Appendix C

Geotechnical Investigation

ROMIG ENGINEERS

GEOTECHNICAL INVESTIGATION

MERCEDES-BENZ/AUDI DEALERSHIPS
1700 AND 1730 EMBARCADERO ROAD
PALO ALTO, CALIFORNIA 94303

Prepared for
Holman Automotive Group, LLC
444 East Kings Highway
Maple Shade, New Jersey 08052

May 2018
Project No. 4300-1

ROMIG ENGINEERS

May 8, 2018
4300-1

Holman Automotive Group, LLC
444 East Kings Highway
Maple Shade, New Jersey 08052

RE: GEOTECHNICAL INVESTIGATION
MERCEDES-BENZ/AUDI DEALERSHIPS
1700 AND 1730 EMBARCADERO ROAD
PALO ALTO, CALIFORNIA

Attention: Mr. Steve Presson

Gentlemen:

In accordance with your request, we have performed a geotechnical investigation for the proposed Mercedes-Benz and Audi Dealerships to be constructed at 1700 and 1730 Embarcadero Road in Palo Alto, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents our geotechnical recommendations for the project.

We refer you to the text of our report for specific recommendations.

Thank you for the opportunity to work with you on this project. If you have any questions or comments about our findings or recommendations for the project, please call.

Very truly yours,

ROMIG ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
MERCEDES-BENZ AND AUDI DEALERSHIPS
1700 AND 1730 EMBARCADERO ROAD
PALO ALTO, CALIFORNIA 94303

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MAY 2018

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GEOTECHNICAL INVESTIGATION
FOR
MERCEDED-BENZ AND AUDI DEALERSHIPS
1700 AND 1730 EMBARCADERO ROAD
PALO ALTO, CALIFORNIA

INTRODUCTION

We are pleased to present this geotechnical investigation report for the proposed Mercedes-Benz and Audi dealerships to be constructed at 1700 and 1730 Embarcadero Road in Palo Alto, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to review our previous work at the site and to provide geotechnical design and construction recommendations for the proposed project.

Project Description

The project consists of constructing an approximately 160,000 square-foot combined Mercedes-Benz and Audi dealership facility at the subject properties in Palo Alto. The approximately 54,000 square foot Mercedes-Benz showroom building will be located along front of the 1700 Embarcadero Road property and will include a ground level showroom, offices, and employee facilities with a large open area at the middle of the building for an automated car stacking system. A two level concrete parking garage will extend across the rear of the existing Audi and new Mercedes-Benz showroom buildings (possibly with a structural separation at the property boundary). The entire roof of the showroom and garage will consist of additional parking. The existing Audi showroom along the front of the 1730 Embarcadero Road property will remain and be structurally separated from the new structure. A detached car wash structure is planned at the rear of 1730 Embarcadero with paved drive aisles and parking along the perimeter of the buildings. Structural loads are expected to be moderate to high as is typical for this type of construction.

The buildings will have a finished first floor elevation of about 11.5 feet. At 1700 Embarcadero Road, site grades vary from about 5.7 feet at the south corner to about 7.2 feet at the north corner. At 1730 Embarcadero Road, site grades vary from about 6.3 feet at the southeast (rear) end to about 9.2 feet at the northwest corner. Approximately 1 to 4.5 feet of fill will be needed below the floor slab and along the perimeter of the building to adjust site grades.

Scope of Work

Our scope of work for this investigation was presented in detail in our agreement with Holman Automotive Group, LLC dated December 6, 2017. In order to complete our investigation, we performed the following work.

- Reviewed readily available geologic and geotechnical literature pertinent to the general area of the site.
- Review of our previous subsurface exploration and laboratory testing conducted at 1700 and 1730 Embarcadero Road and information available in our files concerning the sites.
- Subsurface exploration consisting of drilling, sampling, and logging of one exploratory boring and three cone penetration tests (CPT) in the area of the proposed structure at 1730 Embarcadero Road to supplement our previous exploration work.
- Laboratory testing of selected samples to aid in soil classification and to help evaluate their engineering properties.
- Engineering analysis and evaluation of the subsurface data and laboratory testing to develop geotechnical design criteria for the project.
- Preparation of this report presenting our findings and geotechnical recommendations for the proposed project.

Limitations

This report has been prepared for the exclusive use of Holman Automotive Group, LLC for specific application to developing geotechnical design criteria for the proposed Mercedes-Benz and Audi dealerships to be constructed at 1700 and 1730 Embarcadero Road in Palo Alto, California. We make no warranty, expressed or implied, except that our services are performed in accordance with the geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and recommendations only. In the event there are any changes in the nature, design, or location of the project, or if any future improvements are planned, the conclusions and recommendations presented in this report should not be considered valid unless 1) the project changes are reviewed by us, and 2) the conclusions and recommendations presented in this report are modified or verified in writing.

The analysis, conclusions, and recommendations presented in this report are based on site conditions as they existed at the time of our investigation; the currently planned improvements; review of previous reports relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes occur, we should be advised so that we can review our report in light of those changes.

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on February 2 and 14, 2018. Our subsurface exploration consisted of advancing one exploratory boring to a depth of 50 feet and three CPTs to a depth of 80 feet. The exploratory boring was advanced using a truck-mounted drill equipped with 7.25-inch diameter hollow-stem augers, and the CPTs were advanced using an electronic cone penetration test system (CPT), which was mounted on a truck having a down pressure capacity of 20 tons. The approximate locations of the boring and CPT probes are shown on the Site Plan, Figure 2. The CPT data and boring logs are included on Appendix A and the results of our laboratory tests are included in Appendix B.

Previous Investigation at 1700 Embarcadero Road

Our previous site reconnaissance and subsurface exploration work at 1700 Embarcadero Road were initially performed on June 25, 2009, using track-mounted, electronic cone penetration test (CPT) equipment. Three CPT probes were advanced to a depth of approximately 80 feet below ground surface. Supplemental subsurface exploration work was performed on November 12, 2013 using a Mobile B-53 truck-mounted drill equipped with 8-inch diameter hollow-stem augers in order to obtain additional samples to test soil properties for use as potential off haul material. Four exploratory borings (Borings EB-5 through EB-8) were advanced to depths ranging between 13 and 18 feet. A geotechnical investigation was also conducted for a previously proposed hotel complex by Billy Lin and Associates in 2005 which included four exploratory borings, each advanced to a depth of 46.5 feet. The approximate locations of the borings and CPTs from these previous investigations are also shown on the Site Plan, Figure 2 and the boring and CPT logs are attached in Appendix A.

Previous Investigation at 1730 Embarcadero Road

Our previous site reconnaissance and subsurface exploration work at 1730 Embarcadero Road were performed on September 2, 2014. The subsurface exploration consisted of advancing one exploratory boring to a depth of 11 feet and three CPTs to depths of about 44.9 feet below ground surface. The exploratory boring was advanced using a truck-mounted drill equipped with 8-inch diameter hollow-stem augers, and the CPTs were advanced using an electronic cone penetration test system (CPT), which was mounted on a truck having a down pressure capacity of 20 tons. The approximate locations of the borings and CPTs from this previous investigation is also shown on the Site Plan, Figure 2 and the boring and CPT logs are attached in Appendix A.

Surface Conditions at 1700 Embarcadero Road

The site is located in a commercial area at the east corner of the intersection of Embarcadero Road and East Bayshore Road. The site was occupied by a single-story, wood-frame building which had a wood siding exterior. An asphalt parking lot and access driveways extended around the building. Concrete walkways extended along the perimeter of the building with concrete stairs along the front of the building. The finished floor elevation of the building was several feet higher than the pavement grades. A steel framed, utility tower was located at the west corner of the site. The relatively flat site was landscaped with small to medium shrubs, and medium to large trees.

The asphalt concrete was in fair to poor condition with extensive stress cracks and alligator cracks. Several large asphalt concrete patches were evident throughout the parking lot. The walkways were in adequate condition with only minor offset between slabs. The site appeared to be in similar condition as observed during our previous investigations.

Surface Conditions at 1730 Embarcadero Road

The site is located in a commercial area at the southeast side of Embarcadero Road, to the northeast (and adjacent) to 1700 Embarcadero Road. At the time of our investigation, the site was occupied by the new single story, steel-framed Audi showroom building located at the front of the site and the two-story service parts and repair building to the rear. Asphalt concrete driveway and parking areas extended along the northeast of the buildings and to the rear of the site. The relatively flat site was landscaped with native grasses, small to large shrubs, and small to large trees.

The depth and width of the older dealership building foundation is unknown. Where visible, some hairline cracks were observed at the perimeter stem wall. In addition, cracks up to 1/16-inch wide were observed in the exterior finish surfaces of the building. The asphalt concrete pavement appeared to be in fair condition, with many alligator cracks and long cracks up to about 1 inch wide. The roof downspouts generally discharged adjacent to the foundations and into a closed pipe system at some locations.

According to our construction observation and testing records, the new Audi showroom building was constructed in 2015 to 2016. The showroom building is supported on a structural mat foundation and we understand that the building appears to be performing well.

Subsurface Conditions

At the locations of the recent and previous borings and CPT's, beneath the asphalt pavement, we generally encountered approximately 4 to 8 feet of artificial fill which consisted of dense/hard, clayey sand/sandy lean clay and stiff to hard sandy lean clay of low to moderate plasticity.

Below the fill, we encountered approximately 4.5 to 9.5 feet of younger Bay Mud which consisted of soft fat clay of very high plasticity to soft/firm clay to silt clay and sensitive fine-grained soils (based on CPT correlations). Relatively high water contents were measured on the Bay Mud in several of the borings. The Bay Mud is expected to be highly compressible under new foundation or fill loads.

Beneath the fill and Bay Mud, we encountered stratified layers of stiff to very stiff sandy lean clay/sandy fat clay of moderate to high plasticity, firm to very stiff silty clay with interbeds of firm to stiff clayey silt, and medium dense to dense sand and silty sand (based on CPT correlations) that extended to depths of approximately 28 to 36 feet. We then encountered stiff to very stiff silty clay and clayey silt, interbedded with generally dense to very dense sand and silty sand extending to approximately 80 feet, the maximum depth of our exploration.

Soil Properties Testing

The laboratory testing was conducted on 10 selected soil samples during our previous investigation at 1700 Embarcadero Road (2103) which included sieve analysis including percent passing the No. 200 sieve and Atterberg Limit tests to establish the Liquid Limit and Plasticity Index of the clay material for off haul of a previously planned basement excavation soils.

A summary of the test results are presented in Table 1, on the following page. The results of the sieve analysis of selected samples are presented in the particle size distribution report, Figures B-4 and B-5.

Table 1. Soil Properties Testing
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Boring (2013)	Depth (feet)	Soil Type	Liquid Limit	Plasticity Index	% Passing #200 Sieve
EB-5	1.5-2	Artificial Fill	33	16	51%
EB-5	6-6.5	Bay Mud	88	40	91%
EB-5	13.5-14	Clayey Sand	22	8	28%
EB-6	2.5-3	Artificial Fill	38	21	56%
EB-6	8.5-9	Bay Mud	89	47	95%
EB-7	1.5-2	Artificial Fill	27	12	61%
EB-7	6-6.5	Artificial Fill	43	25	71%
EB-7	11-11.5	Sandy Fat Clay	53	32	66%
EB-8	1.5-2	Artificial Fill	47	29	57%
EB-8	6.5-7	Bay Mud	81	40	92%

Ground Water

Ground water was measured at a depth approximately 8 feet in Boring EB-1 shortly after drilling and sampling was completed. Based on the dynamic pore pressure response, ground water was estimated to be present at a depth of about 9 feet below grade in CPT-1, at about 8.1 feet below grade in CPT-2, and at about 5.5 feet below grade in CPT-3. The borings and CPTs were backfilled immediately after drilling and/or sampling was completed, and therefore a stabilized ground water level may not have been obtained.

Ground water was estimated to be present at a depth of about 7 feet below grade at all the CPT locations at 1700 Embarcadero Road based on the dynamic pore pressure response observed during testing in 2009. Because of the low permeability of the Bay Mud, pore pressure dissipation tests performed at two CPT locations were inconclusive, therefore these ground water levels do not represent stabilized ground water levels. Ground water was measured at a depth of between 9.5 to 14 feet in our supplemental exploratory borings in 2013. Ground water was encountered in the previous borings (Billy Lin and Associates, 2005) between depths of approximately 5 to 7 feet below the ground surface. At the time of our exploration at 1730 Embarcadero Road in 2014, ground water was estimated to be present at a depth of about 8 feet below grade at CPT-1 based on the pore pressure response.

Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, tidal fluctuations, local surface and subsurface drainage patterns, landscaping, and other factors.

Two ground water monitoring wells were previously installed at 1700 Embarcadero Road to facilitate measuring pre-construction ground water levels on the property in November, 2013. The exploratory monitoring wells were permitted through the Santa Clara Valley Water District (SCVWD). The two wells were installed in order to sample ground water and for initial depth to water measurements for the previously proposed hotel complex basement. "Stabilized" ground water levels in these wells after well development showed depth to ground water in MW-1 and MW-2 at a depth of about 7 feet. The location of the monitoring wells are show on Figure 2.

Information contained in Seismic Hazard Zone Report 111 for the Palo Alto 7.5-Minute Quadrangle (California Geological Survey, 2006) indicates the depth to the historic high ground water level in the area of the site is approximately 5 feet or less. Based on our experience at other sites in the area, we expect that ground water will be present in the fill above the Bay Mud and that the stabilized ground water level could seasonally be as high as approximately 3 feet below grade.

Corrosion Potential Testing

Corrosion potential tests were performed by Cooper Testing Laboratory on two samples of surface fill obtained from the CPT locations. The soil samples were tested for resistivity, pH, sulfate content, chloride content, and redox potential. The results of these tests are presented in Appendix B.

Resistivity of the lab-saturated soil samples measured in accordance with ASTM Test G57 ranged from 1,502 to 4,158 ohm-cm. These test results suggest the surface soils may be severely corrosive.

The pH of the soil samples ranged from 7.9 to 8.0. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride content was <2 mg/kg (ppm) for each sample. The oxidation-reduction potential (Redox) ranged from -34 to 123 mv.

The water-soluble sulfate content of the samples that were tested in accordance with California Test Method 417-modified were measured to be <5 parts per million (<0.0005% by dry weight). Table 19A-A-4 of the California Building Code classifies a water-soluble sulfate content of 0.0 to 0.10% by dry weight as producing negligible sulfate exposure. The Bay Mud soils encountered at depth however would be expected to have moderate to high sulfate content.

Due to the Bay Mud and salt water environment, for specific long-term corrosion control design recommendations, it may be beneficial to retain a corrosion engineer to evaluate the corrosion potential and protection for buried metal and concrete elements.

GEOLOGIC SETTING

We have briefly reviewed our local experience and geologic literature pertinent to the area of the site. The information that we reviewed for this study indicates the site is underlain by Historic Artificial fill, af (Brabb, Graymer and Jones, 2000). These deposits are generally found to consist of loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations. Thickness is variable and may exceed 30 meters in some places. Some of the fill is compacted and quite firm, but fill placed before 1965 is nearly everywhere not compacted and consists of dumped materials. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

Based on information presented in a report titled "Geologic and Engineering Aspects of San Francisco Bay Fill" (CDMG, 1969), the surface fill is mapped as being underlain by approximately 10 feet of soft, compressible, younger Bay Mud (CDMG, 1969). The young Bay Mud covers most of the bottom of the San Francisco Bay and some of the Bay margins and generally consists of soft, silty clay, silt, minor fine sand, and shell fragments. The estimated thickness of the young Bay Mud indicated in the reference noted above is shown on the Contour Map of Bay Mud Thickness, Figure 4.

The Seismic Hazards Zones Map of the Palo Alto Quadrangle prepared by the California Geologic Survey (Seismic Hazard Zone Report 111, 2006) indicates the site is located in an area where historical occurrence of liquefaction, or local geological, geotechnical, and ground water conditions indicate a potential for permanent ground displacement from liquefaction such that mitigation would be required. A site specific liquefaction discussion is presented later in this report.

The property and the immediate site vicinity are located in an area that slopes very gently to the east (approximately 10 feet vertically per 1,600 feet laterally, although locally the topography may be steeper). The site is located at an elevation of approximately 6 feet above sea level (see Figure 1).

Faulting and Seismicity

There are no mapped through-going faults within or adjacent to the site and the site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the San Andreas fault, located approximately 7.5 miles southwest of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is remote.

The San Francisco Bay Area is an active seismic region. Earthquakes in the region result from strain energy constantly accumulating because of the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906, and 1989. The faults considered most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras faults. The San Gregorio fault is located approximately 18 miles southwest of the site. The Hayward and Calaveras faults are located approximately 12 and 18 miles northeast of the site, respectively. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 2 on the following page and are shown on the Regional Fault and Seismicity Map, Figure 5.

Table 2. Earthquake Magnitudes and Historical Earthquakes
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Fault	Maximum Magnitude (Mw)	Historical Earthquakes	Estimated Magnitude
San Andreas	7.9	1989 Loma Prieta	6.9
		1906 San Francisco	7.9
		1865 N. of 1989 Loma Prieta Earthquake	6.5
		1838 San Francisco-Peninsula Segment	6.8
		1836 East of Monterey	6.5
Hayward	7.1	1868 Hayward	6.8
		1858 Hayward	6.8
Calaveras	6.8	1984 Morgan Hill	6.2
		1911 Morgan Hill	6.2
		1897 Gilroy	6.3
San Gregorio	7.3	1926 Monterey Bay	6.1

In the future, the subject property will undoubtedly experience severe ground shaking during moderate and large magnitude earthquakes produced along the San Andreas fault or other active Bay Area fault zones. Using information from recent earthquakes, improved mapping of active faults, ground motion prediction modeling, and a new model for estimating earthquake probabilities, a panel of experts convened by the U.S.G.S. have concluded there is a 72 percent chance for at least one earthquake of Magnitude 6.7 or larger in the Bay Area before 2043. The Hayward fault has the highest likelihood of an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 33 percent, while the likelihood on the San Andreas and Calaveras faults is estimated at approximately 22 and 26 percent, respectively (Aagaard et al., 2016).

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the 2016 California Building Code and in ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures." Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class F due to the potential for liquefaction, in accordance with Chapter 20 of ASCE 7-10. In Site Class F, a site specific response analysis can be required to obtain the seismic design parameters, however, for structures with a fundamental period of equal or less than 0.5 second, a site response analysis is not required for areas of liquefiable soils.

Provided that the proposed structure is shown to have a fundamental period of less than 0.5 second, in our opinion, the project may be designed based on the higher values of the seismic design parameters of Site Class D and E, in accordance with Chapter 20 of ASCE 7-10. For site latitude (37.5238), longitude (-122.2686), the design parameters for Site Class D are presented on Table 3 and the design parameters for Site Class E are presented on Table 4 (the higher recommended values are boxed).

Table 3. 2016 CBC Seismic Design Criteria For Site Class D
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Spectral Response Acceleration Parameters			Design Value
Mapped Value for Short Period	-	S_S	1.500
Mapped Value for 1-sec Period	-	S_1	0.601
Site Coefficient	-	F_a	1.0
Site Coefficient	-	F_v	1.5
Adjusted for Site Class	-	S_{MS}	1.500
Adjusted for Site Class	-	S_{M1}	0.901
Value for Design Earthquake	-	S_{DS}	$\boxed{1.00}$
Value for Design Earthquake	-	S_{D1}	0.601

Table 4. 2016 CBC Seismic Design Criteria For Site Class E
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Spectral Response Acceleration Parameters			Design Value
Mapped Value for Short Period	-	S_S	1.500
Mapped Value for 1-sec Period	-	S_1	0.601
Site Coefficient	-	F_a	0.9
Site Coefficient	-	F_v	2.4
Adjusted for Site Class	-	S_{MS}	1.350
Adjusted for Site Class	-	S_{M1}	1.442
Value for Design Earthquake	-	S_{DS}	0.900
Value for Design Earthquake	-	S_{D1}	0.961

Liquefaction Evaluation

Severe ground shaking during an earthquake can cause loose to medium dense granular soils to densify. If the granular soils are below ground water, their densification can cause increases in pore water pressure, which can lead to soil softening, liquefaction, and ground deformation. Soils most prone to liquefaction are saturated, loose to medium dense, silty sands and sandy silts with limited drainage, and in some cases, sands and gravels that are interbedded with or that contain seams or layers of impermeable soil.

To evaluate the potential for earthquake-induced liquefaction of the soils at the site, we performed a liquefaction analysis of the data from the CPT probes following the methods described in the 2008 publication by Idriss and Boulanger titled "Soil Liquefaction During Earthquakes".

The peak ground acceleration (PGA) used for our liquefaction analysis was based on information presented on the Probabilistic Seismic Hazards Mapping Ground Motion Page (CGS, 2018) which indicates that the maximum considered earthquake acceleration (PGA_M) is 0.54g. The depth to ground water used in our liquefaction analysis was 3 feet below grade. The results of our liquefaction evaluation are presented on Figures C-1 through C-9 in Appendix C.

The silt and sand layers encountered at the site below a depth of 5 feet and the maximum of our exploration were considered in our liquefaction analysis. Soils with a soil behavior classified as “clay” and “silty clay to clay” (based on soil the behavior correlations referenced in Appendix A) were considered too clay-rich to liquefy.

Based on our analyses of the CPT data, total settlement that could occur as a result of liquefaction from the design-level earthquake is estimated to range from about 2.5 to 3.7 inches at the ground surface. In our opinion, differential settlement of about 1½- to 2-inch over a horizontal distance of about 50 feet is possible at the ground surface from this amount of total settlement and this differential settlement could affect surface imporvements or structured supported on shallow piers or spread footings. However, since the proposed structures will be supported on pile foundations extending well below a majority of the liquefiable layers, in our opinion, the likelihood of significant damage to the proposed buildings from liquefaction is low.

The clayey soils that we encountered in the exploratory borings were primarily of moderate to high plasticity, generally having a low potential for liquefaction. Because the CPT tests included continuous measurement to a depth of about 80 feet, the CPT evaluation is considered more reliable, in our opinion.

Since there are no open faces or steep creek banks in the immediate site area, it is our opinion that there is a low potential for lateral spreading to occur at the site as a result of an earthquake.

Compressible Bay Mud

As discussed above, up to about 5 to 9.5 feet of relatively soft younger Bay Mud was encountered across the project site, and the Bay Mud is expected to be compressible under new building and fill loads. Based on the documents reviewed, the existing fills across the site appear to have been placed about 40 years ago. Because fill was placed so long ago and its thickness at the site, additional ongoing settlement within the Bay Mud from the existing fill loads is not expected to be significant.

Based on the preliminary grading plan, up to about 4.5 feet of fill may be required to raise the site grades and up to about 3.5 feet of fill may be required to raise the pad grade below the building floor slab. We estimated the amount of consolidation settlement that will occur based on the varying amounts of fill that will be placed. The results of our settlement evaluation for the range of Bay Mud thickness are presented in Table 5 below.

About 70 percent of the total settlement estimated in Table 5 from new fill placement will occur in a time period of about four months to one year, with 90 percent of the total settlement occurring over about one and a half to two years. We recommend that the fill for the building pads and surrounding areas be placed as early as practical.

Table 5. Estimated Fill 30-Year Consolidation Settlement
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Fill Thickness (ft)	Fill Load (psf)	Approximate Consolidation Settlement (inches)
0.5	62.5	0.3 - 0.4
1.0	125	0.7 - 1.2
1.5	187.5	1.2 - 2.1
2.0	250	1.7 - 2.8
2.5	312.5	2.1 - 3.6
3.0	375	2.5 - 4.3
3.5	437.5	2.8 - 4.9
4.0	500	3.1 - 5.2
4.5	652.5	3.4 - 5.6

Since the buildings will be supported on pile foundations, differential settlement will occur between the buildings and the surrounding areas receiving fill. This differential settlement should be considered in the design of entrance slabs or ramps that will not be supported on deep foundations and may need to be adjusted in the future. In addition, the above estimated settlement should be considered during the design of the underground utilities to be constructed within or around the building pads or across portions of the site requiring varying amounts of new fill. The settlement will also place a downdrag load on pile foundations which was considered during our pile analysis.

In addition, the placement of up to about 3 feet of new fill to raise site grades will cause settlement of the showroom building from the weight of the fill being transferred to the underlying Bay Mud along the southeast and southwest portions of the existing Audi showroom building. We estimate that up to approximately 1- to 1.25-inch of total settlement could occur across the southeast and southwest sides of the existing building due to placement of new fill. If the estimated settlements are not tolerable, the settlements may be reduced by specifying that the proposed fill below the adjacent proposed building and traffic drive aisle be a lightweight fill material or lightweight expanded polystyrene, such as Geofoam or a portion of the adjacent proposed building floor could be designed as a raised floor. We recommend that the lightweight fill or raised floor slab be used within a distance of about 20 feet of the existing Audi showroom building. We can provide further input into these alternatives during design.

Geologic Hazards

We briefly reviewed the potential for geologic hazards other than liquefaction and lateral spreading (which were discussed previously) to impact the site, considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below:

- Fault Rupture - The site is not located in a State of California Earthquake Fault Zone or area where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site and the potential for fault rupture at the site is considered low.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Strong ground shaking should therefore be expected several times during the life of the building, as is typical for sites throughout the Bay Area. The structures should be designed in accordance with current earthquake resistance standards.

- Differential Compaction - Differential compaction can occur during moderate and large earthquakes when soft or loose, natural or fill soils are densified and settle, often unevenly across a site. The soils encountered in our CPTs and borings were generally firm to very stiff clay and medium dense to dense sand. However some loosely compacted fill was encountered in the upper 5 to 7 feet in the previous borings advanced at the site. Since the proposed structures are expected to be supported on pile foundations extending well below the fill, in our opinion, the likelihood of structural damage to the proposed buildings from differential compaction is low, however some differential compaction could affect flatwork and pavements supported on existing grades or new fill areas.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed Mercedes-Benz and Audi dealership facility and associated site improvements provided the recommendations presented in this report are followed during design and construction. Specific recommendations are provided in the following sections of this report.

The primary geotechnical concerns are the presence of a shallow ground water table, the presence of soft compressible Bay Mud below the fill, and the probability of liquefaction and liquefaction-induced total and differential settlement at the site as a result of a major earthquake in the loose to medium dense sands encountered between depths of about 15 and 45 feet, particularly in the northwest to north portion of the site. In addition, somewhat variable soil and pile support conditions were encountered across the individual sites and also between the sites themselves as discussed further in the Pile Load Testing section of this report.

Due to the presence of compressible Bay Mud and the anticipated high column loads of the proposed dealership and garage buildings, we recommend that the proposed structures be supported on an auger cast pile foundation system. The piles will gain support in friction and will need to extend below a majority of the liquefaction prone soils encountered to depths of about 45 feet.

In addition, because of the amount of consolidation settlement from new fills to be placed at the site, the floor slabs at the ground level should be designed as structural slabs supported on the pile foundation. Differential settlement should also be considered in the design of entrance slabs or ramps that will not be supported on deep foundations and for underground utilities that connect to the pile supported structures or extend across portions of the site requiring varying amounts of new fill.

Because subsurface conditions may vary from those encountered at the locations of our CPTs and borings and to observe that our recommendations are properly implemented, we recommend that we be retained to 1) review the project plans for conformance with our report recommendations and 2) observe and test the earthwork and foundation installation phases of construction.

PILE FOUNDATIONS

Based on our evaluation of subsurface conditions and anticipated structural loads, the proposed structures should be supported on a deep foundation system, such as auger-cast piles which we understand is considered the preferable pile system. Recommendations for auger cast piles are presented in the following sections of this report.

Auger Cast Piles

In our opinion, the proposed structures may be supported on an auger cast pile foundation. The advantages of auger cast piles may include economy, reduced vibration influence on adjacent buildings, reduced noise during pile installation and the ability to install the piles through the interbedded very dense sand layers that may encounter refusal with a conventional driven pile.

The method, details, and equipment for construction of auger cast piles will be determined by a design/build auger cast pile subcontractor. In our opinion, the auger cast pile design/build contractor should have at least 5 years of auger cast pile experience and a proven track record of successful design and installation of auger cast piles in the Bay

Area. We anticipate the preferred type of auger cast piles will be auger pressure-grouted, partial displacement piles (Partial APGD).

Since a sufficiently thick and continuous end-bearing strata was not encountered during subsurface exploration, the piles will gain support primarily from friction along the pile shaft. The pile capacities were estimated using the data available from the CPTs that were advanced and the method of estimating pile capacity developed by Eslami and Fellenius (1997). This method uses direct readings of the cone tip resistance to estimate pile friction capacity by applying correlation coefficients based on soil type. We also estimated capacities using adhesion factors and shear strength profiles established during our field investigation.

The design lengths for individual 16-inch diameter auger cast piles may be estimated using the allowable capacity curves presented on Figures D-1 (1700 Embarcadero Road) and D-2 (1730 Embarcadero Road) of this report. The allowable pile capacity was calculated based a factor of safety of about 2.0. The axial capacity may be increased by one-third when evaluating for wind or seismic forces. The allowable capacities include a downdrag load caused by consolidation settlement of the Bay Mud from placement of new fill and neglecting the soils below the Bay Bud to a depth of 34 feet, above which a majority of the liquefiable soils occur. The structural engineer/design build contractor should confirm that the total structural load on the piles plus the downdrag load do not exceed the structural capacity of the pile. The actual capacity of auger cast piles will depend on the methods and details of pile installation and will need to be confirmed in the field by static and/or dynamic load tests on auger cast test piles prior to constructing the production piles.

