GEOTECHNICAL INVESTIGATION

HYATT PLACE HOTEL
1470 W SAN CARLOS STREET
SAN JOSE, CALIFORNIA 95126

Prepared for

Mr. Dylan Nguyen
c/o Rescom Development & Investments
2726 Aborn Street, Suite 207
San Jose, California 95121

August 2019

Project No. 4852-1

August 16, 2019
4852-1

Mr. Dylan Nguyen
c/o ResCom Development & Investments
2726 Aborn Street, Suite 207
San Jose, California 95121

RE: GEOTECHNICAL INVESTIGATION
HYATT PLACE HOTEL
1470 W SAN CARLOS STREET
SAN JOSE, CALIFORNIA

Dear Mr. Nguyen:

As requested, we have performed a geotechnical investigation for your proposed hotel development to be constructed at 1470 W. San Carlos Street in an unincorporated area of Santa Clara County near San Jose, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents geotechnical recommendations for the proposed hotel development.

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

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

Very truly yours,

ROMIG ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
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GEOTECHNICAL INVESTIGATION
FOR
HYATT PLACE HOTEL
1470 W SAN CARLOS STREET
SAN JOSE, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for your proposed hotel development to be constructed at 1470 W. San Carlos Street in an unincorporated area of Santa Clara County near San Jose, 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 design and construction of the proposed project.

Project Description

The project consists of constructing a six-story hotel building at the referenced site in San Jose. The hotel building is expected to have a footprint of about 63 by 215 feet (occupying the majority of the site), and will have one level of below-grade parking below the entire building. The site is currently occupied by a commercial building and auto repair shop, which will be demolished prior to construction.

Scope of Work

Our scope of work for this investigation was presented in our agreement with Mr. Dylan Nguyen, dated May 30, 2019. 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 hotel.
- Laboratory testing of selected soil samples to aid in soil classification and to help evaluate the engineering properties of the soils encountered at the site.

- Engineering analysis and evaluation of the surface and subsurface data to develop earthwork guidelines and foundation design criteria for the proposed building.

- Preparation of this report presenting our findings and geotechnical recommendations for the proposed hotel building.

Limitations

This report has been prepared for the exclusive use of Mr. Dylan Nguyen for specific application to developing geotechnical design criteria for the proposed hotel development to be constructed at 1470 W. San Carlos Street in an unincorporated area of Santa Clara County near San Jose, California. We make no warranty, expressed or implied, for the services we performed for this project. Our services are performed in accordance with the geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and recommendations only. In the event there are any changes in the nature, design, or location of the project, or if any future improvements are planned, the conclusions and recommendations presented in this report should not be considered valid unless: 1) the project changes are reviewed by us, and; 2) the conclusions and recommendations presented in this report are modified or verified in writing.

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

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on July 8, 2019. Subsurface exploration was performed using a Mobile B-53 truck-mounted drill equipped with 7-1/4-inch diameter hollow-stem augers. Two exploratory borings were advanced to depths of 30 and 45 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 commercial and residential area along the south side of W San Carlos Street and on the east side of Willard Avenue. At the time of our investigation, the site was occupied by a single-story auto repair shop and a small commercial building along the southern portion of the property. Asphalt concrete and concrete driveway and parking areas were located at the northern and western portions of the site.

The depth and width of the existing building foundations are unknown. The concrete pavement has numerous hairline to 1-inch wide cracks, and the asphalt concrete had alligator cracking in some area. Roof downspouts discharged adjacent to the perimeter foundations.

Subsurface Conditions

At the location of our exploratory borings, we generally encountered about 8 to 11 feet of very stiff to hard sandy lean clay of low to moderate plasticity underlain by about 7 to 8.5 feet of medium dense to dense clayey sand. Beneath the sandy layer, we primarily encountered stiff to very stiff sandy lean clay of low to high plasticity to the maximum depths explored of 45 feet in Boring EB-1, and 30 feet in Boring EB-2. We note that some dense clayey sand was also encountered near the termination depth of Boring EB-1.

A Liquid Limit of 38 and a Plasticity Index of 18 were measured on a sample of near-surface native soil obtained from our Boring EB-1. These test results indicate that the near surface soil at the site generally has low to moderate plasticity and a relatively low potential for expansion.

Ground Water

Free ground water was encountered at a depth of about 30 feet in Boring EB-1, but was not encountered in Boring EB-2 during our field exploration. The borings were backfilled with grout shortly after drilling; therefore a stabilized ground water level may not have been obtained. Ground water information presented in Seismic Hazard Zones Map of the San Jose West Quadrangle prepared by the California Division of Mines and Geology in 2002 indicates that the depth to the historic high ground water in the area of the site is about 30 to 35 feet below ground surface. In addition, based on information presented on the Geotracker website, ground water was measured at depths ranging from about 28 to 46 feet between 1993 and 2008 in the monitoring wells installed at 1295 and 1744 West San Carlos Street (about 1,500 feet east and 2,000 feet west of the project site, respectively). Based on the information above, it is our opinion that the highest projected ground water level to be considered at the site is about 23 feet below the existing ground surface.

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

GEOLOGIC SETTING

We have briefly reviewed our local experience and geologic literature pertinent to the general site area. The information reviewed indicates that the site is located in an area underlain by older alluvial fan deposits (Qhf2) (Wentworth et al., 1999). The deposits generally consist of gravelly sand and sandy/clayey gravel, grading upward to sandy and silty clay. Upstream and near the fan heads the deposits tend to be moderately dense to dense and coarser. The alluvial deposits were deposited by flooding streams where they emerged from constrained channels of the uplands and merge downslope into flood plain and basin deposits. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

The lot and the immediate site vicinity are located in an area that slopes very gently toward the north and northeast. The property is located at an elevation of approximately 110 feet above sea level.

Faulting and Seismicity

There are no mapped through-going faults within or adjacent to the site and the site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the Hayward fault, located approximately 7.2 miles northeast 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 because of the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906 and 1989. The faults considered most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras faults. The San Andreas and San Gregorio faults are located approximately 10 and 25 miles southwest of the site, respectively. The Calaveras fault is located approximately 10 miles northeast of the site. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 1, and are shown on the Regional Fault and Seismicity Map, Figure 4.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Hyatt Place Hotel
San Jose, California

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

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

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the 2016 California Building Code and in ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures." Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class 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 (37.3231), longitude (-121.9164) and Site Class D, design parameters are presented on Table 2, below.

