Appendix E:
Geotechnical Investigation

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GEOTECHNICAL INVESTIGATION

FOR THE FOUR-STORY LIVE/WORK BUILDING
619, 621, AND 625 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA 94010

Prepared for
MR. ED DUFFY
Renovattio Construction
414 Pinehill Road
Hillsborough, California 94010

December 2016
Project No. 3910-1

ROMIG
ENGINEERS

December 6, 2016
3910-1

Mr. Ed Duffy
Renovattio Construction
414 Pinehill Road
Hillsborough, California 94010

RE: GEOTECHNICAL INVESTIGATION
FOUR-STORY LIVE/WORK BUILDING
619, 621, AND 625 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA

Dear Mr. Duffy:

In accordance with your request, we have performed a geotechnical investigation for your proposed live/work (mixed used) building to be constructed at 619, 621, and 625 California Drive 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 project.

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

Thank you for the opportunity to work with you on this project. Please call if you have any questions or comments concerning the findings, conclusions, or recommendations from our investigation.

Very truly yours,

ROMIG ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
FOUR-STORY LIVE/WORK BUILDING
619, 621, AND 625 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA

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DECEMBER 2016

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GEOTECHNICAL INVESTIGATION
FOR
FOUR-STORY LIVE/WORK BUILDING
619, 621, and 625 CALIFORNIA DRIVE
BURLINGAME, CALIFORNIA

INTRODUCTION

We are pleased to present this geotechnical investigation report for your proposed live/work (mixed use) building to be constructed at 619, 621, and 625 California Drive 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 project.

Project Description

The project consists of constructing a four-story live/work (mixed use) building on the three referenced properties in Burlingame. The building is expected to have a ground level concrete podium with three levels of wood frame construction above. The ground level of the building is expected to include interior covered parking, a small lobby, and five office units with the ground floor space totaling 14,164 square feet. The 2nd through 4th floor will consist of 26 residential/work units totaling approximately 34,799 square feet. The 4th floor will also include exterior common and private terraces. The relatively flat approximately 0.45-acre site is currently occupied by an auto repair shop, a parking lot, and residential units. The proposed building will be located centrally across the three lots. Other improvements include a trash/utility enclosure, exterior flatwork, paved parking entrance driveway, and landscaping around the building.

Scope of Work

The scope of our work for this investigation was presented in our agreement with you dated September 12, 2016. In order to accomplish this investigation, we performed the following work.

- Review of geologic, geotechnical, and seismic conditions in the vicinity of the site.
- Subsurface exploration consisting of drilling, sampling, and logging three exploratory borings in the area of the proposed building.
- Laboratory testing of selected samples to aid in soil classification and to help evaluate the engineering properties of the soils encountered in our borings.

- Engineering analysis and evaluation of the subsurface data to develop geotechnical design criteria.
- Preparation of this report presenting our geotechnical findings and recommendations for the project.

Limitations

This report was prepared for the exclusive use of Mr. Ed Duffy for specific application to developing geotechnical design criteria for the currently proposed live/work building to be constructed at 619, 621, and 625 California Drive in Burlingame, California. We make no warranty, expressed or implied, except that our services were 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 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; our understanding of the currently proposed construction; 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

Site reconnaissance and subsurface exploration were performed on October 17, 2016. Subsurface exploration was performed using a truck-mounted drill equipped with 7.25-inch diameter hollow-stem augers. Three exploratory borings were advanced to depths ranging between 20 to 50 feet. The approximate locations of the borings are shown on the Site Plan, Figure 2. The boring logs and the results of our laboratory tests are attached in Appendices A and B, respectively.

Surface Conditions

The site is located in a residential/commercial area at the east corner of the intersection of California Drive and Oak Grove Avenue. At the time of our investigation, the site consisted of three adjacent lots addressed as 619, 621, and 625 California Drive. 619 California Drive (southeast most site) was an asphaltic concrete paved, empty lot. Two large shipping containers were present on the lot. 625 California Drive (middle site) was occupied by a single story, concrete block commercial building. 619 California Drive was occupied by two single-story, wood framed residences that had wood siding exteriors (the rear residence addressed as 1201 Oak Grove Avenue). A concrete driveway extended from California Drive along a portion of the front residence. Concrete and brick flatwork were located along the perimeter of the residences. The residential lot was vegetated with small shrubs and small to medium trees.

The depth and width of the existing building and residence foundations are unknown. Several vertical cracks were observed along the exterior foundation wall of the commercial building. Where visible we observed many vertical cracks up to about ½-inch wide along the exterior stem wall of the two single-story residences. We did not observe the interior of any of the structures. The concrete walkways were in poor condition with surface cracks as wide as about 1-inch. The concrete driveway had several cracks up to ¼-inch in width. The asphalt parking lot at 619 California Drive had numerous hairline to ½-inch wide cracks and was very deteriorated and weathered. The roof downspouts at the two residences discharged into a closed pipe system or adjacent to the perimeter foundation

Subsurface Conditions

At the location of our Exploratory Boring EB-1, which was advanced at 625 California Drive, we encountered approximately 1.5 feet of fill which consisted of very stiff sandy lean clay of low plasticity underlain by approximately 3.5 feet of very stiff sandy lean clay of low plasticity. Beneath the surface clays encountered approximately 45 feet of medium dense to very dense clayey sand and clayey gravel which extended to the maximum depth explored of 50 feet.

At Boring EB-2, which was advanced at the northeast side of 619 California Drive, we encountered approximately 3 feet of fill which consisted of firm sandy lean clay of low plasticity underlain by approximately 14.5 feet of stiff to very stiff sandy lean clay of low plasticity. We then encountered medium dense clayey sand which extended to the maximum depth explored of 35 feet.

At Boring EB-3, which was advanced at the southeast corner of 619 California Drive, we encountered approximately 3 feet of fill which consisted of soft sandy lean clay of low plasticity underlain by approximately 4 feet of stiff to very stiff sandy lean clay of low plasticity. We then encountered medium dense clayey sand which extended to the maximum depth explored of 20 feet.

