Appendix C: Geotechnical Investigation

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GEOTECHNICAL INVESTIGATION
FOR
MIXED-USE BULDING
1214 DONNELLY AVENUE
BURLINGAME, CALIFORNIA 94010

August 2016

Prepared for

Mr. John Britton
c/o W.J. Britton & Co.
1345 Mission Street
San Francisco, California 94103

Project No. 3804-1

ROMIG ENGINEERS, INC.
GEOTECHNICAL & ENVIRONMENTAL SERVICES

ROMIG ENGINEERS, INC.
GEOTECHNICAL & ENVIRONMENTAL SERVICES

August 8, 2016
3804-1

Mr. John Britton
c/o W.J. Britton & Co.
1345 Mission Street
San Francisco, California 94103

RE: GEOTECHNICAL INVESTIGATION
MIXED-USE BUILDING
1214 DONNELLY AVENUE
BURLINGAME, CALIFORNIA

Dear Mr. Britton:

In accordance with your request, we have performed a geotechnical investigation for your proposed mixed-use building to be constructed at 1214 to 1220 Donnelly Avenue in Burlingame, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents our geotechnical recommendations for the proposed construction.

We refer you to the report for our specific recommendations.

Thank you for the opportunity to work with you on this project. If you have any questions concerning our investigation, please call.

Very truly yours,

ROMIG ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
MIXED-USE BUILDING
1214 DONNELLY AVENUE
BURLINGAME, CALIFORNIA 94010

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AUGUST 2016

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GEOTECHNICAL INVESTIGATION
FOR
MIXED-USE BUILDING
1214 DONNELLY AVENUE
BURLINGAME, CALIFORNIA

INTRODUCTION

We are pleased to present this geotechnical investigation report for your proposed mixed-use building to be constructed at 1214 to 1220 Donnelly Avenue in Burlingame, California. The location of the 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 the proposed construction.

Project Description

The project consists of constructing a mixed-use building at your three combined Burlingame properties. The three-story building will have a footprint of approximately 13,924 square feet. The ground floor will be retail space at the front and parking at the rear and the second and third floors will be residential units. The mixed-use building is expected to have an elevator at its east corner. Permeable pavers are planned for the driveway and exterior flatwork at the front of the building. The existing structures on the properties will be demolished prior to construction. Structural loads are expected to be light to moderate as is typical for this type of construction.

Scope of Work

The scope of work of this investigation was presented in detail in our agreement with Mr. John Britton, dated June 9, 2016. In order to accomplish this work, we have performed the following services:

- Review of geologic, geotechnical, and seismic conditions in the vicinity of the site.
- Subsurface exploration consisting of four cone penetration tests (CPT) within the footprint of the proposed mixed-use building.
- Laboratory testing of a near-surface sample to aid in soil classification and to help evaluate the engineering properties of the surface soil.

- Engineering analysis and evaluation of the subsurface data to develop geotechnical design criteria.

- 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 Mr. John Britton for specific application to developing geotechnical design criteria for the proposed mixed-use building at 1214 to 1220 Donnelly Avenue in Burlingame, California. We make no warranty, expressed or implied, for the services we performed for this project. Our services are performed in accordance with 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 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 EXPLORATION AND RECONNAISSANCE

Our site reconnaissance and subsurface exploration were performed on July 1, 2016. The subsurface exploration consisted of advancing four CPTs to depths ranging from 47.2 to 50.5 feet. 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. In addition, a near surface soil sample P-1 was collected using hand-auger equipment. The approximate locations of the CPTs and P-1 are shown on the Site Plan, Figure 2. The CPT logs are attached in Appendix A. The results of our laboratory testing of near-surface sample P-1 is attached in Appendix B.

Surface Conditions

The site is located in a residential and commercial area at the northwest side of Donnelly Avenue. At the time of our investigation, the site was occupied by two, 2-story, wood-frame residences that had wood siding exteriors. A fire damaged residence with only the foundation remaining was located between the two residences. Two accessory structures were located at the rear of 1218 and 1220 Donnelly Avenue. A majority of the remaining site areas were covered with concrete driveways, parking spaces, and exterior flatwork. The properties were relatively flat. At 1214 Donnelly Avenue, a patch of native grass and a large tree were present at the front of the residence.

We expect that the residences are supported on conventional shallow foundations, although the depth and width of the existing foundations are unknown. Due to the wood siding, the exterior stem wall was not visible at 1214 Donnelly Avenue. At 1218 and 1220 Donnelly Avenue, cracks up to about 1/2-inch wide were observed at the exterior stem walls. Cracks up to about 1/4-inch wide were observed at the concrete pavement and exterior flatwork. Roof downspouts generally discharged adjacent to the perimeter foundations.

Subsurface Conditions

At the locations of our CPTs, we generally encountered stiff to hard lean clay, clayey silt and sandy silt throughout the subsurface profile. In CPT-1, CPT-3, and CPT-4, we encountered sand strata between depths of 23 to 37, 22 to 30, and 15 to 28 feet, respectively. In CPT-2, we encountered sand interbeds between depths of 14 to 16, 22 to 32, 38 to 42, and 48 to 50 feet. In addition, firm to stiff clays were encountered between the depths of about 1.5 to 7 feet in CPT-3 and CPT-4.

A Liquid Limit of 31 and a Plasticity Index of 11 were measured on a sample of near-surface soil from sample location P-1. These test results indicate that the surface soil at the site has low plasticity and a low potential for expansion.

We note that portions of the granular and silty soils strata encountered at the site may be susceptible to liquefaction during strong seismic shaking. Details of our liquefaction evaluation are included in the section below titled “Liquefaction Evaluation.”

Ground Water

At the time of our exploration, ground water was estimated to be present at a depth of about 14 feet below grade at our CPTs based on the pore pressure response observed during testing. Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, landscaping, underground drainage patterns, and other factors. It is also possible that perched ground water conditions could develop in the soils during and after significant rainfall or due to landscape watering at the property and the upslope areas.

GEOLOGIC SETTING

We have briefly reviewed our local experience and geologic literature pertinent to the area of the site. The information that we reviewed indicates the site is underlain by Pleistocene-age alluvial fan and fluvial deposits, Qpaf (Brabb, Graymer and Jones, 2000). These deposits are generally expected to consist of brown dense gravely and clayey sand or clayey gravel that becomes finer grained upward typically transitioning to sandy clay. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

The lot and the immediate site vicinity are located in an area that slopes very gently to the north and northeast (approximately 10 feet vertically per 1,000 feet laterally, although locally the topography may be steeper). The site is located at an elevation of approximately 30 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 is considered probable. The closest active fault is the San Andreas fault, located approximately 2.7 miles southwest of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is low.

The San Francisco Bay Area is, however, 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 9 miles southwest of the site. The Hayward and Calaveras faults are located approximately 16 and 24 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, and are shown on the Regional Fault and Seismicity Map, Figure 4.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Mixed-Use Building
Burlingame, 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. 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 an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 14 percent, while the likelihood on the San Andreas and Calaveras faults is estimated at approximately 6 and 7 percent, respectively (Working Group, 2015).

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 Response Acceleration 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 (37.5796) and longitude (-122.3477) and Site Class D, Fa = 1.0, Fv = 1.5, SDs = 1.375 and SD1 = 0.975.

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 soil at the site, we performed a liquefaction analysis of the CPT data from our investigation following the methods described in the 2008 publication by Idriss and Boulanger titled "Soil Liquefaction During Earthquakes". The loose to medium dense sands and silty soils encountered below the assumed highest ground water table, which is estimated to be at a depth of about 10 feet, were considered in our liquefaction analysis. Soils with normalized standard penetration test, $(N_1)_{60}$, greater than 30 blows per feet were considered too dense to liquefy.

