GEOTECHNICAL INVESTIGATION
FOR
PALO ALTO PUBLIC SAFETY BUILDING
AND PARKING GARAGE
SHERMAN AVENUE
PALO ALTO, CALIFORNIA 94306

May 2016

Prepared for

City of Palo Alto
Public Works Department
250 Hamilton Avenue
Palo Alto, California 94301

Project No. 3723-1

ROMIG ENGINEERS, INC.
GEOTECHNICAL & ENVIRONMENTAL SERVICES

May 6, 2016
3723-1

City of Palo Alto
Public Works Department
250 Hamilton Avenue
Palo Alto, California 94301

RE: GEOTECHNICAL INVESTIGATION
PALO ALTO PUBLIC SAFETY BUILDING
AND PARKING GARAGE
SHERMAN AVENUE
PALO ALTO, CALIFORNIA

Attention: Mr. Matt Raschke

Ladies and Gentlemen:

In accordance with your request, we have performed a geotechnical investigation for the Palo Alto Public Safety Building and parking garage proposed for construction on the north side of Sherman Avenue between Ash Street and Park Boulevard Palo Alto, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents our geotechnical conclusions and recommendations for the currently proposed 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. Please call if you have questions or comments about site conditions or our geotechnical recommendations for the proposed project.

Very truly yours,

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SHERMAN AVENUE
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PREPARED FOR:

CITY OF PALO ALTO

PUBLIC WORKS DEPARTMENT

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MAY 2016

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INTRODUCTION

This report presents the results of our geotechnical investigation for the Palo Alto Public Safety Building and parking garage proposed for construction on the north side of Sherman Avenue between Ash Street and Park Boulevard in Palo Alto, California. The approximate location of the project site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for design and construction of the proposed project.

Project Description

The project consists of constructing a three-story Public Safety Building and a three-story parking garage on the north side of Sherman Avenue. The sites for the proposed buildings are currently occupied by public parking lots C-6 and C-7. The proposed Public Safety Building on the 1.2-acre site occupied by parking lot C-6 will have a footprint area of approximately 45,500-square-feet and will include a two-level operational basement below grade and at-grade parking. The proposed parking garage on the 0.93-acre site occupied by parking lot C-7 will have a footprint area of approximately 32,700-square-feet and will include two-levels of below grade parking. A 4,700-square-foot at-grade retail building will be constructed between the northeastern end of the parking garage and Birch Avenue. The below-grade basements for the Public Safety Building and the parking garage are planned to be connected by a tunnel below Birch Street to provide a secondary means of emergency vehicle egress. The site plans included in this report are conceptual.

Scope of Work

The scope of our geotechnical services for the Public Safety Building and parking garage is described in our agreement with the City of Palo Alto, dated March 9, 2016. In order to accomplish this investigation, we performed the following work.

- Review of geologic, geotechnical, seismic, and ground water conditions in the vicinity of the project site.

- Subsurface exploration consisting of drilling, sampling, and logging of three exploratory borings and advancing seven cone penetration test (CPT) probes at the sites for the proposed Public Safety Building and parking garage.
- Laboratory testing of selected soil samples to aid in soil classification and to help evaluate their engineering properties.
- Engineering analysis and evaluation of the available surface and subsurface data to develop geotechnical design criteria for the project.
- Preparation of this geotechnical report presenting our findings, conclusions, and geotechnical recommendations for the currently proposed Public Safety Building and parking garage.

Limitations

This report has been prepared for the exclusive use of the City of Palo Alto for specific application to developing geotechnical design criteria for the proposed Public Safety Building and parking garage on the north side of Sherman Avenue between Ash Street and Park Boulevard in Palo Alto, California. We make no warranty, expressed or implied, for the services we perform for this project. 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 proposed improvements, review of readily available 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 RECONNAISSANCE AND SUBSURFACE EXPLORATION

Site reconnaissance and subsurface exploration were performed on March 31 and April 8, 2016. Subsurface exploration consisted of advancing three exploratory borings to a depth of 44.5 feet and seven CPTs to depths ranging from 43.8 to 44.1 feet. The exploratory borings were advanced using a truck-mounted drill equipped with 8-inch diameter hollow-stem augers. The CPTs were advanced using a truck with a down pressure of 25 tons and an electronic cone penetration test system. The approximate locations of the borings and CPT probes are shown on the Site Plans, Figure 2 and 3. Logs of the exploratory borings and CPTs are included in Appendix A, and the results of our laboratory tests are included in Appendix B.

Surface Conditions

The proposed building sites are located in a commercial area along the north side of Sherman Avenue between Ash Street and Park Boulevard. Birch Street separates the proposed building sites. At the time of our investigation, the relatively flat sites were occupied by parking lots known as Lots C-6 and C-7. These parking lots consisted of asphalt concrete paved parking stalls and drive aisles with concrete islands and landscape planter areas between and along the perimeter of the parking areas. Concrete sidewalks were present along the City streets. Perimeter landscape areas included small shrubs and medium to large trees. The asphalt concrete pavements had numerous hairline to ½-inch wide cracks and large areas of alligator cracking. The concrete walkways had cracks varying from hairline to ¼-inch wide.

Subsurface Conditions

The alluvial soils encountered at the site generally consisted of interbedded layers of stiff to hard, clayey silt, sandy silt, sandy clay, and silty clay, and medium dense to very dense, clayey sand, silty sand, and poorly graded sand. Our borings and CPTs indicate that soils within the anticipated 24-foot depth of the basement excavations are likely to vary from medium dense to very dense sand with variable cohesion to stiff to hard clay.

At the locations of the seven cone penetration test probes, we generally encountered 3.5 to 6 feet of very stiff to hard, clayey silt and/or silty clay with interbeds of silty sand underlain by 5 to 12 feet of dense to very dense, clayey sand and silty sand with interbeds of stiff, sandy silt. These soils were underlain by generally stiff to hard, clayey silt and/or silty clay to a depth of approximately 30 to 32 feet. The consistency of portions of the clayey silt and/or silty clay strata between depths of about 22 to 28 feet is firm. Below a depth of approximately 30 to 32 feet, the CPTs encountered approximately 6 to 10 feet of medium dense to very dense, clean sand to silty sand and stiff to hard, sandy silt and silty clay to a depth of approximately 44 feet, the maximum depth of our CPT exploration.

At the location of Boring EB-1 in the northeastern portion of the Public Safety Building site, we encountered approximately 32.5 feet of stiff to hard, sandy lean clay of low to moderate plasticity underlain by very dense, poorly-graded sand to the maximum depth explored of 44.5 feet.

At Boring EB-2 near the western side of the Public Safety Building site, we encountered approximately 5 feet of stiff to hard, sandy lean clay of low plasticity underlain by approximately 13.5 feet of dense to very dense, clayey sand with gravel. These soils were underlain by approximately 18.5 feet of very stiff, sandy lean clay of moderate plasticity over medium dense to very dense, clayey sand that extended to a depth of 44.5 feet, the maximum depth of our exploration.

At Boring EB-3 in the southeastern portion of the parking garage site, we encountered approximately 16 feet of medium dense to very dense, clayey sand underlain by approximately 6 feet of very stiff, sandy lean clay of moderate plasticity. These soils were underlain by approximately 10 feet of very stiff, sandy fat clay of high plasticity over approximately 8 feet of dense to very dense, poorly graded sand over hard, sandy lean clay of moderate plasticity to a depth of 44.5 feet, the maximum depth of our exploration.

A Liquid Limit of 34 and a Plasticity Index of 15 were measured on a sample of near surface soil obtained from Boring EB-1. These test results indicate the surface and near-surface clays at the site have low plasticity and a low potential for expansion.

A Liquid Limit of 54 and a Plasticity Index of 28 were measured on a sample of sandy fat clay encountered in Boring EB-3 at a depth of 23.5 to 25 feet, indicating the sandy fat clay at this depth and location has high plasticity. Liquid Limits of 46 and 43 and Plasticity Indices of 20 and 17 were measured on samples of sandy lean clay recovered from Borings EB-1 and EB-2 at a depth of 28.5 to 30 feet. These test results indicate the sandy lean clay at these locations has moderate plasticity.

Ground Water

During drilling and sampling on April 8, 2016, ground water was encountered at depths of approximately 21.6, 26.6, and 23.5 feet in Borings EB-1, EB-2, and EB-3, respectively. Pore pressure dissipation tests performed during CPT exploration on March 31, 2016 indicated that ground water was present between depths of about 19.6 to 23.9 feet below the ground surface. The borings and CPTs were backfilled with grout shortly after sampling and penetration testing was completed. The borings may not have been left open for a sufficient length of time for the level of the ground water to stabilize.

Ground water levels in the general area of the site and to the northeast have been artificially-lowered for many years by pumping of ground water from the Oregon-Page Mill Expressway Underpass pump station and from several extraction wells to the southeast. As part of our work, we reviewed records of ground water levels measured intermittently in monitoring wells by SECOR International and/or by Stantec Consulting Services from 2005 through 2013. The information we reviewed indicates that during this period the ground water level in the general area of the site has varied from about 14.3 to 21 feet below the ground surface, and has varied as much as 4 to 6 feet seasonally.

We were told by South Bay Construction's project manager that the depth to ground water prior to dewatering for the basement excavation at 385 Sherman Avenue in 2015 varied from about 21 to 23 feet below grade.

According to information on Plate 1.2 contained in Seismic Hazard Zone Report 111 for the Palo Alto Quadrangle (CGS, 2006), the highest historic ground water level in the area of the site is approximately 17 feet below the ground surface. Generalized depth to first ground water maps from the Santa Clara Valley Water District (2003) and the City of Palo Alto (2007) indicate the highest ground water level that has been encountered in the general area of the site is about 12 to 13 feet below the ground surface.

Based on the information summarized above, we recommend assuming a design ground water level of 12 feet below existing grade for design of the basement structures and the access tunnel. For preliminary planning purposes, it may be assumed that ground water will be encountered during basement excavation at a depth of about 21 to 24 feet after below average to average winter rainfall and at a depth of about 17 to 20 feet after above average winter rainfall. The depth to ground water at the site could be monitored by installation of ground water observation wells. Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, landscaping, surface and subsurface drainage patterns, pumping of ground water from the Oregon-Page Mill Expressway Underpass, and other factors.

City Basement Drainage Requirements

The City of Palo Alto Public Works Department has a policy that requires basement floors and basement walls in this area of Palo Alto to be designed and constructed without underdrains or wall backdrains. Since basement floor underdrains and wall backdrains will not be allowed, the lower level floor will need to be designed to resist uplift pressure from a high ground water level and the basement walls will need to be designed to resist lateral pressure from undrained basement wall backfill. To reduce the potential for leaks and dampness of the basement, the basement floor and walls will need to be effectively water-proofed.

