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GEOCON

GEOCON PROJECT NO. S9840-05-01

NOVEMBER 2013

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. S9840-05-01
November 21, 2013

Sacramento Basketball Holdings, LLC
c/o ICON Venue Group
8101 E. Prentice Avenue, Suite 900
Greenwood Village, Colorado 80111

Attention: Mr. Tom Noonan

Subject: GEOTECHNICAL INVESTIGATION
SACRAMENTO ENTERTAINMENT AND SPORTS CENTER
SACRAMENTO, CALIFORNIA

Dear Mr. Noonan:

In accordance with your authorization, we have performed a geotechnical investigation for the proposed Sacramento Entertainment and Sports Center located between 5th and 7th and J and L Streets in Downtown Sacramento, California.

The accompanying report presents our findings, conclusions, and recommendations pertaining to the geotechnical aspects of designing and constructing the project as presently proposed. This report provides a seismic hazards evaluation and design-level geotechnical recommendations for the proposed improvements. In our opinion, no adverse geotechnical, geologic, or seismic conditions are present that would preclude development at the site provided the recommendations of this report are incorporated into design and construction of the project. The primary geotechnical constraints identified at the site include soft, compressible silt, liquefaction-susceptible sand, and shallow groundwater.

Please contact us if you have any questions concerning the contents of this report. We look forward to reviewing the project plans as they develop further, providing engineering consultation as needed and performing geotechnical observation and testing services during construction.

Sincerely,

GEOCON CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
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GEOTECHNICAL INVESTIGATION

1.0 INTRODUCTION AND SCOPE OF SERVICES

This report presents the results of our geotechnical investigation for the proposed Sacramento Entertainment and Sports Center (ESC) to be located within the east-central portion of the existing Downtown Plaza development roughly within the area bounded by J and L Streets and 5th and 7th Streets in Downtown Sacramento, California. The approximate site location is shown on the Vicinity Map, Figure 1.

The purpose of our investigation was to explore and evaluate subsurface soil, geologic, and seismic conditions at the site and provide conclusions and design-level recommendations for the project as presently proposed. We should review the project plans as they develop further, provide additional geotechnical engineering consultation as needed, and perform geotechnical observation and testing services during construction.

In conjunction with our geotechnical investigation, we collected soil and groundwater samples from seven of the borings for laboratory analysis for environmental contaminants of concern (COCs) including: metals, petroleum, and volatile organic compounds (VOCs). Laboratory test results and discussion of our findings, conclusions, and recommendations are presented in our *Limited Phase II Environmental Site Assessment* report, presented under separate cover.

To prepare this report, we performed the following scope of services:

- Reviewed area geologic maps and other technical literature pertaining to the site and vicinity (see References in Section 9.0 of this report).
- Reviewed preliminary development plans and project descriptions provided by the design team.
- Attended various project design and coordination meetings.
- Performed a site reconnaissance to review project limits, determine exploration equipment access, and mark out exploratory excavation locations.
- Notified subscribing utility companies via Underground Service Alert (USA) a minimum of 48 hours (as required by law) prior to performing exploratory excavations at the site.
- Paid required fees and obtained a subsurface exploration permit from the Sacramento County Environmental Management Department (SCEMD).
- Paid required fees and obtained an encroachment permit from the City of Sacramento to perform exploratory excavations within City right-of-way (ROW).
- Retained the services of a private utility locator to further delineate subsurface utilities near our proposed exploration locations.
- Performed five exploratory borings (B1 through B5) with a truck-mounted drill rig equipped with hollow-stem auger and rotary-wash drilling equipment to depths ranging from approximately 61 to 120 feet.

- Performed six exploratory borings within the interior of the lower parking level (IB1 through IB6) using limited-access, low-overhead drilling equipment.
- Performed two dynamic cone penetrometer (DCP) soundings (DCP4 and DCP6) near borings IB4 and IB6 using a Wildcat DCP.
- Performed five cone penetration test (CPT) soundings (CPT1 through CPT5) using truck-mounted equipment to approximate depths ranging from 50 to 80 feet. Performed shear wave velocity measurements in two of the CPT soundings (CPT1 and CPT5).
- Performed two dilatometer test (DMT) soundings (DMT1 and DMT2) using truck-mounted equipment to approximate depths of 37 and 50 feet.
- Logged the exploratory borings in accordance with American Society for Testing and Materials (ASTM) D2487 which is based on the Unified Soil Classification System (USCS).
- Upon completion, backfilled the borings, CPTs, and DMTs with cement grout in accordance with SCEMD permit requirements.
- Performed laboratory tests to evaluate pertinent geotechnical parameters.
- Prepared this report summarizing our findings, conclusions, and recommendations regarding the geotechnical aspects of designing and constructing the project as presently proposed.

Approximate locations of our subsurface explorations are shown on the Site Plan, Figure 2, and Cross-Sections A-A’ and B-B’, Figures 3 and 4, respectively. Details of our field exploration program including exploratory boring logs, DCP logs, CPT logs, and DMT logs are presented in Appendix A. Details of our laboratory testing program and test results are summarized in Appendix B. Shear wave velocity measurements/profiles and correlated engineering parameters are presented in Appendix C. Results of our liquefaction settlement analysis are presented in Appendix D. Results of our LPILE analysis (laterally loaded pile analysis) are presented in Appendix E.

2.0 SITE AND PROJECT DESCRIPTION

The ESC will be the home for the National Basketball Association (NBA) Sacramento Kings and will also be host to other sporting events and a variety of family and cultural activities. As currently envisioned, the new ESC will be located within the central-eastern portion of the existing Downtown Plaza development roughly within the area bounded by J and L Streets and 5th and 7th Streets in Downtown Sacramento (Vicinity Map, Figure 1).

The existing Downtown Plaza development includes several two- to four-level retail/commercial buildings and two levels of subterranean (below-grade) parking (referred to as “lower level” and “upper level”). The Downtown Plaza commercial buildings are supported on the “roof” level of the parking structure. Based on our review of the plans provided (*Downtown Plaza Parking Structure, Project of the Redevelopment Agency of the City of Sacramento for Downtown Plaza Properties*, prepared by Conrad Engineers, dated November 11, 1975), the lower level is at elevation +10.00 feet, the upper level is at elevation +19.67 feet, and the roof level is at elevation +29.67 feet (datum unspecified). The recent topographic map prepared by Morton & Pitalo (project civil engineer) shows spot elevations of the lower level near the east end of the site to range from approximately +11.7 feet to +11.9 feet, and roof level elevations ranging from approximately +31.8 feet to +32.2 feet, based on the North American Vertical Datum of 1988 (NAVD 88). Therefore, the original plans and current topographic map are based on different datums. *Elevations stated in this report are based on the recent Morton & Pitalo topography, which is based on NAVD 88.*

Based on the 1975 plans, foundations for the existing Downtown Plaza consist of vertical and battered driven concrete and concrete-filled pipe piles with a reported single pile axial capacity of 60 tons. The plans note that "*if 60 tons is not obtainable, in certain areas, 45 ton piles shall be used.*" The lengths of the existing piles are not indicated on the plans. The piles are arranged in groups of 3, 4, 5, 6 and 8 with rectangular and hexagonal pile caps. The plans also show alternate pile cap plans that allow additional piles if needed. Pile caps are connected by a system of reinforced concrete grade beams/tie beams of various sizes. The lower parking level floor is reported to consist of 2 inches of asphalt concrete (AC); which is consistent with conditions encountered in our borings (see Section 3.2). Four, 6-foot-diameter (approximately) combination drainage and ventilation tunnels are located below the lower level floor and traverse the site west to east with an invert elevation of approximately +0.5 feet, which places the tunnel inverts approximately 9 to 10 feet below the lower level. We understand that each tunnel flows to a dedicated sump/wet well equipped with pumps that operate as needed to remove water, which we understand is several times throughout the year.

The eastern portion of the parking structure (below the current East Macy's, 24-hour Fitness, and 660 J Street) was constructed at a later date, and plans were not provided for our review. This portion of parking structure is located at similar elevations to the western portion; however, the lower level floor consists of a concrete slab instead of AC. We do not know the foundation types and if the ventilation/drainage tunnels extend under this area.

Based on the topographic map prepared by Morton & Pitalo, the elevation of the lower parking level, which encompasses the majority of the site, ranges from approximately +11.5 feet to +12.2 feet NAVD 88. The street-level elevations surrounding the site vary: J Street is approximately +30 feet NAVD 88; L Street is approximately +22 feet NAVD 88; 7th Street generally slopes downward from J Street on the north (+30 feet NAVD 88) to L Street on the south (+22 feet NAVD 88); and 5th Street slopes down from J Street on the north (+30 feet NAVD 88) to a low of approximately +12 feet NAVD 88 as it passes under the Downtown Plaza bridge structure and ascends to approximately +21 feet NAVD 88 as it approaches L Street. Site topography is shown on the Site Plan, Figure 2.

The ESC will have an approximate structure footprint of 235,000 square feet and a total gross area of approximately 674,000 square feet. The project will require demolition and removal of the current buildings, parking facilities, and related infrastructure. Adjacent buildings and infrastructure will remain. The ESC will have multiple levels above the event floor, which is currently anticipated to be approximately five feet below the existing lower parking level, at an approximate elevation of +5.6 feet

NAVD 88. In general, construction will require mass excavating approximately 11 feet below the current lower parking level within the footprint of the ESC. The new ECS structure will likely be of structural steel, braced-frame construction with variable structural loading, currently estimated in the range of 200 kips to 3,000 kips per column (service load). Because of the magnitude of structural loads and the subsurface soil conditions, a deep foundation system will be required.

The event floor will likely consist of a pile-supported structural concrete slab (mat slab) with adjacent areas consisting of conventional (non-structural) concrete slabs-on-grade. Since a permanent dewatering system will likely not be permitted by the City, the structure will likely employ a “bathtub” (hydrostatic) design with waterproofing. Based on preliminary design details provided by the design team (dated November 8, 2013), the event floor section may consist of an 18- to 24-inch mat slab designed to span between pile caps. The mat slab will be underlain by a single-ply waterproofing membrane and overlain by 24 inches of crushed concrete fill (derived from demolition operations) and topped with a 6-inch concrete slab. Building utilities (water, sewer, electric) will be located within the crushed concrete fill section.

Other proposed improvements will likely include new underground utility infrastructure and street-level improvements such as sidewalks, courtyards, and landscaping. Grading and improvement plans were not available as of the date of this report. We anticipate that the majority of grading and earthwork will involve mass excavation to attain design grades. We do not anticipate significant fill placement to raise grades around the project area. The approximate proposed footprint of the ESC is shown on the Site Plan, Figure 2.

3.0 SOIL AND GEOLOGIC CONDITIONS

We identified soil and geologic conditions by observing exploratory borings and reviewing referenced geologic literature (Section 9.0). The soil descriptions provided in this report include the USCS symbol where applicable. General subsurface profiles through the site are presented as Cross-Sections A-A’ and B-B’, Figures 3 and 4.

3.1 Regional and Local Geology

The site is located within the Great Valley Geomorphic Province of California, more commonly referred to as the Central Valley. The Central Valley is a broad depression bounded by the Sierra Nevada mountain range to the east and the Coast Ranges to the west. The valley has been filled with a thick sequence of sediments derived from weathering of the adjacent mountain ranges resulting in a stratigraphic section of Cretaceous, Tertiary, and Quaternary deposits.

The site is located near the southern end of the Sacramento Valley, approximately one-quarter mile east of the Sacramento River, approximately one mile south of the confluence of the American River. Alluvial sediments at the site have been deposited primarily during flood stages of the Sacramento and American Rivers.

3.2 Existing Pavement Sections

Our borings were performed in paved areas comprised of hot mix asphalt (HMA) or Portland cement concrete (PCC) overlying aggregate base (AB) or ¾-inch open-graded crushed rock. Table 3.2 summarizes the pavement section material thicknesses encountered in our borings.

TABLE 3.2
SUMMARY OF EXISTING PAVEMENT SECTIONS

Boring ID	Location[1]	HMA[2] (inches)	PCC[3] (inches)	AB[4] (inches)	Open-Graded ¾-inch Crushed Rock
B1	5th Street near L Street	---	8	18	---
B2	5th Street near J Street	---	8½	15½	---
B3	6th Street near Scientology Building	7	---	8	---
B4	K Street / L Street Alley	---	9	---	---
B5	6th Street Entrance to Lower Parking Level	---	8	---	---
IB1	Lower Parking Level	3	---	3	---
IB2	Lower Parking Level	3	---	3	---
IB3	Lower Parking Level	2	---	---	---
IB4	Lower Parking Level	---	6	---	6
IB5	Lower Parking Level	3½	---	12	---
IB6	Lower Parking Level	2½	---	4	---

Notes:
1. *Approximate locations shown on the Site Plan, Figure 2*
2. *HMA = Hot Mix Asphalt*
3. *PCC = Portland cement concrete*
4. *AB = Aggregate Base*
5. *--- = Not Encountered*

3.3 Fill

Below the existing pavement section, we encountered fill in borings B3 and B4 to depths ranging from approximately 10 to 12 feet. We also encountered fill in boring IB4, performed within the lower parking level below the East Macy's building. The fill encountered in boring B3 was presumably placed to raise grades above the common flood level during the early development of Sacramento and generally consisted of poorly-graded fine sand (SP) with occasional brick and wood debris. The fill encountered in boring B4 generally consisted of open-graded ¾-inch crushed gravel and was likely placed as backfill against the retaining walls of the adjacent subterranean parking area. The fill encountered in boring IB4 appears to be isolated (i.e. fill not observed in other interior borings) and generally consisted of sandy silt (ML) with brick, porcelain, and wood fragments and a slight petroleum hydrocarbon odor. Given the site history, it is possible that other areas of fill not identified in our borings are present at the site. We expect that the lateral/vertical extents, composition, and characteristics of the existing fill at the site are highly variable.

3.4 Alluvium

Below the fill, where present, and below the existing pavement sections elsewhere, we encountered alluvial soils. The alluvium can be subdivided into two distinct units: "recent" alluvium and "older" alluvium as described herein. The thickness of individual soil layers vary across the site (see Cross-Sections A-A' and B-B", Figures 3 and 4). The depths used in the following descriptions are referenced to the proposed event floor elevation (+5.6 feet NAVD 88) within the central portion of the arena, generally at Boring B5.

- 0 to 30 feet: "recent" alluvium consisting of interbedded, soft to medium-stiff silt (ML) with thin lenses of silty sand (SM). This material has high in-situ moisture content (approximately 30% to 40%) and is compressible under increased loading.
- 30 to 45 feet: "recent" alluvium consisting of loose to medium dense, poorly graded sand with fines content ranging from approximately 4 to 9%. This material is susceptible to liquefaction during the design-level seismic event.
- 45 to 75 feet: "older" alluvium consisting of dense to very dense poorly graded gravel (GP) with sand and poorly graded sand (SP) with variable amounts of fine gravel. This layer is generally considered the "first" bearing layer for deep foundations in Downtown Sacramento. However, the composition of this layer is variable. Some areas contain a heavy concentration of large gravel and small cobble (borings B2, B3 and B4) and other areas contain very little, if any, gravel (borings B1 and B5).
- 75 feet and deeper: "older" alluvium consisting of interbedded layers of very dense, partially cemented silt (ML) and silty sand (SM). This layer comprises the "second" bearing layer for deep foundations in Downtown Sacramento.

Soil and geologic conditions described in the previous paragraphs and shown on Cross-Sections A-A' and B-B' (Figures 3 and 4), are generalized. Therefore, the reader should consult the boring logs included in Appendix A for soil type, color, moisture, consistency, and USCS classification of the soils encountered at specific locations and elevations. DCP, CPT and DMT logs are also presented in Appendix A. Shear wave velocity profiled and correlated engineering parameters are presented in Appendix C.

4.0 GROUNDWATER

Table 4.0 summarizes the groundwater conditions encountered in our borings performed during the period of October 9 to 23, 2013.

TABLE 4.0
GROUNDWATER OBSERVATIONS

Boring ID	Date	Boring Depth (feet)	Approximate Boring Elevation (feet NAVD 88)	Depth to Groundwater[1] (feet)	Approximate Groundwater Elevation (feet NAVD 88)
B1	10/17/13	101.5	19.5	15	+4.5
B2	10/16/13	61.5	26.5	29	-2.5
B3	10/15/13	72.5	28.5	25	+3.5
B4	10/17/13	76.5	21.5	23	-1.5
B5	10/18/13	121.5	12.5	9	+3.5
IB1	10/22/13	47.5	11.5	7.5	+4
IB2	10/23/13	12	11.5	7.5	+4
IB3	10/23/13	3 (refusal)	11.5	n/a	n/a
IB4/DCP4	10/09/13	53	11.5	8	+3.5
IB5	10/22/13	42	11.5	7.5	+4
IB6/DCP6	10/09/13	45	11.5	7.5	+4

Notes:

1. *Depth to groundwater was measured with a cloth tape upon withdrawing augers from the boring (accuracy within 1 foot). Since measurements were made a short time after drilling, they may not represent stabilized groundwater elevations.*

Review of the *Spring 2007 Sacramento County Groundwater Elevation Map* (County of Sacramento, Water Resources Division, April 2007) indicates that the average springtime (seasonal high) groundwater elevation in the site vicinity is approximately +0 feet mean sea level (MSL) which is approximately +2.2 feet NAVD 88, which corresponds to approximately 20 to 30 feet below street level or about 12 feet below the lower parking level.

Bi-annual depth to groundwater measurements (Spring and Fall) from four existing groundwater wells (SPW-03, SPW-13, SPW-26R, and SPW-14) that generally surround the site were available on the State of California Water Resources Control Board *GeoTracker* online database (http://geotracker.waterboards.ca.gov/). These groundwater measurements were available for the 11-year period between April 2001 and November 2012. We normalized the depth measurements to elevation (based on the surface elevations presented on the well logs, which we assume were referenced to MSL) and plotted the results on Figure 5. As shown on Figure 5, the average groundwater elevation measured in these wells fluctuates seasonally between about elevation +2 feet and +12 feet NAVD 88 with an average around +6 feet NAVD 88. The relative average high groundwater elevation of +12 feet NAVD 88 occurred in April 2006, and the relative average low groundwater elevation of +1 feet NAVD 88 occurred in October 2003 and November 2007.

We understand that Viking Drilling is currently installing several groundwater monitoring wells around the project site to observe depth to groundwater over the next 12 months to aid in evaluating project dewatering conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors. Groundwater depth at the site is likely strongly influenced by the level of water in the adjacent Sacramento River.

5.0 GEOLOGIC HAZARDS

5.1 Regional Active Faults

The numerous faults in Northern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Division of Mines and Geology (CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Hart, 1999). An active fault has experienced surface displacement within the last 11,000 years. A potentially active fault has experienced surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known movement within the past 11,000 years. Faults that have not moved in the last 1.6 million years are considered inactive. Based on our review of geologic maps and reports, the site is not within a currently established Alquist-Priolo (AP) Earthquake Fault Zone. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the project is considered low.

The Northern California region is considered seismically active, and the site could be subjected to ground shaking in the event of an earthquake on one of the many active Northern California faults. Table 5.1 summarizes the distance of known active faults within 62 miles of the site, based on the computer program *EQFAULT* (Version 3, Blake, 2000).

TABLE 5.1
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance from Site (miles)	Maximum Earthquake Magnitude, M_w
Foothills Fault System	22	6.5
Great Valley, Segment 3	27	6.9
Great Valley, Segment 4	27	6.6
Great Valley, Segment 5	30	6.5
Hunting Creek – Berryessa	39	7.1
Concord – Green Valley	39	6.9
Great Valley, Segment 6	39	6.2
West Napa	48	6.5
Greenville	52	6.9
Great Valley, Segment 2	55	6.4
Rodgers Creek	61	7.0
Bartlett Springs	61	7.1
Calaveras (No. of Calaveras Res)	61	6.8

5.2 Historical Earthquakes and Ground Shaking

The Sacramento region of Northern California has a history of relatively low seismicity in comparison with more active seismic regions such as the Bay Area or Southern California. The two most commonly referred to earthquakes that resulted in some reported building damage in Downtown Sacramento are the Winters and Vacaville events in 1892. There are no reported occurrences of seismic-related ground failure in the Sacramento region due to earthquakes.

We used the United States Geological Survey (USGS) computer program *2008 Interactive Deaggregations* to estimate the peak ground acceleration (PGA) and modal (most probable) magnitude associated with the Maximum Considered Earthquake (MCE) with a 2,475-year return period. The USGS estimated PGA is 0.37g and the modal magnitude is 6.77.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site. The site could be subjected to ground shaking in the event of an earthquake along the faults mentioned above or other area faults.

5.3 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with earthquakes. Primary factors that trigger liquefaction are: strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions.

The site is not located in a currently established State of California Seismic Hazard Zone for liquefaction. In addition, we are not aware of any reported historical instances of liquefaction in the greater Sacramento area. However, soil and groundwater conditions exist at the site that may be susceptible to seismic-induced liquefaction under the design-level seismic event.

The current standard of practice, as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California"* requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. We used the computer software program CLiq (Version 1.7, Geologmiski) and the in-situ soil parameters measured in the CPT soundings. The software utilizes the 1998 National Center for Earthquake Engineering Research (NCEER) method of analysis which was developed with the broad consensus of national geotechnical earthquake engineering experts. We assumed a groundwater depth of 0 (zero) feet, an earthquake magnitude of 6.77, and a PGA of 0.31g (PGA_M adjusted for Site Class per ASCE 7-10, Eq. 11.8-1). Results of our liquefaction analysis are presented in Appendix D.

Based on the results of our analyses, there is the potential for liquefaction at the site within a sandy soil layer generally present between depths of approximately 25 to 40 feet below the event floor level (approximately Elevations -20 feet to -35 feet NAVD 88, see Cross-Sections A-A' and B-B'). The overlying silt layer and underlying dense sand and gravel layer does not appear to be susceptible to liquefaction.

Consequences of liquefaction may include ground surface settlement, ground loss (sand boils), and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert a large enough force to break through overlying, non-liquefiable layers. Based on work by Youd and Garris (1995) which modified and advanced the work of Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils or fissures. An apparently continuous, non-liquefiable, silt stratum at least 30 feet thick is present across the site. In our opinion, this layer is sufficiently thick to reduce the potential for ground loss due to sand boils or fissures.

Therefore, the likely consequence of potential liquefaction at the site is ground surface settlement, which could impart negative skin friction (downdrag) loads on the deep foundations after a liquefaction event. Dynamic settlement of the soils that experience liquefaction may occur after earthquake shaking has ceased. We estimated potential dynamic settlements of liquefied soil layers using the methodology developed by Tokimatsu and Seed (1987). The results of the analysis indicate potential total liquefaction settlements of approximately 1 to 5 inches. Conventional geotechnical practice is to assume that differential settlements may approach 50 percent of the calculated total settlement.

5.4 Lateral Spreading/Dynamic Stability

Lateral spreading is a dynamic instability phenomenon in which soil moves laterally during seismic shaking and is often associated with liquefaction. Areas subject to lateral spreading are typically underlain by liquefiable soil and can be gently sloping areas or flat areas adjacent to "free face" geometry. Given the apparently variable and discontinuous nature of the liquefiable soil at the site and the lack of free face geometry near the site, the potential for significant lateral spreading is low.

5.5 Seismic Site Classification

In accordance with the 2013 California Building Code (CBC), we evaluated seismic Site Class in accordance with Chapter 20 of ASCE 7-10. As discussed in Section 5.3 of this report, there is a potential for liquefaction at the site; however, there is a low potential for failure or collapse (sand boils/ground loss), therefore Site Class "F" does not apply. Based on the subsurface conditions encountered in our borings and CPT/DMT soundings, the three required criteria for Site Class "E" (Section 20.3.2 of ASCE 7-10) do not apply. Therefore, we evaluated seismic Site Class on the basis of in-situ shear wave velocity measurements performed in conjunction with CPT2 and CPT5. Results are presented in Appendix C (Figures C1 and C2). The calculated average shear wave velocity (per ASCE 7-10, Eq. 20.4-1) ranges from approximately 640 to 780 feet/sec. Therefore, the site is classified as Site Class "D" per Table 20.3-1 of ASCE 7-10. Seismic design criteria based on Site Class "D" per the 2013 CBC are provided in Section 5.6.

5.6 2013 CBC Seismic Design Criteria

Table 5.6 summarizes seismic design criteria per the 2013 CBC.

TABLE 5.6
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC / ASCE 7-10 Reference
Site Class	D	--- / Table 20.3-1
S_S – Mapped Spectral Response Acceleration for Short Period	0.680g	Figure 1613.3.1(1) / Figure 22-1
S_1 – Mapped Spectral Response Acceleration for 1-sec Period	0.295g	Figure 1613.3.1(2) / Figure 22-2
F_a – Short Period Site Coefficient	1.256	Table 1613.3.3(1) / Table 11.4-1
F_v – Long Period Site Coefficient	1.811	Table 1613.3.3(2) / Table 11.4-2
S_{MS} – Maximum Considered Earthquake Spectral Response Acceleration, 5% damped, 0.2-sec period, adjusted for Site Class	0.854g	Eq. 16-37 / Eq. 11.4-1
S_{M1} – Maximum Considered Earthquake Spectral Response Acceleration, 5% damped, 1-sec period, adjusted for Site Class	0.533g	Eq. 16-38 / Eq. 11.4-2
S_{DS} – Design Earthquake Spectral Response Acceleration, 5% damped, 0.2-sec period	0.569g	Eq. 16-39 / Eq. 11.4-3
S_{D1} – Design Earthquake Spectral Response Acceleration, 5% damped, 1-sec period	0.356g	Eq. 16-40 / Eq. 11.4-4
MCE_G **PGA** - Maximum Considered Earthquake Geometric Mean for Site Class B	0.232g	--- / Figure 22-7
PGA_M – MCE_G PGA adjusted for Site Class	0.31g	--- / Eq. 11.8-1

Conformance to the criteria in Table 5.6 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

5.7 Expansive Soil

Laboratory Plasticity Index and Expansion Index test results for the fine-grained soils at the site indicate low expansion potential. Mitigation and/or special design considerations with respect to expansive soil is not necessary for the project.

5.8 Soil Corrosion Screening

We performed a soil corrosion potential screening by conducting laboratory testing on a representative near-surface soil sample. The laboratory test results and published screening levels are presented in Appendix B.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General Discussion

6.1.1 No soil or geologic conditions were encountered during our investigation that would preclude development of the site as planned, provided the recommendations contained in this report are incorporated into design and construction of the project.

6.1.2 Based on our findings, evaluation, and analyses to date, we have identified the following geotechnical constraints for the site:

- Soft, compressible silt soil within the top 25 feet
- Potentially liquefiable sand between 25 and 40 feet
- Shallow groundwater

These conditions impact design and construction of the project. Discussion of these geotechnical constraints and specific mitigation, design, and construction recommendations are provided herein.

6.1.3 Due to the presence of soft, compressible silty soil and underlying potentially liquefiable sand, and the magnitude of anticipated structural loading, the use of shallow foundations for support of the structure is not feasible due to excessive post-construction settlement and potential liquefaction-induced settlement. Therefore, the ESC structure should be supported on deep foundations bearing within the dense sand gravel and underlying layers at approximately 40 to 50 feet below proposed event floor elevation (approximate elevations -35 feet to -45 feet NAVD 88. Possible deep foundation types currently under consideration include augercast pressure-grout displacement (APGD) piles, conventional augercast pressure-grout (APG) piles and drilled-displacement concrete filled pipe piles (DDCFPP). These deep foundation types are designed and installed by specialty geotechnical contractors. General recommendations are provided herein.

6.1.4 Due to the compressible nature of the silts within 25 feet of proposed grade, placing additional fill around the project area may induce settlement. If the project requires placing additional fill around the perimeter of the proposed arena, we should review the proposed details and evaluate the potential settlement-related impacts.

6.1.5 The foundations for the existing Downtown Plaza consist of vertical and battered driven concrete and concrete-filled pipe piles with a reported single pile axial capacity of 60 tons. The actual as-built lengths of the existing piles are unknown. If re-use of the existing foundation system is planned, pile integrity and dynamic load testing should be performed to confirm the as-built length(s) and capacity of the existing piles. We can provide specific recommendations for pile testing if needed.

6.1.6 In conjunction with our geotechnical investigation, we collected soil and groundwater samples from seven of the borings for laboratory analysis for environmental contaminants of concern (COCs) including: metals, petroleum, and volatile organic compounds (VOCs). Laboratory test results and a detailed discussion of our findings are presented in our *Limited Phase II Environmental Site Assessment* report, presented under separate cover. A general summary follows. Laboratory analysis of the samples indicate that the primary COCs were petroleum and VOCs in groundwater. No significant concentrations of the COCs were detected in the soil samples. Petroleum in the groundwater samples was identified by the lab as diesel and oil-range organics (DRO and ORO). No gasoline-range organics were detected. DRO and ORO were detected at concentrations exceeding regulatory screening levels for drinking water in four of seven shallow groundwater samples collected and analyzed. VOCs including cis-1,2-dichloroethene, 1,1-dichloroethane, and vinyl chloride were also detected in two of the seven groundwater samples at concentrations exceeding drinking water criteria. Shallow groundwater beneath the site will not be a source of drinking water for the site; however, the presence of petroleum and VOCs in shallow groundwater indicate that water produced from construction dewatering and soil/water mixtures generated during deep foundation construction will have to be properly managed. Groundwater pumped from and around the site for dewatering operations will likely need to be temporarily stored, profiled (sampled and analyzed), pre-treated if necessary and then discharged in accordance with a specific water discharge permit to be obtained by the contractor's dewatering subcontractor. Saturated soil cuttings potentially generated during deep foundation construction will also need to be temporarily contained, profiled, then disposed of offsite. Offsite disposal could include reuse as fill on other properties if the material is found to not be impacted with COCs or disposal at a licensed landfill. As part of the deep foundation design for the project, we recommend performing a comprehensive installation and testing program (see Section 6.9.4) to verify constructability, installation production rates, and structural capacity. Samples of the soil cuttings generated during test pile installation should be collected and analyzed for the COCs to evaluate offsite disposal and/or reuse options.

6.1.7 Because of the anticipated potential liquefaction-induced settlement (estimated to range from 1 to 5 inches), we recommend that the event level floor consist of a structural slab designed to span between pile caps. Assuming some degree of damage is acceptable; slabs for ancillary areas may consist of conventional (non-structural) slabs-on-grade. Potential damage may include significant cracking and displacements that may require extensive repairs and/or replacement. The foundation recommendations presented in this report are intended to mitigate and design for the effects of liquefaction settlement on the proposed structure.

6.1.8 Soft, wet alluvial silt is anticipated to be exposed throughout the mass excavation for the ESC structure. We anticipate that these soils will be extremely unstable under construction equipment traffic. Use of rubber-tire construction equipment on these soils may further deteriorate conditions and equipment may become stuck. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance. Stabilization of the bottom of the excavation will likely be required in order to provide a firm working surface upon which heavy equipment can operate. Preliminary stabilization recommendations are provided herein.

6.1.9 Groundwater will likely be encountered during mass excavation, pile cap excavations, underground utility construction, and deep foundation construction. Due to the proposed depth of construction excavations, temporary dewatering measures will be required to control groundwater seepage during excavation and construction. Recommendations for temporary construction dewatering are provided herein.

6.1.10 We understand that ESC structure will be designed and waterproofed to withstand hydrostatic pressure from the seasonally fluctuating groundwater. This type of design (fully waterproofed “bathtub”) will generate uplift forces as a result of groundwater which can seasonally rise as high as +12 feet NAVD 88. These uplift forces must be accounted for in the project design. Temporary dewatering must be maintained during construction until the building loads are heavy enough to resist the buoyant forces or the appropriate anchoring is in place.

6.1.11 Permanent waterproofing of subterranean walls and slabs will be required. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method that would provide protection to subterranean walls, floor slabs and foundations. In addition, an experienced waterproofing inspector should be retained to check proper installation of the system during construction.

6.1.12 Improvements which are not supported on deepened foundations, such as walkways, paving, and utilities, may still be subject to seismic and/or static settlement. The client should consider the flexibility of the products and pavements being installed. Interlocking pavers typically conform to differential settlement and are easily repaired. Utilities traversing through existing site soils should use flexible connections in order to minimize the damage to underground installations caused by potential soil movements.

6.1.13 Construction of ESC structure will involve mass excavation below the existing lower parking level by approximately 11 feet. Due to the proximity of adjacent improvements (streets, sidewalks, structures) excavation sloping will likely not be possible and shoring will be required. There are many options for temporary earth retention, such as soldier pile and lagging, sheet piles, cast-in-place tangent piers, and others. Preliminary recommendations are provided herein.

6.1.14 Due to the presence of shallow groundwater at the site, onsite stormwater infiltration into the ground is not considered feasible and should not be used for this project.

6.1.15 We should be retained to review the project plans as they develop further, provide engineering consultation as-needed, and perform geotechnical observation and testing services during construction. The recommendations contained in this report are preliminary until verified during construction by representatives of our firm.

6.2 Excavation Characteristics

6.2.1 In our opinion, site soils can be excavated with light to moderate effort using conventional heavy duty grading equipment. Slumping and caving should be expected in un-shored excavations, especially where saturated or granular soils are encountered.

6.2.2 Optimum moisture content for the silty soil expected to be exposed in the project mass excavation is approximately 15%. Measured in-situ moisture content for soil encountered in our borings ranged from approximately 30% and 40%, which is significantly above optimum. Therefore, significant instability should be expected in the mass excavation bottom. Preliminary stabilization recommendations are provided on Section 6.6.

6.2.3 Earthwork and underground utility contractors should be aware of the high in-situ moisture content, moisture sensitivity, and potential compaction/workability difficulties. The contractor should expect that, at a minimum, aerating/drying soils will be required to achieve proper compaction. If aerating/drying the soils is too slow based on weather conditions at the time, lime- or cement-treatment may be an alternative. We should evaluate unstable soil conditions in the field at the time of construction and determine the type, level, and extent of mitigation alternatives as necessary.

6.2.3 Project excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as shoring.

6.2.4 Temporary excavation slopes must meet Cal-OSHA requirements as appropriate. We anticipate that the majority of excavations will be classified as Cal-OSHA "Type C" soil. Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and to make appropriate recommendations where necessary. It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements, which may be damaged by earth movements.

6.2.5 The excavation support recommendations provided by Cal-OSHA are generally geared towards protecting human life and not necessarily towards preventing damage to nearby structures or surface improvements. The contractor should be responsible for using the proper active shoring systems, sloping, or underpinning measures to prevent damage to any structure or improvements located near excavations.

6.3 Groundwater and Construction (Temporary) Dewatering

6.3.1 The average groundwater elevation measured in four nearby wells fluctuates seasonally between about elevation +1 feet and +12 feet NAVD 88 with an average around +6 feet NAVD 88 (Figure 5). The average groundwater elevation measured in our borings performed during the period of October 9 through 23, 2013, was approximately +4 feet NAVD 88. Based on this information, we recommend assuming a seasonal high groundwater elevation of +12 feet NAVD 88 for design purposes.

6.3.2 We understand that Viking Drilling (dewatering contractor) is planning to install several groundwater monitoring wells around the project site to further evaluate depth to groundwater over the next 12 months; therefore, additional groundwater elevation information will become available as the project design progresses.

6.3.3 Given the anticipated groundwater conditions, temporary dewatering will likely be necessary during construction activities associated with the subterranean event level and associated pile cap/underground utility excavations. To increase excavation bottom stability, groundwater levels should be lowered at least 3 feet below planned excavation depths. *However, to reduce potential settlement-related impacts to adjacent structures, utilities, and improvements, groundwater should not be lowered below elevation zero feet NAVD 88.* This amount of dewatering should be sufficient to allow for stabilizing the bottom of the mass excavation, however, groundwater may be present in within individual pile cap excavations. Localized pumping may be required at pile caps to allow placement of rebar and concrete. We expect that groundwater will primarily occur within silty sand (granular) lenses within the predominantly fine-grained silt soils. These soils are generally difficult to dewater due to the slow permeability of the silt and the isolated, random nature of the granular zones/lenses.

6.3.4 A qualified dewatering consultant should be retained to assess flow rates and anticipated effective drawdown during the design phase of the project. We anticipate that several, closely spaced perimeter and interior well points operating far in advance of construction (at least several weeks) will be necessary to effectively dewater the site.

6.3.5 If perimeter wells and interior well points are not effective, water may be collected and controlled within the excavation through the use of gravel-filled trenches (French drains). The number and locations of the French drains can be adjusted during excavation activities as necessary to collect and control groundwater. The French drains would then direct the collected seepage to a sump where it could be pumped out of the excavation. Alternatively, a gravel blanket drain may be placed over the entire excavation bottom which would have additional stabilizing benefits. Additional discussion is provided in Section 6.6.

6.4 Temporary Excavation Shoring and Support

6.4.1 Construction of ESC structure will involve mass excavation below the existing lower parking level by approximately 11 feet. Due to the proximity of adjacent improvements (streets, sidewalks, structures) excavation sloping will likely not be possible and shoring will be required. There are many options for temporary earth retention, such as soldier pile and lagging, sheet piles, cast-in-place tangent piers, and others.

