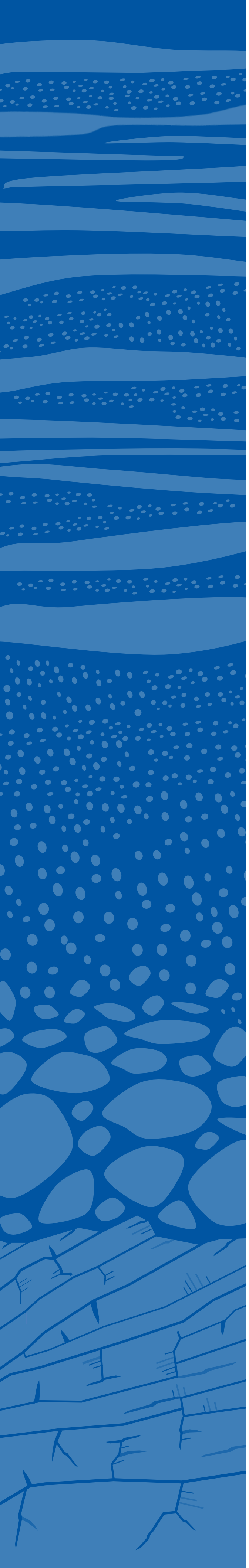
Appendix B

Geotechnical Investigation



ROMIG
ENGINEERS

GEOTECHNICAL INVESTIGATION

HAMPTON INN
1090 S. DE ANZA BOULEVARD
SAN JOSE, CALIFORNIA 95129

Prepared for
CUPERTINO DE ANZA HOSPITALITY LLC
P.O. BOX 446
Cupertino, California 95015

April 2017

Project No. 4017-1

ROMIG
ENGINEERS

April 18, 2017
4017-1

Cupertino De Anza Hospitality, LLC
P. O. Box 466
Cupertino, California 95015

RE: GEOTECHNICAL INVESTIGATION
HAMPTON INN
1090 S. DE ANZA BOULEVARD
SAN JOSE, CALIFORNIA

Attention: Mr. Jerry Kwok

Gentlemen:

As requested, we have performed a geotechnical investigation for the proposed Hampton Inn to be constructed at 1090 S. De Anza Boulevard in 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 project.

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

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

Very truly yours,

ROMIG ENGINEERS, INC.

Tom W. Porter

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GEOTECHNICAL INVESTIGATION
HAMPTON INN

1090 S. DE ANZA BOULEVARD
SAN JOSE, CALIFORNIA 95129

PREPARED FOR:
CUPERTINO DE ANZA HOSPITALITY, LLC
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APRIL 2017

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GEOTECHNICAL INVESTIGATION
FOR
HAMPTON INN
1090 S. DE ANZA BOULEVARD
SAN JOSE, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed 90 room Hampton Inn to be constructed at 1090 S. De Anza Boulevard, 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 51,279 square foot four-story hotel building at the referenced site in San Jose. The building is expected to have one level of basement parking below the entire footprint that extends about 12 feet below grade. The basement will be accessed through a ramp at the north side of the building. The ground level of the hotel will include the lobby, business center, fitness room, guest rooms, and porte-cochere. A swimming pool is planned at the central courtyard of the hotel building and over the basement. A portion of the north and east perimeter driveway may need to consist of a pier supported deck in order to protect tree roots of the adjacent large trees. The property is currently occupied by an active Chevron Service Station and attached car wash. The Chevron service station and car wash which currently occupies the site will be demolished. Structural loads are expected to be moderate as is typical for this type of construction.

Scope of Work

Our scope of work for this investigation was presented in our agreement with Cupertino De Anza Hospitality LLC dated February 9, 2017. In order to complete our investigation, we performed the following work.

- Review of geologic and geotechnical literature in our files pertinent to the general area of the site including the April 14, 2014, Geotechnical Engineering Report prepared by Earth Systems Pacific.

- Subsurface exploration consisting of drilling, sampling, and logging three exploratory borings in the area of the proposed building.

- 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 construction.