GEOLOGIC AND SEISMIC SETTING

As part of our investigation, we reviewed our local experience and geologic literature pertinent to the general area of the site. The information reviewed indicates the site is located in an area underlain by Holocene-age flood plain deposits, Qhfp (Brabb, Graymer, and Jones, 2000). These deposits are generally expected to consist of unconsolidated to moderately consolidated, moderately sorted, fine sand, silt, and clayey silt deposited at the edge of coarse-grained alluvial fans and interfingered with coarse-grained and fine-grained alluvium. The mapped geology in the general area of the site is shown on the Vicinity Geologic Map, Figure 4.

The project site and the immediate site vicinity are located in an area that slopes very gently down to the east and northeast (approximately 10 feet vertically per 2,000 feet laterally although locally the topography may be steeper). The surface of the project site varies from approximately elevation 33 to 35 feet above sea level.

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 must be considered. The closest active fault is the San Andreas Fault, which is located approximately 5.5 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 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 16 miles southwest of the site. The Hayward and Calaveras Faults are located approximately 14 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 1 on the following page.

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. The Working Group on California Earthquake Probabilities, a panel of experts that are periodically convened to estimate the likelihood of future earthquakes based on the latest science and ground motion prediction modeling, concluded there is a 72 percent chance for at least one earthquake of Magnitude 6.7 or larger in the Bay Area before 2045. The Hayward fault has the highest likelihood of producing an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 14 percent, while the likelihood of a similar event occurring on the San Andreas and Calaveras faults is estimated at approximately 6 and 7 percent, respectively (Working Group, 2015).

Table 1. Earthquake Magnitudes and Historical Earthquakes
Palo Alto Public Safety Building and Parking Garage
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

Earthquake Design Parameters

The State of California requires that all buildings be designed in accordance with the seismic design provisions presented in the 2013 California Building Code and in ASCE 7, “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 D, stiff soil, in accordance with Chapter 20 of ASCE 7-10. Spectral acceleration response parameters S_S and S_1, and site coefficients Fa and Fv, may be taken directly from the U.S.G.S. website based on the longitude and latitude of the site. For the site latitude (37.4269 degrees) and longitude (-122.1431 degrees) and Site Class D, Fa = 1.0, Fv = 1.5, SDs = 1.027 and SD1 = 0.706.

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 our borings and CPT probes following the methods described in the 2008 publication by Idriss and Boulanger titled "Soil Liquefaction During Earthquakes".

The peak ground acceleration (PGA_M) used for our liquefaction analysis was based on information presented on the Probabilistic Seismic Hazards Mapping Ground Motion Page (CGS, 2016), which indicates the maximum considered earthquake acceleration (PGA_M) is 0.618g. The depth to ground water used in our liquefaction analysis was the recommended design high ground water level of 12 feet below existing grade.

The silt and sand layers encountered at the site below a depth of 12 feet and above the maximum depth of our exploration were considered in our liquefaction analysis. Soils with a soil behavior type classified as "clay" and "silty clay to clay" (based on the soil behavior correlations referenced in Appendix A) were considered too clay-rich to liquefy.

Based on our analyses of data from the borings and CPTs, we concluded that medium dense portions of the interbedded sand and sandy silt strata encountered at the site could liquefy during a major earthquake. Our analysis indicates that liquefaction-induced total settlement at the ground surface as a result of the design-level earthquake would be in the range of about ¼- to ¾-inch. We estimate that differential settlement of about ¼- to ½-inch could occur across the basement structures from liquefaction of the underlying sand and sandy silt strata. Liquefaction-induced differential settlement of about ¼- to ¾-inch could occur across on-site buildings supported at-grade, and between the basement structures and the adjacent at-grade buildings.

As part of our work, we also evaluated the potential for earthquake-induced liquefaction of the clays encountered in our borings below the design ground water level using the guidelines described in CDMG Special Publication 117 (1997) and the methods described in the 2006 publication by Bray and Sancio titled "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils." According to Bray and Sancio (2006), fine grained soils need to satisfy the following criteria in order to be considered potentially susceptible to severe strength loss and liquefaction during an earthquake:

$PI < 12$:	$Wc/LL > 0.85$
$12 < PI < 18$:	$Wc/LL > 0.80$
$PI > 18$:	Not susceptible to liquefaction

The results of our laboratory tests and liquefaction evaluation of the clays encountered at the site are presented on Table 2 below. Based on the test results summarized in Table 2 and on our interpretation of the Liquid Limit and plasticity of the soils encountered in the borings in relation to their water content, the screening method by Bray and Sancio (2006), indicates the clays encountered in our borings and CPTs have sufficiently high plasticity and/or sufficiently low water content so as to be considered not susceptible to liquefaction during severe ground shaking.

Table 2: Results of Liquefaction Evaluation of On-site Clays
Palo Alto Public Safety Building and Parking Garage
Palo Alto, California

Boring No.	Layer Depth (ft)	Soil Type	Liquid Limit (%)	Plasticity Index (%)	Water Content (%)	Plasticity Index >18	Water Content > 0.80 x LL	Susceptible To Liquefaction
EB-1	28.5-30	CL	46	20	35	Yes	No	No
EB-2	28.5-30	CL	43	17	32	No	No	No
EB-3	23.5-25	CH	54	28	30	Yes	No	No

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

Geologic Hazards

As part of our investigation, we briefly reviewed the potential for geologic hazards other than liquefaction and lateral spreading, which were discussed above, to impact the site and the proposed buildings 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 be present below 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 proposed structures, as is typical for sites throughout the Bay Area. The Public Safety Building and parking garage 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 above the water table are densified and settle, often unevenly across a site. The soils encountered in our borings and CPTs were generally composed of stiff to hard clay and medium dense to very dense sand. In our opinion, the likelihood of significant differential soil compaction affecting the proposed Public Safety Building and parking garage is low provided the recommendations presented in this report are followed during design and construction.

CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical viewpoint, the site is suitable for the proposed Public Safety Building and parking garage provided the recommendations presented in this report are followed during project design and construction.

The primary geotechnical concerns for the proposed buildings are: the need for temporary shoring of the basement excavations; the likelihood that ground water will be present above the depth of the basement excavations requiring dewatering, depending on the time of year the excavations are made; the need to design and water-proof the floors and walls of the basements and access tunnel for a projected high ground water level of 12 feet below existing grade, and; the likelihood of severe ground shaking at the site during a major earthquake.

Based on an anticipated finished floor elevation of 21 feet below existing grade, the lower level basement floors are expected to bear on firm to stiff clay. For adequate support of the superstructures and basement walls, and to resist hydrostatic uplift pressure on the lower floor of the basements, the Public Safety Building and parking garage should be supported on a reinforced concrete mat foundation. The access tunnel between the proposed basements should also be constructed with a reinforced concrete floor or mat slab. Prior to mat construction, the mat subgrades should be prepared and compacted as recommended in the section of this report titled “Earthwork.” The proposed at-grade retail building may be supported on a conventional spread footing foundation bearing in stiff undisturbed native soil.

We have not provided recommendations regarding the method or details for water-proofing of the basements or access tunnel since design of water-proofing systems is outside of our scope of services and expertise. Installing adequate water-proofing below the lower level mat slabs and tunnel floor, and behind the basement and tunnel walls is essential for the success of the basement and tunnel structures.

We note that portions of the sand strata encountered in the borings and CPTs within the anticipated depth of the basement excavations were judged to have limited cohesion and to be prone to sloughing and/or caving if excavated near-vertical. This information should be considered by the contractor when determining safe temporary slopes and when selecting options for temporary excavation shoring.

Because subsurface conditions may vary from those encountered at the locations of our borings and CPTs, and to observe that our recommendations are properly implemented, we recommend that Romig Engineers be retained to: 1) review the project plans for conformance with our recommendations, and; 2) observe and test during shoring of the basement excavation, preparation of the mat subgrade, and construction of foundations, concrete flatwork, and pavements.

FOUNDATIONS

Basement Mat Foundation

The proposed access tunnel and the basements and basement retaining walls for the Public Safety Building and parking garage should be supported on a reinforced concrete mat foundation bearing on undisturbed native soil. The mat foundations may be designed for an average allowable bearing pressure of up to 3,000 pound per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading. A maximum localized bearing pressure of 4,000 pounds per square foot from dead plus live loads may be used at concentrated column and wall loads. The mat foundations should also be designed to resist hydrostatic uplift pressure resulting from a design ground water level at 12 feet below the existing ground surface.

Mat foundations should be reinforced to provide structural continuity and to permit spanning of local irregularities. A modulus of subgrade reaction (Kv1) of 100 pounds per cubic inch (pci) may be assumed for the mat subgrade. This value is based on a 1-foot square bearing area and should be scaled to account for mat foundation size effects. A modulus of subgrade reaction (Kv) of 20 to 40 pci may be assumed for preliminary design of the mat foundations, and the modulus confirmed when building loads are available.

The surface of the excavation for the basement mat should be cleaned of all loose or soft soil and debris. A member of our staff should observe the basement excavation and evaluate whether scarification and/or surface compaction of the bottom of the excavation is needed. A thin working slab or 6-inch thick section of crushed rock or aggregate base could be placed across the bottom of the prepared and approved mat subgrade, if required by the water-proofing consultant or contractor.

Lateral Loads For Basement Foundations

Lateral loads may be resisted by friction between the water-proofing system below the mat and the supporting subgrade and by passive soil pressure acting against the sides of foundations elements and basement walls. The structural engineer should consult with the water-proofing consultant and/or manufacturer for the coefficient of friction that should be assumed for design. Lateral resistance may also be provided by passive soil pressure acting against the sides of the mat foundation provided the mat is cast neat in a foundation excavation or shored excavation, or backfilled with properly compacted soil. An equivalent fluid pressure of 350 pounds per cubic foot may be assumed for passive soil resistance, where appropriate.

Basement Water-Proofing

We have not provided recommendations regarding the method or details for basement water-proofing since design of water-proofing systems is outside of our scope of services and expertise. Installing adequate water-proofing below and behind the edges of the basement mat slab and behind the basement walls is essential for the success of the basement structure. Placing concrete with a low water:cement ratio should be considered as one step of good damp-proofing as discussed below. The water-proofing system below the basement mat may be placed on a thin working slab or on a layer of crushed rock or aggregate base, as described previously, or on other materials determined to be more appropriate by the water-proofing consultant and the contractor.