Due to ground settlement from the compressible Bay Mud across the project sites, the upper portions of the piles within about 12 feet from top of the piles at 1700 Embarcadero Road within about 15 feet from top of the piles at 1730 Embarcadero Road will place a downdrag load on the piles. This load is estimated to be 54 kips and 67 kips for 1700 and 1730 Embarcadero Road, respectively. The structural section of the piles should be designed for the downdrag load plus the selected pile capacity from Figure D-1 and D-2.

As indicated on Figures D-1 and D-2, an 100-foot-long 16-inch diameter auger cast pile will have allowable capacity of about 207 kips at 1700 Embarcadero Road and 175 kips at 1730 Embarcadero Road, when considering dead plus live loads. Please note that some adjustment of the recommended allowable pile capacities may be appropriate following completion of the indicator pile program and dynamic pile monitoring.

Depending on the method and details of pile installation, it is possible that field load testing of auger cast test piles will establish that the allowable capacity of auger cast piles is higher than the pile capacity shown on Figure D-1.

The allowable uplift capacity of auger cast piles may be assumed to be 75 percent of the allowable downward capacity but no more than the allowable structural capacity of the pile in tension, as determined by the auger cast pile designer.

Pile Groups

The center-to-center spacing of auger cast piles in pile groups should be at least three pile diameters. With at least this minimum spacing, we expect the auger cast pile designer will determine that a pile group reduction factor is not required.

Lateral Loads on Piles and Pile Caps

To estimate the lateral load capacity of individual piles, we used L-Pile, a program that models lateral pile capacity and load/deflection response. Our lateral pile analyses were intended to model 80-foot-long, 16-inch diameter auger cast piles with an assumed pile concrete compressive strength of at least 6,000 pounds per square inch and a modulus of elasticity of $4.4x10^6$ pounds per square inch. An axial compression load of 115 kips was assumed to act on the head of the piles during lateral loading.

Our analysis used typical average soil conditions and no factor of safety was included. The structural engineer may need to use an appropriate factor of safety for their design, as appropriate. The calculated deflection, bending moment, and shear versus pile depth for various lateral loads under free head and fixed head conditions for auger cast piles are presented on Figures D-3 through D-8 of this report.

Our lateral capacity analyses were based on a single pile condition. For the pile groups, the shear planes in the soil overlap and therefore the resistance for an individual pile within the group is less than that of a single pile. To account for the reduction of soil resistance due to group effects, we recommend multiplying the lateral loads corresponding to a given deflection by the factors in Table 6 below for pile center-to-center spacing ranging from 2.5 to 4 times the shaft diameter.

For example, a 4 x 5 pile group with a center-to-center pile spacing (S/D) of 3 times the shaft diameter would use p-multipliers of 0.54 and 0.52 for loads applied in the direction of (perpendicular to) the 4 and 5 pile rows, respectively.

Table 6. Average P-Multipliers for Various Pile Groups
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Number of Rows*	PILE SPACING (S/D) 2.5	3	4
2	0.61	0.68	0.79
3	0.50	0.59	0.72
4	0.45	0.54	0.69
5	0.42	0.52	0.67

* Number of pile rows in the direction of loading

Lateral loads may also be resisted by passive soil pressure acting against the sides of pile caps and grade beams cast neat in excavations. We recommend that passive soil resistance simulated by an equivalent fluid pressure of 350 pounds per cubic foot be used for design, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the pile cap is not covered and protected by a concrete slab or pavement.

Pile Load Testing

Preliminary estimates of auger cast pile capacity will need to be confirmed in the field by static and/or dynamic load tests on auger cast test piles prior to constructing production piles. The number and length of test piles to be driven should be determined by the geotechnical engineer once the foundation plan has been finalized. On a preliminary basis, we expect that eight to twelve test piles should be installed across the proposed building areas to confirm the required final pile lengths. In addition, the test piles should extend well below a depth of 45 feet to extend below the liquefiable strata to better estimate pile skin friction. The subsurface investigation data indicates that variable soil strength conditions exist and a variation in pile capacity test results between the two sites and across each individual site should be expected. We recommend that the number and location of the test piles be selected to be able to establish the full range of pile capacities across the sites. Some of the test piles also should be located close to selected CPT locations.

The test piles should be installed with the same equipment that will be used to construct the production piles. The test piles should be constructed with continuous observation and monitoring by our staff. Pile load testing should also be monitored by our staff, and the results of the load testing used to confirm the final length and configuration of the production piles and pile caps.

Installation of Production Piles

We note that the actual load capacity and performance of auger cast piles are highly dependent on the method of installation, the contractor's experience, and the equipment that is used. Therefore, monitoring the installation of the auger cast piles will be essential to confirm the integrity and capacity of the piles. We recommend that only specialized contractors with proper equipment be considered for this project, and that all piles be installed under the continuous observation of the geotechnical engineer to confirm that the pile foundations are constructed in accordance with the recommendations presented in this report. For quality assurance purposes, we recommend that each auger cast pile rig be equipped with a Pile Installation Recorder (PIR), or comparable instrumentation, in order to accurately monitor the installation of each pile.

Pile Foundation Settlement

Based on the recommended maximum allowable pile load capacity described above, we estimate that total 30-year pile settlement will be less than ¾-inch to mobilize the allowable static capacity of the auger cast piles. Differential static settlements between adjacent columns is expected be less than about ½-inch. Additional settlement is expected from placement of new fill below the pavement and parking structure floor adjacent to the existing Audi Showroom building as discussed earlier.

As discussed above, some settlement of the more granular/less plastic strata is possible from liquefaction from the design level earthquake. Based on our liquefaction analysis, a significant percentage of the seismic related settlement will occur in the upper 34 feet of soils. However, since the piles are expected to extend well below the liquefiable strata and the potentially liquefiable strata were neglected in our pile capacity analysis to a depth of 34 feet, seismic settlement is not expected to significantly impact the pile supported structures.

Liquefaction related settlement could also affect the Audi showroom building and proposed at-grade improvements following strong seismic shaking. Further discussion regarding the at-grade improvements is presented below.

MINOR SITE IMPROVEMENT FOUNDATIONS

Isolated Spread Footings

In our opinion, it may be possible to support miscellaneous landscape improvements, such as low site retaining walls, privacy/sound walls, or other landscaping features on conventional spread footings bearing on stiff onsite surface fill soils. Once the type of structures to be supported on shallow foundations are known, these preliminary recommendations may need to be updated for the specific loading and type of improvement proposed. In general, footings should have a minimum width of 15 inches and extend at least 24 inches below the bottom of slabs-on-grade and at least 24 inches below exterior finish grade. Footings may be designed for allowable bearing pressures of 2,000 pounds per square foot for dead plus live loads, with a one-third increase allowed for total loads including wind or seismic forces. The weight of the footings can be neglected for design purposes.

All footings located adjacent to utility lines or other footings should bear below a 1:1 plane extended upward from the bottom edge of the utility trench. All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities.

The bottom of all footing excavations should be cleaned of loose material. Our representative should observe the excavations to confirm that they are founded in suitable materials and have been properly cleaned prior to placing concrete forms and reinforcing steel. If soft or loose materials are encountered at the foundation bearing depth, our field representative may require over-excavation and/or compaction before the reinforcing steel is placed or may require a deeper footing embedment depth.

Lateral loads will be resisted by friction between the bottom of the footings and the supporting subgrade. A coefficient of friction of 0.30 may be assumed for design of improvements supported in fill. Lateral resistance may also be provided by passive soil pressure acting against foundations cast neat in footing excavations or backfilled with properly compacted structural fill. We recommend that passive soil resistance simulated by an equivalent fluid pressure of 300 pounds per cubic foot be used for design, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing is not covered and protected by a concrete slab or pavement.

Drilled Piers

As an alternative to isolated footings, it may be possible to support "flagpole" type site improvements such as lighting poles, permanent fencing, and sound walls also may be supported on a drilled pier foundation bearing in stiff fill soil above the compressible Bay Mud. The piers for these improvements should extend at least 6 feet below existing grade or as required by the structural engineer to resist overturning. The piers may be designed for an allowable skin friction in native soil of 400 pounds per square foot for dead plus live loads, with a one-third increase allowed when considering additional short-term wind or seismic loading. The uplift capacity of the piers may be based on a skin friction value of 350 pounds per square foot. The vertical resistance of the soil against the upper 2 feet of the piers should be neglected in design. For better performance, piers should extend through the entire fill and Bay Mud and be embedded into the underlying stiff soils.

Lateral loads on the piers may be resisted by passive earth pressure based on an equivalent fluid pressure of 300 pounds per cubic foot acting on 2 times the projected area of the pier. Passive resistance provided by the upper 2 feet of soil should be neglected in design.

Pier drilling should be observed by a member of our staff to confirm that the pier holes extend at least the required minimum depth, expose the anticipated competent material, and are properly cleaned of all loose or soft soil and debris. The pier depths recommended above may require adjustment if differing conditions are encountered during pier drilling.

Concrete should be placed in the pier excavations as soon as practical after drilling. Depending upon the depth of the pier holes, ground water seepage may be encountered during pier drilling and it is possible that ground water seepage could cause some sloughing or caving conditions, if the piers are not cast soon after drilling. This can be further evaluated during drilling of the initial piers. If ground water cannot be successfully pumped from the pier holes, concrete should be placed by the tremie method. The use of a drilling fluid or casing could be considered if significant caving occurs during pier hole drilling or placement of concrete.

Settlement of Minor Site Improvements

We are not aware of any heavy surface or landscape improvements that are planned for the project. When the actual loads and foundation configuration of the surface and landscape improvements are available, we should be contacted and settlement analyses may need to be performed based on the actual loads, footing sizes, and sensitivity of the surface improvements to differential settlement.

As discussed above, ground settlement could occur in the areas where the exterior site grades will be revised with new fill. The amount of settlement will vary across the site based on the thickness of the fill that will be placed and the thickness of the underlying compressible Bay Mud. The estimated consolidation settlement discussed in the above section titled “Compressible Bay Mud” should be considered during the design of any surface improvements to be constructed on shallow footing or drilled pier foundations supported in the fill above the Bay Mud. In addition, miscellaneous structures that are sensitive to differential settlement preferably should not be located in areas where the thickness of new fill will vary significantly across the improvement area or deep foundations or a deeper drilled pier foundation should be considered.

Additional differential settlement to at-grade improvements may occur as a result of liquefaction caused by severe ground shaking during a major earthquake, as discussed earlier.

SLABS-ON-GRADE

General Slab Considerations

The surface and near surface fill soils at this site generally have a low potential for expansion. To reduce the potential for movement of the slab subgrade, at least the upper 6-inches of surface soil should be scarified and compacted at a moisture content at least 2 percent above the laboratory optimum. The native or fill soil subgrade should be kept moist up until the time the non-expansive fill and/or aggregate base is placed. Slab subgrades and non expansive fill should be prepared and compacted as recommended in the section of this report titled “Earthwork.” Exterior flatwork should be underlain by a layer of non expansive fill as discussed below. The non expansive fill should consist of aggregate base rock or a clayey soil with a plasticity index of 15 or less.

Considering the potential for expansive soil movements of the surface soils, we expect that a reinforced slab will perform better than an unreinforced slab. Consideration should also be given to using a control joint spacing on the order of 2 feet in each direction for each inch of slab thickness.

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 4 inches of Class 2 aggregate base. We recommend that exterior slabs-on-grade be constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs.

Interior Slabs

We understand that concrete floors at the ground level of the structures will be designed and constructed as structural slabs spanning across the foundations. In our opinion, structural slabs should be constructed on a properly prepared and compacted soil subgrade. In areas where dampness of concrete floor slabs would be undesirable, such as within building interiors, concrete slabs should be underlain by at least 4 inches of clean, free-draining gravel, such as ½-inch to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used. The crushed rock should be compacted with vibratory equipment.

To reduce vapor transmission up through at-grade concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor retarder meeting the minimum ASTM E 1745, Class C requirements or better. If moisture-sensitive floor coverings are proposed and/or additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms (such as 15-mil thick "Stego Wrap Class A") may be used rather than a Class C vapor retarder. The vapor retarder or barrier should be placed directly below the concrete slab. Sand above the vapor retarder/barrier is not recommended. The vapor retarder/barrier should be installed in accordance with ASTM E 1643. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations.

The permeability of concrete is affected significantly by the water:cement ratio of the mix, with lower water:cement ratios producing more damp-resistant slabs and higher strength. Where moisture protection is important and/or where the concrete will be placed directly on the vapor barrier, the water:cement ratio should be 0.45 or less. To increase the workability of the concrete, mid-range plasticizers may be added to the mix. Water should not be added to the mix unless the slump is less than specified and the water:cement ratio will not exceed 0.45. Other steps that may be taken to reduce moisture transmission through concrete slabs-on-grade include moist curing for 5 to 7 days and allowing the slab to dry for a period of two months or longer prior to placing floor coverings. Prior to installation of floor coverings, it may be appropriate to test the slab moisture content for adherence to the manufacturer's requirements to determine whether a longer drying time is necessary.

RETAINING WALLS

We recommend retaining walls with level backfill that are not free to deflect or rotate, such as building walls or elevator pits, be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot, plus an additional uniform lateral pressure of 8H pounds per square foot, where H is the height of the wall in feet. Where retaining walls are assumed to be undrained, such as for the elevator pit walls, these walls should be designed to resist an equivalent fluid pressure of 80 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot (where H is the height of the wall in feet).

Retaining walls with level backfill that are free to rotate, such as site retaining walls (if any), may be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot. Retaining walls with backfill that slopes at about 2:1 (horizontal:vertical) should be designed to resist an equivalent fluid pressure of 65 pounds per cubic foot for walls free to rotate, with 8H added as recommended above for walls not free to rotate. Wherever retaining walls or elevator pit walls will be subjected to surcharge loads, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge load for restrained walls and one-third of the surcharge load for unrestrained walls.

Based on the site peak ground acceleration (PGA), on Seed and Whitman (1970); Al Atik and Sitar (2010); and Lew et al. (2010); seismic loads on retaining walls that can yield may be simulated by a line load of $2H^2$ (in pounds per foot, where H is the wall height in feet). Seismic loads on walls that cannot yield may be subjected to a seismic load as high as about $8H^2$. This seismic surcharge line load should be assumed to act at 1/3H above the base of the wall (in addition to an active wall design pressure of 45 pounds per cubic foot).

To prevent buildup of water pressure from surface water infiltration, a subsurface drainage system could be installed behind retaining walls, otherwise the walls should be designed for undrained pressures as discussed above. The drainage system should consist of a 4-inch diameter perforated pipe (perforations placed down) embedded in a section of 1/2- to 3/4-inch, clean, crushed rock at least 12 inches wide. Backfill above the perforated drain line should also consist of 1/2- to 3/4-inch, clean, crushed rock to within about 1½ to 2 feet below exterior finished grade. A filter fabric should be wrapped around the crushed rock to protect it from infiltration of native soil. The upper 1½ to 2 feet of backfill should consist of compacted native soil. The perforated pipe should discharge into a free-draining outlet or sump that pumps to a suitable location. Damp-proofing of the walls should be included in areas where wall dampness and efflorescence would be undesirable.

Miradrain, Enkadrain or other drainage fabrics approved by our office may be used for wall drainage as an alternative to the gravel drainage system described above. If used, the drainage fabric should extend from a depth of about 1 foot below the top of the wall backfill down to the drain pipe at the base of the wall. A minimum 12-inch wide section of ½-inch to ¾-inch clean crushed rock and filter fabric should be placed around the drainpipe, as recommended previously.

Backfill placed behind the basement walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used for compaction of wall backfill, the walls should be temporarily braced.

Building retaining walls should be supported on a pile foundation designed in accordance with the recommendations presented previously. Low landscaping walls may be supported on conventional continuous shallow or drilled pier foundations as presented previously.

VEHICLE PAVEMENTS

Asphalt Concrete Pavements

We understand the existing vehicle pavements will be removed, finished grades adjusted slightly for improved surface water drainage, and new asphalt concrete pavements constructed. The new pavement sections will be supported on the existing variable clayey sandy fill soils, which may be assumed to have an R-value of 18 for design purposes. Following Procedure 630 of Caltrans Highway Design Manual, we developed the minimum recommended pavement section thicknesses presented on Table 7, below.

The Traffic Indices used in our pavement thickness calculations are considered reasonable values for this development and are based on engineering judgment rather than on detailed traffic projections. Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of the Caltrans Standard Specifications, latest edition, except that compaction should be based on ASTM Test D1557.

We recommend that measures be taken to limit the amount of surface water that seeps into the aggregate base and subgrade below vehicle pavements, particularly where the pavements are adjacent to landscape areas. Seepage of water into the pavement base material tends to soften the subgrade, increasing the amount of pavement maintenance that is required and shortening the pavement service life. Deepened curbs extending

4-inches below the bottom of the aggregate base layer are generally effective in limiting excessive water seepage. Other types of water cutoff devices or edge drains may also be considered to maintain pavement service life.

Table 7. Minimum Asphalt Concrete Pavement Sections
Mercedes-Benz and Audi Dealerships
Palo Alto, California

Traffic Loading Condition	Design Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)	Total Thickness (inches)
Automobile Parking	4.0	3.0	5.0	8.0
Automobile Access	4.5	3.0	7.0	10.0
Light Truck Traffic	5.0	3.0	8.0	11.0
	5.5	3.0	10.0	13.0
Moderate Truck Traffic	6.0	4.0	9.0	13.0
Heavy Truck Traffic	7.0	4.0	12.0	16.0

*****Caltrans Class 2 Aggregate Base (minimum R-value = 78).

Portland Cement Concrete Pavements

If Portland Cement Concrete (PCC) pavements are to be used on portions of the site, the minimum required thickness of the PCC pavements should be based on the anticipated traffic loading, the modulus of rupture of the concrete that will be used for pavement construction, and the composition and supporting characteristics of the soil subgrade below the pavement section.

To provide a general guideline for the minimum required thickness of PCC pavements, we used information in the Portland Cement Association publication titled “Thickness Design for Concrete Highway and Street Pavements.” We assumed “low” subgrade support from the on-site fills, typical residential street traffic (trucks with maximum single axle loads of 22 kips and maximum tandem axle loads of 36 kips), aggregate-interlock joints (i.e. no dowels), no concrete shoulder or curb, a modulus of rupture of concrete of 550 psi (which correlates to a concrete compressive strength of approximately 3,700 psi), at least 8 inches of Class 2 aggregate base below the PCC pavement, and 20-year pavement service life. Sufficient control joints should be incorporated in the design and construction to limit and control cracking.

Based on the design assumptions described above, a PCC pavement with a thickness of at least 6 inches would be adequate for average daily truck traffic (ADTT) of one; a thickness of at least 6.5 inches would be adequate for ADTT of 13; and a thickness of at least 7 inches would be adequate for ADTT of 110.

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as existing foundations and pavements, utilities to be abandoned, vegetation, root systems, loose surface fills, topsoil, etc. should be cleared from areas of the site to be built on or paved. The actual stripping depth should be determined by a member of our staff in the field at the time of construction. Excavations that extend below finished grade should be backfilled with structural fill that is water-conditioned, placed, and compacted as recommended in the section of this report titled “Compaction.”

After the site has been properly cleared, stripped, and excavated to the required grades, exposed soil surfaces in areas to receive structural fill or slabs-on-grade should be scarified to a depth of 6 inches, moisture conditioned, and compacted as recommended for structural fill in the section of this report titled "Compaction."

On-site native soils, slab and pavement subgrades, footing, grade beam and pile cap excavations, and utility trench excavation, should be kept in a moist condition throughout the construction period.

Material For Fill

All on-site soil containing less than 3 percent organic material by weight (ASTM D2974) may be suitable for use as structural fill. Structural fill should not contain rocks or pieces larger than 6 inches in greatest dimension and no more than 15 percent larger than 2.5 inches. Imported, non-expansive fill should have a Plasticity Index no greater than 15, should be predominately granular, and should have sufficient binder so as not to slough or cave into foundation excavations or utility trenches. A member of our staff should approve proposed import materials prior to their delivery to the site.

Recycling of Existing Building and Pavement Materials

Portions of the concrete floors, and foundations of the existing buildings, and other miscellaneous concrete that are present are expected to be pulverized on-site and reused as part of the proposed construction. If these materials are properly crushed and handled, the pulverized materials will be suitable for use as structural fill, non-expansive fill, and subbase, and possibly as Class 2 aggregate base below vehicle pavements.

If the on-site asphalt concrete is properly pulverized and handled, the pulverized asphalt concrete should be suitable for use as aggregate base or subbase below exterior flatwork, walkways, and vehicle pavements depending on the gradation of the pulverized asphalt concrete material. We also expect the majority of the existing aggregate base below pavements, buildings, and slabs will be able to be used as structural fill, non-expansive fill, subbase, or aggregate base, depending on how the materials are handled. We do not recommend that recycled asphalt concrete be used as non-expansive fill below the footprint of the building.

Compaction

Scarified soil surfaces and all structural fill should be compacted in uniform lifts no thicker than 8-inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 8 on the following page. The relative compaction and moisture content recommended in Table 8 is relative to ASTM Test D1557, latest edition.

Table 8. Compaction Recommendations
Mercedes-Benz and Audi Dealerships
Palo Alto, California

General	**Relative Compaction***	**Moisture Content***
• Scarified subgrade in areas to receive structural fill.	90 percent	Above optimum
• Structural fill composed of native soil.	90 percent	Above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
• Structural fill below a depth of 4 feet.	93 percent	Above optimum
Pavement Subgrade		
• On-site soil.	95 percent	Near optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soil.	90 percent	Near optimum
• Imported sand.	93 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Temporary Slopes, Excavations and Dewatering

Ground water should be expected in the bottom of utility trench and manhole excavations that extend down to or below the ground water elevations described previously. If this occurs, provisions will need to be made for dewatering and maintaining sidewall stability during placement and compaction of pipe bedding and backfill.

The contractor should be responsible for the design and construction of all temporary slopes, excavations, and shoring. Shoring and bracing should be designed and installed in accordance with applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards.

If deep excavations are required that extend into or close to the soft saturated Bay Mud, they may be prone to sloughing and/or caving if excavated near-vertical, and could become unstable. If excavations will extend into the Bay Mud, sheet piles or an equivalent method may be required to support the walls of the excavations. This information should be considered by the contractor when establishing temporary shoring/bracing/cut slope criteria for any deep utility trench excavations and other temporary cuts. Excavations that extend below ground water will require flatter inclinations or temporary shoring. If deep excavations are required, we can provide further input as needed.

Because of the potential variation of the surface and near-surface soils, field modification of temporary cut slopes and excavations may be required. Unstable materials near trenches, excavations, and slopes should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

Protection of structures near excavations and trenches will also be the responsibility of the contractor. In our experience, a preconstruction survey is generally performed to document existing conditions prior to construction, with intermittent monitoring of the structures during construction.

Finished Slopes

We recommend that finished slopes be cut or filled to an inclination no steeper than 3:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion that would require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted to with erosion resistant vegetation.

<u>Surface Drainage</u>

Finished grades should be designed to prevent ponding and to drain surface water away from foundations and edges slabs and pavements, and toward suitable collection and discharge facilities. Slopes of at least 2 percent are recommended for flatwork and pavement areas with 5 percent preferred in landscape areas within 8 feet of the structures, where possible. At a minimum, splash blocks should be provided at the ends of downspouts to carry surface water away from perimeter foundations. Preferably, downspout drainage should be collected in a closed pipe system that is routed to a storm drain system or other suitable discharge outlet.

Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during first two years following construction. We recommend that an as-built plan be prepared to show the locations of all surface and subsurface drain lines and clean-outs. Drainage facilities should be periodically checked to verify that they are continuing to function properly. The drainage facilities will probably need to be periodically cleaned of silt and debris that may build up in the lines.

FUTURE SERVICES

<u>Plan Review</u>

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations contained in this report. We should be provided with these plans as soon as possible upon completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review process. In addition, it should be noted that many of the local building and planning departments now require "clean" geotechnical plan review letters prior to acceptance of plans for their final review. Since our plan reviews typically result in recommendations for modification of the plans, our generation of a "clean" review letter often requires two iterations. At a minimum, we recommend the following note be added to the plans:

"Earthwork, slab subgrade preparation, foundation construction, pile installation and load testing, pavement subgrade and aggregate base construction, backfilling of walls and utility trenches, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated May 8, 2018. Romig Engineers should be notified at least 48 hours in advance of any earthwork and should observe and test during earthwork and foundation construction as recommended in the geotechnical report."

Construction Observation and Testing

The earthwork and foundation phases of construction should be observed and tested by us to 1) establish that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the design concepts, specifications and recommendations; and 3) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on a limited amount of subsurface exploration. The nature and extent of variation across the site may not become evident until construction. If variations are exposed during construction, it will be necessary to reevaluate our recommendations.

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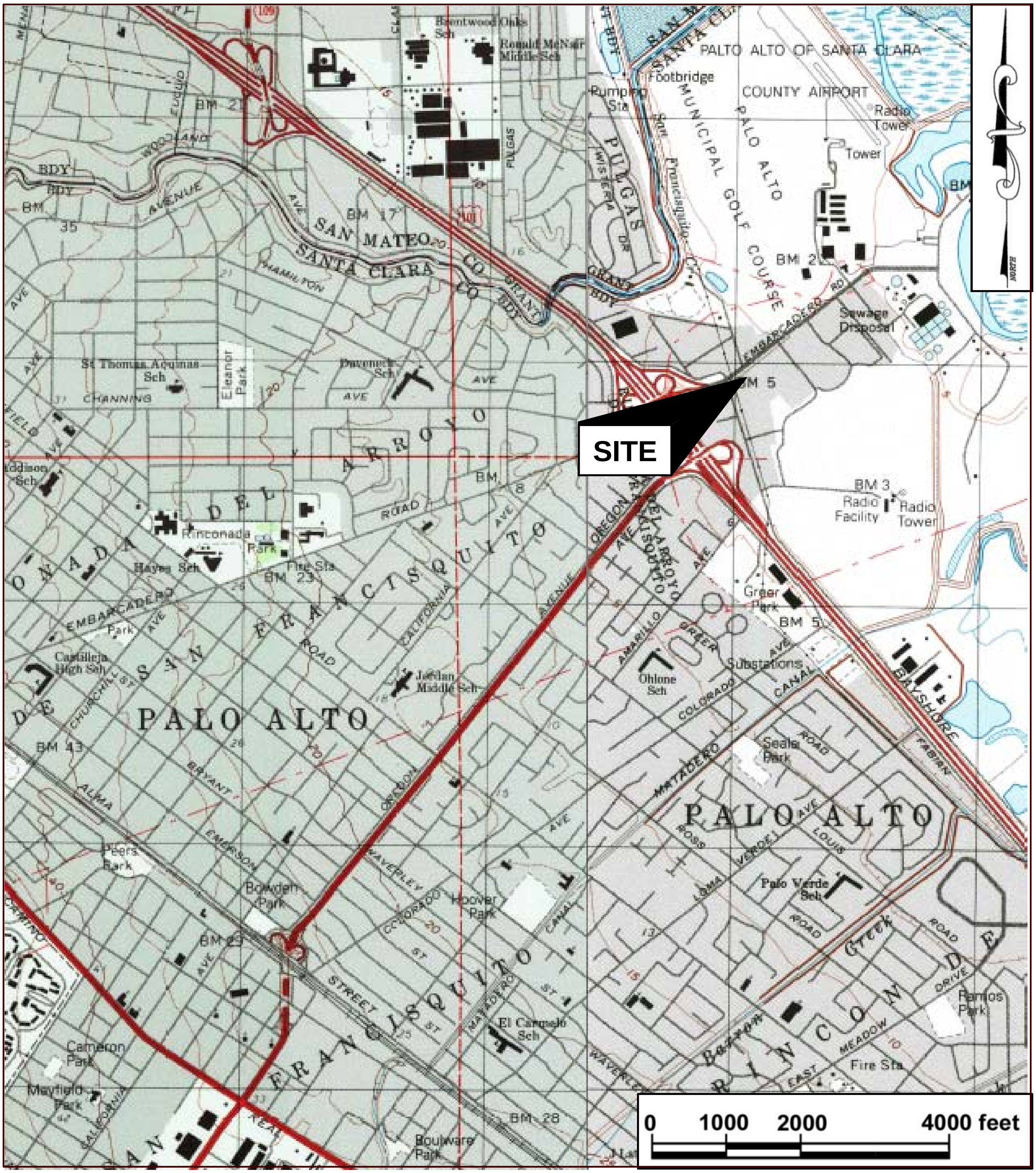
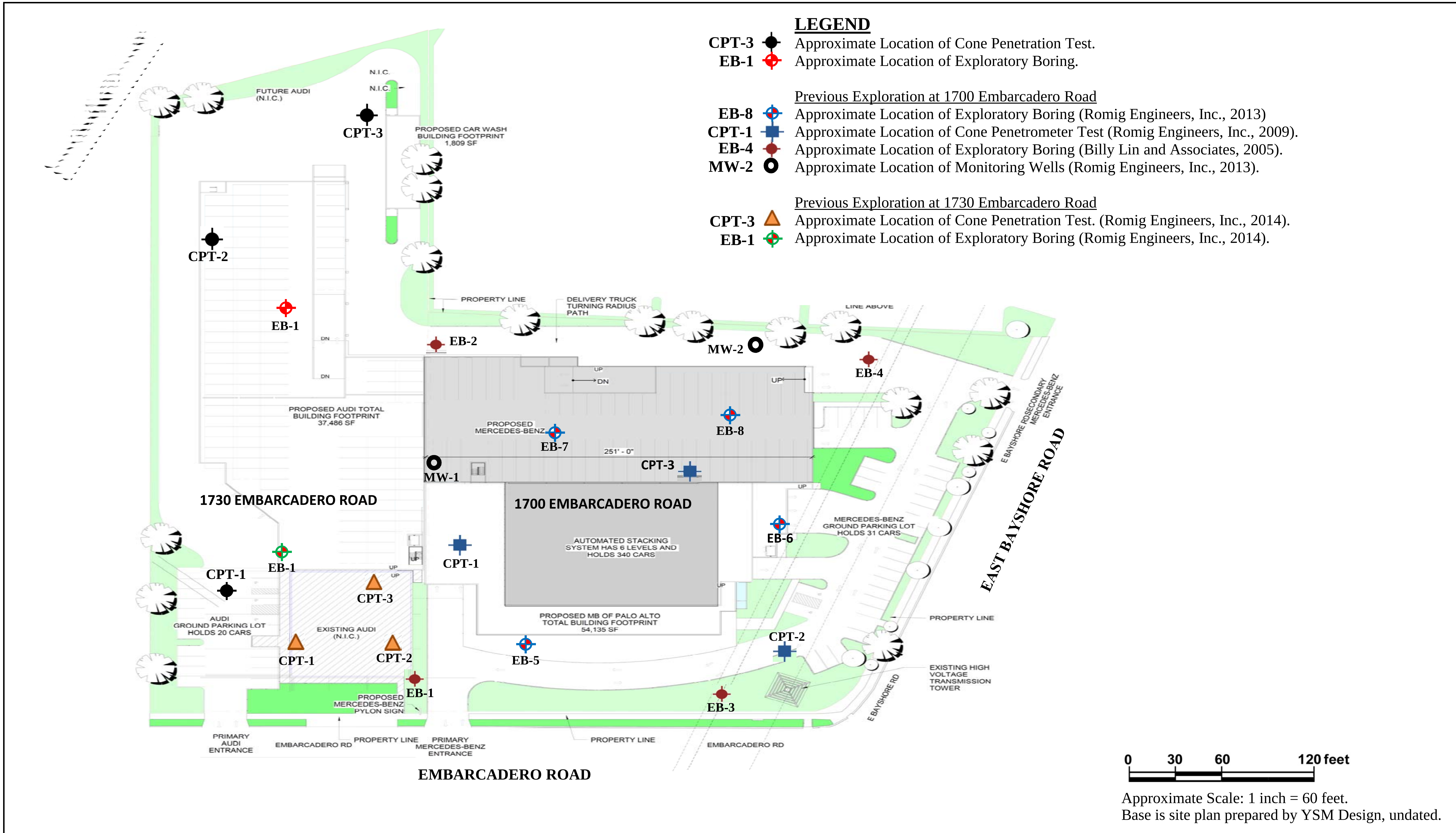


Scale: 1 inch = 2000 feet
Base is United States Geological Survey Palo and Mountain View Alto 7.5 Minute Quadrangle, dated 1997.