6.4.2 The design of temporary earth retention systems must consider soil and groundwater conditions, surcharge loading, depth and width of the excavated area, as well as excavation staging and sequencing. At this time, the excavation edge limits, sequencing, and methods of temporary and permanent earth retention are not well defined. It is possible that the existing retaining walls may be incorporated into the temporary and/or permanent earth retention systems. A qualified shoring contractor/consultant should be retained to provide a design for temporary shoring.

6.4.3 The structural engineer in conjunction with Geocon and the shoring contractor/consultant should evaluate the as-built conditions of the existing retaining walls and possible incorporation into temporary and/or permanent earth retention systems. Once an earth retention strategy has been developed, we should provide lateral earth pressure design recommendations in an addendum to this report.

6.5 Materials for Fill

6.5.1 Excavated near-surface soils generated from cut operations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or rocks larger than 6 inches in maximum dimension. As previously discussed, excavated soils will have high in-situ moisture content and significant aerating/drying and/or chemical treatment will be required to achieve proper compaction.

6.5.2 Import fill material should be primarily granular with a "low" expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.5.3 Environmental characteristics and corrosion potential of import soil materials should also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.6 Excavation Bottom Stabilization

6.6.1 Due to high in-situ moisture content, significant instability should be expected in the mass excavation bottom. Stabilization measures will likely be necessary in order to provide access for construction equipment. The use of low contact-pressure tracked equipment should be considered to reduce disturbance and deterioration of the exposed excavation bottom.

6.6.2 Since we do not know the magnitude and extent of soft or unstable areas, our field representative should provide mitigation recommendations in the field at the time of construction. For planning purposes, we are providing three stabilization alternatives for consideration: (1) over-excavation and placement of a gravel mat (layer) over a durable geosynthetic fabric; (2) chemical treatment with high-calcium quicklime; and (3) placing a layer of large, angular rock or crushed concrete. The appropriateness and effectiveness of these alternatives will depend on the severity of the instability.

6.6.3 Stabilization may be accomplished by placing at least 12 to 18 inches of open-graded, angular ¾-inch gravel over a stabilization geotextile fabric (Mirafi 500X or equivalent). This procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Adjacent edges of fabric should be overlapped at least 2 feet or as recommended by the manufacturer. In order to reduce disturbance to the soft subgrade, we recommend using low-contact pressure, tracked equipment to perform the gravel spreading operations. Heavy equipment may operate on the completed gravel mat. The gravel should be compacted/consolidated to a dense state utilizing track equipment or a drum roller.

6.6.4 The gravel mat, if used, should be coordinated with the temporary dewatering system for the site. The gravel and fabric system is highly permeable and may enhance and facilitate dewatering efforts as well as provide a stable platform for construction equipment. We recommend that the grading and dewatering contractors coordinate efforts for this system.

6.6.5 Alternatively, based on the experience of the contractor and equipment being utilized, it may be possible to create a stable excavation bottom by blending high-calcium quicklime into the wet soils exposed in the excavation bottom. We anticipate that a minimum 18-inch-thick lift of lime-treated soil will be required. We anticipate lime content required for stabilization will be approximately 5% or less by dry weight. However, laboratory analyses should be performed to confirm that this percentage is effective.

6.6.6 Lime-treatment operations require the use of large construction equipment to spread and mix the lime. The presence of existing foundations (piles) and possibly other obstructions may hamper lime-treatment production. If lime-treatment is performed, we recommend that the contractor consider creating a stockpile of lime-treated soil for later use during smaller earthwork operations, such as shallow utility trench backfill.

6.6.7 Once the lime-treated soil has been mixed, mellowed (24-hours), re-mixed, compacted, and allowed to cure for a minimum of two days, we recommend placing at least 4 to 6 inches of crushed aggregate or AB over the treated soil to protect/enhance the durability of the section. The stabilized soil is essentially a "crust" that will bridge over the underlying soft wet soils and heavy construction equipment could damage the existing bridging capacity of the soil crust resulting pumping, instability, and deterioration. The aggregate surfacing should enhance the durability of the section.

6.6.8 Lime-treated soil has alkalinity (pH typically greater than 12) and is severely detrimental to landscaping. If used, lime-treatment should be limited to areas below the proposed building and pavement areas. *The designers of subsurface improvements in lime-treated areas should be informed of the chemical characteristics of lime-treated soil.*

6.6.9 Another stabilization alternative consists of placing a thin lift of 3 to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete would also be acceptable for this purpose. Material generated from onsite concrete crushing operations could be used. The crushed rock/broken concrete should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. In order to prevent excessive disturbance to a soft subgrade, we recommend using low-contact pressure tracked equipment for stabilization operations.

6.7 Grading and Earthwork

6.7.1 Earthwork operations should be observed and fills tested for recommended compaction and moisture content by a representative of Geocon.

6.7.2 References to relative compaction and optimum moisture content in this report are based on the latest ASTM D1557 Test Procedure.

6.7.3 Prior to earthwork operations, a pre-construction conference with representatives of the client, grading contractor, and Geocon should be held at the site. Site preparation, soil handling and/or the grading plans should be discussed at the pre-construction conference.

6.7.4 Site preparation should begin with complete removal of existing structures (including slabs, footings, and appurtenances), subsurface structures, underground utilities, debris, and existing pavements (HMA, PCC, and AB). Existing pile foundations not planned for re-use should be cut-off at least 2 feet below finished subgrade elevation. Any existing wells should be abandoned in accordance with Sacramento County requirements. Any encountered deleterious debris such as wood, brick, trash, etc. should be excavated and removed from the site.

6.7.5 If structurally supported slabs are used for the event floor, removal and re-compaction of existing undocumented fill (if present) under the ESC structure is not necessary. However, in areas with conventional slabs-on-grade, existing fill should be removed and replaced with engineered fill to provide uniform slab support.

6.7.6 Excavations or depressions resulting from demolition and removals, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.7.7 Once the mass excavation bottom has been established, Geocon should observe the exposed conditions and coordinate with the grading and dewatering contractors to evaluate the appropriate bottom stabilization alternatives as discussed in Section 6.6.

6.7.8 Remedial grading in the form of partial removal and re-compaction should be performed under ancillary structures (such as planter/landscape walls, monuments, trash enclosures, or similar structures) located at street-level. To provide more uniform support, we recommend removing and re-compacting at least 24 inches of existing soil below the bottom of footings for these improvements. We should review project plans as they develop further to better identify areas where remedial grading may be required.

6.7.9 Areas to receive fill and/or pavements/flatwork should be scarified at least 12 inches, uniformly moisture-conditioned to near optimum moisture content and compacted to at least 90% relative compaction. Scarification and re-compaction operations should be performed in the presence of our representative.

6.7.10 Engineered fill should be compacted in horizontal lifts not exceeding 8 inches (loose thickness) and brought to final design elevations. Each lift should be moisture-conditioned to near optimum moisture content, and compacted to at least 90% relative compaction.

6.7.11 The top 6 inches of final vehicular pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned to near optimum moisture content and compacted to at least 95% relative compaction. Final pavement subgrade should be finished to a smooth, unyielding surface. We further recommend proof-rolling the subgrade with a loaded water truck (or similar equipment with high contact pressure) to verify the stability of the subgrade prior to placing AB.

6.8 Underground Utilities

6.8.1 Current design details (November 8, 2013) indicate that building utilities will be located in a crushed concrete fill zone above mat slab which is pile supported. Due to the potential for liquefaction-induced settlement, consideration should be given to using flexible connections where utilities exit the building footprint.

6.8.2 Underground utility trenches should be backfilled with properly compacted material. Pipe bedding, shading, and trench backfill should conform to the requirements of the appropriate utility authority. Soil excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation, or rock/cementations larger than 6 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 6 inches. Lifts should be compacted to a minimum of 90% relative compaction near optimum moisture content. Compaction should be performed by mechanical means only; jetting of trench backfill should not be allowed.

6.9 Deep Foundations

6.9.1 Due to the presence of soft, compressible silt soil and underlying potentially liquefiable sand, and the magnitude of anticipated structural loading, the use of shallow foundations for support of the ESC structure is not feasible due to excessive post-construction settlement and potential liquefaction-induced settlement. However, shallow foundations or cast-in-drilled-hole (CIDH) concrete piers may be used for lightly-loaded ancillary structures not structurally connected to the ESC structure, such as planter/landscape walls, monuments, light poles, or similar structures where estimated post-construction settlements are tolerable. Recommendations for shallow foundations for lightly-loaded ancillary structures are provided in Section 6.10.

6.9.2 The ESC structure should be supported on deep foundations bearing within the dense sand gravel and underlying cemented silts and sands layer at and below approximately 40 to 50 feet below proposed event floor elevation (approximately Elevation -35 feet to -45 feet NAVD 88). A discussion of deep foundation types, project delivery methods, axial and lateral capacity estimates, and installation and load testing program recommendations are provided herein.

6.9.1 Deep Foundation Types and Project Delivery Method

6.9.1.1 There is a wide variety of deep foundation "pile" types available and each pile type behaves differently depending on installation and construction methods. Each pile type has specific advantages and disadvantages with respect to structural capacity, constructability, production rates, cost, and a host of other factors. The two major pile types include (1) manufactured "fixed-length" piles, such as pre-cast concrete, steel, or timber piles, and (2) drilled, cast-in-place piles. Each of these pile types includes "displacement" and "non-displacement" versions. Displacement piles move the soil laterally during installation (i.e. does not excavate or remove the soil) while non-displacement piles either cut through the soil (in the case of driven piles) or removes the soil (in the case of drilled piles). Displacement piles typically develop higher capacity due to the densification achieved as a result of soil displacement.

6.9.1.2 Due to the variable depths and variations within the dense sand and gravel layer in Downtown Sacramento, the use of fixed-length piles can be problematic due to early refusal and/or deeper penetration; both or which may require post-installation modifications to the pile such as cutting or splicing, which can add significant cost. In addition, we understand that pile driving noise and potential vibrations are undesirable for the project. Therefore, we do not recommend the use of fixed-length, driven piles for the project.

6.9.1.3 Because of the desire to reduce the amount of spoils generated from pile installation, the project team is currently considering low-vibration, drilled-displacement piles for support of the ESC structure. Possible pile types currently under consideration include drilled-displacement concrete filled pipe (DDCFP) piles, augercast pressure-grout displacement (APGD) piles, and conventional (non-displacement) augercast pressure grout piles (APG). Although conventional APG piles generate more spoils than APGD piles, they may prove more cost-effective from a constructability/production viewpoint. A brief discussion of each pile type follows:

- **Drilled-Displacement Concrete Filled Pipe (DDCFP) Piles:** consist of close-ended, conical-tipped steel pipes that are drilled into the ground using high torque and crowd pressure. Steel fins, a single flight, or helix welded to the tip displaces soil laterally during installation. Proprietary names for these pile types include Tubex®, EDTTEX®, Torque-Down®, and others. We understand that 12.75-inch Torque-Down® DDCFP piles were successfully installed at the nearby Aura Residential Tower (SEC 6th and L Streets) to depths on the order of 60 to 90 feet generating ultimate downward capacities ranging from 500 to 700 kips. However, it has been our experience on several Sacramento area projects that these piles suffer from installation difficulties, particularly early refusal in the dense sand and gravel layer (i.e. installation refusal prior to reaching the specified tip elevation). Methods to alleviate this problem include pre-drilling, which can reduce the capacity benefits of full soil displacement, generates excess spoils, and significantly reduces installation production rates. Some contractors have developed specific pile tips designed to better penetrate the soils.

- **Augercast Pressure Grouted Displacement (APGD) Piles** are installed using a specialized plugged auger that laterally displaces the soil as it is advanced into ground. Once the desired depth is reached, the plug is removed and high-strength grout is pumped under pressure as the auger is withdrawn. After the auger is removed, the required steel reinforcement is then "wet-set" into the pile to complete the installation. This pile type produces few spoils, approximately 20% or less of the theoretical hole volume. Installation difficulties associated with APGD piles can include early refusal in soils that are not "displaceable", such as very dense gravels and cemented soils; both are possible conditions at this site.

- **Augercast Pressure Grouted (APG) Piles** are similar to APGD piles but are not displacement piles. These piles are often referred to as "drilled replacement" piles. These piles are installed using a plugged continuous flight auger that is advanced into ground. Once the desired depth is reached, the plug is removed and high-strength grout is pumped under pressure as the auger is withdrawn. As the auger is withdrawn, the flighted soil is removed from the hole and replaced with grout. After the auger is removed, the required steel reinforcement is then "wet-set" into the pile to complete the installation. This pile type produces approximately 100% to 120% of the theoretical hole volume of spoils. Unlike DDCFP piles and APGD piles, APG piles generally do not experience early refusal due to the installation method. This pile type has been used successfully on many projects in Sacramento including the 500 Capitol Mall Highrise Office Building (5th Street and Capitol Mall) in which 85- to 90-foot-long, 18-inch diameter APG piles achieved 600 to 700 kip ultimate downward capacity.

6.9.1.4 The pile types discussed above are typically designed and installed by specialty geotechnical contractors. Constructability, installation production, performance, and capacity will vary depending on the contractor's experience, skill, equipment, materials, and installation procedures. For this site, difficult soil conditions include soft soils on the order of 40 to 50 feet thick overlying the dense sand and gravel bearing layer which will need to be penetrated some distance in order to develop pile capacity. *We strongly recommend performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity. Recommendations are provided in Section 6.9.4.* The program should include sampling and analysis of soil cuttings generated during test pile installation (if any) for the COCs to evaluate offsite disposal and/or reuse options.

6.9.1.5 Due to the large number of piles expected for this project and the anticipated compressed construction schedule, we recommend implementing a design-build or design-assist approach where one or more specialty foundation contractors are engaged to design and develop specifications for the deep foundation system. The specialty foundation contractor should prepare a complete design-build submittal with design details, calculations, estimated capacities, installation procedures, proposed load testing procedures, acceptance criteria, and quality control procedures. Geocon should perform a geotechnical review of the design-build submittal.

6.9.2 Axial Capacity and Negative Skin Friction

6.9.2.1 Deep foundation systems will generate the majority of their vertical load-carrying capacity from skin friction and end-bearing in the dense sand and gravel and underlying layers located beneath the site at approximately -40 feet to -50 feet NAVD 88 (45 to 55 feet below event floor).

6.9.2.2 Because of the highly variable structural loading for individual columns for the ESC structure (ranging from approximately 200 kips to 3,000 kips), it may be advantageous to use two different capacity "classes" of piles to optimize the foundation design: a lower capacity pile for lighter column loads and a higher capacity pile for heavier column loads. Table 6.9.2 presents estimated pile lengths and minimum embedment depths into the dense sand and gravel for DDCFP piles, APGD piles, and APG piles for two pile classes. *Pile lengths presented in Table 6.9.2 are estimates only and are intended for planning purposes. Actual pile lengths should be verified by the design-build contractor and confirmed based on the pile load testing program.* Geocon should review, and if necessary, can assist the design-build contractor in developing a suitable testing program; preliminary recommendations are provided in Section 6.9.4.

TABLE 6.9.2
AXIAL CAPACITY – ESTIMATED PILE LENGTHS

Pile Type	Class 200 Capacity[1]		Class 300 Capacity[2]	
	Minimum Embedment into Dense Sand/Gravel	Estimated Pile Length Below Pile Cap[3] (feet)	Minimum Embedment into Dense Sand/Gravel	Estimated Pile Length Below Pile Cap[3] (feet)
12.75-inch DDCFP Piles[4]	20	60	30	70
16-inch APGD Piles[5]	15	55	20	60
16-inch APG Piles[6]	20	60	30	70

Notes:
1. *200 kip allowable capacity (D+L), 400 kip ultimate capacity*
2. *300 kip allowable capacity (D+L), 600 kip ultimate capacity*
3. *Assumed elevation +0 feet NAVD 88*
4. *Drilled-Displacement Concrete Filled Pipe Pile*
5. *Auger Pressure Grout Displacement Pile*
6. *Auger Pressure Grout (non-displacement) Pile*

6.9.2.3 Single pile uplift capacity can be taken as 50 percent of the allowable downward capacity. Tension piles should be properly reinforced to transfer uplift forces to the pile tip. We note that deeper embedment into the gravel layer may be required to achieve the allowable compression or tension capacities depending on the pile type and construction methods.

6.9.2.5 Negative skin friction or "downdrag" can occur when the settlement of the surrounding soil exceeds the downward movement of the pile shaft. Should liquefaction occur at the site, negative skin friction could result due to subsequent settlement of the soil overlying the liquefiable material. Estimated negative skin friction forces are approximately 45 kips for the 12.75-inch DDCFP piles and 55 kips for the 16-inch APGD/AGP piles. Since liquefaction settlement and downdrag phenomena occur after seismic shaking has stopped, downdrag loads on the piles should be evaluated in conjunction with static building loads and not seismic loading conditions. Potential negative skin friction loads should be accounted for when determining allowable capacities based on load tests.

6.9.2.5 If pile spacing is at least 3 times the maximum dimension of the pile, a reduction in axial capacity for group effects is not considered necessary. Geocon should be contacted for review if piles are spaced closer than 3 times the maximum dimension of the pile.

6.9.3 Lateral Resistance

6.9.3.1 We performed lateral analysis using the computer program LPILE Plus 6 (2011) to evaluate laterally loaded, single piles. We modeled the 12.75-inch DDCFP piles as steel pipe piles with a 3/8-inch wall thickness and minimum yield strength of 45,000 pounds per square inch (psi). For the 16-inch APG/APGD piles, we assumed a concrete strength of 4,000 psi, a pile modulus of elasticity of 3,605,000 psi, and a pile moment of inertia of 3,217 in^4. We modeled both free- and fixed-head conditions using the parameters summarized in Table 6.9.3. Plots of lateral deflection, shear, and bending moment are included in Appendix E.

TABLE 6.9.3
SOIL PARAMETERS USED FOR L-PILE ANALYSIS

Soil Layer	Soil Type	Elevation (ft NAVD 88)		Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (deg)	Strain at 50% Stress e_{50}	Soil Modulus K (pci)
		Top	Bottom					
1	Silt	0	-25	55	600	0	0.01	30
2	Liquefiable Sand	-25	-40	55	0	30	---	20
3	Sand	-40	-100	68	0	36	---	100

Notes:

Groundwater assumed to be at Elevation 0 feet NAVD 88.
pcf= pounds per cubic foot
pci = pounds per cubic inch
psf = pounds per square foot

6.9.3.2 Our lateral analysis represents the probable response of a single pile under short-term loading conditions. Lateral capacity of a group of piles is generally less than that of a single pile for groups with individual pile spacing less than 6 to 8 diameters. The magnitude of reduction varies with the number and arrangement of the pile group. Once the pile group arrangement(s) have been determined, we should be contacted for review.

6.9.3.3 Additional resistance to lateral loading may be provided by passive pressure acting on the sides of the piles caps and grade beams (if any). An allowable passive resistance of 175 pounds per cubic foot (pcf) equivalent fluid pressure may be used, assuming saturated soil conditions. Frictional resistance along the bottom of the pile caps and grade beams should be neglected.

6.9.4 Installation and Load Test Program

6.9.4.1 We recommend performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity. The purposes of the test program will be to verify installation conditions, production rates, and axial capacity. In addition, the program should include sampling and analysis of soil cuttings generated during test pile installation (if any) for the COCs to evaluate offsite disposal and/or reuse options. A representative of Geocon should be present to observe test pile installation and load testing. The information obtained from the pile load testing should be used to evaluate the need to modify pile lengths to achieve design capacities, as well as develop installation criteria that can be used during construction of production piles.

6.9.4.2 At a minimum, we recommend installing at least five pre-production "indicator" piles, one each located near the corners and center of the proposed ESC structure to capture the varying subsurface conditions across the site. The indicator piles should be tested in compression and tension. The project structural engineer should evaluate the need for lateral load testing. If compression and tension tests cannot be performed on the same pile, additional indicator piles will be necessary. The sacrificial test piles should be instrumented with strain gauges at various locations along the length of the pile and at the pile toe such that the skin friction for the portion of the pile embedded in the silts and sands above the dense sand and gravel layer can be separated from the total capacity measured during the test. The strain gauge instrumentation will provide the load distribution along the length of the pile which will be important in evaluating capacity under seismic conditions.

6.9.4.3 Static compression load tests should be performed in accordance with ASTM D1143, and static uplift (tension) tests should be performed in accordance with ASTM D3689. Indicator piles should attain the ultimate design compression capacity (200% of the allowable design load + negative skin friction load) and 125% of the design uplift load for tension. If the required capacities are not reached, additional indicator piles should be constructed and tested.

6.9.4.4 Prior to performing static load testing of the APGD/APG piles, we recommend performing Thermal Integrity Profiling (TIP) as well as low-strain impact integrity testing (Pile Integrity Testing or "PIT") in accordance with ASTM D5882. The purpose of the TIP and PIT will be to verify the pile integrity, physical properties of the constructed pile (cross-sectional area, length, continuity and presence of cracks, cold-joints, necking or bulging) and to establish a correlation between the static load test results, TIP, and PIT results. As a quality assurance measure during construction, TIP and PIT should be performed on 10% of the production piles. The frequency of TIP and PIT may be increased by the geotechnical engineer if installation difficulties are encountered. If the results of TIP and PIT indicate questionable pile properties, additional high-strain dynamic testing (ASTM D4945) may be required to verify production pile capacity. For the DDCFP piles, we recommend performing high strain dynamic load tests on at least 5% of the production piles.

6.9.5.5 At the completion of indicator pile testing and prior to construction of production piles, the design-build foundation contractor should prepare a written report of the static load tests, TIP, PIT, and dynamic load testing. This report should contain a load test evaluation in accordance with Section 1810.3.3.1.3 of the 2013 CBC and a discussion of the pile working capacity obtained from the testing. The contractor should also prepare a report after the completion of the production piles. The report should present graphical information of the construction procedures, tip elevations, depth into competent material, grout volumes (for the APGD/APG piles), and a statement that the foundations can accept the design loading conditions.

6.10 Potential Reuse of Existing Foundations

6.10.1 It is possible that the existing pile foundations may be re-used in some fashion for the new structure. It is unlikely that portions of the existing piles remaining after mass excavation and cut-off would provide reliable downward axial capacity. However, the piles may be engaged to provide uplift resistance to hydrostatic forces or provide additional vertical support for the mat slab in a potential liquefaction event.

6.10.2 As-built information including lengths of the piles is currently not available. Therefore, pile testing such as PIT and static tension testing would be required to evaluate the integrity and capacity of the existing piles. We can provide specific testing recommendations if reuse is proposed.

6.11 Ancillary Structure Foundations

6.11.1 Foundations for lightly-loaded ancillary structures not structurally connected to the ESC structure, such as planter/landscape walls (up to 6 feet high), monuments, trash enclosures, or similar structures, may be supported on conventional foundations bearing on a minimum of 24 inches of newly placed engineered fill placed in accordance with the recommendations of this report.

6.11.2 Shallow foundations may be designed for an allowable bearing capacity of 1,500 pounds per square foot (psf), and should be a minimum of 12 inches wide and embedded at least 12 inches below the lowest adjacent grade. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

6.11.3 Allowable passive pressure used to resist lateral movement of footings may be assumed to be equal to a fluid weighing 250 pcf. The allowable coefficient of friction to resist sliding of footings is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for footing design provided that the frictional resistance is reduced by 50%.

6.11.4 Continuous footings should be reinforced with at least four No. 4 reinforcement bars, two each placed near the top and bottom of the footing to allow footings to span isolated soil irregularities. The reinforcement recommended above is for soil characteristics only and is not intended to replace reinforcement required for structural considerations. The project structural engineer should evaluate the need for additional reinforcement.

6.11.5 Light poles and similar structures may be supported on straight-shaft, CIDH concrete piers which may be designed using formulae from the 2013 CBC. An allowable lateral soil-bearing pressure (CBC parameters S_1 in equation 18-1 and S_3 in equations 18-2 and 18-3) of 150 psf per foot of depth may be used. If ½-inch deflection at the ground surface is acceptable, this value may be doubled.

6.11.6 The bottom of the pier excavation should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material, and a flat cleanout plate is necessary. Concrete should be placed within the excavation as soon as possible after the auger/cleanout plate is withdrawn to reduce the potential for caving.

6.11.7 If seepage or groundwater is encountered, water should be pumped from the pier excavation prior to placement of concrete. Concrete should be placed by tremie methods from the bottom of the hole keeping the tremie pipe below the surface of the concrete at all times. Concrete should have a minimum 28-day design strength of 3,000 psi.

6.11.8 A Geocon representative should observe foundation excavations prior to placing reinforcing steel or concrete to observe that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.12 Interior Slabs-on-Grade

6.12.1 As previously discussed, a structural concrete slab designed to span between pile caps should be used for the main event floor. Assuming some degree of damage is acceptable, slabs for ancillary areas may consist of conventional (non-structural) slabs-on-grade.

6.12.2 Slab thickness and reinforcement should be determined by the structural engineer. Based on preliminary details provided by the design team, the structural mat slab will be approximately 18 to 24 inches thick and conventional slabs will be 6 inches thick. Control joint spacing should conform to ACI, PCA or similar guidelines.

6.12.3 If building pad soils become dry, they should be re-moistened prior to concrete slab-on-grade construction. Building pads should be moistened to at least optimum moisture content prior to placing the vapor barrier. Moisture content should be verified by Geocon prior to placing the vapor barrier.

6.13 Waterproofing / Concrete Moisture Protection Considerations

6.13.1 Permanent waterproofing of subterranean walls and slabs will be required. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations. In addition, an experienced waterproofing inspector should be retained to check proper installation of the system during construction.

6.13.2 Migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner and design team, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field.

6.13.3 In areas where waterproofing materials are not present, a minimum 10-mil-thick vapor barrier meeting ASTM E1745-97 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) may be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.13.4 At least 4 inches of ½ or ¾ inch crushed rock, with no more than 5 percent passing the No. 200 sieve, may be placed below the vapor barrier to serve as a capillary break.

6.13.5 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.13.6 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.14 Retaining Walls

6.14.1 At the time of this report, the structure edge limits, details, and types of permanent retaining walls are not well defined. New retaining walls will likely consist of the walls associated with the subterranean event floor area and be directly supported by the ESC deep foundation system. Lateral support for the walls may be provided by the structure itself or by tieback anchors. We should review the proposed retaining wall details with the project structural engineer and provide additional design parameters and recommendations as needed.

6.14.2 Preliminary design of retaining walls and buried structures may be based on the lateral earth pressures (equivalent fluid pressure) summarized in Table 6.14.

TABLE 6.14
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active (Above Groundwater)	45 pcf
Active (Below Groundwater)	85 pcf
At-Rest (Above Groundwater)	65 pcf
At-Rest (Below Groundwater)	95 pcf
Passive (Above Groundwater)	250 pcf
Passive (Below Groundwater)	175 pcf
Seismic Earth Pressure[1]	10 pcf

1. Applicable for walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. Conventional triangular distribution. Should be combined with ACTIVE lateral earth pressure for seismic case analysis.

6.14.3 The soil pressures above assume that the backfill material within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall will be composed of the existing onsite soils.

6.14.4 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared addressing specific surcharge conditions throughout the project, if necessary.

6.14.4 In addition to the recommended earth pressure, the upper 10 feet of subterranean walls adjacent to streets should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, the traffic surcharge may be neglected.

6.14.5 If not designed for hydrostatic conditions, retaining walls should be provided with a drainage system and waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.14.6 Moisture affecting below grade walls is a common post-construction complaint. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

6.15 Elevator Pit Design

6.15.1 Elevator pit slabs and retaining walls should be designed by the project structural engineer. As a minimum, the pit slab-on-grade should be at least 5 inches thick and reinforced with No. 4 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed using the lateral earth pressures provided in Section 6.14. The elevator pit should be structurally supported by the ESC foundation system.

6.15.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared addressing specific surcharge conditions throughout the project, if necessary.

6.15.3 We suggest that the elevator pit walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

6.15.4 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to an existing pile foundation, especially if the drilling is performed after the foundation is in place.

6.15.5 Casing will be required if caving is experienced in the drilled excavation, especially if the excavation is conducted below the groundwater level. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces since the casing will be below the groundwater level. Continuous observation of the drilling and installation of the elevator piston by Geocon is highly recommended.

6.15.6 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

6.16 Concrete Sidewalks and Flatwork

6.16.1 Sidewalk, curb, and gutter within City right-of-way should be designed and constructed in accordance with the latest City of Sacramento standards and details as applicable. We note that the City of Sacramento requires 6 inches of compacted Class 2 AB below sidewalks.

6.16.2 Exterior concrete flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint.

6.16.3 Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

6.16.4 The recommendations of this report are intended to reduce the potential for cracking of slabs. However, even with the incorporation of these recommendations, concrete flatwork may exhibit some cracking due to soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.17 Rigid Concrete Pavement

6.17.1 Rigid concrete pavement may be used in vehicular traffic areas, such as loading and parking. Based on the soil conditions encountered at the site, concrete pavement should consist of at least 6 inches Portland Cement Concrete (PCC) overlying at least 12 inches of Class 2 AB meeting the requirements of Section 26 of the Caltrans Standard Specifications.

6.17.2 Subgrade soils should be prepared in accordance with the recommendations of the geotechnical report. Class 2 AB and subgrade should be compacted to at least 95% relative compaction near optimum moisture content. Subgrade should be proof-rolled with a loaded water truck to verify stability.

6.17.3 Concrete should have a minimum 28-day compressive strength of 4,000 psi. Adequate construction and crack control joints should be used to control cracking inherent in concrete construction. It would be advantageous to provide minimal reinforcement, such as No. 3 steel bars placed 18 inches on center in both horizontal directions to help control cracking. Consideration should be given to providing maximum control joint spacing of 12 feet in both directions for a 6-inch-thick slab. Adequate dowels should also be used at joints to facilitate load transfer and reduce vertical offset. In addition, the recommendations above pertaining to depended curbs, moisture cut-offs, and subsurface drainage applies to concrete pavements, sidewalks and flatwork, as well as asphalt pavements.

6.17.4 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.18 Site Drainage

6.18.1 Proper site drainage is critical to reduce the potential for differential soil movement, soil expansion, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to building foundations. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with the 2013 CBC or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices.

6.18.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.18.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall (deepened concrete curb, plastic root barrier, or similar cutoff) along the edge of the pavement that extends at least 4 inches into the soil subgrade below the bottom of the base material.

6.18.4 We recommend that roof drains be connected to water-tight drainage piping connected to the storm drain system. However, we understand that Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. At a minimum, the water from the roof drains should be directed away from buildings. Consideration should be given to draining roofs to lined planter boxes or placing liners below the proposed landscape areas to prevent infiltration of the water. Geocon can be contacted for additional recommendations.

6.18.5 Experience has shown that even with these provisions, subsurface seepage may develop in areas where no such water conditions existed prior to site development. This is particularly true where a substantial increase in surface water infiltration has resulted from an increase in landscape irrigation.

7.0 FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review the improvement plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for other's interpretation of our recommendations, and therefore the future performance of the project.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, we should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials or environmental contamination was not part of our scope of services.

This report is issued with the understanding that it is the responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought to the attention of the design team for the project and incorporated into the plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The recommendations contained in this report are preliminary until verified during construction by representatives of our firm. Changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. Additionally, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated partially or wholly by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.

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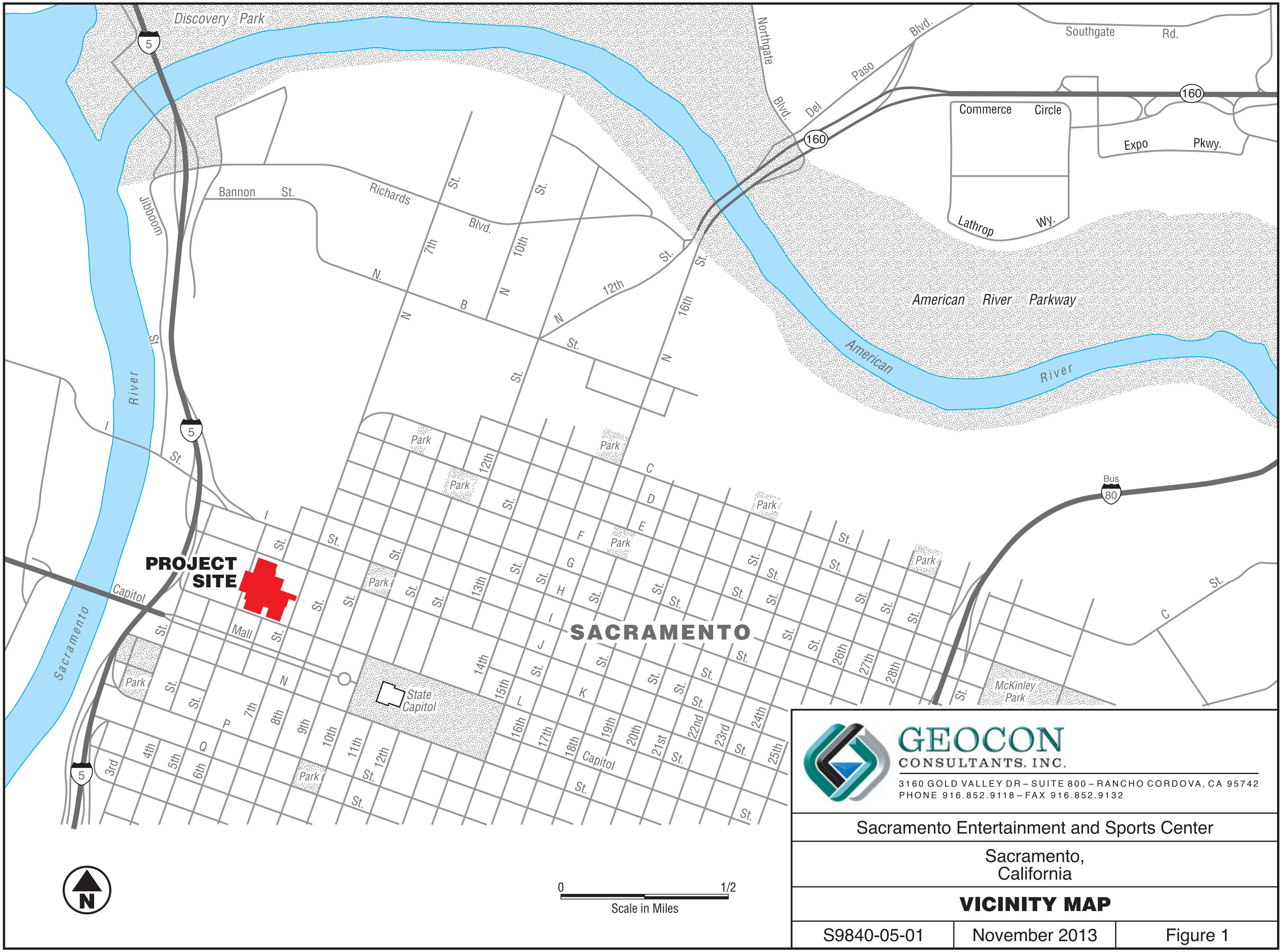