Table 2. 2016 CBC Seismic Design Criteria
Hyatt Place Hotel
San Jose, California

Spectral Response Acceleration Parameters		Design Value
Mapped Value for Short Period -	S_S	1.500
Mapped Value for 1-sec Period -	S_1	0.600
Site Coefficient -	F_a	1.0
Site Coefficient -	F_v	1.5
Adjusted for Site Class -	S_{MS}	1.500
Adjusted for Site Class -	S_{M1}	0.900
Value for Design Earthquake -	S_{DS}	1.000
Value for Design Earthquake -	S_{D1}	0.600

Geologic Hazards

As part of our investigation, we reviewed the potential for geologic hazards to impact the site and the proposed hotel development, 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 considered low.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Strong ground shaking should therefore be expected several times during the life of the building, as is typical for sites throughout the Bay Area. The building should be designed 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, silty sand, uniformly graded sands and sandy silt. Since saturated loose sands and other soils prone to liquefaction were not encountered during our exploration, and since ground water level is expected to be relatively deep, in our opinion, the likelihood of significant liquefaction occurring at the site is low. In addition, the site is not located in a State of California liquefaction hazard zone.

- Dynamic Densification - Dynamic densification occurs during moderate and large earthquakes when unsaturated soft/loose, natural or fill soils densify and settle, often unevenly across a site. To evaluate the potential for earthquake-induced dynamic densification of the medium dense sands encountered at the site, we performed a settlement analysis following the methods presented in the US Army Corps of Engineers EM1110-1-1904.

  Based on our analysis, the medium dense sand encountered in our Boring EB-1 between depths of about 8 to 12 feet is potentially prone to dynamic densification when subjected to a peak ground acceleration (PGA) of 0.50g, the PGA_M for maximum considered earthquake based on ASCE 7-10.

  Based on the results of our analysis, we estimate that total dynamic settlement of up to about 1/4-inch may occur at the ground surface due to dynamic densification at the medium dense sand encountered in Boring EB-1 during a design level earthquake. Since the proposed building is expected to have a basement level extending below the medium dense sand encountered in Boring EB-1, in our opinion, dynamic densification settlement is not expected to significantly affect the proposed building.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed hotel building with a full basement 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.

Based on the depth of the proposed basement, the basement foundation is expected to bear primarily on very stiff to hard clays and dense sands. In our opinion, the building may be supported on a basement mat foundation bearing on dense sands or stiff clayey soils. Any miscellaneous site improvements planned at-grade may be supported on conventional spread footing foundations.

Prior to mat construction, the mat subgrade should be prepared and compacted as recommended in the "Earthwork" section of this report. At this time, building loads are not available. During design, our office should be retained to finalize the preliminary foundation design and building settlement criteria presented in this report.

We note that the sandy strata encountered in the borings within the basement excavation depth were judged to have limited cohesion and may be prone to sloughing and/or caving if excavated near-vertical. Temporary basement excavation shoring should be designed and installed accordingly. This information should be considered by the contractor when establishing temporary shoring/sloping criteria for basement excavation, as needed.

Because subsurface conditions may vary from those encountered at the location 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 the recommendations presented in this report and; 2) observe and test during earthwork, foundation, shoring, drainage and slab construction.

FOUNDATIONS

Mat Foundation

In our opinion, the proposed building and basement walls may be supported on a reinforced concrete mat foundation bearing in undisturbed native soil. The mat may be designed for an average allowable bearing pressure of up to 3,000 pounds per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading. A maximum localized bearing pressure of 3,500 pounds per square foot from dead plus live loads may be used at concentrated column or wall loads.

The mat should be reinforced to provide structural continuity and to permit spanning of local irregularities. On a preliminary basis, a modulus of subgrade reaction (Kv1) of 100 pounds per cubic inch may be assumed for the mat subgrade. This value is based on a 1-foot square bearing area and should be scaled to account for mat foundation size effects. Alternatively, once building loads and estimated post construction differential settlement are available, a modulus of subgrade reaction (Kv) may be estimated for the mat subgrade (typically on the order of 10 to 25 pci). The mat should also be designed with sufficient thickness and reinforcing to span over localized weak compressible areas.

The bottom of the excavation for the basement mat should be cleaned of all loose to medium dense or relatively soft soil and debris. A member of our staff should observe the excavation and evaluate whether scarification and compaction or proof rolling of the bottom of the excavation is needed. A 6-inch section of crushed rock or a thin working slab could be placed as a working surface on the prepared and approved mat subgrade.

Basement Water Proofing

We have not provided recommendations regarding the method or details for basement damp-proofing since design of damp-proofing systems is outside of our scope of services and expertise. Installing adequate damp-proofing below and behind the edges of the basement floor and behind the basement walls is essential for the success of the basement structure. Placing concrete with a low water cement ratio should be considered as one step of good damp-proofing as discussed below. The damp-proofing system below the basement mat may be placed directly on a layer of ¾-inch crushed rock or a thin working slab (as discussed previously), or alternative methods as determined by the water-proofing consultant and/or contractor.

Shallow Foundations

In our opinion, any miscellaneous improvements planned at-grade may be supported on conventional spread footings bearing in undisturbed stiff native soil. The footings should have a width of at least 15 inches and should extend at least 28 inches below exterior grade, and 24 inches below the bottom of concrete slabs-on-grades, whichever is deeper.

Footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 2,500 pounds per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading.

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

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

Lateral Loads for Basement Mat

Lateral loads may be resisted by friction between the bottom of the mat/footings and the supporting subgrade, and by passive soil pressure acting against the mat or basement walls cast neat in foundation excavations or backfilled with properly compacted structural fill. The below values given for coefficient of friction and passive soil resistance are ultimate values. We recommend that a factor of safety of 1.5 be applied in the design.

An ultimate coefficient of friction of 0.45 may be assumed for at-grade footings bearing directly on native soil. However, since it is likely that a water-proofing membrane will be installed between the bottom of the mat and subgrade soil, the structural engineer should consult with the water-proofing consultant for the coefficient of friction between the membrane and subgrade soil.

Ultimate passive soil resistance may be simulated by an equivalent fluid pressure of 450 pounds per cubic foot beginning at the ground surface or basement subgrade, where appropriate. The ultimate passive soil resistance acting on the mat foundation should be limited to 2,500 pounds per square foot. This passive pressure assumes lateral deflection at the top of the mat foundation on the order of ¼- to ½-inch.

Settlement

On a preliminary basis, 30-year post-construction total settlement due to static loads is not expected approximately 1-inch across the building mat foundation. Post-construction differential settlement of about ¾-inch between interior columns and perimeter basement walls is also estimated across the 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 and/or differential settlement.

