GEOTECHNICAL INVESTIGATION

THREE-STORY MIXED-USE BUILDING
3585 EL CAMINO REAL
PALO ALTO, CALIFORNIA 94306

Prepared for
Ms. Fangzhou Song
c/o KSS Management
22000 Rolling Hills Road
Saratoga, California 95070

August 2017
Project No. 4088-1

August 9, 2017
4088-1

Ms. Fangzhou Song
c/o KSS Management
22000 Rolling Hills Road
Saratoga, California 95070

**RE: GEOTECHNICAL INVESTIGATION
THREE-STORY MIXED-USE BUILDING
3585 EL CAMINO REAL
PALO ALTO, CALIFORNIA**

Dear Ms. Song:

In accordance with your request, we have performed a geotechnical investigation for the proposed three-story mixed-use building to be constructed at 3585 El Camino Real in Palo Alto, California. The accompanying report summarizes the results of our field exploration, laboratory testing and engineering analysis, and presents geotechnical recommendations for the proposed building.

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

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

Very truly yours,

ROMIG ENGINEERS, INC.

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AUGUST 2017

ROMIG ENGINEERS

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3585 EL CAMINO REAL
PALO ALTO, CALIFORNIA 94306

INTRODUCTION

We are pleased to present this geotechnical investigation report for the proposed three-story mixed-use building to be constructed at 3585 El Camino Real in Palo Alto, 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 design and construction recommendations for the proposed building.

Project Description

The project consists of constructing a three-story mixed-use building at the subject property in Palo Alto. The building is expected to have a footprint of about 2,100 square feet and will generally be located at the southern portion of the lot. We note the building will not include a basement level. The remainder of the lot will consist of a parking lot. The existing industrial building at the northern portion of the lot will be demolished prior to construction.

Scope of Work

Our scope of work for this investigation was presented in detail in our agreement with Ms. Fangzhou Song, dated May 8, 2017. In order to complete our 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 two 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 surface and near-surface soils encountered at the site.

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

Limitations

This report has been prepared for the exclusive use of Ms. Fangzhou Song for specific application to developing geotechnical design criteria for the proposed three-story mixed-use building to be constructed at 3585 El Camino Real in Palo Alto, California. We make no warranty, expressed or implied, for the services we performed for this project. Our services were performed in accordance with geotechnical engineering principles generally accepted at this time and location. The report was prepared to provide engineering opinions and recommendations only. In the event that there are any changes in the nature, design or location of the project, or if any future improvements are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) The project changes are reviewed by us, and 2) The conclusions and recommendations presented in this report are modified or verified in writing.

The analysis, conclusions, and recommendations presented in this report are based on site conditions as they existed at the time of our investigation; the currently planned improvements; review of 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.

PREVIOUS ENVIRONMENTAL WORK

Based on our brief review of the environmental report prepared by Frey Environmental, Inc. (dated January 9, 2009), and the case closure report prepared by the County of Santa Clara Department of Environmental Health (dated August 31, 2016), we understand that an auto repair and gasoline station formerly occupied the subject site, and two 2,000-gallon leaking underground storage tanks (USTs) were removed in 1986. We understand the UST excavation, located within the southern portion of the lot, was backfilled with about 12 to 15 feet of undocumented fill (refer to Figure 2).

These reports also indicated that residual petroleum-based contaminants were left in-place below the central and northern portions of the site, below a depth of about 20 feet. The County stated in their case closure report that no further action was required regarding the former petroleum release at the site.

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on June 5, 2017. Subsurface exploration was performed using a Mobile B-53 truck-mounted drill rig equipped with hollow-stem augers. Two exploratory borings were advanced to depths of 25 and 30 feet. The approximate locations of the borings are presented 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 mixed residential and commercial area north of the intersection of El Camino Real and Matadero Avenue. At the time of our investigation, the northern portion of the lot was occupied by a single story garage structure which appeared to be of wood-frame construction. The remainder of the lot was unoccupied, and covered with a concrete and asphalt concrete surface.

Based on the age, we expect that the existing garage structure is likely supported on a shallow foundation system, although the depth and width of the foundations are unknown. No obvious significant distress was noted at the building exterior. The concrete and asphalt concrete surface generally appeared to be in fair to poor condition, where many alligator cracks were noted.

Subsurface Conditions

At the location of our Exploratory Boring EB-1, which was advanced within the former UST pit backfill near the south corner of the property, we encountered about 15 feet of fill consisting of a mixture of stiff to very stiff sandy lean clay of moderate plasticity and loose to medium dense clayey sand. Beneath the fill, we encountered about 6.5 feet of native, stiff sandy lean clay of moderate plasticity underlain by medium dense to dense clayey sand to the maximum depth explored of 30 feet.

At Boring EB-2, which was advanced near the north side of the proposed building, we encountered about 12 feet of stiff to very stiff sandy lean/fat clay of moderate to high plasticity underlain by about 9.5 feet of stiff to very stiff sandy lean clay of low to moderate plasticity. The clayey soils were underlain by dense silty sand to the maximum depth explored of 25 feet.

We noted a noticeable hydrocarbon odor in three soil samples collected between depths of about 18.5 to 30 feet in Boring EB-1, and in one soil sample collected between depths of about 23.5 and 25 feet in Boring EB-2 that appeared to be related to residual petroleum-based contaminants, as discussed above. We also note that the soils encountered within the upper 15 feet in our Boring EB-1 appeared to be backfill soils that were placed during the former UST pit backfill. Based on the resistance blow count and appearance of the fill soils, the former UST backfill did not appear to have been placed and compacted to today's standards as engineered fill and these undocumented fills will likely settle over the years, particularly during times of seasonally heavy rainfall or irrigation when the fill becomes wet, or during strong seismic shaking.