VICINITY MAP
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE 1
MAY 2018
PROJECT NO. 4300-1

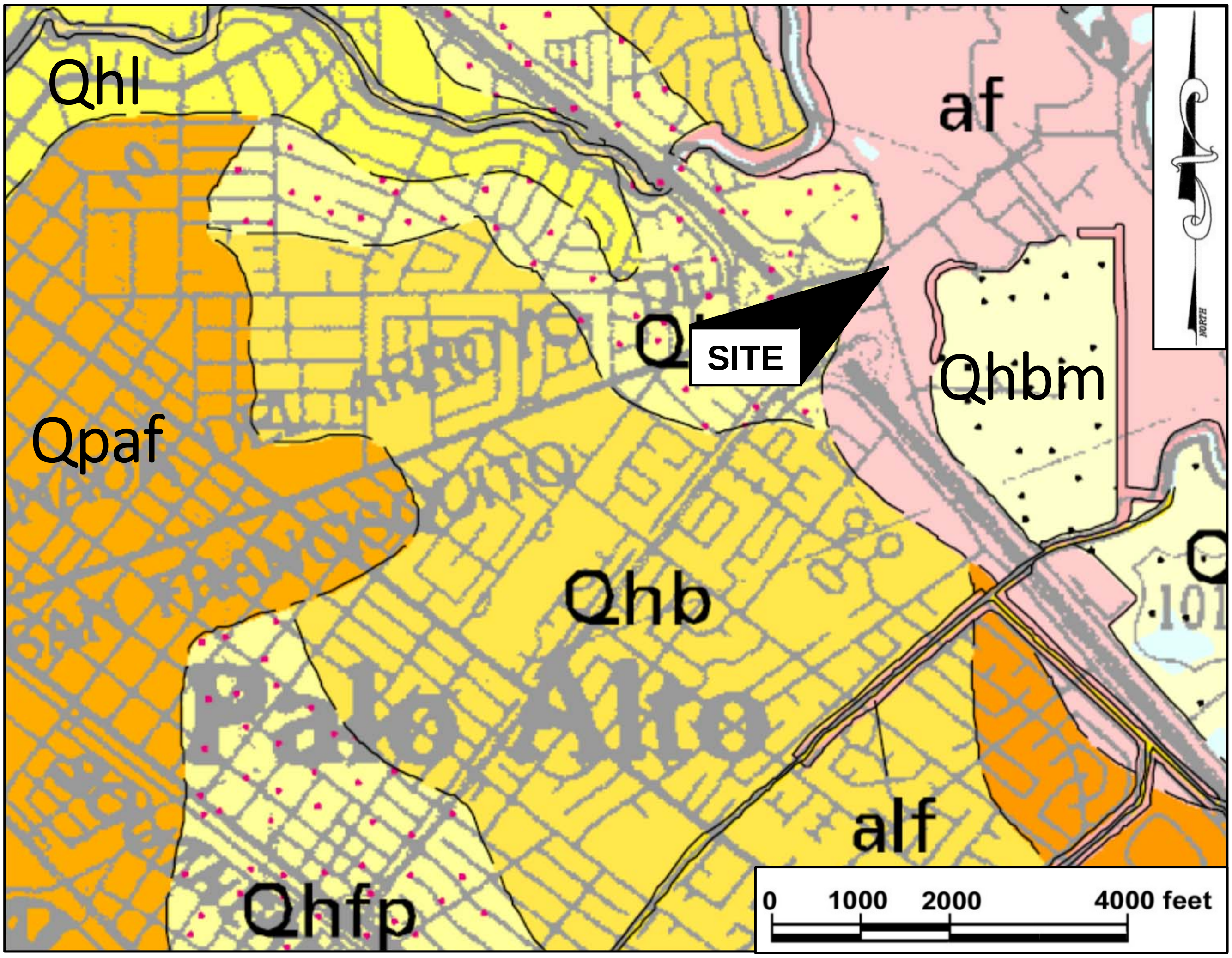
ROMIG ENGINEERS



SITE PLAN
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

ROMIG ENGINEERS

FIGURE 2
MAY 2018
PROJECT NO. 4300-1



MAP LEGEND

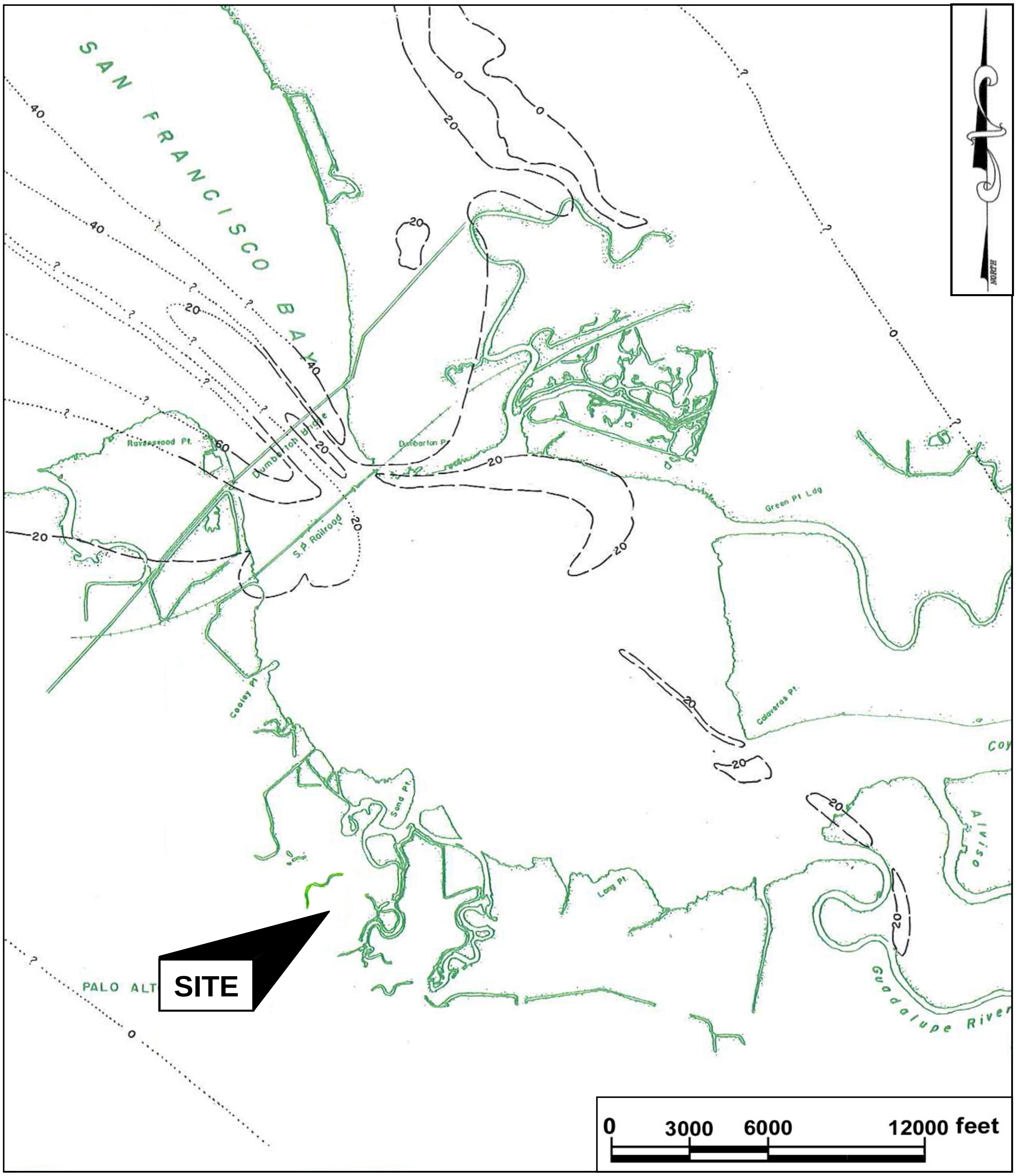
af	Artificial Fill
alf	Artificial Levee Fill
Qhbm	Bay Mud
Qhb	Basin Deposits
Qhfp	Floodplain Deposits
Qhl	Natural Levee Deposits
Qpaf	Alluvial Fan and Fluvial Deposits
———— / — — - - - -	Geologic Contact - dashed where approximate, dotted where inferred.

Scale: 1 inch = 2000 feet
Base is Geologic Map of Palo Alto 30 x 60 Minute Quadrangle (Brabb, Graymer, and Jones, 2000).

VICINITY GEOLOGIC MAP
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE 3
MAY 2018
PROJECT NO. 4300-1

ROMIG ENGINEERS



Scale: 1 inch = 6000 feet

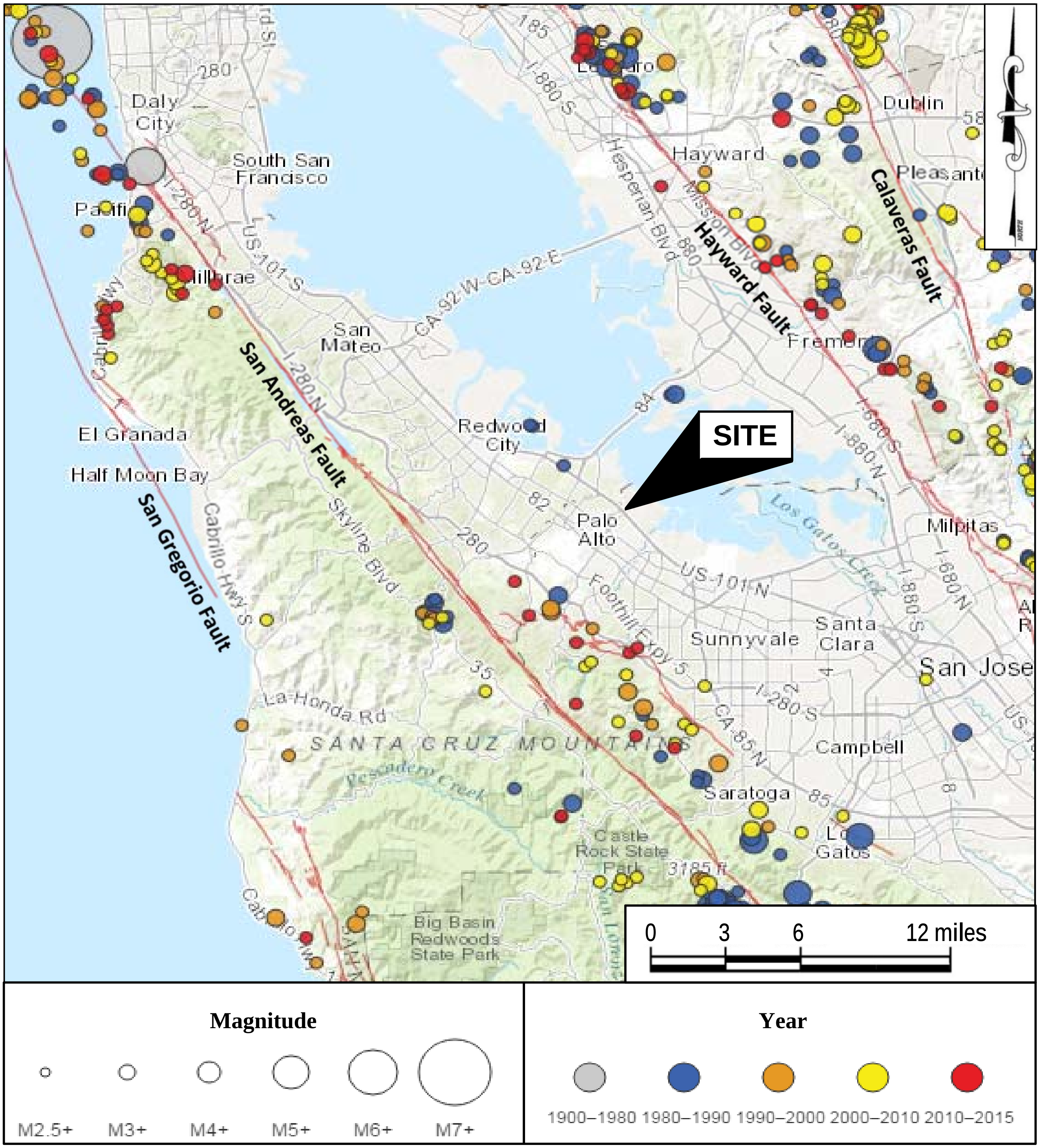
Base is Geolgic and Engineering Aspects of San Francisco Bay Fill, Special Report 97, Map of "Thickness of Younger Bay Mud", (CDMG, 1966).

VICINITY MAP | **FIGURE 4**

MERCEDES BENZ AND AUDI DEALERSHIPS | MAY 2018

PALO ALTO, CALIFORNIA | PROJECT NO. 4300-1

ROMIG ENGINEERS



Earthquakes with M5+ from 1900 to 1980, M2.5+ from 1980 to January 2015. Faults with activity in last 15,000 years. Based on data sources from Northern California Earthquake Data Center and USGS Quaternary Fault and Fold Database, accessed May 2015.

REGIONAL FAULT AND SEISMICITY MAP
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE 5
MAY 2018
PROJECT NO. 4300-1

ROMIG ENGINEERS

APPENDIX A

SUMMARY OF FIELD INVESTIGATION DATA

Subsurface exploration at the sites were performed by means of exploratory borings and Cone Penetration Test (CPT) probes to explore subsurface conditions.

Four cone penetration test (CPT) probes advanced at 1700 Embarcadero Road in 2009 were performed by California Push Technologies, Inc. using a track-mounted, Geoprobe Model 6625CPT rig to advance an electronic cone penetration test (CPT) probe. Four cone Penetration Tests (CPT) were performed by Middle Earth Geo Testing, Inc. at 1730 Embarcadero Road in 2014 and 2018 using an integrated electronic cone system. The CPT soundings were performed in accordance with ASTM standards (D 5778-95). A 20 ton capacity cone was used for the sounding. The cone had a tip area of 10 cm^2 and friction sleeve area of 150 cm^2. The logs of our CPT's are attached in this Appendix.

The soils encountered during drilling of the borings were logged by our representative and samples were obtained at depths appropriate to the investigation. The samples were taken to our laboratory where they were evaluated and classified in accordance with the Unified Soil Classification System. The logs of our borings and the previous borings conducted at the sites, and a summary of the soil classification system used on the logs (Figure A-1), are included in this appendix.

The standard penetration test resistance was determined by dropping a 140-pound hammer through a 30-inch free fall and recording the blows required to drive the 2-inch (outside diameter) sampler 18 inches. The standard penetration test (SPT) resistance is the number of blows required to drive the sampler the last 12 inches and is recorded on the boring logs at the appropriate depths. Soil samples were also collected using 3.0-inch O.D. drive samplers. The blow counts shown on the logs for these larger samplers do not represent SPT values and have not been corrected in any way.

The locations of the CPTs and borings were determined by pacing using the site plan provided to us. The CPT and boring locations should be considered accurate only to the degree implied by the method used. The CPT and boring logs and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the subsurface conditions.

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USCS SOIL CLASSIFICATION

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (< 50 % Fines)	GRAVEL	CLEAN GRAVEL (< 5% Fines)	GW	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL with FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND	CLEAN SAND (< 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (> 50 % Fines)	SILT AND CLAY Liquid limit < 50%		ML	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY Liquid limit > 50%		MH	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.
BEDROCK			BR	Weathered bedrock.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 to 4
LOOSE	4 to 10
MEDIUM DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
FIRM	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12 " 3" 0.75" 4 10 40 200

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

* Standard Penetration Test (SPT) resistance, using a 140 pound hammer falling 30 inches on a 2 inch O.D. split spoon sampler; blow counts not corrected for larger diameter samplers.

^ Unconfined Compressive strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

KEY TO SAMPLERS

- Modified California Sampler (3-inch O.D.)
- Mid-size Sampler (2.5-inch O.D.)
- Standard Penetration Test Sampler (2-inch O.D.)

KEY TO EXPLORATORY BORING LOGS **FIGURE A-1**
MERCEDES BENZ AND AUDI DEALERSHIPS MAY 2018
PALO ALTO, CALIFORNIA PROJECT NO. 4300-1

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-53 with 7-1/4" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 8 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 2/2/18

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
4 inches Asphalt Concrete and 2 inches Aggregate Base Rock		AC		0					
Artificial Fill: Brown, Sandy Lean Clay, very moist, fine to coarse grained sand, moderate plasticity, trace, sub-rounded to rounded gravel, gray mottling.	Very Stiff to Hard	CL				28	18	0.8	2.3
■ Liquid Limit = 39, Plasticity Index = 19.				5		58	13	0.5	1.5
Young Bay Mud: Gray-Brown, Fat Clay, wet, fine grained sand, high plasticity.	Firm to Stiff	CH				9	73	0.2	<0.5
▼ Ground water measured at 8 feet after drilling.				▼					
				10		6	87	0.1	<0.5
Brownish-gray, Sandy Lean Clay, moist, fine to coarse grained sand, fine sub-angular to rounded gravel, moderate plasticity, orange and white mottling.	Very Stiff	CL		15		25	22		2.3
Brown, Clayey Sand, very moist, fine to coarse grained sand.	Very Dense	SC							
● 16% Passing No. 200 Sieve.				20		56	22		

Continued on Next Page

DRILL TYPE: Mobile Drill B-53 with 7-1/4" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 8 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 2/2/18

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Clayey Sand, very moist, fine to coarse grained sand.	Very Dense	SC		20					
Orange to brown, Sandy Lean Clay, very moist, fine to medium grained sand, moderate plasticity, gray mottling.	Very Stiff	CL		25		29	21		3.0
				30		16	28		
Brown, Poorly Graded Sand, moist, fine to coarse grained sand, fine to coarse sub-angular to rounded gravel.	Very Dense	SP		35		50/6"	12		
				40		70	13		

Continued on Next Page

DRILL TYPE: Mobile Drill B-53 with 7-1/4" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 8 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 2/2/18

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Poorly Graded Sand, moist, fine to coarse grained sand, fine to coarse sub-angular to rounded gravel.	Very Dense	SP		40					
● 13% Passing No. 200 Sieve.				45	●	45	13		
Grayish-brown, Sandy Fat Clay, very moist, fine grained sand, high plasticity.	Very Stiff	CH							
				50		21	20		
Bottom of Boring at 50 feet.									
				55					
				60					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

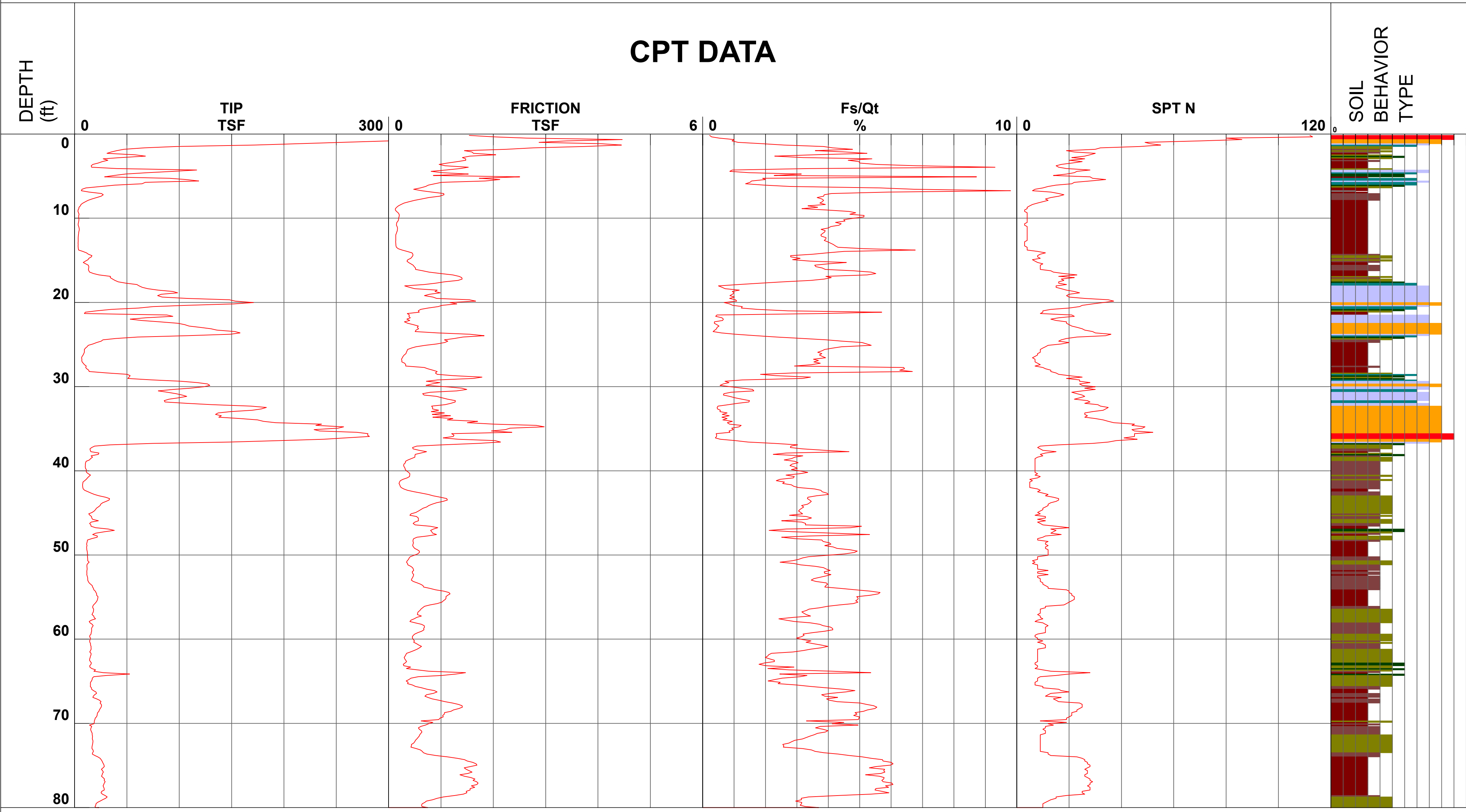
*Measured using Torvane and Pocket Penetrometer devices.

Romig Engineers

Project	Audi Palo alto	Operator	BH-RB	Filename	SDF(053).cpt
Job Number	4300-1	Cone Number	DDG1418	GPS	
Hole Number	CPT-01	Date and Time	2/14/2018 8:38:23 AM	Maximum Depth	80.54 ft
EST GW Depth During Test			9.00 ft		

Net Area Ratio .8

CPT DATA



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 (*)

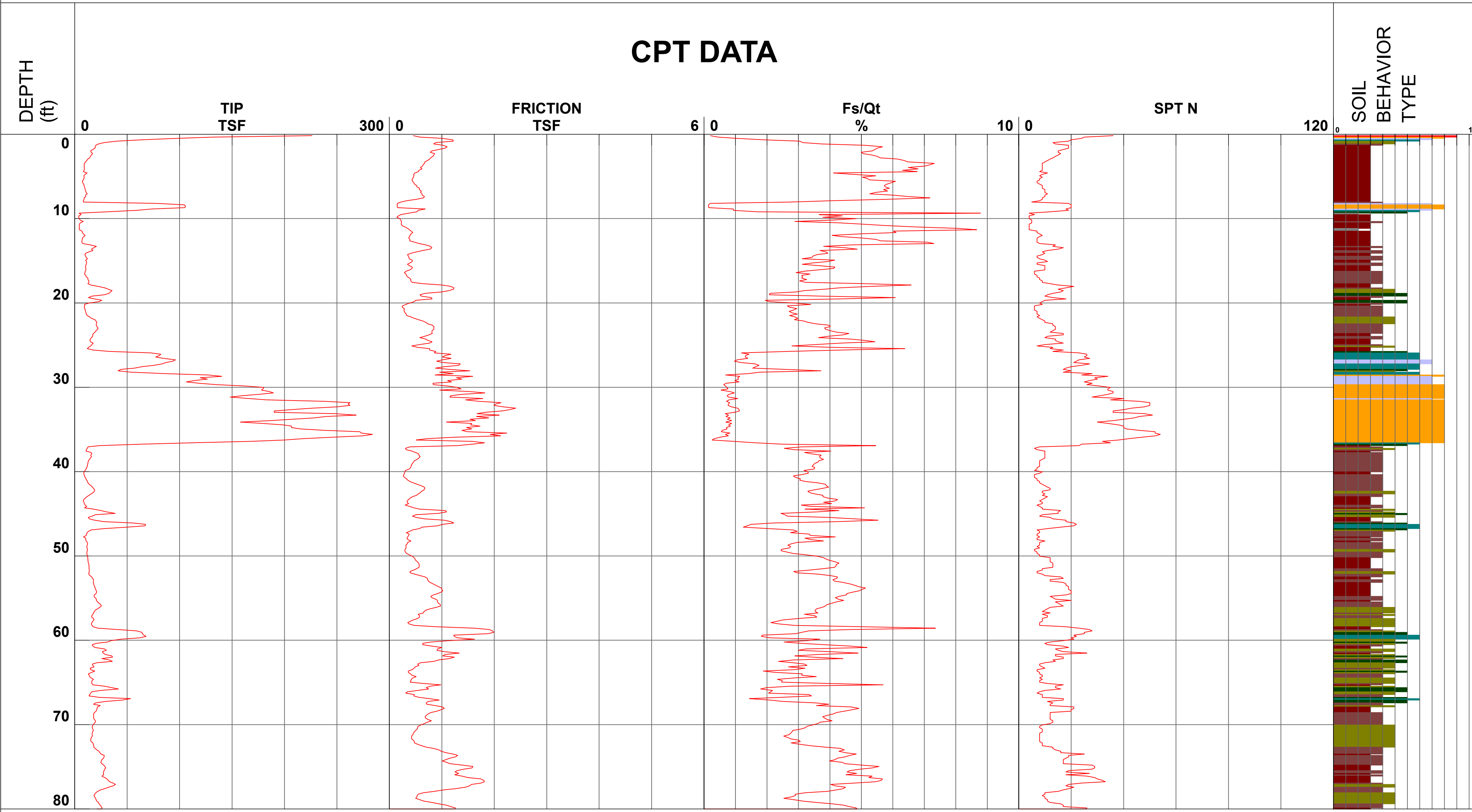
Cone Size 10cm squared

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

Romig Engineers

Project	Audi Palo alto	Operator	BH-RB	Filename	SDF(054).cpt
Job Number	4300-1	Cone Number	DDG1418	GPS	
Hole Number	CPT-02	Date and Time	2/14/2018 9:42:25 AM	Maximum Depth	80.54 ft
EST GW Depth During Test		8.10 ft			

Net Area Ratio .8



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 (*)

