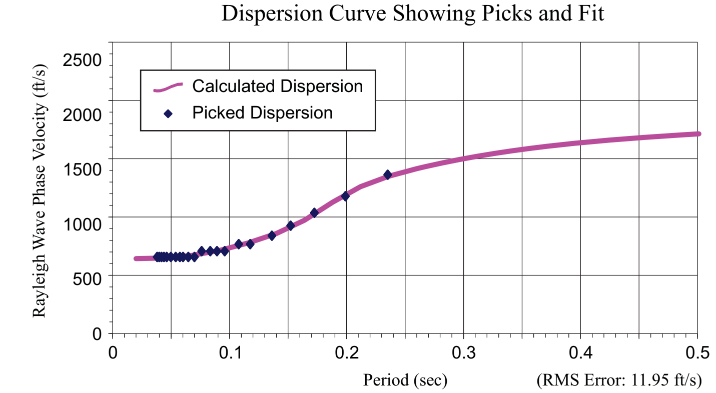


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Figure A-2. Shear-Wave Velocity Model
University of California, Santa Barbara-West Campus
Santa Barbara, California

JOB NO.:	4953-12-0132	REVISIONS:
DATE:	05/09/2012	
SCALE:	As shown	
DRAWN BY:	PWK	
CHECKED BY:	JRK	