A Liquid Limit of 28 and a Plasticity Index of 8 were measured on a sample of near-surface soil obtained from Boring EB-1. These test results indicate that the surface and near-surface soils at the site generally have low plasticity and a low potential for expansion.

Ground Water

Ground water was encountered during drilling and sampling at a depth of approximately 23 feet in Boring EB-1, approximately 11 feet in Borings EB-2, and approximately 14 feet in Boring EB-3. The borings were backfilled with grout shortly after drilling, therefore a stabilized ground water level was not measured. Our work experience in the immediate area of the site indicates that the stabilized ground water table has varied from about 5 to 7 feet below surface grades at nearby project sites.

Based on our experience, we expect that the highest projected ground water level at the site could be seasonally as high as approximately 6 feet below grade. Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, tidal fluctuations, local surface and subsurface drainage patterns, landscaping, and other factors.

GEOLOGIC SETTING

As part of our investigation, we reviewed our local experience and geologic literature in our files pertinent to the general area of the site. The information reviewed indicates the site is located in an area mapped as Holocene-age medium-grained alluvium, Qam (Pampeyan, 1994). The alluvium is described as unconsolidated to moderately consolidated, moderately sorted sand and silty to clayey sand. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

Based on information presented in a report titled "Geologic and Engineering Aspects of San Francisco Bay Fill" (CDMG, 1969), the site is mapped outside the area which is considered to be underlain by compressible younger Bay Mud (CDMG, 1969). The estimated extent and thickness of the young Bay Mud in the immediate site area is shown on the Contour Map of Bay Mud Thickness, Figure 4.

The lot and immediate site vicinity are located in an area that slopes very gently to the north towards the San Francisco Bay. The site is located at an elevation of approximately 23 feet above sea level.

Faulting and Seismicity

There are no mapped through-going faults across or immediately 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 remote.

The San Francisco Bay Area is, however, an active seismic region. Earthquakes in the region result from strain energy constantly accumulating due to 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 9.1 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 5.

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).

Table 1. Earthquake Magnitudes and Historical Earthquakes
Live/Work 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

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the 2016 California Building Code and in ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures." Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class 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 figures and tables in the 2016 California Building Code and in the lookup tables at the U.S.G.S. website based on the latitude and longitude of the site. For the site latitude (37.5819) and longitude (-122.3510) and Site Class D, SDs = 1.385g and SD1 = 0.983g.

Geologic Hazards

As part of our investigation, we reviewed the potential for geologic hazards to impact the site and the proposed building, considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below and in the following sections of our report.

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

- 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 service center facility, as is typical for sites throughout the Bay Area. The building should be designed in accordance with current earthquake resistance standards.

Liquefaction and Dynamic Densification

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. A State of California liquefaction hazard zone had not been established yet for this site area.

The clayey sand encountered at the site below the highest projected ground water depth, which is estimated to be about 6 feet below the ground surface, was 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.

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

In addition to liquefaction, we analyzed the potential for dynamic densification of the medium dense sandy soils when a deeper ground water condition is present. Dynamic densification occurs during moderate and large earthquakes when soft or loose, natural or fill soils densify and settle, often unevenly across a site. To evaluate the potential for earthquake-induced dynamic densification, we performed a settlement analysis of the data from our borings following the methods presented at the US Army Corps of Engineers EM1110-1-1904.

Potentially liquefiable soils and/or soils prone to dynamic densification were encountered in Boring EB-1 between depths of approximately 5 to 7 feet, 12 to 20 feet, and 27 to 32 feet, and in Boring EB-2 between depths approximately of 17.5 to 35 feet, and in Boring EB-3 between depths of approximately 7 to 20 feet. These clayey sands and gravelly sands are potentially prone to liquefaction or dynamic densification when subjected to the maximum considered earthquake acceleration (PGA_M) of 0.81g based on the Probabilistic Seismic Hazards Mapping Ground Motion Page (CGS, 2016). Based on the results of our analysis of these sand and gravel layers, we estimate that total settlement of about 0.6- to 1.1-inches could occur within these sand strata due to severe ground shaking caused by a major earthquake. In our opinion, differential settlement of about ½- to ¾-inch over a horizontal distance of about 50 feet is possible at the ground surface from this amount of total settlement.

Several feet of soft to firm surface fill was encountered along the southeast area of the site. In our opinion, some static and seismic related differential settlement of slabs-on-grade and exterior flatwork/pavement areas is possible in areas where the existing fill is not excavated and properly compacted.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed four-story live/work building provided the recommendations presented in this report are followed during design and construction. Specific geotechnical recommendations for the project are presented in the following sections of this report.

The primary geotechnical concerns for the proposed improvements are the presence of up to 3 feet of soft to firm surface fill material encountered in our borings, the medium dense sand strata that are susceptible to liquefaction and dynamic densification, and the potential for severe ground shaking during a major earthquake. In our opinion, the proposed building may be supported on mat or conventional spread footing foundation bearing in stiff native soils below any existing fill, or on properly compacted structural fill. These preliminary foundation recommendations are based on the anticipated structural loading conditions. However, once the specific dead and live loads and the foundation configuration have been developed, we should update the range of expected foundation settlement and determine if revision to these preliminary recommendations are appropriate.

In our opinion, any existing fill not removed during grading for the building pad should be excavated and recompacted below the building, exterior flatwork, and any other site improvements during site preparation. The reworking of the fill and subgrade preparation should proceed as recommended in the section of this report titled "Earthwork."

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

FOUNDATIONS

Spread Footing Foundations

In our opinion, the building may be supported on a conventional spread footing foundation system bearing on stiff native soils or properly compacted structural fill. All continuous footings should have a width of at least 15 inches and should extend at least 30 inches below exterior grade and at least 24 inches below the bottom of concrete slabs-on-grade, whichever is deeper. Continuous footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 3,000 pounds per square foot for dead loads, 3,500 pounds per square foot for live loads with a one-third increase allowed when considering additional short-term wind or seismic loading.