Foundations for At-Grade Building

In our opinion, the proposed single-story retail building may be supported on conventional continuous foundations and on isolated spread footings bearing on undisturbed stiff to very stiff native soil. Footings should have a width of at least 15 inches and should be embedded at least 24 inches below exterior finished grade and at least 18 inches below the bottom of concrete slabs-on-grade, whichever provides deeper embedment. Footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 2,500 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 weight of the footings may be neglected for design purposes.

We recommend that isolated footings and portions of continuous footings parallel to the parking garage basement wall be supported on undisturbed native soil below any basement wall backfill. Surcharge pressures from these footings should be applied to the basement walls in accordance with the criteria presented in the section of this report titled “Basement and Tunnel Retaining Walls.” In general, footings and slabs located over basement wall backfill, if any, should be designed to span across the backfill zone.

Footings located adjacent to utility lines should be embedded below a 1:1 plane extending up from the bottom edge of the utility trench. Continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities.

A member of our staff should observe all foundation excavations prior to placement of reinforcing steel to confirm that they have at least the minimum recommended dimensions, expose suitable bearing material, and have been properly cleaned of all loose or disturbed soil and debris. If soft, loose or disturbed soils are encountered in the bottom of the foundation excavations, our field representative will require these soils to be removed before reinforcing steel and concrete is placed.

Lateral Loads for At-Grade Foundations

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. Lateral resistance may also be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations or backfilled with compacted structural fill. We recommend assuming an equivalent fluid pressure of 300 pounds per cubic foot for passive soil resistance, where appropriate. At least the upper foot of passive soil resistance should be neglected where soil adjacent to the footing will be landscaped or subject to softening from rainfall and/or surface water runoff. Passive soil resistance may be assumed to start at the surface where the soil adjacent to the foundation is covered and protected by a concrete slab-on-grade or pavement.

Foundation Settlement

On a preliminary basis, thirty year total settlement of basement mat foundations is expected to be no greater than about ¾-inch and differential settlement across the mats is expected to be about ½-inch under static loading conditions. Thirty year differential settlement due to static loads is not expected to exceed ¾-inch across the at-grade retail building provided the foundations are designed and constructed as recommended. These preliminary settlement estimates can be updated when building loads are available

SLABS-ON-GRADE

General Slab Considerations

To reduce the potential for movement of slab subgrades, at least the upper 6-inches of the slab subgrade should be scarified and compacted at a moisture content above the laboratory optimum. Slab subgrades should be kept moist up until the time the non-expansive fill, crushed rock and vapor barrier, 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 and interior slabs-on-grade should be underlain by a layer of non-expansive fill as discussed below. The non-expansive fill should consist of crushed rock, Class 2 aggregate base, or clayey soil with a Plasticity Index no greater than 15.

Considering the potential for some movement of soil subgrades, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should be given to using a control joint spacing of about 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 6 inches of Class 2 aggregate base. To improve performance, exterior slabs-on-grade, such as for patios, should be constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs and into the underlying base and subgrade.

Interior Slabs

Concrete slab-on-grade floors (other than the basement mat) should be constructed on a layer of non-expansive fill at least 6-inches thick. In areas where dampness of concrete floor slabs would be undesirable, such as within the interior of the retail building, concrete slabs should be underlain by at least 6 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 for this capillary break material. The crushed rock should be densified with vibratory equipment and may be considered as the non-expansive fill recommended above.

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 meeting the minimum ASTM E1745, 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 E1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms, such as 15-mil Stego Wrap Class A, should be used rather than a Class C vapor retarder. The vapor retarder or vapor barrier should be placed directly below the concrete slab. Installation of a layer of sand above the membrane vapor retarder/vapor barrier is not recommended. The vapor retarder/vapor barrier should be installed in accordance with ASTM E1643. 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 would be appropriate to test the slab moisture content for adherence to the manufacturer's requirements and to determine whether a longer drying time is necessary.

BASEMENT AND TUNNEL RETAINING WALLS

We recommend the portion of the basement retaining walls and access tunnel that is above the design ground water depth of 12 feet be designed to resist an equivalent fluid pressure of at least 40 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot (where H is the overall height of the wall in feet). Since the City of Palo Alto will not allow a drainage system to be installed behind the basement or tunnel walls (or below the basement mat foundation), the design lateral pressure on the basement and tunnel walls should be increased by at least 40 pounds per cubic foot (i.e. to at least 80 pcf plus 8H) below the design ground water level. In addition, some provision should be made in design of the basement and tunnel walls to account for undrained wall backfill conditions above the design ground water level.

To account for approximately 6 feet of perched ground water behind the basement and tunnel walls that are above the design ground water level, we recommend adding a line load surcharge of 720 pounds per lineal foot behind the basement and tunnel walls. Since perched water conditions could develop at various depths behind the basement and tunnel walls, we recommend the line load surcharge be applied at various depths (such as mid-depth between floors) to check the wall design for perched water conditions. Where the basement and tunnel walls will be subjected to surcharge loads, such as from foundations, construction loading, or traffic on adjacent streets, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Based on peak ground acceleration at the site, Seed and Whitman (1970), Al Atik and Sitar (2010), and Lew et al. (2010), seismic loads on retaining walls that cannot yield, such as the basement and tunnel walls, may be simulated by a line load of $10H^2$ (in pounds per foot, where H is the overall height of the wall in feet). This seismic surcharge line load may be assumed to act at 1/3H above the base of the wall (in addition to the drained active wall design pressure of 40 pounds per cubic foot above the design ground water level, and in addition to the undrained active pressure of 80 pounds per cubic foot below the design ground water level). This seismic surcharge on the basement and tunnel walls may also be applied as an equivalent fluid pressure of 20 pounds per cubic foot.

Backfill (if any) behind the basement and tunnel 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 may need to be temporarily braced.

The basement and tunnel retaining walls should be supported on a mat foundation designed and constructed in accordance with the recommendations presented previously.

EXCAVATION SHORING

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

Due to the anticipated variation of the on-site soils, field modification of temporary cut slopes may be required. Unstable materials encountered on slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination. Protection of structures, streets, and utilities around and near utility trenches and basement and tunnel excavations will be the responsibility of the contractor. A pre-construction survey and daily monitoring of the excavation shoring system and the streets and structures around the basement and tunnel excavations is recommended.

We anticipate the temporary basement excavation shoring system will consist of tied-back soldier beams and lagging or soil nails with shot-crete facing. Basement excavation and dewatering is assumed to produce drained conditions behind the basement shoring system. In our opinion, tied-back soldier beam and lagging shoring should be designed to resist an at-rest pressure simulated by a uniform lateral pressure of at least 28H, where H is the depth of the basement excavation. This design pressure may be assumed to decrease uniformly to zero above the upper row of tie-backs.

Design and construction of excavation shoring should be performed in a manner that will control lateral deflection of the excavation sidewalls and differential movement of the ground and streets adjacent to the basement and tunnel excavations to an acceptable level. This may require tensioning of tie-backs and/or internal bracing of soldier beams.

If overlapping soil:cement columns are used between soldier beams in lieu of lagging, or to reduce ground water flow into the excavation, the lateral soil pressures recommended for design of the basement walls should be used for shoring design with the depth to the start of undrained pressure based on the ground water level during construction.

Allowable skin friction support on soldier beam piers is estimated to be about 450 pounds per square foot from the clays and sands below the bottom of the excavation (although frictional support in the sands will depend on the method of pier construction, the depth that ground water is lowered, and the amount of disturbance of the sand layers caused by pier construction). An allowable end-bearing capacity of up to 6,000 psf may be included in calculations of vertical pier capacity provided the contractor uses good workmanship and removes all drilling spoils from the pier excavations. Some amount of downward movement of the shoring piers, perhaps one-half to one inch or so, will occur as the loads are applied to the on-site soils. The amount of downward pier deflection will depend on the condition of the bottom and sides of the pier holes at the time the concrete is placed.

The shoring designer should consider the presence and condition of the firm to stiff, silty clay and/or clayey silt that was encountered in the CPTs at a depth of about 20 to 28 feet below grade when selecting passive soil pressure and determining the minimum required depth of shoring embedment below the bottom of the excavations.

If soils nails are used for temporary support of the basement excavation, the following design parameters may be assumed: soil unit weight = 120 pcf; friction angle = 22 degrees; cohesion = 600 psf, and ultimate bond stress = 1,600 psf. Bond stress should be confirmed by the shoring contractor by load testing of the nails during construciotn.

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as existing pavements, utilities to be abandoned, vegetation, root systems, surface fill, 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 soils, foundation and utility trench excavations, and slab and pavement subgrades should be kept in a moist condition throughout the construction period.

A member of our staff should observe and evaluate the basement excavation to determine whether scarification and/or compaction of the excavation bottom is needed.

If a temporary ramp is constructed to access the basement excavations, the ramp should be properly backfilled with compacted on-site soil as recommended in this report for structural fill. A member of our staff should observe and test during backfilling of temporary entrance ramps.

Material for Fill

On-site soil containing less than 3 percent organic material by weight (ASTM D2974) should be suitable for use as structural fill (but not for non-expansive 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 in maximum dimension. 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 evaluate and approve proposed import materials prior to their delivery to the site.

Temporary Dewatering of Basement Excavations

During subsurface exploration at the site on March 31, 2016 and April 8, 2016, ground water was encountered at depths of approximately 19½ to 26½ feet below existing grade. Ground water was reportedly present at depths of approximately 21 to 23 feet below grade prior to the start of dewatering of the basement excavation at 385 Sherman Avenue southeast of the proposed parking structure. The historic high ground water level in the area of the site is approximately 17 feet below grade according to Seismic Hazards Zone Report 111 for the Palo Alto Quadrangle. The ground water depth in the area of the site projected from monitoring well data varied from about 14.3 to 21 feet below grade during the period from 2005 to 2013, with seasonal water depth fluctuations of about 4 to 6 feet.

Based on the available ground water depth information, temporary dewatering of the anticipated 23 to 24 foot deep basement excavations should be anticipated. In our opinion, it would be desirable for dewatering to draw-down and maintain the ground water level at least 2 feet below the bottom of the basement excavations during construction. Selection of equipment and methods of dewatering should be left up to the contractor. Ground water observation wells could be installed on or near the site to confirm and monitor the depth to ground water prior to and during construction.

Special considerations, such as temporary storage, testing for environmental quality, and/or water treatment under permit, may be required prior to discharge of ground water from dewatering of the site during construction. It would be possible to reduce the volume of ground water the flows laterally into the basement excavations during dewatering by installing overlapping soil/cement columns as part of the excavation shoring system around the perimeter of the basement excavations.

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 3 below. The relative compaction and moisture content recommended in Table 3 is relative to ASTM Test D1557, latest edition.