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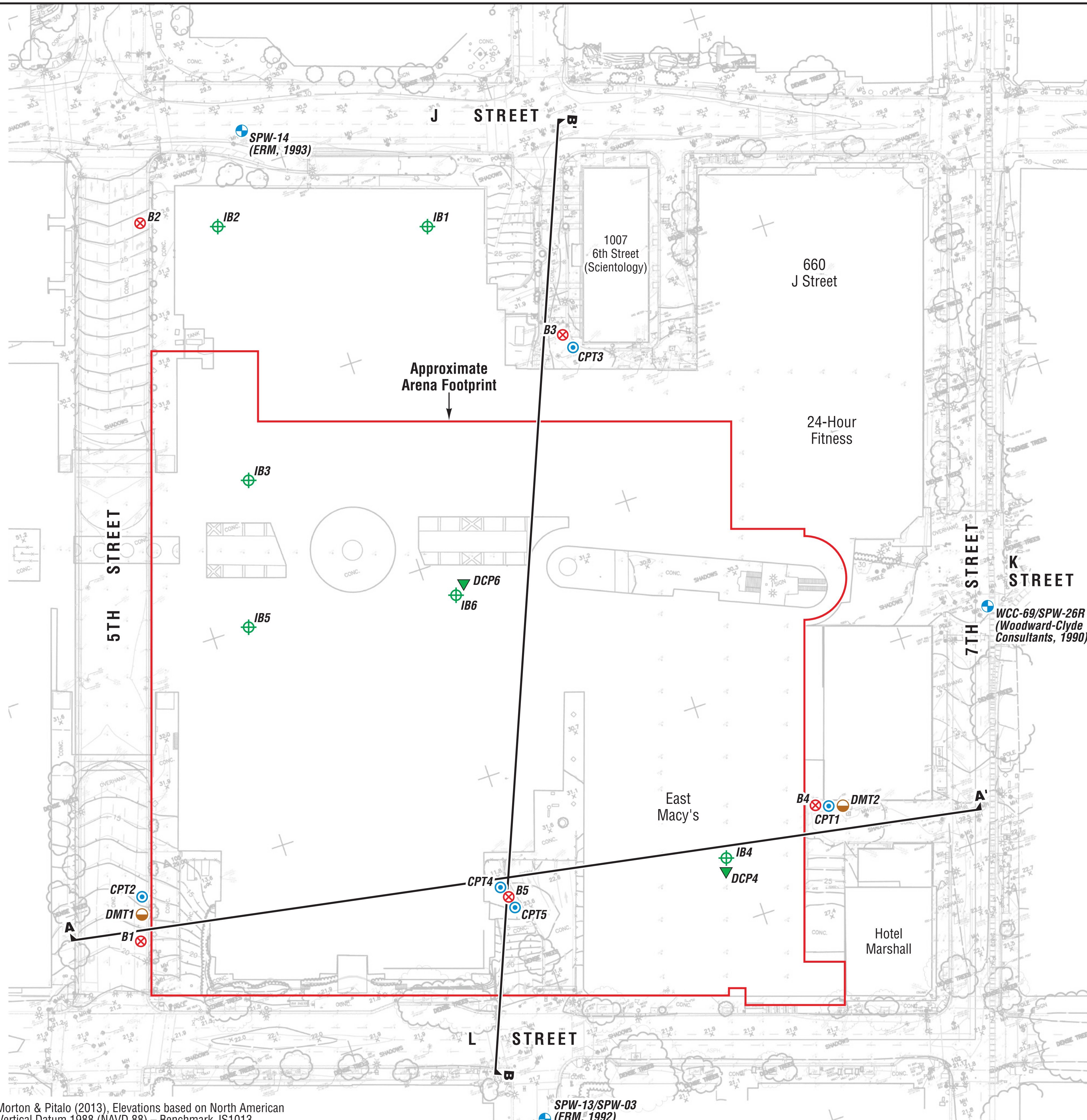
Sacramento Entertainment and Sports Center

Sacramento,
California

VICINITY MAP

S9840-05-01	November 2013	Figure 1



LEGEND:

B5	Approximate Exploratory Boring Location
IB6	Approximate Interior Exploratory Boring Location
CPT5	Approximate Cone Penetrometer Test Location
DCP6	Approximate Dynamic Cone Penetrometer Location
DMT2	Approximate Dilatometer Test Location
WCC-69	Approximate Existing Groundwater Monitoring Well Location
A A'	Approximate Cross-Section Location (Figures 3 and 4)

0 100
Scale in Feet

Site Plan Ref: Morton & Pitalo (2013), Elevations based on North American Vertical Datum 1988 (NAVD 88) – Benchmark JS1013

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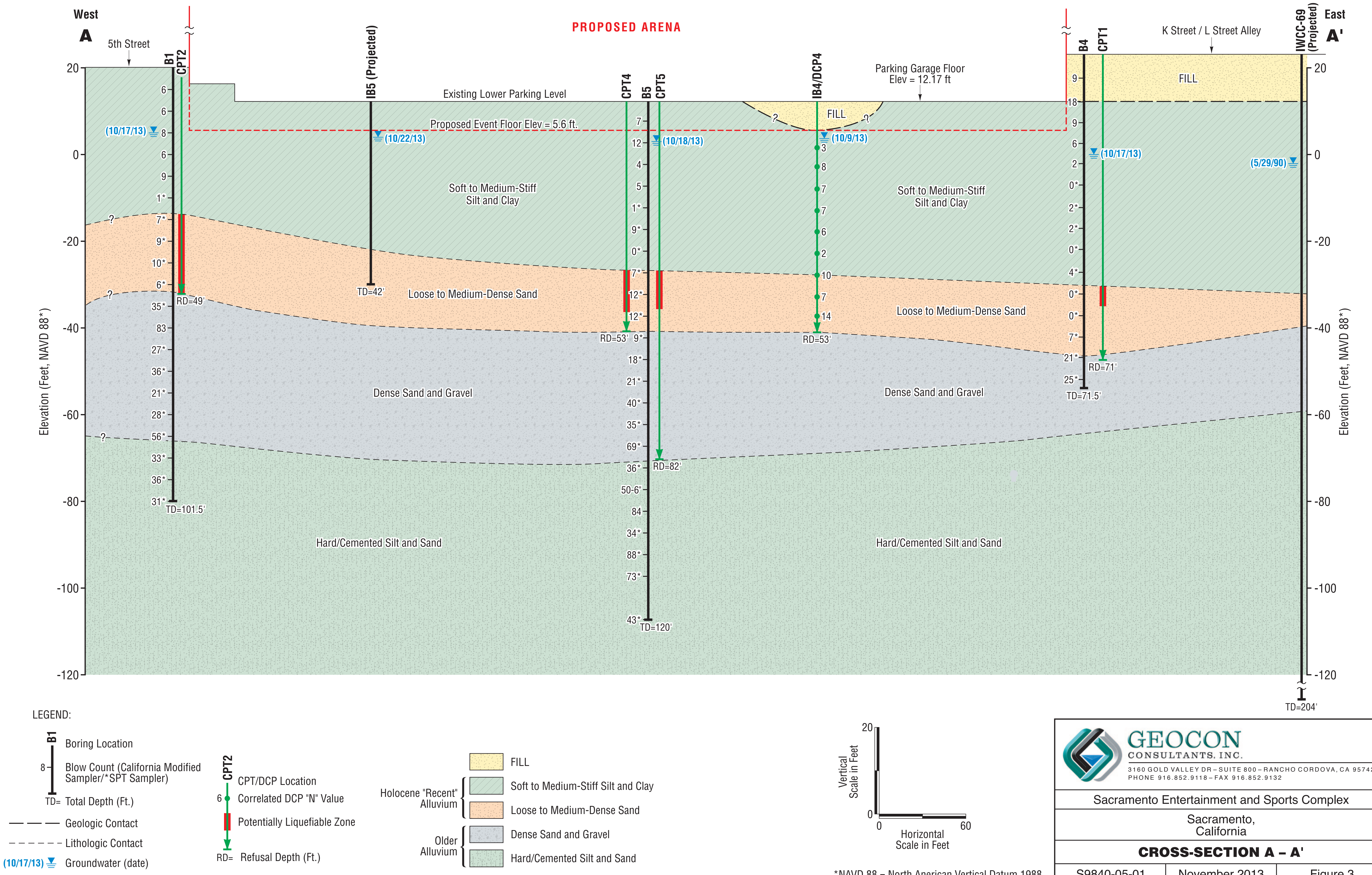
SITE PLAN

S9840-05-01	November 2013	Figure 2



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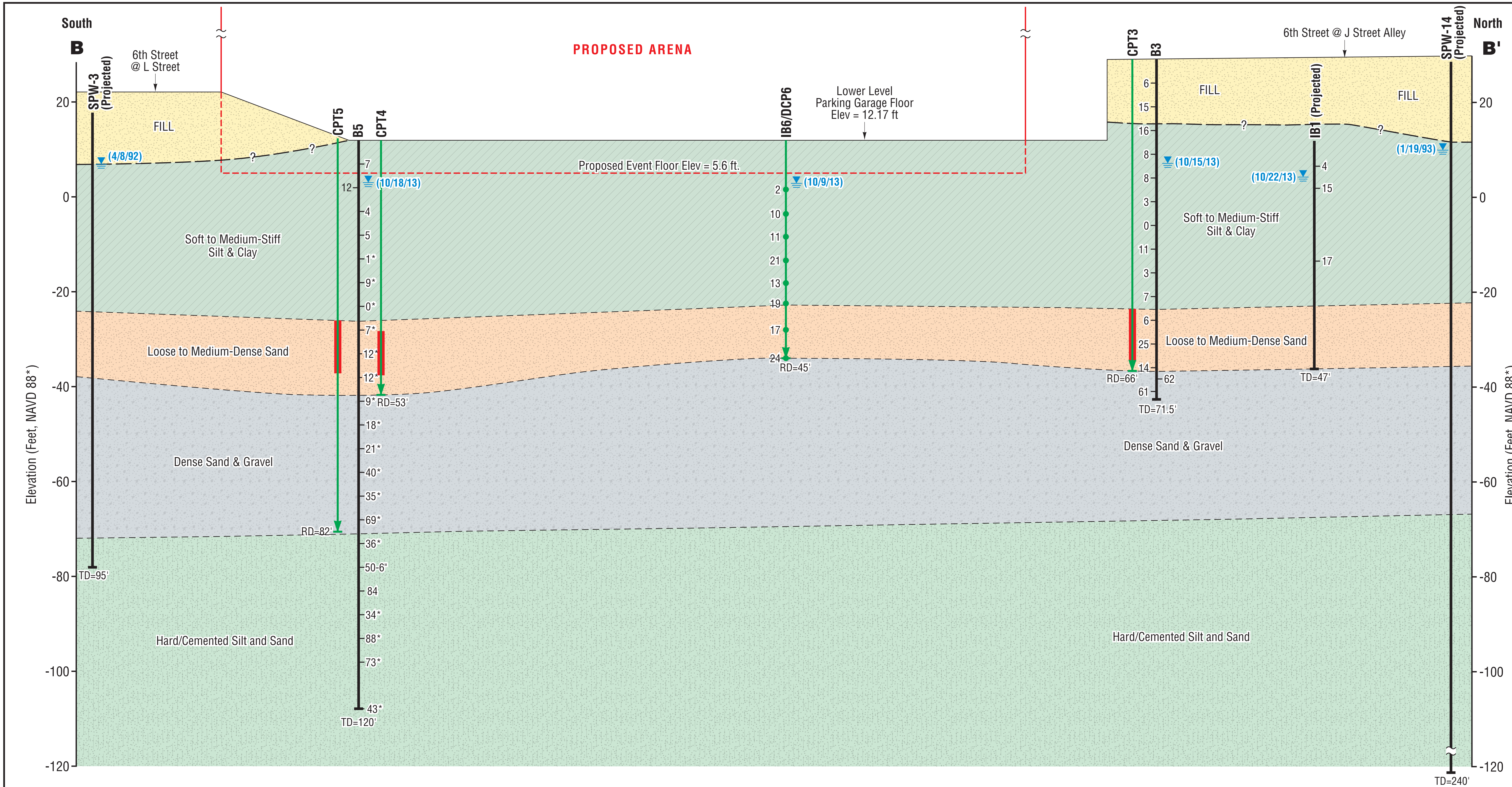
CROSS-SECTION A – A'

S9840-05-01	November 2013	Figure 3



LEGEND:

B1	Boring Location
8	Blow Count (California Modified Sampler/*SPT Sampler)
TD=	Total Depth (Ft.)
———	Geologic Contact
- - - -	Lithologic Contact
(10/17/13)	Groundwater (date)
CPT2	CPT/DCP Location
6	Correlated DCP "N" Value
	Potentially Liquefiable Zone
RD=	Refusal Depth (Ft.)

	FILL
Holocene "Recent" Alluvium	Soft to Medium-Stiff Silt & Clay
	Loose to Medium-Dense Sand
Older Alluvium	Dense Sand and Gravel
	Hard/Cemented Silt and Sand

Vertical Scale in Feet: 0 – 20
Horizontal Scale in Feet: 0 – 60

*NAVD 88 = North American Vertical Datum 1988

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Sacramento Entertainment and Sports Complex

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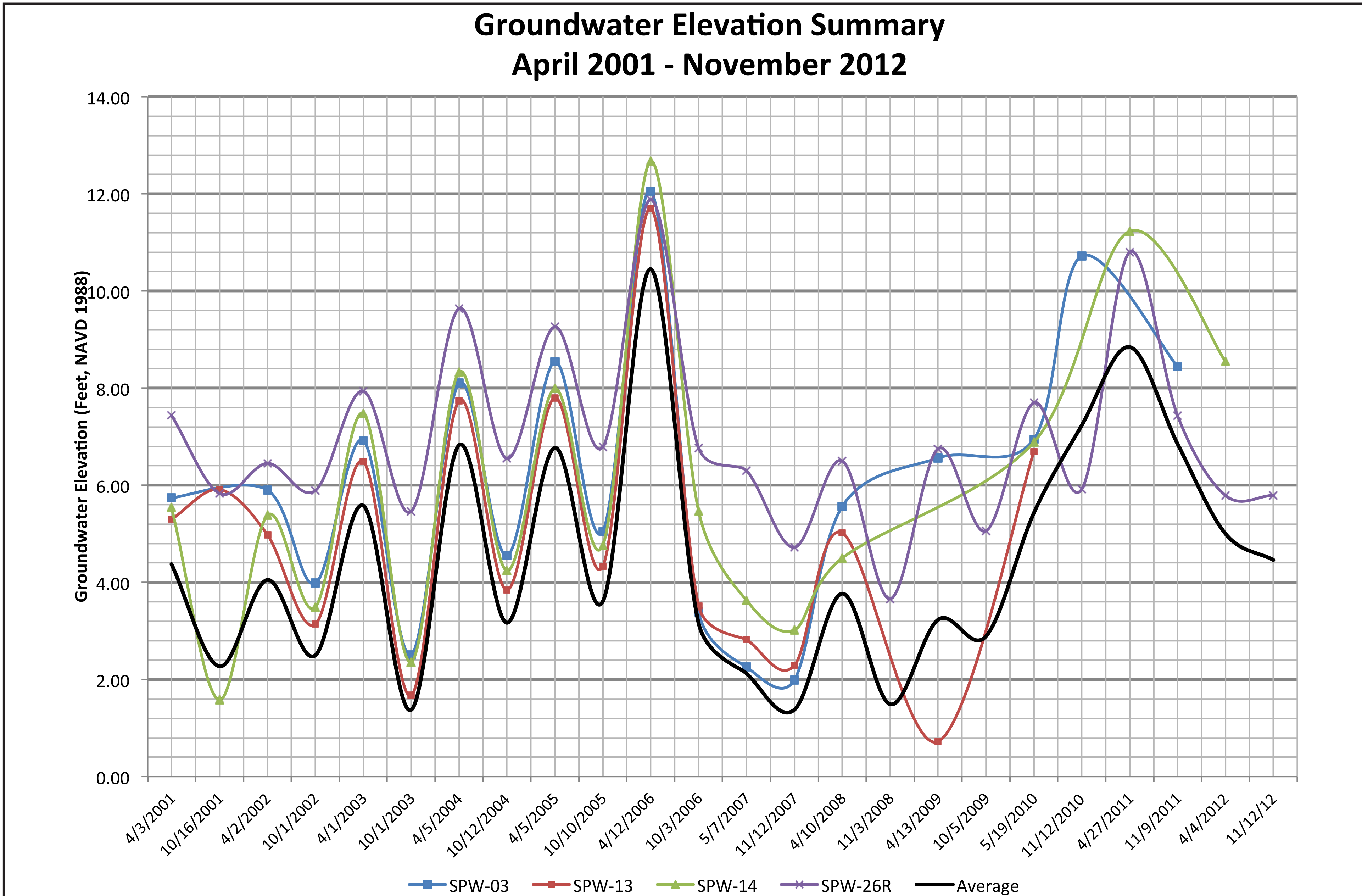
CROSS-SECTION B – B'

S9840-05-01	November 2013	Figure 4



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Groundwater Elevation Summary

Sacramento Entertainment and Sports Complex		
Sacramento, California		
S9840-05-01	November 2013	Figure 5

APPENDIX A

APPENDIX A

FIELD EXPLORATION

Our geotechnical field exploration was performed from October 9 to 23, 2013 and consisted of advancing eleven exploratory borings (B1 through B5 and IB1 through IB6), two dynamic cone penetrometer soundings (DCP4 and DCP6), five cone penetration test (CPT1 to CPT5) soundings, and two dilatometer soundings (DMT1 and DMT2) at the approximate locations shown on the Site Plan, Figure 2.

Exploratory borings were performed using a truck-mounted CME-75 drill rig equipped with 8-inch outside-diameter hollow-stem augers and a 4.875-inch rotary-wash drilling system. Sampling was accomplished using a 140-pound, automatic hammer with a 30-inch drop. Samples were obtained with a 3-inch OD, split-spoon (California Modified) sampler and a 2-inch OD Standard Penetration Test (SPT) sampler. The number of blows required to drive the samplers the last 12 inches (or portion thereof) of the 18-inch sampling interval were recorded on the boring logs.

The DCP soundings were then performed using a hand-operated Wildcat DCP to provide in-situ measurements of soil density and consistency.

The CPTs were performed using a 20-ton push capacity truck-mounted rig. CPT parameters including tip resistance (q_c), sleeve friction (f_s) and dynamic pore pressure (U) were measured at approximately 2-inch intervals as the cone advanced. Soil behavior types were determined using correlations developed by Robertson and Campanella (1988). The cone was equipped with a seismic transducer and shear wave velocity measurements were also collected at approximate 5-foot intervals in two of the soundings.

The dilatometer test is performed in-situ by pushing a blade-shaped, precision-instrument into the soil. The blade is equipped with an expandable membrane on one side that is pressurized until the membrane moves horizontally into the surrounding soil. Readings of the pressure required to move the membrane to a point that is flush with the blade (A – pressure) and to a point 1.1 mm into the surrounding soil (B – pressure) are recorded. The pressure is subsequently released and, in permeable soils below the groundwater table, a pressure reading is recorded as the membrane returns to the flush position (C – pressure). The test sequence is performed at 0.2-meter (8-inch) intervals to obtain a comprehensive soil profile. A material index (Id), a horizontal stress index (Kd) and a dilatometer modulus (Ed) are obtained directly from the dilatometer data.

Marchetti (1980) developed a soil classification system based on the material index. According to this system, soils with Id values less than 0.6 are classified as clay. Soils classified as sand have an Id value greater than 1.8. Material index values between 0.6 and 1.8 indicate silty clay to silty sand soils. Empirical relationships between the horizontal stress index and the coefficient of lateral earth pressure (K_o) have been developed by Lunne et al. (1990) for clays and by Schmertmann (1983) for uncemented sands. While Lunne's method makes use of dilatometer data exclusively, Schmertmann utilizes both dilatometer and cone penetration data to estimate K_o. Since the dilatometer is strain-controlled, the measured difference between the B-pressure and A-pressure readings (corrected for membrane stiffness) and cavity expansion theory can be used to directly measure the soil stiffness. Assuming a Poisson's ratio, the dilatometer modulus is correlated to shear modulus, Young's modulus, and constrained modulus.

The dilatometer soundings completed at this site were advanced to depths ranging from approximately 37 to 50 feet.

Subsurface conditions encountered in the exploratory borings were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488-90). This system uses the Unified Soil Classification System (USCS) for soil designations. The logs depict soil and geologic conditions encountered and depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing. Logs of the exploratory borings, DCPs, CPTs and DMTs are presented herein as follows:

- Figure A1, Key to Logs
- Figures A2 through A36, Logs of Exploratory Borings and Dynamic Cone Penetration Soundings
- Figures A37 through A41, Logs of CPT Soundings (CPT1 through CPT5)
- Figures A42 and A43, Dilatometer Soundings (DMT1 and DMT2)

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	**GRAVELS** MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVELS, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	**SILTS AND CLAYS** LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TEST PIT LOG LEGEND

pp	—	Pocket Penetrometer (tsf)
tsf	—	Tons Per Square Foot
LL	—	Liquid Limit
PI	—	Plasticity Index
	—	Shelby Tube Sample
	—	Bulk Sample
	—	SPT Sample
	—	Modified California Sample
▼	—	Groundwater Level (At Completion)
▽	—	Groundwater Level (First Encountered)

PENETRATION RESISTANCE

SAND AND GRAVEL		
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*
VERY LOOSE	0 - 4	0 - 7
LOOSE	4-10	7 - 17
MEDIUM DENSE	10-30	17 - 48
DENSE	30-50	48 - 85
VERY DENSE	OVER 50	OVER 85

SILT AND CLAY			
CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY SOFT	0 - 2	0 - 2	0 - 0.25
SOFT	2 - 3	2 - 4	0.25 - 0.50
MEDIUM STIFF	3 - 8	4 - 10	0.50 - 1.0
STIFF	8 - 15	10 - 20	1.0 - 2.0
VERY STIFF	15 - 30	20 - 48	2.0 - 4.0
HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

Geocon Consultants, Inc.
3160 Gold Valley Drive, Suite 800
Rancho Cordova, CA 95742
Telephone: 916-852-9118
Fax: 916-852-9132

GEOCON

Key to Logs

Project: Sacramento ESC
Location: Sacramento, CA
Number: S9840-05-01
Figure: A1

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING B1

ELEV. (MSL.) **19.5** DATE COMPLETED **10/17/2013**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**CONCRETE:** 10 inches			
1				GM	**AGGREGATE BASE:** 18 inches Dense, moist, brown, rounded, Silty GRAVEL with Sand			
2								
3				ML	**ALLUVIUM** Medium stiff, moist, brown, SILT with Clay			
4								
5					PP = 1.0 tsf			
6	B1-6.0					6	84.2	37.8
7								
8								
9								
10					PP = 1.0 tsf			
11	B1-11.0					6		
12								
13								
14								
15			▼		- becomes saturated			
16	B1-16.0			ML	soft, wet, brown, SILT with interbeds of Silty Sand 80.6% fines	8		36.4
17								
18								
19								
20					- no recovery			
21						6		
22								
23								
24				SM	Loose, wet, brown, Silty SAND			

Figure A2, Log of Boring, page 1 of 5

GEOCON

SAMPLE SYMBOLS
- ☐ ... SAMPLING UNSUCCESSFUL
- ☒ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

BORING B1

ELEV. (MSL.) 19.5 | DATE COMPLETED 10/17/2013
ENG./GEO. Sean Dixon | DRILLER V&W Drilling
EQUIPMENT CME 75 HSA/Mud Rotary | HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25				SM	- becomes gray, saturated			35.6
26	B1-26.0				- 37.7% fines - 31.6% fines	9	88.7	33.5
27								
28				ML	Soft, wet, gray, Sandy SILT			
29								
30	B1-30.0					1		
31								
32								
33								
34				SP-SM	Loose, saturated, gray, fine-grained Poorly graded SAND with Silt			
35	B1-35.0				- 12.7% fines	7		32.4
36								
37								
38								
39								
40	B1-40.0				- becomes medium-grained sand - 6.4% fines	9		27.6
41								
42								
43								
44								
45	B1-45.0				- 8.5% fines	10		26.3
46								
47								
48				SM	Loose, wet, fine-grained Silty SAND			
49								

Figure A3, Log of Boring, page 2 of 5

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B1

ELEV. (MSL.) **19.5** DATE COMPLETED **10/17/2013**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50	B1-50.0		-lense of sandy silt	6		58.1
52		SP	Dense, wet, gray, Poorly graded SAND			
55	B1-55.0		- becomes dense	35		
58		SP-SM	Very dense, saturated, light brown, Poorly graded fine-grained SAND with Silt			
60						24.2
61	B1-61.0			83		
64		SM	Medium dense, saturated, gray, Silty SAND with Gravel			
65	B1-65.0			27		
70	B1-70.0		- trace gravel up to 3/4" - 12.6% fines	26		16.4
73		SP-SM	Medium dense, wet, gray, Poorly graded SAND with Silt and Gravel			

Figure A4, Log of Boring, page 3 of 5

GEOCON

SAMPLE SYMBOLS ... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED) ... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B1

ELEV. (MSL.) **19.5** DATE COMPLETED **10/17/2013**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
75	B1-75.0			SP-SM	- becomes fine-grained sand, no gravel - 7.2% fines	21		33.8
80	B1-80.0				- becomes medium-grained sand - 7.6% fines	28		22.2
85	B1-85.0				- dense, trace rounded gravel approximately 3/4"	56		
86				ML/SM	Hard, saturated, light brown, SILT with fine-grained Sand interbedded with Silty Sand, non-plastic			
90	B1-90.0				- 46.8% fines	33		40.9
95	B1-95.0				- weakly cemented	36		

Figure A5, Log of Boring, page 4 of 5

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B1

ELEV. (MSL.) **19.5** DATE COMPLETED **10/17/2013**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
100	B1-100.0				- becomes more sandy	31		
101								
					BORING TERMINATED AT 101.5 FEET GROUNDWATER ENCOUNTERED AT 15.0 FEET BACKFILLED WITH NEAT CEMENT GROUT			

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING B2

ELEV. (MSL.) **26.5** DATE COMPLETED **10/16/2013**
ENG./GEO. **Joshua Lewis** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**CONCRETE:** 8.5 inches			
1					**AGGREGATE BASE:** 15.5 inches Dense, moist, brown, rounded, Silty GRAVEL with Sand			
2				ML	**ALLUVIUM** Very stiff to hard, damp, brown, SILT with fine-grained Sand PP = 4.5 tsf			
3								
4								
5								
6	B2-6.0			SP-SM	Loose, damp, brown, Poorly graded fine- to medium-grained SAND with Silt	9		
7								
8								
9								
10				ML	Stiff, damp, brown, SILT with fine-grained Sand PP = 1.5 tsf			
11	B2-11.0					7		
12								
13								
14								
15				ML	Soft, moist, brown, Clayey SILT PP < 0.5 tsf			
16	B2-16.0					5		
17								
18								
19								
20					- medium stiff, no recovery, drove sampler back down with sand catcher and collected disturbed sample			
21	B2-21.0					8		
22								
23								
24								

Figure A7, Log of Boring, page 1 of 3

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B2

ELEV. (MSL.) 26.5 DATE COMPLETED 10/16/2013

ENG./GEO. Joshua Lewis DRILLER V&W Drilling

EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25				ML	Stiff, moist, dark green, SILT, trace fine-grained sand, micaceous			
26	B2-26.0				PP = 2.0 tsf	12		
27								
28								
29			▼					
30					- very soft, wet			
31	B2-31.0				PP < 0.5 tsf	1	77.1	45.6
32					- switch to mud rotary			
33				SM	Loose, wet, dark green, Silty SAND			
34								
35								
36	B2-36.0				- 15.2% fines	14	89.8	31.4
37								
38				SP	Loose, wet, dark green, Poorly graded SAND			
39								
40	B2-40.0					5		36.3
41								
42								
43								
44								
45					-medium dense			
46	B2-46.0					17		25.8
47								
48								
49								

Figure A8, Log of Boring, page 2 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B2

ELEV. (MSL.) **26.5** DATE COMPLETED **10/16/2013**
ENG./GEO. **Joshua Lewis** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50	B2-50.0				- loose - 4.7% fines	7		35.8
51								
52								
53				GP	Dense, wet, dark green, Poorly graded GRAVEL with Sand			
54								
55								
56	B2-56.0					78		7.0
57								
58								
59								
60				GW	Dense, wet, dark green, Well graded GRAVEL			
61	B2-61.0					51		

BORING TERMINATED AT 61.5 FEET
GROUNDWATER ENCOUNTERED AT 29.0 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A9, Log of Boring, page 3 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING B3

ELEV. (MSL.) 28.5 DATE COMPLETED 10/15/2013
ENG./GEO. Joshua Lewis DRILLER V&W Drilling
EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT CONCRETE:** 7 inches			
1					**AGGREGATE BASE:** 8 inches			
2				SP	**FILL** Very loose, damp, brown, Poorly graded fine-grained SAND, micaceous, brick fragments			
6	B3-6.0					6		
10				ML	**ALLUVIUM** Loose, moist, dark brown, fine-grained Sandy SILT			
11	B3-11.0					15		
15				ML	Stiff, moist, dark brown, SILT with Sand			
16	B3-16.0					18		
20				ML	Stiff, moist, dark brown, fine-grained Sandy SILT PP = 2.0 tsf			
21	B3-21.0					8	90.9	30.3
24			▼					

Figure A10, Log of Boring, page 1 of 3

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B3

ELEV. (MSL.) **28.5** DATE COMPLETED **10/15/2013**
ENG./GEO. **Joshua Lewis** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25				ML	Stiff, moist to wet, dark brown, Sandy SILT			
26	B3-26.0				- 85.5% fines	8	87.9	34.0
27								
28								
29								
30				ML	Soft, wet, dark brown, SILT, low plasticity PP < 0.5 tsf			
31	B3-31.0					3		
32								
33								
34								
35	B3-35.0				- very soft PP < 0.5 tsf	0		41.2
36								
37								
38								
39								
40					- stiff, dark green PP = 1.5 tsf			
41	B3-41.0					11	83.0	39.3
42								
43								
44								
45	B3-45.0				- soft	3		42.3
46								
47								
48								
49								

Figure A11, Log of Boring, page 2 of 3

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B3

ELEV. (MSL.) 28.5 DATE COMPLETED 10/15/2013
ENG./GEO. Joshua Lewis DRILLER V&W Drilling
EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50				CL-ML	Medium stiff, wet, dark green, Silty CLAY, moderate plasticity PP = 0.5 tsf			
51	B3-51.0					7		
52								
53								
54								
55	B3-55.0			SP-SM	Loose, wet, dark green, Poorly graded fine-grained SAND with Silt - 10.3% fines	6		30.5
56								
57								
58								
59								
60	B3-60.0				- medium dense, no recovery	25		
61								
62								
63								
64								
65					- heaving sands in auger			
66	B3-66.0				- 7.9% fines	14	96.4	25.2
67					- switch to mud rotary			
68				GP	Dense, wet, dark gray, Poorly graded coarse GRAVEL, few fine gravel			
69								
70	B3-70.0					62		
71					- losing drilling mud into formation			
72	B3-72.0					61		

BORING TERMINATED AT 72.5 FEET
GROUNDWATER ENCOUNTERED AT 25.0 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A12, Log of Boring, page 3 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B4

ELEV. (MSL.) 21.5 DATE COMPLETED 10/17/2013

ENG./GEO. Sean Dixon DRILLER V&W Drilling

EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**CONCRETE:** 9 inches			
1				GP	**FILL** Loose, dry to damp, gray, Poorly graded coarse GRAVEL, angular, 3/4"			
2								
3								
4								
5								
6	B4-6.0					4		
7								
8								
9								
10								
11	B4-11.0					18		
12				ML	**ALLUVIUM** Medium stiff, moist, light brown and gray SILT with fine-grained Sand			
13								
14								
15								
16	B4-16.0					9		
17								
18								
19								
20					PP = 0.75 tsf			
21	B4-21.0					6	87.6	35.4
22								
23			▼					
24								

Figure A13, Log of Boring, page 1 of 4

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B4

ELEV. (MSL.) **21.5** DATE COMPLETED **10/17/2013**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25					- becomes saturated			
26	B4-26.0					2		
27								
28								
29								
30	B4-30.0			ML	Soft, saturated, gray, SILT	0		40.4
31								
32								
33								
34								
35	B4-35.0				- 91.8% fines	2		44.1
36								
37								
38								
39								
40	B4-40.0					2		38.0
41								
42								
43								
44								
45	B4-45.0				- no recovery	0		
46								
47								
48								
49								

Figure A14, Log of Boring, page 2 of 4

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B4

ELEV. (MSL.) **21.5** DATE COMPLETED **10/17/2013**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50	B4-50.0	SM	Soft, saturated, gray, Silty SAND - 41.4% fines	4		49.6
51						
52						
53		ML	Soft, saturated, gray, fine-grained Sandy SILT - 62.9% fines			
54						
55	B4-55.0			0		46.5
56						
57						
58						
59						
60	B4-61.0			0		
61						
62						
63						
64		SM	Loose, saturated, gray, Silty SAND, trace organics			
65	B4-65.0			7		50.1
66						
67						
68						
69						
70	B4-70.0		- 38.4% fines	21		39.6
71		GP	Dense, saturated, gray, Poorly graded coarse GRAVEL, rounded			
72						
73						
74						

Figure A15, Log of Boring, page 3 of 4

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B4

ELEV. (MSL.) 21.5 DATE COMPLETED 10/17/2013
ENG./GEO. Sean Dixon DRILLER V&W Drilling
EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
75	B4-75.0				- 9" slough	25		
76								

BORING TERMINATED AT 76.5 FEET
GROUNDWATER ENCOUNTERED AT 23.0 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A16, Log of Boring, page 4 of 4

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B5

ELEV. (MSL.) 12.5 DATE COMPLETED 10/18/2013
ENG./GEO. Sean Dixon DRILLER V&W Drilling
EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**CONCRETE:** 8 inches			
1				ML	**ALLUVIUM** Medium stiff, moist, brown, SILT with fine-grained SAND PP = 0.75 tsf			
2								
3								
4								
5								
6	B5-6.0					7		
7								
8								
9			▼					
10					- becomes stiff, saturated			
11	B5-11.0					12		
12								
13								
14								
15					- becomes medium stiff, saturated gray		79.1	42.2
16	B5-16.0					4		
17								
18								
19								
20							78.2	43.4
21	B5-21.0					5		
22								
23								
24				ML	Soft, saturated, gray, fine-grained Sandy SILT			

Figure A17, Log of Boring, page 1 of 5

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B5

ELEV. (MSL.)	12.5	DATE COMPLETED	10/18/2013
ENG./GEO.	Sean Dixon	DRILLER	V&W Drilling
EQUIPMENT	CME 75 HSA/Mud Rotary	HAMMER TYPE	Automatic

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25	B5-25.0		Soft, saturated, gray, fine-grained Sandy SILT	1		
26						
27						
28						
29						
30	B5-30.0	SM	Loose, saturated, gray, Silty fine-grained SAND - 32.9% fines	9		33.2
31						
32						
33						
34		ML	Very soft, saturated, gray, SILT			
35	B5-35.0		- 19.7% fines (sand lense)	0		45.6
36						
37						
38						
39						
40	B5-40.0			7		
41			- 6" sand lense			
42						
43						
44						
45	B5-45.0			12		
46		SP	Medium dense, saturated, gray, Poorly graded Sand, few silt			
47						
48						
49						

Figure A18, Log of Boring, page 2 of 5

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING B5

ELEV. (MSL.) **12.5** DATE COMPLETED **10/18/2013**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50	B5-50.0				- no recovery with sand catcher	12		
51								
52								
53				SM	Loose, saturated, gray, Silty SAND			
54								
55	B5-55.0				- 26% fines	9		24.4
56								
57								
58								
59								
60	B5-60.0			SP-SM	Loose, saturated, gray, Poorly graded SAND with Silt - 9.2% fines	18		33.1
61								
62								
63								
64				SP-SM	Medium dense, saturated, gray, Poorly graded coarse- to medium-grained SAND with Silt and fine Gravel			
65	B5-65.0					21		
66								
67								
68								
69								
70	B5-70.0				- becomes dense - 5.9% fines	40		15.7
71								
72								
73								
74								

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B5

ELEV. (MSL.) 12.5 DATE COMPLETED 10/18/2013
ENG./GEO. Sean Dixon DRILLER V&W Drilling
EQUIPMENT CME 75 HSA/Mud Rotary HAMMER TYPE Automatic

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
75	B5-75.0			SP-SM	Dense, saturated, gray, Poorly graded coarse- to medium-grained SAND with fine Gravel	35		
80	B5-80.0			SP	Medium dense, saturated, gray, Poorly graded coarse- to medium-grained SAND	69		
83.5				ML	Hard, saturated, light brown and gray, SILT, cemented			
85	B5-85.0					36		
90	B5-80.0					50/6"		
93.5				SP	Very dense, saturated, gray Poorly graded SAND			
96	B5-96.0					84		

Figure A20, Log of Boring, page 4 of 5

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B5

ELEV. (MSL.) **12.5** DATE COMPLETED **10/18/2013**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **CME 75 HSA/Mud Rotary** HAMMER TYPE **Automatic**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
100	B5-100.0				- becomes dense	34		
101								
102								
103				ML	Hard, saturated, gray and brown, SILT, cemented			
104								
105	B5-105.0					88		
106								
107								
108								
109								
110	B5-110.0					73		
111								
112								
113								
114								
115								
116								
117								
118								
119					- interlayered with silty sand			
120	B5-120.0					43		
121								

BORING TERMINATED AT 121.5 FEET
GROUNDWATER ENCOUNTERED AT 9.0 FEET
BACKFILLED WITH NEAT CEMENT GROUT

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING IB1

ELEV. (MSL.) **11.5** DATE COMPLETED **10/22/2013**

ENG./GEO. **Sean Dixon** DRILLER **California Geotech**

EQUIPMENT **Superman 5" HSA** HAMMER TYPE **Manual Donut 40# 30"**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT:** 3 inches			
1				ML	**AGGREGATE BASE:** 3 inches Rounded, GRAVEL with Sand and Silt			
2	IB1-BULK(1.5-5)				**ALLUVIUM** Soft, moist, brown, SILT			
3								
4					- becomes light brown, wet			
5	IB1-5.0					4		
6								
7			▼		- becomes saturated			
8								
9								
10	IB1-10.0				- becomes medium stiff	15		
11								
12								
13								
14								
15	IB1-15.0				- fine silt, saturated			
16					- generating little cuttings			
17								
18								
19								
20								
21								
22								
23	IB1-22.5				- becomes gray	7		
24								

Figure A22, Log of Boring, page 1 of 2

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING IB1

ELEV. (MSL.)	11.5	DATE COMPLETED	10/22/2013
ENG./GEO.	Sean Dixon	DRILLER	California Geotech
EQUIPMENT	Superman 5" HSA	HAMMER TYPE	Manual Donut 40# 30"

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25								
26								
27								
28								
29								
30								
31					Soft, saturated, brown, SILT (cuttings)			
32								
33								
34								
35								
36								
37								
38								
39								
40								
41								
42								
43								
44								
45								
46								
47					- gravelly drilling			
					BORING TERMINATED AT 47.5 FEET GROUNDWATER ENCOUNTERED AT 7.0 FEET BACKFILLED WITH NEAT CEMENT GROUT			

Figure A23, Log of Boring, page 2 of 2

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

BORING IB2

ELEV. (MSL.) 11.5 | DATE COMPLETED 10/23/2013
ENG./GEO. Sean Dixon | DRILLER California Geotech
EQUIPMENT Superman 4" SSA | HAMMER TYPE N/A

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT:** 3 inches			
					AGGREGATE BASE: 3 inches Rounded, GRAVEL with Sand and Silt			
1				ML				
2	IB2-BULK(2-5)				**ALLUVIUM** Soft to medium stiff, moist, brown, SILT			
3								
4					- becomes light brown and wet			
5								
6								
7			▼					
8					- becomes saturated			
9								
10					- becomes gray			
11								
12					BORING TERMINATED AT 12.0 FEET GROUNDWATER ENCOUNTERED AT 7.5 FEET BACKFILLED WITH NEAT CEMENT GROUT			

Figure A24, Log of Boring, page 1 of 1

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING IB3

ELEV. (MSL.) **11.5** DATE COMPLETED **10/23/2013**

ENG./GEO. **Sean Dixon** DRILLER **California Geotech**

EQUIPMENT **Superman 4" SSA** HAMMER TYPE **N/A**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT:** 2 inches			
1				GM	**FILL** Dense, damp to moist, light brown, rounded, GRAVEL to COBBLE with Sand and Silt			
2								
3					- refusal on dense gravels and cobbles			

REFUSAL AT 3.0 FEET
GROUNDWATER NOT ENCOUNTERED
BACKFILLED WITH NEAT CEMENT GROUT

Figure A25, Log of Boring, page 1 of 1

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING IB4

ELEV. (MSL.) 11.5 DATE COMPLETED 10/9/2013
ENG./GEO. Sean Dixon DRILLER GEOCON
EQUIPMENT Hand Auger 4" HAMMER TYPE N/A

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**CONCRETE:** 6 inches			
1				GP	**CRUSHED ROCK** Medium dense, damp to moist, gray, 3/4" open graded crushed rock			
2				ML	**FILL** Medium stiff, moist, gray, SILT with brick and ceramic fragments and wood debris			
3								
4					-slight Petroleum hydrocarbon odor			
5	IB4-5.0			ML	**ALLUVIUM** Medium stiff, moist, gray, SILT			
6								
7								
8	IB4-8.0		▼		- becomes saturated - Continues as DCP4			37.1

BORING TERMINATED AT 8.5 FEET
GROUNDWATER ENCOUNTERED AT 8.0 FEET
BORING CONTINUED AS DYNAMIC CONE PENETRATION (DCP) SOUNDING - DCP4

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

WILDCAT DYNAMIC CONE LOG

Geocon Consultants, Inc.