The results of our analysis indicate that portions of the loose to medium dense sands and silty soils encountered in the CPTs between depths of about 15 to 50 feet could liquefy when subjected to a peak ground acceleration (PGA) of 0.81g, the PGA_M for maximum considered earthquake based on ASCE 7-10. Total ground surface settlement that could occur as a result of liquefaction from the design-level earthquake is estimated to range from about 1/8 to 3/8 inches in the areas of our CPTs. In our opinion, differential settlement of about 1/4-inch across a horizontal distance of 50 feet is possible from liquefaction during seismic shaking, and the estimated differential dynamic settlement should be considered during the structural design of the foundation system.

Geologic Hazards

In addition to liquefaction potential, we briefly reviewed the potential for geologic hazards to impact the development, considering the geologic setting and the soil encountered during our investigation. The results of our review are presented below.

- Fault Rupture - The site is not located in an 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 design life of the building, as is typical for sites throughout the Bay Area. The building 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. Since the materials encountered in our CPTs above the assumed highest ground water level were generally stiff clays, in our opinion, the likelihood of significant differential compaction affecting the building is low, provided the recommendations presented in our report are followed during design and construction.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed mixed-use building, provided the recommendations presented in this report are followed during design and construction. The primary geotechnical concerns for the proposed building are the variable stiffness of the near surface clays and the potential for severe ground shaking at the site during a major earthquake. Our subsurface exploration indicates that the surface clays are less stiff and more compressible at the west portion of the building than at the east portion. In our opinion, the mixed-use building may be supported on a mat foundation or a spread footing foundation. To help reduce the effects of the variable support conditions on the proposed building, we recommend that the foundations have added reinforcing to provide a stiffer foundation more capable of tolerating differential soil movement. Detailed recommendations are provided in the following sections of our report.

Because subsurface conditions may vary from those encountered at the location of our CPTs, 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 recommendations; and 2) observe and test during earthwork and foundation construction.

FOUNDATIONS

Mat Foundation

In our opinion, the mixed-use building may be supported on a reinforced concrete mat foundation bearing on a properly prepared and compacted soil subgrade and a non-expansive fill section. The mat may be designed for an average allowable bearing pressure of 1,500 pounds per square foot for combined dead plus live loads, with maximum localized bearing pressures of 2,500 pounds per square foot at column or wall loads. These pressures may be increased by one-third for total loads including wind or seismic forces. These pressures are net values; the weight of the mat may be neglected in design.

The mat 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 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. Alternatively, based on the anticipated building load and estimated post construction differential static settlement, a modulus of subgrade reaction (Kv) of 20 pounds per cubic inch (pci) may be assumed for the mat subgrade. The mat should also be designed with sufficient depth and reinforcing to span an unsupported length of at least 12 feet and cantilever a distance of at least 5 feet under full dead loads.

In our opinion, the mat slab should be underlain by at least 6 inches of free-draining gravel, such as ½- to ¾-inch clean crushed rock, as described in the section of the report titled "Interior Slabs." In addition, the mat foundation should be designed with a thickened perimeter edge. The thickened perimeter edge should have a width of at least 12 inches, should extend at least 24 inches below exterior grade, and at least 12 inches below the bottom of mat, whichever is deeper. Where permeable pavers are planned immediately adjacent to the building foundation, the nearby thickened perimeter edge should extend at least 24 inches below the bottom of the permeable rock section. Preferably, the subgrade soils below the permeable rock section should be sloped away from the building perimeter.

The mat subgrade should be scarified, prepared and compacted as recommended in the section titled "Compaction." Just prior to mat construction, the subgrade should be proof-rolled to provide a smooth firm surface for mat support. Our representative should observe and test during the preparation and compaction of the mat subgrade and crushed rock section.

Spread Footing Foundation

As an alternative to a mat foundation, in our opinion, the mixed-use building may be supported on a conventional spread footing foundation bearing on stiff native soils. Footings should have a width of at least 15 inches and should extend at least 28 inches below exterior finished grade, at least 24 inches below the bottom of concrete slabs-on-grade, whichever is deeper. In addition, where permeable pavers are planned immediately adjacent to the building foundation, the nearby spread footings should extend at least 24 inches below the bottom of the permeable rock section. Footings may be designed for an allowable bearing pressure of 2,000 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 during design.

All footings located adjacent to utility lines should bear below a 1:1 plane extending up from the bottom edge of the utility trench. In our opinion, all continuous footings should be reinforced with sufficient top and bottom steel reinforcement to provide structural continuity and to permit spanning of local irregularities. We also recommend that individual continuous foundations be designed with sufficient depth and reinforcing to span an unsupported length of at least 12 feet and cantilever a distance of at least 5 feet under full dead load. Isolated footings should generally be avoided where differential settlements would be problematic.

The bottom of all footing excavations should be cleaned of loose and soft soil and debris. A member of our staff should observe all footing excavations prior to placement of reinforcing steel to confirm that they have at least the recommended minimum dimensions, expose suitable material, and have been properly cleaned. If relatively soft or loose soil or debris is encountered in the foundation excavations, our field representative will require these materials to be removed and may require a deeper footing embedment depth before reinforcing steel and concrete is placed or alternatively the excavations may be deepened and replaced with properly compacted fill. .

Lateral Loads for Mat/Footings

For the mat foundation, the structural engineer should consult with the vapor barrier manufacturer for the coefficient of friction to be assumed for design. 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. In addition to friction, lateral resistance may also be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations or backfilled with properly compacted structural fill. We recommend assuming an equivalent fluid pressure of 300 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the foundations is not covered and protected by a concrete slab or pavement

Settlement

Thirty year post construction differential movement due to static loads is not expected to exceed about 3/4-inch across the mixed-use building provided the foundation for the structure is designed and constructed as recommended. In addition to static settlement, as stated in the above sections, we estimate that liquefaction-induced differential settlement on the order of about 1/4-inch could occur over a horizontal distance of 50 feet across the property from the analyzed seismic event. The estimated dynamic settlement should be considered during the structural design of the building and their foundation systems. During structural design, the estimated building settlement may be updated when building loads are available.

SLABS-ON-GRADE

General Mat/Slab Considerations

To reduce the potential for movement of the soil subgrades below the mat and concrete slabs-on-grade, at least the upper 6-inches of the surface soil should be scarified, moisture conditioned, and compacted at a moisture content above the laboratory optimum. The native soil subgrade should be kept moist up until the time the non-expansive fill, crushed rock and vapor barrier, and/or aggregate base section is installed. Slab subgrades and non-expansive fill should be prepared and compacted as recommended in the section of this report titled "Earthwork."

Overly soft or moist soils should be removed from mat/slab-on-grade areas. The subgrade conditions should be reviewed by our field representative. Exterior flatwork and building mat should be underlain by a layer of non-expansive fill as described below. The non-expansive fill should consist of imported soil with a Plasticity Index no greater than 15, preferably Class 2 aggregate base.

Considering the potential for some differential movement of the surface and near-surface soils, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should 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 6 inches of Class 2 aggregate base. To improve performance, exterior slabs-on-grade, such as for patios, may 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. In our opinion, the thickened edges should be at least 8 inches wide and ideally should extend at least 4 inches below the bottom of the underlying aggregate base layer.

Interior Slabs

The mat and interior concrete slab-on-grade floors should be constructed on a layer of non-expansive fill at least 6-inches thick that is placed and compacted 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 6 inches of clean, free-draining gravel, such as 1/2-inch to 3/4-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 layer should be densified and leveled with vibratory equipment, and may be considered as the upper portion of the non-expansive fill recommended above.