All footings located adjacent to utility lines should bear below a 1:1 plane extending up from the bottom edge of the utility trench. We recommend that continuous foundations be designed with sufficient depth and reinforcing to tolerate the estimated differential settlement.

Our representative should observe all footing excavations prior to placement of reinforcing steel to confirm that they expose suitable material and have been properly cleaned. If soft or loose soils are encountered in the foundation excavations, our field representative may require overexcavation and/or compactive effort or a deeper footing depth before the reinforcing steel is placed.

Structural Mat Foundation

As an alternative to the spread footing foundation described above, the building may be supported on a reinforced concrete mat foundation bearing on a properly prepared and compacted soil subgrade. The mat may be designed for an average allowable bearing pressure of 2,000 pounds per square foot for combined dead plus live loads, with maximum localized bearing pressures of 3,000 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. It would be preferable for the mat foundation to have a thickened perimeter edge that extends at least 8 inches into the soil subgrade below the bottom of the mat or at least 4 inches below the base of the capillary break rock section. This should improve edge stiffness, reduce the potential for mat slab dampness, and increase resistance to lateral loads imposed on the mat.

The mat should be reinforced to provide structural continuity and to permit spanning of local irregularities. A modulus of subgrade reaction (Kv1) of 70 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 differential static settlement, on a preliminary basis a modulus of subgrade reaction (Kv) of 15 pounds per cubic inch (pci) may be assumed for the mat subgrade.

The mat foundation should be reinforced to provide structural continuity and to permit spanning of local irregularities. We recommend the mat be designed with sufficient depth and reinforcing to be able to tolerate the estimated differential settlements.

Prior to mat construction, the mat subgrade should be proof-rolled to provide a smooth firm surface for mat support. Where dampness of the mat would be undesirable, a high-quality membrane vapor barrier should be installed below the mat as described in the section of this report titled "Slabs-on-Grade."

Lateral Loads

Lateral loads may be resisted by base friction between the vapor barrier or damp proofing membrane below the mat and the supporting subgrade and by passive soil pressure acting against the sides of the mat foundation. The structural engineer should consult with the membrane manufacturer for the coefficient of friction to be assumed for design.

Lateral loads may 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 footing design.

In addition, lateral resistance may 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. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing or mat will be landscaped or subject to softening from rainfall and/or surface water runoff.

Settlement

Based on the bearing capacity values presented above, on a preliminary basis, in our experience, the 30-year post-construction differential settlement due to static loads is not expected to exceed 1-inch across the proposed building, provided the building foundations are designed and constructed as recommended. Less differential movement would be expected across a structural mat foundation. Once the range of dead and live loads and the foundation configuration have been developed, we should update the magnitude of total and differential foundation settlement to help establish if an adjustment should be made to the allowable bearing capacity values.

Additional differential settlement may occur as a result of liquefaction and dynamic densification caused by severe ground shaking during a major earthquake, as discussed earlier.

SLABS-ON-GRADE

General Slab Considerations

The surface and near surface soils at this site have a low potential for expansion. To reduce the potential for movement of the slab subgrade, at least the upper 8-inches of expansive soil should be scarified and compacted at a moisture content at least 2 percent above the laboratory optimum. The native or fill soil subgrade should be kept moist up until the time the non-expansive fill and/or aggregate base is placed. Slab subgrades and non expansive fill should be prepared and compacted as recommended in the section of this report titled "Earthwork." Exterior flatwork should be underlain by a layer of non expansive fill as discussed below. The non expansive fill should consist of aggregate base rock or a clayey soil with a plasticity index of 15 or less.

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

Exterior Flatwork

Near surface 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. We recommend that exterior slabs-on-grade be constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs.

Interior Slabs

Concrete slab-on-grade floors for the building (other than the mat slab) should be constructed on a layer of non-expansive fill at least 10-inches thick and constructed on a properly prepared and compacted soil subgrade. Since the ground level garage floor for the building will support vehicle loads, we recommend that the garage floor slabs should be designed more heavily reinforced and at least 5 to 6 inches in thickness, in our opinion.

In areas where dampness of concrete floor slabs or mat would be undesirable, such as within building interiors, concrete slabs and mat should be underlain by at least 4 inches of clean, free-draining gravel, such as ½-inch to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used. The crushed rock should be compacted with vibratory equipment. To reduce vapor transmission up through at-grade concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor retarder meeting the minimum ASTM E 1745, Class C requirements or better. If moisture-sensitive floor coverings are proposed and/or additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms (such as 15-mil thick "Stego Wrap Class A") may be used rather than a Class C vapor retarder. The vapor retarder or barrier should be placed directly below the concrete slab. Sand above the vapor retarder/barrier is not recommended. The vapor retarder/barrier should be installed in accordance with ASTM E 1643. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations.

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

VEHICLE PAVEMENTS

Asphalt Concrete Pavements

Based on the anticipated composition of the surface soils, and an estimated traffic index for the proposed pavement loading conditions, we developed the minimum pavement sections presented in Table 2 below based on Procedure 630 of the Caltrans Highway Design Manual.

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

Table 2. Pavement Sections
Four-Story Live/Work Building
Burlingame, California

Traffic Loading Condition	Design Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)	Total Thickness (inches)
Automobile Parking	4.0	3.0	7.0	10.0
Automobile Access	4.5	3.0	8.0	11.0
Light Truck Traffic	5.0	3.0	9.0	12.0
Moderate Truck Traffic	6.0	4.0	11.0	15.0

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

We recommend that measures be taken to limit the amount of surface water that seeps into the aggregate base and subgrade below vehicle pavements, particularly where the pavements are adjacent to landscape areas. Seepage of water into the pavement base material tends to soften the subgrade, increasing the amount of pavement maintenance that is required and shortening the pavement service life. Deepened curbs extending 4-inches below the bottom of the aggregate base layer are generally effective in limiting excessive water seepage. Other types of water cutoff devices or edge drains may also be considered to maintain pavement service life.