SLABS-ON-GRADE

General Slab Considerations

To reduce the potential for movement of the slab subgrade, at least the upper 6 inches of subgrade soil should be scarified and compacted at a moisture content above the laboratory optimum. The soil subgrade should be kept moist up until the time the non-expansive fill, aggregate base, and/or vapor barrier 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 and interior slabs-on-grade should be underlain by a layer of non-expansive fill as recommended below. The non-expansive fill should consist of Class 2 aggregate base or clayey soil with a Plasticity Index of 15 or less.

Considering the potential for some differential movement of the surface and near-surface soils, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should be given to using a control joint spacing on the order of 2 feet in each direction for each inch of slab thickness.

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 6 inches of Class 2 aggregate base. To improve performance, exterior slabs-on-grade, such as for patios and wide walkways, 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.

Basement Mat

In our opinion, the basement mat and parking ramp (prior to installation of the water proofing) may be placed directly on a layer of ¾-inch crushed rock or a thin working slab, or alternative methods as determined by the water-proofing consultant and/or contractor. A member of our staff should observe the excavation and evaluate whether or not scarification and compaction or proof rolling of the bottom of the excavation below the basement mat and ramp is needed.

As discussed previously, installing adequate damp-proofing below and behind the edges of the basement floor and behind the basement walls is essential for the success of the basement structure.

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 (or basement retaining walls) and higher strength. Where moisture protection is important and/or where the concrete will be placed directly on the damp-proofing, 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.

Subsurface Drainage

Although a high ground water table is not expected, a subslab drain system could be installed below the basement mat to reduce the possibility of water pressure developing below the basement floor and floor damp-proofing system. If installed, perforated pipes for the basement drainage system should be installed at the bottom of the basement excavation. The basement drainage system should include a minimum 6-inch-thick blanket of free-draining gravel, such as 1/2- or 3/4-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, below the basement mat.

Prior to placing the gravel blanket, the subgrade below the gravel layer should be surface compacted and covered with a filter fabric, such as TC Mirafi 140N. The gravel drain should extend up and around the sides of the mat and basement walls.

Drain pipes around the basement walls should consist of 4-inch diameter perforated PVC pipes with perforations placed down installed at bottom of the wall excavation. The perforated pipes should discharge to a suitable sump and pump system. To minimize vapor transmission through the basement mat, a high-quality water-proof membrane should be placed over the crushed rock and around the edges of the mat foundation. A schematic sketch of the basement drainage system is presented in Figure 5.

BASEMENT WALLS

We recommend that retaining walls with level backfill that are not free to deflect or rotate, such as the basement walls, be designed to resist an at-rest equivalent fluid pressure of 45 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot, where H is the height of the wall in feet. If the basement walls will be designed as undrained and subject to ground water pressure, they should be designed to resist an equivalent fluid pressure of 80 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot. Where retaining walls will be subjected to surcharge loads, such as from foundations, construction loading, or traffic on adjacent streets, the walls should also be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Based on the site peak ground acceleration (PGA), on Seed and Whitman (1970); Al Atik and Sitar (2010); and Lew et al. (2010); seismic loads on basement retaining walls that cannot yield may be simulated by a line load of $11H^2$ (in pounds per foot, where H is the wall height in feet). This seismic surcharge line load should be assumed to act at 1/3H above the base of the wall (in addition to the active wall design pressure of 40 or 80 pounds per cubic foot for drained and undrained condition, respectively). The additional 8H pounds per square foot discussed above need not to be applied for seismic condition.

As noted above, a reliable water-proofing system should be installed below and around the edges of the foundation and slab floor as well as behind the basement walls.

If the basement is designed for drained conditions, in order to prevent buildup of water pressure from surface water infiltration, a subsurface drainage system should be installed behind the walls (and the perched ground water condition recommended above may be eliminated). The drainage system should consist of a 4-inch diameter perforated pipe (perforations placed down) embedded in a section of 1/2- to 3/4-inch, clean, crushed rock at least 12 inches wide. Backfill above the perforated drain line should also consist of 1/2- to 3/4-inch, clean, crushed rock to within about 1½ to 2 feet below exterior finished grade. A filter fabric should be wrapped around the crushed rock to protect it from infiltration of native soil. The upper 1½ to 2 feet of backfill should consist of compacted native soil. The perforated pipe should discharge into a sump that pumps to a suitable location. Damp-proofing of the walls should be included in areas where wall dampness and efflorescence would be undesirable.

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

Backfill (if any) behind the retaining walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used for compaction of wall backfill, the walls may need to be temporarily braced.

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

TEMPORARY BASEMENT EXCAVATION SHORING

Since the basement excavation is expected to extend to a depth of at least 12 to 15 feet and the excavation will be located in a close proximity of the adjacent properties and streets, temporary shoring should be provided during basement excavation. The walls of the basement excavation may be supported by several methods including tiebacks, soldier beams and wood lagging, soil nails, braced shoring or potentially other methods. The choice should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate.

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Support of any adjacent existing structures and improvements without distress should also be the contractor's responsibility. We recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review. In addition, it should be the contractor's responsibility to undertake a preconstruction survey with benchmarks and photographs of the adjacent properties.

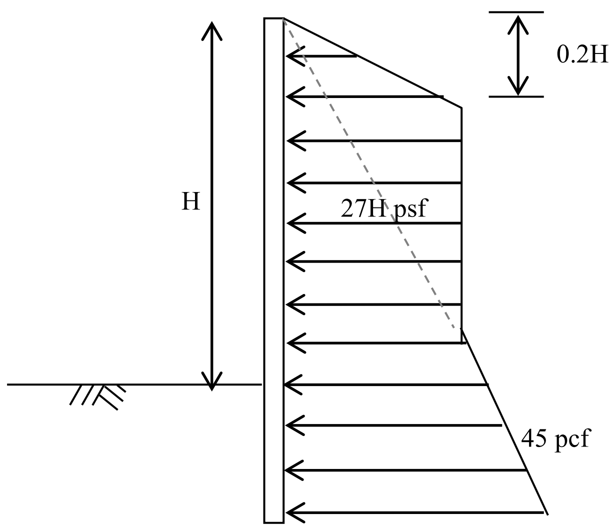
On a preliminary basis, the following geotechnical design parameters are provided for the basement shoring and support. The shoring engineer and contractor who are responsible for performance of the shoring system may recommend alternative values based on their experience and the allowable deflection needed for the site and adjacent structures.