Table 3. Compaction Recommendations
Palo Alto Public Safety Building and Parking Garage
Palo Alto, California

	Relative Compaction*	Moisture Content*
General		
• Scarified subgrade in areas to receive structural fill or slabs.	90 percent	Above optimum
• Structural fill composed of on-site soil.	90 percent	Above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
• Structural fill below a depth of 5 feet.	92 percent	Above optimum
Vehicle Pavement Areas		
• Upper 6-inches of soil below aggregate base.	95 percent	Near optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soil.	90 percent	Above optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Surface Drainage

Finished grades should be designed to prevent ponding of water and to drain surface water runoff 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 structures, where possible. At a minimum, roof downspouts should be arranged to discharge onto hardscape or concrete splash blocks that are sloped to promote drainage of roof water away from perimeter foundations.

Storm water 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

Plan Review

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations presented in this report. We should be provided with these plans as soon as possible upon their completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review process.

We expect the City of Palo Alto will require Romig Engineers to prepare a "clean" geotechnical plan review letter for this project prior to their approval of the plans for construction. 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 that the following note be added to the plans.

"All earthwork, excavation shoring, slab subgrade preparation, foundation and slab construction, basement and tunnel wall construction and backfilling, pavement construction, and site drainage should be performed as recommended in the geotechnical report, dated May 6, 2016, prepared by Romig Engineers, Inc. Romig Engineers should be notified at least 48 hours in advance of any earthwork, excavation shoring, foundation, slab, or pavement construction, and should observe and test during earthwork, excavation shoring, and foundation, slab, and pavement construction as recommended in the geotechnical report."

Construction Observation and Testing

All earthwork, excavation shoring, foundation, slab, and pavement construction should be observed and tested by Romig Engineers to: 1) confirm that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the project 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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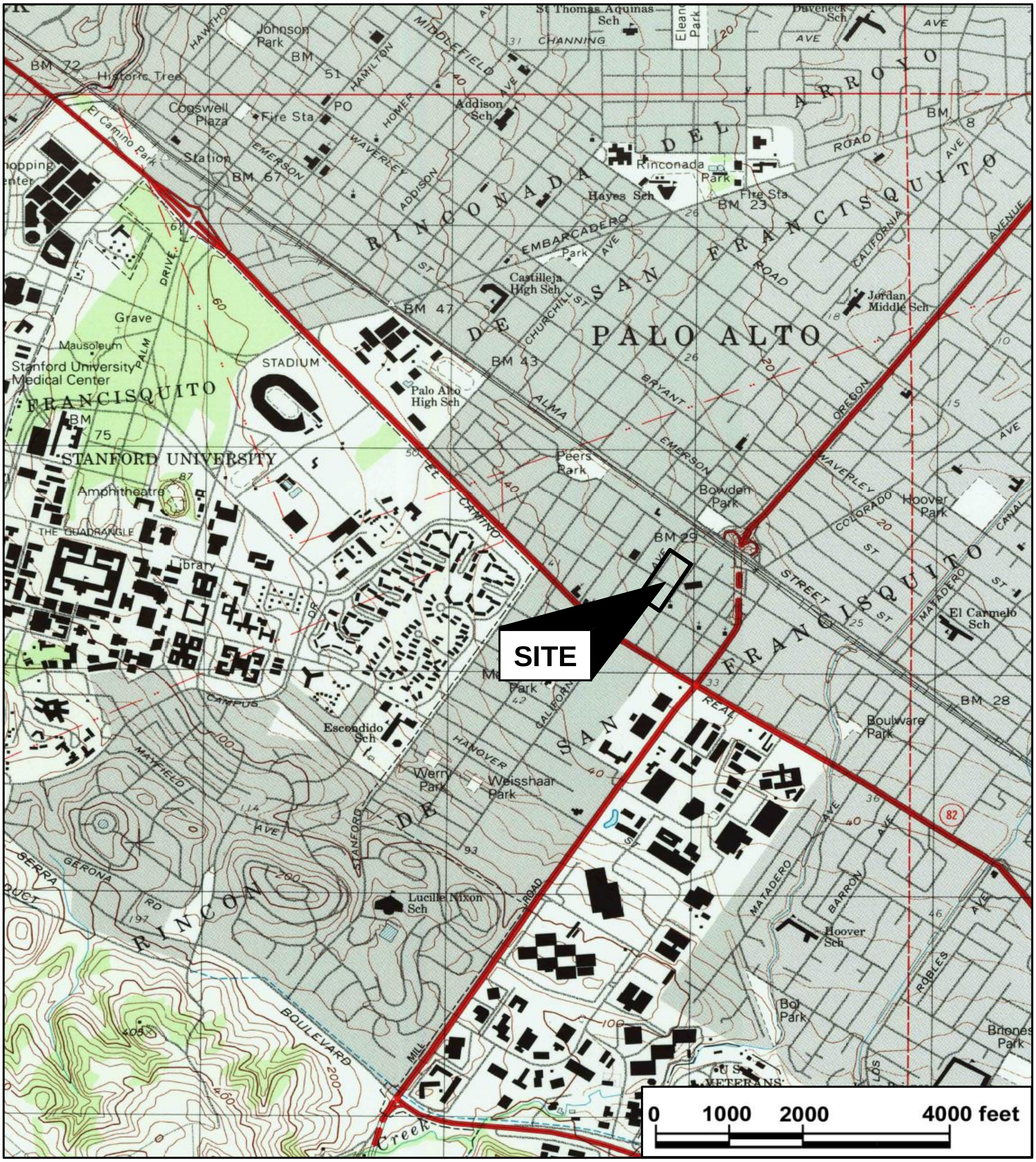
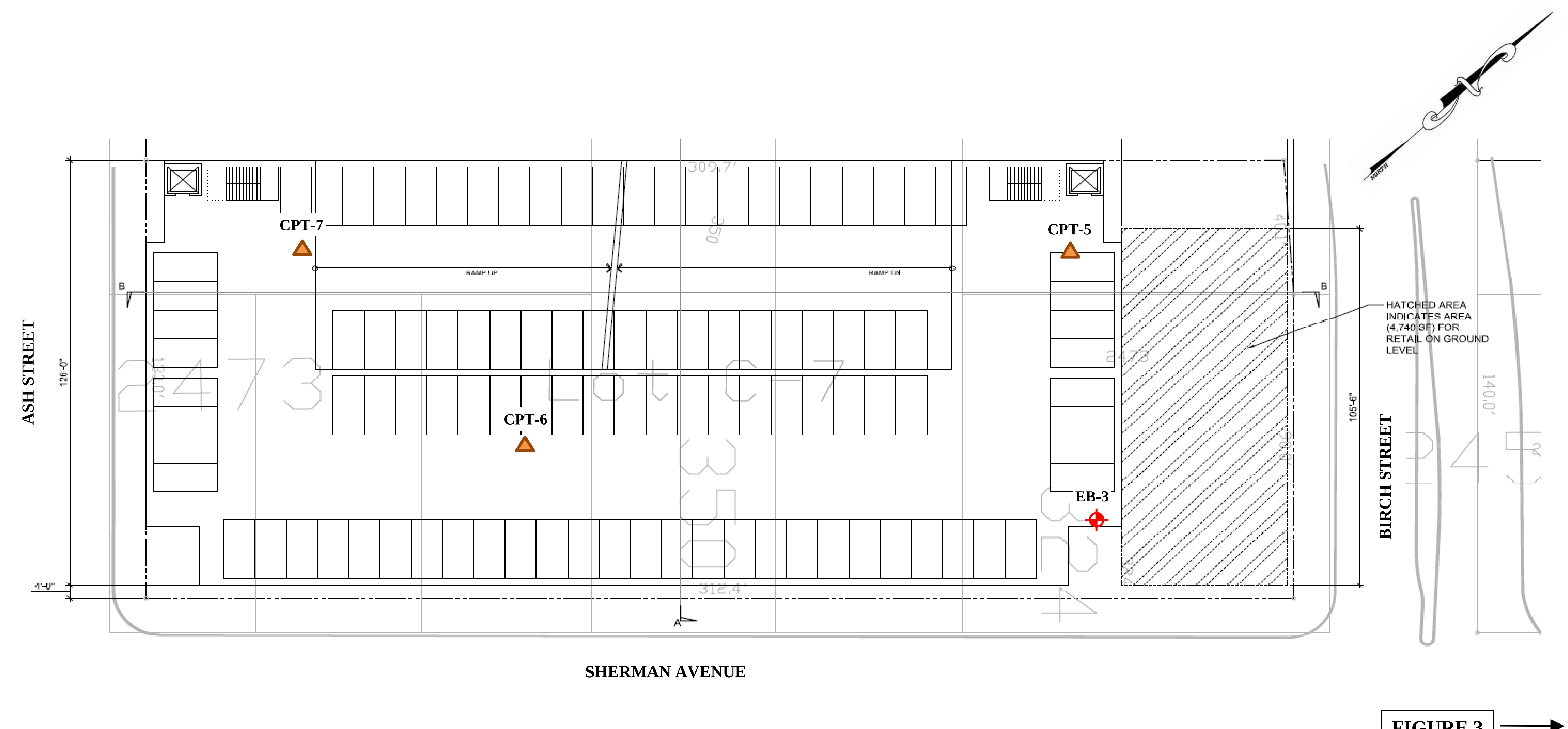


Scale: 1 inch = 2000 feet
Base is United States Geological Survey Palo Alto 7.5 Minute Quadrangle, dated 1997.

VICINITY MAP **FIGURE 1**
CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE MAY 2016
PALO ALTO, CALIFORNIA PROJECT NO. 3723-1



LEGEND

CPT-7 Approximate Location of Cone Penetrometer Test.

EB-3 Approximate Location of Exploratory Boring.

Approximate Scale: 1 inch = 30 feet.

Base is Typical Level Parking Plan, dated September 22, 2015, prepared by Watry Design, Inc.

CONCEPTUAL SITE PLAN - PARKING GARAGE
CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE
PALO ALTO, CALIFORNIA

FIGURE 2
MAY 2016
PROJECT NO. 3723-1

ROMIG ENGINEERS, INC.

LEGEND

CPT-4 Approximate Location of Cone Penetrometer Test.

EB-2 Approximate Location of Exploratory Boring.