Cone Size 10cm squared

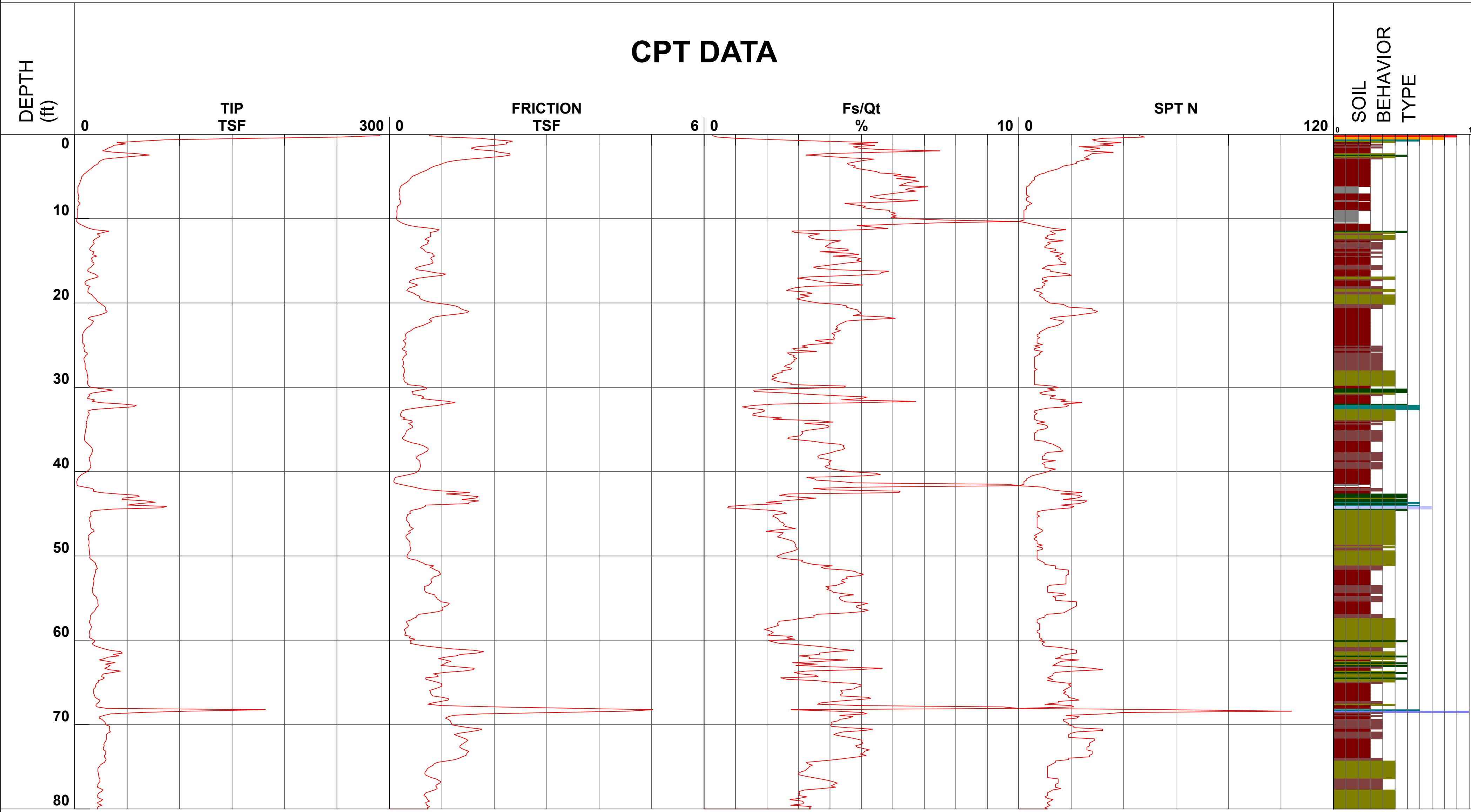
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Audi Palo alto	Operator	BH-RB	Filename	SDF(055).cpt
Job Number	4300-1	Cone Number	DDG1418	GPS	
Hole Number	CPT-03	Date and Time	2/14/2018 11:58:29 AM	Maximum Depth	80.54 ft
EST GW Depth During Test		7.50 ft			

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

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

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 7.34 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 7 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/13/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
PAVEMENT (1½" AC over 4" AB*)		--	--	--							
CLAYEY SAND, fine- to coarse-grained, silty, with trace of coarse-grained, angular gravels (rock fragments). (FILL)		Gray	Moderate Compaction	SC	1						
					2		37	7.6	118.0		
					3						
					4						
SILTY SAND, fine-grained (FILL)		Dark Bluish Gray	Very Loose Compaction	SM	5						
					6		6	32.4	83.6		
					7			▼ Elev. 0.34			
					8	Elev -0.66					
FAT CLAY, with some fine-grained sands and trace of fine-grained gravels; desiccated, (Bay Mud)		White & Gray mottled	Firm	CH	9						
					10						
(*1 Test: Liquid limit = 60%, Plastic Limit = 17%, PI = 43%) (*2 Test: Initial void ratio = 0.960, maximum consolidation pressure = ±1,900 psf)					11	*1, *2	5	31.2	87.6		
					12						
					13						
					14	Elev -6.66					
CLEAN SAND, fine to coarse grained, with significant amount of fine-grained, round gravels; poorly graded		Grayish Brown	Medium Dense	SP	15						
(*3 Test: 74% fine to coarse sands, 23% fine gravels, and 3% fines)					16	*3	18	15.5	114.4		88.0
					17						
					18						
(grading with some silts below 18 feet)		Gray	Very Dense		19						
					20						

Remark:

▼ Groundwater Table

* AC: Asphaltic Concrete, AB: Aggregate Base

California Split Spoon Sampler, 2-inch inside diameter

Billy Lin and Associates

5088 CRIBARI BLUFFS
SAN JOSE, CALIFORNIA 95135
PHONE: (408) 531-9889 FAX: (408) 531-8913
E-MAIL: billy.lin@sbcglobal.net

EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-1	1 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 7.34 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 7 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/13/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
CLEAN SAND (continued)		Gray	Very Dense	SP							
(*1 Test: 62% fine to coarse sands, 30% fine to coarse gravels and 8% fines)					21	*1	69	11.8	124.6		90.0
					22						
					23						
					24						
SILTY CLAY		Light Brown	Stiff	CL	25			23.6	108.4	1.7	
					26		25				
CLEAN SAND, fine to coarse grained, fine gravelly, with some silts; poorly graded; locally with clayey sand pockets		Grayish Brown	Dense	SP				10.9	115.8		
					27						
					28						
					29						
		Greenish Gray			30						
(*2 Test: 60% fine to coarse sands, 32% fine gravels, and 8% silts)					31	*2	32	16.6	116.4		99.8
					32						
					33						
					34						
(graded to fine-grained sands below 34 feet)		Gray & Brown mottled	Medium Dense		35						
					36		12	18.9	112.3		
					37						
					38	Elev -30.66					
LEAN CLAY, with some fine-grained sands		Bluish Gray	Very Stiff	CL-CH							
					39						
					40						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

Billy Lin and Associates

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-1	2 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 7.34 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 7 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/13/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
LEAN CLAY (continued)		Bluish Gray	Very Stiff	CL-CH							
					41	*1	26	24.5	102.5		99.6
(*1 Test: Liquid limit = 49%, plastic limit = 17%, PI = 32%)					42						
					43						
CLEAN SAND, fine to medium grained, with some coarse gravels		Grayish Brown	Dense	SP	44						
					45						
					46		29	21.6	104.4		
BOTTOM OF BORING = ±46.5 FEET					47						
					48						
					49						
					50						
					51						
					52						
					53						
					54						
					55						
					56						
					57						
					58						
					59						
					60						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-1	3 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 8.74 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 6 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/13/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
PAVEMENT (2" AC over 6" AB)		--	--	--							
SILTY CLAY, with fine- to coarse-grained sands and some fine- to coarse-grained gravels (rock & brick fragments) (FILL)		Brown	Loose to Moderate Compaction	CL	1–2		15	17.9	108.8		
SILTY CLAY (FILL)		Light Gray & Brown mottled	Loose to Moderate Compaction	CL	4–6	Elev. +2.74	10	22.4	101.3	2.0	
FAT CLAY, with organics, (Bay Mud)		Bluish Gray	Soft	CH	7	Elev. +1.74					
(*1 Test: Liquid limit = 73%, Plastic Limit = 32%, PI = 41%)					11	*1	4	81.6	52.6		99.8
SILTY CLAY, locally with pockets of fine-grained sands and fine-grained gravels		Greenish Gray	Stiff	CL	13	Elev -4.26					
					16		15	23.0	102.3	1.5	
(grading significantly siltier below 18 feet; locally with clayey) sand lenses)		Greenish Gray & Brown mottled			18–20						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

* AC: Asphaltic Concrete, AB: Aggregate Base

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-2	1 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 8.74 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 6 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/13/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
SILTY CLAY (continued)		Greenish Gray & Brown mottled	Stiff	CL							
(*1 Test: Liquid limit = 32%, Plastic Limit = 21%, PI = 11%)					21	*1	12	27.2	96.8		99.0
					22						
(Grading with significantly less silts, but more fine-grained sands with depth below 22 feet)					23						
		Light Brown & some white mottled			24						
					25						
					26		8	27.1	96.3	0.6-1.1	
					27						
					28						
					29						
SANDY CLAY, fine- to coarse-grained sands		Brown	Stiff	CL	30						
(*2 Test: Liquid limit = 32%, Plastic Limit = 17%, PI = 15%)					31	*2 *3	11	26.2	99.5		
(*3 Test: Initial void ratio = 0.726, maximum consolidation pressure = ± 2,150 psf)					32						
					33						
					34						
					35						
(with a 6" layer of fine- to coarse-grained sand and trace of sub-angular coarse-grained gravels below 35½ feet)					36		9	22.1	104.2		
					37						
					38	Elev -29.26					
LEAN CLAY, with some fine-grained sands		Bluish Gray	Very Stiff	CL-CH	39						
					40						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-2	2 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 8.74 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 6 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/13/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
LEAN CLAY (continued)		Light Bluish Gray	Very Stiff	CL-CH	41		23	28.1	94.5	2.6	
					42						
					43						
					44						
					45						
SANDY SILT to SILTY SAND, fine-grained		Light Gray	Medium Dense	ML-SM	46		16	27.7	94.7		
BOTTOM OF BORING = ±46.5 FEET					47						
					48						
					49						
					50						
					51						
					52						
					53						
					54						
					55						
					56						
					57						
					58						
					59						
					60						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-2	3 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	**SURFACE ELEVATION:** 8.07 feet (±)	**LOGGED BY:** Billy Lin
GROUNDWATER: 5 feet below existing ground surface	**BORING DIAMETER:** 8 inches	**DATE DRILLED:** 10/14/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
PAVEMENT (1" AC* over 3" AB*)		--	--	--							
SILTY CLAY, with significant amount of fine- to coarse-grained sands, and some fine-grained gravels		Dark Grayish Brown	Moderate Compaciton	CL	1		29	6.4	124.7		
FILL					2						
					3						
(grading very silty below 3.5 feet)		Brown	Loose Compaction		4						
					5			▼	Elev. +3.07		
(*1 Test: Liquid limit = 31%, Plastic Limit = 20%, PI = 11%)					6	*1	7	27.0	90.6		84.6
					7	Elev. +1.07					
FAT CLAY, with organics, **(Bay Mud)**		Bluish Gray	Soft	CH	8						
					9						
					10						
(*2 Test: Liquid limit = 72%, Plastic Limit = 30%, PI = 42%)					11	*2	4	83.1	51.9		99.7
					12	Elev -3.93					
CLEAN SAND, fine- to coarse-grained, fine gravelly, with some silts; poorly graded		Gray	Loose	SP	13						
		Gray & Brown mottled	Medium Dense		14						
					15						
(*3 Test: 72% fine to coarse sands, 18% fine gravels, and 10% silts)					16	*3	17	15.1	115.9		89.7
					17						
					18						
					19						
					20						

Remark:

▼ Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

* AC: Asphaltic Concrete, AB: Aggregate Base

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-3	1 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 8.07 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 5 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/14/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
CLEAN SAND (continued)		Gray & Brown mottled	Medium Dense	SP	21		20	15.4	114.2		
					22						
					23						
(grading with no gravel below 24 feet)		Brown			24						
					25						
(*1 Test: 97% fine to medium sands, 3% fines)		Greenish Gray	Loose		26	*1	2	20.2	105.4		91.0
					27						
(grading fine gravelly again below 28 feet)		Brown	Medium Dense		28						
					29						
					30						
					31		22	17.7	114.6		
					32						
(grading to fine-grained **CLEAN SAND** below 33 feet)					33						
					34						
					35						
			Elev -27.93		36		11	17.4	110.7		
LEAN CLAY, with some fine-grained sands		Bluish Gray	Very Stiff	CL-CH	37						
					38						
					39						
					40						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-3	2 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 8.07 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 5 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/14/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
LEAN CLAY (continued)		Bluish Gray	Very Stiff	CL-CH							
					41		20	26.0	97.5	2.3	
					42						
					43						
					44						
					45						
					46		13	32.6	88.3		
BOTTOM OF BORING = ±46.5 FEET					47						
					48						
					49						
					50						
					51						
					52						
					53						
					54						
					55						
					56						
					57						
					58						
					59						
					60						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-3	3 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	**SURFACE ELEVATION:** 6.50 feet (±)	**LOGGED BY:** Billy Lin
GROUNDWATER: 6 feet below existing ground surface	**BORING DIAMETER:** 8 inches	**DATE DRILLED:** 10/14/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
PAVEMENT (2" AC over 3" AB)											
SILTY CLAY, with significant amount of fine-grained sands, and with trace of fine-grained gravels (broken rock fragments) FILL		Dark Grayish	Moderate Compaciton	CL	1						
					2	▯	21	13.3	116.8		
					3						
					4	Elev. +2.50					
FAT CLAY, with organics, desiccated, (Bay Mud)		Light Bluish Gray	Firm	CH	5						
(*1 Test: Liquid limit = 76%, Plastic Limit = 31%, PI = 45%)					6	*1	6	57.9	64.4	0.5	96.5
					7	Elev. +0.50					
					8	Elev -1.50					
SILTY CLAY		Bluish Gray	Stiff	CL	9						
					10						
					11	▯	14	24.0	99.2	1.7	
					12						
					13						
					14						
SANDY CLAY, fine- to coarse-grained sands		Light Bluish Gray	Firm to Stiff	CL	15						
(*2 Test: 28% fine to coarse sands, 2% fine gravels, and 70% silts & clays) (grading sandier below 16 feet)					16	*2	9	26.8	97.8	0.8	99.6
					17						
SILTY CLAY		Light Gray & Brown mottled	Stiff	CL	18						
					19						
					20	▯					

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

* AC: Asphaltic Concrete, AB: Aggregate Base

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-4	1 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	**SURFACE ELEVATION:** 6.50 feet (±)	**LOGGED BY:** Billy Lin
GROUNDWATER: 6 feet below existing ground surface	**BORING DIAMETER:** 8 inches	**DATE DRILLED:** 10/14/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
SILTY CLAY (continued)		Light Gray & Brown mottled	Stiff	CL	21		15	24.8	99.8	1.7	
					22						
					23						
(grading with some fine-grained sands below 23½ feet; locally with lenses and pockets of fine-grained sands)		Light Brown Reddish Brown mottled	Firm		24						
					25						
					26		6	25.1	101.7		
					27	Elev -20.50					
CLEAN TO SILTY SAND, fine to coarse grained; locally with sandy clay pockets		Greenish Gray	Loose	SP-SM	28						
					29						
					30						
(*1 Test: 74% fine to coarse sands, 2% fine gravels, and 24% silts & clays)					31	*1	8	24.1	102.1		99.8
					32						
					33						
					34						
					35						
(grading with fine-grained, round, gravels also below 35 feet)			Elev -29.50		36		9	32.3	86.2		
LEAN CLAY, with some fine-grained sands		Light Bluish Gray	Stiff	CL-CH	37						
					38						
					39						
					40						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-4	2 of 3

DRILL RIG: Truck-Mounted CME-75 with continuous hollow-stem flight augers	SURFACE ELEVATION: 6.50 feet (±)	LOGGED BY: Billy Lin
GROUNDWATER: 6 feet below existing ground surface	BORING DIAMETER: 8 inches	DATE DRILLED: 10/14/2005

DESCRIPTION	Symbol	Color	Consistency	Soil Type	Depth (feet)	Sampler	Penetration Resistance (blows/foot)	In-Place Moisture Content (%)	In-Place Dry Density (pcf)	Pocket Penetrometer Strength (tsf)	Saturation (%)
LEAN CLAY (continued)		Light Bluish Gray	Stiff	CL-CH							
					41		13	30.9	90.1	2.0	
					42						
					43						
					44						
					45						
					46		12	35.7	81.7	0.8	
BOTTOM OF BORING = ±46.5 FEET					47						
					48						
					49						
					50						
					51						
					52						
					53						
					54						
					55						
					56						
					57						
					58						
					59						
					60						

Remark:

Groundwater Table

California Split Spoon Sampler, 2-inch inside diameter

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EXPLORATORY BORING LOGS

HOTEL/CONDOMINIUM COMPLEX
1700 EMBARCADERO ROAD, PALO ALTO, CA

Project Number	Date	Boring Number	Page No.
267-02	11/16/2005	EB-4	3 of 3

DRILL TYPE: Mobile Drill B-53 with 8" Hollow Stem Auger **LOGGED BY:** TWP

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 11/12/13

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
2-inches of asphalt concrete over 4-inches of baserock.				0					
Artificial Fill: Brown, Clayey Sand/Sandy Lean Clay, moist, fine to coarse sand, fine to course gravel, low plasticity. ■ Liquid Limit = 33, Plasticity Index = 16. ● 51% Passing No. 200 Sieve. Becoming dark grayish brown, moderate plasticity.	Dense/ Hard	SC/ CL		5		45	13		
Young Bay Mud: Blue gray, Fat Clay, wet, fine sand, high plasticity. ■ Liquid Limit = 88, Plasticity Index = 40. ● 91% Passing No. 200 Sieve. ▼ Ground water measured at 9.5 feet shortly after drilling.	Soft	CH		▼ 10		3	62		
Gray, Clayey Sand, moist, fine to medium sand. ■ Liquid Limit = 22, Plasticity Index = 8. ● 28% Passing No. 200 Sieve.	Medium Dense	SC				10			
Bottom of Boring at 14.5 feet. Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.				15 20					

EXPLORATORY BORING LOG EB-5 **BORING EB-5**

WU-CHUNG HOTEL COMPLEX DECEMBER 2013

PALO ALTO, CALIFORNIA PROJECT NO. 2317-1A

DRILL TYPE: Mobile Drill B-53 with 8" Hollow Stem Auger **LOGGED BY:** TWP

DEPTH TO GROUND WATER: 9.5 Feet. **SURFACE ELEVATION:** NA **DATE DRILLED:** 11/12/13

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
2-inches of asphalt concrete over 4-inches of baserock.				0					
Artificial Fill: Brown, Sandy Lean Clay, very moist, fine to medium sand, fine gravel, moderate plasticity, gray mottling. ■ Liquid Limit = 38, Plasticity Index = 21. ● 56% Passing No. 200 Sieve.	Very Stiff	CL		5		21	28		
Young Bay Mud: Blue gray, Fat Clay, wet, fine sand, high plasticity. ■ Liquid Limit = 89, Plasticity Index = 47. ● 95% Passing No. 200 Sieve. ▼ Ground water measured at 14 feet shortly after drilling. Transitioning from gray to brown.	Soft	CH		10, ▼, 15		3	81		
Brown, Sandy Fat Clay, very moist, fine to medium sand, moderate to high plasticity.	Very Stiff	CL/ CH				18	23		
Bottom of Boring at 18 feet. Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.				20					

EXPLORATORY BORING LOG EB-6
WU-CHUNG HOTEL COMPLEX
PALO ALTO, CALIFORNIA

BORING EB-6
DECEMBER 2013
PROJECT NO. 2317-1A

DRILL TYPE: Mobile Drill B-53 with 8" Hollow Stem Auger **LOGGED BY:** TWP

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 11/12/13

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
2-inches of asphalt concrete over 4-inches of baserock.				0					
Artificial Fill: Brown, Sandy Lean Clay with gravel, moist, fine to coarse sand, fine to course gravel, low plasticity. ■ Liquid Limit = 27, Plasticity Index = 12. ● 61% Passing No. 200 Sieve. Becoming dark brown to black, Sandy Lean Clay, moist, fine to medium sand, low to moderate plasticity. ■ Liquid Limit = 43, Plasticity Index = 25. ● 71% Passing No. 200 Sieve.	Very Stiff to Hard	CL		5		50/5"			
Brown, Sandy Lean Clay/Sandy Fat Clay, moist, fine to medium sand, moderate to high plasticity (Bay Mud crust?).	Stiff	CL/ CH		10		15			
Gray, Sandy Fat Clay, very moist, fine sand, high plasticity (possible stiffer area of Bay Mud). ■ Liquid Limit = 53, Plasticity Index = 32. ● 66% Passing No. 200 Sieve.	Very Stiff	CH				23	32		
Bottom of Boring at 13 feet. Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.				15 20					

EXPLORATORY BORING LOG EB-7
WU-CHUNG HOTEL COMPLEX
PALO ALTO, CALIFORNIA

BORING EB-7
DECEMBER 2013
PROJECT NO. 2317-1A

DRILL TYPE: Mobile Drill B-53 with 8" Hollow Stem Auger **LOGGED BY:** TWP

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** NA **DATE DRILLED:** 11/12/13

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
2-inches of asphalt concrete over 4-inches of baserock.				0					
Artificial Fill: Brown, Sandy Lean Clay with gravel, moist, fine to coarse sand, fine gravel, moderate plasticity. ■ Liquid Limit = 47, Plasticity Index = 29. ● 57% Passing No. 200 Sieve.	Stiff	CL				11			
Young Bay Mud: Blue gray, Fat Clay, wet, fine sand, high plasticity. ■ Liquid Limit = 81, Plasticity Index = 40. ● 92% Passing No. 200 Sieve. ▼ Ground water measured at 10 feet shortly after drilling.	Soft	CH		5		4	78		
Brown and gray, Sandy Lean Clay/Sandy Fat Clay, very moist, fine sand, moderate plasticity.	Stiff	CL/ CH		15		14	25		
Bottom of Boring at 15.5 feet.				20					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-8
WU-CHUNG HOTEL COMPLEX
PALO ALTO, CALIFORNIA

BORING EB-8
DECEMBER 2013
PROJECT NO. 2317-1A

CALIFORNIA PUSH
TECHNOLOGIES
INCORPORATED

California Push Technologies Inc.
104 Constitution Drive Suite 2
Menlo Park CA 94025
(C-57 License #884827)

office: 650 854 0300
fax: 650 854 0301

www.cptinc.com

Cone penetration testing and soil sampling methods description.

Rig Description

Our services are based on the state-of-the-art, Geoprobe Model 6625CPT rig, a limited-access, self-anchoring, 20-ton push capacity, track-mounted push platform for dedicated Geotechnical CPT applications with the unique and valuable added ability to quickly perform intermittent or continuous soil sampling.

Weight = ~ 9,500 pounds
Surface load = ~ 4.5 psi
Push capacity = ~ 20 tons; self-anchoring achieved using 10- or 15-inch diameter helical soil anchors driven 4- to 10-feet into the soil
Sampling hammer percussion rate = 32 Hz & 20,000 lbs force/blow
Length = ~ 12 feet; Width = ~ 7 feet
Height (folded) = 7 feet; Height (unfolded) = 14 feet

CPT Description

Our Geoprobe 6625CPT incorporates the Swedish-made Geotech AB Cone Penetration Testing tools which meet the ASTM D-5778 Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils. Cones have 10 cm^2 tips and 150 cm^2 friction sleeves, and include a porous filter and pressure sensor located in the u_2 position directly behind the cone. The cone and porous filter are saturated under vacuum with glycerin to promote rapid equilibration with in-situ pore pressures. Cones are advanced at the ASTM standard rate of 2 cm/second. Baseline readings are performed both before and after each push to check for load cell drift. The cone measures bearing (max load = 100 MPa ~ 1044 TSF), friction sleeve (max load = 1.0 MPa ~ 10.4 TSF), and dynamic pore pressure (max load = 2.5 MPa ~ 363 psi) at 2 cm or 4 cm intervals (client's choice) and this data is plotted in real-time and recorded on a laptop computer adjacent to the push platform. Holes are grouted upon completion of each push, or at the end of each day, as site conditions and regulations warrant.

The basic equation to determine the depth to the free water surface from the pore pressure dissipation test is;

Depth to phreatic surface = [Dissipation depth] – [equilibrium pore pressure / unit weight of H2O x unit conversation factor]

…where;

1) Surface elevation is always assumed to be 0 feet
2) Dissipation depth = the depth (feet) below surface elevation where the cone advancement was paused while waiting for equilibrium pore pressure to be achieved
3) Equilibrium pore pressure = the pore pressure after an elapsed time where no increase or decrease in pore pressure is occurring, in pounds per square inch (psi)
4) Unit weight of water = 62.3 pounds per cubic foot (lb/ft^3)
5) Unit conversion factor (for dimensional analysis): 1 psi = 144 lb/ft^3

From the dissipation plots, simply read the dissipation depth and dissipated pressure for the values to plug into the equation above. On the plots, pore pressure (psi) is on the abscissa and log time (seconds) is on the ordinate.

Sampling Description

Geoprobe® brand Dual Tube Sampling Systems are efficient methods of collecting continuous soil cores with the added benefit of a cased hole. Dual tube sampling uses two sets of probe rods to collect continuous soil cores. One set of rods is driven into the ground as an outer casing (2.2 or 3.25 inches in diameter). These rods receive the driving force from the hammer and provide a sealed hole from which soil samples may be recovered without the threat of cross contamination. The second, smaller set of rods are placed inside the outer casing. The smaller rods hold a sample liner in place as the outer casing is driven one sampling interval. The small rods are then retracted to retrieve the filled liner. Soil samples are collected in 1.85-inch diameter or 1.125-inch diameter clear PVC sample sheaths.

Interpretations

Soil behavior type (SBT), SPT N60 energy ratio, undrained shear strength, OCR, and unit weights are calculated and/or are interpretations generated by the CPT-Pro software based on empirical relationships derived in the following references;

P.K. Robertson, R.G. Campanella, D. Gillespie, and J. Greig, 1986, Use of Piezometer Cone Data, Proceedings of the ASCE Specialty Conference In Situ '86: Use of In Situ Tests in Geotechnical Engineering; pp. 1263-1280.

P.K. Roberston, 1990, Soil Classification Using the Cone Penetration Test, Canadian Geotechnical Journal, 27(1), pp. 151-158.

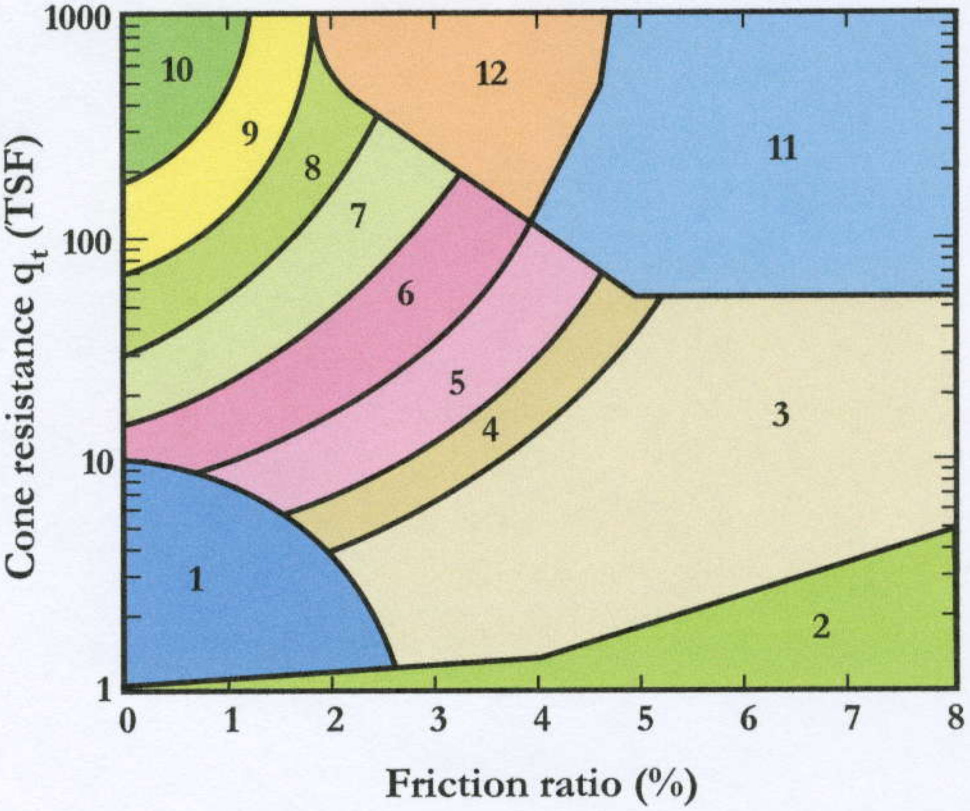
T. Lunne, P.K. Robertson, and J.J.M. Powell, 1997, Cone Penetration in Geotechnical Practice, Taylor and Francis Publishing.

CPT Inc. makes no recommendation on which soil behavior type analysis is "most-correct". The engineer should be aware of the limitations of using CPT data to derive soil behavior type and other engineering parameters and is encouraged to review the above references to better understand the applicability and limitations of CPT data. It is sometimes not possible to determine soil type based solely on tip resistance, sleeve friction, and dynamic pore pressure response, and confirmatory samples may be required.

Please do not hesitate to contact CPT Inc. if you have questions.

Sincerely,
John Rogie

President
California Push Technologies, Inc.