A Liquid Limit of 50 and a Plasticity Index of 27 were measured on a native sample of near-surface soil obtained from Boring EB-2. These test results indicate that the surface and the near-surface soils at the site have high plasticity and a high potential for expansion.

Ground Water

Free ground water was encountered at a depth of approximately 23.5 feet in our borings during our subsurface exploration. The borings were backfilled with grout immediately after drilling and sampling was completed; therefore a stabilized ground water level measurement was not obtained. 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.

We have encountered ground water at depths ranging from about 15 to 18 feet below grade at two properties located within a radius of about 650 feet of the subject site. In addition, information presented in Seismic Hazard Zones Report for the Palo Alto Quadrangle (California Geological Survey, 2006) indicates that the historical high ground water level near the site is about 13 feet.

GEOLOGIC SETTING

Geologic information that we reviewed for this study indicates the site is underlain by Holocene-age flood plain deposits, Qhfp (Brabb, Graymer and Jones, 2000). These deposits are generally found to consist of medium to dark gray, sandy to silty clay. Lenses of coarser material (silt, sand, and pebbles) may be locally present. The geology of the general area of the site is shown on the Vicinity Geologic Map, Figure 3.

The lot and the immediate site vicinity are located in an area that slopes down gently to the northeast towards the San Francisco Bay. The site is located at an elevation of approximately 35 feet above mean sea level (Figure 1).

Faulting and Seismicity

There are no mapped through-going faults across or adjacent to the site, and the site is not located in a State of California Earthquake Fault Zone, an area where the potential for fault rupture is considered probable. The closest active fault is the San Andreas fault, which is located approximately 5.6 miles southwest of the property. Thus, the likelihood of surface rupture 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 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 experienced large, destructive earthquakes in 1838, 1868, 1906, and 1989.

The faults considered most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras faults. The San Gregorio fault is located approximately 16 miles southwest of the site. The Hayward and Calaveras faults are located approximately 13 and 17 miles northeast of the site, respectively. These faults and significant earthquakes that have been documented in the Bay Area are listed on Table 1 below, and are shown on the Regional Fault and Seismicity Map, Figure 4.

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
Three-Story Mixed-Use Building
Palo Alto, California

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

Earthquake Design Parameters

The State of California 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 Response Acceleration parameters and site coefficients may be taken directly from the U.S.G.S. website based on the longitude and latitude of the site. For site latitude (37.4190), longitude (-122.1338) and Site Class D, design parameters are presented on Table 2 on the following page.

Table 2. 2016 CBC Seismic Design Criteria
Three-Story Mixed-Use Building
Palo Alto, California

Spectral Response Acceleration Parameters			Design Value
Mapped Value for Short Period	-	S_S	1.536
Mapped Value for 1-sec Period	-	S_1	0.702
Site Coefficient	-	F_a	1.0
Site Coefficient	-	F_v	1.5
Adjusted for Site Class	-	S_{MS}	1.536
Adjusted for Site Class	-	S_{M1}	1.053
Value for Design Earthquake	-	S_{DS}	1.024
Value for Design Earthquake	-	S_{D1}	0.702

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.

- 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 to occur 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 proposed construction, as is typical for sites throughout the Bay Area. The building should be designed and constructed in accordance with current earthquake resistance standards.

- Liquefaction - Liquefaction occurs when saturated sandy soils lose strength during earthquake shaking. Ground settlement often accompanies liquefaction. Soils most susceptible to liquefaction are saturated, loose, sandy silts, silty sands, and uniformly graded sands. Our borings generally encountered primarily stiff clayey soils and relatively dense sands; therefore, the likelihood of significant liquefaction occurring in the soils encountered at the site is low. The site is also not included within a State of California liquefaction hazard zone.

- 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. As discussed previously, about 15 feet of undocumented fill was encountered at the former UST pit backfill near the south corner of the property; and this fill is expected to be prone to some dynamic densification settlement. Due to the variable and unknown nature of this fill material, it is difficult to estimate the amount of potential dynamic settlement at the site. However, since the proposed building is expected to be supported on relatively deep drilled piers extending below the fill, the likelihood of significant differential compaction affecting the proposed building is low provided the recommendations presented in this report are followed during design and construction.

- Expansive Soil - Based upon the results of the laboratory testing, the surface and near-surface soils encountered at the site are highly expansive and subject to expansion and contraction during wetting/drying cycles. Since we recommend supporting the proposed building on relatively deep drilled piers, in our opinion, the likelihood of significant damage to the proposed building from expansive soil movement is low. A potential also exists for damage to pavements, flatwork, and other improvements supported at grade or near grade.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed building, provided the recommendations presented in this report are followed during design and construction. The primary geotechnical concerns for the proposed building are:

- The presence about 15 feet of undocumented fill encountered in Boring EB-1 near the southern corner of the lot, which was related to the former UST pit backfill operation.

- The presence of highly expansive native surface and near-surface soils at the site, outside of the former UST backfill area.

- The potential for severe ground shaking at the site during a major earthquake.

As disussed previously, a portion of the proposed building is expected be underlain by undocumented fill, within the area of the former UST pit backfill (refer to Figure 2). The remainder of the proposed building footprint is expected to be underlain by highly expansive soils. In our opinion, the building will likely be subject to differential movement due to static settlement of the undocumented fill under the building loads, particularly if the fill soils becomes wet or saturated, and due to dynamic densification settlement during moderate to string earthquake shaking.

In addition, the building will likely be subject to differential movement due to expansive soil movement from significant volume changes caused by seasonal fluctuations in the soil moisture content of the native clayey soils. Due to the variable subsurface conditions encountered in our borings across the limits of the building, we recommend supporting the building on a pier and grade beam foundation system. In addition, a structural slab spanning across the pier and grade beam foundations is recommended, at least within the former UST backfill area.