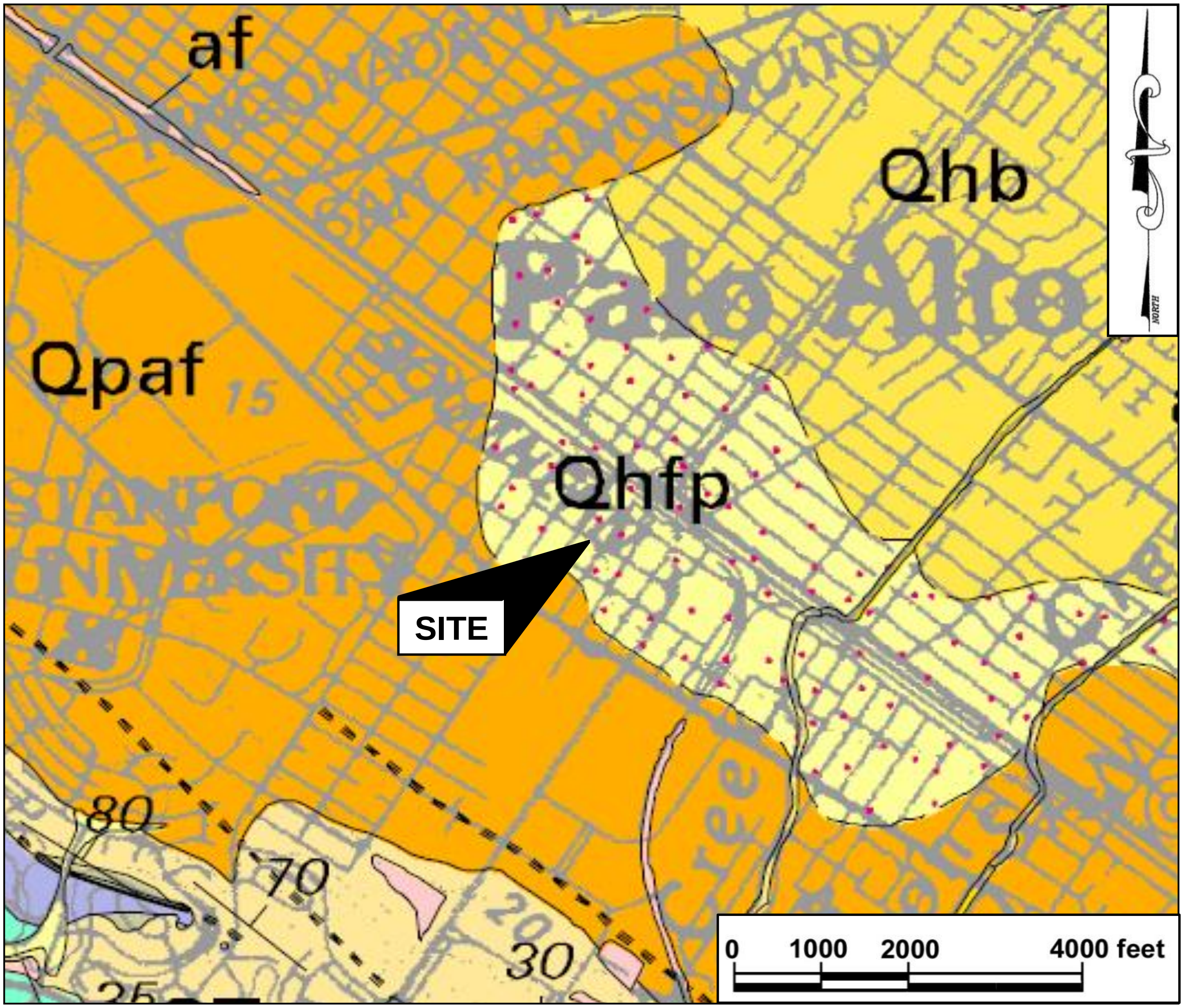
Approximate Scale: 1 inch = 30 feet.

Base is site plan, dated August 31, 2015, prepared by Ross Drulis Cusenbery Architecture.

CONCEPTUAL SITE PLAN - PUBLIC SAFETY BUILDING
CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE
PALO ALTO, CALIFORNIA

FIGURE 3
MAY 2016
PROJECT NO. 3723-1

ROMIG ENGINEERS, INC.



LEGEND

af	Artifical fill		Geologic Contact - dashed where approximate, dotted where inferred.
Qhb	Basin deposits		
Qhfp	Floodplain deposits	?	Fault - dashed where approximate, dotted where inferred.
Qpaf	Alluvial fan and fluvial deposits	35	Strike and dip of bedding

Base is Geologic Map of Palo Alto 30 x 60 Minute Quadrangle (Brabb, Graymer, and Jones, 2000).

VICINITY GEOLOGIC MAP **FIGURE 4**
CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE MAY 2016
PALO ALTO, CALIFORNIA PROJECT NO. 3723-1

APPENDIX A

FIELD INVESTIGATION

Subsurface exploration at the site was performed by means of three exploratory borings and seven Cone Penetration Test (CPT) probes. The CPT probes were performed by Middle Earth Geo Testing, Inc. using a 25 ton truck with an integrated electronic cone system. The CPT soundings were performed in accordance with ASTM D5778-95. The cone that was used had a tip area of 10 cm^2 and friction sleeve area of 150 cm^2. Logs of the CPT probes are included in this Appendix.

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

Several tests were performed in the field during drilling and sampling. 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 diameter 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 plans that were provided to us. The CPT and boring locations should be considered accurate only to the degree implied by the method used.

The CPT logs, the exploratory boring logs, and the 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**

CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE MAY 2016

PALO ALTO, CALIFORNIA PROJECT NO. 3723-1

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 21.6 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
3-inches asphalt concrete.		AC		0					
Brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity, small roots.	Stiff	CL							
■ Liquid Limit = 34, Plasticity Index = 15.						12	27	0.5	1.5
Brown to light brown, fine to coarse grained sand.	Hard					42	14		>4.5
Some manganese oxide staining.				5		42	17		>4.5
Light brown, increased sand.									
Decreased sand and gravel.	Stiff to Very Stiff			10		17	24		
Fine to medium grained sand.				15		35	19		2.0
Moderate plasticity.				20		18	22		1.5
Continued on next page.									

EXPLORATORY BORING LOG EB-1 **BORING EB-1**

CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE PAGE 1 OF 3

PALO ALTO, CALIFORNIA MAY 2016

PROJECT NO. 3723-1

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 21.6 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

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 to light brown, Sandy Lean Clay, very moist, fine to medium grained sand, moderate plasticity.	Very Stiff	CL		20					
▼ Ground water encountered during drilling at 21.6 feet.				▼					
Becomes light brown to light gray, fine grained sand.						18	26	1.1	2.5
				25					
■ Liquid Limit = 46, Plasticity Index = 20.									
				30		17	35		1.0
Brown, Poorly Graded Sand, moist, fine to coarse grained sand, fine angular to rounded gravel.	Very Dense	SP							
				35		50/5"	17		
				40		63	17		
Continued on next page.									

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 angular to rounded gravel.	Very Dense	SP		40					
						61	15		
Bottom of Boring at 44.5 Feet.				45					
				50					
				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.

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 26.6 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
3-inches asphalt concrete.		AC		0					
Brown, Sandy Lean Clay, moist, fine to coarse grained sand, low plasticity, roots.	Stiff to Very Stiff	CL				13	15		3.0
Brown to light brown, fine angular to rounded gravels, light orange mottling.	Hard			5		35	11		>4.5
Brown, Clayey Sand and gravel, slightly moist, fine to coarse grained sand, fine angular to rounded gravel.	Very Dense to Dense	SC				61	10		
				10	●	48	10		
● 22% Passing No. 200 Sieve.									
Light brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity, light orange mottling.	Very Stiff	CL		15		17	14		
				20		25	19	0.6	2.5

Continued on next page.

EXPLORATORY BORING LOG EB-2 **BORING EB-2**

CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE PAGE 1 OF 3

PALO ALTO, CALIFORNIA MAY 2016

PROJECT NO. 3723-1

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 26.6 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

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)*
Light brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity, light orange mottling.	Very Stiff	CL		20					
Becomes light brown, moderate plasticity.						17	24	0.8	
				25					
▼ Ground water encountered during drilling at 26.6 feet.				▼					
■ Liquid Limit = 43, Plasticity Index = 17.				30		19	32		
Brown, Clayey Sand, wet, fine to coarse grained sand.	Very Dense	SC							
				35		70	21		
Increased low plasticity fines.									
● 25% Passing No. 200 Sieve.	Medium Dense			40		29	21		
Continued on next page.									

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 26.6 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

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, moist, fine to coarse grained sand.	Medium Dense	SC		40					
Fine rounded to angular gravel.	Dense					39	23		
Bottom of Boring at 44.5 Feet.				45					
				50					
				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.

EXPLORATORY BORING LOG EB-2 **BORING EB-2**

CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE PAGE 3 OF 3

PALO ALTO, CALIFORNIA MAY 2016

PROJECT NO. 3723-1

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 23.5 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
4-inches asphalt concrete.		AC		**0**					
Brown, Clayey Sand, slightly moist, fine to coarse grained sand, fine angular to rounded gravel.	Medium Dense	SC				25	7		
Increased gravel, light orange mottling.	Very Dense					77	7		
				5					
						62	11		
	Medium Dense			**10**		28	10		
● 32% Passing No. 200 Sieve.	Very Dense			**15**	●	40	5		
Light brown, Sandy Lean Clay, moist, fine to medium grained sand, moderate plasticity, light orange mottling.	Stiff to Very Stiff	CL		**20**		17	22	0.8	

Continued on next page.

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 23.5 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

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)*
Light brown, Sandy Lean Clay, moist, fine to medium grained sand, moderate plasticity, light orange mottling.	Stiff to Very Stiff	CL		20					
Olive brown, Sandy Fat Clay, very moist, fine to medium grained sand, high plasticity, brown mottling. ▼ Ground water encountered during drilling at 23.5 feet. ■ Liquid Limit = 54, Plasticity Index = 28.	Stiff to Very Stiff	CH		▼ 25		19	30	1.5	
				30		16	33		
Brown, Poorly Graded Sand, moist, fine to coarse grained sand, fine angular to rounded gravel, low plasticity fines.	Very Dense	SP		35		50/6"	12		
	Dense			40		38	23		
Continued on next page.									

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 23.5 Feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 04/08/2016

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)*
Gray, Sandy Lean Clay, moist, fine to medium grained sand, moderate plasticity.	Hard	CL		40					
						33	27		
				45					
Bottom of Boring at 44.5 Feet.									
				50					
				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.