On a preliminary basis, in our opinion, if soldier beam and wood lagging will be used for temporary shoring, piers may be designed to support an active lateral soil pressure of at least 45 pounds per cubic foot across the entire vertical excavation cut. Where deflection of the shoring wall is not desired, such as sections near neighboring buildings, the shoring wall should be designed to resist an at-rest equivalent fluid pressure of 45 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot, where H is the height of the wall in feet.

A skin friction of 350 pounds per square foot may be assumed for the stitch piers when calculating the allowable vertical capacity of the piers. Passive soil resistance of 450 pounds per cubic foot may be assumed to act on the stitch piers over 2 pier diameters when calculating the minimum depth of the piers required to resist lateral loads; at least the upper foot of passive resistance should be neglected in design.

If a tie-back system will be used, on a preliminary basis, the temporary shoring may be designed to support a lateral soil pressure based on the following trapezoidal distribution. Below the trapezoidal distribution, the active pressure discussed above (i.e., 45 pounds per cubic foot, starting at the top of the shoring wall) should extend to the bottom of the wall foundation. As discussed above, where deflection of the shoring wall is not desired, such as sections near neighboring buildings, the shoring wall should be designed to resist an additional uniform lateral pressure of 8H pounds per square foot, where H is the height of the wall in feet.

Figure A. Earth Pressure Design for Excavation Bracing



Where shoring walls will be subjected to surcharge loads, such as from adjacent foundations, sloping backfill, vehicle/traffic loads, or construction, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Some vertical and lateral deflection should be expected to occur in the temporary shoring system, which could result in ground settlement adjacent to the shoring. The amount of vertical and lateral deflection at the shoring face (for active condition) is typically on the order of 1 to 2 percent of the total excavation depth (H) (reducing to ground settlement on the order of about ½ to 1 percent of H within a lateral distance of H equal to the total excavation depth). If this amount of deflection and settlement is not tolerable, the shoring system should be designed for a higher active or at-rest pressure discussed above in order to limit the potential deflections.

Larger deflections than estimated above are possible depending upon how the shoring is constructed and/or backfilled. The contractor should monitor vertical and lateral deflections as the basement excavation, shoring installation and building construction proceeds and modify the design as needed to control deflections to acceptable amounts. In addition, it should be the contractor's responsibility to undertake a preconstruction survey with benchmarks and photographs of the adjacent properties.

Concrete should be placed in the pier excavations as soon as practical after drilling. Ground water seepage likely will be encountered during pier drilling and it is possible that ground water seepage 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 the tremie method. If caving conditions occur, scheduling several concrete placements each day of drilling or use of a drill casing may be required.

Tie Backs

Tie backs may be installed to laterally support the shoring system as needed. The tie backs may be designed with allowable bond strength between the native soil and the anchors of 1,200 pounds per square foot. This bond strength (with a factor of safety of at least 1.5) should be confirmed in the field during the initial stages of construction with proof load testing as required by the shoring designer. The actual bond strength and pull-out capacity of the tie back is dependent upon the installation method and should be confirmed in the field during construction with performance and proof load testing; our representative should observe the testing to verify that the needed capacities are obtained.

The design bond length will depend on the anchor spacing and desired capacity; however we suggest a minimum bond length of 10 feet beyond the active soil wedge behind the shoring walls would generally be appropriate. We suggest that the minimum unbonded length within the active zone of the tie-backs may be assumed to be the length in front of a 60 degree slope (from horizontal) projected up from the base of the retaining wall.

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as existing foundations, pavements, flatwork, 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.

A member of our staff should observe the basement excavation to evaluate whether scarification and compaction or proof rolling of the excavation bottom is needed. If a temporary ramp is constructed to access portions of the basement excavation, the ramp should be properly backfilled with compacted on-site soil as recommended in this report for structural fill. A member of our staff should observe and test during backfilling of the temporary entrance ramp and basement walls.

Material For Fill

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

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
Hyatt Place Hotel
San Jose, California

General	Relative Compaction*	Moisture Content*
• Scarified subgrade in areas to receive structural fill.	90 percent	Above optimum
• Structural fill composed of native soil.	90 percent	Above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
• Structural fill below a depth of 5 feet.	93 percent	Above optimum
Pavement Areas		
• Upper 6-inches of soil below aggregate base.	95 percent	Near optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site 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. Preferably, downspout drainage should be collected in a closed pipe system that is routed to a storm drain system or other suitable discharge outlet.

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

FUTURE SERVICES

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. In addition, it should be noted that many of the local building and planning departments now require "clean" geotechnical plan review letters prior to acceptance of plans for their final review. Since our plan reviews typically result in recommendations for modification of the plans, our generation of a "clean" review letter often requires two iterations. At a minimum, we recommend the following note be added to the plans.