Portland Cement Concrete Pavements

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

To provide a general guideline for the minimum required thickness of PCC pavements, we used information in the Portland Cement Association publication titled "Thickness Design for Concrete Highway and Street Pavements." We assumed "low" subgrade support from the on-site soils, considering typical residential street traffic (up to 25 daily trucks with maximum single axle loads of 22 kips and maximum tandem axle loads of 36 kips), aggregate-interlock joints (i.e. no dowels), no concrete shoulder or curb, a modulus of rupture of concrete of 550 psi (which correlates to a concrete compressive strength of approximately 3,700 psi), at least 10 inches of Class 2 aggregate base below the PCC pavement, and 20-year pavement service life. Sufficient control joints should be incorporated in the design and construction to limit and control cracking.

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

EARTHWORK

Clearing and Subgrade Preparation

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

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

On-site soils, foundation and utility trench excavations, and slab and pavement subgrades should be kept in a moist condition throughout the construction period.

Building Pad Recommendations

In our opinion, the existing fill should be excavated and recompacted below the building, exterior flatwork, pavements, and other site improvements, with a 5 foot overbuild, where possible. The fill should be excavated down to stiff native soil and compacted under our direction. Imported backfill materials should be approved by a member of our staff prior to delivery to the site. The backfill should be moisture conditioned, and compacted as recommended in the section of this report titled "Compaction." A member of our staff should observe and test during re-working of the building pad, as required.

Material For Fill

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

Temporary Slopes, Excavations and Dewatering

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 current OSHA excavation and trench safety standards.

Due to the potential for variation of the on-site soil, field modification of temporary cut slopes may be required. Unstable materials encountered on excavations and slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

As discussed above, ground water will could seasonally be as high as approximately 6 feet below grade. Therefore, construction dewatering may be required depending on the depth of temporary excavations for utility trenches and the ground water level at the time of excavation.

Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions, they should be aware that modifications to the dewatering system may be required during construction depending on the conditions encountered.

Preferably, dewatering of deep utility trench excavations should be carried out in such a manner as to maintain the ground water a minimum of 2 feet below the bottom of the trench excavations. The contractor should design a system to achieve this. Depending upon the depth and dimensions of the excavations, we anticipate that dewatering may be able to be accomplished from pumping from sumps.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the quality of the ground water, and environmental impacts at the site or at nearby locations. These requirements may include storage, testing and/or treatment under permit prior to discharge.

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

Finished Slopes

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

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 on the following page. The relative compaction and moisture content recommended in Table 3 is relative to ASTM Test D1557, latest edition.

Table 3. Compaction Recommendations
Live/Work Building
Burlingame, California

	Relative Compaction*	Moisture Content*
General		
• Scarified subgrade in areas to receive structural fill.	90 percent	At least 2 percent above optimum
• Structural fill composed of native soil.	90 percent	At least 2 percent above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
Pavement Areas		
• Upper 6-inches of soil below aggregate base.	95 percent	Near optimum
• Aggregate base.	95 percent	At least 2 percent above optimum
Utility Trench Backfill		
• On-site soil.	90 percent	Near optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Surface Drainage

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

Infiltration basins or bioswales, if any, preferably should not be placed within about 10 feet of shallow foundation supported structures or slab or flatwork areas. Drains should be provided for infiltration basins that direct water to an appropriate outlet as required by the civil engineer.

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. 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 often do result in recommendations for additional changes to 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 general note sections of the architectural, structural, and civil plans:

"Earthwork, utility trench backfilling, slab subgrade preparation, foundation and slab construction, pavement construction, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated December 6, 2016. Romig Engineers should be notified at least 48 hours in advance of any earthwork or foundation construction and should observe and test during earthwork and foundation construction as recommended in the geotechnical report."

Construction Observation and Testing

Earthwork and foundation construction should be observed and tested by us to 1) confirm 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 exposed during construction, it will be necessary to reevaluate our recommendations.

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REFERENCES

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-10.

California Building Standards Commission, and International Code Council, 2016 California Building Code, California Code of Regulations, Title 24, Part 2.

California Department of Conservation, Division of Mines and Geology (DMG), 1994, Fault-Rupture Hazard Zones in California, Special Publication 42.

California Division of Mines and Geology, 1969, Geologic and Engineering Aspects of San Francisco Bay Fill.

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

Pampeyan, Earl H., 1994, Geologic Map of the Montara Mountain and San Mateo 7-1/2' Quadrangles, San Mateo, County, California, U.S. Geological Survey Map I-2390.

U.S.G.S., 2016, U.S. Seismic Design Maps, Earthquake Hazards Program, http://earthquake.usgs.gov/designmaps/us/application.php

Working Group on California Earthquake Probabilities (WGCEP), 2015, Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF 3), U.S. Geological Survey Open File Report 2013-1165.