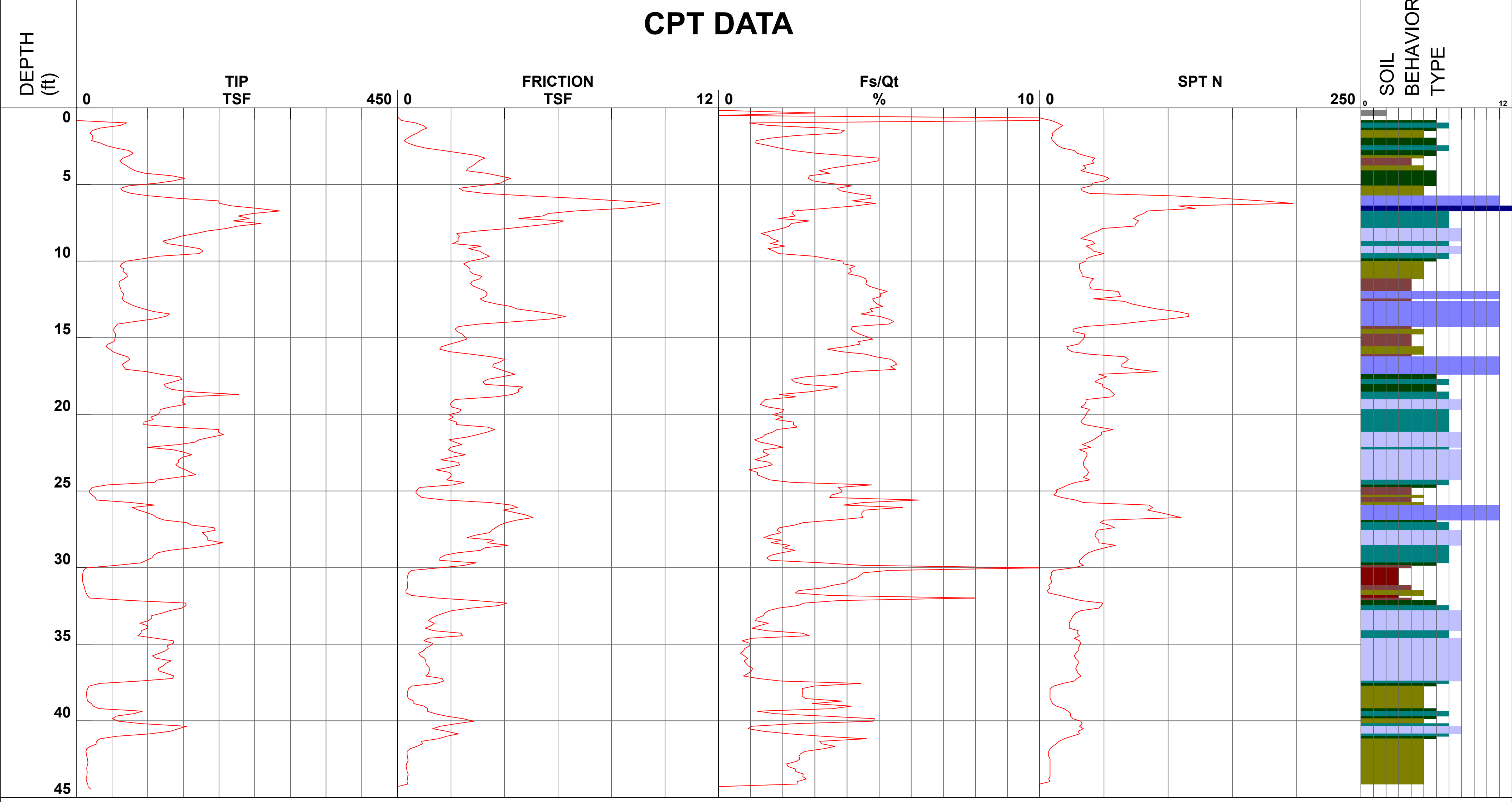
EXPLORATORY BORING LOG EB-3 **BORING EB-3**
CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE
PALO ALTO, CALIFORNIA

MAY 2016
PROJECT NO. 3723-1

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(667).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-01	Date and Time	3/31/2016 8:31:11 AM	Maximum Depth	44.45 ft
EST GW Depth During Test		22.10 ft			

Net Area Ratio .8



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

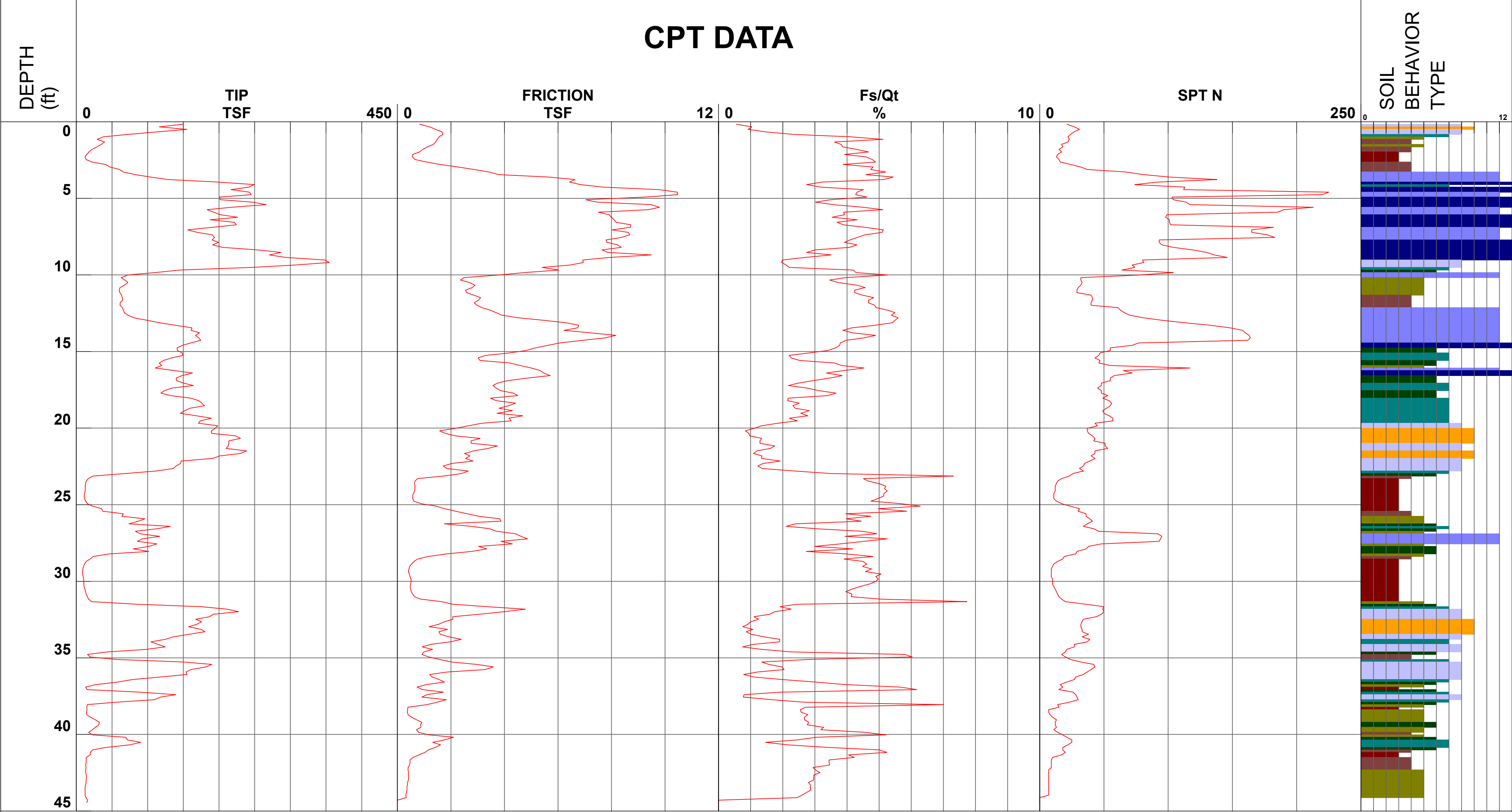
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(668).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-02	Date and Time	3/31/2016 9:33:24 AM	Maximum Depth	44.45 ft
EST GW Depth During Test			22.00 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

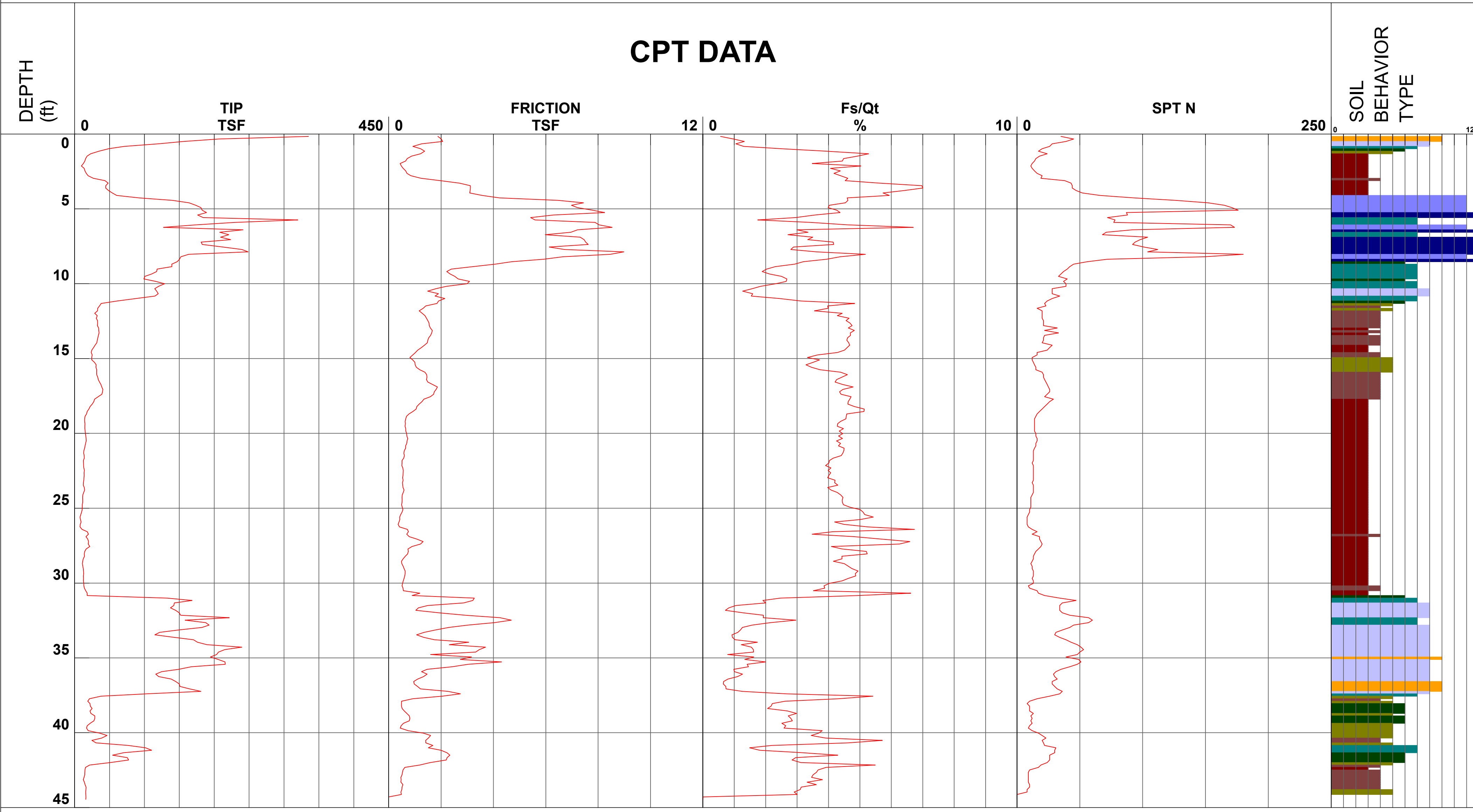
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(669).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-03	Date and Time	3/31/2016 10:17:28 AM	Maximum Depth	44.45 ft
EST GW Depth During Test		19.60 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