PROJECT NUMBER: S9840-05-01
DATE STARTED: 10-09-2013
DATE COMPLETED: 10-09-2013

HOLE #: DCP4
CREW: SMD JAE
PROJECT: Sacramento ESC
ADDRESS: K Street
LOCATION: Sacramento, CA

SURFACE ELEVATION: 12
WATER ON COMPLETION: 8
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 1 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 2 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 3 ft	0	0.0		0	VERY LOOSE	VERY SOFT
- 1 m	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 4 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 5 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 6 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 2 m	0	0.0		0	VERY LOOSE	VERY SOFT
- 7 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 8 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	2	6.8	•	1	VERY LOOSE	VERY SOFT
-	2	6.8	•	1	VERY LOOSE	VERY SOFT
- 9 ft	1	3.4		0	VERY LOOSE	VERY SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
-	5	17.1	••••	4	VERY LOOSE	SOFT
- 3 m 10 ft	4	13.7	•••	3	VERY LOOSE	SOFT
-	5	15.3	••••	4	VERY LOOSE	SOFT
-	4	12.2	•••	3	VERY LOOSE	SOFT
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
- 11 ft	5	15.3	••••	4	VERY LOOSE	SOFT
-	8	24.5	•••••••	6	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
- 12 ft	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	8	24.5	•••••••	6	LOOSE	MEDIUM STIFF
-	10	30.6	••••••••	8	LOOSE	MEDIUM STIFF
- 4 m 13 ft	12	36.7	••••••••••	10	LOOSE	STIFF

WILDCAT DYNAMIC CONE LOG

PROJECT: Sacramento ESC PROJECT NUMBER: S9840-05-01

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
-	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 14 ft	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
-	12	33.2	•••••••••	9	LOOSE	STIFF
-	14	38.8	•••••••••••	11	MEDIUM DENSE	STIFF
- 15 ft	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	9	24.9	•••••••	7	LOOSE	MEDIUM STIFF
-	9	24.9	•••••••	7	LOOSE	MEDIUM STIFF
- 16 ft	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
- 5 m	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
-	10	25.4	•••••••	7	LOOSE	MEDIUM STIFF
- 17 ft	10	25.4	•••••••	7	LOOSE	MEDIUM STIFF
-	12	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	11	27.9	••••••••	7	LOOSE	MEDIUM STIFF
- 18 ft	11	27.9	••••••••	7	LOOSE	MEDIUM STIFF
-	12	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	13	33.0	•••••••••	9	LOOSE	STIFF
- 19 ft	13	33.0	•••••••••	9	LOOSE	STIFF
-	10	25.4	•••••••	7	LOOSE	MEDIUM STIFF
- 6 m	10	25.4	•••••••	7	LOOSE	MEDIUM STIFF
- 20 ft	11	25.6	•••••••	7	LOOSE	MEDIUM STIFF
-	10	23.3	••••••	6	LOOSE	MEDIUM STIFF
-	7	16.3	••••	4	VERY LOOSE	SOFT
- 21 ft	8	18.6	•••••	5	LOOSE	MEDIUM STIFF
-	9	21.0	••••••	5	LOOSE	MEDIUM STIFF
-	9	21.0	••••••	5	LOOSE	MEDIUM STIFF
- 22 ft	9	21.0	••••••	5	LOOSE	MEDIUM STIFF
-	14	32.6	•••••••••	9	LOOSE	STIFF
-	14	32.6	•••••••••	9	LOOSE	STIFF
- 7 m 23 ft	13	30.3	••••••••	8	LOOSE	MEDIUM STIFF
-	14	30.2	••••••••	8	LOOSE	MEDIUM STIFF
-	14	30.2	••••••••	8	LOOSE	MEDIUM STIFF
- 24 ft	16	34.6	••••••••••	9	LOOSE	STIFF
-	14	30.2	••••••••	8	LOOSE	MEDIUM STIFF
-	12	25.9	•••••••	7	LOOSE	MEDIUM STIFF
- 25 ft	12	25.9	•••••••	7	LOOSE	MEDIUM STIFF
-	13	28.1	••••••••	8	LOOSE	MEDIUM STIFF
-	14	30.2	••••••••	8	LOOSE	MEDIUM STIFF
- 26 ft	12	25.9	•••••••	7	LOOSE	MEDIUM STIFF
- 8 m	11	23.8	••••••	6	LOOSE	MEDIUM STIFF
-	11	22.1	••••••	6	LOOSE	MEDIUM STIFF
- 27 ft	11	22.1	••••••	6	LOOSE	MEDIUM STIFF
-	11	22.1	••••••	6	LOOSE	MEDIUM STIFF
-	12	24.1	••••••	6	LOOSE	MEDIUM STIFF
- 28 ft	13	26.1	•••••••	7	LOOSE	MEDIUM STIFF
-	12	24.1	••••••	6	LOOSE	MEDIUM STIFF
-	14	28.1	••••••••	8	LOOSE	MEDIUM STIFF
- 29 ft	15	30.2	••••••••	8	LOOSE	MEDIUM STIFF
-	12	24.1	••••••	6	LOOSE	MEDIUM STIFF
- 9 m	12	24.1	••••••	6	LOOSE	MEDIUM STIFF

WILDCAT DYNAMIC CONE LOG

PROJECT: Sacramento ESC PROJECT NUMBER: S9840-05-01

	BLOWS	RESISTANCE	GRAPH OF CONE RESISTANCE		TESTED CONSISTENCY	
DEPTH	PER 10 cm	Kg/cm²	0 50 100 150	N'	SAND & SILT	CLAY
- 30 ft	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	12	22.7	••••••	6	LOOSE	MEDIUM STIFF
-	9	17.0	••••	4	VERY LOOSE	SOFT
- 31 ft	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	10	18.9	•••••	5	LOOSE	MEDIUM STIFF
-	9	17.0	••••	4	VERY LOOSE	SOFT
-	7	13.2	•••	3	VERY LOOSE	SOFT
- 32 ft	10	18.9	•••••	5	LOOSE	MEDIUM STIFF
-	9	17.0	••••	4	VERY LOOSE	SOFT
- 10 m	9	17.0	••••	4	VERY LOOSE	SOFT
- 33 ft	10	18.9	•••••	5	LOOSE	MEDIUM STIFF
-	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	8	15.1	••••	4	VERY LOOSE	SOFT
- 34 ft	5	9.5	••	2	VERY LOOSE	SOFT
-	4	7.6	••	2	VERY LOOSE	SOFT
-	6	11.3	•••	3	VERY LOOSE	SOFT
- 35 ft	4	7.6	••	2	VERY LOOSE	SOFT
-	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	8	15.1	••••	4	VERY LOOSE	SOFT
- 11 m 36 ft	7	13.2	•••	3	VERY LOOSE	SOFT
-	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
- 37 ft	15	28.4	••••••••	8	LOOSE	MEDIUM STIFF
-	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	15	28.4	••••••••	8	LOOSE	MEDIUM STIFF
- 38 ft	15	28.4	••••••••	8	LOOSE	MEDIUM STIFF
-	16	30.2	••••••••	8	LOOSE	MEDIUM STIFF
-	17	32.1	•••••••••	9	LOOSE	STIFF
- 39 ft	16	30.2	••••••••	8	LOOSE	MEDIUM STIFF
- 12 m	18	34.0	•••••••••	9	LOOSE	STIFF
-	19	35.9	••••••••••	10	LOOSE	STIFF
- 40 ft	19	35.9	••••••••••	10	LOOSE	STIFF
-	15	28.4	••••••••	8	LOOSE	MEDIUM STIFF
-	13	24.6	•••••••	7	LOOSE	MEDIUM STIFF
- 41 ft	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	11	20.8	••••••	5	LOOSE	MEDIUM STIFF
-	12	22.7	••••••	6	LOOSE	MEDIUM STIFF
- 42 ft	12	22.7	••••••	6	LOOSE	MEDIUM STIFF
-	13	24.6	•••••••	7	LOOSE	MEDIUM STIFF
- 13 m	14	26.5	•••••••	7	LOOSE	MEDIUM STIFF
- 43 ft	15	28.4	••••••••	8	LOOSE	MEDIUM STIFF
-	12	22.7	••••••	6	LOOSE	MEDIUM STIFF
-	8	15.1	••••	4	VERY LOOSE	SOFT
- 44 ft	8	15.1	••••	4	VERY LOOSE	SOFT
-	9	17.0	••••	4	VERY LOOSE	SOFT
-	7	13.2	•••	3	VERY LOOSE	SOFT
- 45 ft	6	11.3	•••	3	VERY LOOSE	SOFT
-	13	24.6	•••••••	7	LOOSE	MEDIUM STIFF
-	18	34.0	•••••••••	9	LOOSE	STIFF
- 14 m 46 ft	25	47.3	•••••••••••••	13	MEDIUM DENSE	STIFF

Figure A29

HOLE #: DCP4

WILDCAT DYNAMIC CONE LOG

PROJECT: Sacramento ESC

PROJECT NUMBER: S9840-05-01

	BLOWS	RESISTANCE	GRAPH OF CONE RESISTANCE		TESTED CONSISTENCY	
DEPTH	PER 10 cm	Kg/cm²	0 50 100 150	N'	SAND & SILT	CLAY
-	24	45.4	•••••••••••••	12	MEDIUM DENSE	STIFF
-	16	30.2	••••••••	8	LOOSE	MEDIUM STIFF
- 47 ft	14	26.5	•••••••	7	LOOSE	MEDIUM STIFF
-	16	30.2	••••••••	8	LOOSE	MEDIUM STIFF
-	18	34.0	•••••••••	9	LOOSE	STIFF
- 48 ft	15	28.4	••••••••	8	LOOSE	MEDIUM STIFF
-	14	26.5	•••••••	7	LOOSE	MEDIUM STIFF
-	16	30.2	••••••••	8	LOOSE	MEDIUM STIFF
- 49 ft	20	37.8	••••••••••	10	LOOSE	STIFF
- 15 m	18	34.0	•••••••••	9	LOOSE	STIFF
-	22	41.6	••••••••••••	11	MEDIUM DENSE	STIFF
- 50 ft	26	49.1	••••••••••••••	14	MEDIUM DENSE	STIFF
-	24	45.4	•••••••••••••	12	MEDIUM DENSE	STIFF
-	27	51.0	••••••••••••••	14	MEDIUM DENSE	STIFF
- 51 ft	25	47.3	•••••••••••••	13	MEDIUM DENSE	STIFF
-	20	37.8	••••••••••	10	LOOSE	STIFF
-	22	41.6	••••••••••••	11	MEDIUM DENSE	STIFF
- 52 ft	16	30.2	••••••••	8	LOOSE	MEDIUM STIFF
-	21	39.7	•••••••••••	11	MEDIUM DENSE	STIFF
- 16 m	23	43.5	••••••••••••	12	MEDIUM DENSE	STIFF
-	26	49.1	••••••••••••••	14	MEDIUM DENSE	STIFF
- 53 ft	30	56.7	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-						
-						
- 54 ft						
-						
-						
- 55 ft						
-						
- 17 m						
- 56 ft						
-						
-						
- 57 ft						
-						
-						
- 58 ft						
-						
-						
- 18 m 59 ft						
-						
-						
- 60 ft						
-						
-						
- 61 ft						
-						
-						
- 62 ft						
- 19 m						

WILDCAT.XLS

Figure A30

PROJECT NO. **S9840-05-01** PROJECT NAME **Sacramento ESC**

BORING IB5

ELEV. (MSL.) **11.5** DATE COMPLETED **10/22/2013**

ENG./GEO. **Sean Dixon** DRILLER **California Geotech**

EQUIPMENT **Superman 4" SSA** HAMMER TYPE **N/A**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT:** 3.5 inches			
1					**AGGREGATE BASE:** 12 inches			
2	IB5-BULK(1.5-5)			ML	**ALLUVIUM** Soft to medium stiff, moist, gray, SILT			
3								
4								
5								
6								
7			▼					
8	IB5-8.0				- becomes saturated, brown			
9								
10								
11								
12					- becomes gray			
13								
14								
15	IB5-15.0							
16								
17								
18								
19								
20								
21								
22								
23								
24								

Figure A31, Log of Boring, page 1 of 2

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING IB5** ELEV. (MSL.) **11.5** DATE COMPLETED **10/22/2013** ENG./GEO. **Sean Dixon** DRILLER **California Geotech** EQUIPMENT **Superman 4" SSA** HAMMER TYPE **N/A** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25								
26								
27								
28								
29								
30								
31								
32								
33								
34								
35								
36								
37					- becomes sandy (based on cuttings)			
38								
39								
40								
41								
42					BORING TERMINATED AT 42.0 FEET GROUNDWATER ENCOUNTERED AT 7.5 FEET BACKFILLED WITH NEAT CEMENT GROUT			

Figure A32, Log of Boring, page 2 of 2

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING IB6

ELEV. (MSL.) **11.5** DATE COMPLETED **10/9/2013**

ENG./GEO. **Sean Dixon** DRILLER **GEOCON**

EQUIPMENT **Hand Auger 4"** HAMMER TYPE **N/A**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT:** 2.5 inches			
1				GM	**AGGREGATE BASE** Rounded, GRAVEL with Silt and Sand			
2	IB6-2.0			ML	**ALLUVIUM** Medium stiff, moist, light brown, SILT			
3								
4	IB6-4.0							
5								
6	IB6-6.0							
7			▼					
8					- becomes saturated			
9					- Continues as DCP6			

BORING TERMINATED AT 9.0 FEET
GROUNDWATER ENCOUNTERED AT 7.5 FEET
BORING CONTINUES AS DYNAMIC CONE
PENETRATION (DCP) SOUNDING - DCP6

Figure A33, Log of Boring, page 1 of 1

IN PROGRESS S9840-05-01 SAC ESC.GPJ 11/12/13

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

WILDCAT DYNAMIC CONE LOG

Geocon Consultants, Inc.

PROJECT NUMBER: S9840-05-01
DATE STARTED: 10-09-2013
DATE COMPLETED: 10-09-2013

HOLE #: DCP6
CREW: SMD JAE
PROJECT: Sacramento ESC
ADDRESS: K Street
LOCATION: Sacramento, CA

SURFACE ELEVATION: 12
WATER ON COMPLETION: 8
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 1 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 2 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 3 ft	0	0.0		0	VERY LOOSE	VERY SOFT
- 1 m	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 4 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 5 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 6 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 2 m	0	0.0		0	VERY LOOSE	VERY SOFT
- 7 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 8 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 9 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	3	10.3	••	2	VERY LOOSE	SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
- 3 m 10 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	3	9.2	••	2	VERY LOOSE	SOFT
-	4	12.2	•••	3	VERY LOOSE	SOFT
-	4	12.2	•••	3	VERY LOOSE	SOFT
- 11 ft	5	15.3	••••	4	VERY LOOSE	SOFT
-	5	15.3	••••	4	VERY LOOSE	SOFT
-	5	15.3	••••	4	VERY LOOSE	SOFT
- 12 ft	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
- 4 m 13 ft	11	33.7	•••••••••	9	LOOSE	STIFF

WILDCAT DYNAMIC CONE LOG

PROJECT: Sacramento ESC PROJECT NUMBER: S9840-05-01

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	12	33.2	•••••••••	9	LOOSE	STIFF
-	12	33.2	•••••••••	9	LOOSE	STIFF
- 14 ft	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
- 15 ft	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
-	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
- 16 ft	13	36.0	••••••••••	10	LOOSE	STIFF
- 5 m	13	36.0	••••••••••	10	LOOSE	STIFF
-	13	33.0	•••••••••	9	LOOSE	STIFF
- 17 ft	13	33.0	•••••••••	9	LOOSE	STIFF
-	13	33.0	•••••••••	9	LOOSE	STIFF
-	14	35.6	••••••••••	10	LOOSE	STIFF
- 18 ft	14	35.6	••••••••••	10	LOOSE	STIFF
-	12	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	13	33.0	•••••••••	9	LOOSE	STIFF
- 19 ft	16	40.6	•••••••••••	11	MEDIUM DENSE	STIFF
-	18	45.7	•••••••••••••	13	MEDIUM DENSE	STIFF
- 6 m	17	43.2	••••••••••••	12	MEDIUM DENSE	STIFF
- 20 ft	18	41.9	••••••••••••	11	MEDIUM DENSE	STIFF
-	19	44.3	••••••••••••	12	MEDIUM DENSE	STIFF
-	22	51.3	••••••••••••••	14	MEDIUM DENSE	STIFF
- 21 ft	24	55.9	••••••••••••••••	15	MEDIUM DENSE	STIFF
-	24	55.9	••••••••••••••••	15	MEDIUM DENSE	STIFF
-	25	58.3	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 22 ft	27	62.9	••••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	27	62.9	••••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	30	69.9	••••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
- 7 m 23 ft	33	76.9	••••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
-	31	67.0	•••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
-	30	64.8	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
- 24 ft	30	64.8	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
-	30	64.8	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
-	34	73.4	•••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
- 25 ft	35	75.6	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
-	35	75.6	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
-	34	73.4	•••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
- 26 ft	34	73.4	•••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
- 8 m	30	64.8	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
-	24	48.2	•••••••••••••	13	MEDIUM DENSE	STIFF
- 27 ft	24	48.2	•••••••••••••	13	MEDIUM DENSE	STIFF
-	25	50.3	••••••••••••••	14	MEDIUM DENSE	STIFF
-	32	64.3	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
- 28 ft	26	52.3	•••••••••••••••	14	MEDIUM DENSE	STIFF
-	25	50.3	••••••••••••••	14	MEDIUM DENSE	STIFF
-	22	44.2	••••••••••••	12	MEDIUM DENSE	STIFF
- 29 ft	19	38.2	•••••••••••	10	LOOSE	STIFF
-	27	54.3	•••••••••••••••	15	MEDIUM DENSE	STIFF
- 9 m	22	44.2	••••••••••••	12	MEDIUM DENSE	STIFF

Figure A35

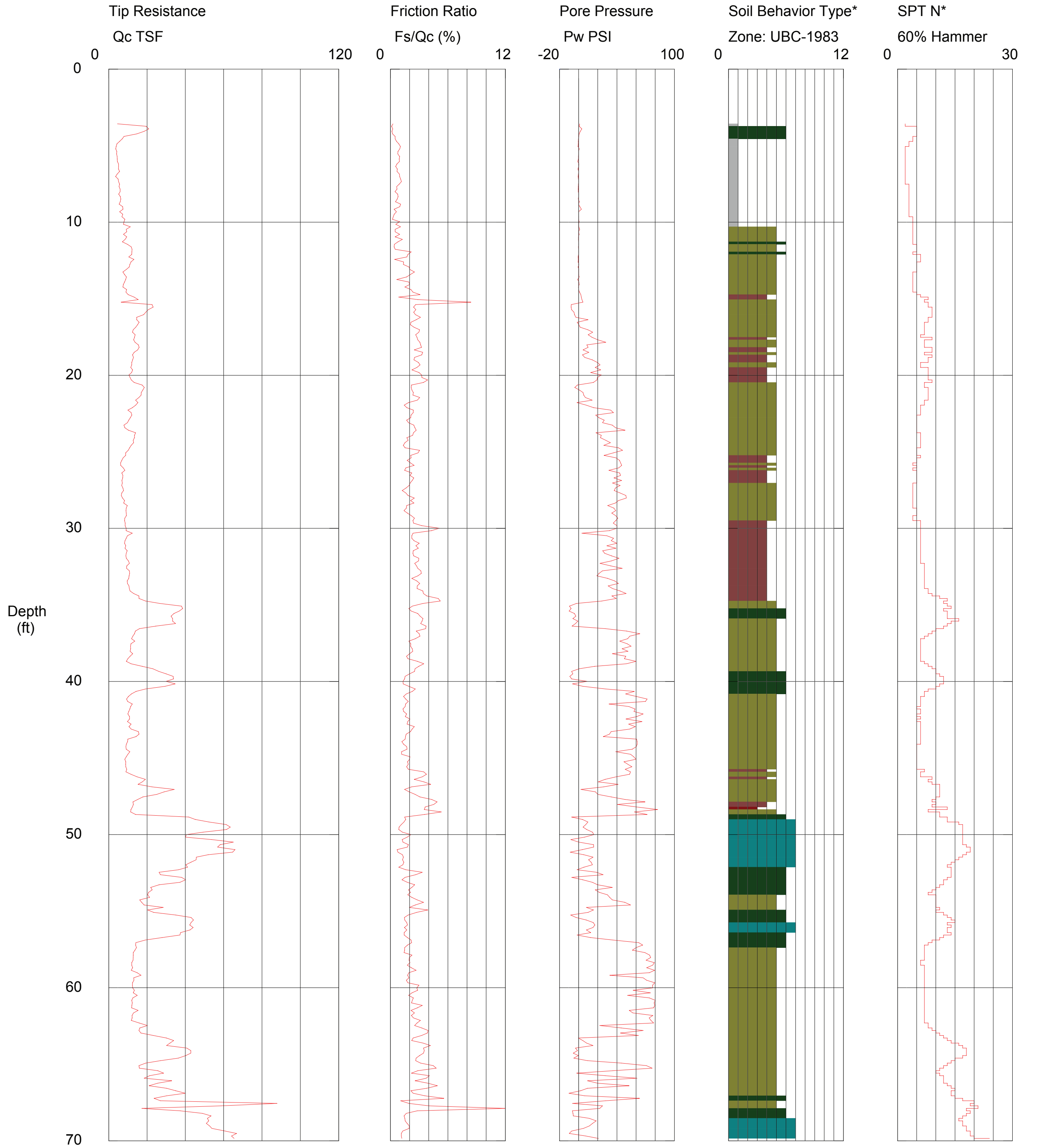
WILDCAT DYNAMIC CONE LOG

PROJECT: Sacramento ESC PROJECT NUMBER: S9840-05-01

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
- 30 ft	25	47.3		13	MEDIUM DENSE	STIFF
-	22	41.6		11	MEDIUM DENSE	STIFF
-	17	32.1		9	LOOSE	STIFF
- 31 ft	17	32.1		9	LOOSE	STIFF
-	16	30.2		8	LOOSE	MEDIUM STIFF
-	15	28.4		8	LOOSE	MEDIUM STIFF
-	21	39.7		11	MEDIUM DENSE	STIFF
- 32 ft	26	49.1		14	MEDIUM DENSE	STIFF
-	28	52.9		15	MEDIUM DENSE	STIFF
- 10 m	28	52.9		15	MEDIUM DENSE	STIFF
- 33 ft	28	52.9		15	MEDIUM DENSE	STIFF
-	31	58.6		16	MEDIUM DENSE	VERY STIFF
-	46	86.9		24	MEDIUM DENSE	VERY STIFF
- 34 ft	39	73.7		21	MEDIUM DENSE	VERY STIFF
-	37	69.9		19	MEDIUM DENSE	VERY STIFF
-	38	71.8		20	MEDIUM DENSE	VERY STIFF
- 35 ft	36	68.0		19	MEDIUM DENSE	VERY STIFF
-	39	73.7		21	MEDIUM DENSE	VERY STIFF
-	40	75.6		21	MEDIUM DENSE	VERY STIFF
- 11 m 36 ft	38	71.8		20	MEDIUM DENSE	VERY STIFF
-	38	71.8		20	MEDIUM DENSE	VERY STIFF
-	34	64.3		18	MEDIUM DENSE	VERY STIFF
- 37 ft	36	68.0		19	MEDIUM DENSE	VERY STIFF
-	32	60.5		17	MEDIUM DENSE	VERY STIFF
-	27	51.0		14	MEDIUM DENSE	STIFF
- 38 ft	27	51.0		14	MEDIUM DENSE	STIFF
-	27	51.0		14	MEDIUM DENSE	STIFF
-	29	54.8		15	MEDIUM DENSE	STIFF
- 39 ft	32	60.5		17	MEDIUM DENSE	VERY STIFF
- 12 m	32	60.5		17	MEDIUM DENSE	VERY STIFF
-	35	66.2		18	MEDIUM DENSE	VERY STIFF
- 40 ft	32	60.5		17	MEDIUM DENSE	VERY STIFF
-	38	71.8		20	MEDIUM DENSE	VERY STIFF
-	40	75.6		21	MEDIUM DENSE	VERY STIFF
- 41 ft	37	69.9		19	MEDIUM DENSE	VERY STIFF
-	37	69.9		19	MEDIUM DENSE	VERY STIFF
-	40	75.6		21	MEDIUM DENSE	VERY STIFF
- 42 ft	38	71.8		20	MEDIUM DENSE	VERY STIFF
-	40	75.6		21	MEDIUM DENSE	VERY STIFF
- 13 m	50	94.5		-	MEDIUM DENSE	VERY STIFF
- 43 ft	44	83.2		23	MEDIUM DENSE	VERY STIFF
-	44	83.2		23	MEDIUM DENSE	VERY STIFF
-	48	90.7		-	MEDIUM DENSE	VERY STIFF
- 44 ft	49	92.6		-	MEDIUM DENSE	VERY STIFF
-	49	92.6		-	MEDIUM DENSE	VERY STIFF
-	44	83.2		23	MEDIUM DENSE	VERY STIFF
- 45 ft	45	85.1		24	MEDIUM DENSE	VERY STIFF
-						
-						
- 14 m 46 ft						

WILDCAT.XLS

Figure A36



Maximum Depth = 70.83 feet

Depth Increment = 0.164 feet

1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT1

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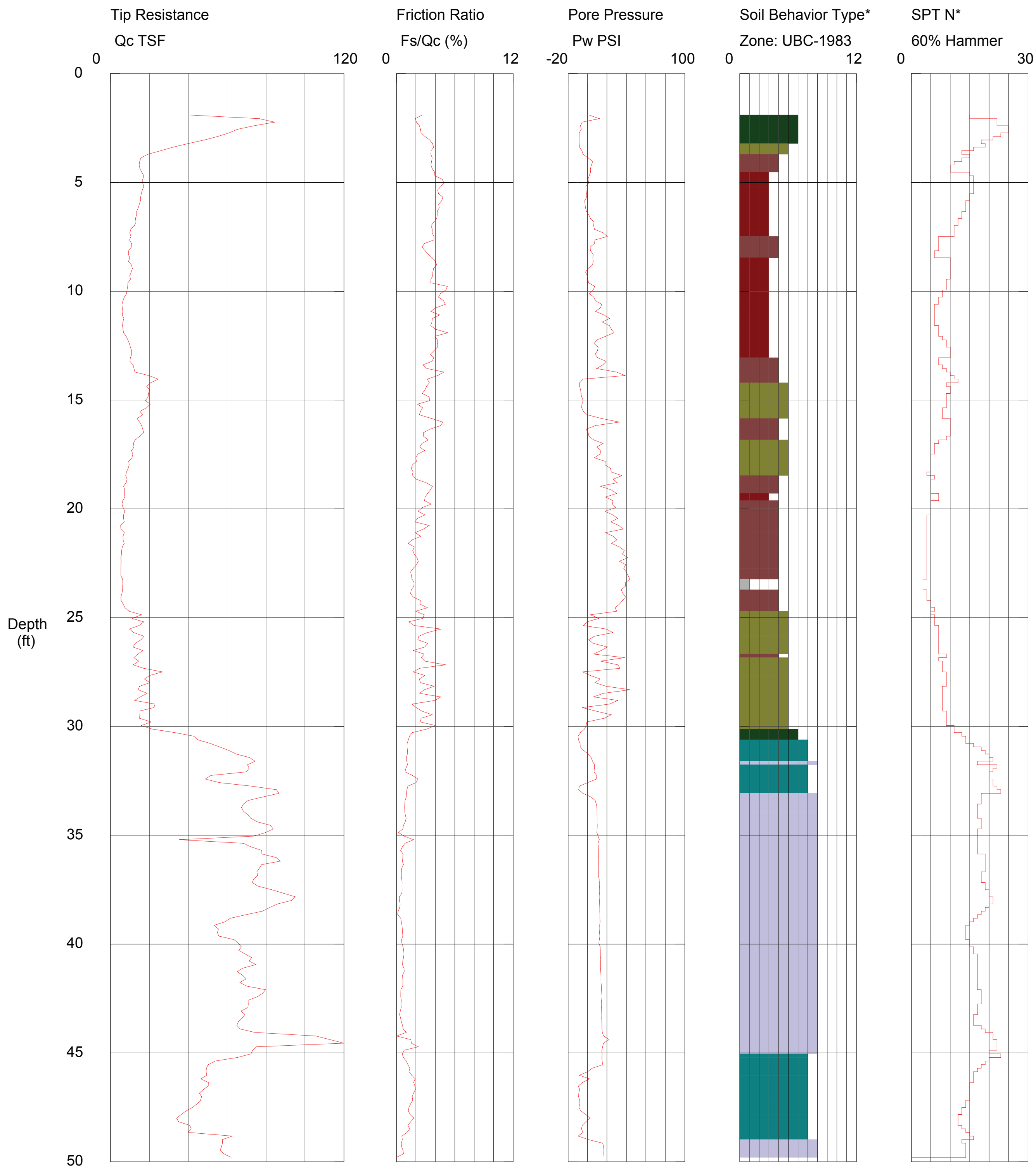
S9840-05-01	November 2013	Figure A37



Maximum Depth = 49.80 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT2

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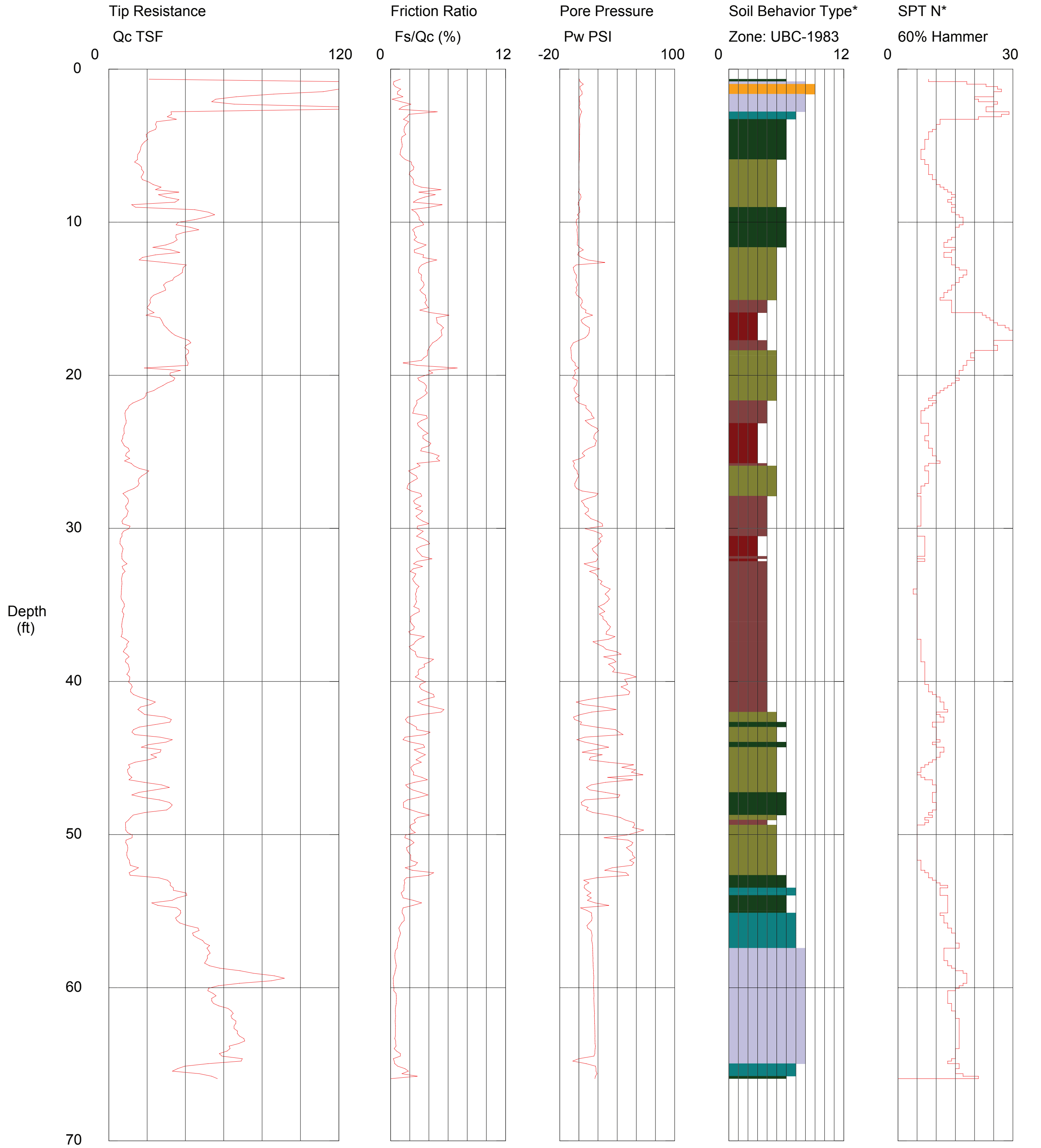
S9840-05-01	November 2013	Figure A38