To reduce vapor transmission up through concrete floors, the crushed rock section should be covered with a high quality, UV-resistant 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”) or other waterproofing membrane. The vapor barrier should be placed directly below the concrete mat/slab. Sand above the vapor barrier is not recommended. The vapor 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 concrete mix, with lower water:cement ratios producing more damp-resistant mat/slabs and stronger concrete. 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 can be added to the mix. Water should not be added to the concrete 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 the concrete slabs-on-grade include moist curing for 5 to 7 days and allowing the mat/slab to dry for a period of two months or longer prior to placing floor coverings. Also, prior to installation of the floor covering, it may be appropriate to test the mat/slab moisture content for adherence to the manufacturer's requirements and to determine whether a longer drying time is necessary.

ELEVATOR PIT WALLS

The elevator pit walls should be designed to support the wall backfill and any surcharge loads acting on the wall backfill. If the elevator pit walls are designed and constructed as undrained walls, the walls should be designed to resist an undrained at-rest pressure simulated by an equivalent fluid pressure of 80 pounds per cubic foot plus an additional uniform lateral pressure of 8H in pounds per square foot, where H is the height of the wall in feet. If the elevator pit walls are drained, the elevator pit walls should 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 the elevator pit walls will be subjected to surcharge loads, such as from adjacent foundations or construction loading, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Based on site peak ground acceleration, Seed and Whitman (1970), Al Atik and Sitar (2010), and Lew et al. (2010), seismic loads on walls that cannot yield, such as the elevator pit walls, may be subjected to a seismic load as high as about $13H^2$. 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 or 80 pounds per cubic foot).

To prevent buildup of water pressure from surface water infiltration, a subsurface drainage system could be installed behind the walls. 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 feet below exterior finished grade. A layer of filter fabric should be wrapped around the crushed rock to protect it from infiltration of native soil. The upper 1½ feet of backfill should consist of compacted native soil. The perforated pipe should discharge into a 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 panels approved by our office may be used for wall drainage as an alternative to the gravel drainage system described above. If used, the drainage panels 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 1/2-inch to 3/4-inch clean crushed rock and filter fabric should be placed around the drainpipe, as recommended previously.

Backfill placed behind the 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 elevator pit walls should be supported on a spread footing or mat foundation designed in accordance with the recommendations presented previously.

DRIVEWAY PAVEMENT

For light residential type traffic using asphalt concrete, we recommend the driveway pavement section consist of at least 3 inches of asphalt concrete on at least 10 inches of Class 2 aggregate base.

If the driveway will be constructed with Portland cement concrete (PCC), we recommend the driveway pavement consist of at least 5 inches of PCC on at least 10 inches of Class 2 aggregate base. Un-reinforced concrete for the 5-inch-thick driveway pavement should have a 28-day compressive strength of at least 3,500 psi. PCC pavements should be laterally constrained with curbs or shoulders and sufficient control joints should be incorporated in the design and construction to limit and control cracking.

If occasional heavy truck traffic is expected, the aggregate base section should be increased to at least 12 inches thick. The soil subgrade and aggregate base below the pavement section should be prepared and compacted as recommended below. The use of a moisture cut-off or thickened edge along the edges of the driveway would be desirable in order to reduce water seepage below the edges of the driveway and into the underlying aggregate base and subgrade, which can lead to premature pavement distress.

PERMEABLE PAVING STONES

The permeable paving stones should be underlain by at least 10 inches of Class 2 aggregate base, at least 8 inches of #57 graded aggregate, or at least 12 inches of Class 2 permeable material (per Caltrans Standard Specifications, Section 68). If the paving stones are supported on #57 graded aggregate or on Class 2 permeable material, provisions should be made for draining water that collects in the base material. The drainage system should include a subdrain that connects to a suitable discharge location or an overflow device, such as a bubbler. Subgrade and base materials should be compacted as recommended in our geotechnical report. The minimum required thickness of the graded aggregate layer will be confirmed during final design when more information is available. Installation of reinforcing fabric or geogrid under the aggregate layer would increase the service life of the pavement.

EARTHWORK

Clearing & Subgrade Preparation

All deleterious materials, such as existing structures, slabs, pavements, concrete, utilities, vegetation, fill soils, relatively soft soils, root systems, topsoil, etc. should be cleared from areas to be built on or paved. The actual stripping depth should be determined by a member of our staff at the time of construction. Excavations that extend below finish grade should be backfilled with structural fill that is water-conditioned, placed, and compacted as recommended in the section 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 titled "Compaction."

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. 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. Our representative should approve proposed import materials prior to their delivery to the site.

Compaction

Scarified surface soils and all structural fill should be compacted in uniform lifts no thicker than 8 inches in pre-compacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 2. The relative compaction and moisture content recommended in Table 2 is relative to ASTM Test D1557, latest edition.

Table 2. Compaction Recommendations
Mixed-Use Building
Burlingame, California

General	Relative Compaction*	Moisture Content*
• Scarified subgrade in areas to receive fill or slabs.	90 percent	Above optimum
• Structural fill	90 percent	Above optimum
• Structural fill below a depth of 4 feet.	93 percent	Above optimum
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 non-expansive soil.	90 percent	Near optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Temporary Slopes and Excavations

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

Because of the potential for 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 near excavations will also be the responsibility of the contractor. In our experience, a preconstruction survey is generally performed to document existing conditions near property lines prior to construction, with intermittent monitoring of the structures during construction.

Utility Trench Backfill

Utility trench excavations should be excavated and backfilled in accordance with all applicable local, state and federal safety regulations, including the current OSHA excavation and trench safety standards. All trench backfill material should be moisture conditioned and compacted as recommended in the section of this report titled "Compaction." Utility penetrations through elevator pit walls or footings should be properly sealed. Proper compaction of utility trenches below pavement areas is essential to prevent future settlement and the resulting damage and maintenance costs of the pavement.

Utilities with sand bedding can become conduits to bring subsurface water below building and pavements particularly when located adjacent to well irrigated landscaping areas. Where utility trenches interface with the building pad or pavement areas, an impermeable plug should be installed to limit the potential for subsurface water to flow along the utility trench and saturate subgrade soils. In our opinion, the impermeable plug could consist of compacted clayey on-site soil, lean concrete slurry, or other approved impermeable material.

Finished Slopes

We recommend that finished slopes be cut or filled to an inclination preferably no steeper than 2.5:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion that may require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted with erosion-resistant vegetation.

Surface Drainage

Finished grades should be designed to prevent ponding of water and to direct surface water runoff away from foundations, and edges of 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 discharge ends of roof downspouts to carry water away from perimeter foundations. Preferably, roof downspout water should be collected in a closed pipe system that is routed to a storm drain system or other suitable location.

The 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 showing the location of the surface and subsurface drain lines and clean outs be developed. The drainage facilities should be periodically checked to verify that they are continuing to function properly, and likely will need to be periodically cleaned of silt and debris which 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 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, foundation construction, mat/slab subgrade and non-expansive fill preparation, elevator pit wall drainage and backfilling, permeable paver construction, pavement construction, utility trench backfill, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated August 8, 2016. 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 number of borings. The nature and extent of variation across the site may not become evident until construction. If variations are then exposed, it will be necessary to reevaluate our recommendations.

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American Society of Civil Engineers, 2013, Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-10.

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Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute (EERI), Oakland, California.