We note that piers to be constructed within the limits of the former UST pit backfill should extend into undisturbed stiff/dense native soil below the fill. These piers may need extend as deep as about 20 feet below the ground surface, depending on the extent of fill encountered at each pier location. The piers located outside the limits of the former fill could be designed with a shallower embedment depth. Specific geotechnical recommendations are presented in the following sections of this report.

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 project plans for conformance with our report recommendations; and 2) observe and test during earthwork and foundation construction.

FOUNDATIONS

Pier and Grade Beam Foundation

In our opinion, the proposed building should be supported on a pier and grade beam foundation system extending into competent native soil, below the upper fill. In our opinion, the piers constructed within the limits of the former UST pit backfill should have a diameter of at least 16 inches, extend at least 20 feet below the bottom of the grade beams, and at least 5 feet into undisturbed stiff/dense native soil, whichever is deeper. The piers located outside the limits of the former fill should have a diameter of at least 16 inches should extend at least 12 feet below the bottom of the grade beams. The structural engineer may require the piers to extend deeper to resist vertical and lateral loads.

Piers should be designed for an allowable skin friction of 150 pounds per square foot for dead plus live loads within the fill soils, and an allowable skin friction of 350 pounds per square foot for dead plus live loads within the native soils. For piers located within the limits of the former UST pit backfill, skin friction of the soil against the upper 6 feet of the piers should be neglected in design.

For piers located outside the limits of the former fill, skin friction of the soil against the upper 1 foot of the piers should be neglected in design. Piers should be reinforced in the vertical direction with the equivalent of at least four No. 5 bars. Piers should have a center-to-center spacing of at least three pier diameters.

We recommend that relatively rigid grade beams be constructed between piers as required by the structural engineer. Grade beams should be sufficiently reinforced top and bottom, as determined by the structural engineer. The grade beam should extend at least 12 inches below the slab subgrade elevation to help limit the infiltration of surface runoff beneath the proposed building. Due to the potential for expansive soil movement, the grade beams should be designed for an uplift pressure of 2,000 pounds per square feet, acting against the bottom of the grade beam surface.

Pier drilling should be observed by our representative to confirm that the pier holes extend the required minimum depth, are embedded into competent native soil, and are properly cleaned of all loose or soft to firm soil, fill and debris. The pier depths recommended above may require adjustment if differing conditions are encountered during drilling. Our field representative should confirm the extent of fill during pier drilling, particularly within the limits of the former UST pit backfill.

Concrete should be placed in the pier holes as soon as practical after drilling is completed. Limited ground water may seep into the pier holes during pier drilling and it is possible that ground water seepage or the granular portions of the fill could cause some sloughing or caving of the pier holes. This can be further evaluated during drilling of the initial piers. If ground water cannot be effectively pumped from the pier holes, concrete will need to be placed in the pier holes by tremie method. If caving conditions occur, scheduling more than one concrete placement per day or casing of the pier holes may be required.

Lateral Loads

Lateral loads on the piers may be resisted by passive earth pressure based upon an equivalent fluid pressure of 300 pounds per cubic foot, acting on 1.5 times the projected area of the pier. The passive resistance of the upper 2 feet of the piers should be neglected in design.

Settlement

Thirty-year post-construction differential movement due to static loads is not expected to exceed about 3/4-inch across the proposed building supported on a drilled pier and grade beam foundation, provided the foundations are designed and constructed as recommended previously.

SLABS-ON-GRADE

General Slab Considerations

The native near-surface soils at the site have a high expansion potential. Expansive soils have a tendency to expand due to increases in moisture content and shrink as they dry. This can result in some slab cracking and heave regardless of the geotechnical measures implemented. Our recommendations below will help mitigate the impacts of the expansive soils beneath slabs-on-grade, but will not eliminate the risk entirely.

To reduce the potential for movement of the slab subgrade, at least the upper 6-inches of expansive soil should be scarified and compacted at a moisture content at least 3 percent 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 is placed. 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 slab-on-grade areas. 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.

For better slab performance, at least the upper 2 to 3 feet, and preferably the full depth, of the existing fill outside the building footprint, should be over-excavated and compacted to modern day construction standards on a series of level benches. If the undocumented fills are not reworked below flatwork or improvement areas, there would be a potential for differential settlement across these areas, particularly during times of seasonally heavy rainfall or irrigation when the fill becomes wet, or during strong seismic shaking.

For interior slabs, as discussed previously, the slab should be designed and constructed as a structural slab spanning across the pier foundations.

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 12 inches of Class 2 aggregate base. The potential for distress to exterior slabs due to expansive soil movements could be reduced by placing and compacting 6 inches of non-expansive fill, or aggregate base, below the minimum 12-inch thick layer of aggregate base recommended above (i.e., a total of 18 inches of non-expansive fill).

To improve performance, exterior slabs-on-grade 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-on-Grade

Concrete slab-on-grade floors outside the former UST backfill area should be constructed on a layer of non-expansive fill at least 18 inches thick that is placed and compacted on a properly prepared and compacted soil subgrade. Due to the potential for expansive soil movement, we recommend that slab-on-grade floors be at least 5 inches thick, and be reinforced with sufficient steel reinforcement to span across local irregularities.

In areas where dampness of concrete floor slabs would be undesirable, such as within the building interior, concrete slabs should be underlain by at least 4 inches of free-draining gravel, such as ½- to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used for this capillary break material. The crushed rock 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 floor slabs, 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"). The vapor barrier should be placed directly below the concrete 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 effected significantly by the water:cement ratio of the concrete mix, with lower water:cement ratios producing more damp-resistant 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 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 slab moisture content for adherence to the manufacturer's requirements and to determine whether a longer drying time is necessary.