Maximum Depth = 65.94 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT3

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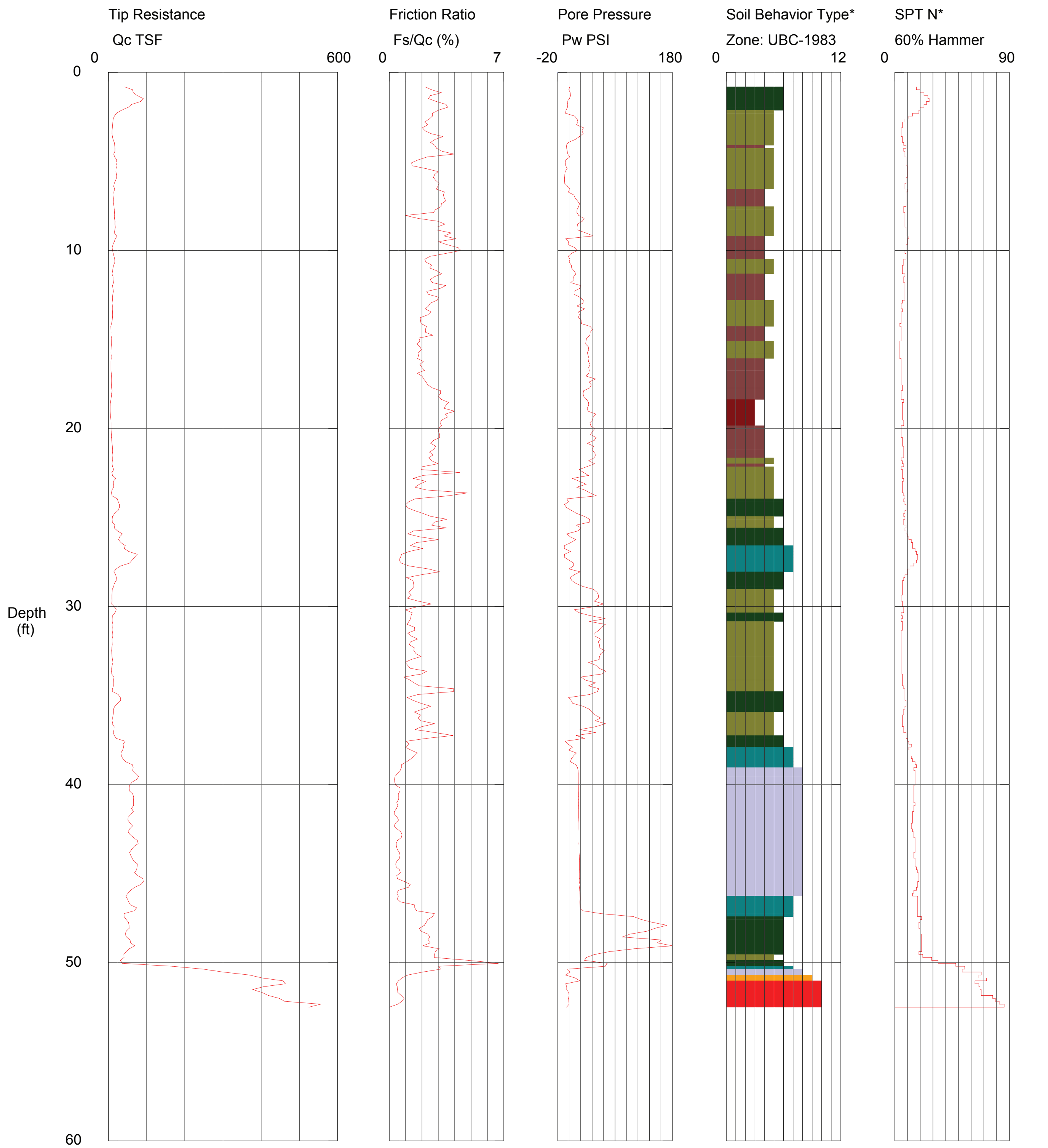
Sacramento Entertainment and Sports Complex		
Sacramento, California		
S9840-05-01	November 2013	Figure A39



Maximum Depth = 52.49 feet

Depth Increment = 0.164 feet

1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT4

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Sacramento Entertainment and Sports Complex

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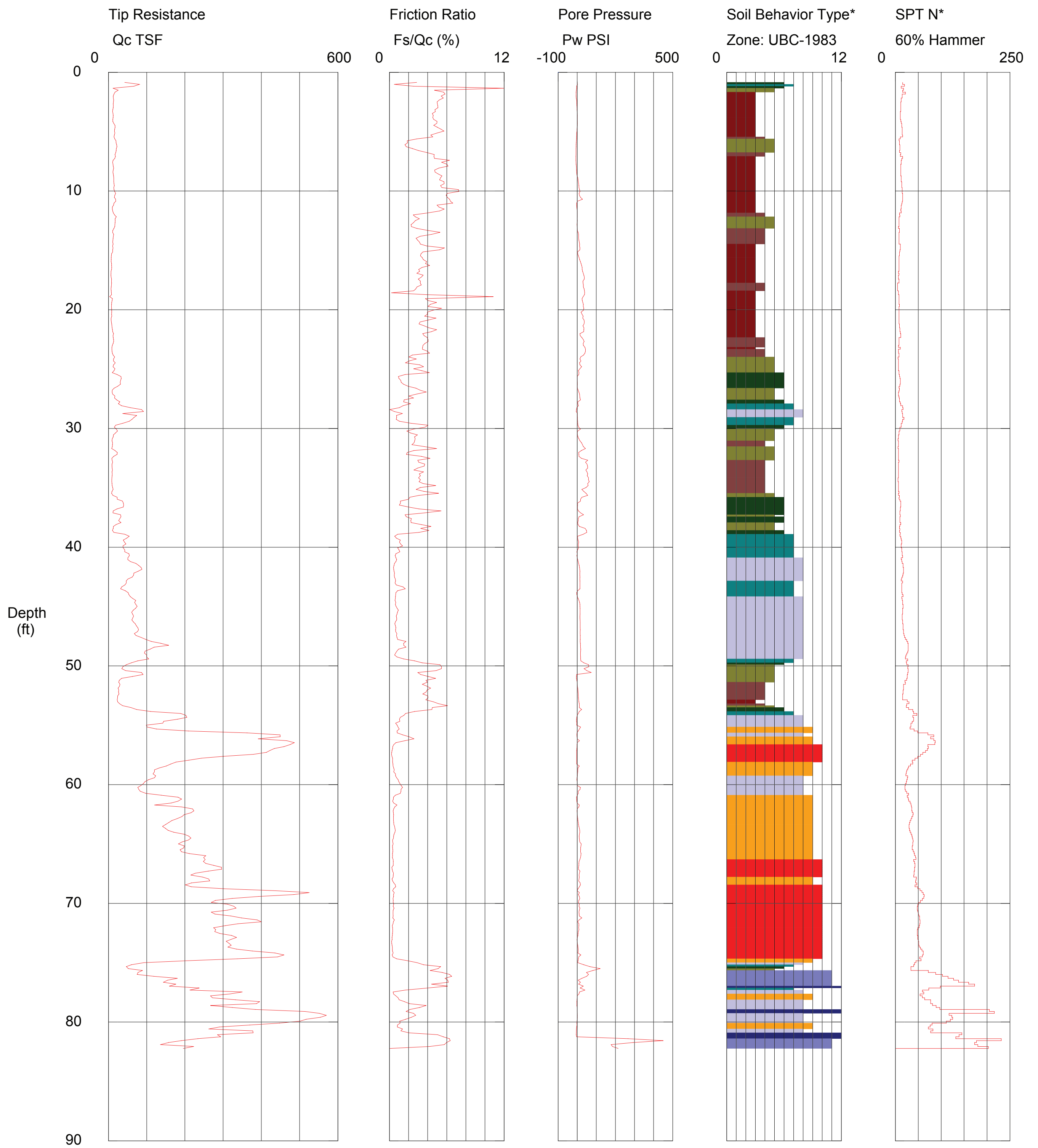
California

S9840-05-01	November 2013	Figure A40



Maximum Depth = 82.22 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT5

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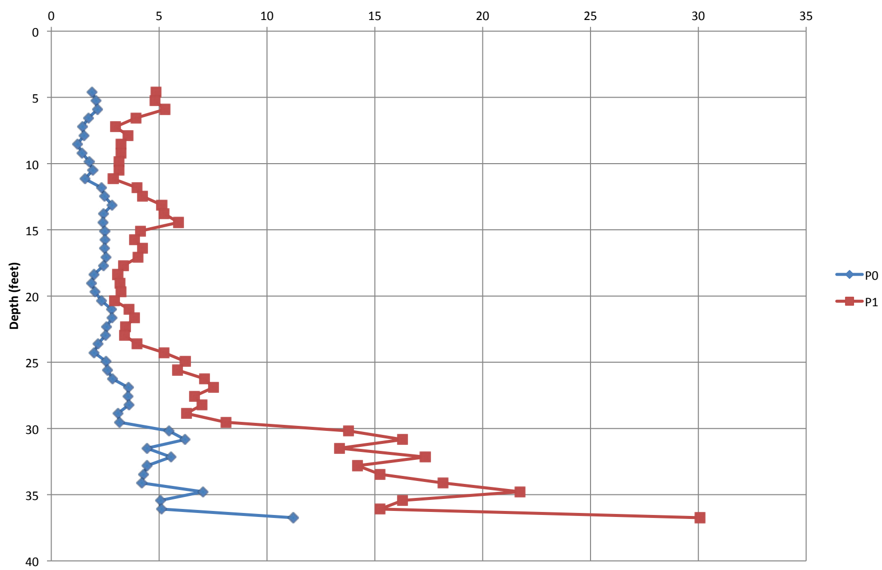
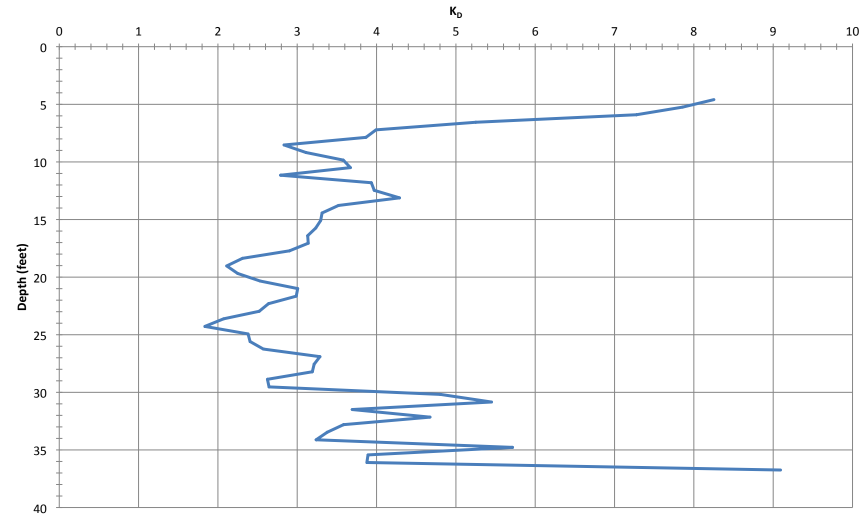
S9840-05-01	November 2013	Figure A41

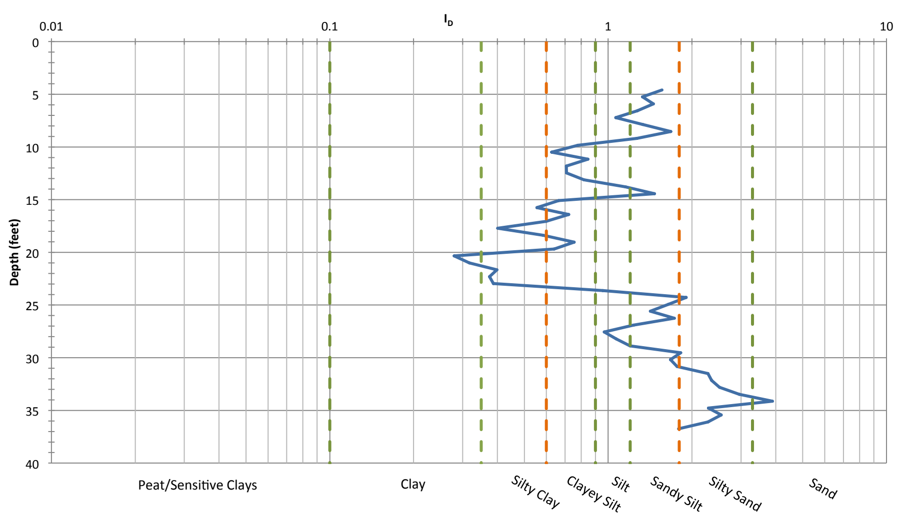
P0, P1



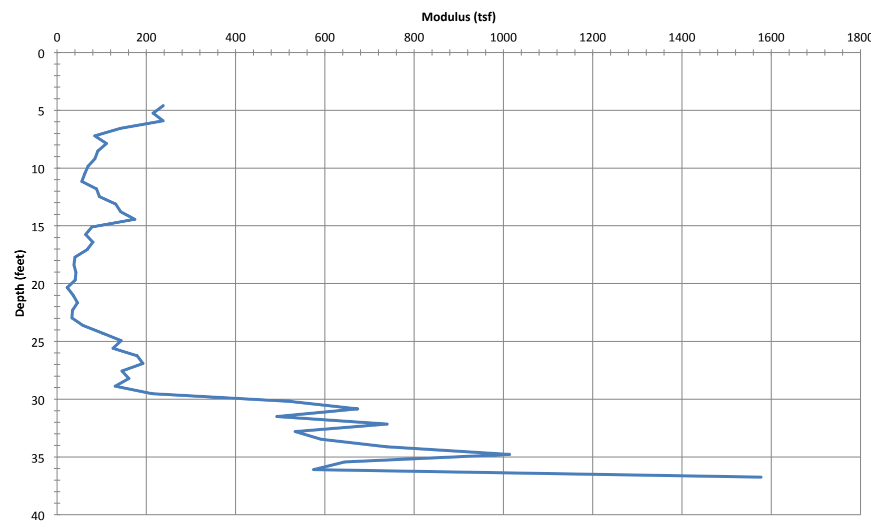
Horizontal Stress Index



Soil Index



DMT Modulus



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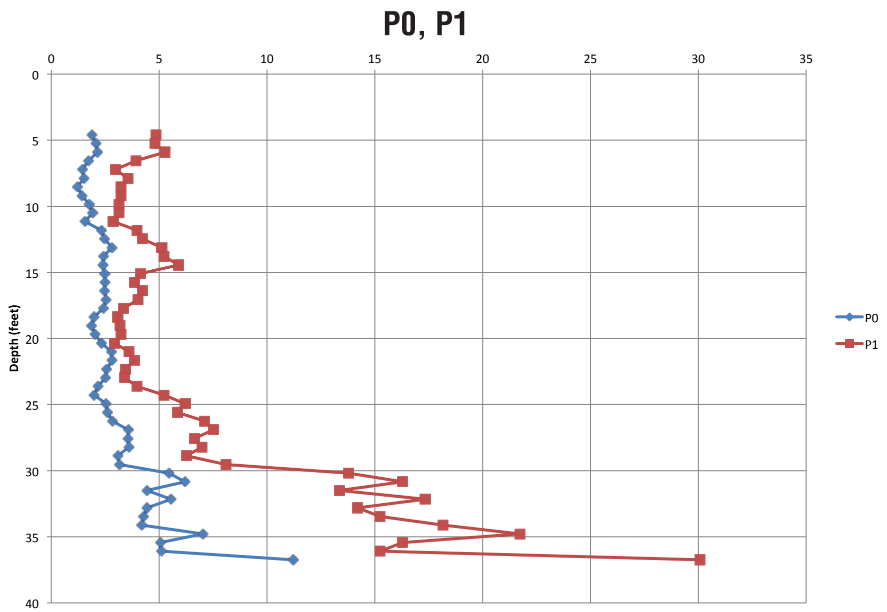
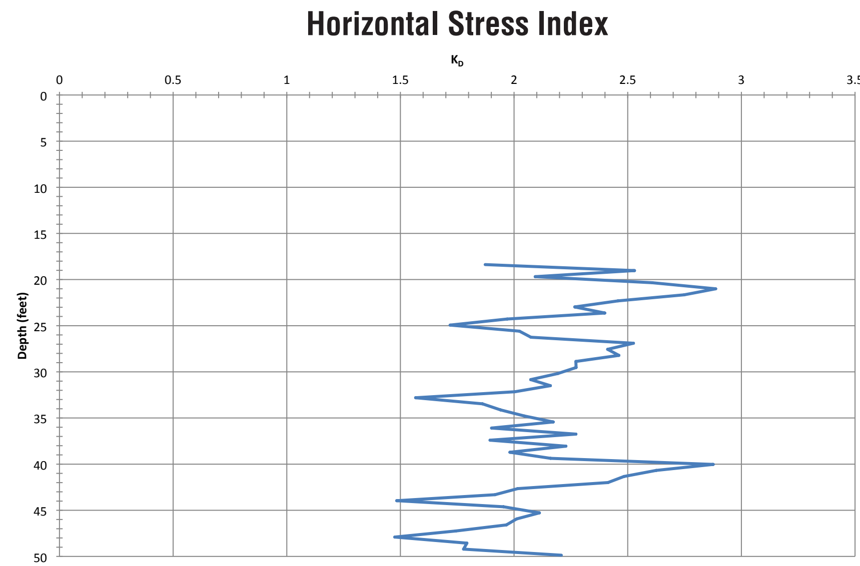
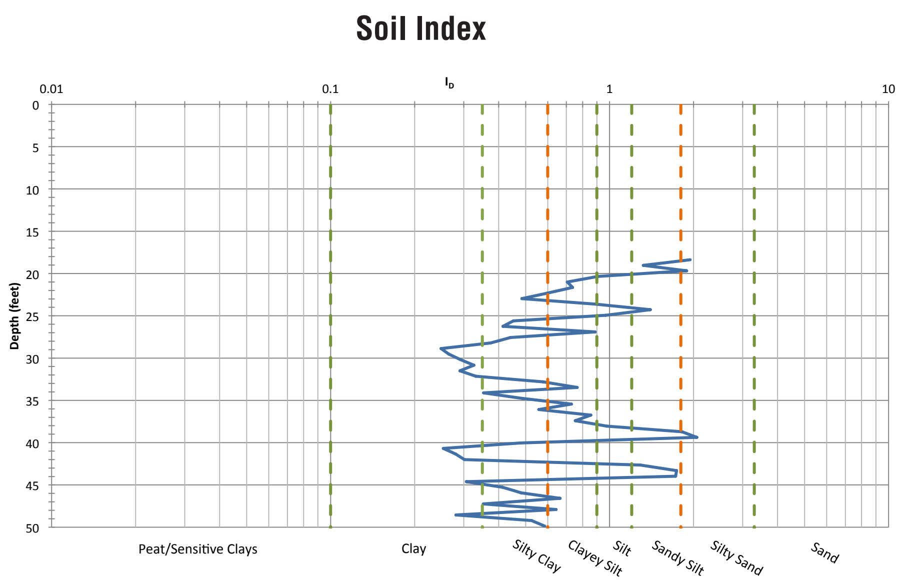
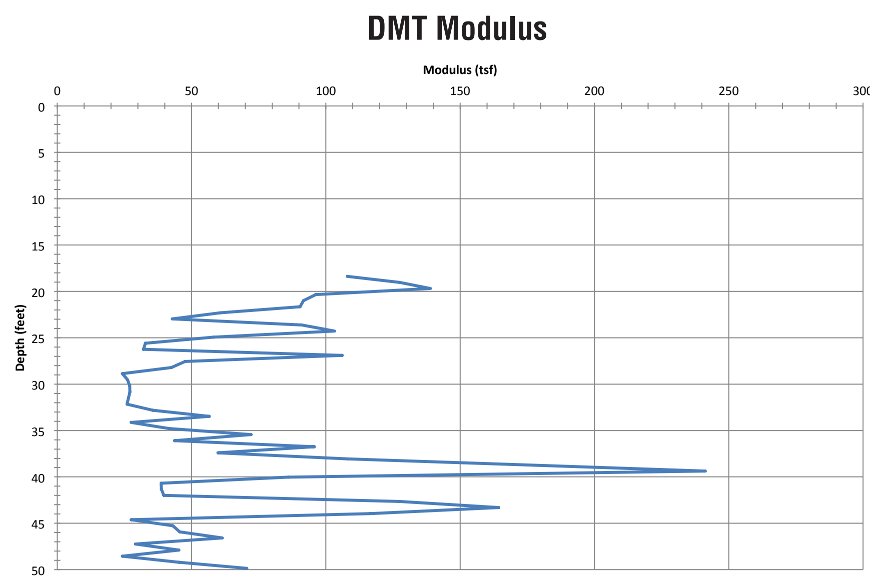
DMT1

Sacramento Entertainment and Sports Center		
Sacramento, California		
S9840-05-01	November 2013	Figure A42









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DMT2

Sacramento Entertainment and Sports Center		
Sacramento, California		
S9840-05-01	November 2013	Figure A43

APPENDIX B

APPENDIX B

LABORATORY TESTING PROGRAM

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their in-situ dry density and moisture content, grain size distribution, plasticity characteristics, expansion potential, shear strength parameters, and corrosion potential. The results of the laboratory tests are presented on the following tables and pages.

TABLE B1
CORROSION PARAMETER TEST RESULTS
(CALIFORNIA TEST METHODS 643, 417, AND 422)

Sample No.	Sample Depth (ft.)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm) / (%)	Sulfate (ppm) / (%)
Composite: IB1, IB2, IB3	1 – 5	7.83	1,960	21.5 / 0.00215	79.3 / 0.00793
B1-30-31.5	30 – 31.5	6.22	2,570	9.1 / 0.00091	70.0 / 0.00700
B4-65-65.5	65 – 65.5	6.05	2,550	19.0 / 0.00190	74.7 / 0.00747

*Caltrans considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil samples at the site:

- The pH is equal to or less than 5.5.
- The resistivity is equal to or less than 1,000 ohm-cm.
- Chloride concentration is equal to or greater than 500 parts per million (ppm).
- Sulfate concentration is equal to or greater than 2,000 ppm.

According to the 2013 California Building Code Section 1904.1 which refers to the durability requirements of American Concrete Institute (ACI) 318 (Chapter 4), Type II cement may be used where soluble sulfate levels in soil are below 2,000 ppm.

TABLE B2
EXPANSION INDEX TEST RESULTS (ASTM D4829)

Sample No.	Sample Depth (ft.)	Moisture Content		Dry Density		Expansion Index	Expansion Potential based on Expansion Index
		Before Test (%)	After Test (%)	Before Test (pcf)	After Test (pcf)		
Composite: IB1, IB2, IB3	1 – 5	13.0	25.5	100.9	96.7	**33**	Low

Sample ID	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Expansion Index	%<#200 Sieve	Water Content (%)	Dry Density (pcf)
B1-6	6	37	29	8			37.8	84.2
B1-16	16					80.6	36.4	
B1-25	25	NP	NP	NP		37.7	35.6	
B1-26	26					31.6	33.5	88.7
B1-35	35					12.7	32.4	
B1-40	40					6.4	27.6	
B1-45	45					8.5	26.3	
B1-50	50					51.1	58.1	
B1-60	60						24.2	
B1-70	70					12.6	16.4	
B1-75	75					7.2	33.8	
B1-80	80					7.6	22.2	
B1-90	90					46.8	40.9	
B2-31	31	39	33	6			45.6	77.1
B2-36	36					15.2	31.4	89.8
B2-40	40					5.8	36.3	
B2-46	46					4.3	25.8	
B2-50	50					4.7	35.8	
B2-56	56						7.0	
B3-21	21						30.3	90.9
B3-26	26					85.5	34.0	87.9
B3-35	35	35	29	6			41.2	
B3-41	41						39.3	83.0
B3-45	45	36	28	8			42.3	
B3-55	55					10.3	30.5	
B3-66	66					7.9	25.2	96.4
B4-21	21						35.4	87.6
B4-30	30	38	28	10			40.4	
B4-35	35					91.8	44.1	
B4-40	40						38.0	
B4-50	50					41.4	49.6	
B4-55	55					62.9	46.5	
B4-65	65						50.1	
B4-70	70					38.4	39.6	

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Summary of Laboratory Results

Project: Sacramento Entertainment & Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B1

Sample ID	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Expansion Index	%<#200 Sieve	Water Content (%)	Dry Density (pcf)
B5-15.5	15.5	36	31	5			42.2	79.1
B5-20	20						43.4	78.2
B5-30	30					32.9	33.2	
B5-35	35						45.6	
B5-45	45					19.7		
B5-55	55					26.0	24.4	
B5-60	60					9.2	33.1	
B5-70	70					5.9	15.7	
IB 1,2,5 comp.	0-5				33			
IB4-8	8	30	27	3		94.8	37.1	

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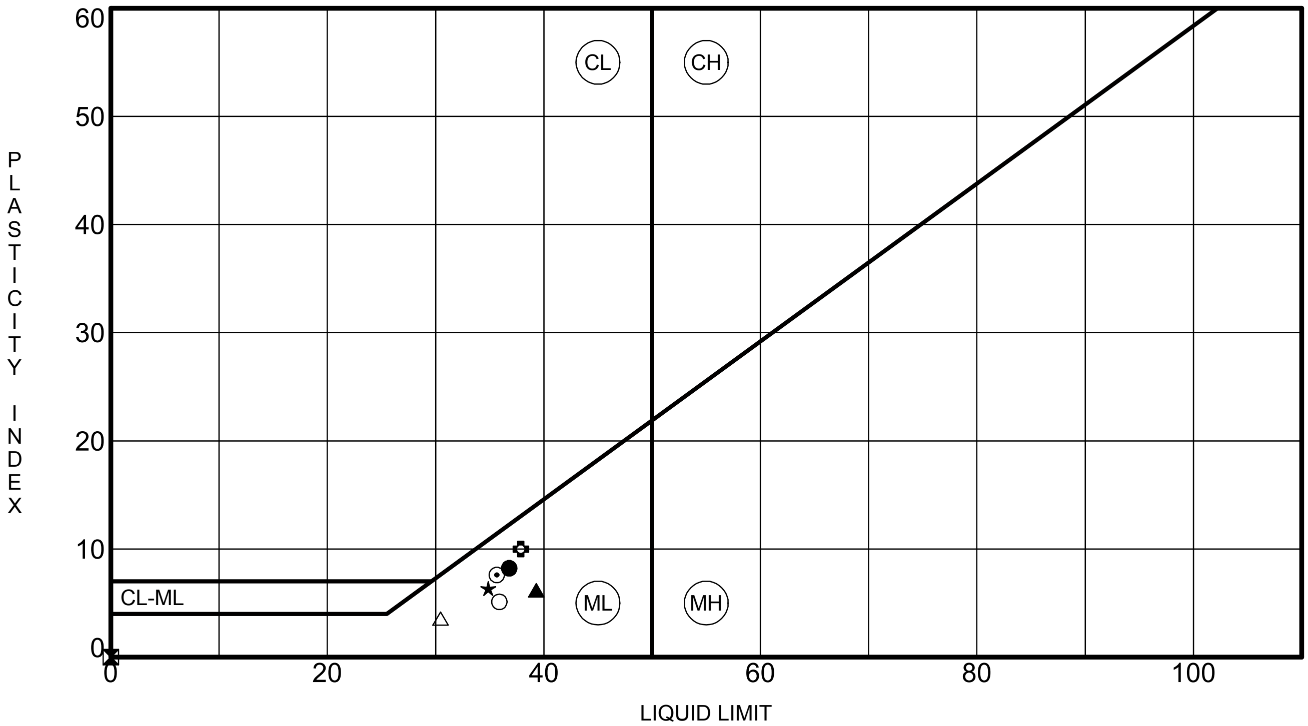
Summary of Laboratory Results

Project: Sacramento Entertainment & Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B1



	Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Unified Soil Classification Description	Preparation Method
●	**B1-6**	**37**	**29**	**8**		**SILT (ML)**	**wet**
⊠	**B1-25**	**NP**	**NP**	**NP**	**37.7**	**SILTY SAND(SM)**	**wet**
▲	**B2-31**	**39**	**33**	**6**		**SILT (ML)**	**wet**
★	**B3-35**	**35**	**29**	**6**		**SILT (ML)**	**wet**
⊙	**B3-45**	**36**	**28**	**8**		**SILT (ML)**	**wet**
✿	**B4-30**	**38**	**28**	**10**		**SILT (ML)**	**wet**
○	**B5-15.5**	**36**	**31**	**5**		**SILT (ML)**	**wet**
△	**IB4-8**	**30**	**27**	**3**	**94.8**	**SILT(ML)**	**wet**

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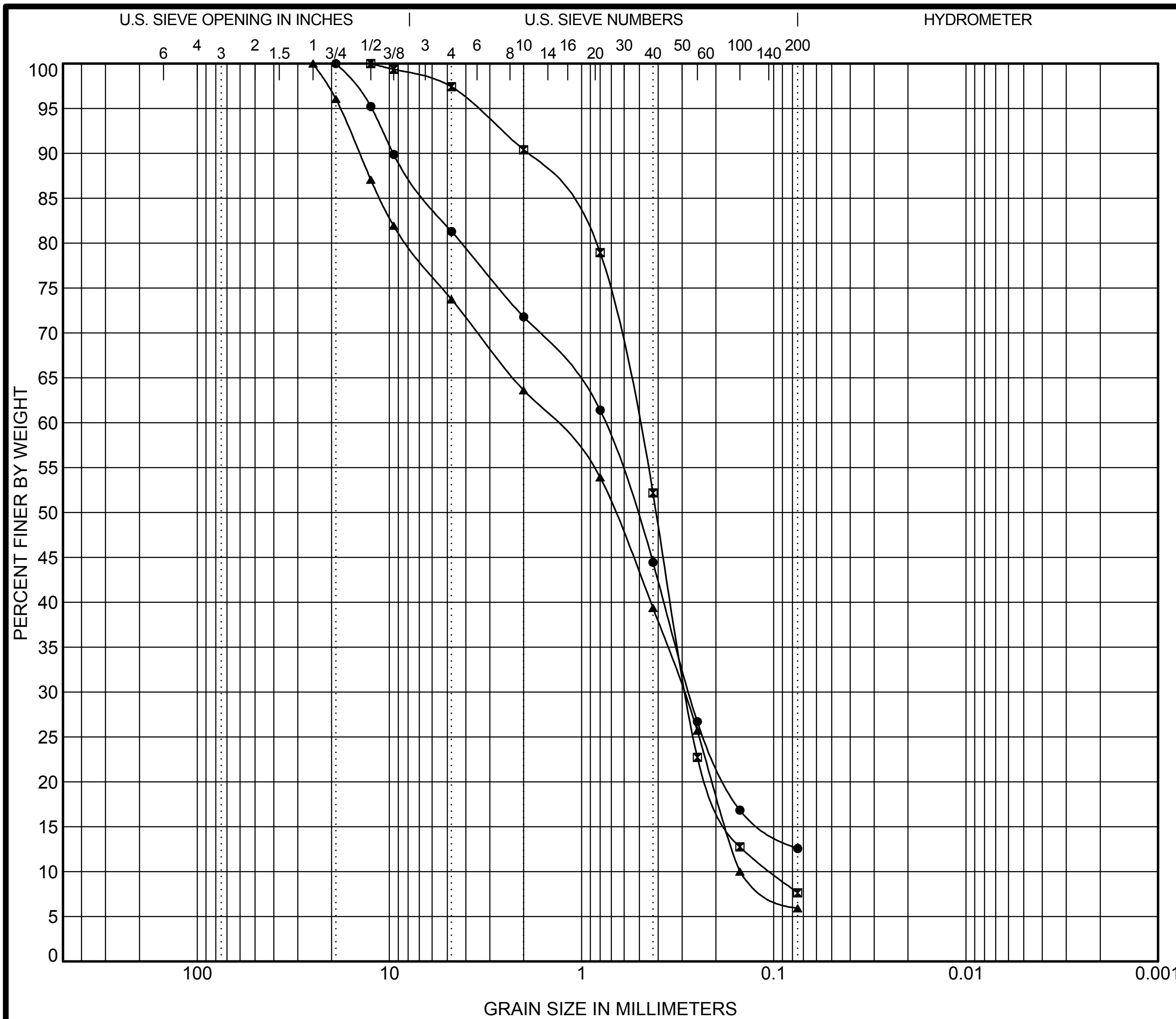
ATTERBERG LIMITS (ASTM D4318)

Project: Sacramento Entertainment & Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample No.	Classification	LL	PL	PI	Cc	Cu
●	**B1-70**	**Silty SAND with gravel (SM)**					
⊠	**B1-80**	**Poorly graded SAND with silt (SP-SM)**				**1.54**	**5.0**
▲	**B5-70**	**Poorly graded SAND with gravel and silt (SP-SM)**				**0.41**	**9.5**

	Sample No.	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B1-70**	**19**	**0.759**	**0.276**		**18.7**	**68.7**	**12.6**	
⊠	**B1-80**	**12.5**	**0.511**	**0.285**	**0.103**	**2.6**	**89.8**	**7.6**	
▲	**B5-70**	**25**	**1.423**	**0.295**	**0.15**	**26.3**	**67.8**	**5.9**	

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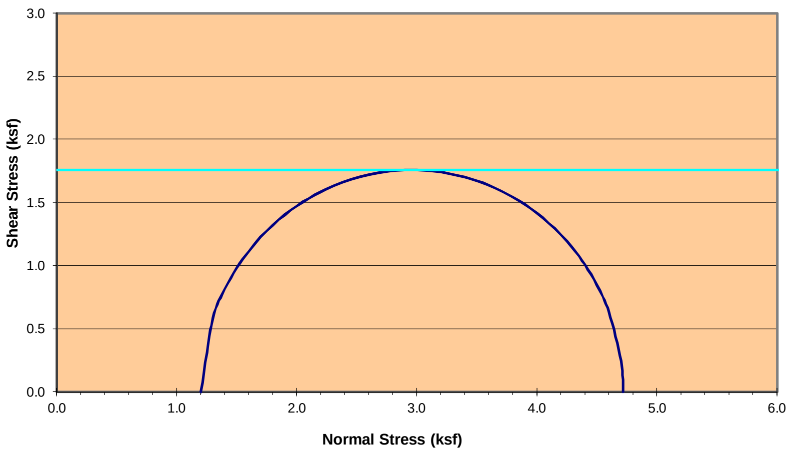
GRAIN SIZE DISTRIBUTION (ASTM D422, D6913)

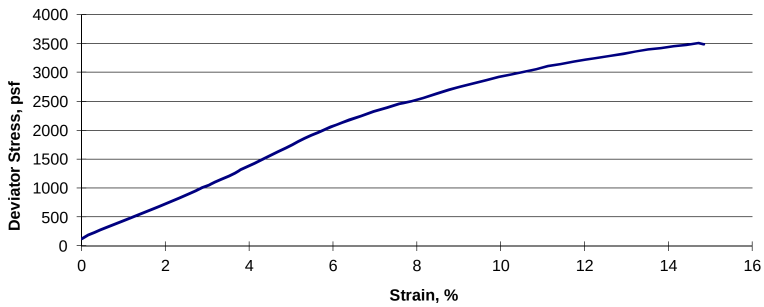
Project: Sacramento Entertainment & Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B3

GRAIN SIZE COPY 2 S9840-05-01 SAC ESC.GPJ US_LAB.GDT 11/11/13

MOHR'S CIRCLES



STRESS-STRAIN



Failure Photo

Sample Description	
Sample Number	B2
Sample Depth (feet)	26
Material Description	dark grayish brown Sandy SILT
Initial Conditions at Start of Test	
Height (inch)	4.52
Diameter (inch)	2.40
Moisture Content (%)	27.3
Dry Density (pcf)	105.3
Estimated Specific Gravity	2.8
Saturation (%)	113.9
Shear Test Conditions	
Strain Rate (%/min)	0.9376
Major Principle Stress at Failure (psf)	4720
Minor Principle Stress, Cell Pressure (psf)	1200
Deviator Stress at Fail (psf)	3520
Test Results	
Friction Angle ϕ, (degrees)	0
Cohesion, (psf)	1760

Note: Strength attibuted to cohesion with no value of friction assigned

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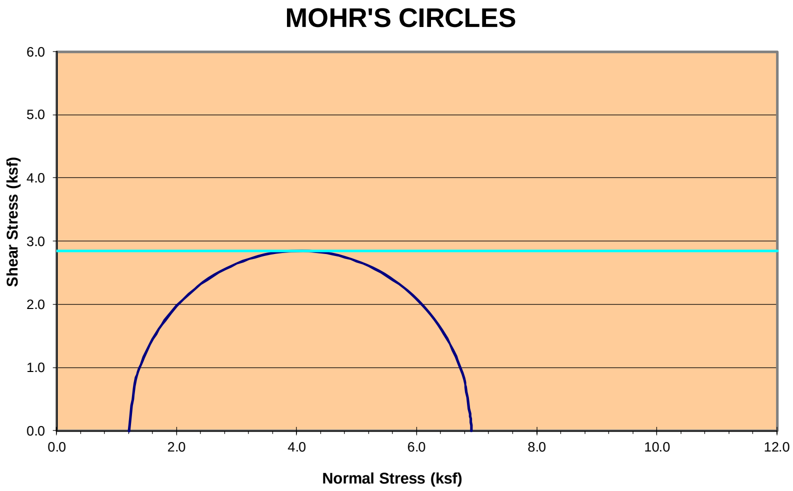
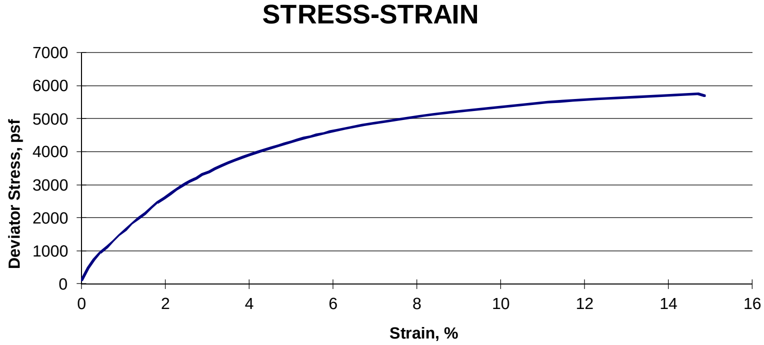
Triaxial Shear Strength - UU Test (single)