"Earthwork, foundation construction, shoring installation, mat and/or slab subgrade preparation, utility trench backfill, basement wall drainage and backfill, subslab drainage installation, pavement construction, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated August 16, 2019. 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) 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 presented 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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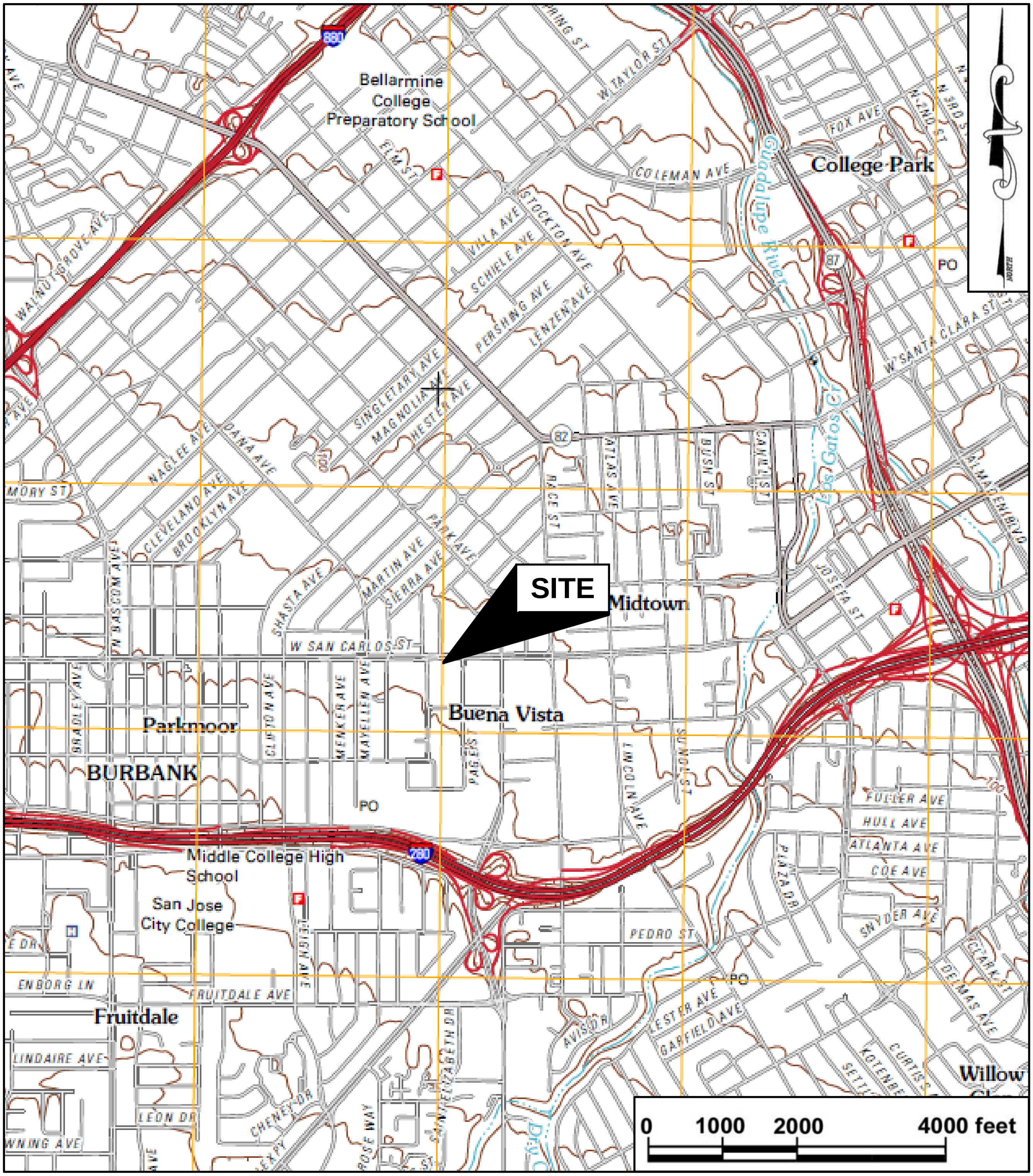
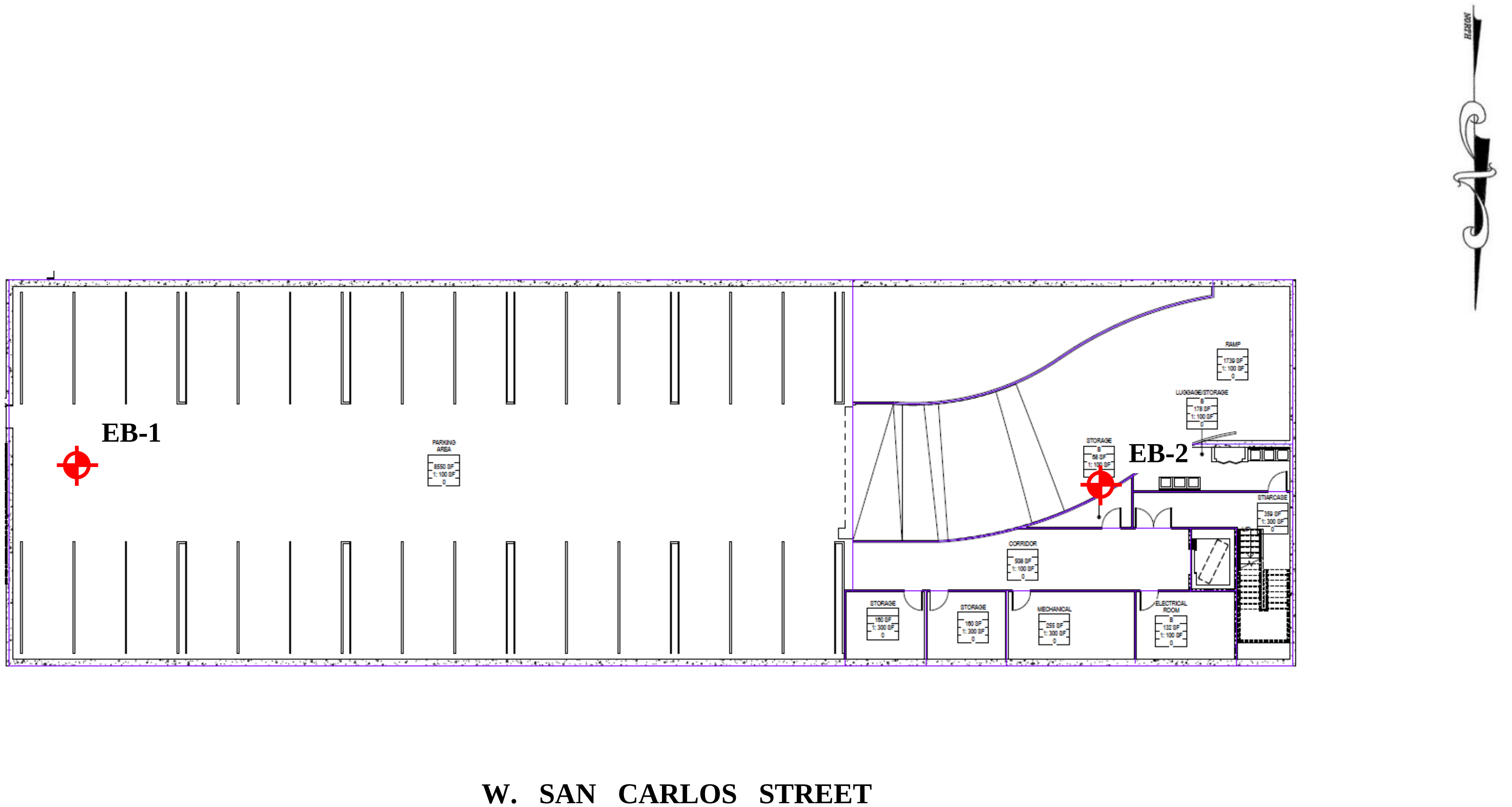


Scale: 1 inch = 2000 feet

Base is United States Geological Survey San Jose West 7.5 Minute Quadrangle, dated 2012.

VICINITY MAP
RESCOM HYATT PLACE HOTEL
SAN JOSE, CALIFORNIA

FIGURE 1
AUGUST 2019
PROJECT NO. 4852-1



LEGEND

EB-2 Approximate Locations of Exploratory Borings.

Approximate Scale: 1 inch = 30 feet.

Base is site plan prepared by Shad Design Group, dated December 3, 2018.