Zone	Soil Behavior Type
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 (overconsolidated or cemented)
12	sand to clayey sand (overconsolidated or cemented)

Source: Robertson, P.K., Campanella, R.G., Gillespie, D., and Greig, J., 1986, Use of Piezometer Cone Data. Proceedings of the ASCE Specialty Conference In Situ 86: Use of In Situ Tests in Geotechnical Engineering.

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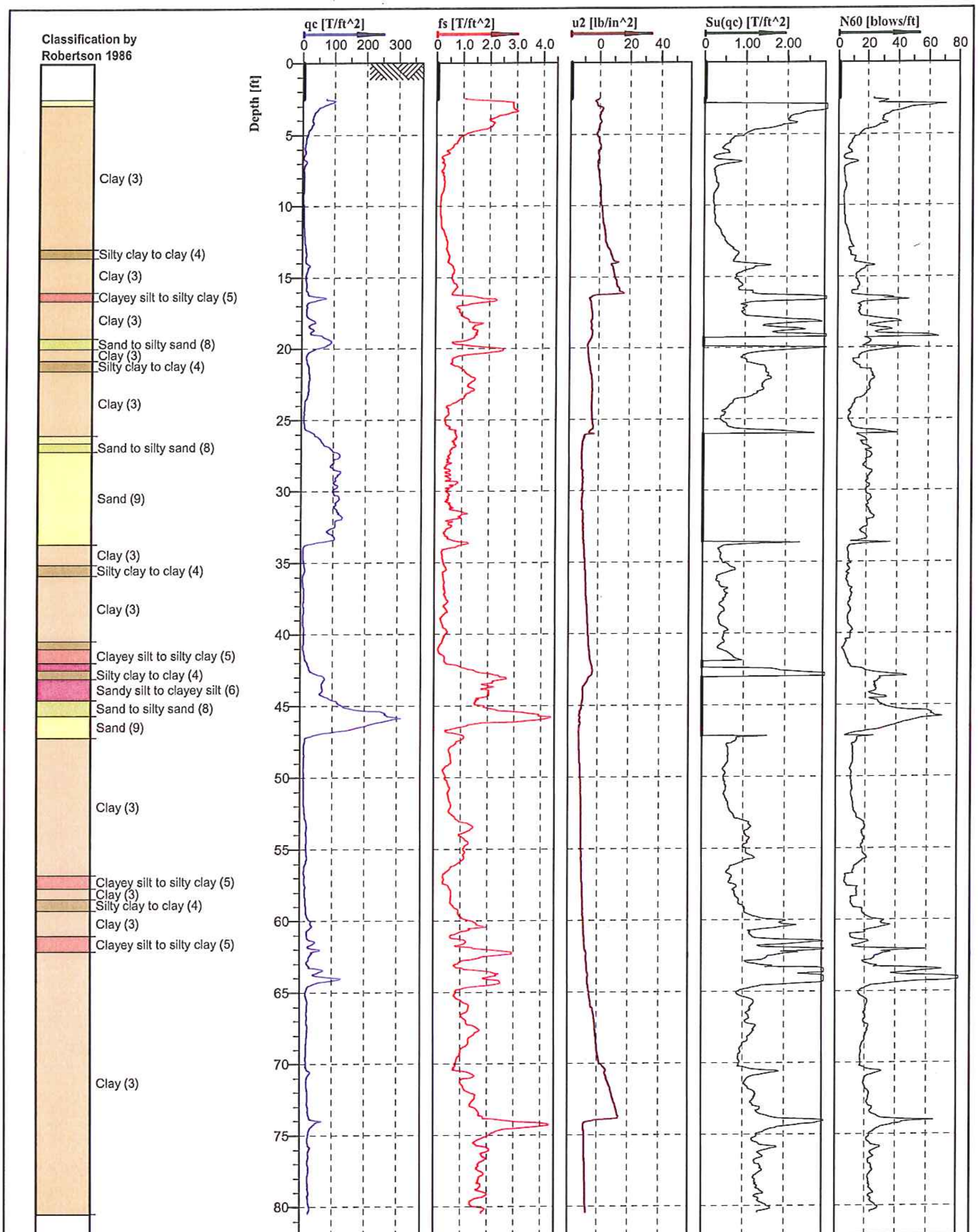
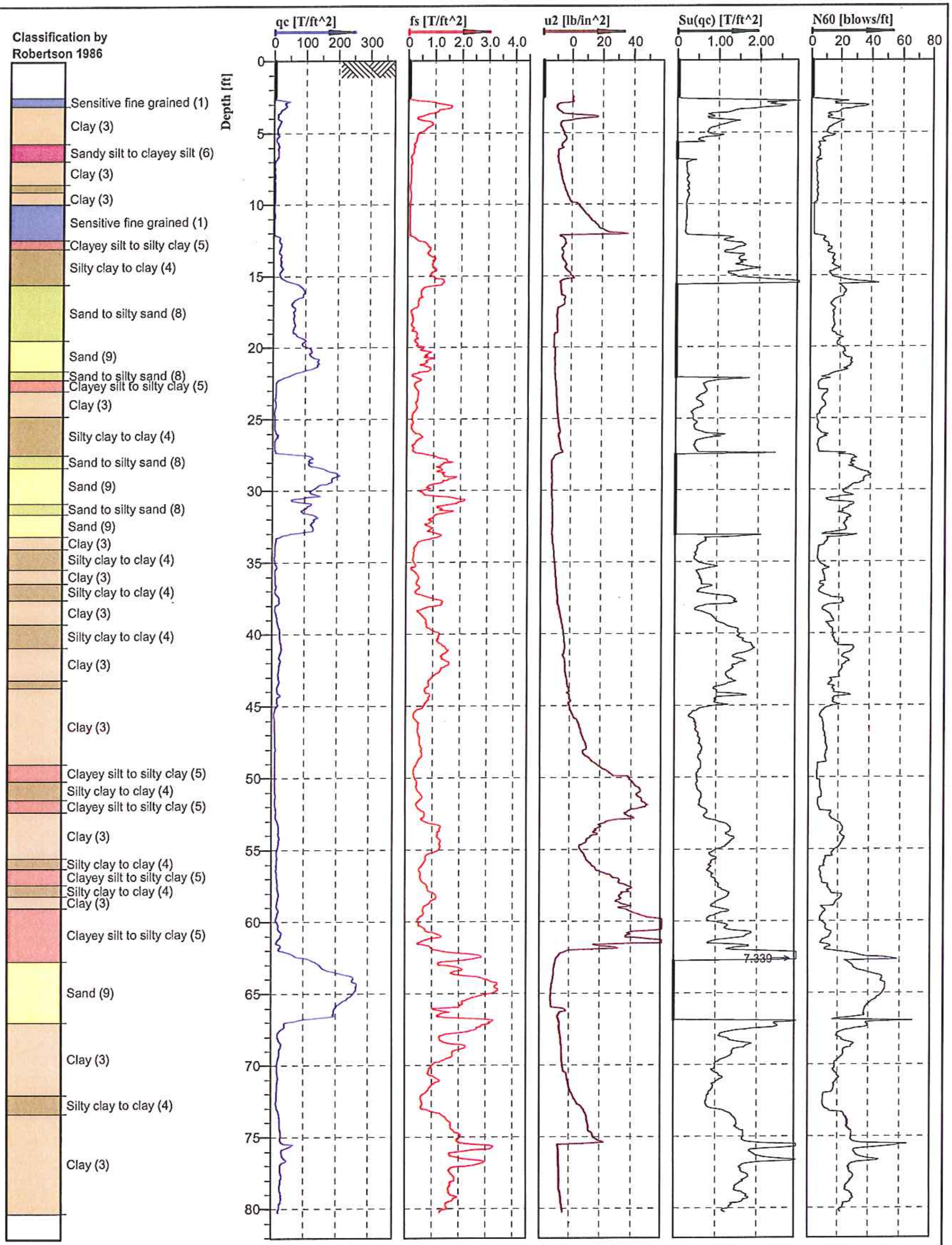
Soil Behavior Type (SBT) Model



Location: Palo Alto, California	Position:	Ground level:	Test no: CPT-1
Project ID: 2317-1	Client: Romig Engineers	Date: 6/25/2009	Scale: 1 : 115
Project: Wu-Chang Project		Page: 1/1	Fig:
		File: CPT-1.cpd	

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cone No: 3794
Tip area [cm2]: 10
Sleeve area [cm2]: 150

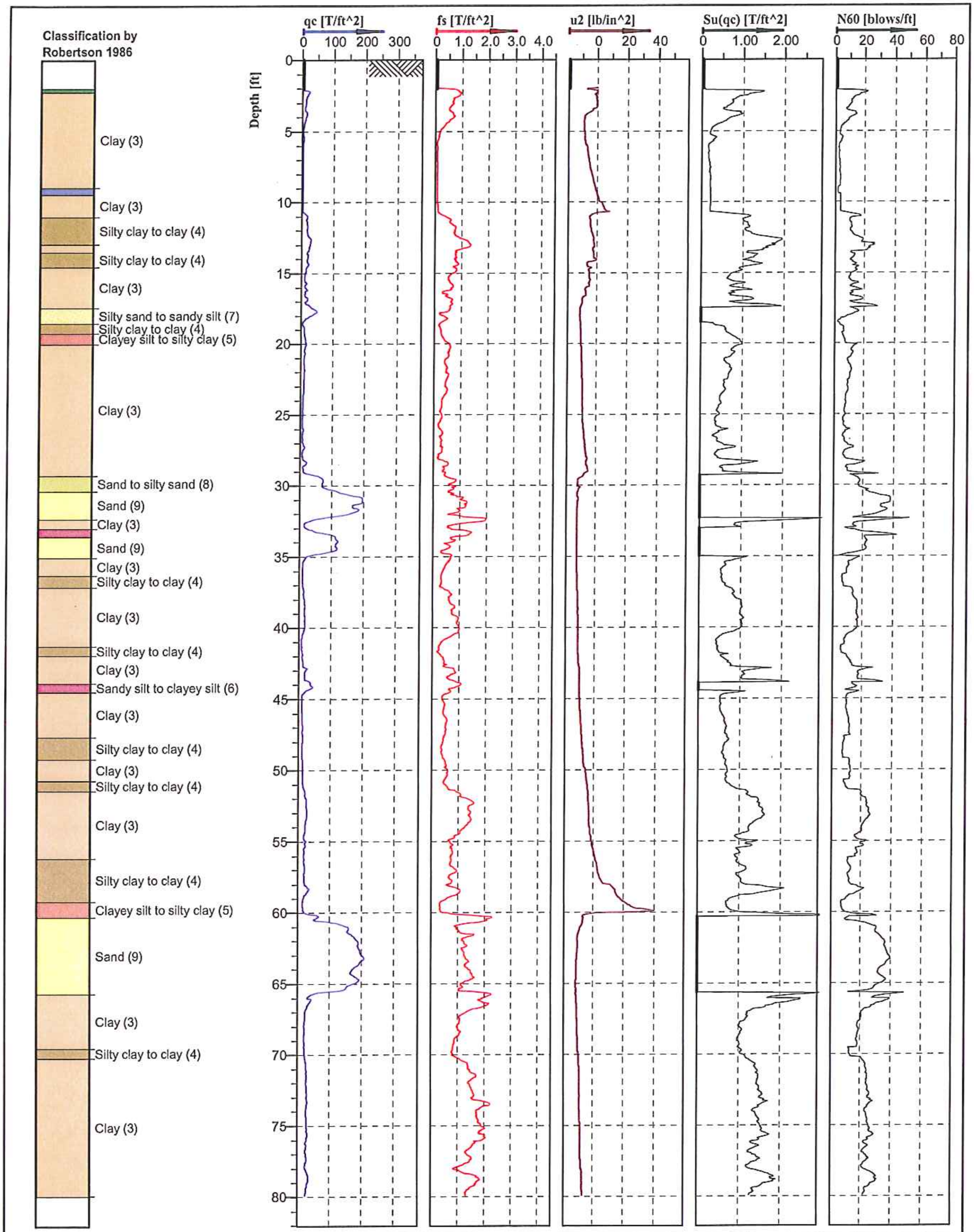


U2

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cone No: 3794
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Palo Alto, California	Position:	Ground level:	Test no: CPT-2
Project ID: 2317-1	Client: Romig Engineers	Date: 6/25/2009	Scale: 1 : 115
Project: Wu-Chang Project		Page: 1/1	Fig:
		File: CPT-2.cpd	



CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

U2

Cone No: 3794
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Palo Alto, California	Position:	Ground level:	Test no: CPT-3
Project ID: 2317-1	Client: Romig Engineers	Date: 6/25/2009	Scale: 1 : 115
Project: Wu-Chang Project		Page: 1/1	Fig:
		File: CPT-3.cpd	

DRILL TYPE: Mobile Drill B-53 with 7-1/4" Hollow Stem Auger **LOGGED BY:** JF

DEPTH TO GROUND WATER: Not Encountered. **SURFACE ELEVATION:** 8 ft **DATE DRILLED:** 09/02/14

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	SPT RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Fill: Brown, Sandy Lean Clay, moist, fine to medium grained sand, fine subangular gravel, low plasticity.	Stiff to Very Stiff	CL		0					
						25	11		
						14	18		4.5
				5		12	16		3.8
Wood debris encountered at 5.5 feet.									
Young Bay Mud: Dark Gray, Fat Clay, wet, fine grained sand, high plasticity.	Soft to Firm	CH				7	44		1.5
						3	42		0.8
						4	68		0.3
				10		6	65		
Bottom of Boring at 11 feet.									
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.				15					
*Measured using Torvane and Pocket Penetrometer devices.									

EXPLORATORY BORING LOG EB-1
AUDI PALO ALTO SHOWROOM
PALO ALTO, CA

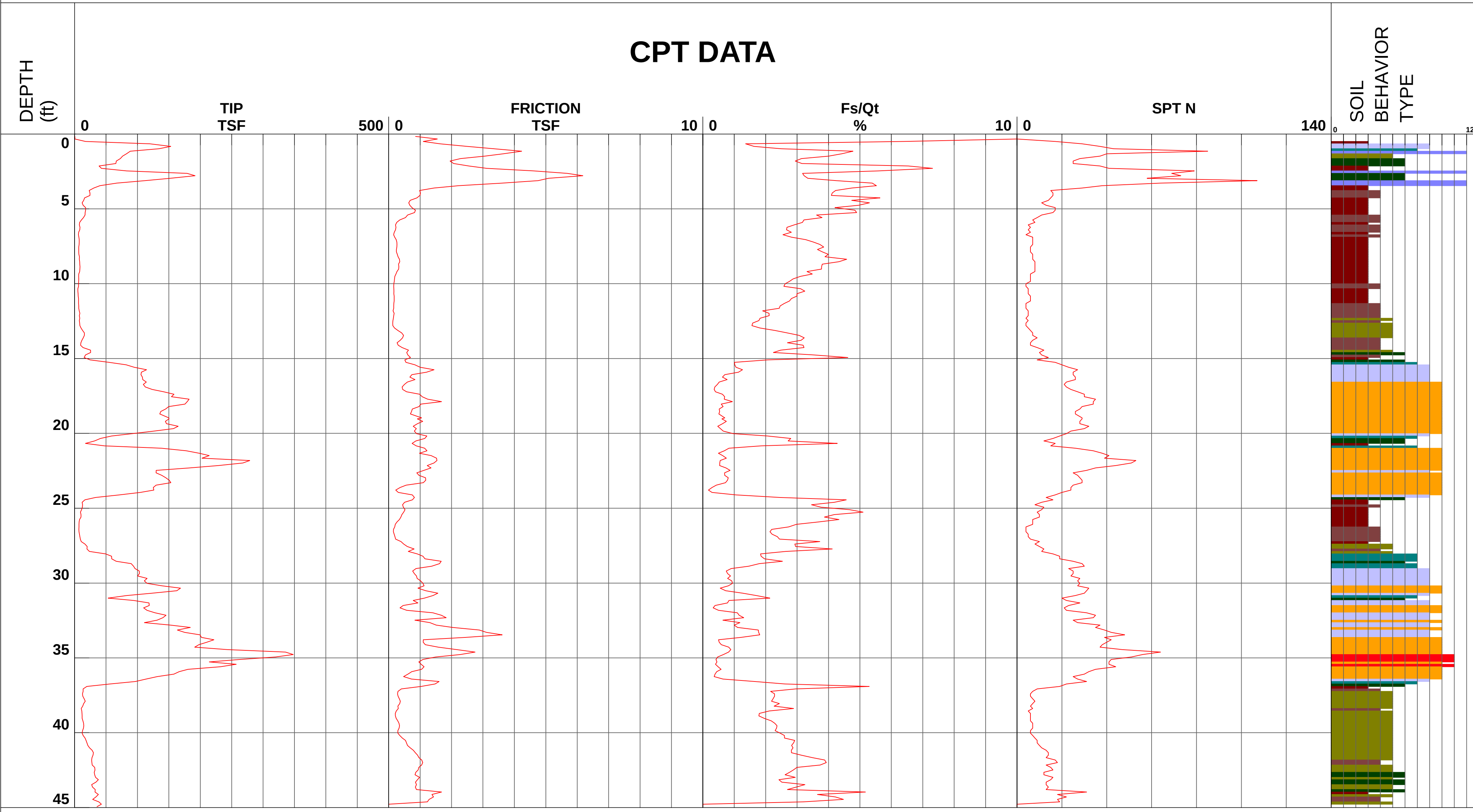
BORING EB-1
NOVEMBER 2014
PROJECT NO. 3247-1

Romig Engineers

Project	Audi Palo Alto Showroom	Operator	CB/MM	Filename	SDF(253).cpt
Job Number	3247-1	Cone Number	DDG1298	GPS	
Hole Number	CPT-01	Date and Time	9/2/2014 8:10:21 AM	Maximum Depth	44.95 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

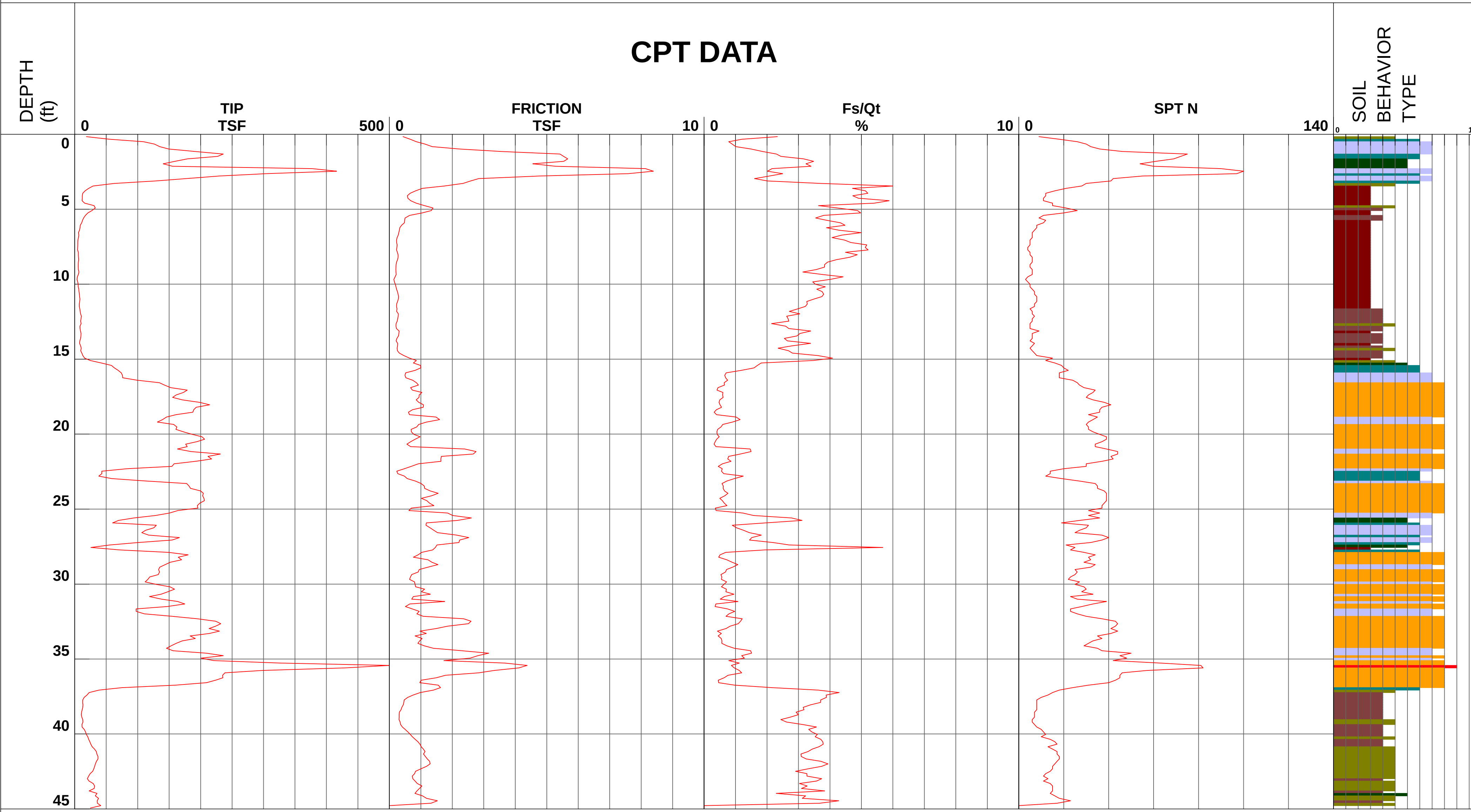
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Audi Palo Alto Showroom	Operator	CB/MM	Filename	SDF(254).cpt
Job Number	3247-1	Cone Number	DDG1298	GPS	
Hole Number	CPT-02	Date and Time	9/2/2014 8:57:47 AM	Maximum Depth	44.95 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8

CPT DATA



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 (*)

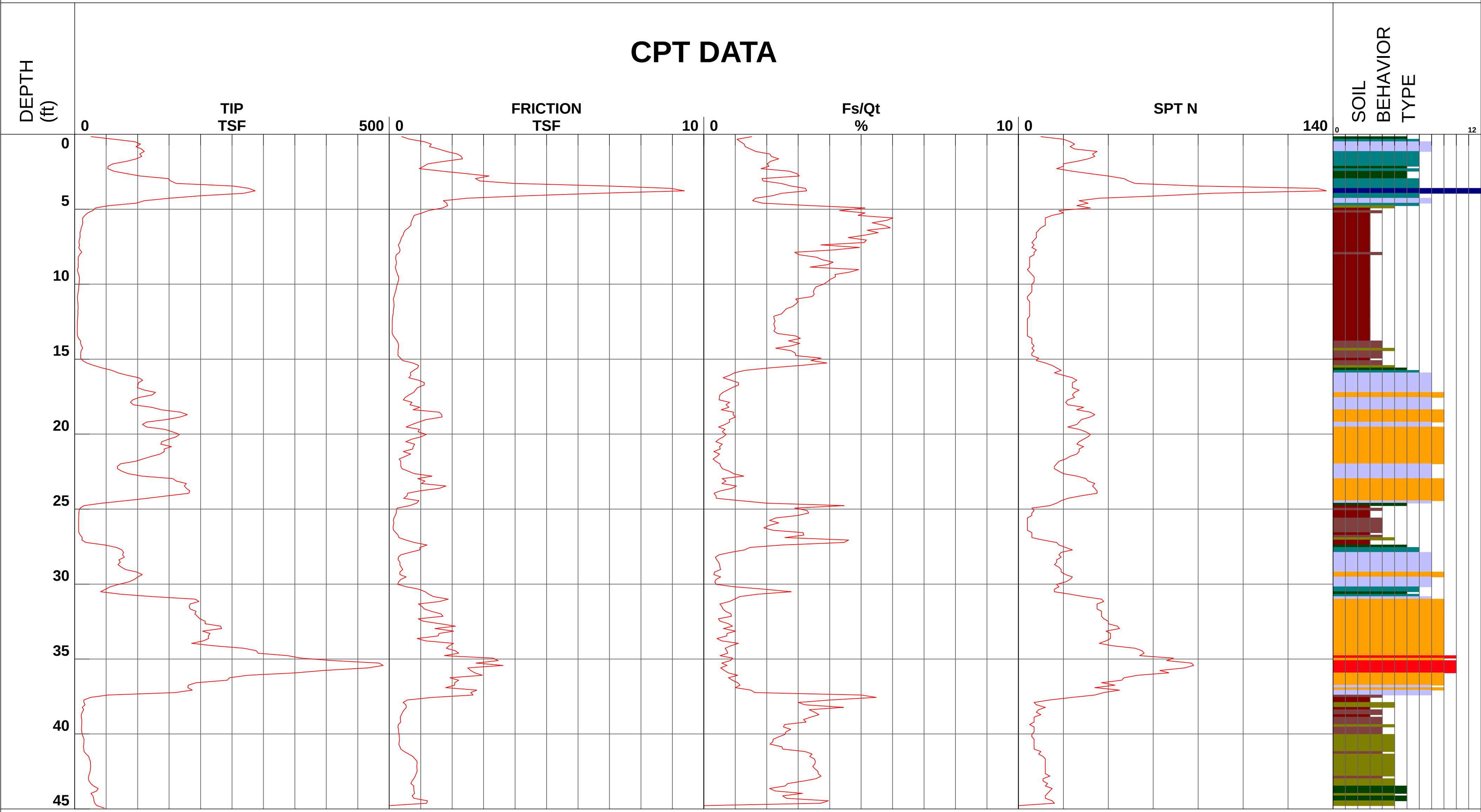
Cone Size 10cm squared

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

Romig Engineers

Project	Audi Palo Alto Showroom	Operator	CB/MM	Filename	SDF(255).cpt
Job Number	3247-1	Cone Number	DDG1298	GPS	
Hole Number	CPT-03	Date and Time	9/2/2014 9:34:35 AM	Maximum Depth	44.95 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8



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 (*)

Cone Size 10cm squared

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

APPENDIX B

SUMMARY OF LABORATORY TEST RESULTS

Samples from the subsurface explorations were selected for tests to help evaluate the physical and engineering properties of the soils. The tests performed are briefly described below.

The natural moisture content was determined in accordance with ASTM D2216 on selected samples recovered from the borings. This test determines the moisture content, representative of field conditions, at the time the samples were collected. The results are presented on the boring logs at the appropriate sample depths.

The Atterberg Limits were determined on eleven samples in accordance with ASTM D4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The results of these tests are presented in Figure B-1, B-2, and B-3 and on the boring logs at the appropriate sample depths.

The amount of silt and clay-sized material present was determined on two sample of soil in accordance with ASTM D422. The results are presented on the log of Boring EB-1 (2018) at the appropriate sample depths.

The particle size distribution was determined on ten samples of soil in accordance with ASTM D422. The results of these tests are presented in Figure B-4 and B-5 and on the boring logs at the appropriate sample depths.

The following corrosion potential tests were performed by Cooper Testing Laboratory on six samples of surface and near-surface soil from the site: resistivity, pH, chloride content, sulfate content, and Redox Potential (Oxidation/Reduction Potential). The test methods that were used and the results of these tests are included in this appendix.