United States Geological Survey, 2015, United States Seismic Design Maps, http://earthquake.usgs.gov/designmaps/us/application.php

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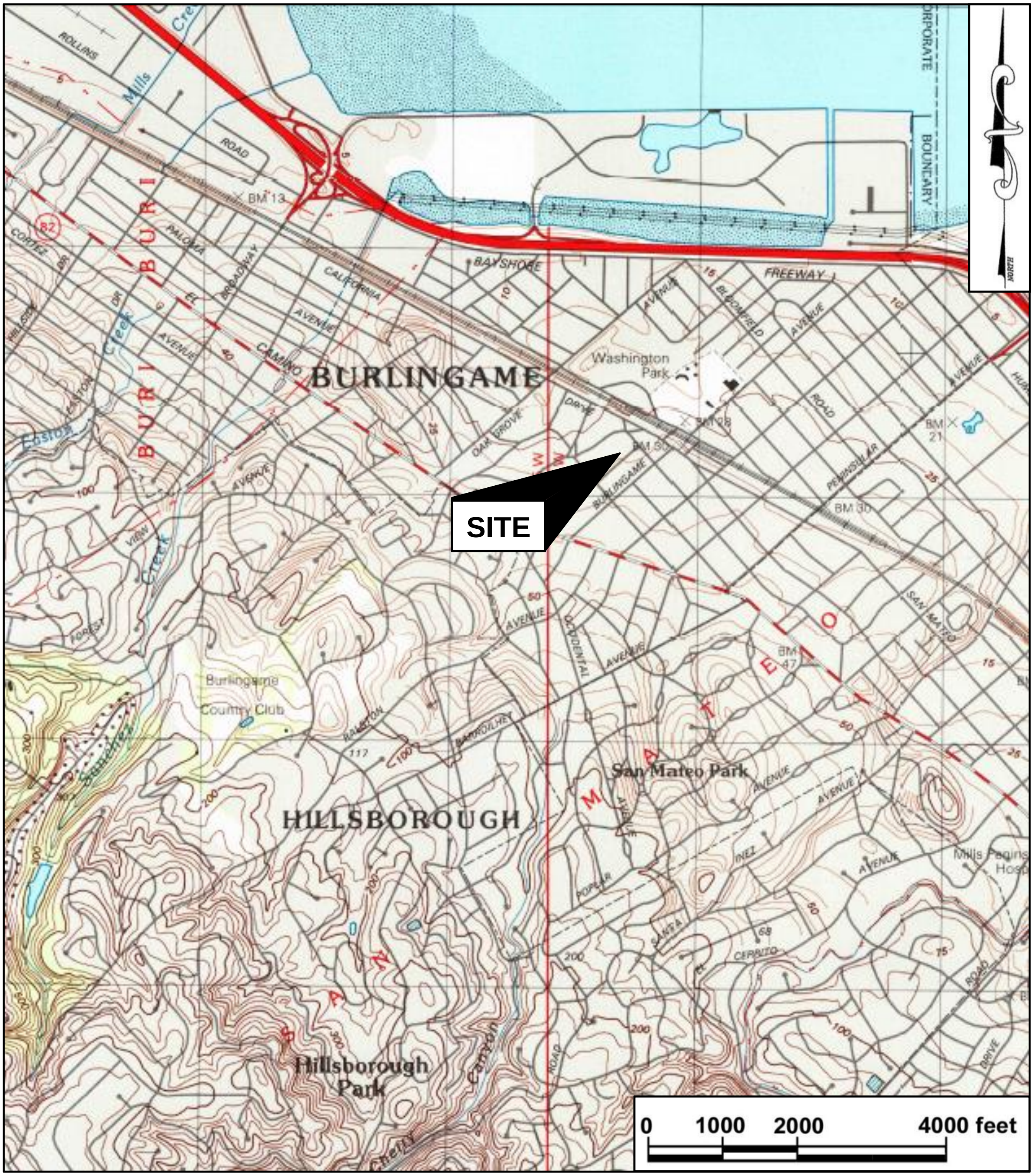
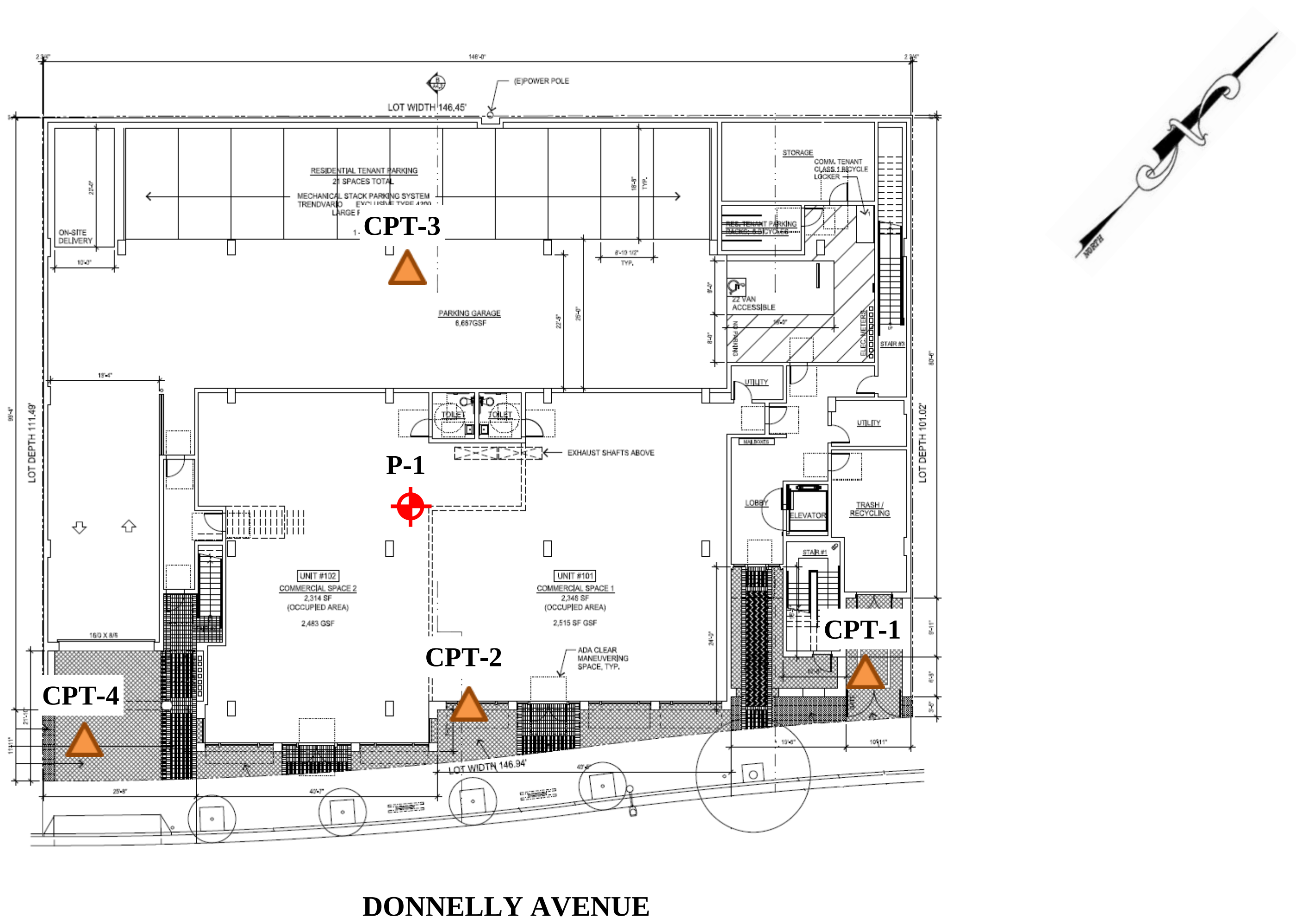


Scale: 1 inch = 2000 feet
Base is United States Geological Survey San Mateo 7.5 Minute Quadrangle, dated 1997.