Project: Sacramento Entertainment and Sports Center
Location: Sacramento, CA
Number: S98440-05-01
Figure: B4





Failure Photo

Sample Description	
Sample Number	B3
Sample Depth (feet)	16
Material Description	dark grayish brown Sandy SILT
Initial Conditions at Start of Test	
Height (inch)	4.95
Diameter (inch)	2.40
Moisture Content (%)	27.3
Dry Density (pcf)	96.1
Estimated Specific Gravity	2.8
Saturation (%)	92.2
Shear Test Conditions	
Strain Rate (%/min)	0.9903
Major Principle Stress at Failure (psf)	6900
Minor Principle Stress, Cell Pressure (psf)	1200
Deviator Stress at Fail (psf)	5700
Test Results	
Friction Angle ϕ, (degrees)	0
Cohesion, (psf)	2850

Note: Strength attibuted to cohesion with no value of friction assigned

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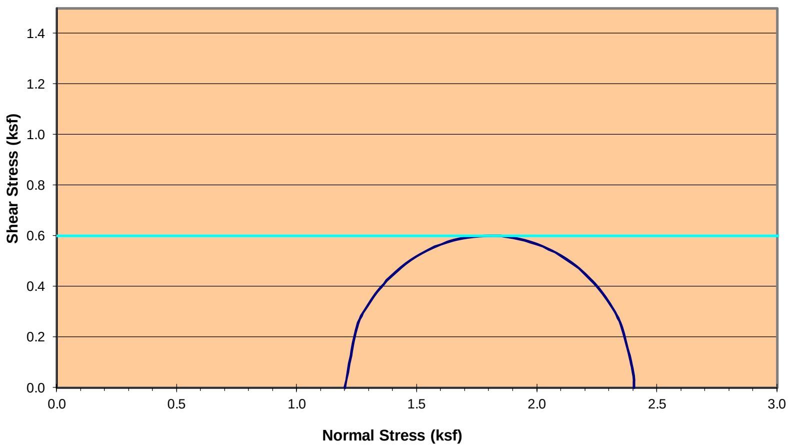
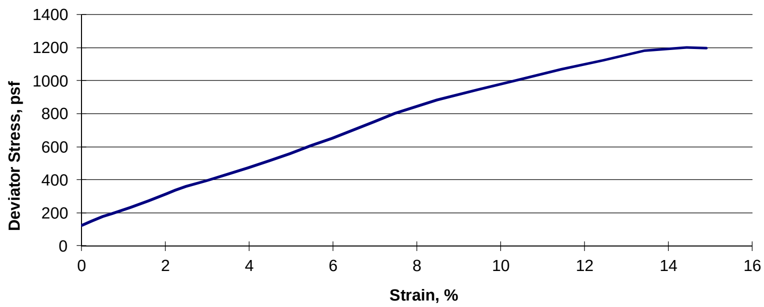
Triaxial Shear Strength - UU Test (single)

Project: Sacramento Entertainment and Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B5

MOHR'S CIRCLES



STRESS-STRAIN



Failure Photo

Sample Description	
Sample Number	B3
Sample Depth (feet)	10
Material Description	Dark brown SILT
Initial Conditions at Start of Test	
Height (inch)	4.90
Diameter (inch)	2.38
Moisture Content (%)	47.9
Dry Density (pcf)	76.1
Estimated Specific Gravity	3.0
Saturation (%)	99.6
Shear Test Conditions	
Strain Rate (%/min)	0.9930
Major Principle Stress at Failure (psf)	2400
Minor Principle Stress, Cell Pressure (psf)	1200
Deviator Stress at Fail (psf)	1200
Test Results	
Friction Angle ϕ, (degrees)	0
Cohesion, (psf)	600

Note: Strength attibuted to cohesion with no value of friction assigned

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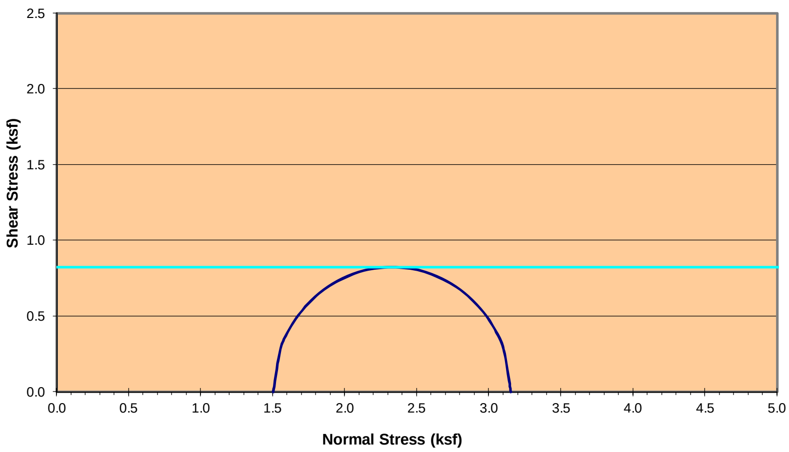
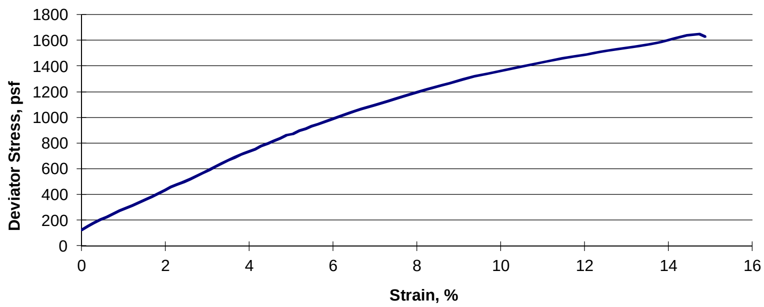
Triaxial Shear Strength - UU Test (single)

Project: Sacramento Entertainment and Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B6

MOHR'S CIRCLES



STRESS-STRAIN



Failure Photo

Sample Description	
Sample Number	B5
Sample Depth (feet)	15
Material Description	Dark brown SILT
Initial Conditions at Start of Test	
Height (inch)	4.90
Diameter (inch)	2.39
Moisture Content (%)	44.0
Dry Density (pcf)	79.1
Estimated Specific Gravity	2.9
Saturation (%)	99.6
Shear Test Conditions	
Strain Rate (%/min)	0.9918
Major Principle Stress at Failure (psf)	3150
Minor Principle Stress, Cell Pressure (psf)	1500
Deviator Stress at Fail (psf)	1650
Test Results	
Friction Angle ϕ, (degrees)	0
Cohesion, (psf)	824

Note: Strength attibuted to cohesion with no value of friction assigned

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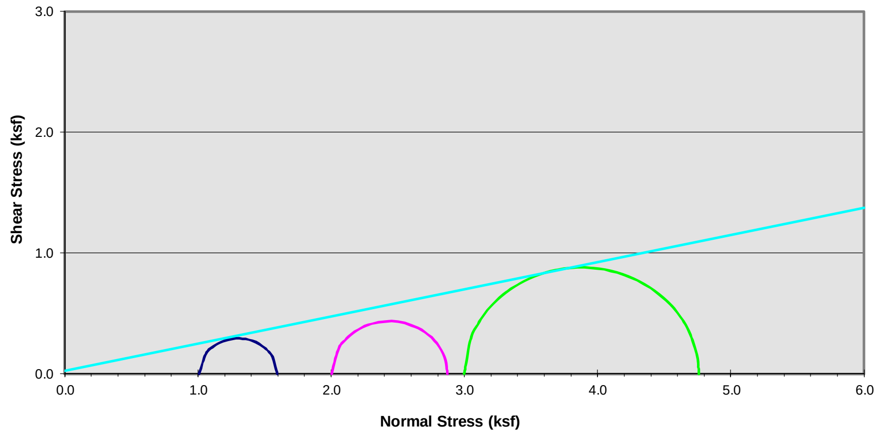
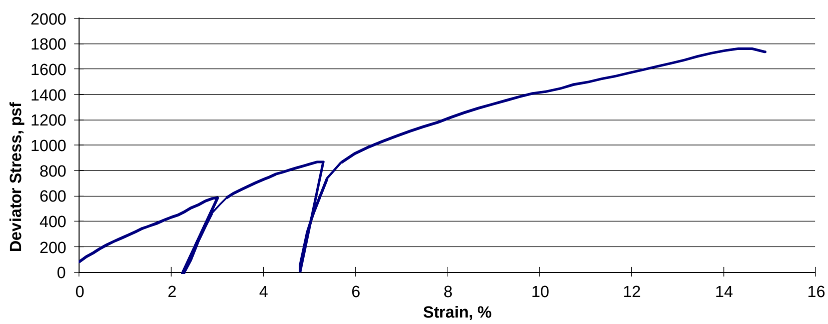
Triaxial Shear Strength - UU Test (single)

Project: Sacramento Entertainment and Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B7

MOHR'S CIRCLES



STRESS-STRAIN



Failure Photo

Test Results

	ϕ, degrees	**12.68**
	c, psf	**25**

Sample Description

	Sample Number	B5-5
	Sample Depth (feet)	5
	Material Description	Dark brown Sandy SILT

Initial Conditions at Start of Stage

Sample ID (psf), minor principal stress	**1010**	**2000**	**3000**
Height (inch)	4.90	4.79	4.62
Diameter (inch)	2.4	2.43	2.47
Moisture Content (%)	32.9	32.9	32.9
Dry Density (pcf)	90.3	90.3	90.3
Saturation (%)	99.7	99.7	99.7

Shear Test Conditions

Strain Rate (%/min)	0.9917	0.8658	0.9929
Major Principle Stress at Failure (psf)	1590	2870	4760
Strain at failure (%)	3.01	5.31	14.32
Deviator Stress and Fail (psf)	590	870	1760

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Triaxial Shear Strength - UU Test (staged)

Project: Sacramento Entertainment and Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B8

CURVE NO. 1

Source of Material: IB1, 2, 5 Bulk Composite

Description of Material: SILT (ML)

Test Method: D1557A

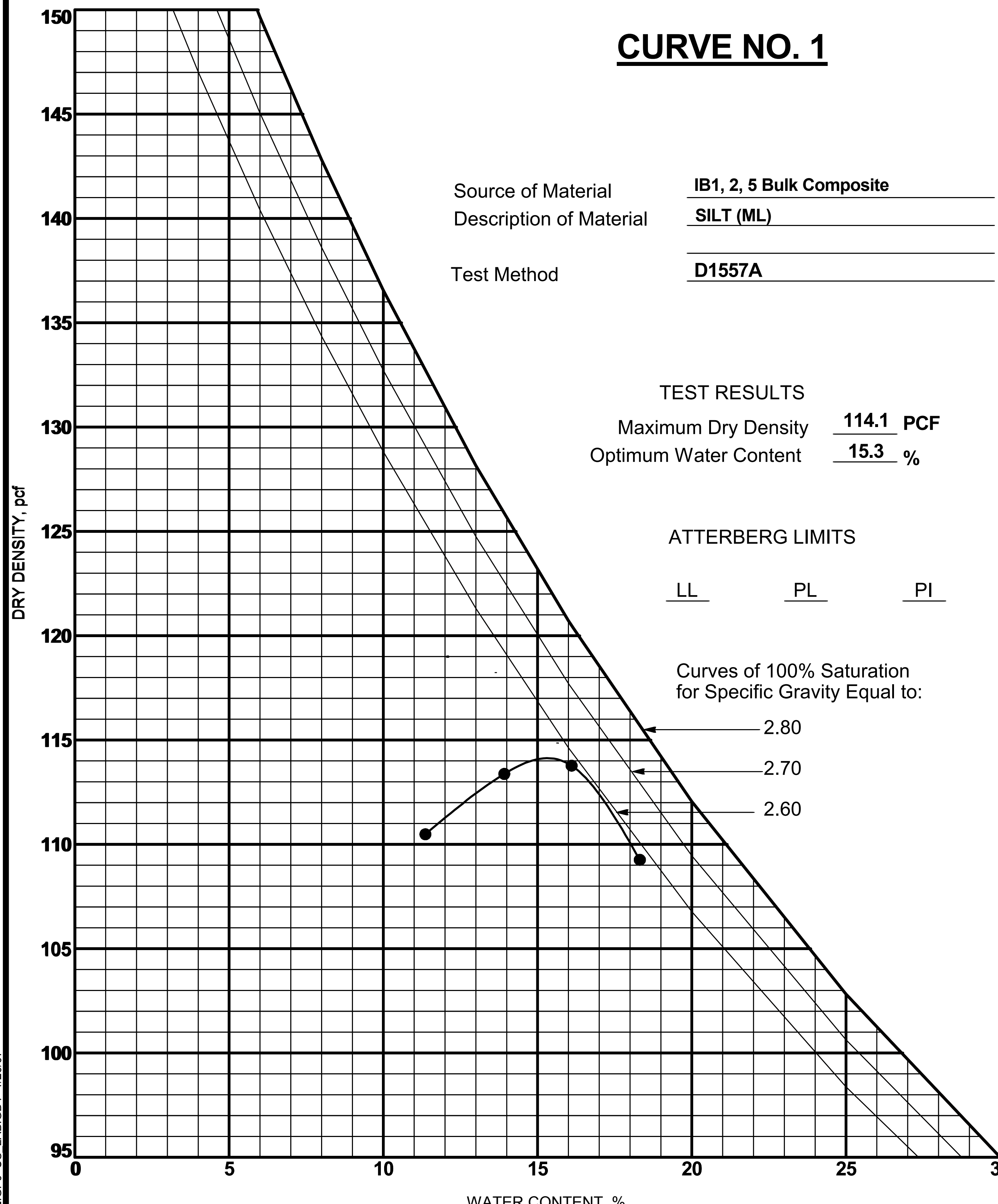
TEST RESULTS

Maximum Dry Density 114.1 PCF

Optimum Water Content 15.3 %

ATTERBERG LIMITS

LL	PL	PI



US COMPACTION COPY 2.GPJ US LAB.GDT 1/26/07

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MOISTURE-DENSITY RELATIONSHIP

Project: Sacramento Entertainment & Sports Center

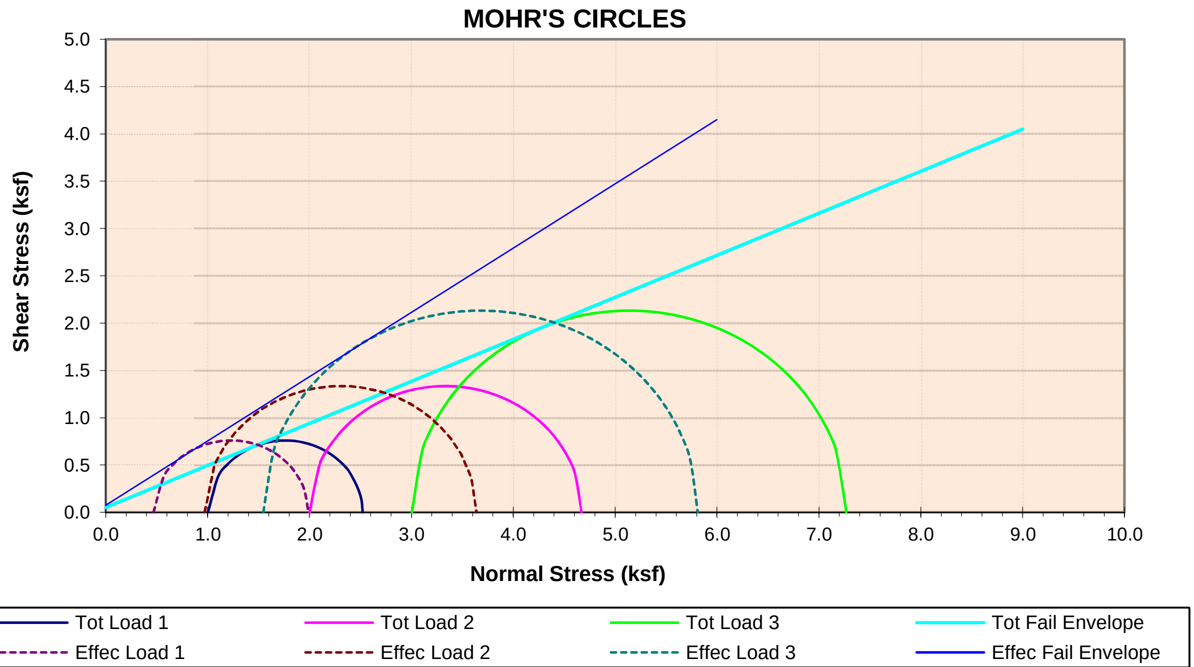
Location: Sacramento, CA

Number: S9840-05-01

Figure: B9

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B1
Sample Number	B1-10
Sample Description	Dark yellowish Brown lean CLAY



Test Results, At Maximum Principal Stress Ratio	Total		Effective
Friction Angle ϕ (degrees)	24.0		34.2
cohesion (psf)	50		75
Initial Conditions at Start of Test	stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure	**1000**	**2000**	**3000**
Height (inch)	4.78	4.65	4.55
Diameter (inch)	2.41	2.42	2.43
Moisture Content (%)	39.1	--	--
Dry Density (pcf)	83.3	--	--
Saturation (%)	99.9	--	--
After Saturation			
Dry Density (pcf)	83.3	--	--
After Consolidation			
Dry Density (pcf)	84.8	--	--
Shear Test Conditions			
Dry Density (pcf)	84.8	86.3	87.3
Moisture Content (%)	--	--	35.3
Saturation (%)	--	--	98.5
Strain rate (%/hr)	3.55	3.65	3.18
Cell pressure (psf)	8230	9250	10250
Initial Back Pressure (psf)	7230	7250	7240
Initial Effective Confining Pressure (psf)	1000	2000	3000
Total Major Principal Stress At Failure (psf)	2520	4670	7270
Effective Major Principal Stress At Failure (psf)	1990	3640	5810
Pore Pressure At Failure (psf)	530	1030	1460
Effective Minor Princial Stress At Failure (psf)	470	970	1550

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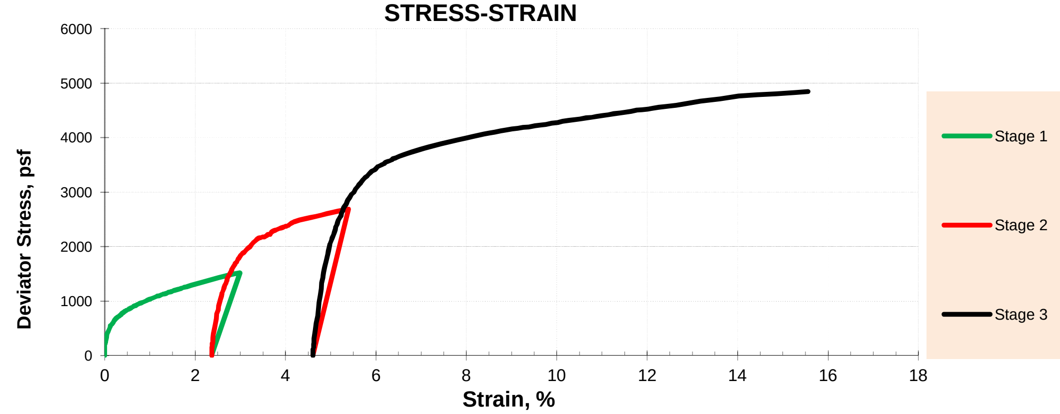
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements (staged)

Project: Sacramento Exercise and Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B10

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

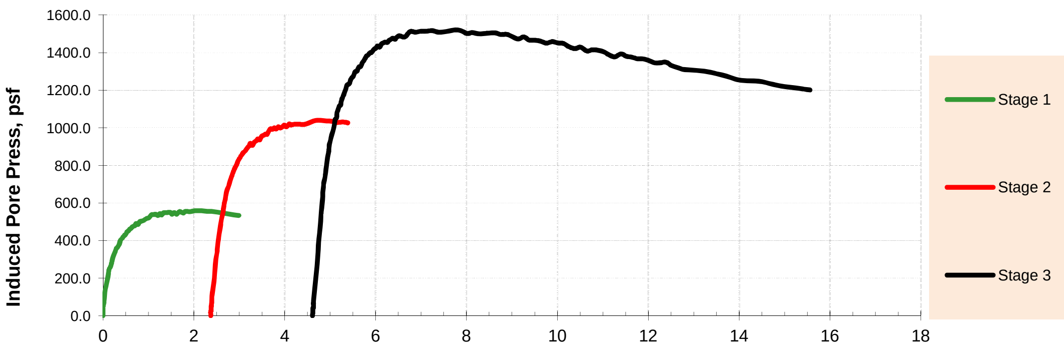
Boring Number	B1
Sample Number	B1-10
Sample Description	Dark yellowish Brown lean CLAY

STRESS-STRAIN

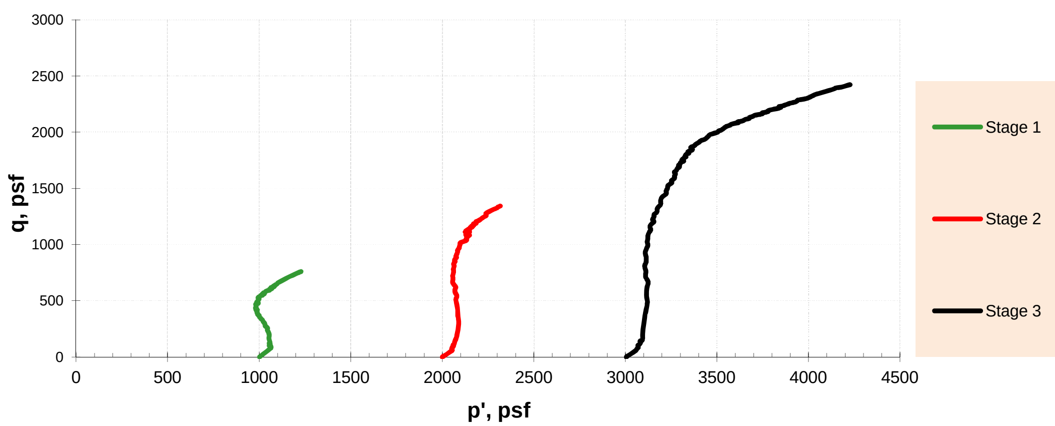


After shear photo

PORE PRESS vs STRAIN



STRESS PATH



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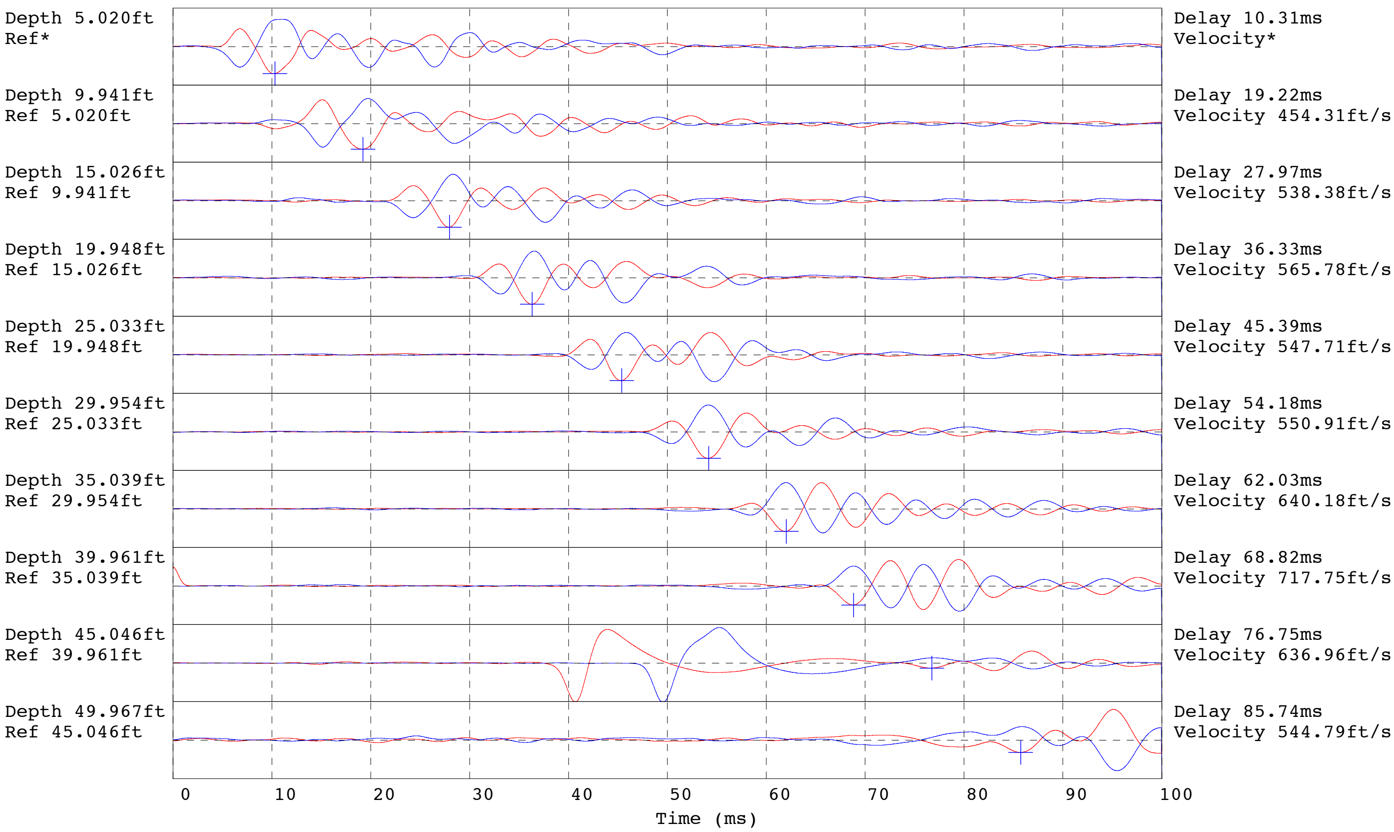
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: Sacramento Exercise and Sports Center
Location: Sacramento, CA
Number: S9840-05-01
Figure: B10

APPENDIX C

APPENDIX C

ENGINEERING PARAMETERS AND CORRELATIONS



Hammer to Rod String Distance 1.52 (m)
* = Not Determined

Shear Wave Velocity – CPT2

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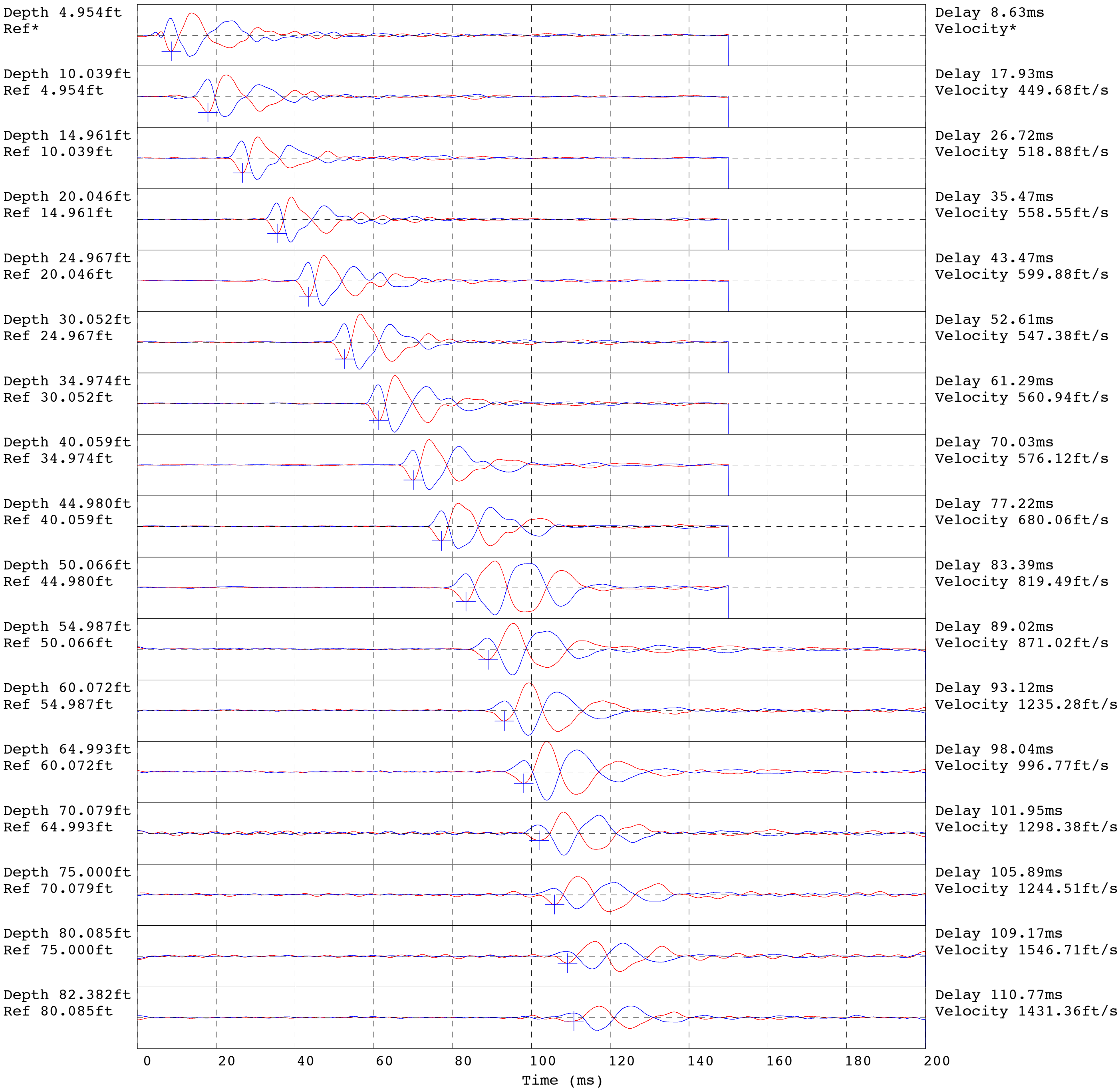
Sacramento Entertainment and Sports Complex

Sacramento,
California

S9840-05-01	November 2013	Figure C1



Hammer to Rod String Distance 1.52 (m)
* = Not Determined

Shear Wave Velocity – CPT5

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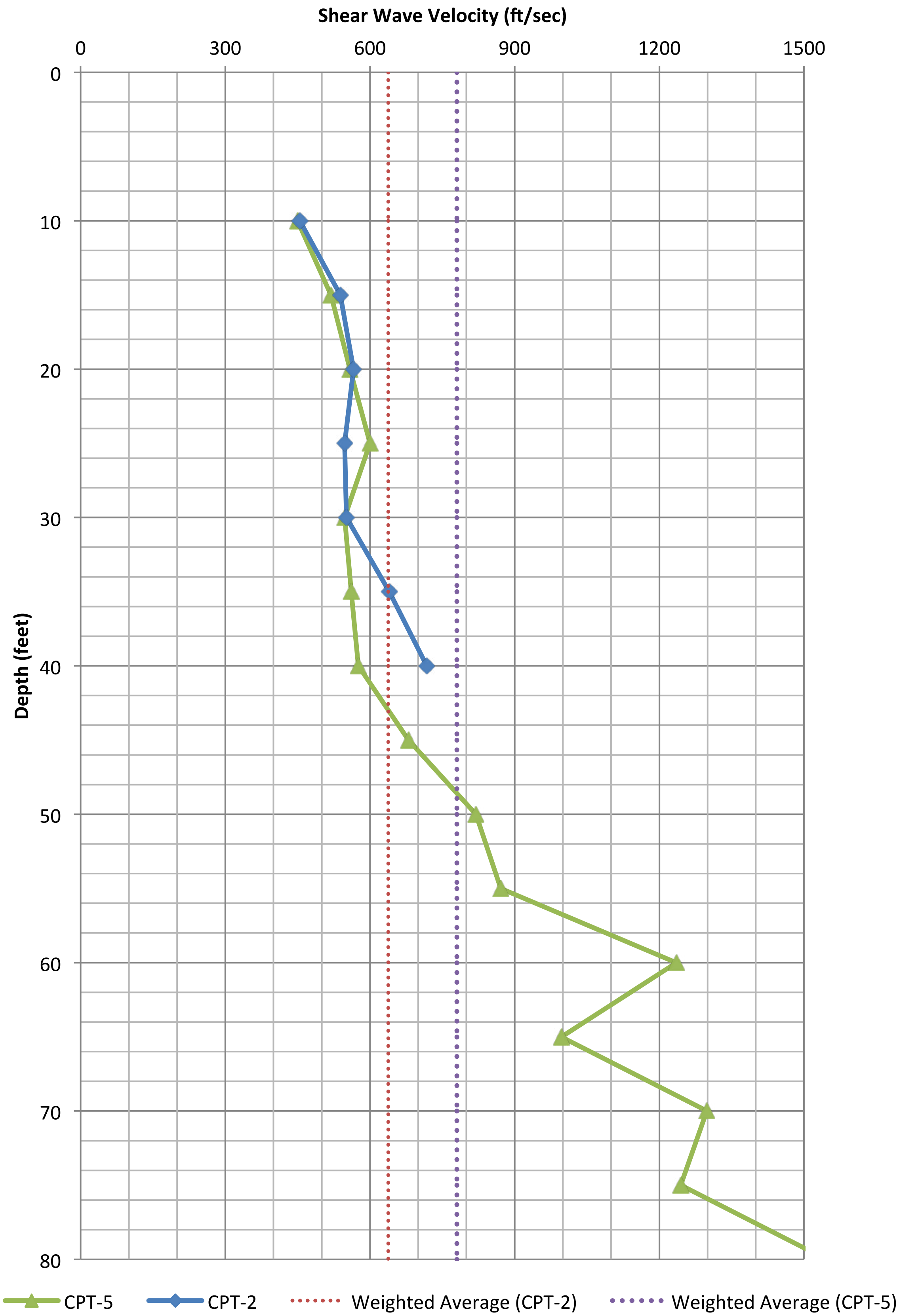
Sacramento, California		
S9840-05-01	November 2013	Figure C2



Shear Wave Velocity vs. Depth

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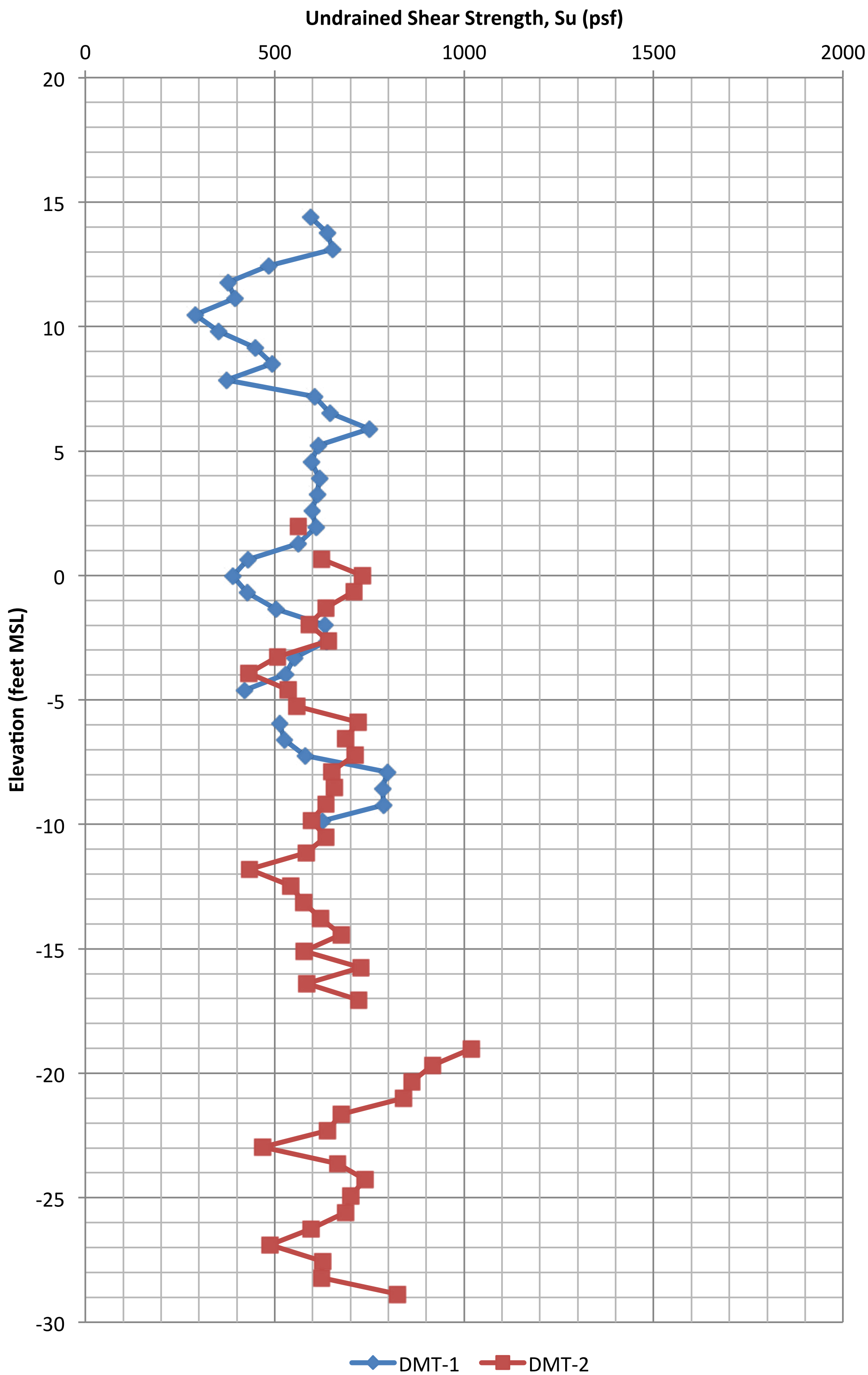
Sacramento, California		
S9840-05-01	November 2013	Figure C3