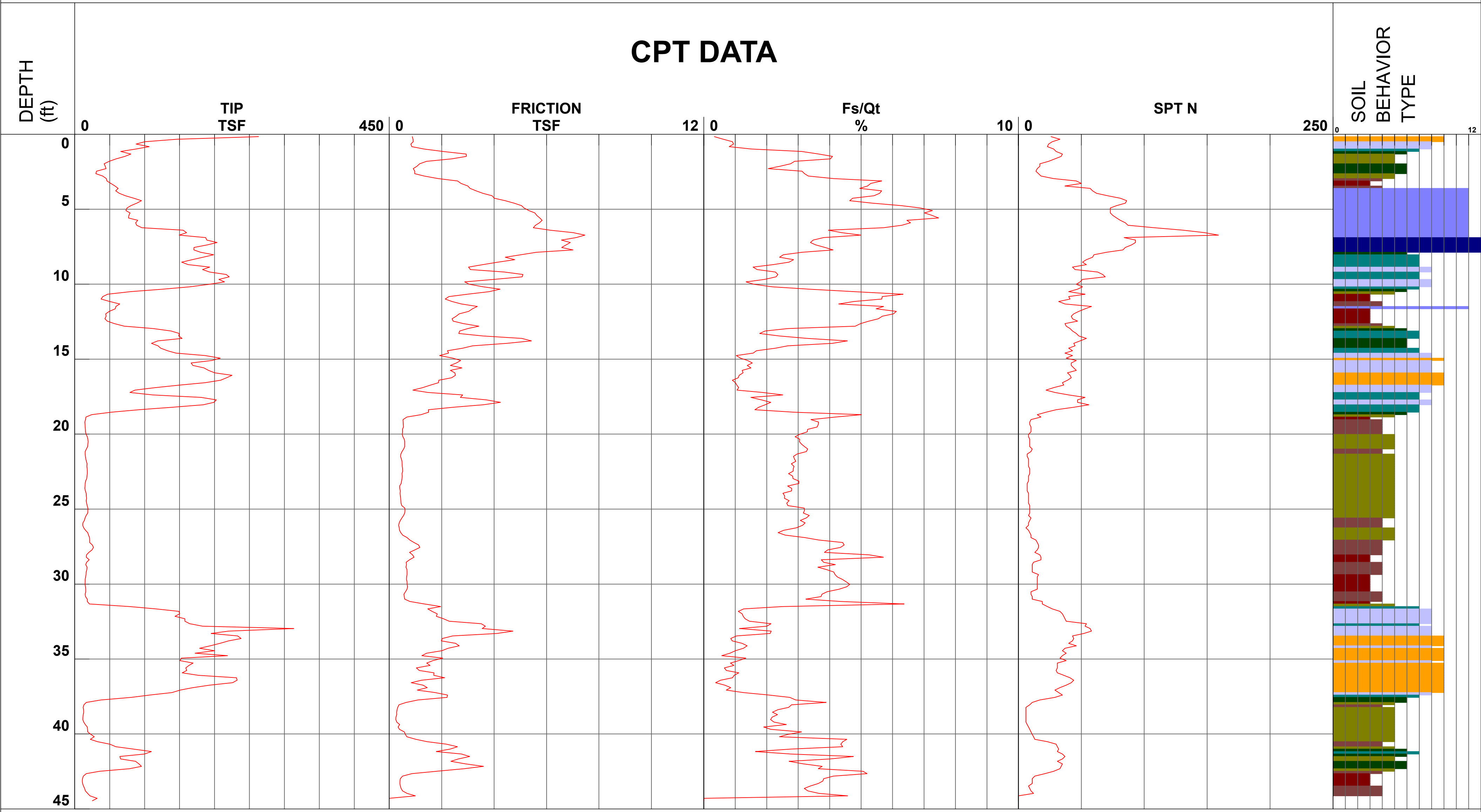
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(670).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-04	Date and Time	3/31/2016 11:07:23 AM	Maximum Depth	44.45 ft
EST GW Depth During Test			22.00 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 (*)

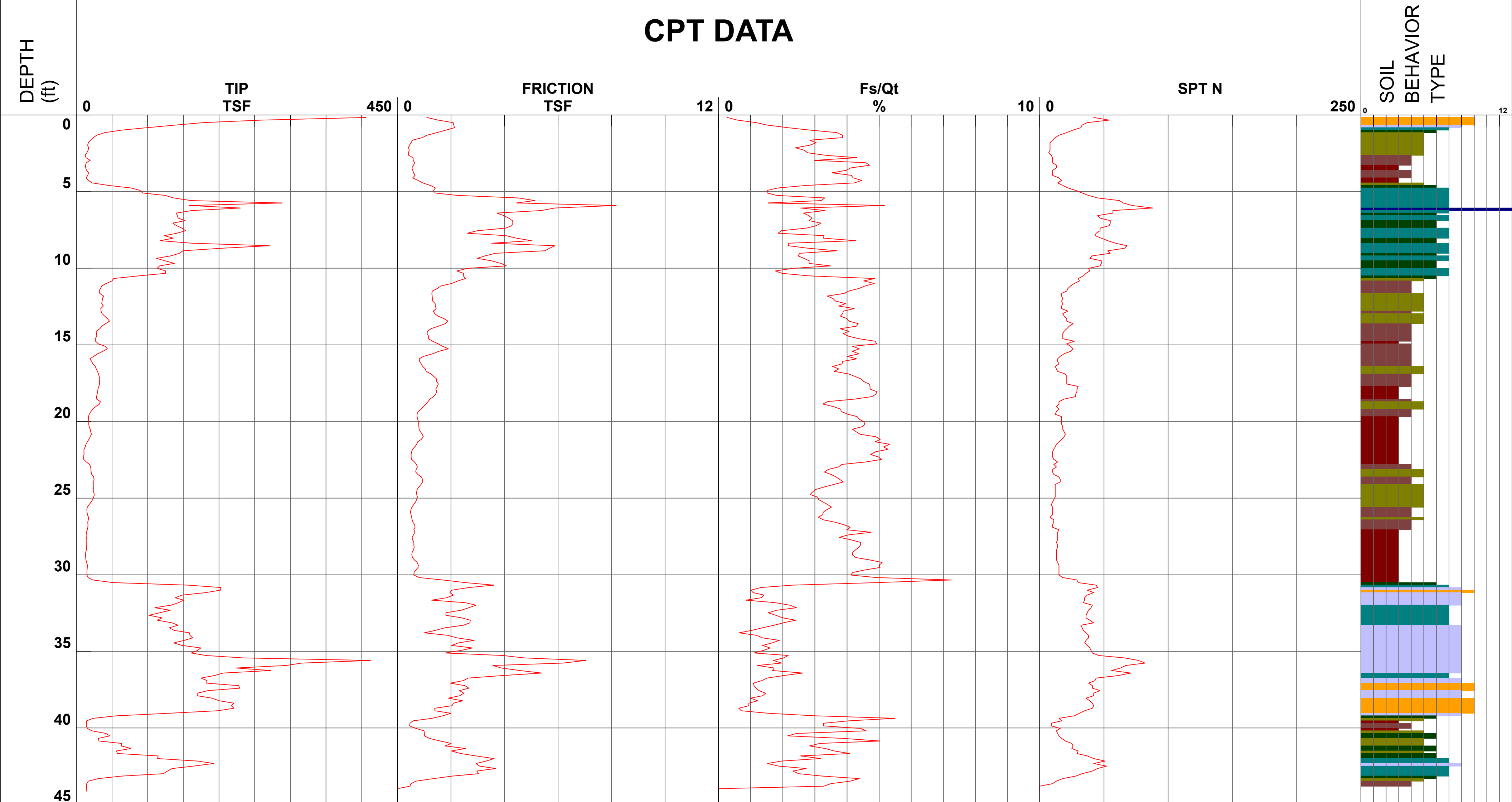
Cone Size 10cm squared

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

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(671).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-05	Date and Time	3/31/2016 12:12:08 PM	Maximum Depth	44.13 ft
EST GW Depth During Test		21.90 ft			