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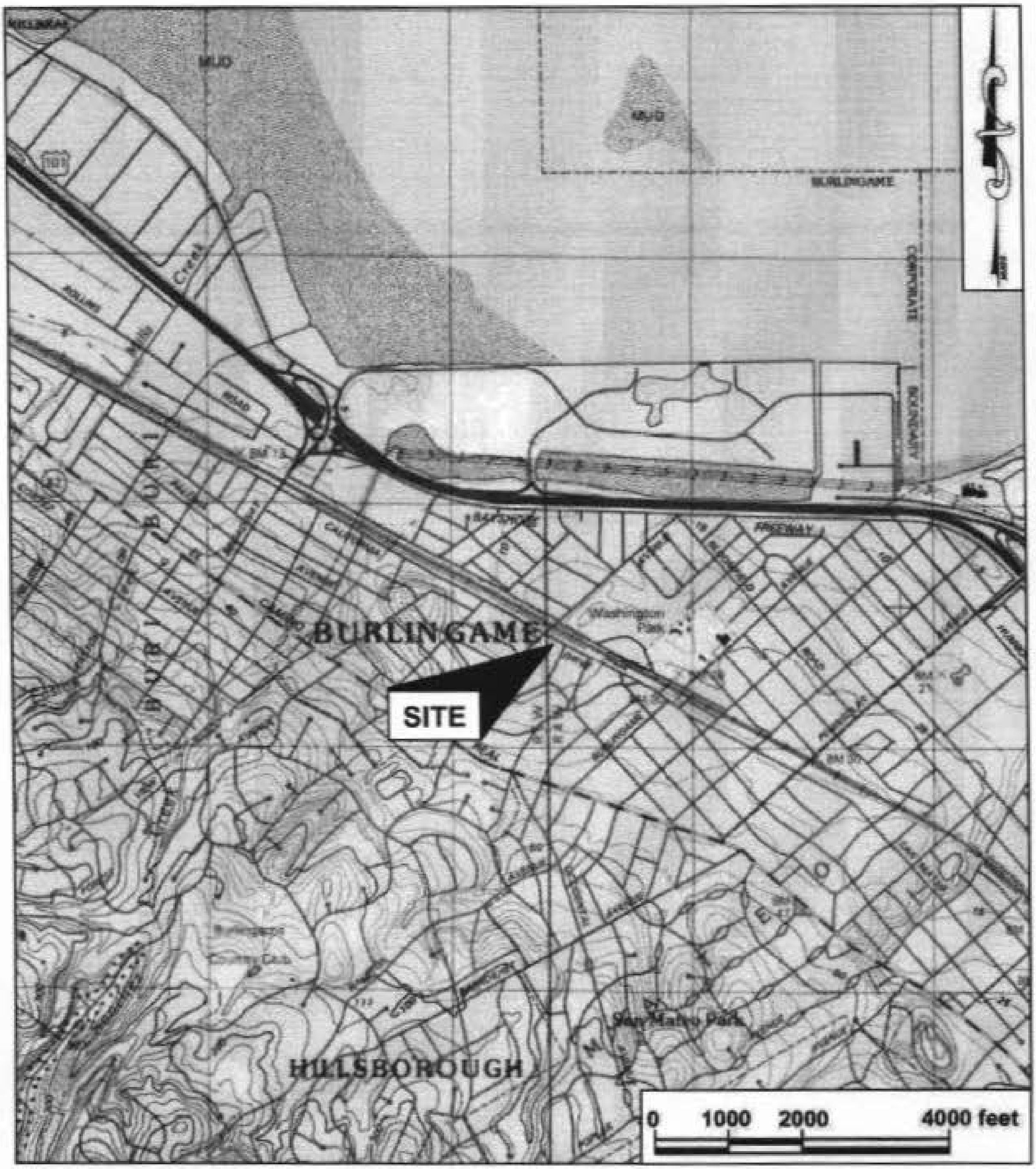
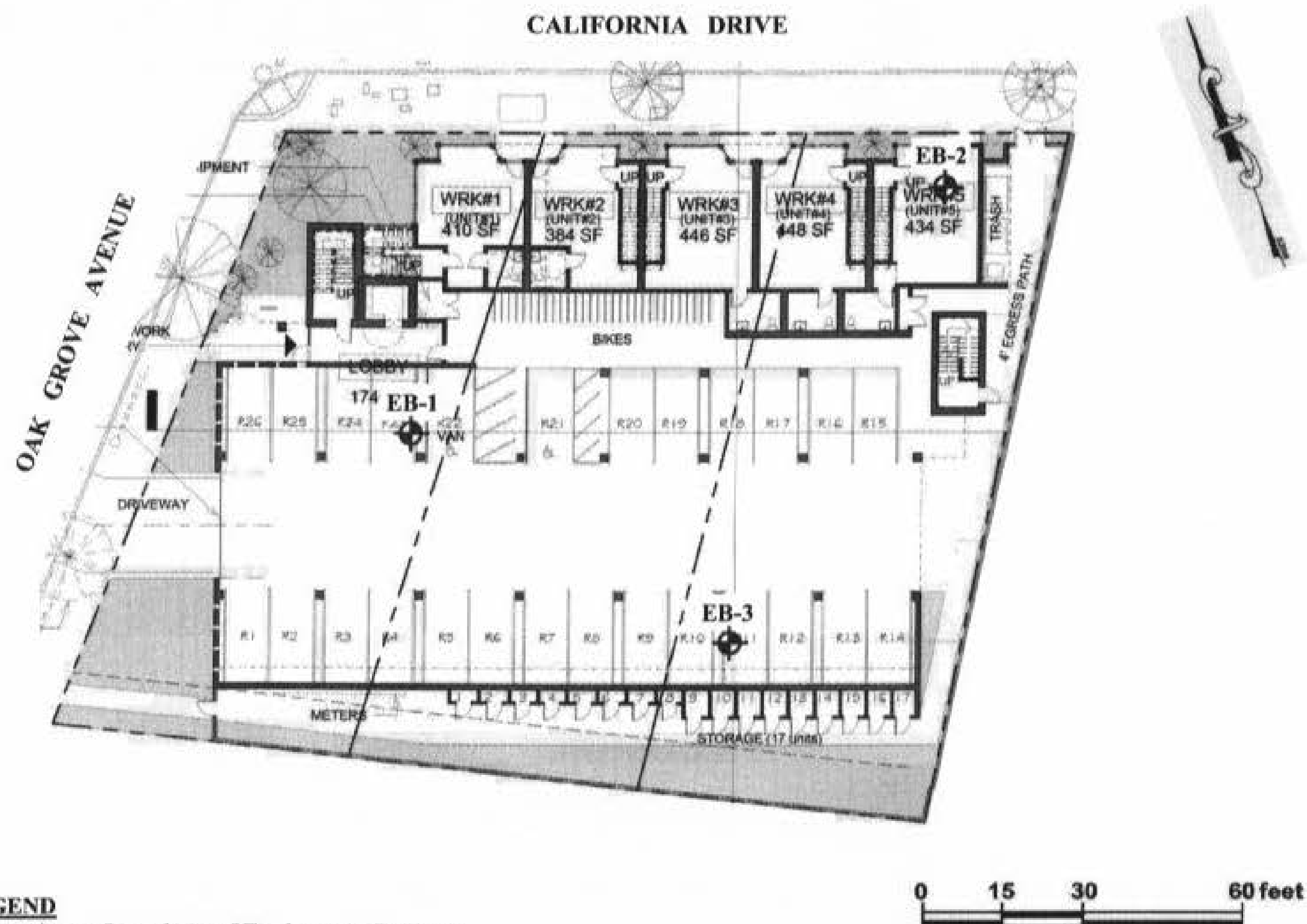


Scale: 1 inch = 2000 feet
Base is United States Geological Survey San Mateo 7.5 Minute Quadrangle, dated 1997.