SITE PLAN
RESCOM HYATT PLACE HOTEL
SAN JOSE, CALIFORNIA

FIGURE 2
AUGUST 2019
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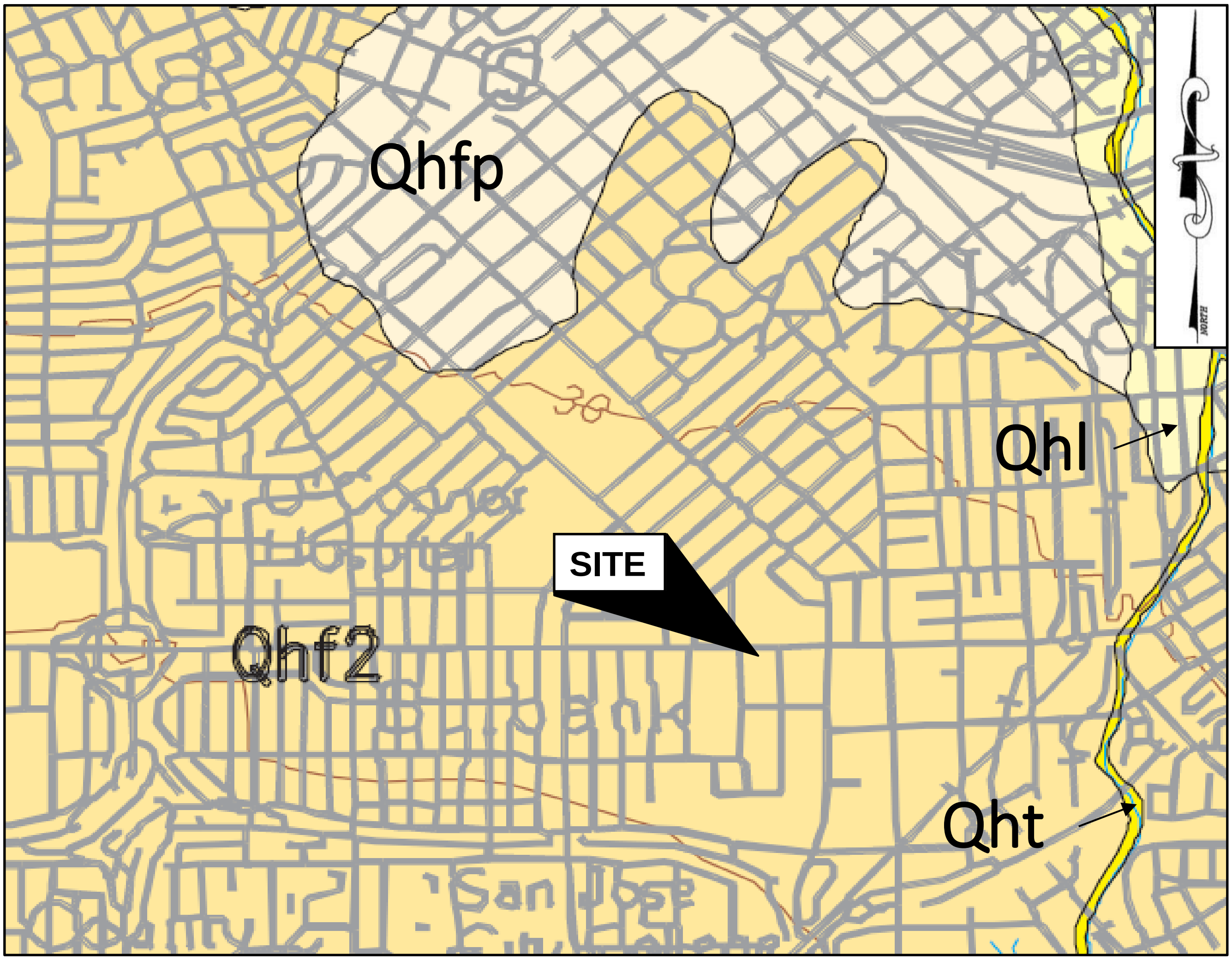

LEGEND

Qhfp	Flood plain deposits
Qhl	Levee deposits
Qht	Stream terrace deposits
Qhf2	Alluvial fan deposits (older)

Geologic Contact - dashed where approximate, dotted where inferred.

Fault - dashed where approximate, dotted where inferred.

0 1000 2000 4000 feet

Scale: 1 inch = 2000 feet
Base is Geologic Map of San Jose 30 x 60 Minute Quadrangle (Wentworth et al., 1999).

VICINITY GEOLOGIC MAP
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SAN JOSE, CALIFORNIA

FIGURE 3
AUGUST 2019
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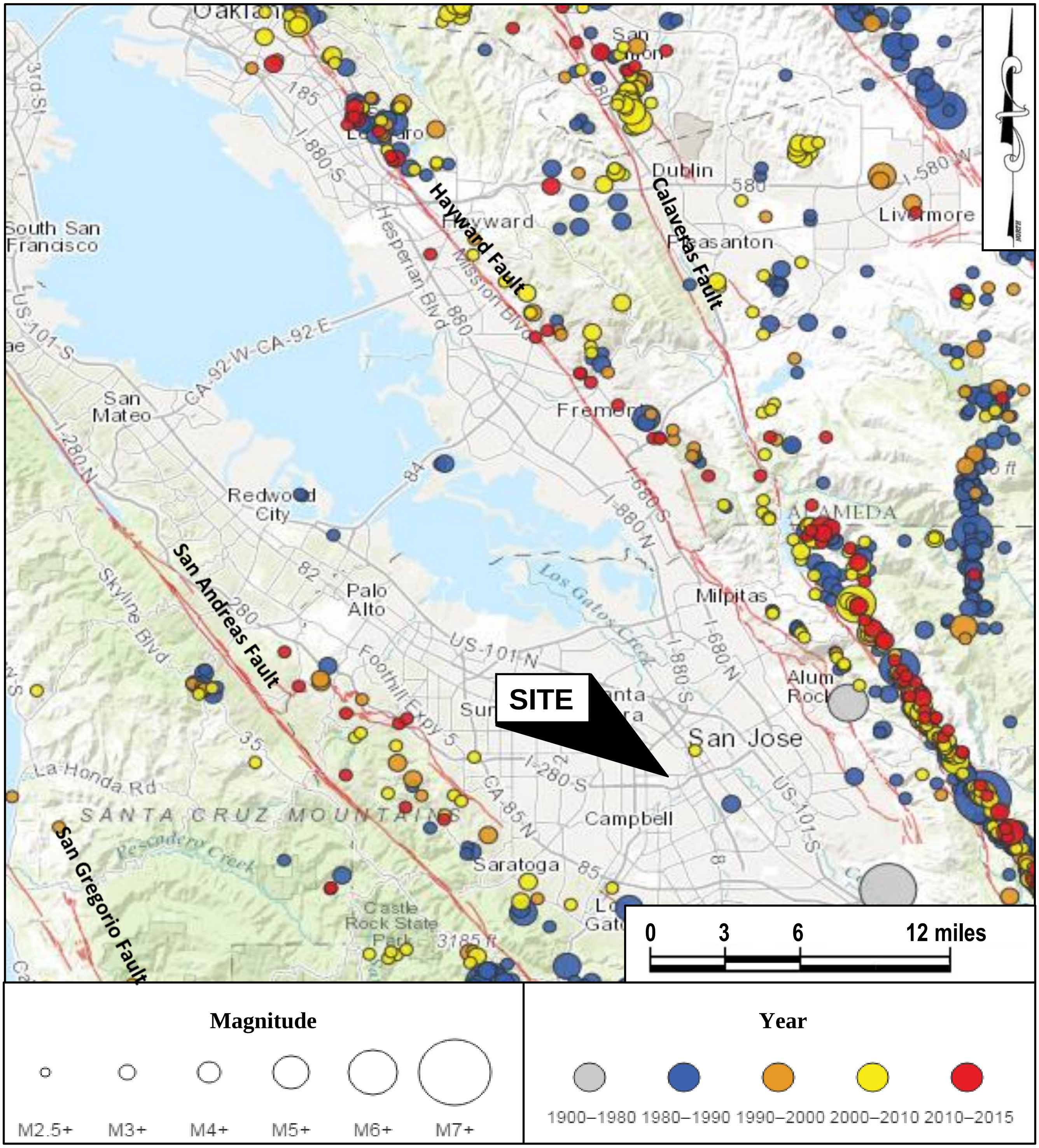