Undrained Shear Strength (Su) Profile within Silt/Clay Correlated from DMT



Undrained Shear Strengh (Su) Profile within Silt

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Sacramento, California		
S9840-05-01	November 2013	Figure C4

APPENDIX D

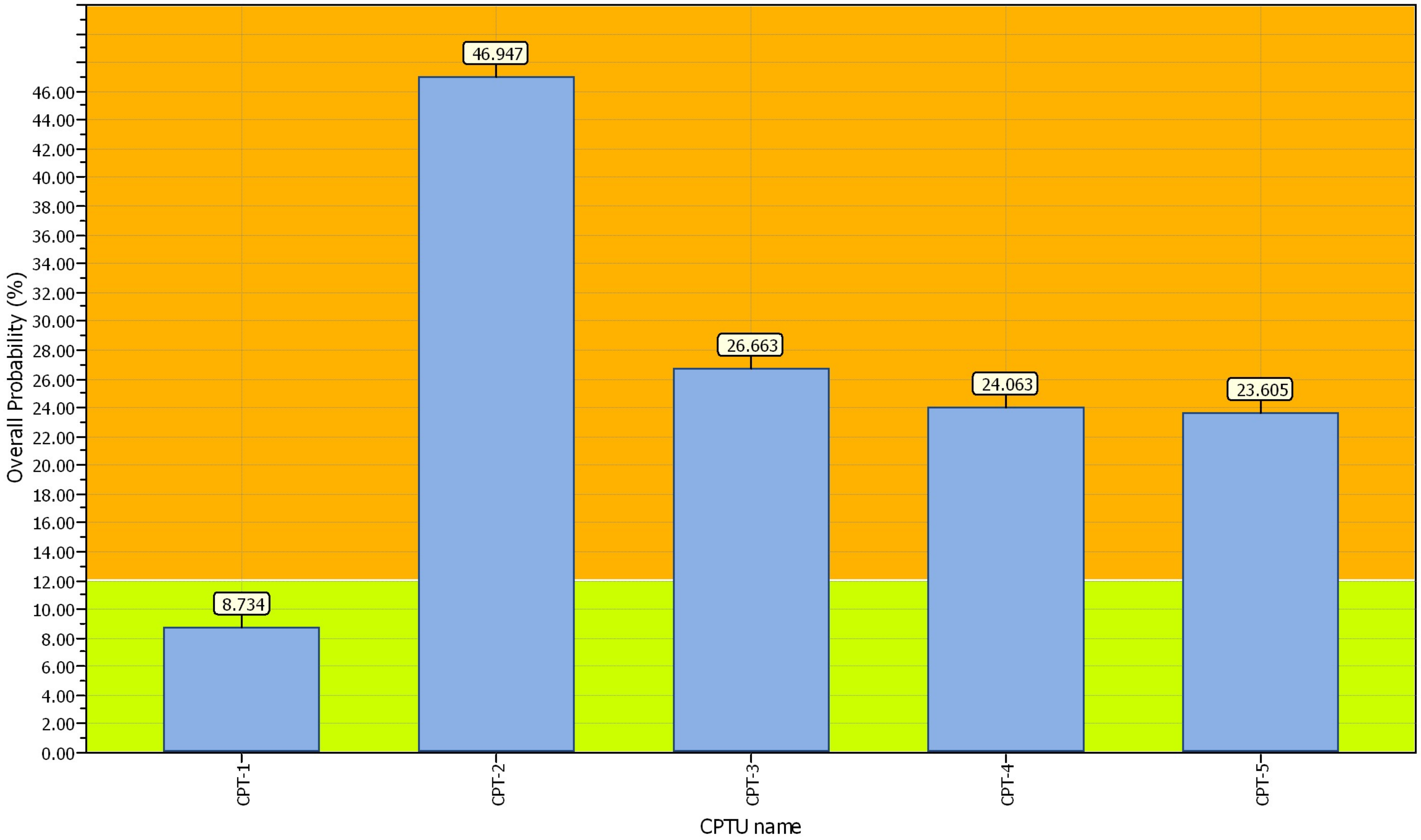
APPENDIX D

LIQUEFACTION ANALYSIS

Project title : Sacramento ESC

Location : Dowtown Sacramento

Overall Probability for Liquefaction report



Probability color scheme

- Very High Probability
- High Probability
- Low Probability

Basic statistics

Total CPT number: 5

20.00% low probability

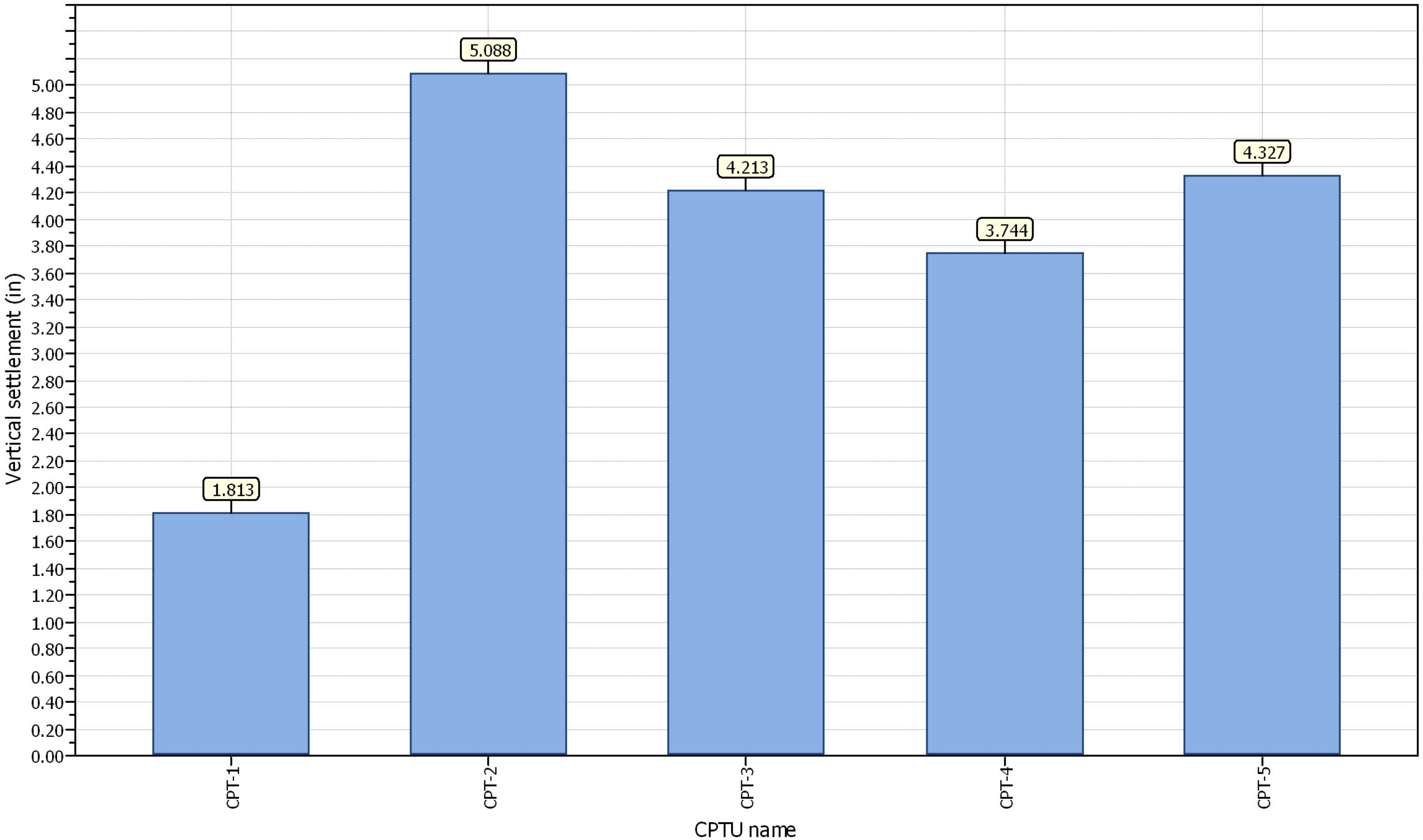
80.00% high probability

0.00% very high probability

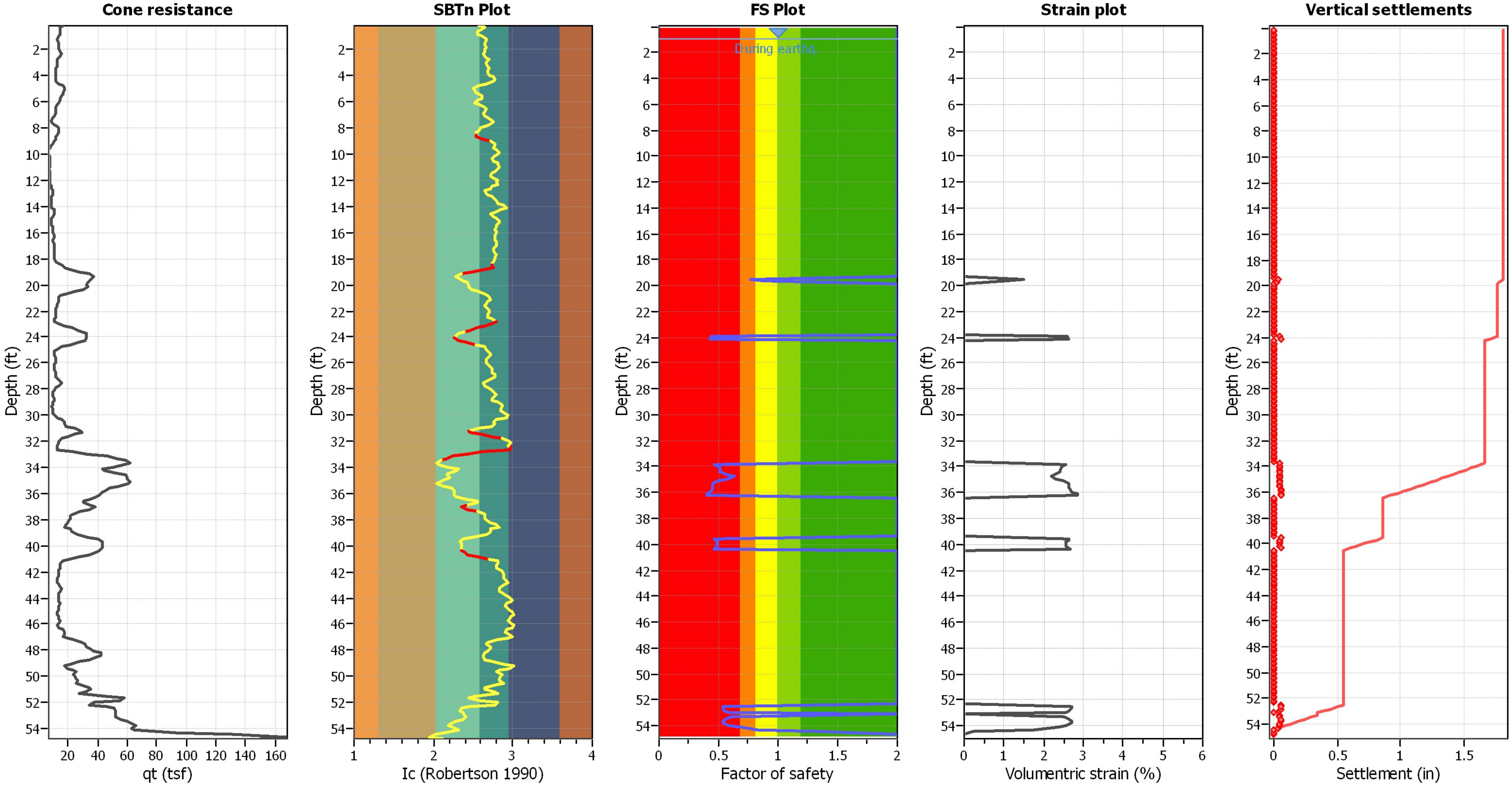
Project title : Sacramento ESC

Location : Dowtown Sacramento

Overall vertical settlements report



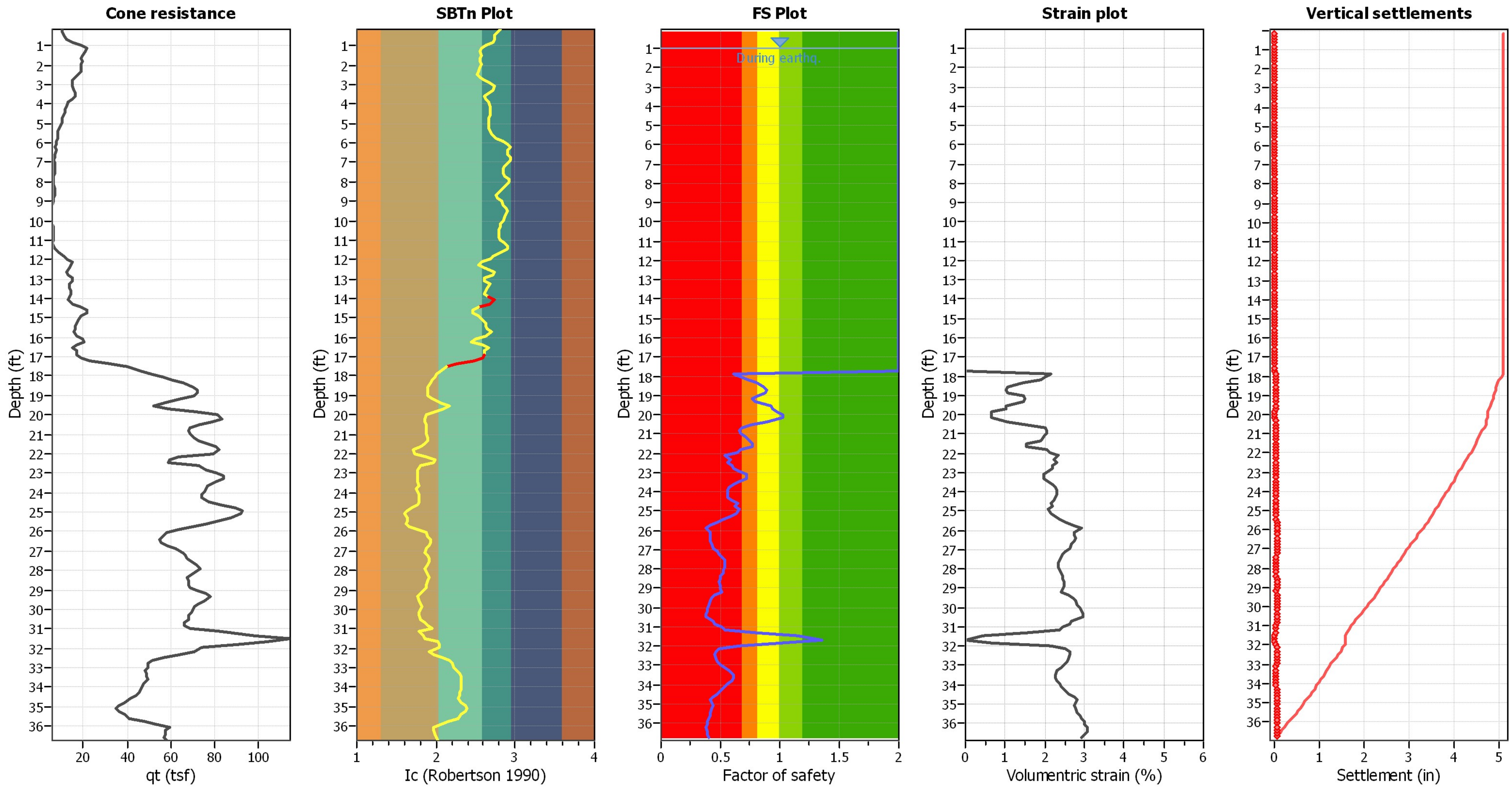
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



Abbreviations

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I_c:	Soil Behaviour Type Index
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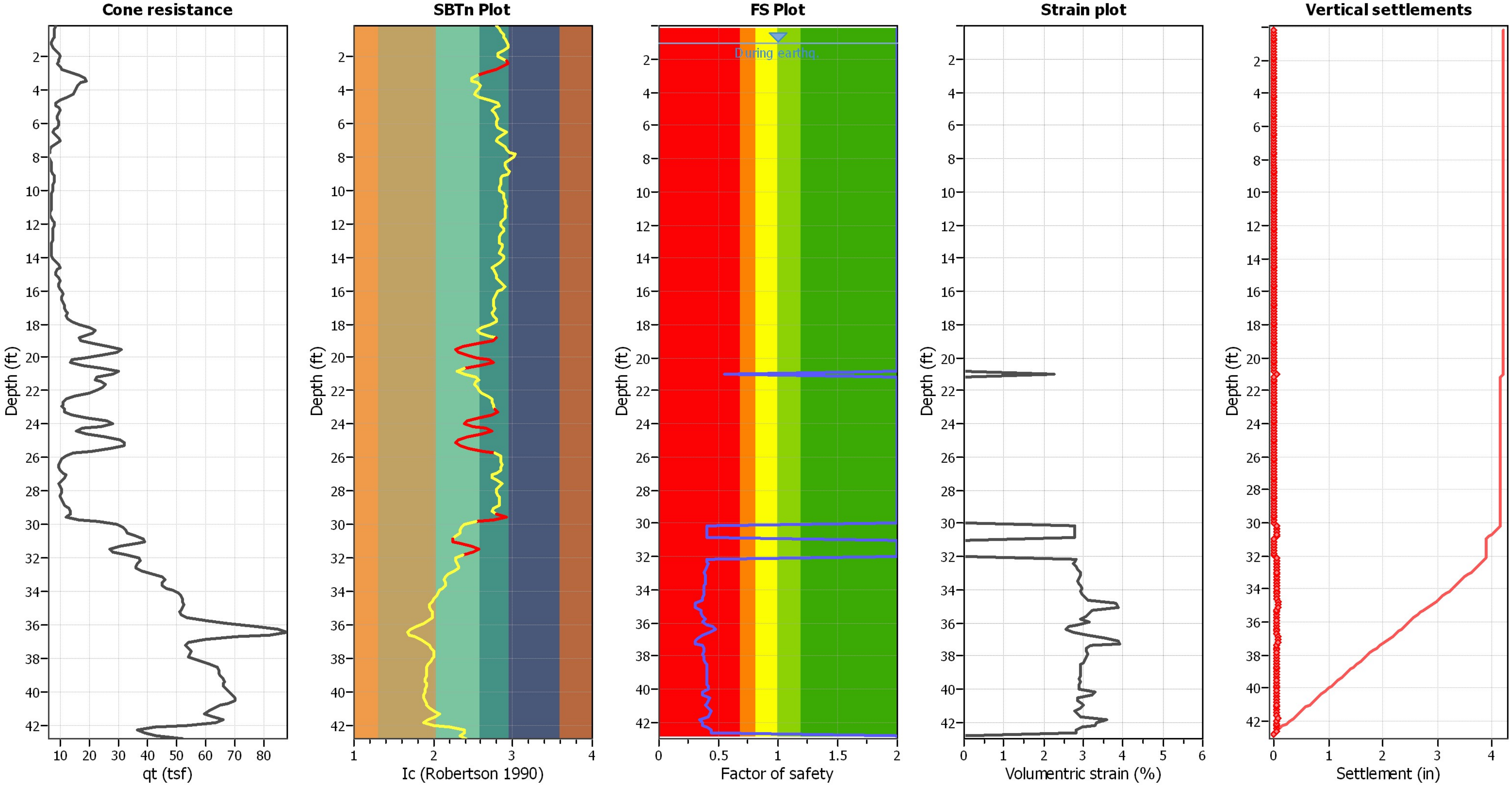
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



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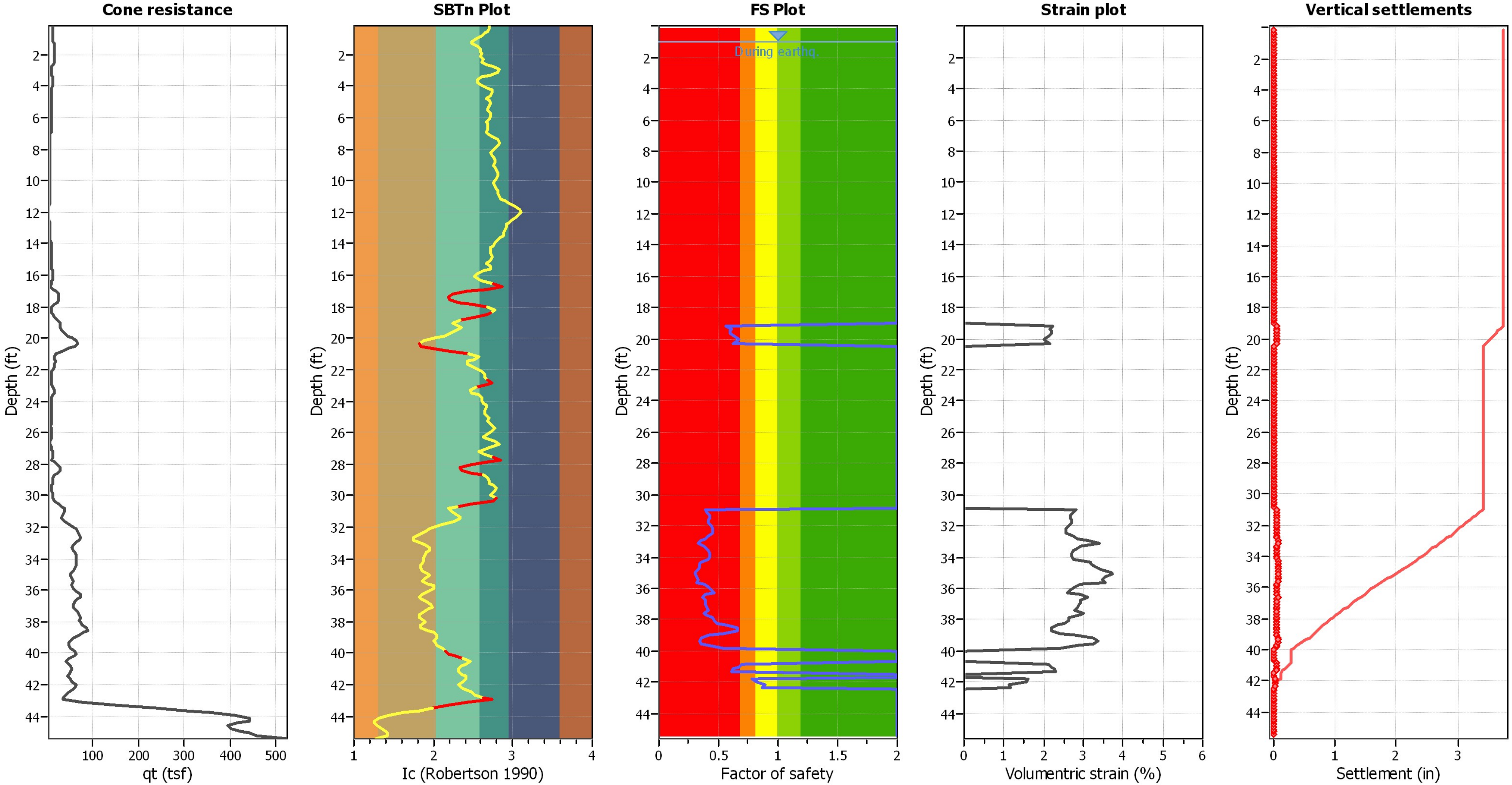

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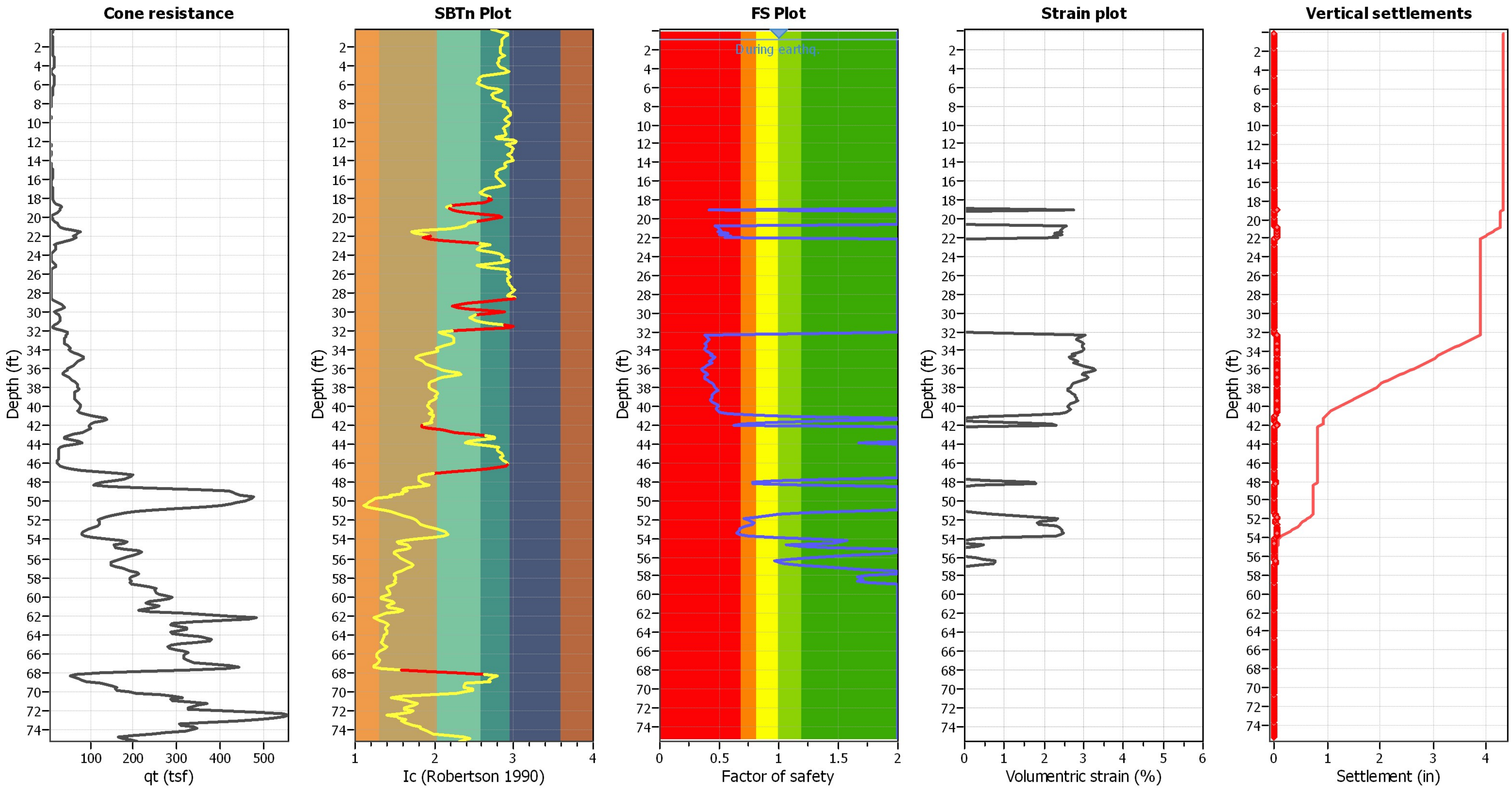
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



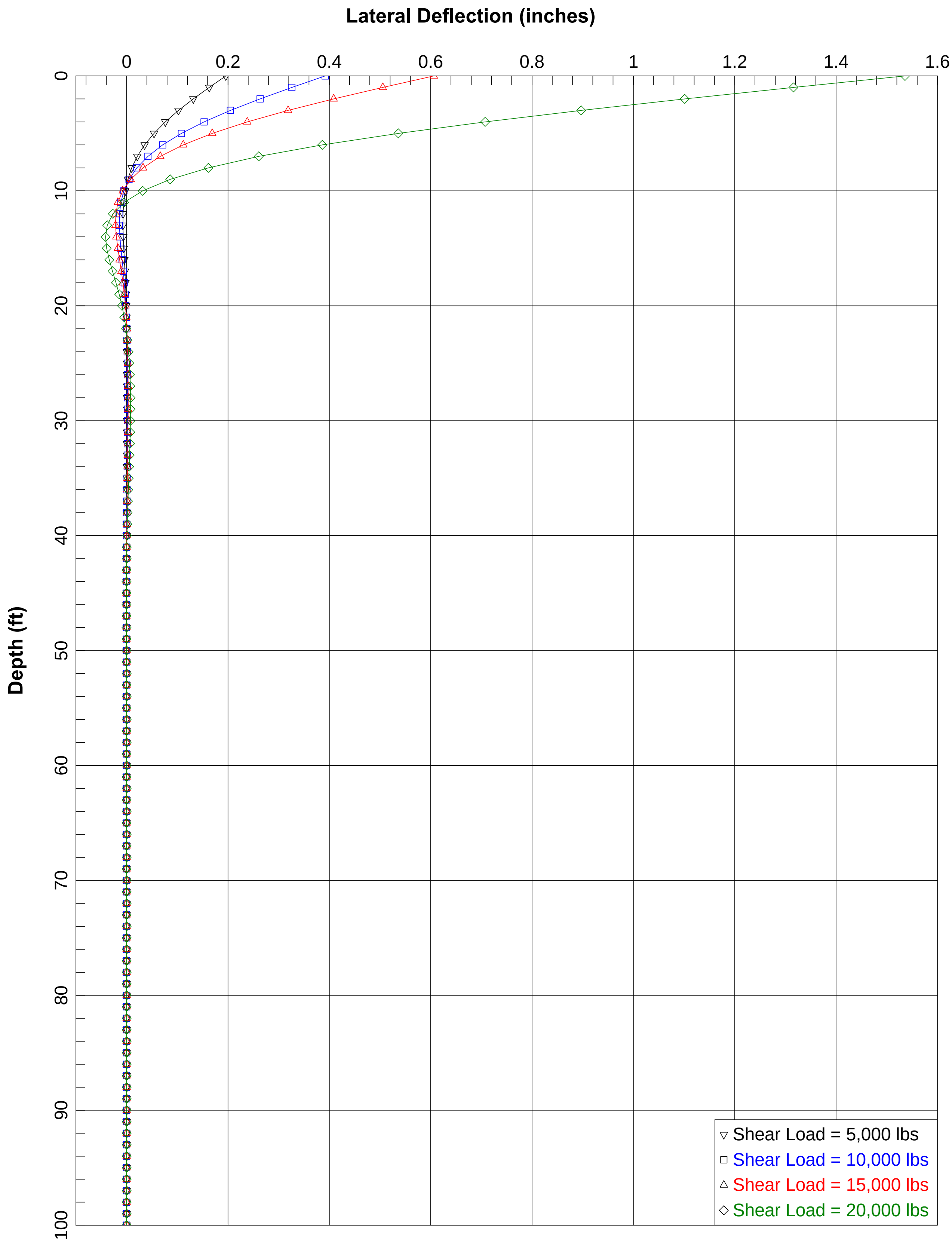
Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