Net Area Ratio .8



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

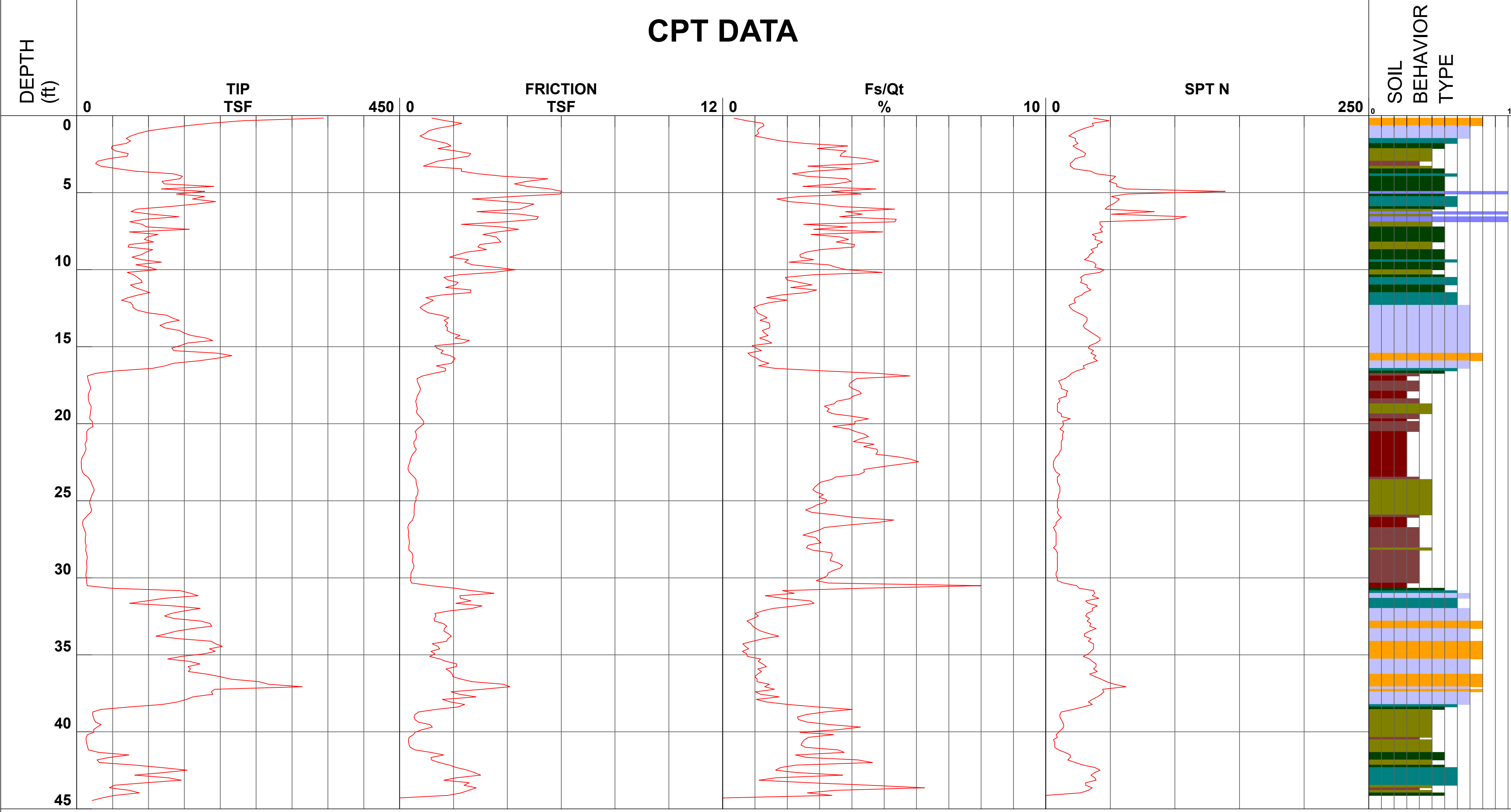
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(672).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-06	Date and Time	3/31/2016 1:11:28 PM	Maximum Depth	44.45 ft
EST GW Depth During Test		22.00 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 (*)

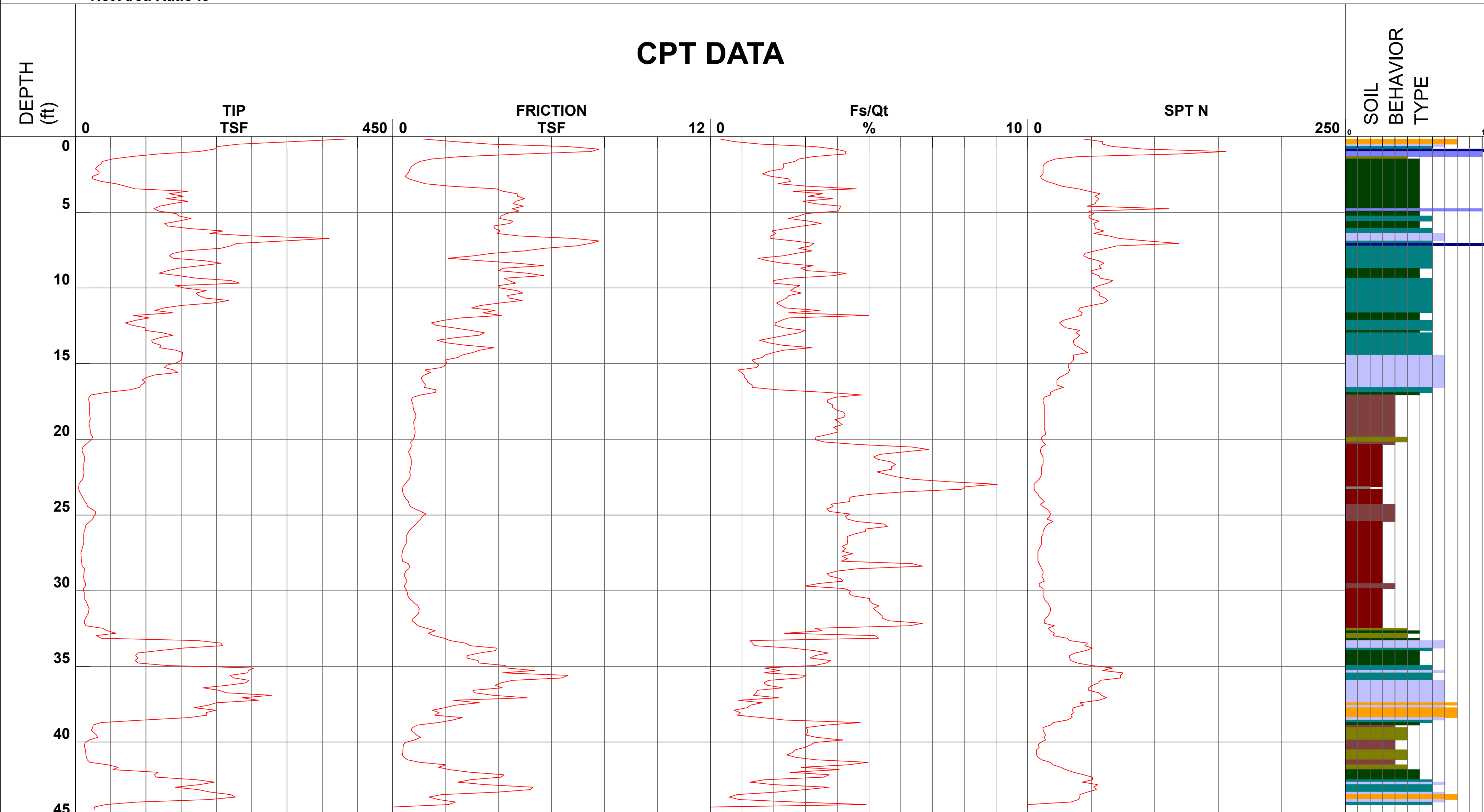
Cone Size 10cm squared

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

Romig Engineering

Project	PA Public Safety Building - Garage	Operator	BH-JH	Filename	SDF(673).cpt
Job Number	3723-1	Cone Number	DDG1350	GPS	
Hole Number	CPT-07	Date and Time	3/31/2016 1:55:08 PM	Maximum Depth	44.45 ft
EST GW Depth During Test		23.90 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 TESTS

Samples collected during subsurface exploration were selected for tests to help evaluate the physical and engineering properties of the soils encountered at the site. The tests that were performed are briefly described below.

The natural moisture content was determined in accordance with ASTM Test D2216 on nearly all 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 depth.

Atterberg Limits were determined on four samples of soil in accordance with ASTM Test 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 and on the boring logs at the appropriate sample depths.

The amount of silt and clay-sized material present was determined on three samples of soil in accordance with ASTM Test D422. The results of these tests are presented on the boring logs at the appropriate sample depths.

❖ ❖ ❖ ❖ ❖

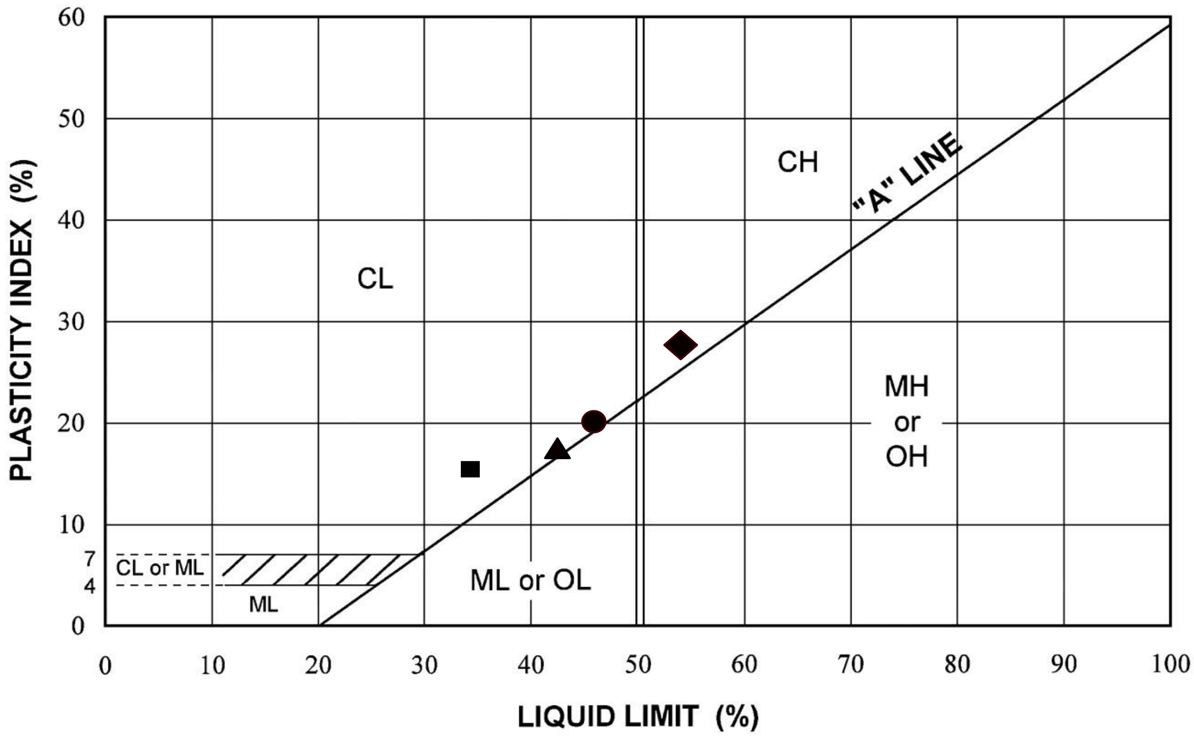


Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	1-2.5	27	34	15	53		CL
●	EB-1	28.5-30	35	46	20	15		CL
▲	EB-2	28.5-30	32	43	17	35		CL
◆	EB-3	23.5-25	30	54	28	14		CH

PLASTICITY CHART **FIGURE B-1**
CITY OF PALO ALTO PUBLIC SAFETY BUILDING AND GARAGE MAY 2016
PALO ALTO, CALIFORNIA PROJECT NO. 3723-1