Earthquakes with M5+ from 1900 to 1980, M2.5+ from 1980 to January 2015. Faults with activity in last 15,000 years. Based on data sources from Northern California Earthquake Data Center and USGS Quaternary Fault and Fold Database, accessed May 2015.

REGIONAL FAULT AND SEISMICITY MAP
RESCOM HYATT PLACE HOTEL
SAN JOSE, CALIFORNIA

FIGURE 4
AUGUST 2019
PROJECT NO. 4852-1

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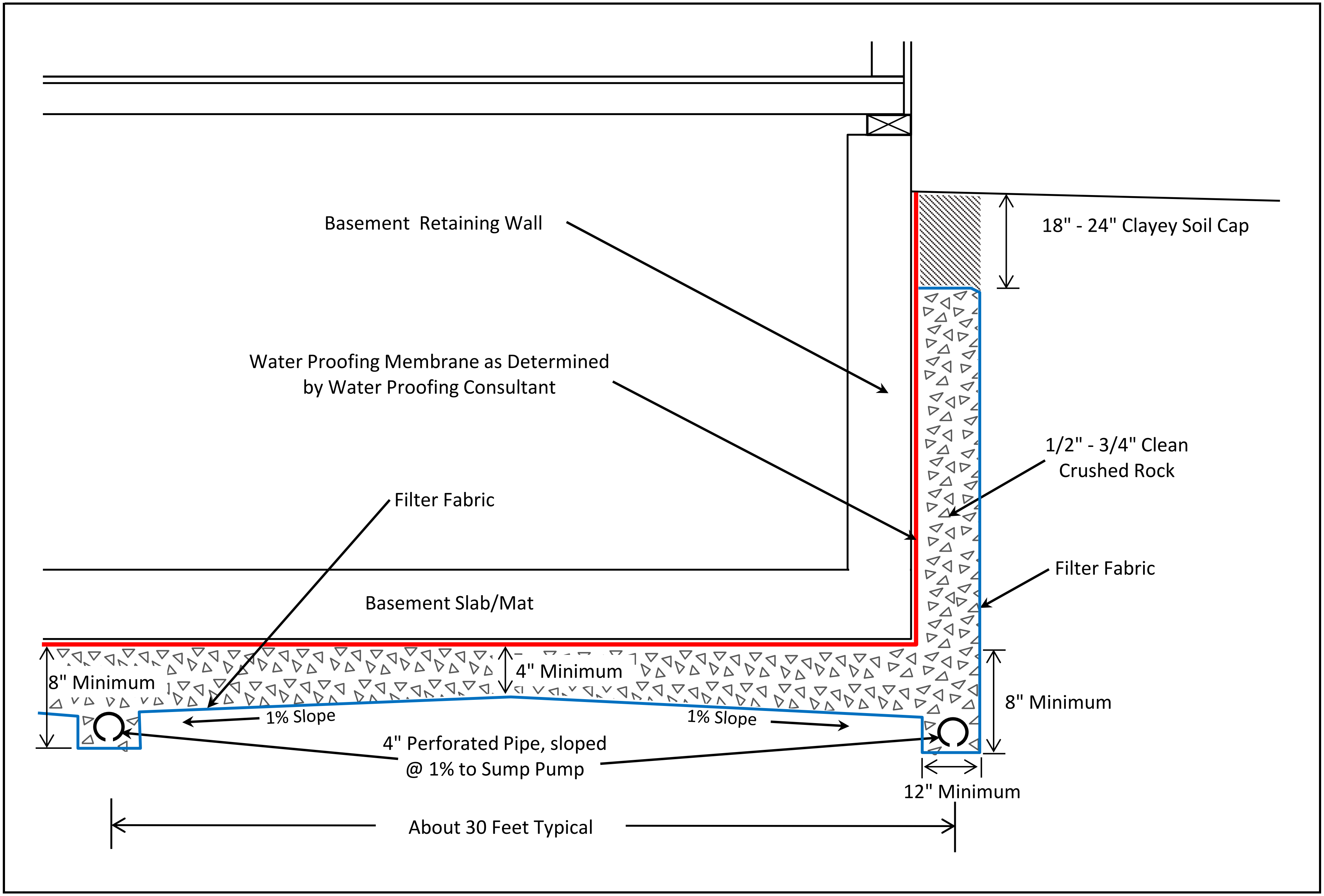

SUBSLAB DRAINAGE DETAIL
RESCOM HYATT PLACE HOTEL
SAN JOSE, CALIFORNIA

FIGURE 5
AUGUST 2019
PROJECT NO. 4852-1

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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 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-inch and 3.0-inch O.D. drive samplers. The blow counts shown on the logs for these larger diameter samplers do not represent SPT values and have not been corrected in any way.