Limitations

This report has been prepared for the exclusive use of Cupertino De Anza Hospitality, LLC for specific application to developing geotechnical design criteria for the proposed Hampton Inn to be constructed at 1090 S. De Anza Boulevard in San Jose, California. We make no warranty, expressed or implied, except that 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 planned improvements; review of previous 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 ENVIRONMENTAL DOCUMENTATION

The property is currently an active Chevron Service Station and attached drive through car wash. We briefly reviewed the Phase 1 Environmental Site Assessment Report, dated February 27, 2015, prepared by Alpha Environmental. The report indicates that 15,000 and 20,000 gallon underground storage tanks are currently located below the southwest corner of the site and we expect that associated piping, dispensers, and other underground station improvements exist at the site. In 1999, three previous underground tanks and associated piping were removed from the site. The depth of the previous and existing underground tanks are unknown. No underground tank removal report or other relevant documentation was available on the state Geotracker website.

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on March 16, 2017. Subsurface exploration was performed using a Mobile B-61 truck-mounted drill equipped with 7.25-inch diameter hollow-stem augers. Three exploratory borings were advanced to depths ranging between 10 to 40 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.

Previous Geotechnical Investigation

A geotechnical investigation was performed at the site by Earth Systems Pacific for design and construction of a previously planned convenience store addition; the results were presented in a report dated April 14, 2014. This previous investigation included two exploratory borings that were both advanced to a depth of 15 feet. Based on our interpretation, the previous borings generally encountered about 9 to 14 feet of very stiff to hard lean clay underlain by dense to very dense clayey sand that extended to the maximum depth of the borings. Ground water was not encountered in the borings during the investigation. The report recommended that the convenience store building be supported on a conventional spread footing foundation. The locations of the borings from the previous investigation are shown on the Site Plan, Figure 2 and the boring logs are attached in Appendix C.

Surface Conditions

The site is located in a commercial area at the northeast corner of the intersection of S. De Anza Boulevard and Via Vico. At the time of our investigation, the site was occupied by a single story retail gas station and attached car wash building which had a wood siding exterior. The canopy that covered the fuel pump islands and slab were located along the south side of the building with the drive through car wash located along the north side. Concrete pavement extended around the building and pump stations. A concrete walkway and trellis covered patio were located along the east side of the building. A two-car carport (vacuum station) was located at the northeast corner of the site. The relatively flat site was landscaped with lawn grass, small shrubs, and small to medium trees located within the landscaping areas along the perimeter of the site.

The depth and width of the existing building foundation is unknown. The perimeter stem walls were generally covered by the wood siding and not visible. The concrete pavement has numerous hairline to 1-inch wide cracks. Roof downspouts discharged into a close pipe drainage system.

Subsurface Conditions

At the location of Boring EB-1, beneath the concrete pavement we encountered approximately 3 feet of fill which consisted of stiff sandy lean clay of low plasticity underlain by approximately 2 feet of very stiff sandy lean clay of low plasticity. Beneath the fill and near surface soil, we encountered approximately 3 feet of very dense clayey sand underlain by very stiff to hard sandy lean clay of low plasticity which extended to a depth of about 33.5 feet. We then encountered dense to very dense clayey sand that extended to the maximum depth explored of 40 feet.

In Boring EB-2, beneath the concrete pavement we encountered approximately 3 feet of fill which consisted of stiff sandy lean clay of low plasticity underlain by approximately 5.5 feet of hard sandy lean clay of low plasticity. Beneath the fill and near surface soil, we encountered approximately 5 feet of dense clayey sand underlain by approximately 5 feet of hard/very dense sandy lean clay/clayey sand and 3.5 feet of very dense clayey sand. We then encountered approximately 6.5 feet of hard sandy lean clay of low plasticity underlain by approximately 5 feet of dense clayey sand underlain by very stiff sandy lean clay of low plasticity that extended to the maximum depth explored of 35 feet.

In Boring EB-3, beneath the concrete pavement we encountered approximately 3 feet of fill which consisted of very stiff sandy lean clay of low plasticity underlain by very stiff to hard sandy lean clay of low plasticity which extended to the maximum depth explored of 10 feet.