Structural Slabs

As discussed previously, interior slabs within the UST backfill area should be supported structurally on the drilled pier foundation system. If structural slabs will be used outside the UST backfill area, the slabs could be constructed with a 2-inch thick void form used below the slabs. If void form will be used, non-expansive fill (such as crushed rock and/or aggregate base) will not be required below the slabs. Where floor dampness is a concern, a water-proofing membrane that will adhere to the concrete (such as preproof or polygard) should be placed between the void form and slab. The contractor will need to exercise care to maintain the integrity of the void forms while placing reinforcing steel and concrete.

VEHICLE PAVEMENTS

Based upon the available laboratory test results and our field investigation, an R-value of 5 appears to be appropriate for design of the parking areas and traffic driveways. Using estimated traffic indices for various pavement loading conditions, we developed the minimum pavement section thicknesses presented in Table 3 on the following page based on the procedure included in Chapter 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.

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

Table 3. Minimum Asphalt Concrete Pavement Section Thicknesses
Three-Story Mixed-Use Building
Palo Alto, California

General Traffic Condition	Traffic Index	AC Thickness (inches)	Aggregate Base* (inches)	Total Section (inches)
Automobile Parking	4.0	3.0	7.0	10.0
Automobile Access	4.5	3.0	8.0	11.0
Light Truck Access	5.0	3.0	10.0	13.0
Moderate Truck Access	6.0	3.5	13.0	16.5
Heavy Truck Access	6.5	3.5	15.0	18.5

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

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as concrete, pavement, abandoned utility lines, soft or loose soils, vegetation, root systems, topsoil, designated fill, etc., should be cleared from areas of the site to be built or paved on. 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."

On-site soils, foundation and utility trench excavations, and slab/flatwork subgrades should be kept in a moist condition throughout the construction period to mitigate the potential effects of the moderately to highly expansive on-site soils on the proposed improvements.

Compaction

Scarified soil surfaces and all structural fill should be placed and 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 4 below. The relative compaction and moisture content recommended in Table 4 is relative to ASTM Test D1557, latest edition.

Table 4. Compaction Recommendations
Three-Story Mixed-Use Building
Palo Alto, California

General	Relative Compaction*	Moisture Content*
• Scarified native subgrade in areas to receive structural fill.	87 to 92 percent	At least 3 percent above optimum
• Structural fill composed of expansive native soils.	87 to 92 percent	At least 3 percent above optimum
• Structural fill composed of undocumented fill soils.	90 percent	Above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
• Structural fill below a depth of 4 feet.	93 percent	At least 2 percent above optimum
Pavement Subgrade		
• Upper 6-inches of soil below aggregate base.	95 percent	3 percent above optimum
• Aggregate base.	95 percent	Above optimum
Utility Trench Backfill		
• On-site expansive native soils.	87 to 92 percent	At least 3 percent above optimum
• On-site undocumented fill soils.	90 percent	Above optimum
• Imported sand.	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

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

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 and shoring 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 the structures near excavations and trenches 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 preferably no steeper than 2.5:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion that could require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted to 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.

Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during the first two years following construction. We recommend preparing an as-built plan showing the locations of surface and subsurface drain lines and clean-outs. The drainage facilities should be periodically checked to verify that they are continuing to function properly. It is likely the drainage facilities will need to be periodically cleaned of silt/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 contained in this report. We should be provided with these plans as soon as possible upon completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review process. In addition, it should be noted that many of the local building and planning departments now require "clean" geotechnical plan review letters prior to acceptance of plans for their final review. Since our plan reviews typically result in recommendations for modification of the plans, our generation of a "clean" review letter often requires two iterations. At a minimum, we recommend the following note be added to the plans:

"Earthwork, slab subgrade and non-expansive fill preparation, foundation and grade beam construction, pier drilling, void form placement, pavement construction, utility trench backfilling, site drainage and grading should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated August 9, 2017. 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