VICINITY MAP
BRITTON MIXED USE BUIDLING
BURLINGAME, CALIFORNIA

FIGURE 1
AUGUST 2016
PROJECT NO. 3804-1



LEGEND

CPT-4 Approximate Locations of Cone Penetration Tests (CPT).

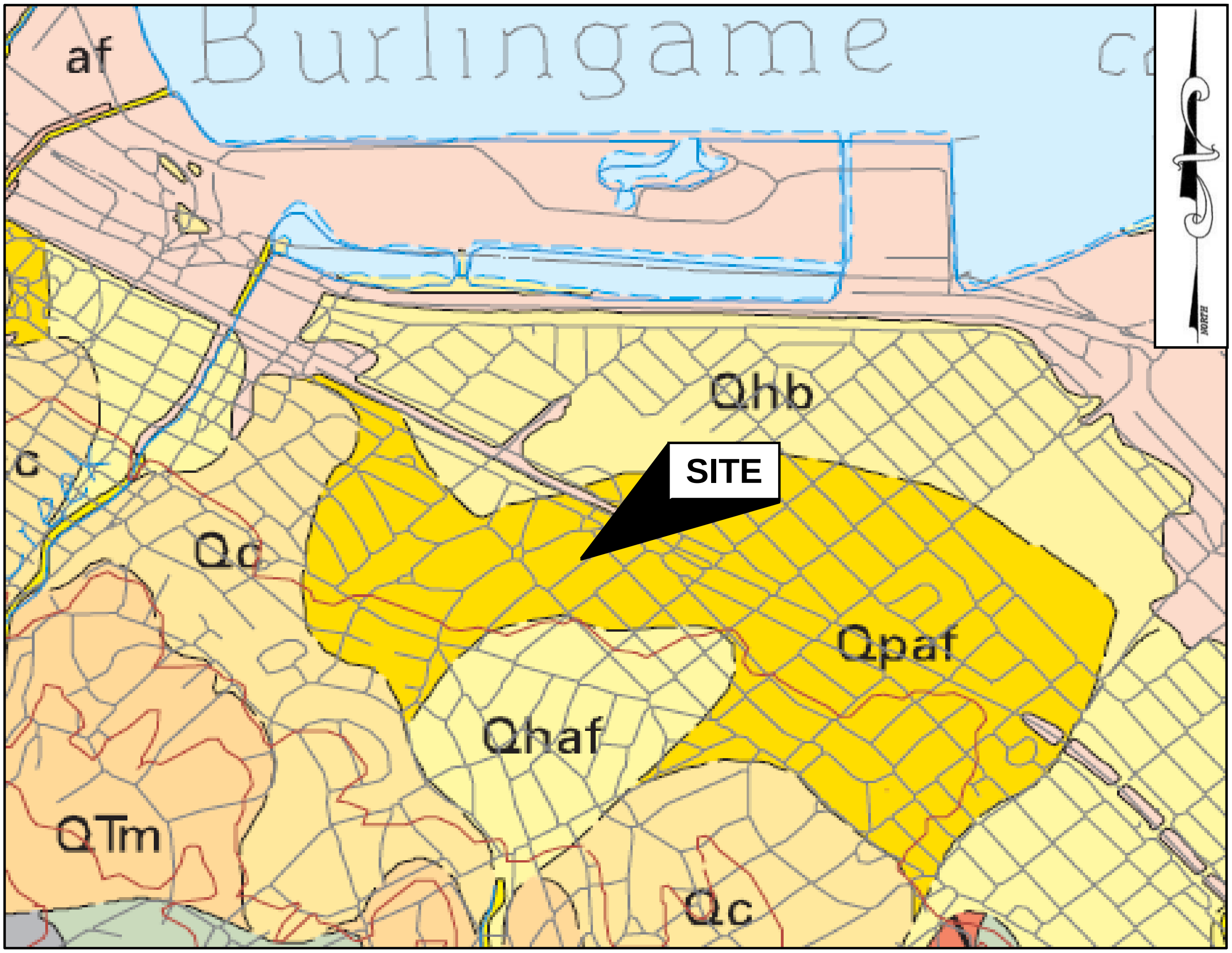
P-1 Approximate Location of near-surface sample.

Approximate Scale: 1 inch = 30 feet.

Base is site plan prepared by Gary Gee Architects, Inc., revised on April 12, 2016.

SITE PLAN
BRITTON MIXED USE BUIDLING
BURLINGAME, CALIFORNIA

FIGURE 2
AUGUST 2016
PROJECT NO. 3804-1



LEGEND

af	Artificial fill	Qc	Colma formation
Qhb	Basin deposits	QTm	Merced formation
Qhaf	Alluvial fab and fluvial deposits		Contact: dashed where inferrred, dotted where concealed
Qpaf	Alluvial fan and fluvial deposits		