The location of the borings were established by pacing using the site plan provided to us 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
RESCOM HYATT PLACE HOTEL
SAN JOSE, CALIFORNIA

FIGURE A-1
AUGUST 2019
PROJECT NO. 4852-1

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

DEPTH TO GROUND WATER: 30 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 7/8/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS✱ (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
4" Concrete over 2" aggregate base.				0					
Dark brown, Lean Clay, moist, fine to coarse grained sand, low to moderate plasticity.	Very Stiff	CL							
■ Liquid Limit = 38, Plasticity Index = 18.					■	22	19		3.5
✻ Dry Density = 104 pcf.					✻	26	13		4.0
				5					
							12		
						34	10		4.0
Light brown, Clayey Sand, moist, fine to coarse grained sand, coarse sub-angular gravel, low plasticity fines.	Medium Dense to Dense	SC							
● 47% Passing No. 200 Sieve.				10	●	23	13		
							4		
● 16% Passing No. 200 Sieve.				15	●	43	6		
Light brown, Lean Clay, moist, fine to coarse grained sand, low plasticity.	Stiff to Very Stiff	CL							
				20		27	26		1.5

Continued on Next Page

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

DEPTH TO GROUND WATER: 30 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 7/8/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Lean Clay, very moist, trace sand, moderate plasticity.	Stiff to Very Stiff	CL		20					
				25		28	34		1.8
▼ Ground water measured at 30 feet after drilling.				▼ 30		21	30		1.5
Increase in sand content.									
				35		33	29		
Decrease sand content.									
Moderate to high plasticity.									
■ Liquid Limit = 46, Plasticity Index = 26.				40	■	24	24		

Continued on Next Page

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

DEPTH TO GROUND WATER: 30 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 7/8/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Lean Clay, moist, trace sand, moderate to high plasticity.	Hard	CL		40					
Brown, Clayey Sand, moist, fine to medium grained sand, low plasticity fines.	Dense	SC		45		45	24		
Bottom of Boring at 45 feet.				50					
				55					
				60					

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

*Measured using Torvane and Pocket Penetrometer devices.

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 7/8/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS✱ (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
4" AC over 34" AB				0					
Dark brown, Sandy Lean Clay, moist, fine to coarse grained sand, low plasticity.	Very Stiff to Hard	CL							
						25	18		
						26	11		4.5
				5					
● 59% Passing No. 200 Sieve.					●	47	10		>4.5
● 53% Passing No. 200 Sieve.				10	●	30	9		
Brown, Clayey Sand, moist, fine to coarse grained sand, fine to coarse sub-angular to sub-rounded gravel, low plasticity fines.	Dense	SC							
✻ Dry Density = 109 pcf.				15	✻	42	3		
Brown, Sandy Lean Clay, moist, fine to coarse grained sand, low plasticity.	Very Stiff	CL					6		
● 63% Passing No. 200 Sieve.				20	●	28	16		
Continued on Next Page									

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 7/8/19

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS✱ (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Lean Clay, moist, fine to medium grained sand, moderate plasticity.	Very Stiff	CL		20					
				25		23	36		2.0
				30		29	26		2.2
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.

APPENDIX B

LABORATORY TESTS

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

The natural moisture content was determined in accordance with ASTM 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 two samples 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 five samples of soil in accordance with ASTM D422. The results are presented on the boring logs at the appropriate sample depths.

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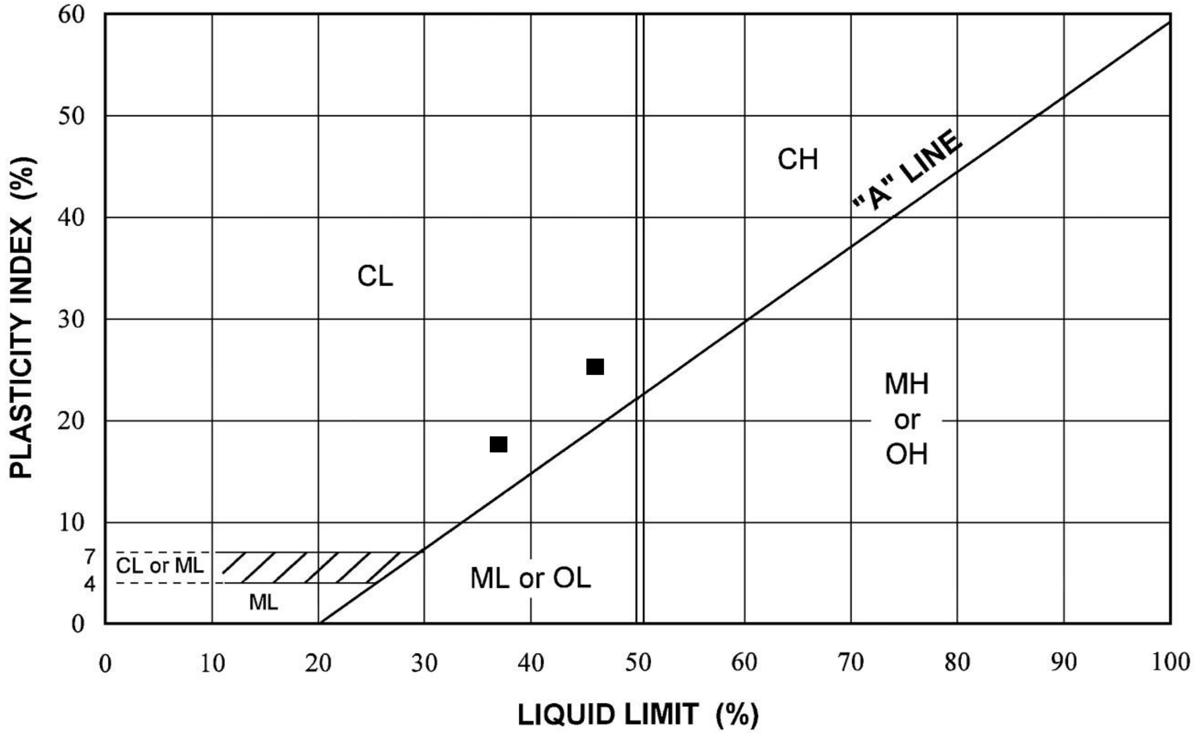

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	1-2.5	13	38	18			CL
■	EB-1	18.5-20	26	46	26			CL

PLASTICITY CHART
RESCOM HYATT PLACE HOTEL
SAN JOSE, CALIFORNIA

FIGURE B-1
AUGUST 2019
PROJECT NO. 4852-1

ROMIG ENGINEERS

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