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

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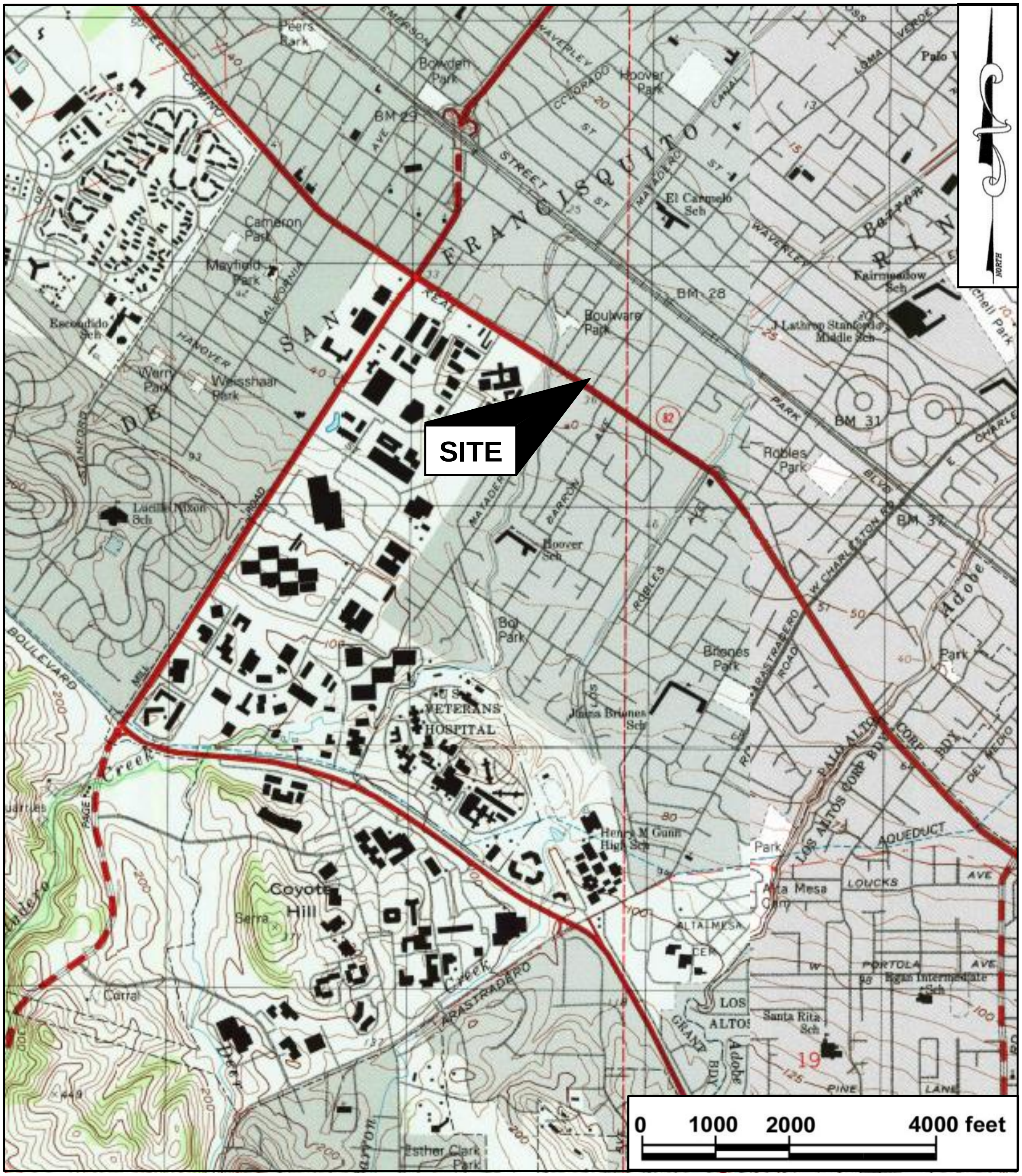


Scale: 1 inch = 2000 feet

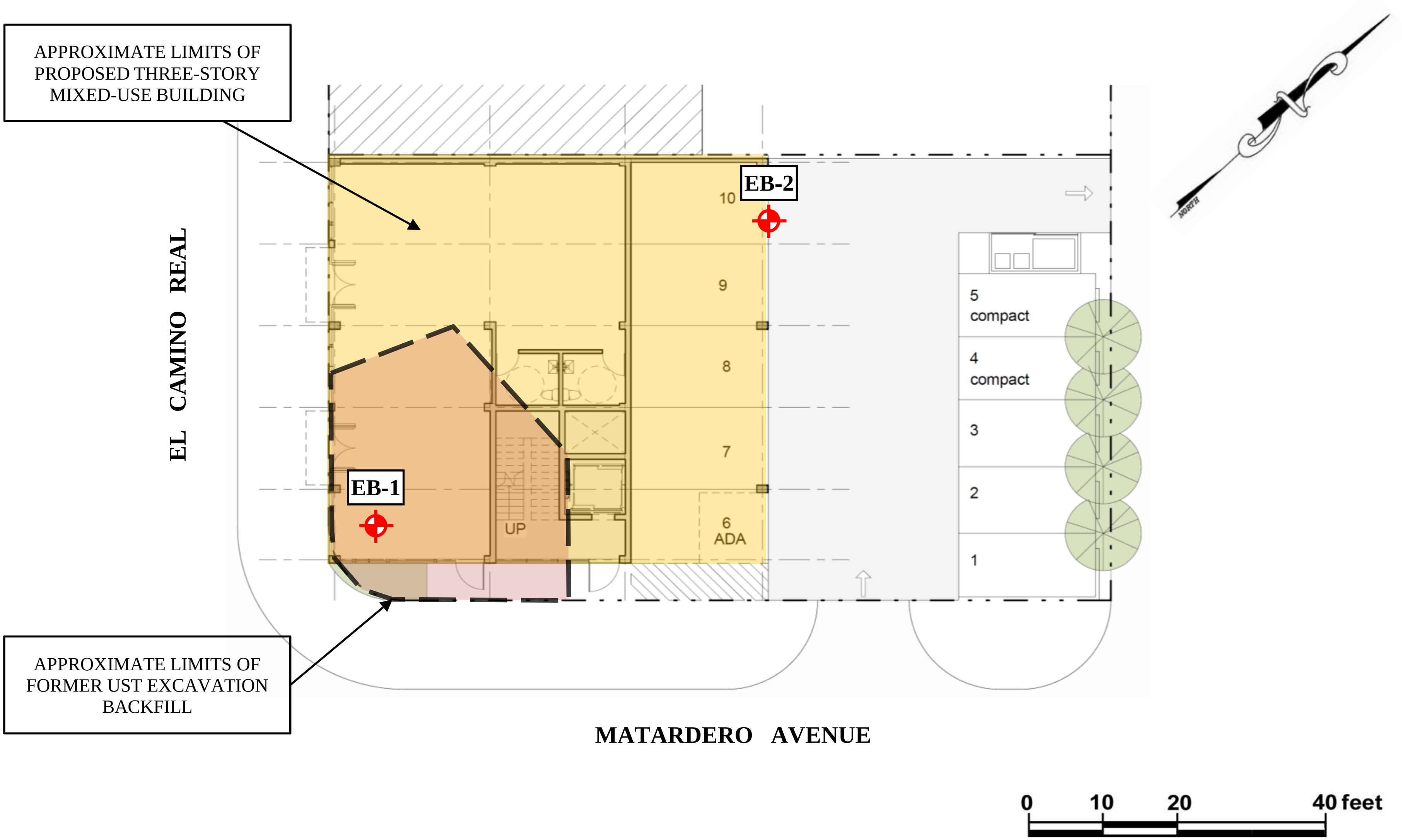
Base is United States Geological Survey Palo Alto and Mountain View 7.5 Minute Quadrangles, dated 1997.