❖ ❖ ❖ ❖ ❖

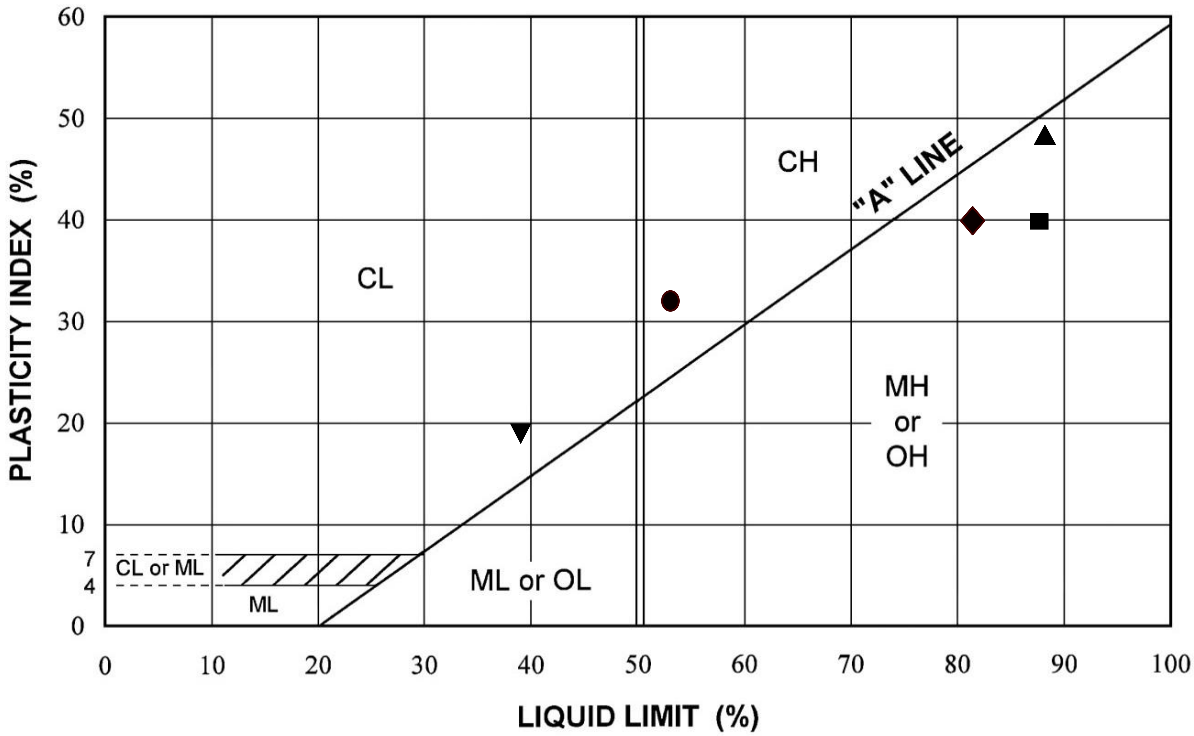
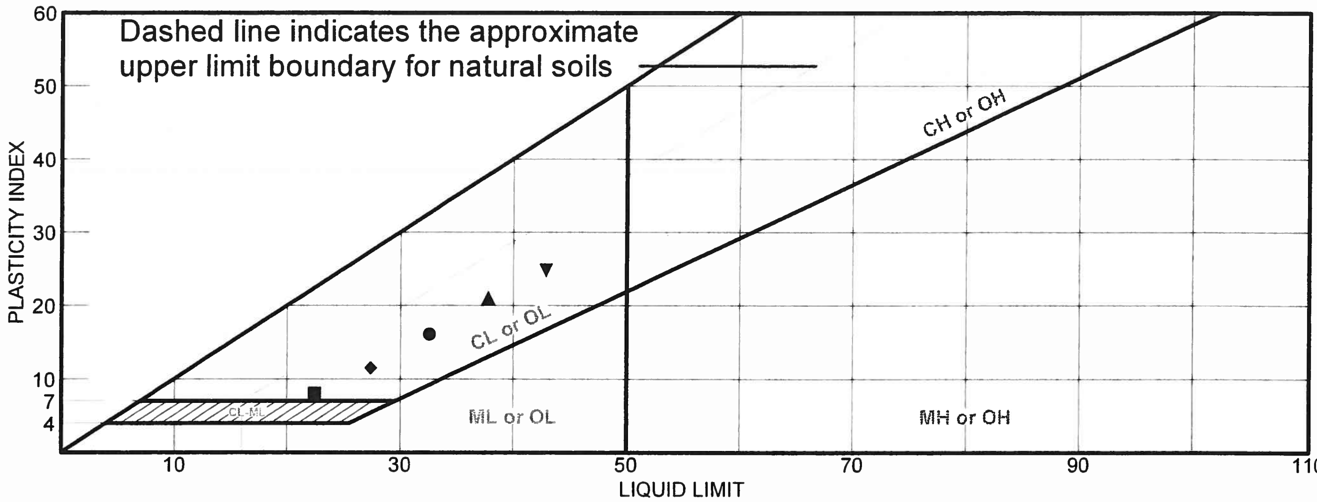


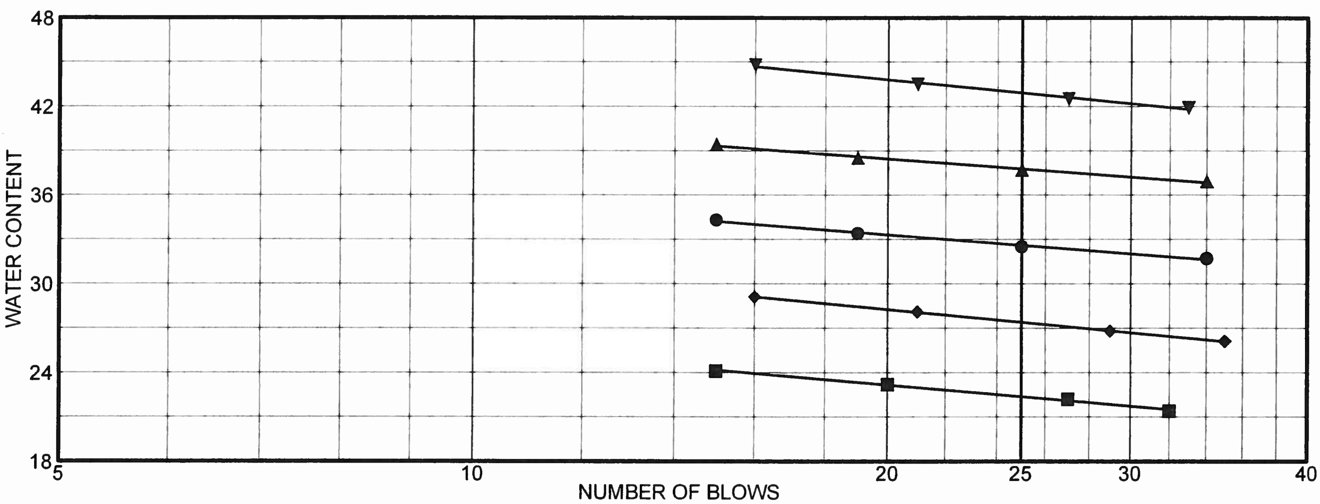
Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
▼	EB-1	3-4.5	13	39	19	-37		CL
■	EB-5 (2013)	6-6.5	62	88	40	35	91	CH/OH
▲	EB-6 (2013)	8.5-9	81	89	48	83	95	CH/OH
●	EB-7 (2013)	12-12.5	32	53	32	34	66	CH
◆	EB-8 (2013)	6-6.5	78	81	40	93	92	CH/OH

PLASTICITY CHART
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE B-1
MAY 2018
PROJECT NO. 4300-1

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Very Dark Brown Sandy Lean CLAY	32.6	16.5	16.1	76.5	51.2	CL
■	Very Dark Bluish Gray Lean Clayey SAND	22.4	14.4	8.0	86.7	27.8	SC
▲	Very Dark Brown Sandy Lean CLAY	37.8	16.8	21.0	74.8	55.7	CL
◆	Very Dark Brown Sandy Lean CLAY	27.4	15.9	11.5	83.7	61.3	CL
▼	Very Dark Brown Lean CLAY w/ Sand	42.9	18.1	24.8	84.5	70.8	CL

Project No. 192-170 **Client:** Romig Engineers, Inc.

Project: Wu-Chung Hotel Complex - 2317-1A

● **Source:** EB-5 **Elev./Depth:** 1.5-2'
■ **Source:** EB-5 **Elev./Depth:** 13.5-14'
▲ **Source:** EB-6 **Elev./Depth:** 2.5-4'
◆ **Source:** EB-7 **Elev./Depth:** 1.5-2'
▼ **Source:** EB-7 **Elev./Depth:** 6-6.5'

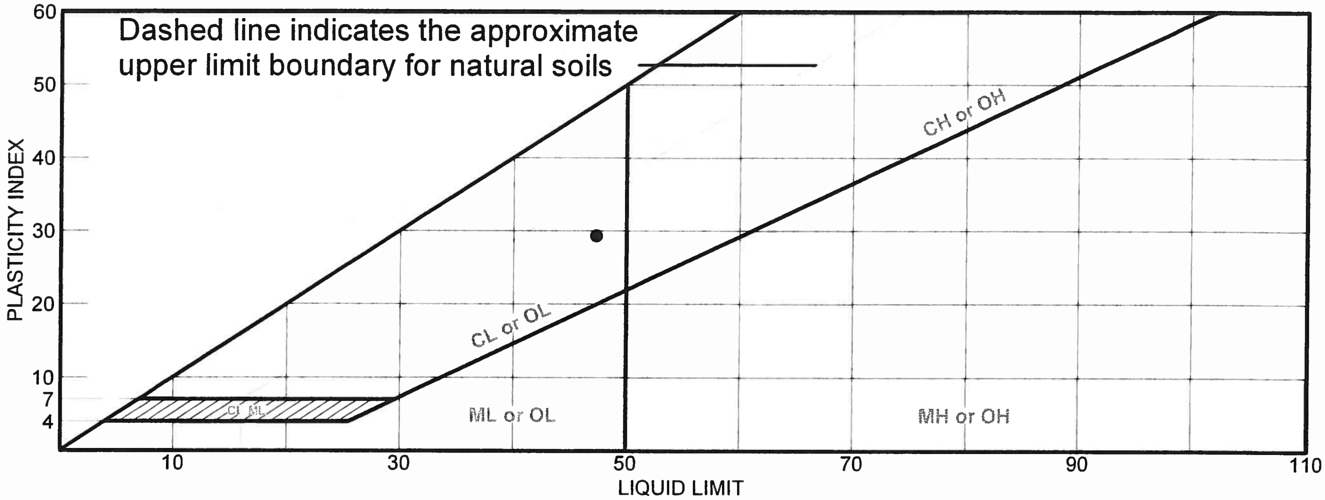
Remarks:
●
■
▲
◆
▼

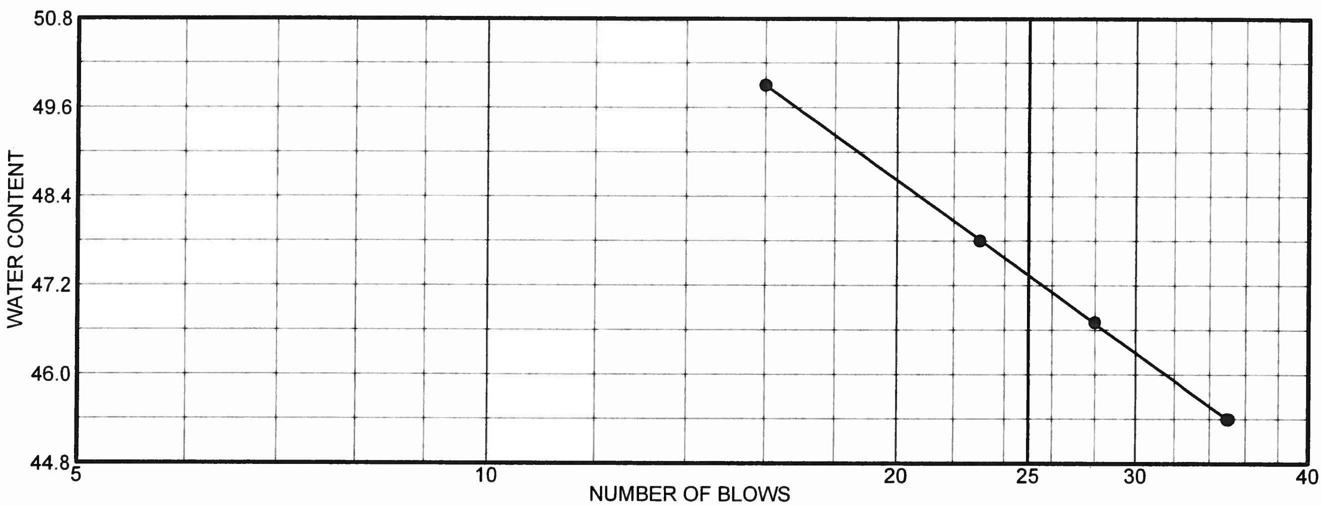
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure B-2

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Very Dark Brown Sandy Lean CLAY	47.3	18.0	29.3	70.4	56.8	CL

Project No. 192-170 **Client:** Romig Engineers, Inc.

Project: Wu-Chung Hotel Complex - 2317-1A

● **Source:** EB-8 **Elev./Depth:** 1.5-2'

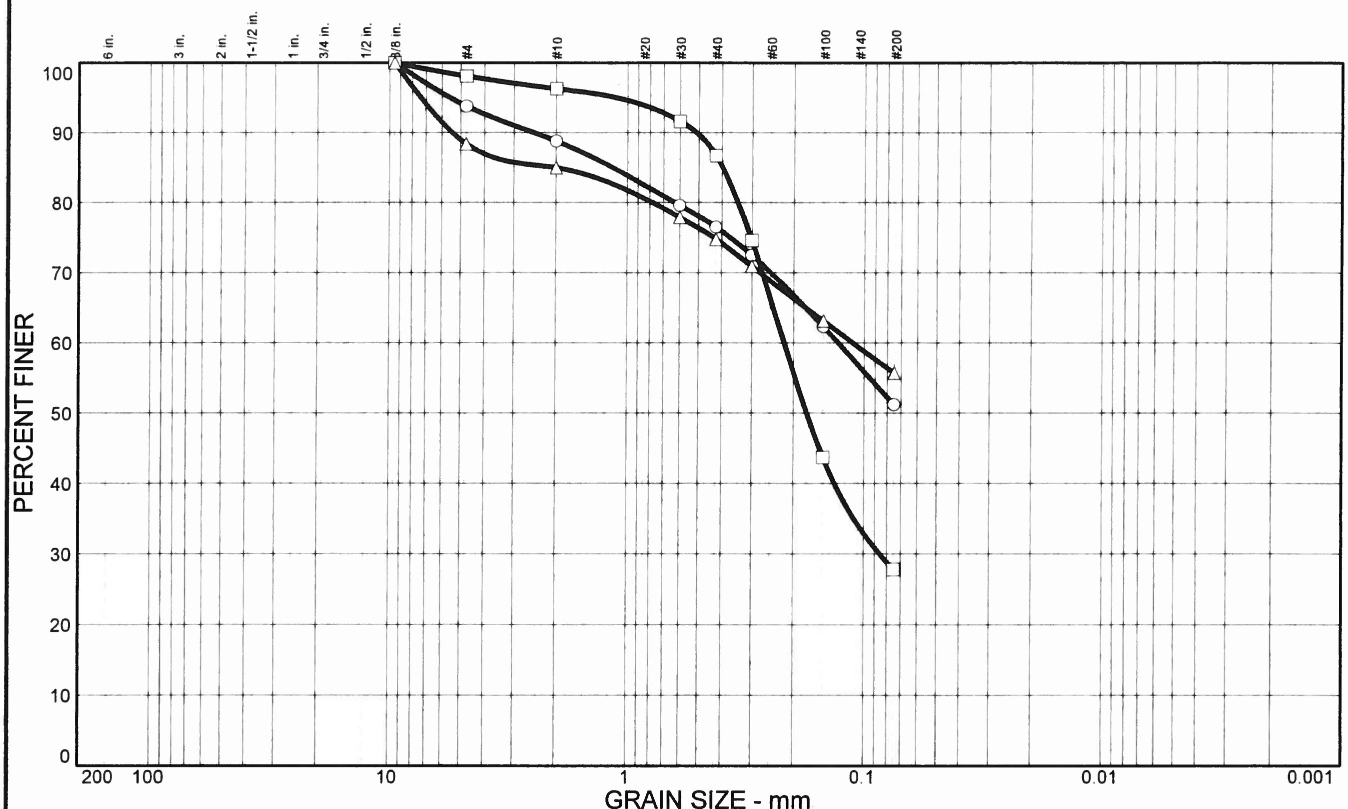
Remarks:
●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure B-3

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		6.2	42.6	51.2		CL		16.5	32.6
□		1.9	70.3	27.8		SC		14.4	22.4
△		11.6	32.7	55.7		CL		16.8	37.8

SIEVE inches size	PERCENT FINER ○	□	△
3/8"	100.0	100.0	100.0
	GRAIN SIZE		
D_{60}	0.130	0.219	0.113
D_{30}		0.0855	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	93.8	98.1	88.4
#10	88.8	96.3	85.0
#30	79.6	91.6	77.9
#40	76.5	86.7	74.8
#50	72.5	74.6	70.9
#100	62.3	43.7	63.1
#200	51.2	27.8	55.7

SOIL DESCRIPTION

○ Very Dark Brown Sandy Lean CLAY

□ Very Dark Bluish Gray Lean Clayey SAND

△ Very Dark Brown Sandy Lean CLAY

REMARKS:

○

□

△

○ Source: EB-5 Elev./Depth: 1.5-2'

□ Source: EB-5 Elev./Depth: 13.5-14'

△ Source: EB-6 Elev./Depth: 2.5-4'

COOPER TESTING LABORATORY

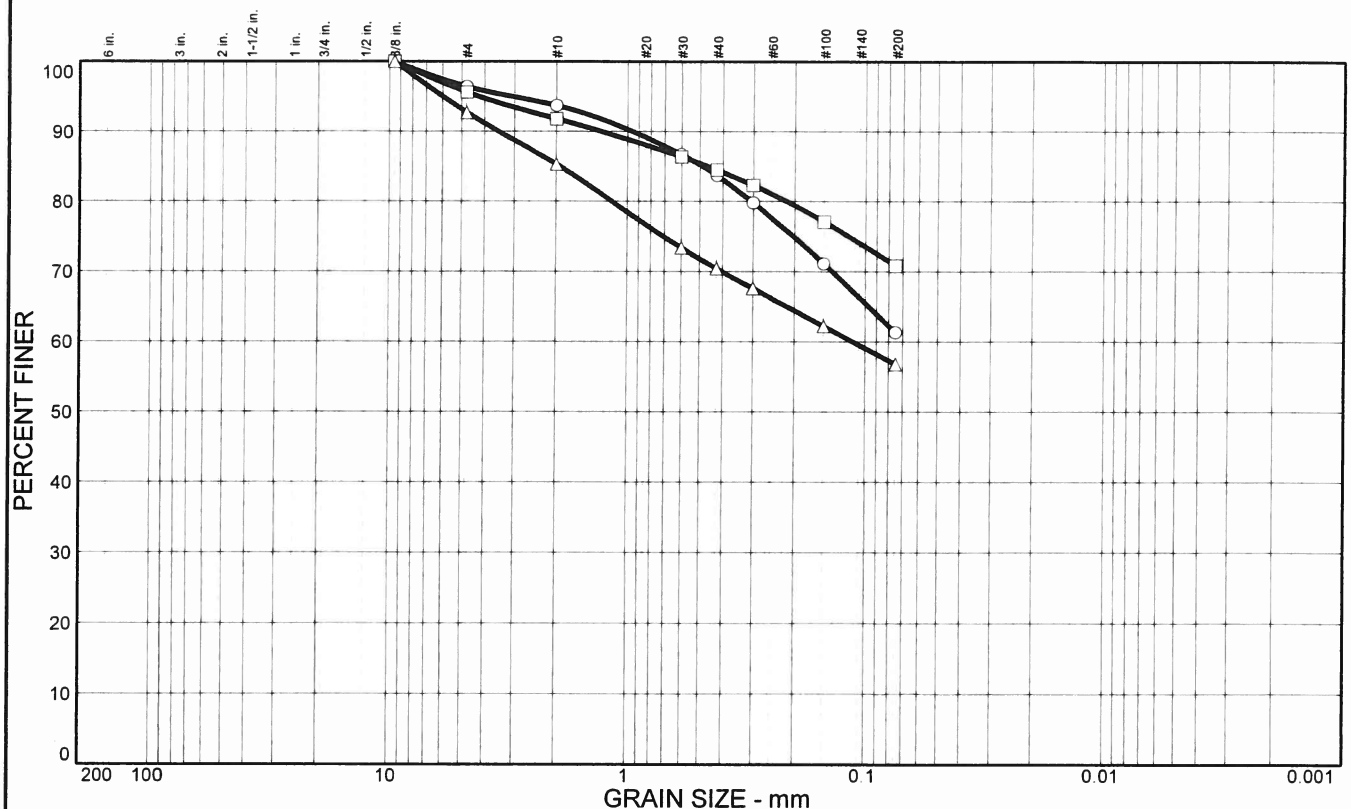
Client: Romig Engineers, Inc.

Project: Wu-Chung Hotel Complex - 2317-1A

Project No.: 192-170

Figure B-4

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		3.6	35.1	61.3		CL		15.9	27.4
□		4.4	24.8	70.8		CL		18.1	42.9
△		7.3	35.9	56.8		CL		18.0	47.3

SIEVE inches size	PERCENT FINER ○	□	△
3/8"	100.0	100.0	100.0
	GRAIN SIZE		
D_{60}			0.113
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	96.4	95.6	92.7
#10	93.7	91.8	85.3
#30	86.7	86.4	73.4
#40	83.7	84.5	70.4
#50	79.8	82.3	67.6
#100	71.1	77.1	62.2
#200	61.3	70.8	56.8

SOIL DESCRIPTION

○ Very Dark Brown Sandy Lean CLAY

□ Very Dark Brown Lean CLAY w/ Sand

△ Very Dark Brown Sandy Lean CLAY

REMARKS:

○

□

△

○ Source: EB-7 Elev./Depth: 1.5-2'

□ Source: EB-7 Elev./Depth: 6-6.5'

△ Source: EB-8 Elev./Depth: 1.5-2'

COOPER TESTING LABORATORY

Client: Romig Engineers, Inc.

Project: Wu-Chung Hotel Complex - 2317-1A

Project No.: 192-170

Figure B-5

Corrosivity Test Summary

COOPER TESTING LABORATORY

CTL # 192-126 **Date:** 7/13/2009 **Tested By:** PJ **Checked:** PJ

Client: Romig Engineers **Project:** Wu Chung Hotel/Condos **Proj. No:** 2317-1

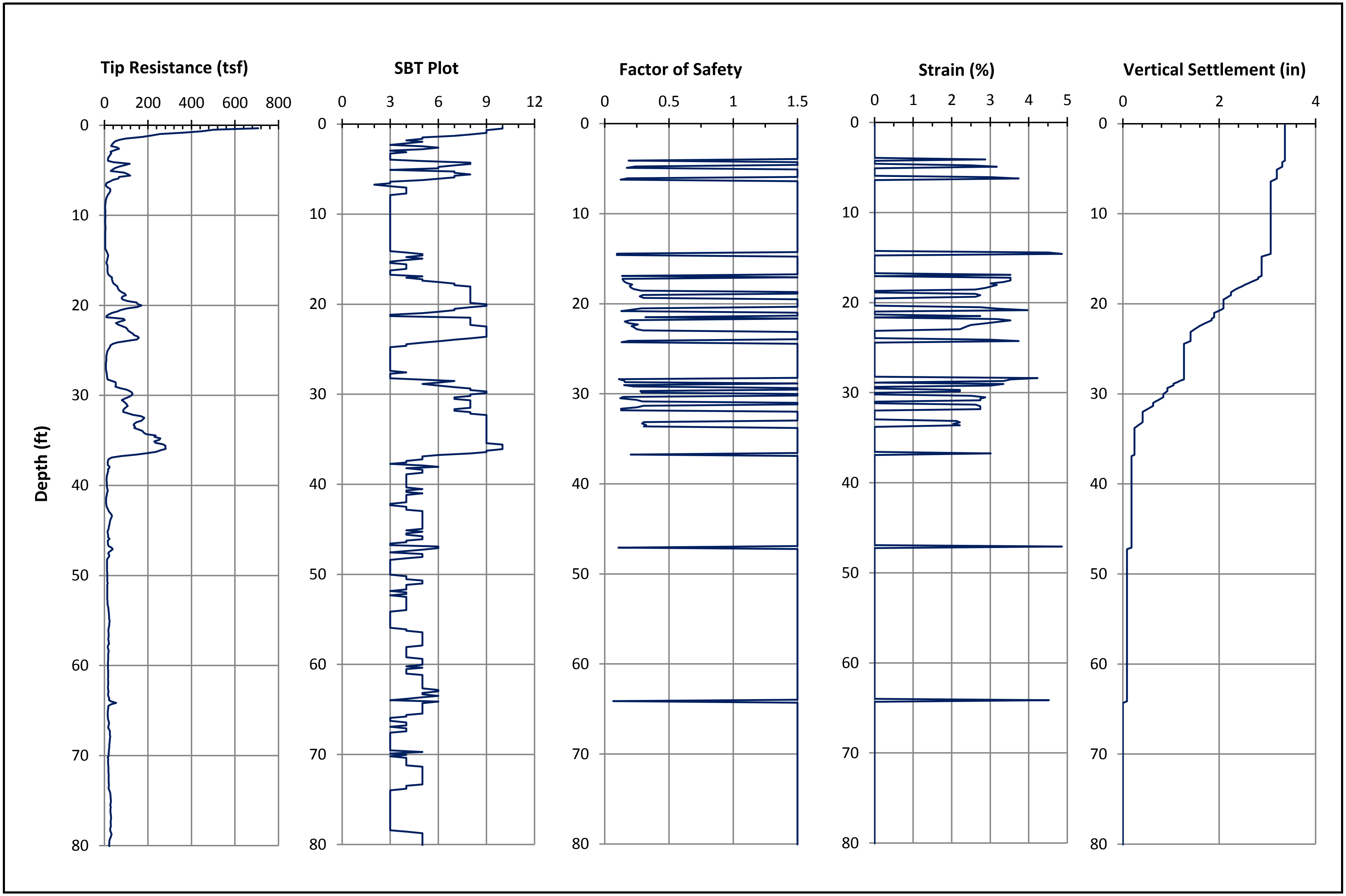
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
CPT-2	-	3	-	-	4,158	<2	<5	<0.0005	8.0	123	-	8.9	Brown Clayey SAND w/ Gravel
CPT-3	-	5	-	-	1,502	<2	<5	<0.0005	7.9	-34	-	20.7	Dark Gray Clayey SAND

APPENDIX C

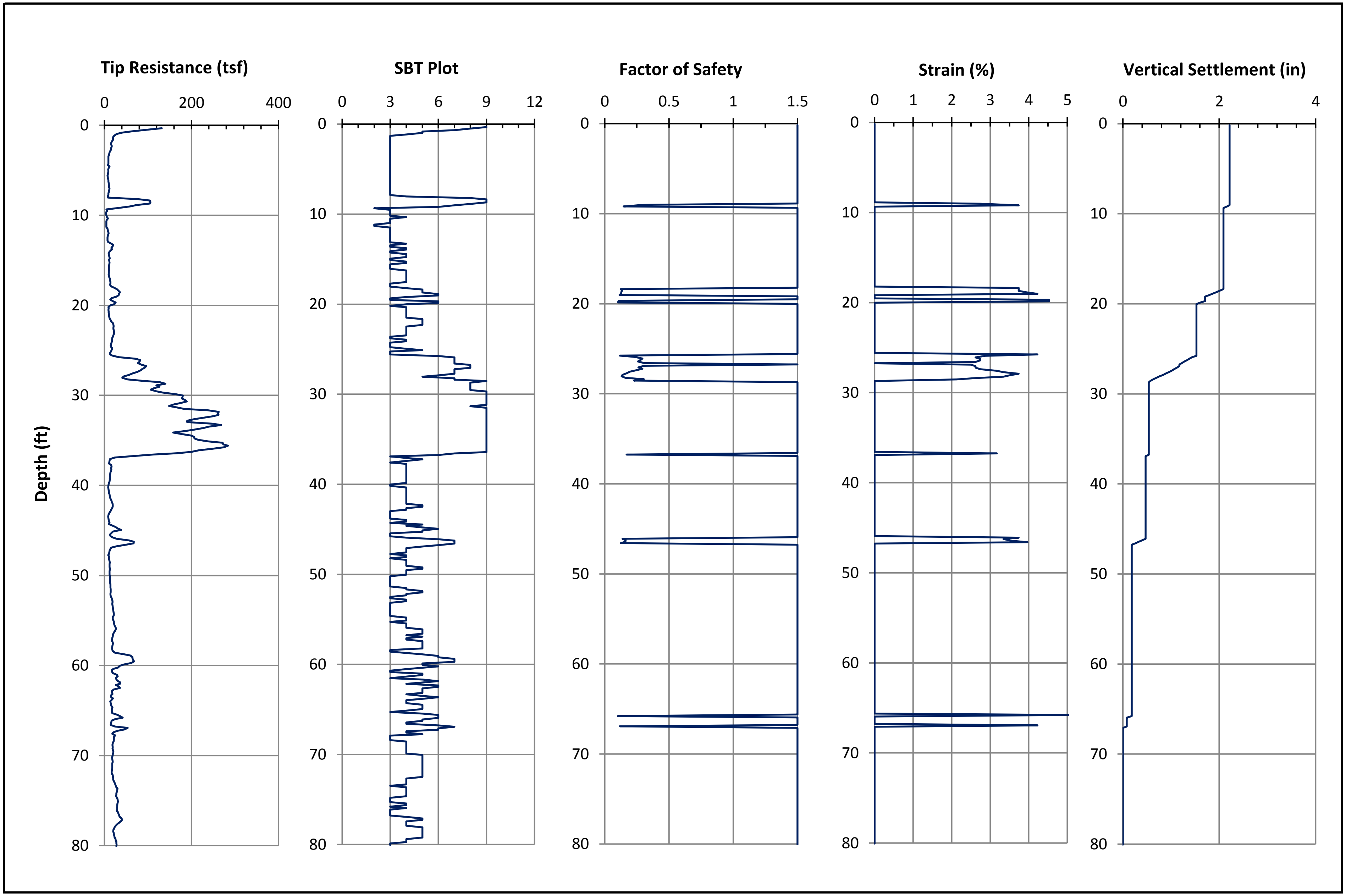
LIQUEFACTION ANALYSIS RESULTS

❖ ❖ ❖ ❖ ❖



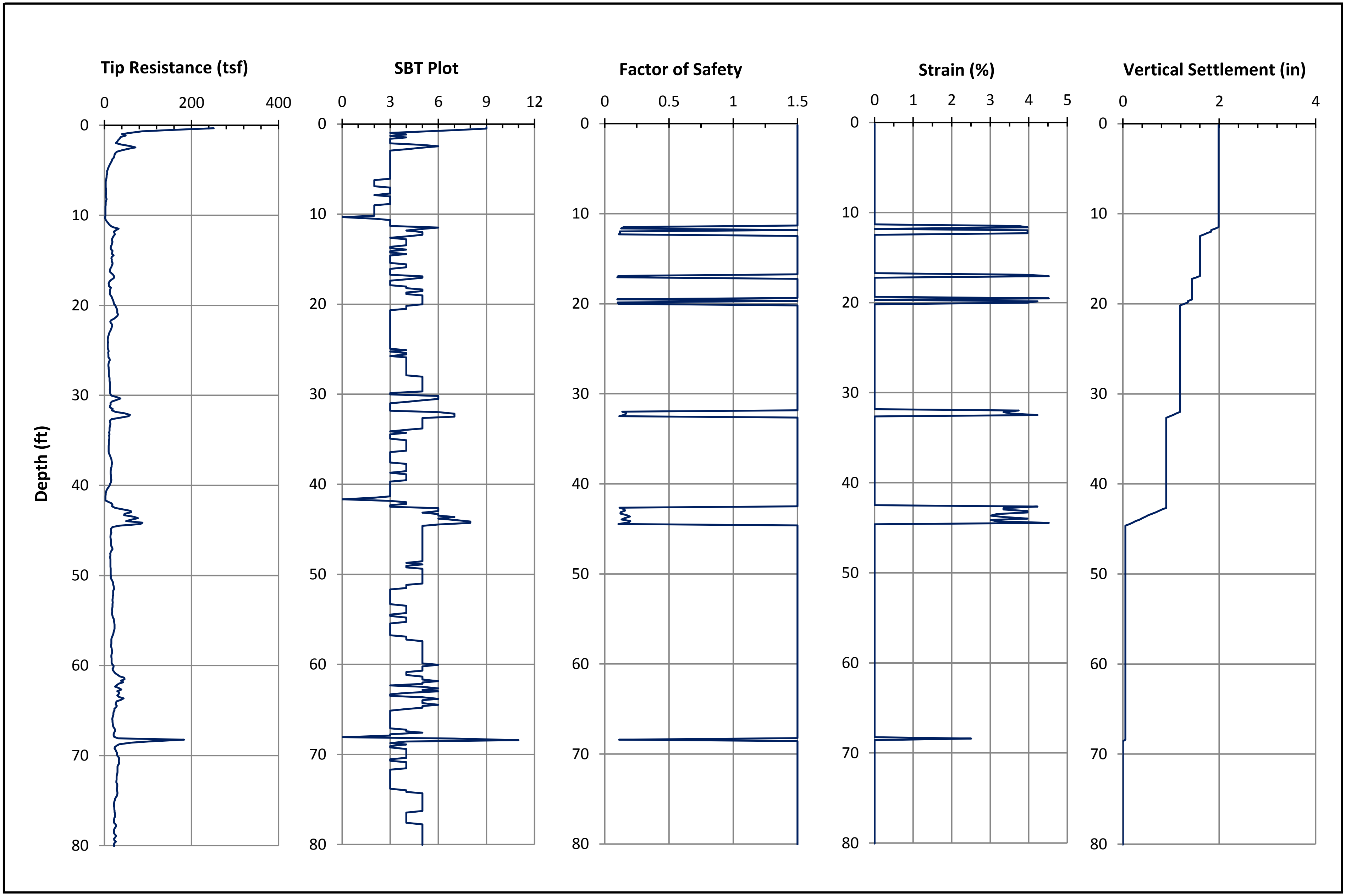
LIQUEFACTION ANLAYSIS FOR CPT-1
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-1
MAY 2018
PROJECT NO. 4300-1