VICINITY MAP
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

FIGURE 1
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS



LEGEND

EB-3 ⯐ Approximate Locations of Exploratory Borings.

Approximate Scale: 1 inch = 30 feet.

Base is site plan prepared by EASA Architecture, dated August 15, 2016.

SITE PLAN
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

FIGURE 2
DECEMBER 2016
PROJECT NO. 3910-1

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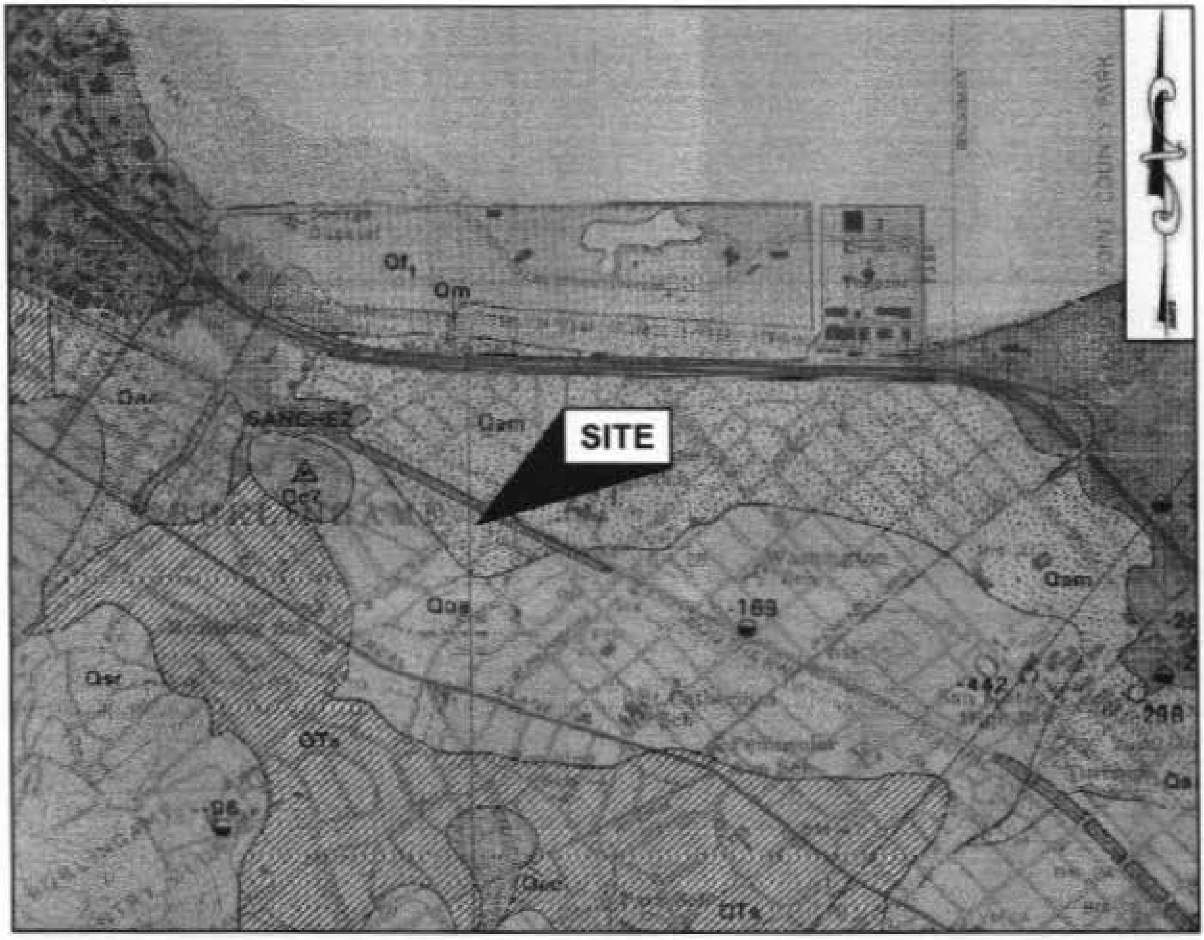

LEGEND

QTs	Sedimentary deposits, undivded
Qam	Medium-grained alluvium
Qac	Coarse-grained alluvium
Qsr	Slope wash, ravine fill, and colluvium
Qf_1	Artificial fill - Unit 1
Qos	Older alluvium

Geologic Contact - dashed where approximate, dotted where inferred.

Fault - dashed where approximate, dotted where inferred.

0 1000 2000 4000 feet

Base is Geologic Map of the Montara Mountain and San Mateo 7.5 Minute Quadrangles, San Mateo County, California, (Pampeyan, 1994).