VICINITY MAP | **FIGURE 1**

THREE-STORY MIXED-USE BUILDING | AUGUST 2017

PALO ALTO, CALIFORNIA | PROJECT NO. 4088-1

ROMIG ENGINEERS



LEGEND

EB-2 Approximate Locations of Exploratory Borings.

Approximate Scale: 1 inch = 20 feet.

Base is site plan prepared by Joseph Bellomo Architects, dated March 23, 2017.

SITE PLAN
THREE-STORY MIXED-USE BUILDING
PALO ALTO, CALIFORNIA

FIGURE 2
AUGUST 2017
PROJECT NO. 4088-1

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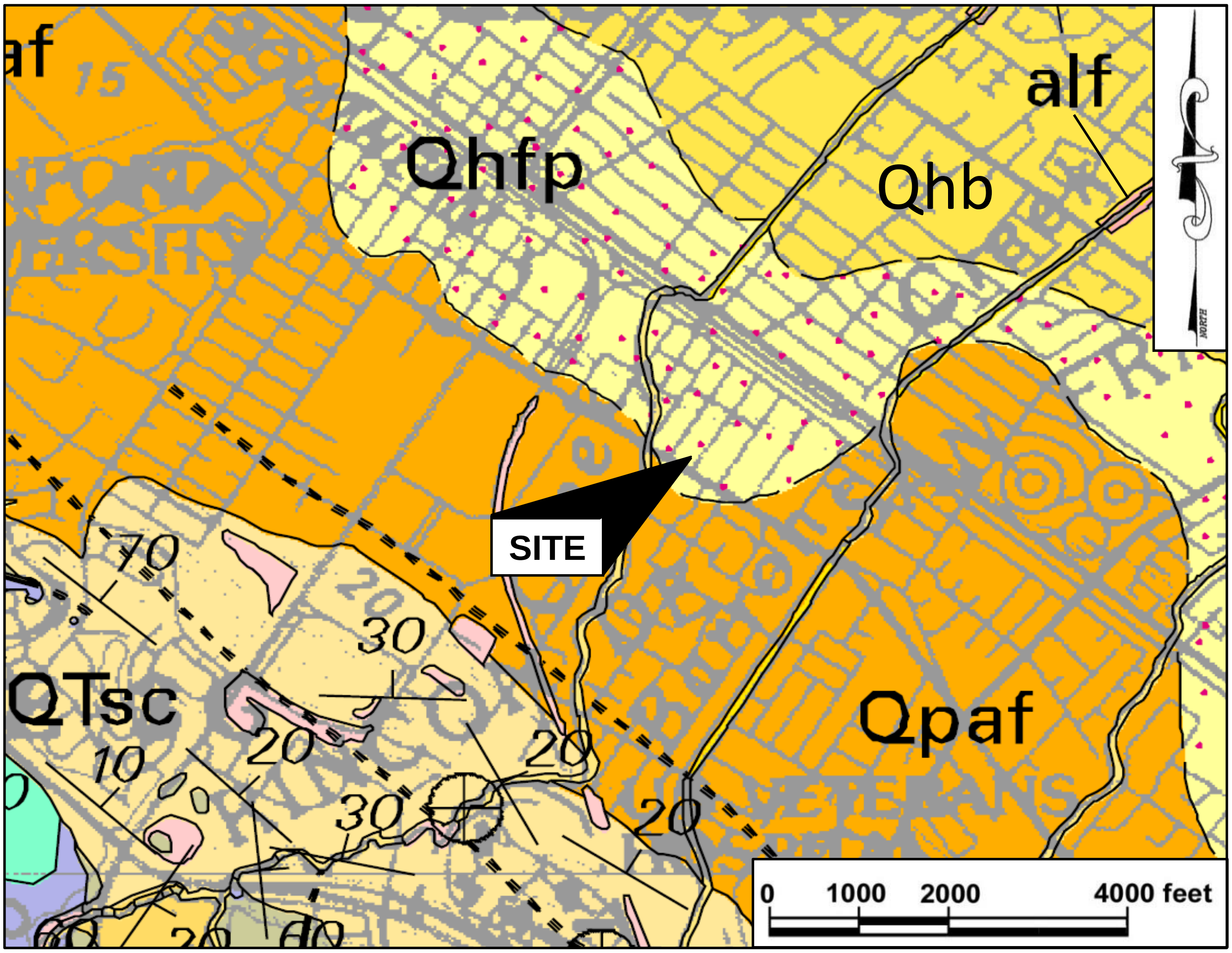

MAP LEGEND

alf	Artificial levee fill		Geologic Contact - dashed where approximate, dotted where inferred.
Qhb	Basin deposits		Fault - dashed where approximate, dotted where inferred.
Qhfp	Floodplain deposits		
Qpaf	Alluvial fan and fluvial deposits	30	Strike and dip of bedding
QTsc	Santa Clara Formation		