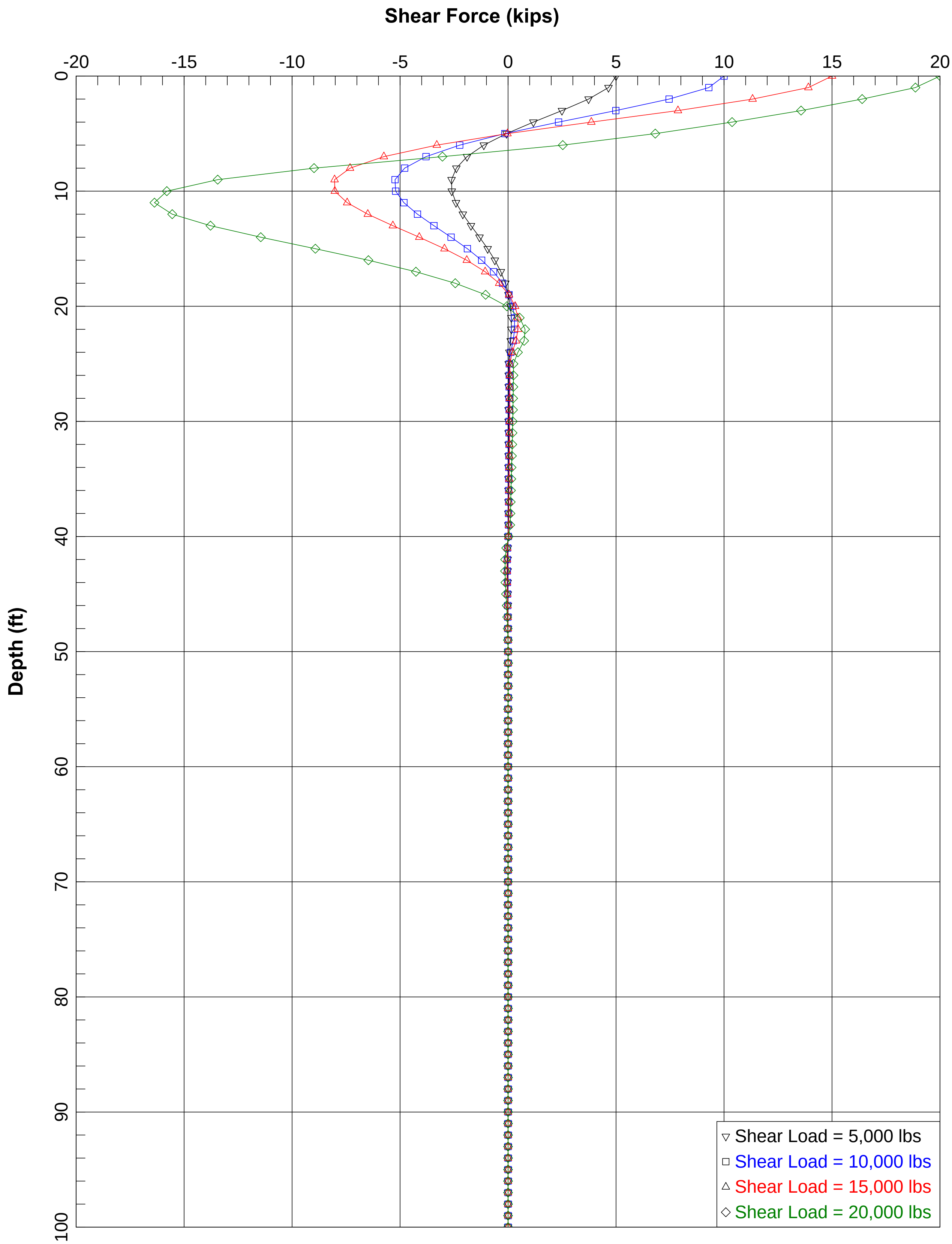
APPENDIX E

APPENDIX E

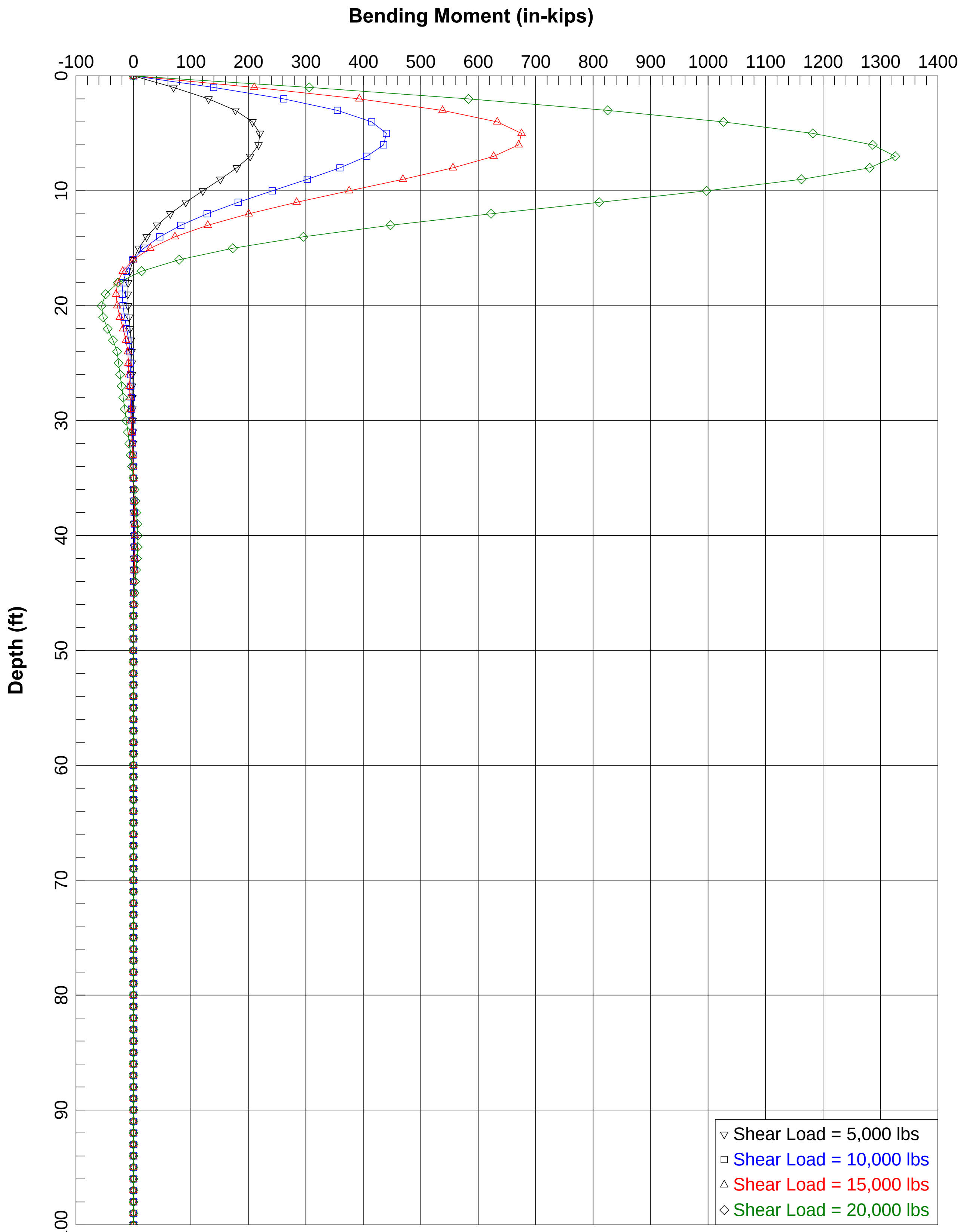
LPILE ANALYSIS



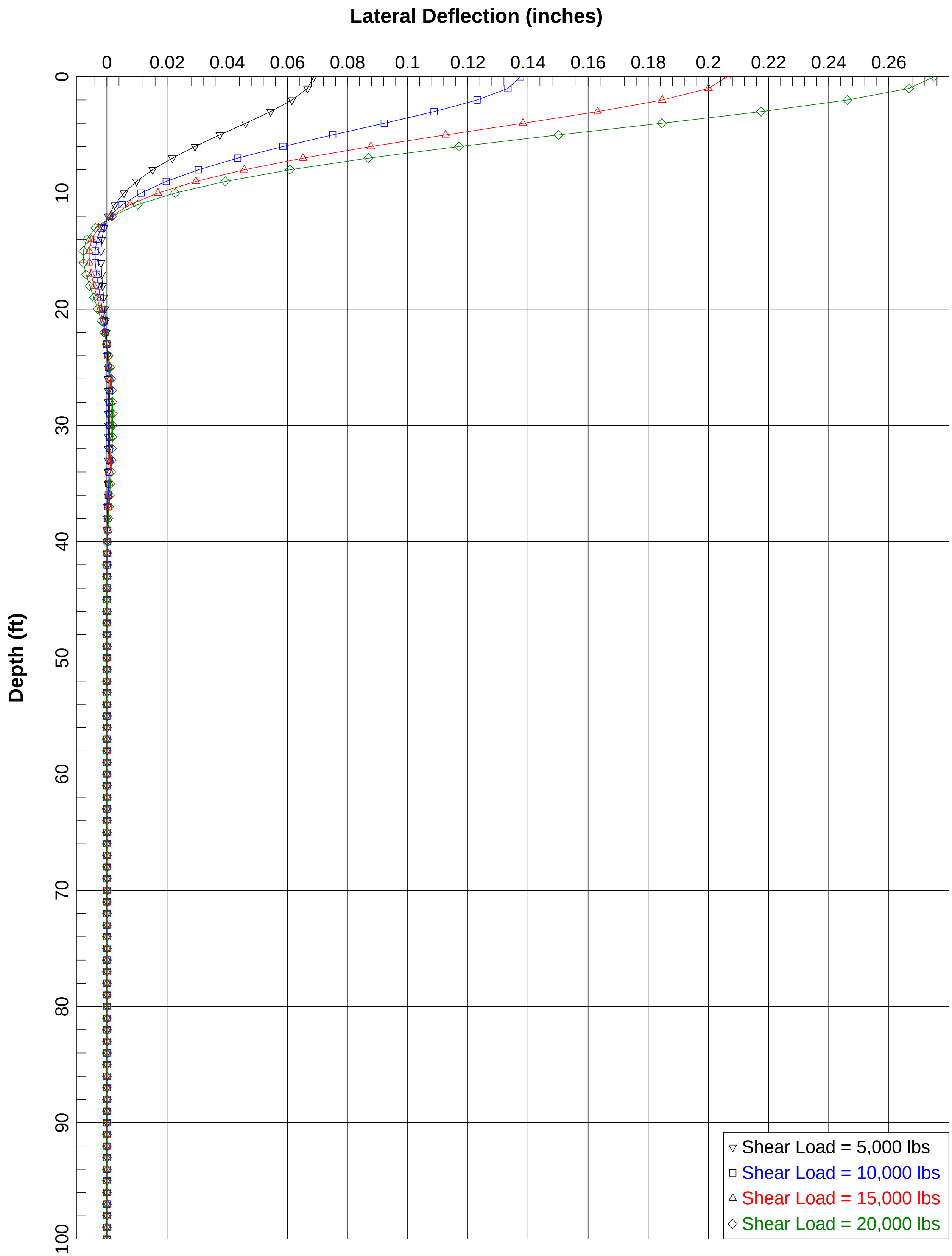
Sacramento ESC - 12.75 inch Pipe Pile - Free Head Condition



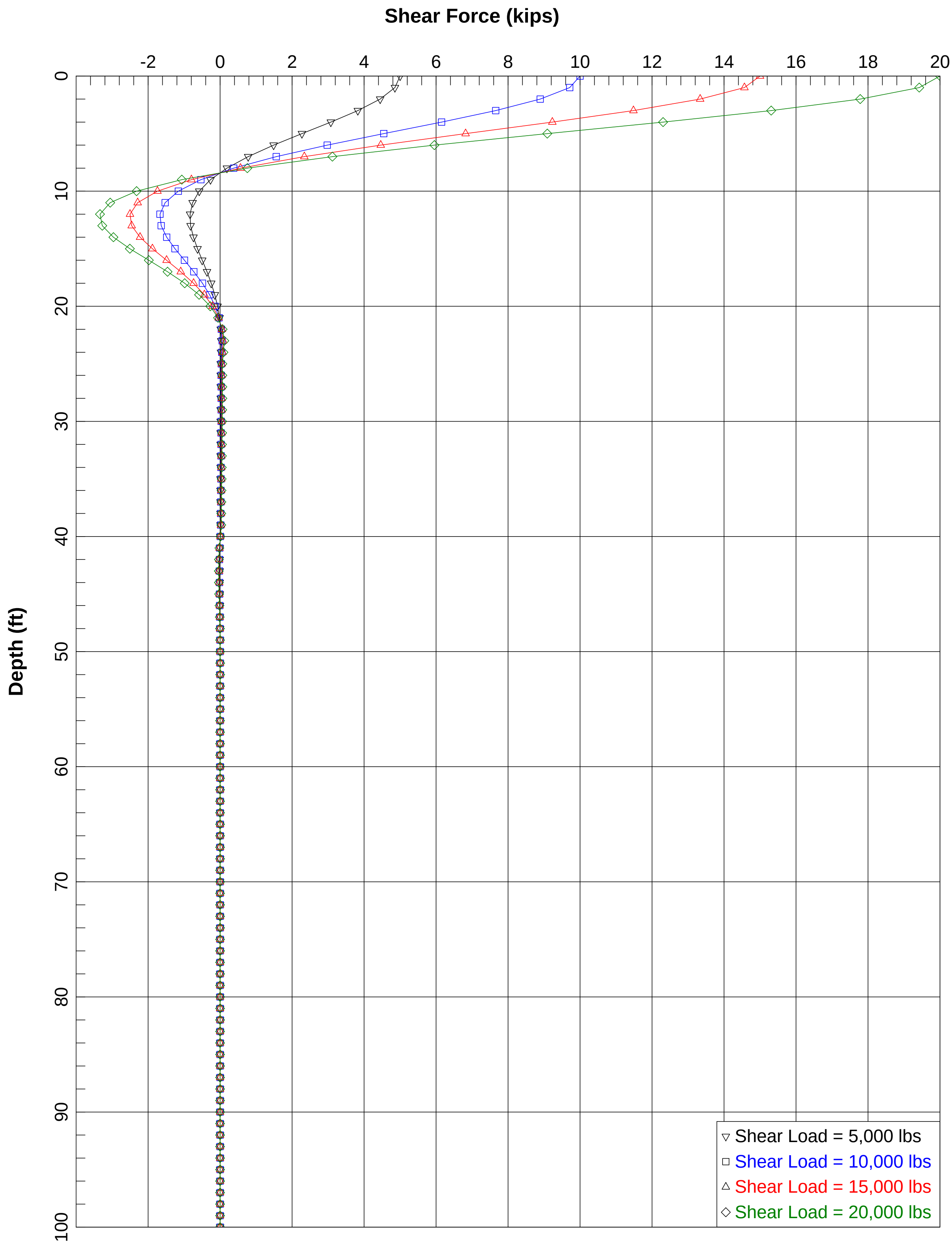
Sacramento ESC - 12.75 inch Pipe Pile - Free Head Condition



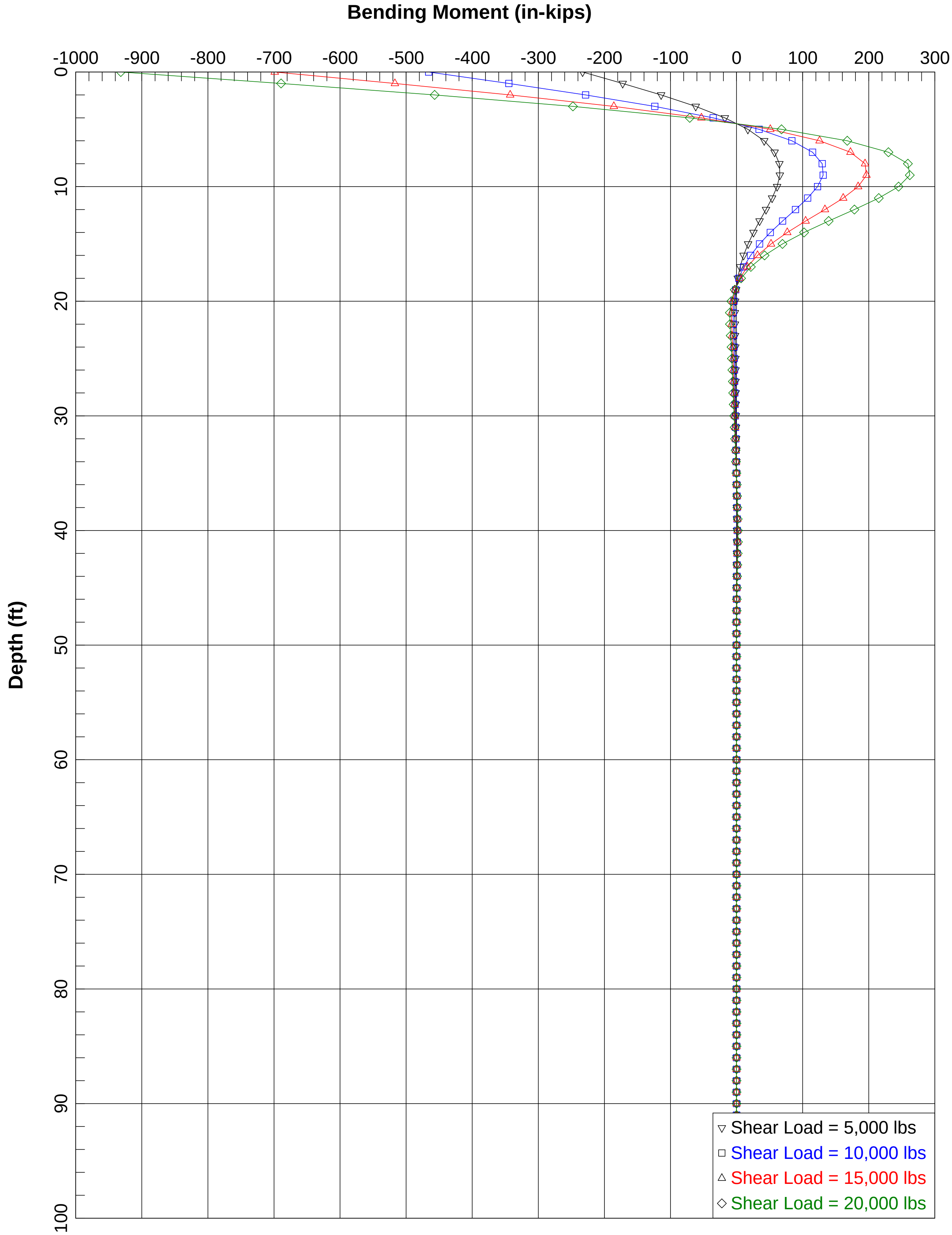
Sacramento ESC - 12.75 inch Pipe Pile - Free Head Condition



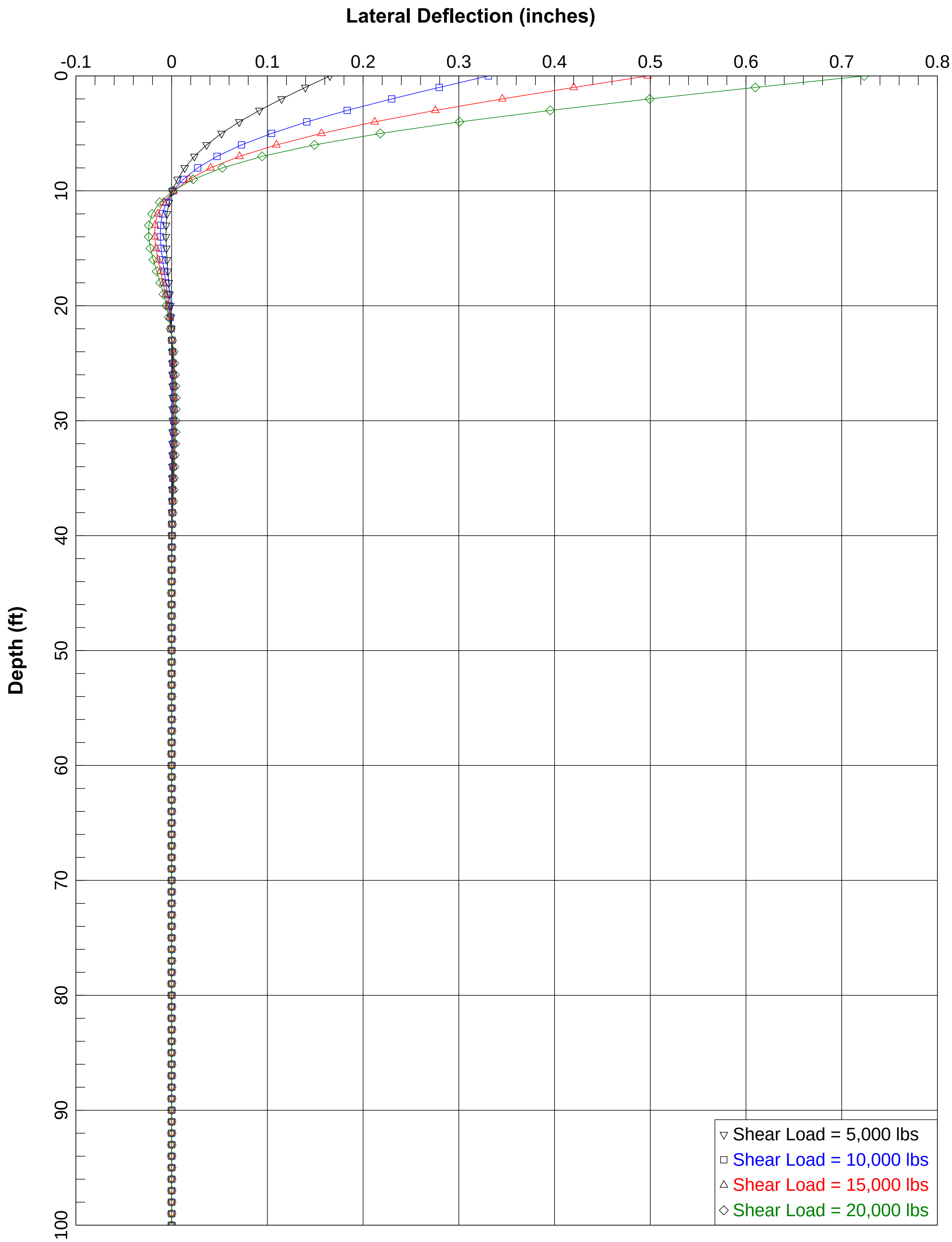
Sacramento ESC - 12.75 inch Pipe Pile - Fixed Head Condition



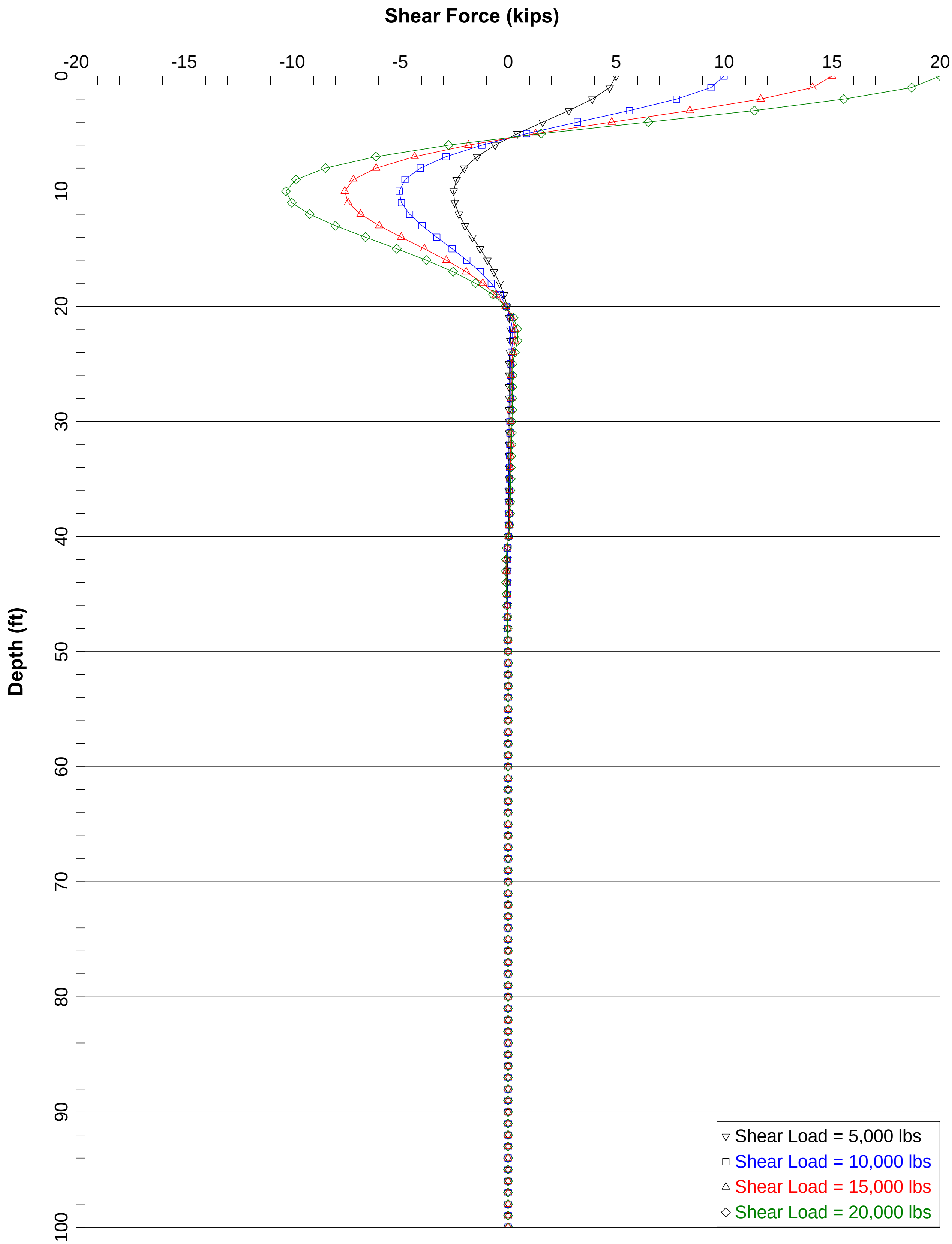
Sacramento ESC - 12.75 inch Pipe Pile - Fixed Head Condition



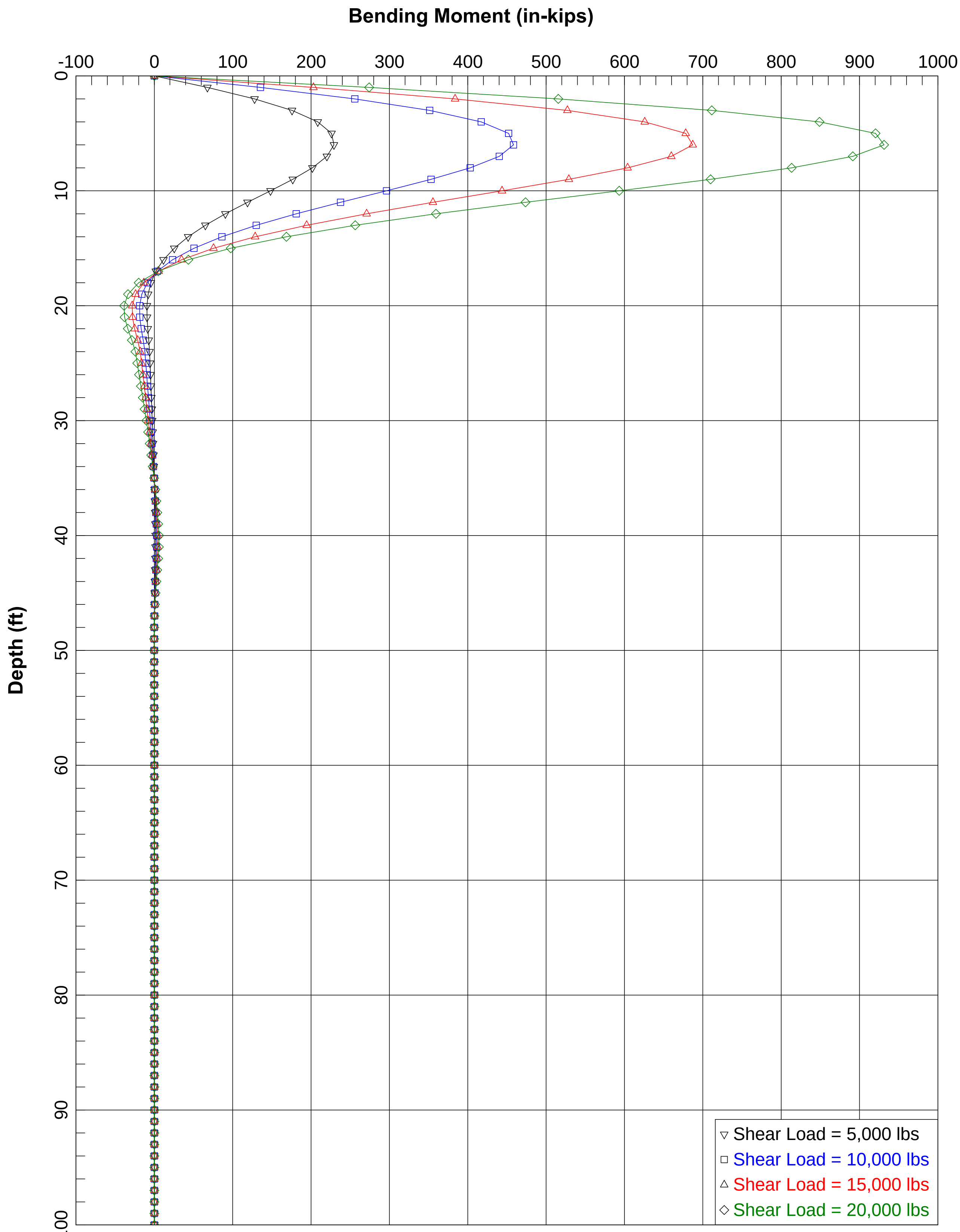
Sacramento ESC - 12.75 inch Pipe Pile - Fixed Head Condition



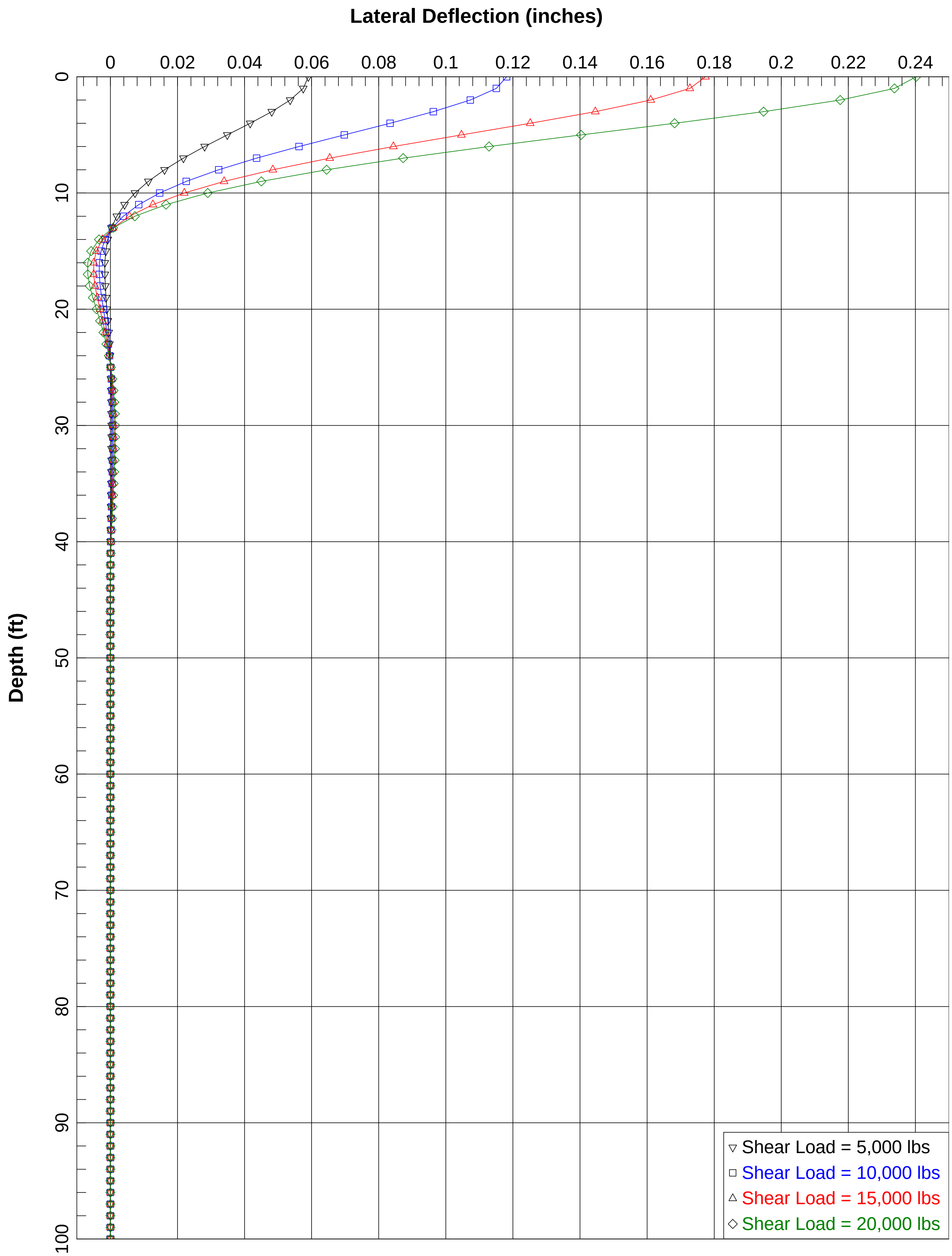
Sacramento ESC - 16 inch APG/APGD Pile - Free Head Condition



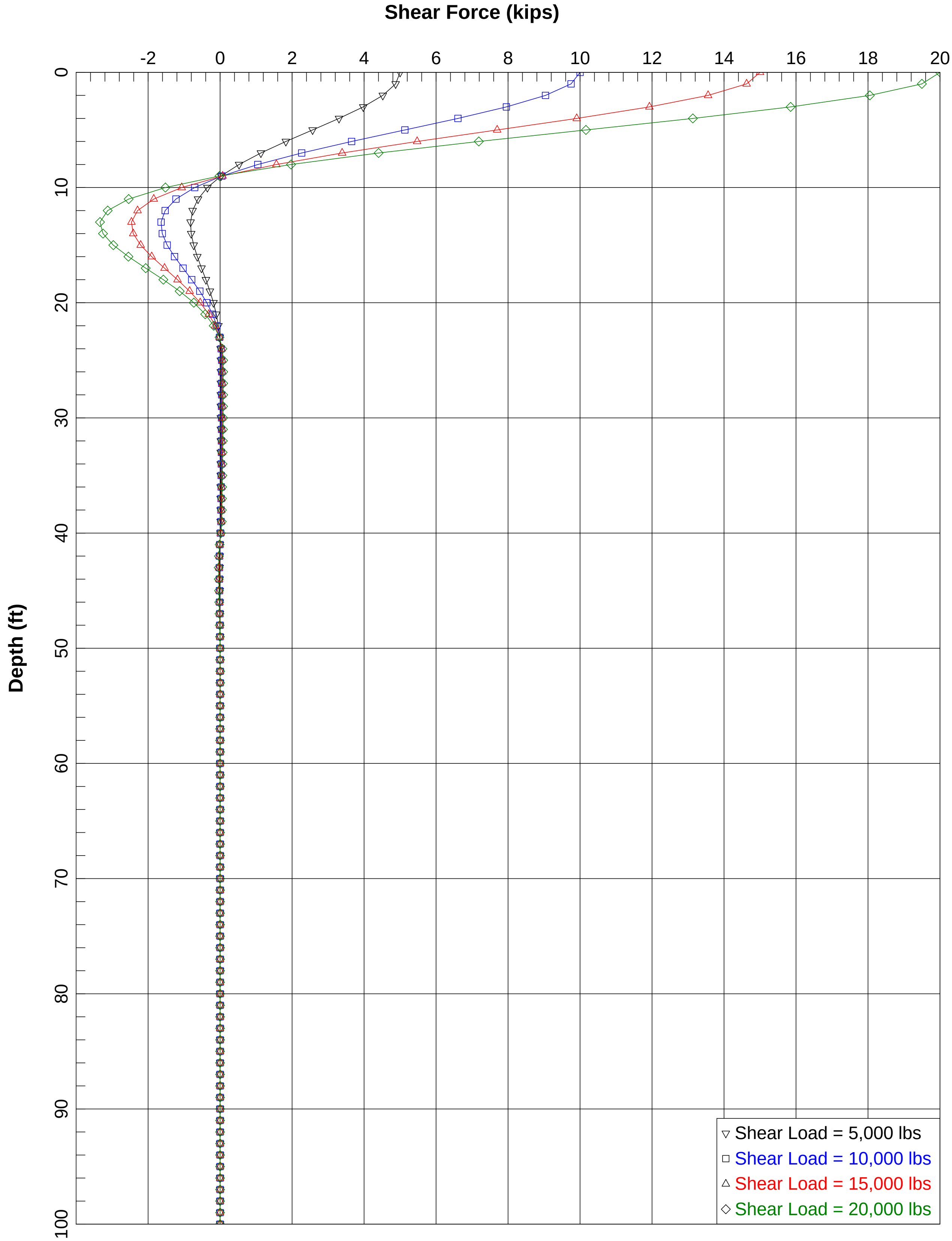
Sacramento ESC - 16 inch APG/APGD Pile - Free Head Condition



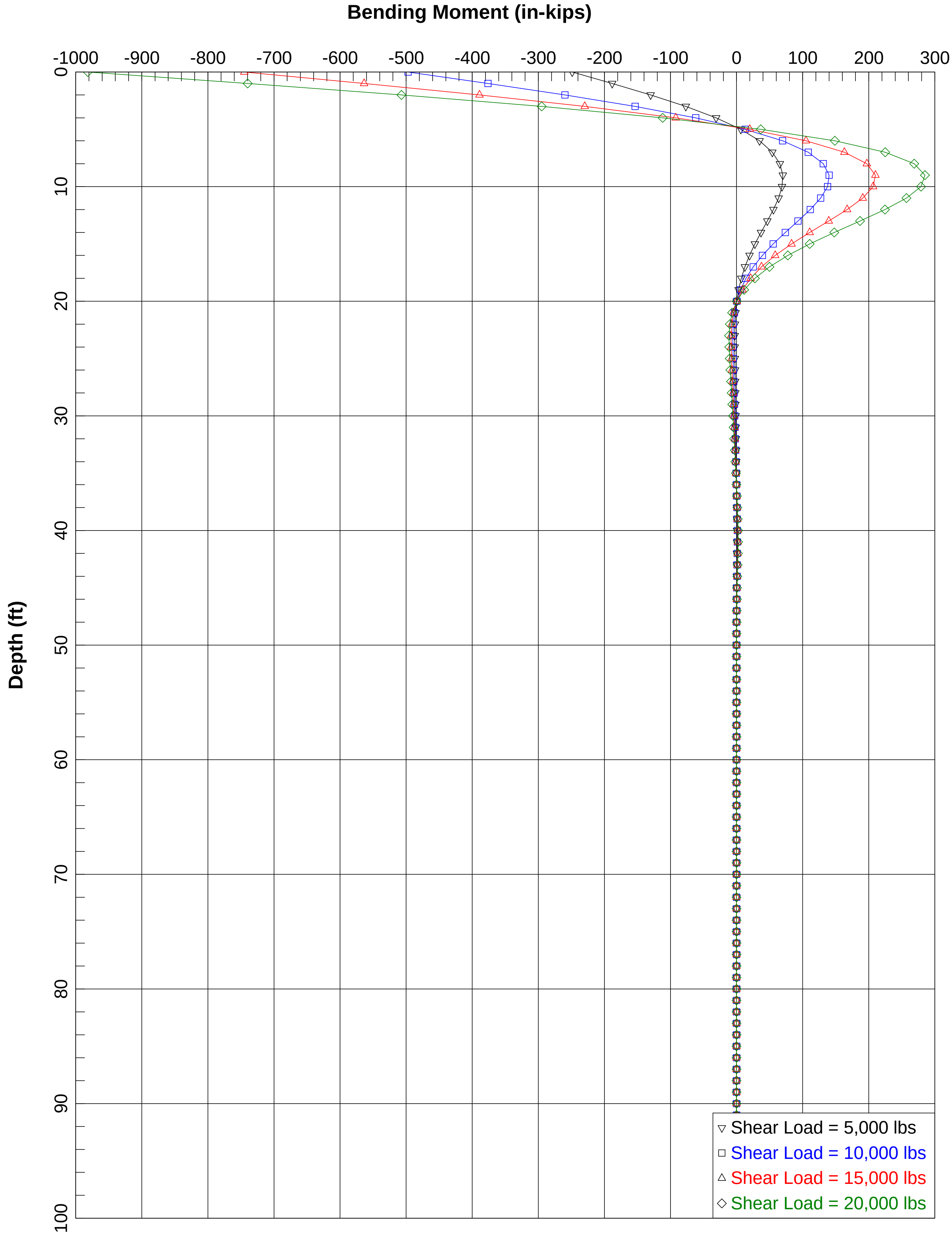
Sacramento ESC - 16 inch APG/APGD Pile - Free Head Condition



Sacramento ESC - 16 inch APG/APGD Pile - Fixed Head Condition



Sacramento ESC - 16 inch APG/APGD Pile - Fixed Head Condition



Sacramento ESC - 16 inch APG/APGD Pile - Fixed Head Condition