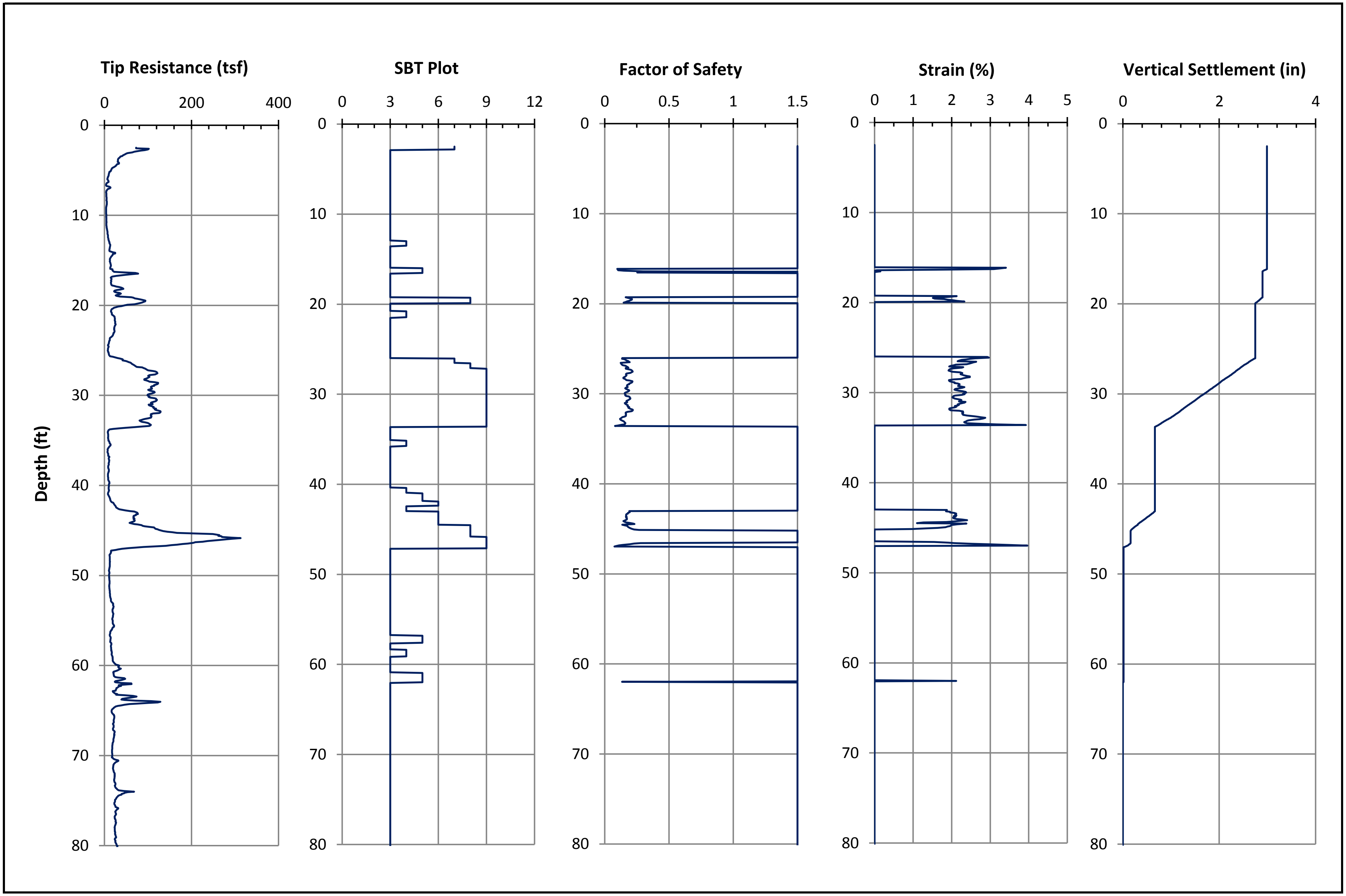
LIQUEFACTION ANLAYSIS FOR CPT-2
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-2
MAY 2018
PROJECT NO. 4300-1



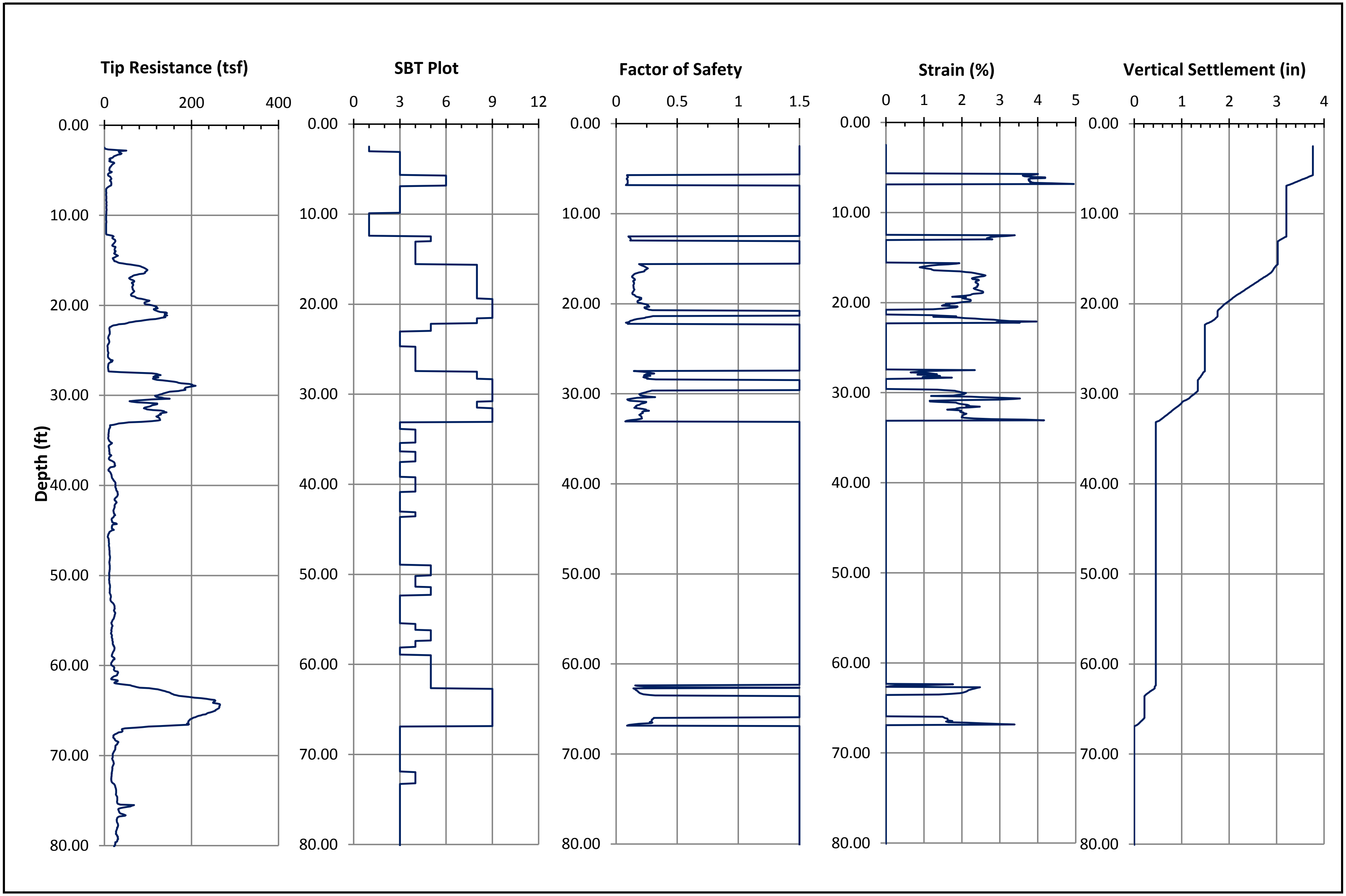
LIQUEFACTION ANLAYSIS FOR CPT-3
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-3
MAY 2018
PROJECT NO. 4300-1



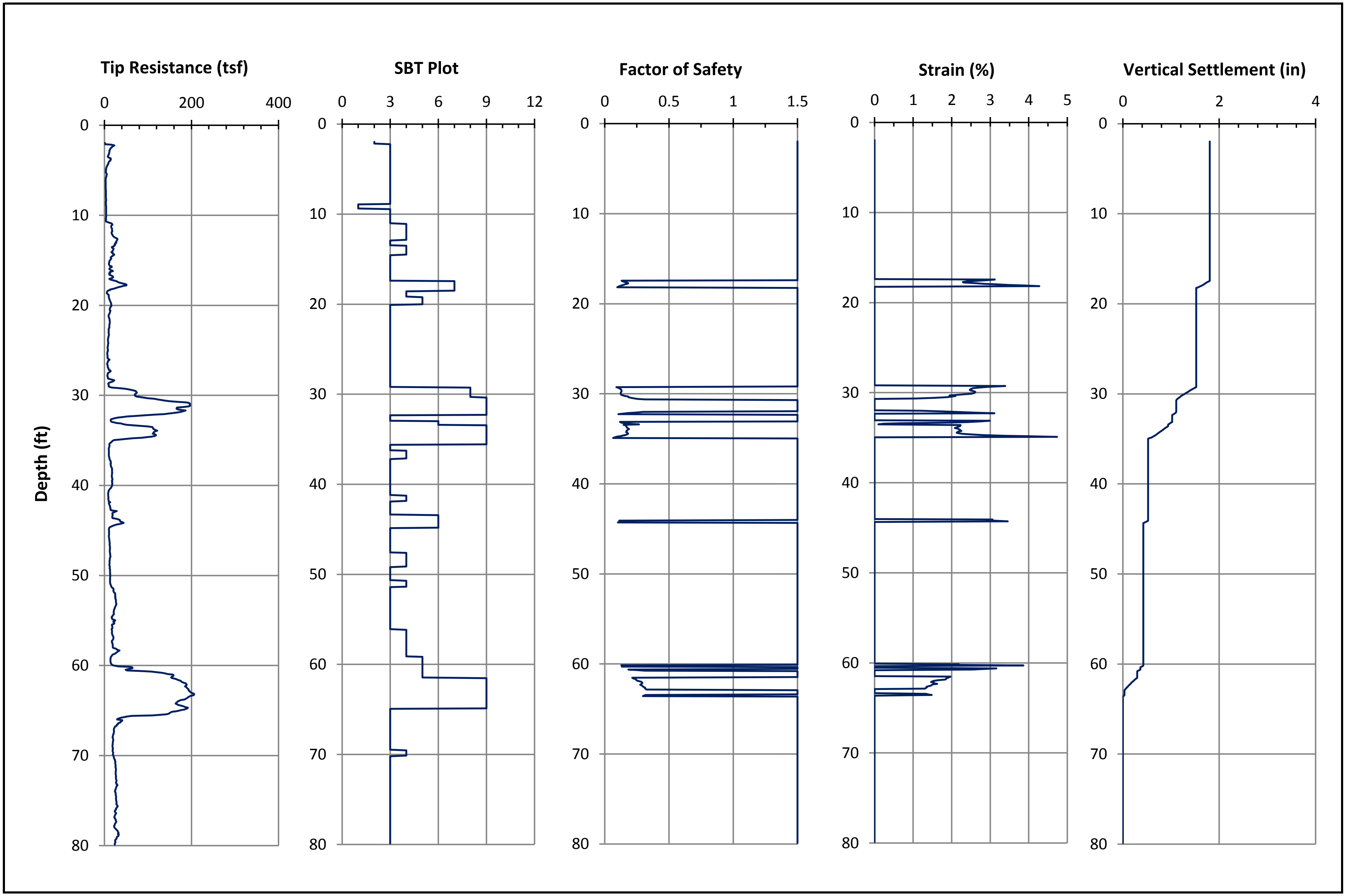
LIQUEFACTION ANLAYSIS FOR PREVIOUS CPT-1 (2009)
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-4
MAY 2018
PROJECT NO. 4300-1



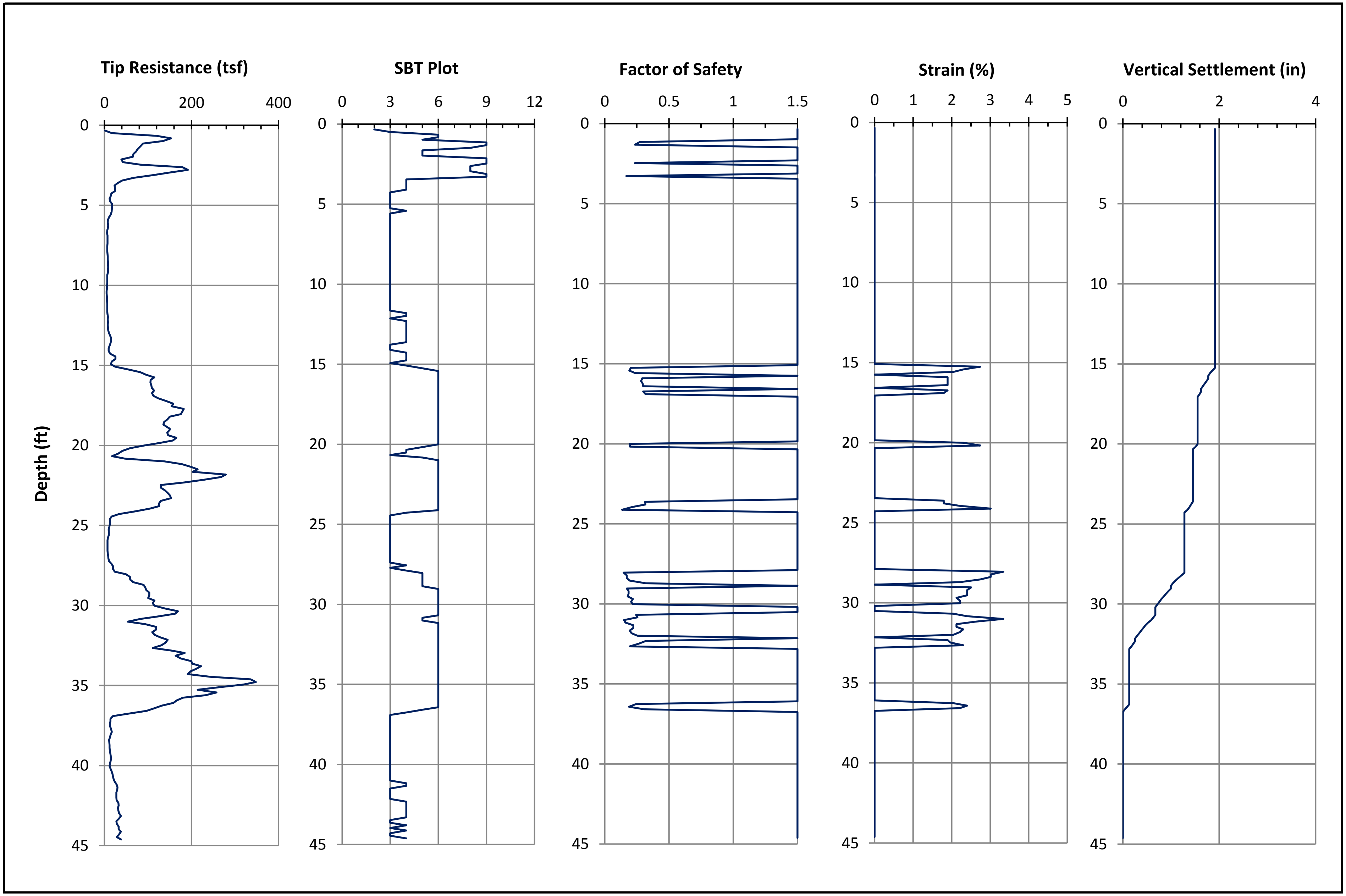
LIQUEFACTION ANLAYSIS FOR PREVIOUS CPT-2 (2009)
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-5
MAY 2018
PROJECT NO. 4300-1



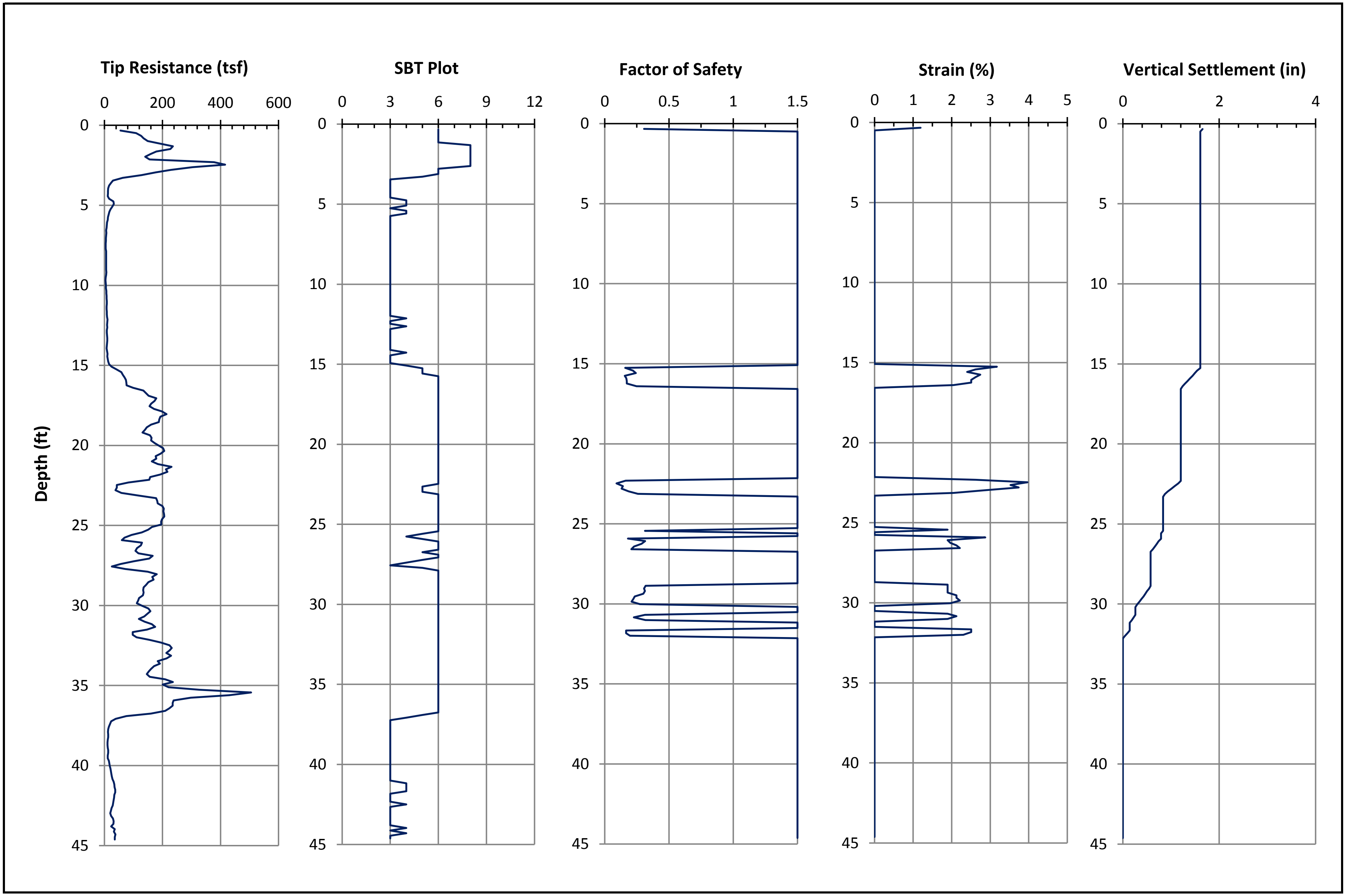
LIQUEFACTION ANLAYSIS FOR PREVIOUS CPT-3 (2009)
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-6
MAY 2018
PROJECT NO. 4300-1



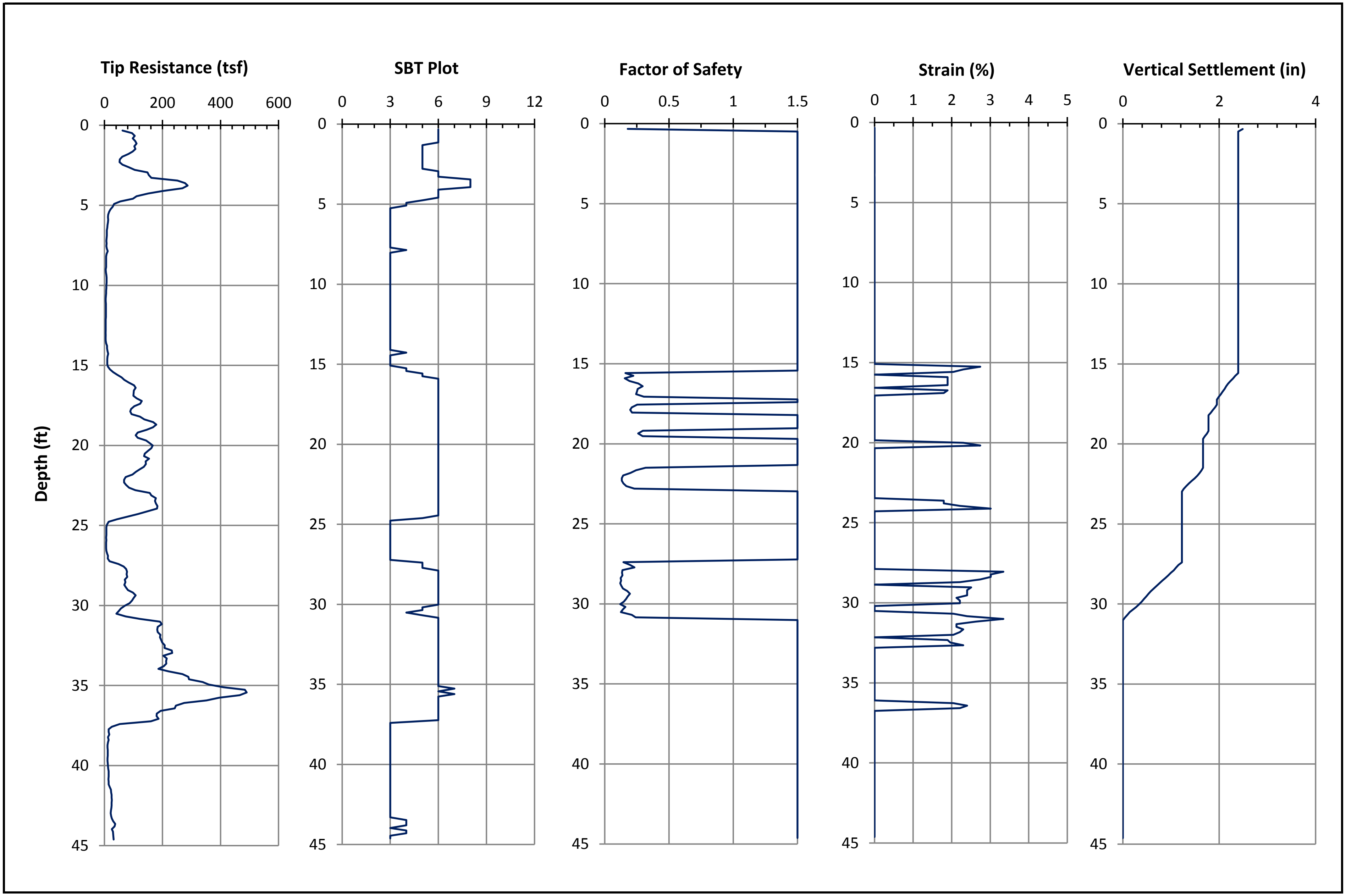
LIQUEFACTION ANLAYSIS FOR PREVIOUS CPT-1 (2014)
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-7
MAY 2018
PROJECT NO. 4300-1



LIQUEFACTION ANLAYSIS FOR PREVIOUS CPT-2 (2014)
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-8
MAY 2018
PROJECT NO. 4300-1



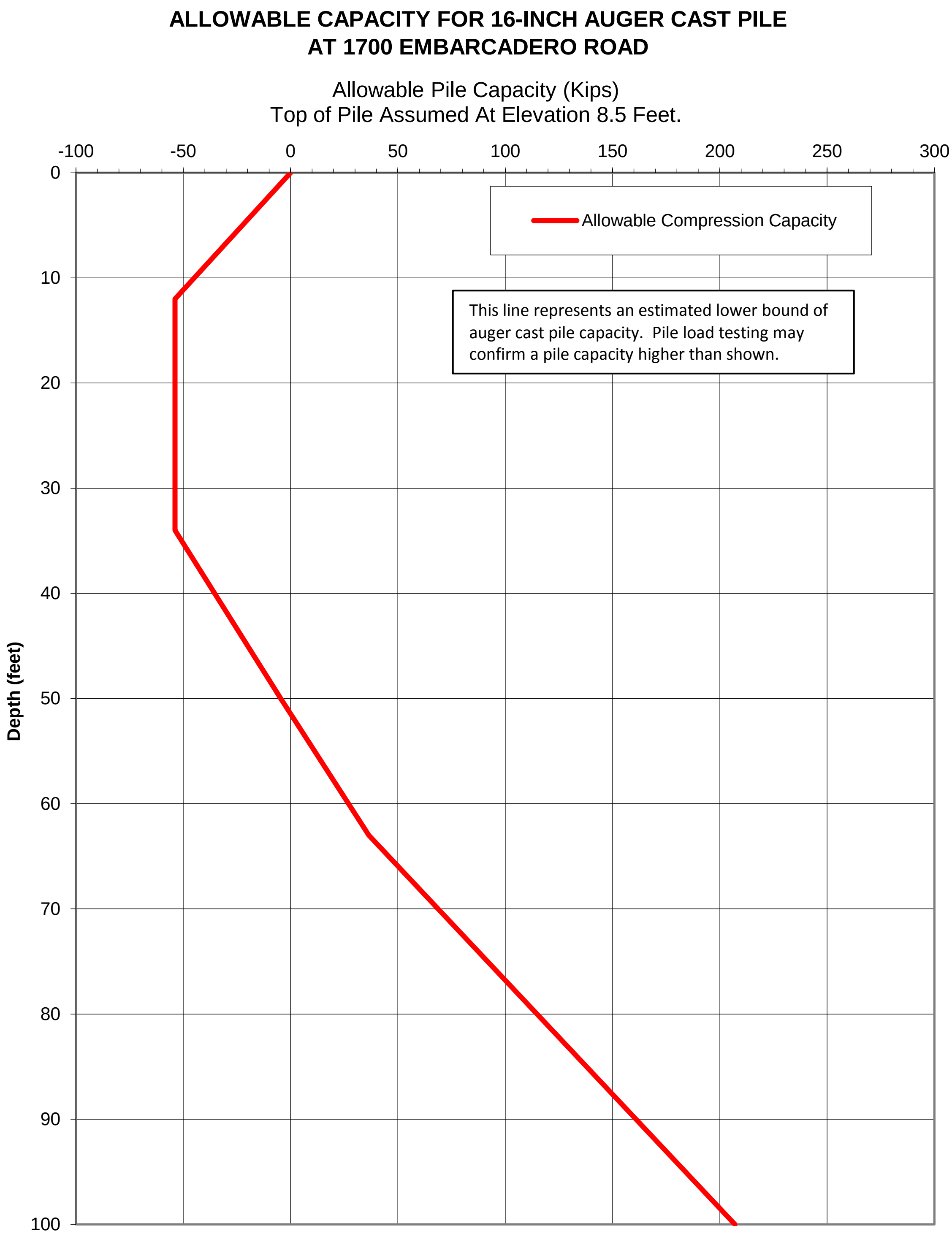
LIQUEFACTION ANLAYSIS FOR PREVIOUS CPT-3 (2014)
MERCEDES BENZ AND AUDI DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE C-9
MAY 2018
PROJECT NO. 4300-1