VICINITY GEOLOGIC MAP
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

FIGURE 3
DECEMBER 2016
PROJECT NO. 3910-1

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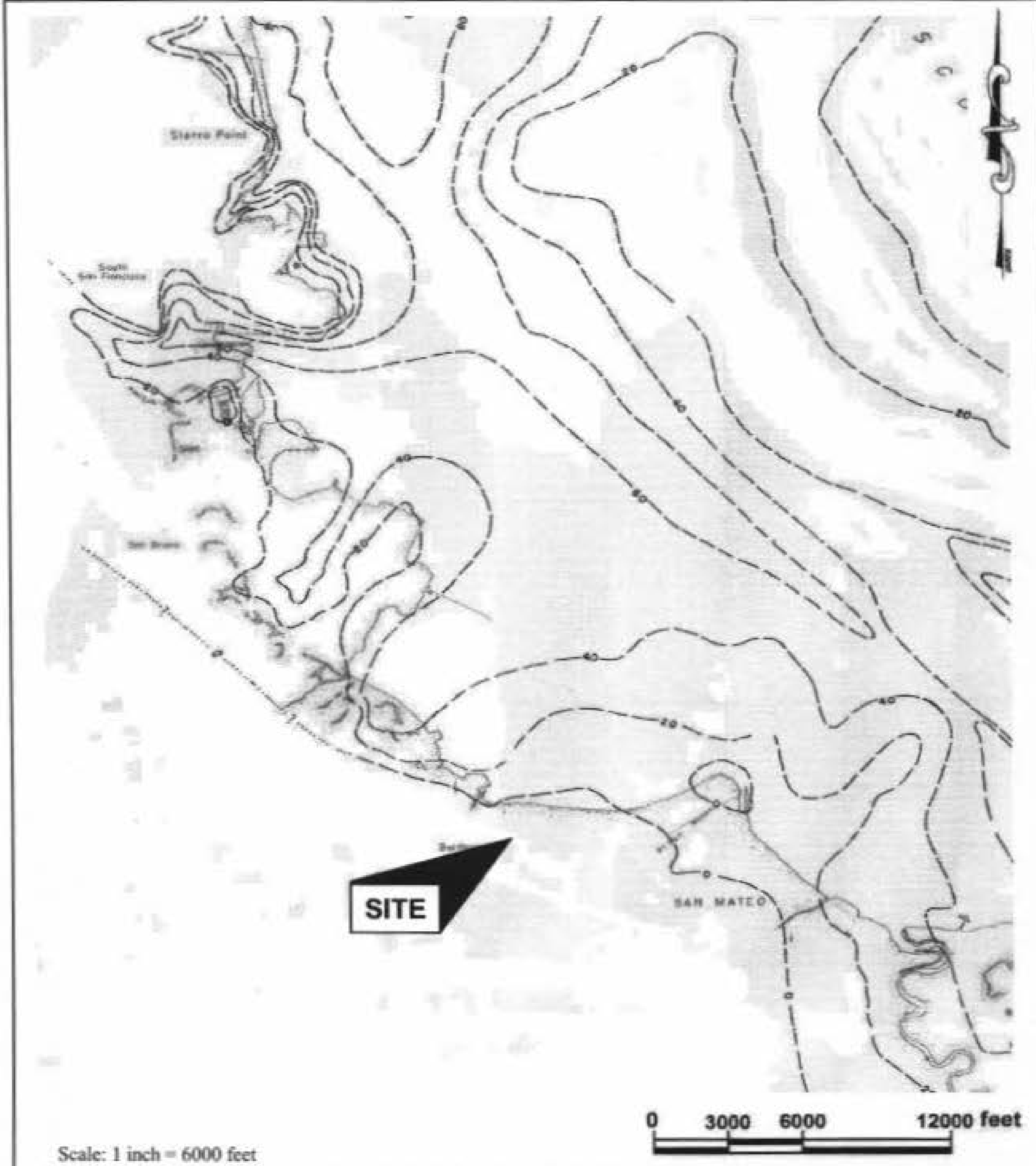

Scale: 1 inch = 6000 feet
Base is Geologic and Engineering Aspects of San Francisco Bay Fill, Special Report 97, Map of "Thickness of Younger Bay Mud", (CDMG, 1966).

CONTOUR MAP OF YOUNG BAY MUD THICKNESS
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

FIGURE 4
NOVEMBER 2016
PROJECT NO. 3910-1

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APPENDIX A

FIELD INVESTIGATION

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

Several tests were performed in the field during drilling. 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 borings log at the appropriate depth. Soil samples were also collected using a 2.5-inch O.D. drive sampler. The blow counts shown on the logs for the 2.5-inch sampler do not represent SPT values and have not been corrected in any way.

The locations of the borings were established by pacing, using the site plan provided to us. The locations should be considered accurate only to the degree implied by the method used.

The boring logs and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the subsurface conditions.

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USCS SOIL CLASSIFICATION

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

RELATIVE DENSITY

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

CONSISTENCY

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

GRAIN SIZES

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

12" | 3" | 0.75" | 4 | 10 | 40 | 200

SIEVE OPENINGS — U.S. STANDARD SERIES SIEVE

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

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

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

KEY TO SAMPLERS

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

KEY TO EXPLORATORY BORING LOGS
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

FIGURE A-1
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: 23 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 10/17/16

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)*
Fill: Orange to dark brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity, tan mottling.	Very Stiff	CL		0					
Light brown, Sandy Lean Clay, moist, fine grained sand, low plasticity, dark orange and black mottling.	Very Stiff	CL				26	11		>4.5
■ Liquid Limit = 28, Plasticity Index = 8.						20	14		4.0
				5					
Light brown, Clayey Sand, moist, fine to coarse grained, fine subangular gravel, low plasticity fines, dark orange and black mottling.	Medium Dense to Dense	SC				28	15		
● 42% Passing No. 200 Sieve.									
Very moist.				10		33	20		
● 44% Passing No. 200 Sieve.				15		28	20		
				20		29	18		
Continued on Next Page									

EXPLORATORY BORING LOG EB-1
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

BORING EB-1
PAGE 1 OF 3
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: 23 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 10/17/16

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, Clayey Sand, moist, fine to coarse grained, low plasticity fines, dark orange and black mottling.	Very Dense	SC		20					
▼ Ground water encountered during drilling at 23 feet.				25		59	11		
Light brown, Clayey Gravel, very moist, fine to coarse grained, fine to medium subangular to subrounded gravel, low plasticity fines.	Medium Dense	GC		30		20	11		
Light brown, Clayey Sand with gravel, very moist, fine to medium grained, fine angular gravel, low plasticity fines.	Dense	SC		35		43	14		
				40		31	19		
Continued on Next Page									