Scale: 1 inch = 2000 feet

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

VICINITY GEOLOGIC MAP
THREE-STORY MIXED-USE BUILDING
PALO ALTO, CALIFORNIA

FIGURE 3
AUGUST 2017
PROJECT NO. 4088-1

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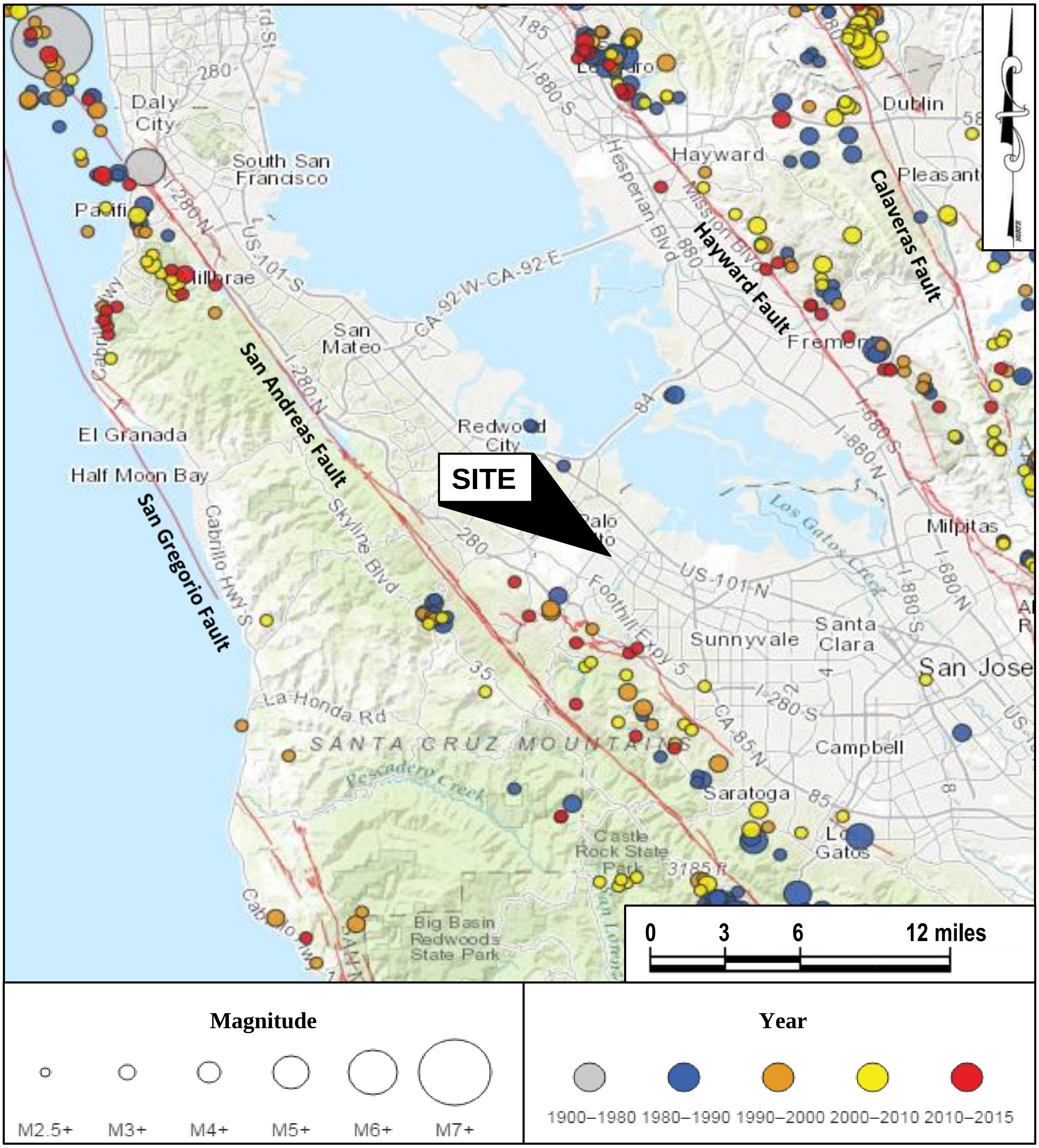

Earthquakes with M5+ from 1900 to 1980, M2.5+ from 1980 to January 2015. Faults with activity in last 15,000 years. Based on data sources from Northern California Earthquake Data Center and USGS Quaternary Fault and Fold Database, accessed July 2017.

REGIONAL FAULT AND SEISMICITY MAP
THREE-STORY MIXED-USE BUILDING
PALO ALTO, CALIFORNIA

FIGURE 4
AUGUST 2017
PROJECT NO. 4088-1

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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 evaluated and classified in accordance with the Unified Soil Classification System. The logs of our borings, and a summary of the soil classification system used on the logs (Figure A-1), are 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 boring logs at the appropriate depths. Soil samples were also collected using 2.5- and 3.0-inch O.D. drive samplers. The blow counts shown on the logs for these larger samplers do not represent SPT values and have not been corrected in any way.

The location of the borings were established by pacing using preliminary site plan prepared by Joseph Bellomo Architects, dated March 23, 2017, and 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
THREE-STORY MIXED-USE BUILDING
PALO ALTO, CALIFORNIA

FIGURE A-1
AUGUST 2017
PROJECT NO. 4088-1

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

DEPTH TO GROUND WATER: 23.5 feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 06/05/17

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)*
Approximately 4-inch thick concrete slab.				0					
Fill: Grayish brown to dark brown, Sandy Lean Clay/Clayey Sand, moist, fine to coarse grained sand, moderate plasticity fines, fine sub-angular to sub-rounded gravel, fat clay lenses, some roots.	Stiff to Very Stiff/ Loose to Medium Dense	CL/ SC				20	17		
Concrete fragments and rubber debris				5		11	19		
No sample recovery between 6 to 7.5 feet. Possible oversized debris/fragments.						54			
● 73% Passing No. 200 Sieve.				10		9	25		
				15		16	18		
Native: Grayish brown and greenish gray, Sandy Lean Clay, moist, fine to medium grained sand, moderate plasticity, white mottling, noticeable hydrocarbon odor.	Stiff	CL		20		15	22		
Continued on Next Page									