APPENDIX D

PILE CAPACITY ANALYSIS RESULTS

❖ ❖ ❖ ❖ ❖

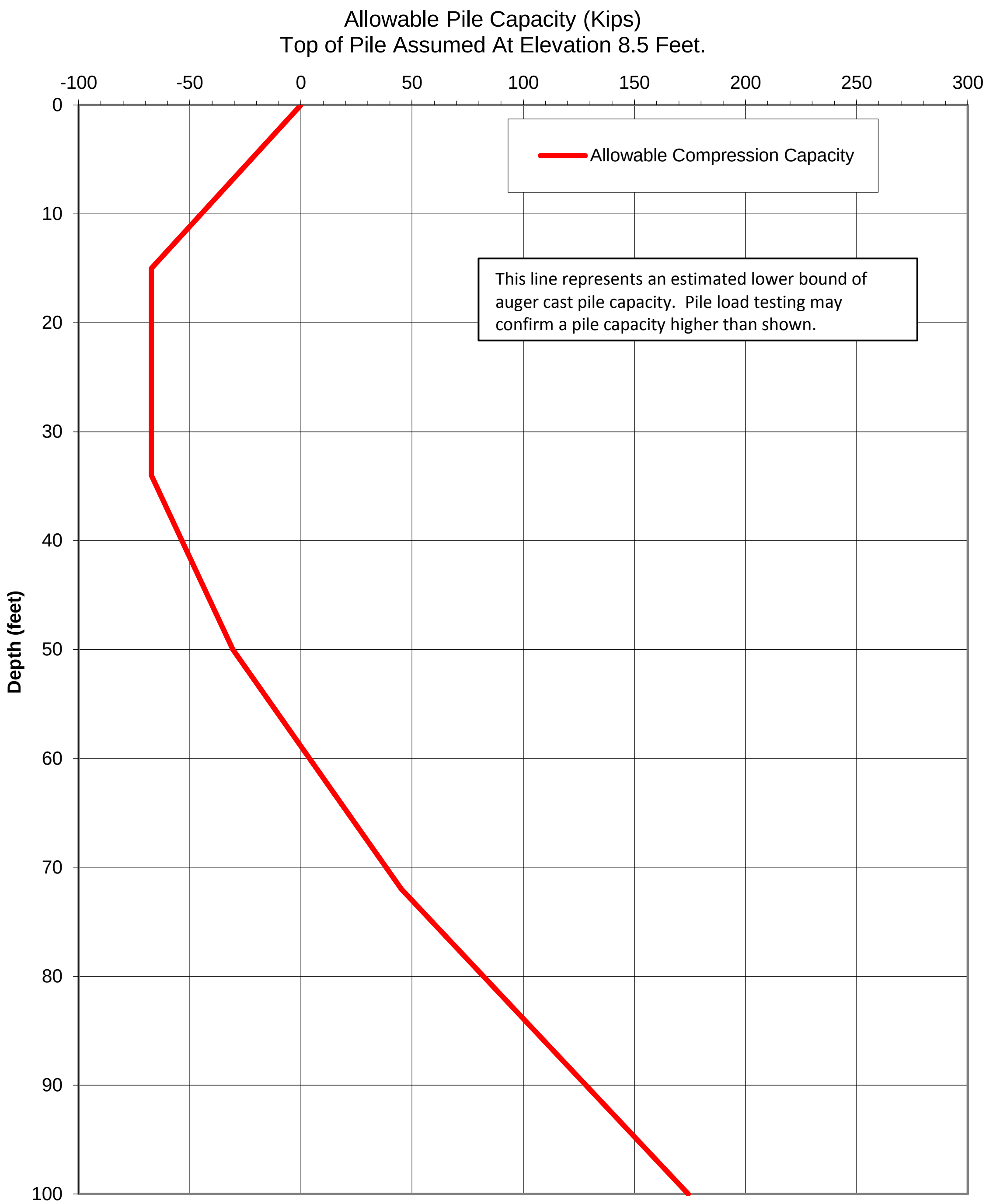


ALLOWABLE 16-INCH AUGER CAST PILE CAPACITY **FIGURE D-1**

MERCEDES-BENZ AND AUDI DEALERSHIP MAY 2018

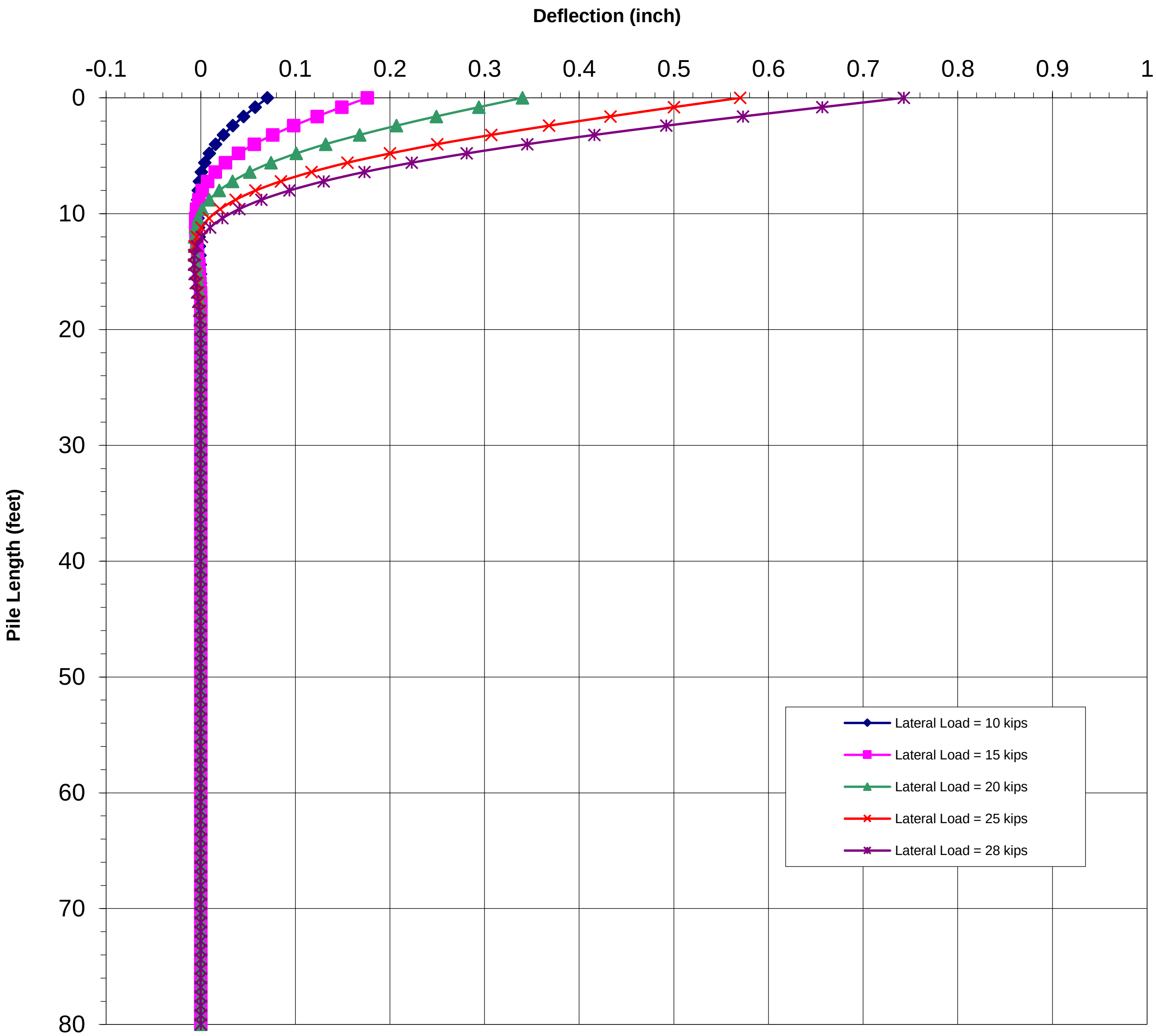
PALO ALTO, CALIFORNIA PROJECT NO. 4300-1

ALLOWABLE CAPACITY FOR 16-INCH AUGER CAST PILE
AT 1730 EMBARCADERO ROAD



ALLOWABLE 16-INCH AUGER CAST PILE CAPACITY | **FIGURE D-2**
MERCEDES-BENZ AND AUDI DEALERSHIP | MAY 2018
PALO ALTO, CALIFORNIA | PROJECT NO. 4300-1

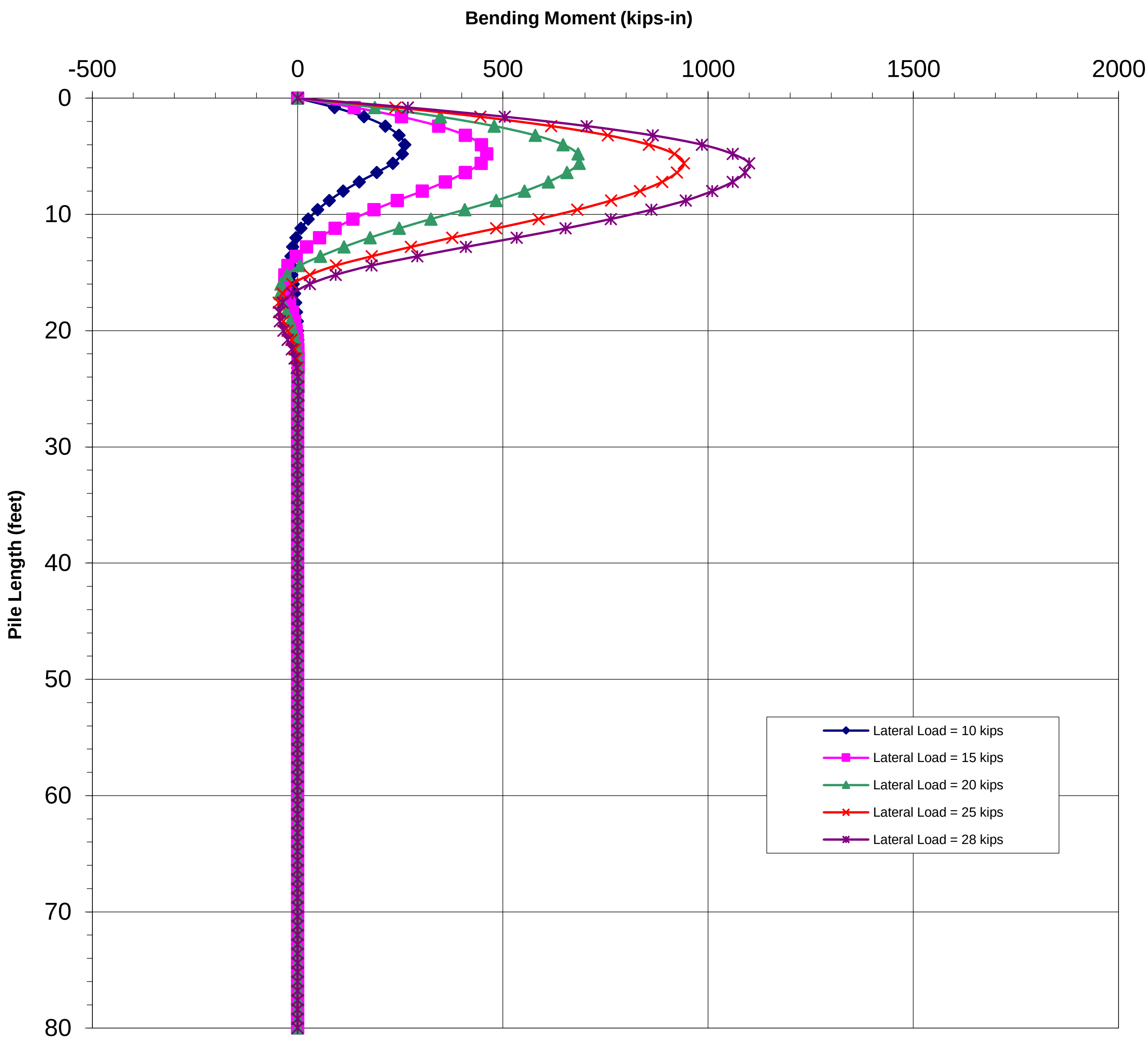
LATERAL DEFLECTION VS. DEPTH
16-inch Auger Cast Pile, Free Head Condition



LATERAL PILE DEFLECTION - FREE HEAD CONDITION
MERCEDES AND AUDI BENZ DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE D-3
MAY 2018
PROJECT NO. 4300-1

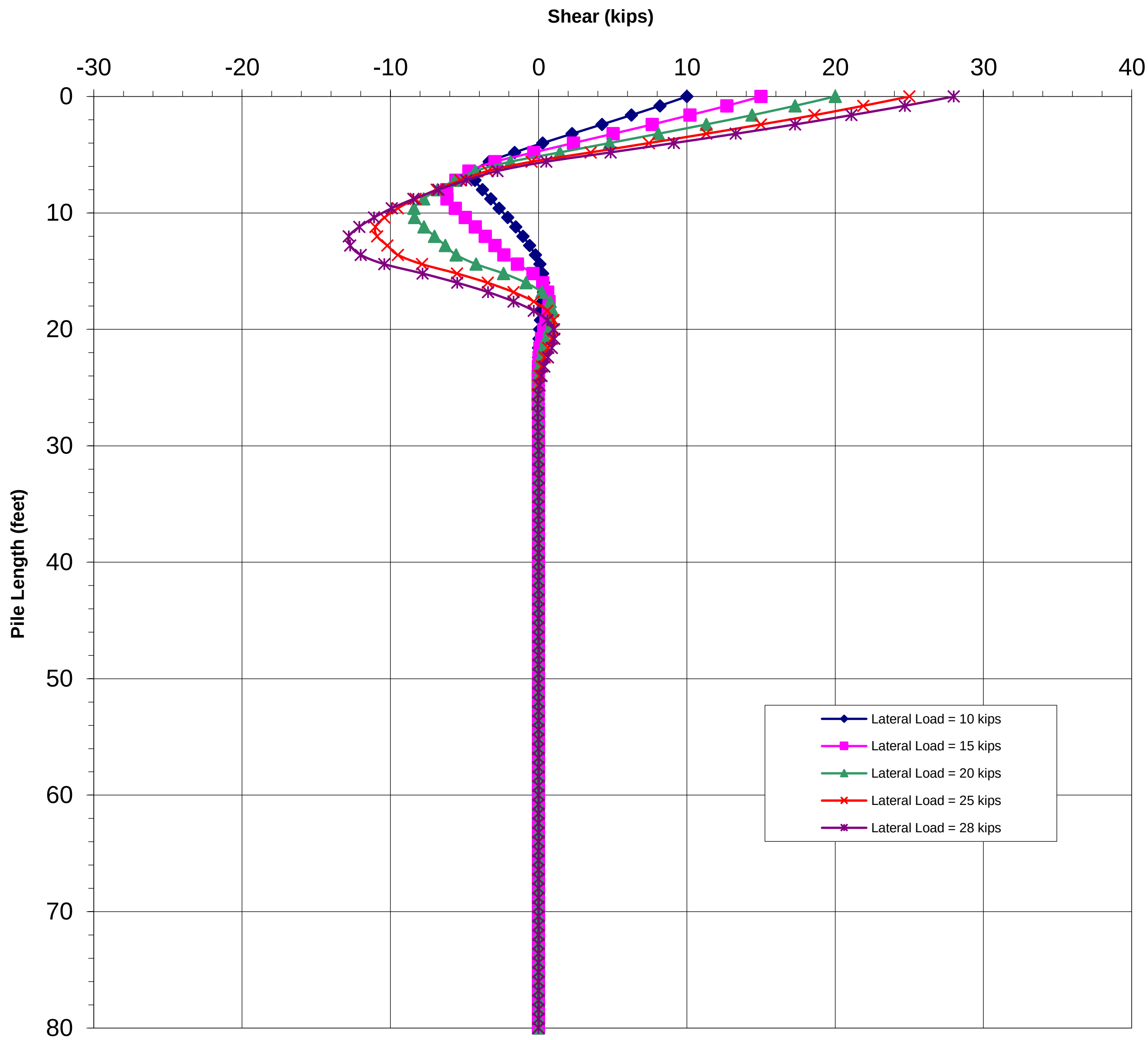
BENDING MOMENT VS. DEPTH
16-inch Auger Cast Pile, Free Head Condition



PILE BENDING MOMENT - FREE HEAD CONDITION
MERCEDES AND AUDI BENZ DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE D-4
MAY 2018
PROJECT NO. 4300-1

SHEAR VS. DEPTH
16-inch Auger Cast Pile, Free Head Condition

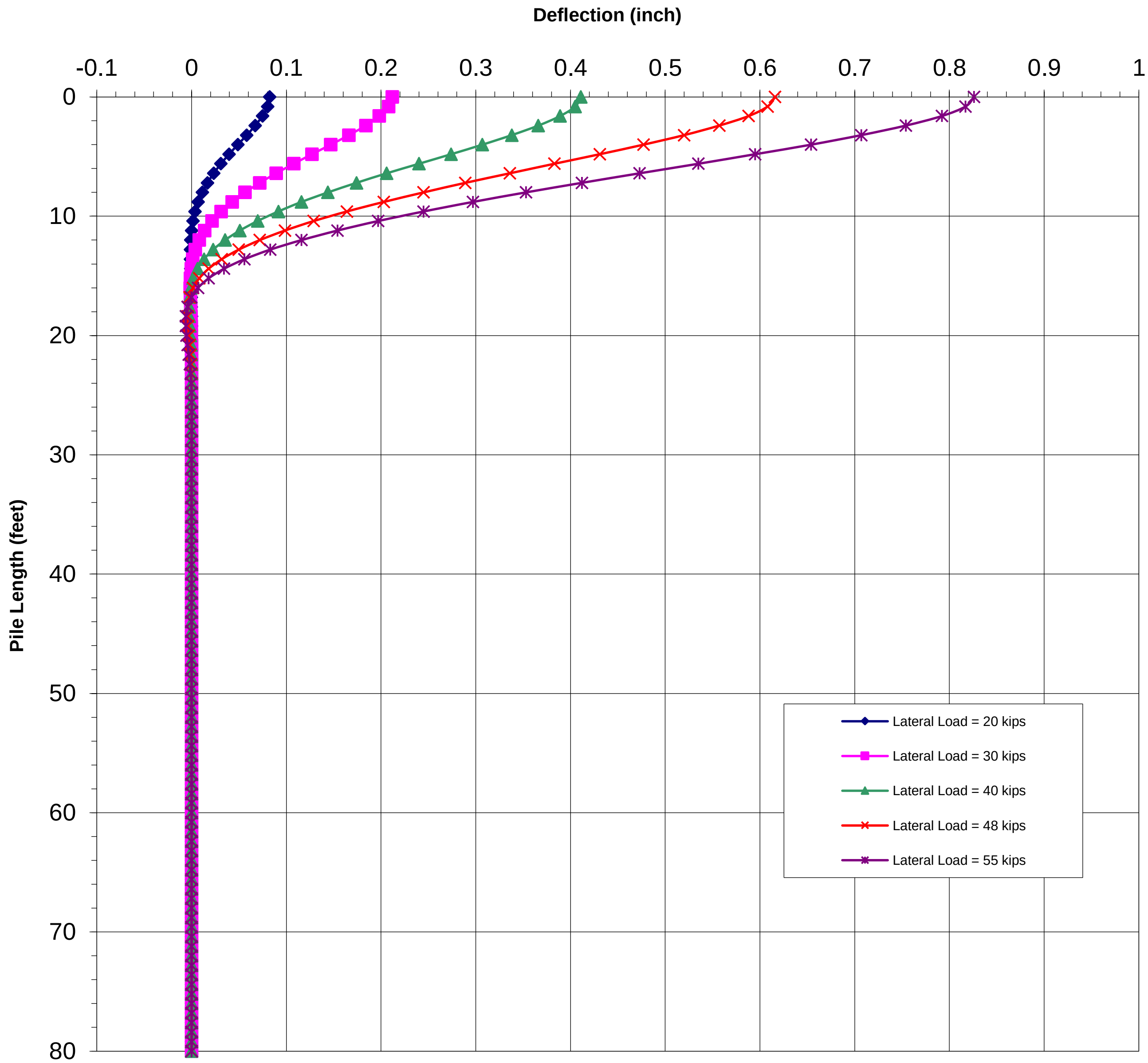


PILE SHEAR - FREE HEAD CONDITION **FIGURE D-5**

MERCEDES AND AUDI BENZ DEALERSHIPS MAY 2018

PALO ALTO, CALIFORNIA PROJECT NO. 4300-1

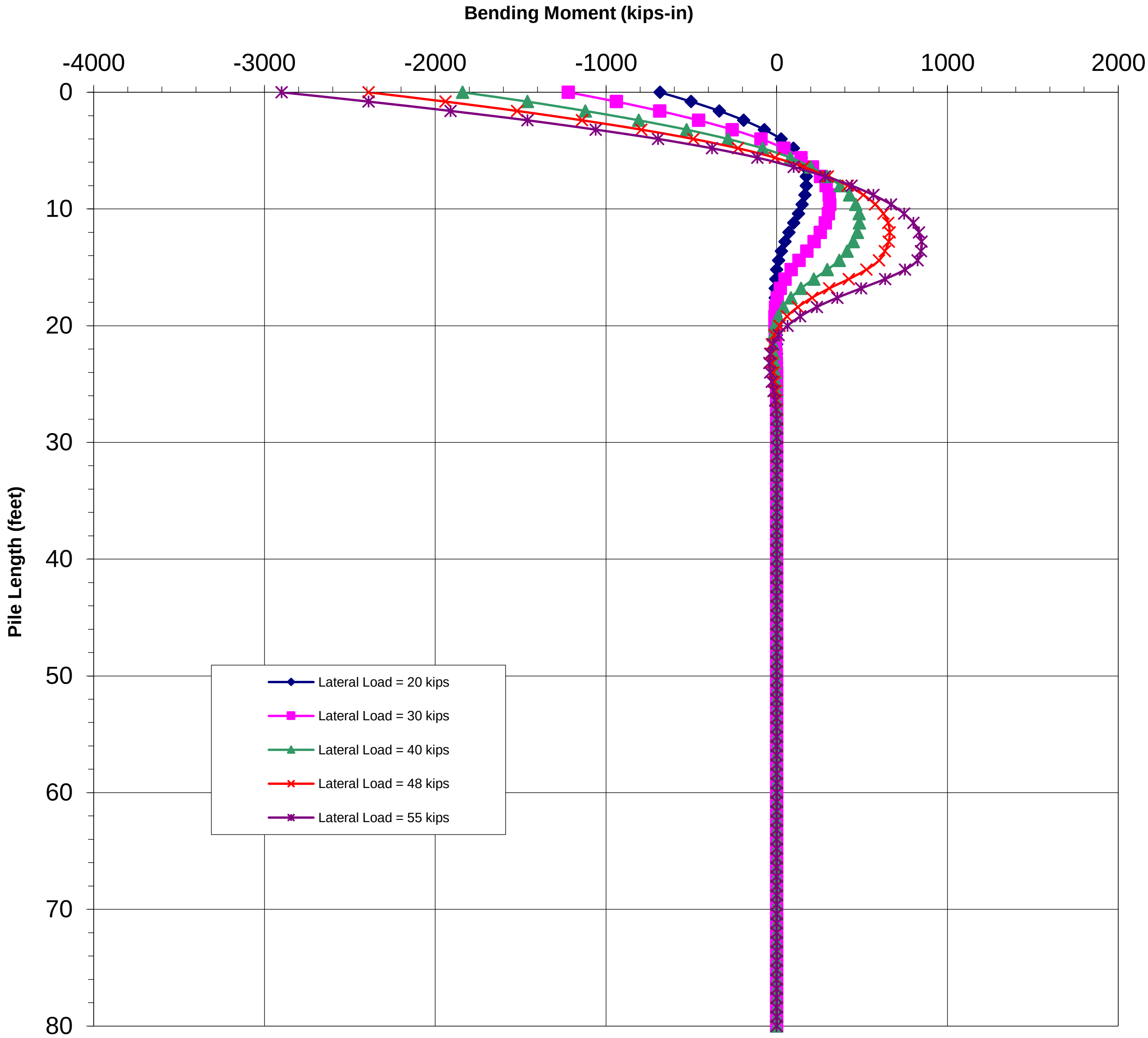
LATERAL DEFLECTION VS. DEPTH
16-inch Auger Cast Pile, Fixed Head Condition



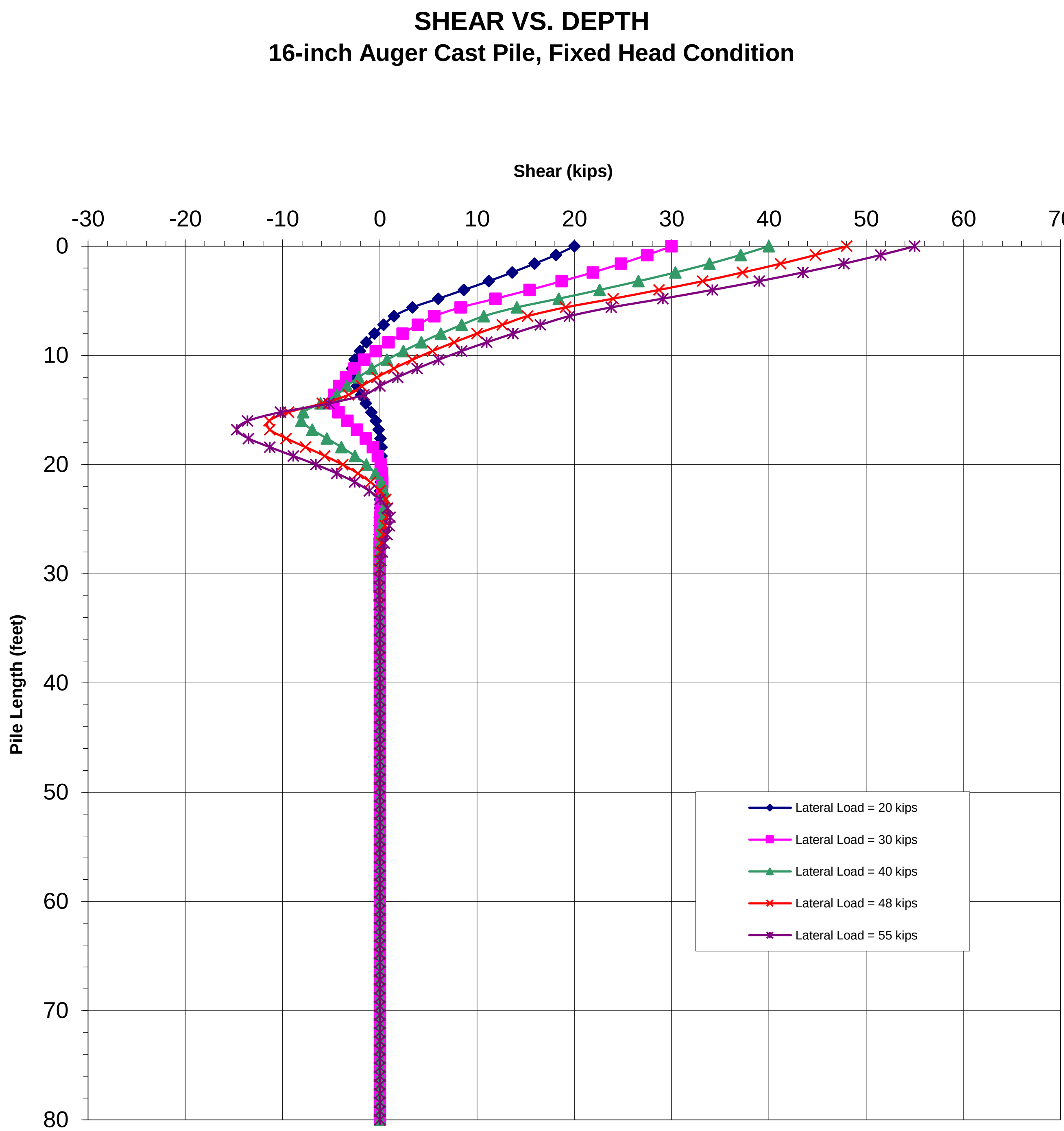
LATERAL PILE DEFLECTION - FIXED HEAD CONDITION
MERCEDES AND AUDI BENZ DEALERSHIPS
PALO ALTO, CALIFORNIA

FIGURE D-6
MAY 2018
PROJECT NO. 4300-1

BENDING MOMENT VS. DEPTH
16-inch Auger Cast Pile, Fixed Head Condition



PILE BENDING MOMENT - FIXED HEAD CONDITION **FIGURE D-7**
MERCEDES AND AUDI BENZ DEALERSHIPS MAY 2018
PALO ALTO, CALIFORNIA PROJECT NO. 4300-1



PILE SHEAR - FIXED HEAD CONDITION **FIGURE D-8**

MERCEDES AND AUDI BENZ DEALERSHIPS MAY 2018

PALO ALTO, CALIFORNIA PROJECT NO. 4300-1

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