EXPLORATORY BORING LOG EB-1
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

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

LOGGED BY: RL

DEPTH TO GROUND WATER: 23 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 10/17/16

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, Clayey Sand with gravel, very moist, fine to coarse grained, fine angular gravel, low plasticity fines.	Dense to Very Dense	SC		40					
● 37% Passing No. 200 Sieve.						37	20		
				50		52	16		
Bottom of Boring at 50 feet.									
				55					
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.				60					

EXPLORATORY BORING LOG EB-1
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

BORING EB-1
PAGE 3 OF 3
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: 11 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 10/17/16

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)*
5-inches of asphalt concrete.		AC		0					
Fill: Dark brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity.	Firm	CL				6	13		1.0
Light brown, Sandy Lean Clay, moist, fine grained sand, trace subangular gravel, low plasticity, dark orange and black mottling.	Stiff to Very Stiff	CL		5		16	15		>4.5
● 54% Passing No. 200 Sieve.						18	16		4.5
Fine to coarse subangular to subrounded gravel.				10		27	15		0.8
▼ Ground water encountered during drilling at 11 feet.					▼				
Increase in sand and gravel content, very moist.				15		30	20		4.5
Light brown, Clayey Sand, wet, fine to coarse grained, fine to coarse subangular gravel, low plasticity fines, dark orange and black mottling.	Medium Dense	SC							
● 34% Passing No. 200 Sieve.						21	17		
Continued on Next Page									

EXPLORATORY BORING LOG EB-2
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: 11 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 10/17/16

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, Clayey Sand, very moit, fine to coarse grained, fine subangular gravel, low plasticity fines, dark orange and black mottling.	Medium Dense	SC		20					
● 34% Passing No. 200 Sieve.				25	●	19	17		
				30		27	23		2.3
				35		15	30		2.3
Bottom of Boring at 35 feet.									
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.				40					

EXPLORATORY BORING LOG EB-2
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

BORING EB-2
PAGE 2 OF 2
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: 14 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 10/17/16

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)*
5-inches of asphalt concrete.		AC		0					
Fill: Light brown, Sandy Lean Clay, very moist, fine grained sand, low plasticity, dark brown and orange mottling.	Soft	CL				3	17		1.0
Light brown to brown, Sandy Lean Clay, very moist, fine sand, low plasticity, dark brown mottling.	Stiff to Very Stiff	CL				14	16	0.4	3.3
				5		28	18	0.8	2.3
Light brown, Clayey Sand, very moist, fine to coarse grained, fine subangular gravel, low plasticity fines.	Medium Dense	SC							
● 39% Passing No. 200 Sieve.				10		23	19		
▼ Ground water encountered during drilling at 14 feet.				15		28	19		
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.				20		26	18		
Bottom of Boring at 20 feet.									

EXPLORATORY BORING LOG EB-3
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

BORING EB-3
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS

APPENDIX B

LABORATORY TESTS

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

The natural moisture content was determined in accordance with ASTM D 2216 on nearly all of the samples recovered from the borings. This test determines the moisture content, representative of field conditions, at the time the samples were collected. The results are presented on the boring logs, at the appropriate sample depths.

The Atterberg Limits were determined on 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 and on the log of Boring EB-1 at the appropriate sample depth.

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

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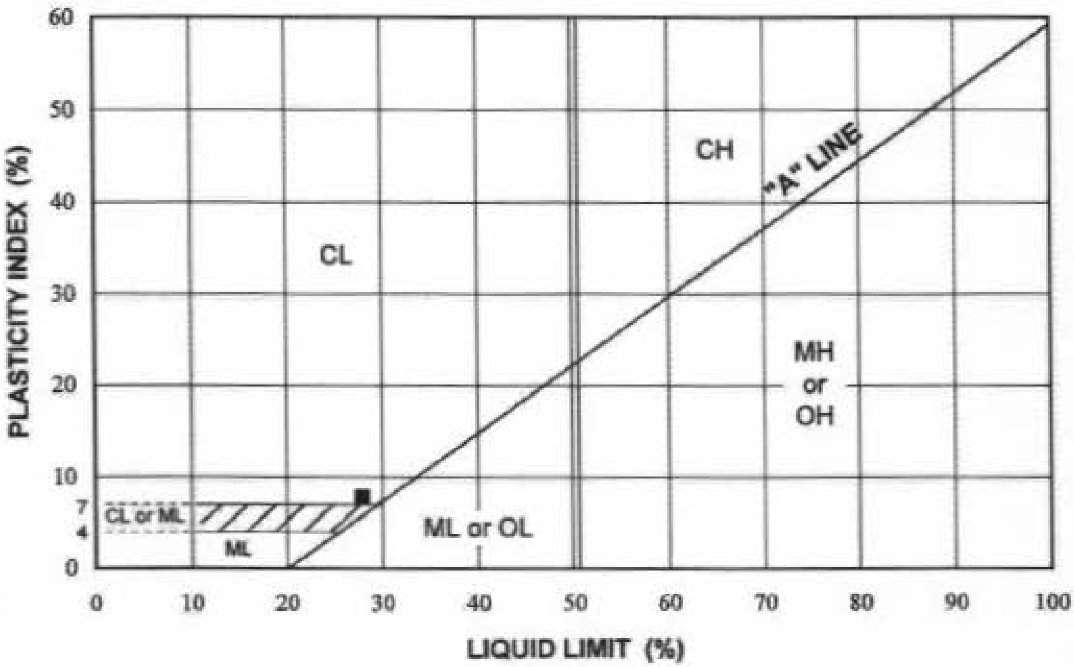

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	3-4.5	14	28	8	-75		CL

PLASTICITY CHART
FOUR-STORY LIVE/WORK BUILDING
BURLINGAME, CALIFORNIA

FIGURE B-1
DECEMBER 2016
PROJECT NO. 3910-1

ROMIG ENGINEERS

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