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

DEPTH TO GROUND WATER: 23.5 feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 06/05/17

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)*
Grayish brown and greenish gray, Sandy Lean Clay, moist, fine to medium grained sand, moderate plasticity, noticeable hydrocarbon odor.	Stiff	CL		20					
Grayish brown, Clayey Sand, moist, fine to coarse grained sand, fine to coarse sub-angular to sub-rounded gravel, low plasticity fines, noticeable hydrocarbon odor. ▼ Ground water measured at 23.5 feet after drilling. ● 32% Passing No. 200 Sieve.	Medium Dense to Dense	SC		▼ 25	●	25	18		
Appears to transition to silty sand.				30		55	29		
Bottom of Boring at 30 feet.				35 40					

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

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-1
THREE-STORY MIXED-USE BUILDING
PALO ALTO, CALIFORNIA

BORING EB-1

AUGUST 2017
PROJECT NO. 4088-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: 23.5 feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 06/05/17

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)*
Native: Dark brown, Sandy Lean Clay/Fat Clay, moist, fine grained sand, moderate to high plasticity.	Stiff to Very Stiff	CL/ CH		0					
■ Liquid Limit = 50, Plasticity Index = 27.					■	12	23		2.3
				5		32	20		3.3
Color transitions to grayish brown.						32	25		2.3
				10		28	19		2.8
Light grayish brown, Sandy Lean Clay, moist, fine to medium grained sand, low to moderate plasticity, some clayey sand lenses.	Stiff to Very Stiff	CL							
				15		17	24		
White mottling.									
● 72% Passing No. 200 Sieve.				20	●	29	24		

Continued on Next Page

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

DEPTH TO GROUND WATER: 23.5 feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 06/05/17

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 grayish brown, Sandy Lean Clay, moist, fine to medium grained sand, low to moderate plasticity.	Very Stiff	CL		20					
Grayish brown and greenish gray, Silty Sand, moist, fine to coarse grained sand, fine sub-angular to sub-rounded gravel, low plasticity fines, noticeable hydrocarbon odor. ▼ Ground water measured at 23.5 feet after drilling.	Dense	SM		▼ 25		45	27		
Bottom of Boring at 25 feet.				30					
				35					
				40					

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

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-2
THREE-STORY MIXED-USE BUILDING
PALO ALTO, CALIFORNIA

BORING EB-2

AUGUST 2017
PROJECT NO. 4088-1

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APPENDIX B

LABORATORY TESTS

Samples from 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 D2216 on nearly all of the soil 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-2 at the appropriate sample depth.

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

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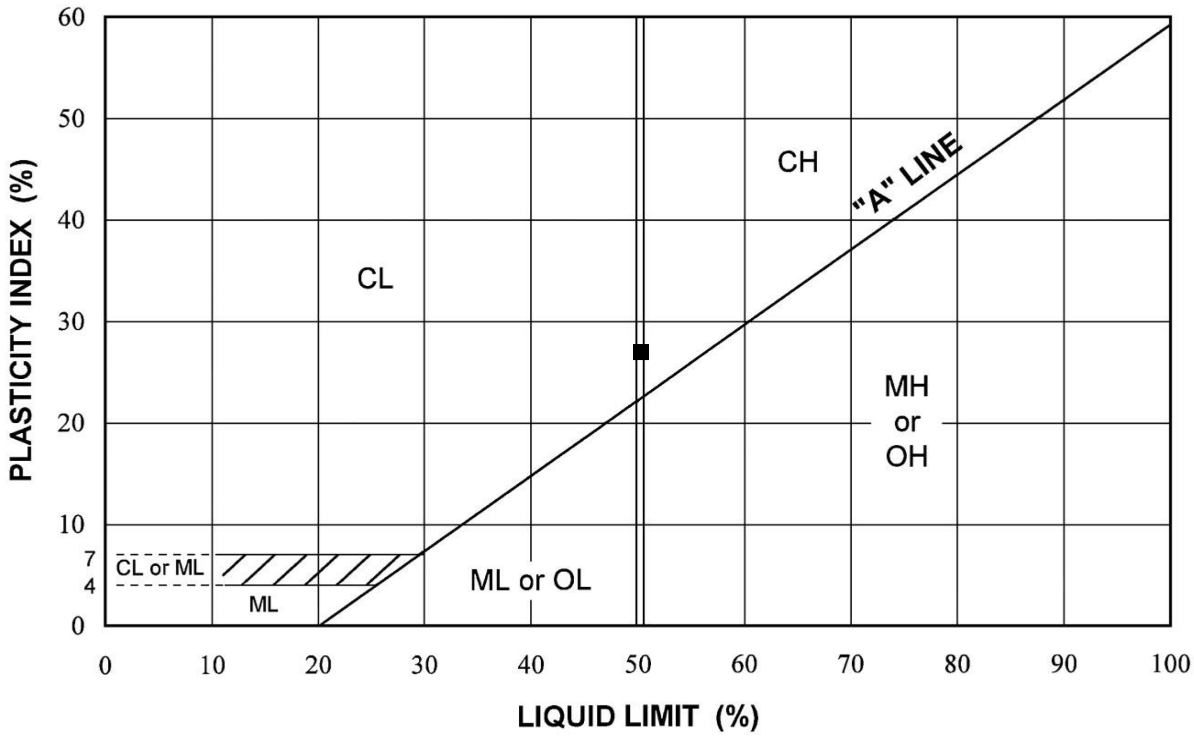

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-2	1-2.5	23	50	27	0		CL/CH

PLASTICITY CHART **FIGURE B-1**

THREE-STORY MIXED-USE BUILDING AUGUST 2017

PALO ALTO, CALIFORNIA PROJECT NO. 4088-1

ROMIG ENGINEERS

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