0 1000 2000 4000 feet

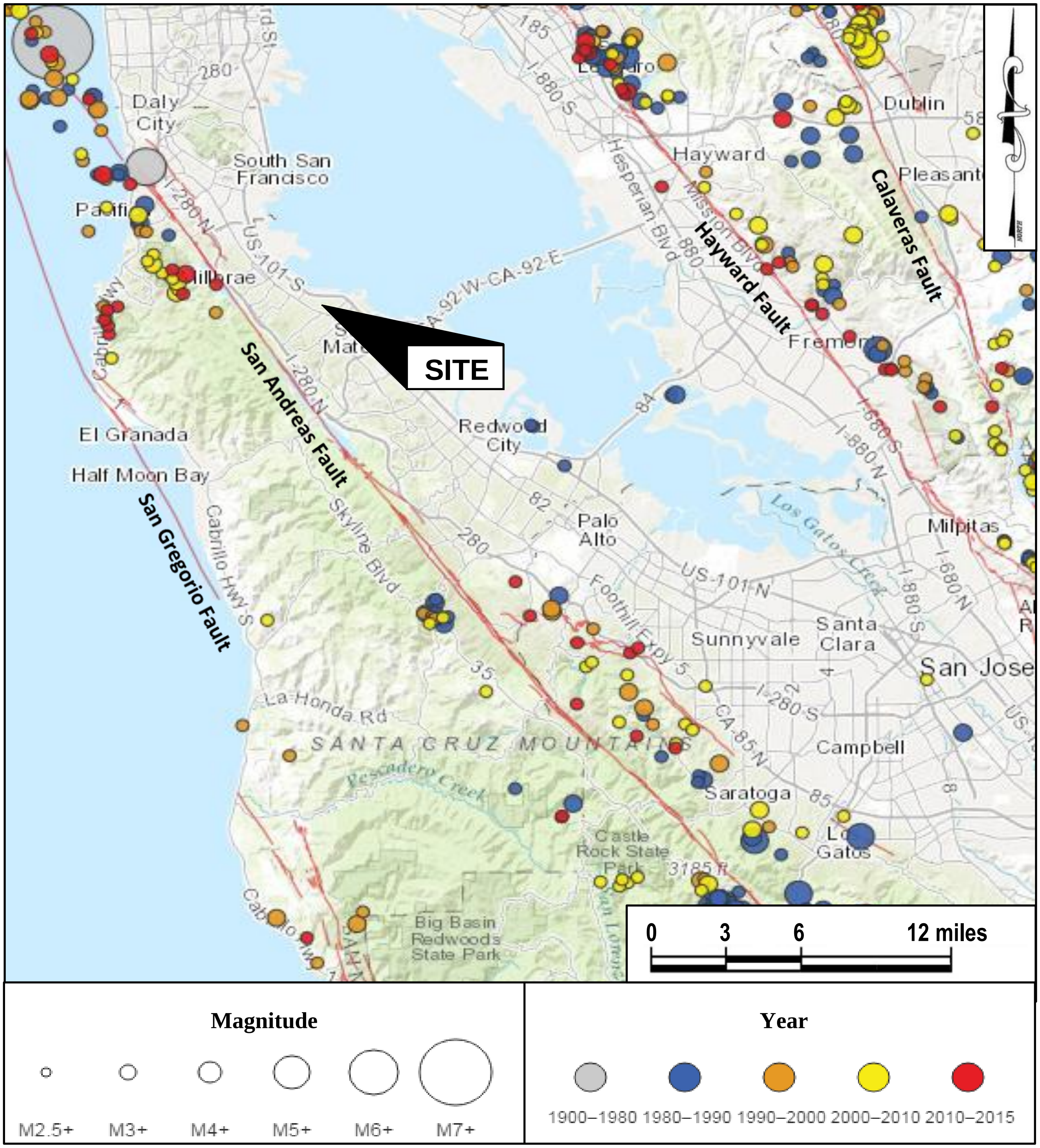
Scale: 1 inch = 2000 feet

Base is USGS Geologic Map of San Mateo County, Brabb, Graymer, and Jones, dated 2000.

VICINITY GEOLOGIC MAP **FIGURE 3**

BRITTON MIXED USE BUIDLING AUGUST 2016

BURLINGAME, CALIFORNIA PROJECT NO. 3804-1



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
BRITTON MIXED USE BUIDLING
BURLINGAME, CALIFORNIA

FIGURE 4
AUGUST 2016
PROJECT NO. 3804-1

APPENDIX A

FIELD INVESTIGATION

The Cone Penetration Tests (CPT) were carried out by Middle Earth Geo Testing, Inc. of Orange, CA 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 all of the soundings. The cone had a tip area of 10 cm^2 and friction sleeve area of 150 cm^2. The logs of our CPTs are attached in this Appendix.

The locations of the CPTs and probe were established by pacing using the site plan prepared by Gary Gee Architects, Inc., revised on April 12, 2016, and should be considered accurate only to the degree implied by the method used

The CPT 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 location where sampling and testing were conducted. The passage of time may also result in changes in the subsurface conditions.

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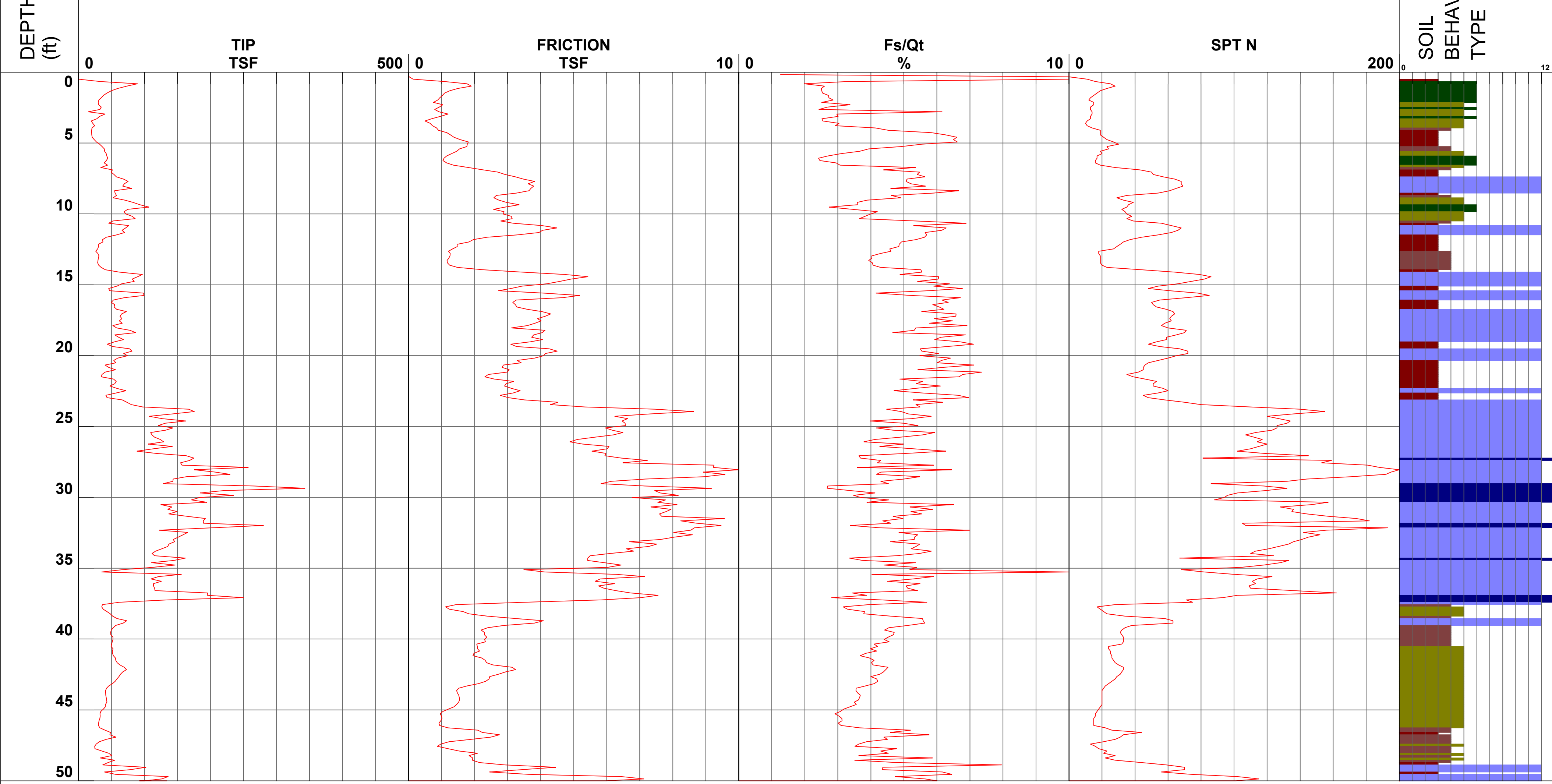
Romig Engineers

Middle Earth Geo Testing Inc.

Project	Britton Mixed use Building	Operator	BH-JH	Filename	SDF(026).cpt
Job Number	1214-1220	Cone Number	DDG1350	GPS	
Hole Number	CPT-01	Date and Time	7/1/2016 3:07:11 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		14.00 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

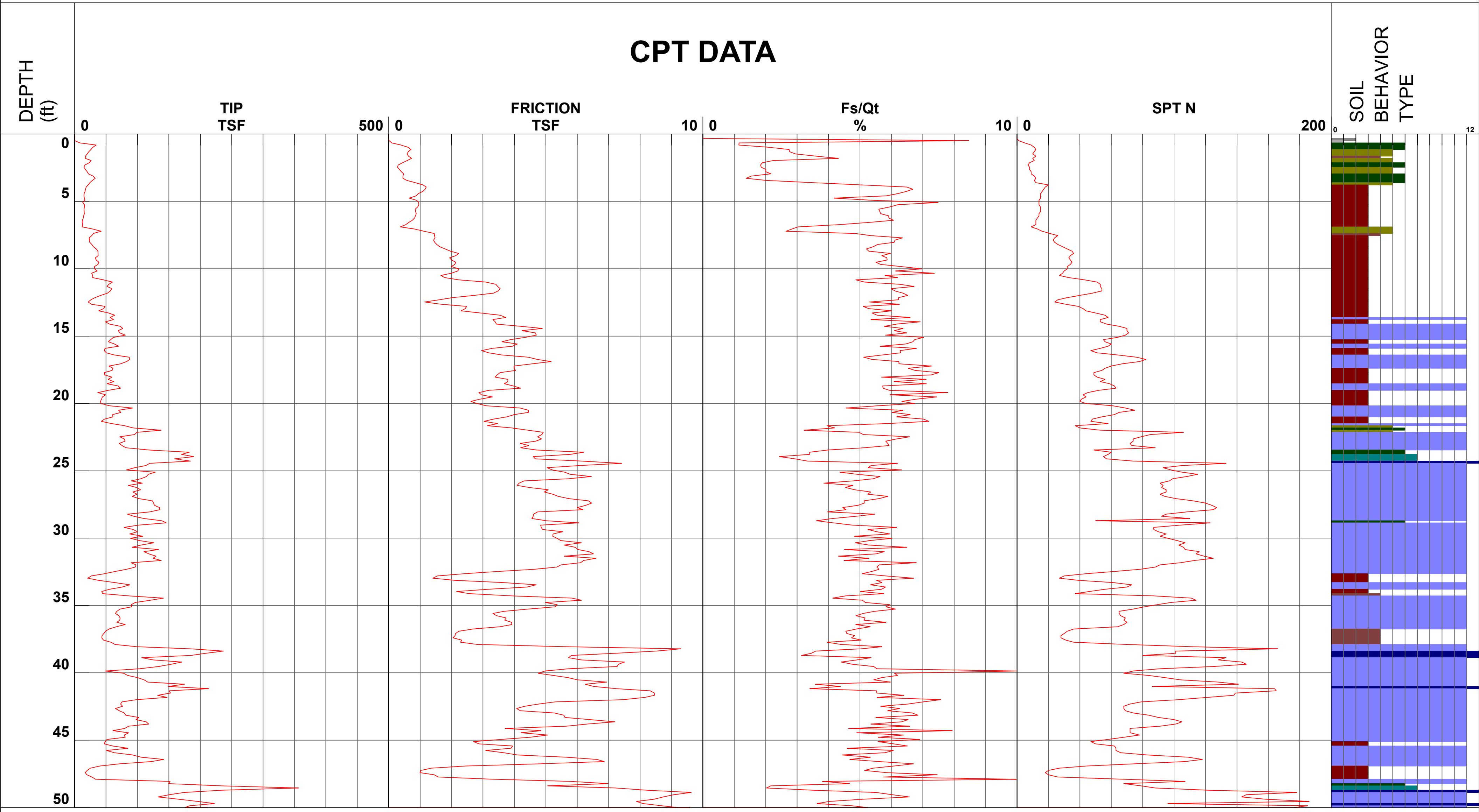
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Britton Mixed use Building	Operator	BH-JH	Filename	SDF(024).cpt
Job Number	1214-1220	Cone Number	DDG1350	GPS	
Hole Number	CPT-02	Date and Time	7/1/2016 1:05:24 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		14.00 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

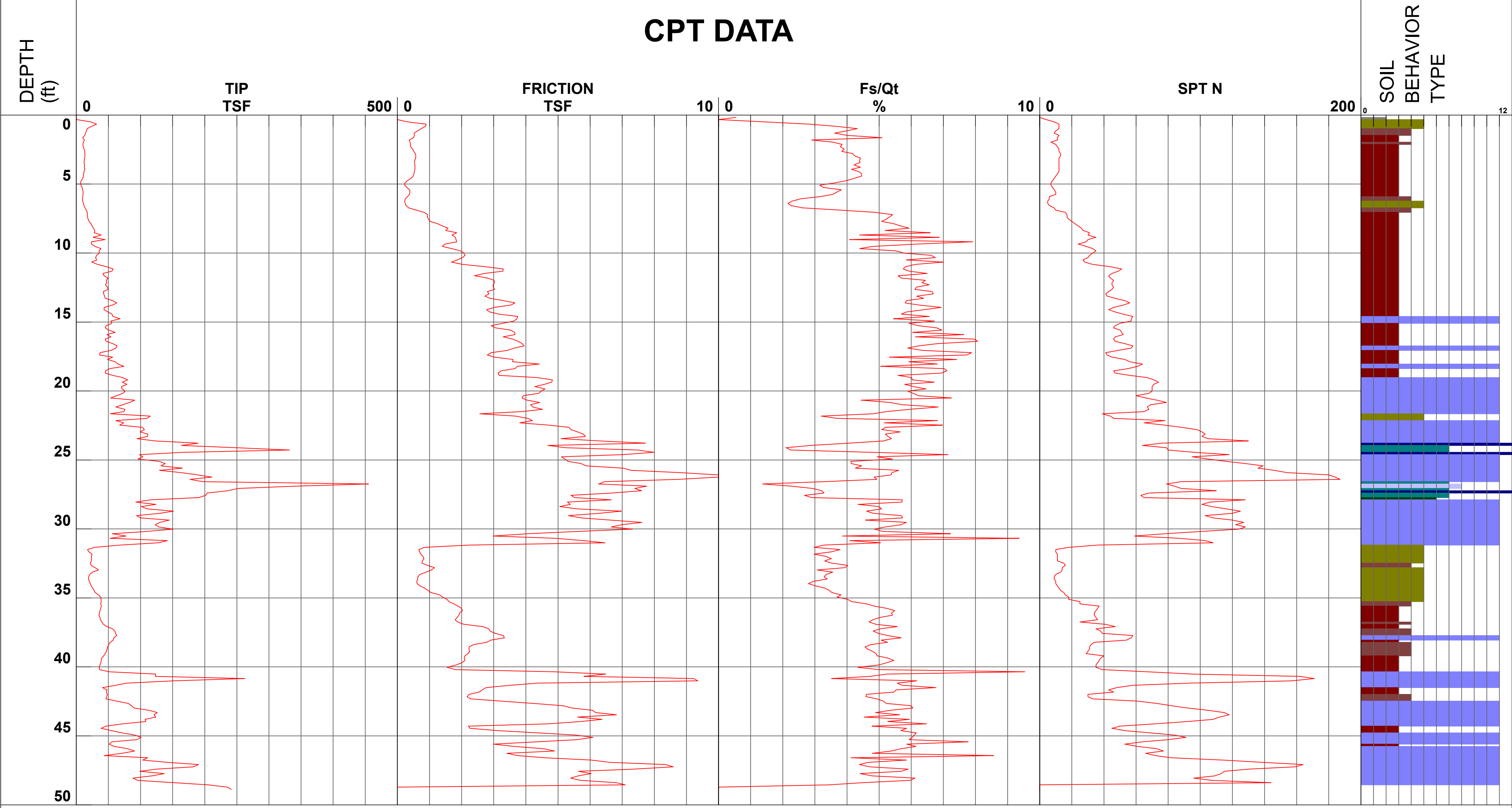
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Britton Mixed use Building	Operator	BH-JH	Filename	SDF(025).cpt
Job Number	1214-1220	Cone Number	DDG1350	GPS	
Hole Number	CPT-03	Date and Time	7/1/2016 2:00:24 PM	Maximum Depth	48.88 ft
EST GW Depth During Test		14.00 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 (*)

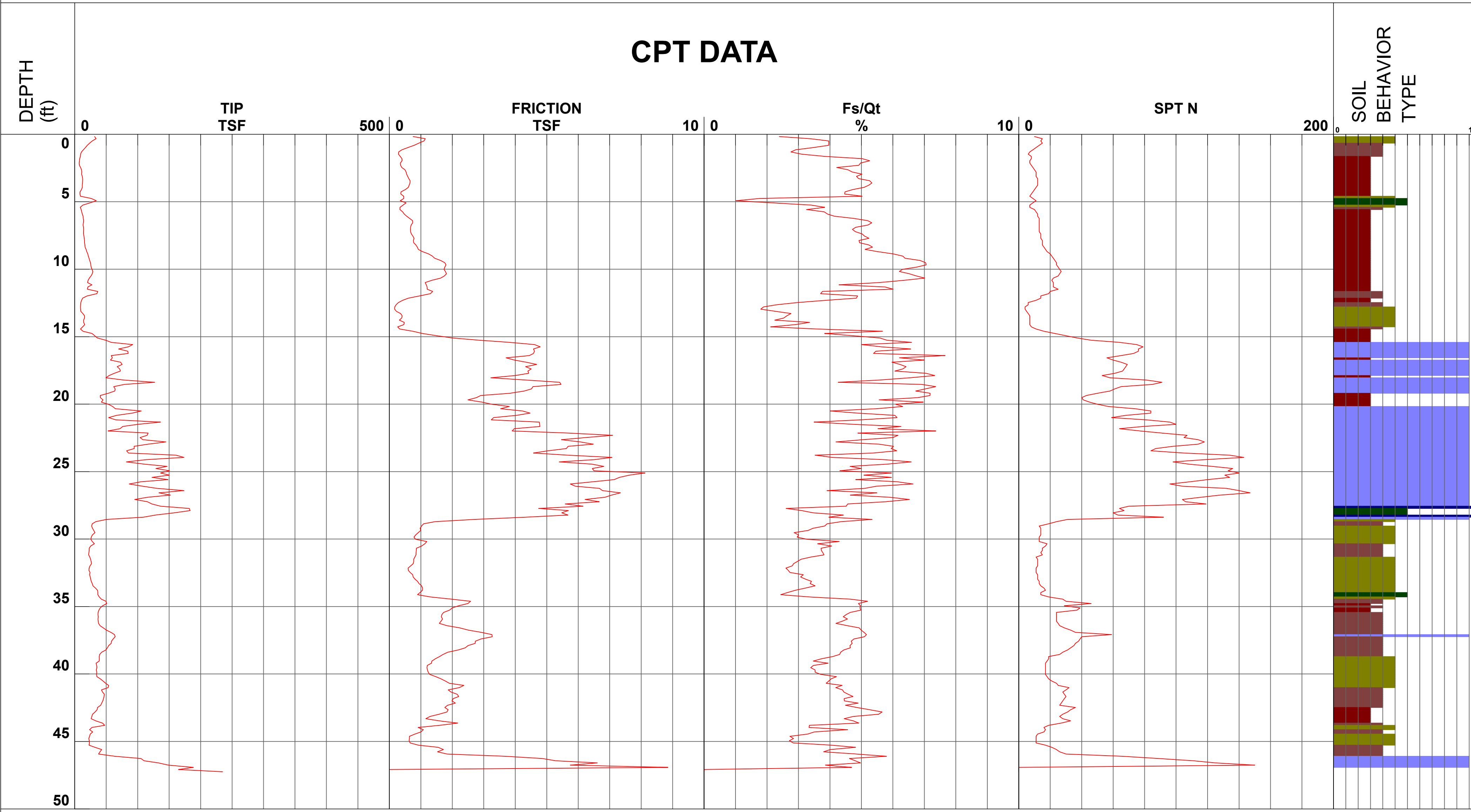
Cone Size 10cm squared

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

Romig Engineers

Project	Britton Mixed use Building	Operator	BH-JH	Filename	SDF(028).cpt
Job Number	1214-1220	Cone Number	DDG1350	GPS	
Hole Number	CPT-04	Date and Time	7/1/2016 4:03:45 PM	Maximum Depth	47.24 ft
EST GW Depth During Test			14.00 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

LABORATORY TESTS

The Atterberg Limits were determined on one sample of soil in accordance with ASTM D4318. The Atterberg limits are the moisture content within which the soil is workable or plastic. The results of this test are presented in Figure B-1.

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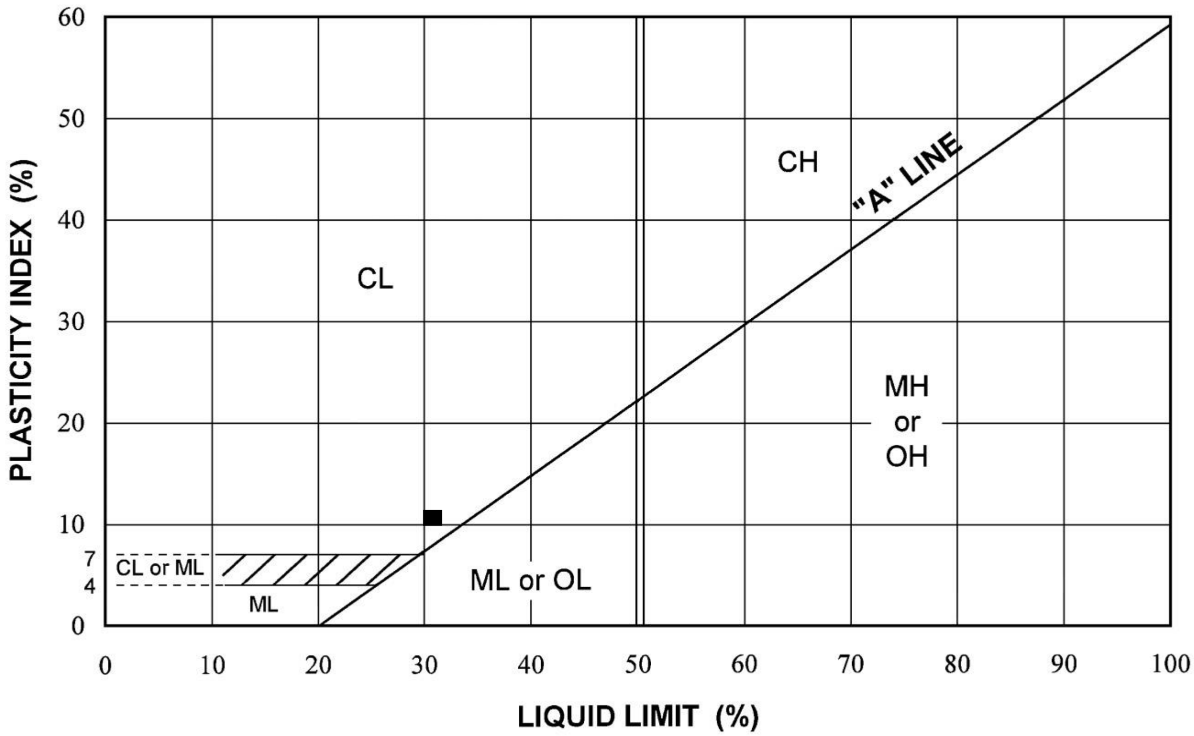

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	P-1	1.5-2.5	11	31	11			CL

PLASTICITY CHART
BRITTON MIXED USE BUIDLING
BURLINGAME, CALIFORNIA

FIGURE B-1
AUGUST 2016
PROJECT NO. 3804-1