A Liquid Limit of 32 and a Plasticity Index of 15 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 generally has low plasticity and a low potential for expansion.

Ground Water

Free ground water was not encountered in our borings during or immediately following our field exploration. The borings were backfilled with grout shortly after drilling, therefore a stabilized ground water level may not have been obtained. As discussed earlier, ground water was also not encountered in the previous Earth Systems Pacific borings in 2014. Information presented in Seismic Hazard Zone Report 068 for the

Cupertino Quadrangle (California Geological Survey, 2006) indicates the historical high ground water level in the area of the site is greater than 50 feet below grade. 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

As part of our investigation, we briefly reviewed our local experience and geologic information in our files pertinent to the general area of the site. Geologic information for the area indicates the site is underlain by Pleistocene-age alluvial fan and fluvial deposits, Qpaf (Brabb, Graymer and Jones, 2000). These alluvial fan and fluvial deposits are generally expected to consist of dense, gravelly and clayey sand or clayey gravel that becomes finer grained upward transitioning into sandy clay. The geology of the general area of the site is shown on the Vicinity Geologic Map, Figure 3.

The lot and immediate site vicinity are located in an area that slopes very gently to the northeast (approximately 10 feet vertically per 1,500 feet laterally, although locally the topography may be steeper). The site is located at an elevation of approximately 270 feet above sea level.

Faulting and Seismicity

There are no mapped through-going faults within or adjacent to the site and the site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the San Andreas fault, located approximately 4.9 miles southwest of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is remote.

The San Francisco Bay Area is an active seismic region. Earthquakes in the region result from strain energy constantly accumulating because of the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906, and 1989. The faults considered most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras faults. The San Gregorio fault is located approximately 19 miles southwest of the site. The Hayward and Calaveras faults are located approximately 13 and 16 miles northeast of the site, respectively. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 1 below and are shown on the Regional Fault and Seismicity Map, Figure 4.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Hampton Inn
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. The Working Group On California Earthquake Probabilities, a panel of experts that are periodically convened to estimate the likelihood of future earthquakes based on the latest science and ground motion prediction modeling, concluded there is a 72 percent chance for at least one earthquake of Magnitude 6.7 or larger in the Bay Area before 2045. The Hayward fault has the highest likelihood of an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 14 percent, while the likelihood on the San Andreas and Calaveras faults is estimated at approximately 6 and 7 percent, respectively (Working Group, 2015).

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the 2016 California Building Code and in ASCE 7-10, “Minimum Design Loads for Buildings and Other Structures.” Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class D, stiff soil, in accordance with Chapter 20 of ASCE 7-10. Spectral acceleration response parameters S_S and S_1, and site coefficients Fa and Fv, may be taken directly from the figures and tables in the 2016 California Building Code and in the lookup tables at the U.S.G.S. website based on the latitude and longitude of the site. For the site latitude (37.3064) and longitude (-122.0318) and Site Class D, SDs = 1.420 and SD1 = 0.758.

Geologic Hazards

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

- Fault Rupture - The site is not located in a State of California Earthquake Fault Zone or area where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site and the potential for fault rupture at the site is 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 - The Seismic Hazard Zones Map of the Cupertino Quadrangle (California Geological Survey, 2002) does not include the site within a State of California liquefaction hazard zone, an area that may be underlain by soils that could be potentially susceptible to liquefaction during a major earthquake. Since a relatively deep ground water level is expected at the site and the soils encountered at the site were dense to very dense clayey sands which are not considered susceptible to liquefaction, in our opinion, the likelihood of damage from liquefaction occurring at the site is low provided the building is designed and constructed in accordance with the recommendations presented in this report.

- Differential Compaction - Differential compaction can occur during moderate and large earthquakes when soft or loose, natural or fill soils are densified and settle, often unevenly across a site. Since the soils encountered in our borings above the projected high ground water level were generally stiff to hard clays and dense to very dense sands which are not prone to differential compaction, in our opinion, the probability of significant differential compaction at the site is low.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed Hampton Inn 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 proposed basement elevation, the basement foundation is expected to bear primarily on very stiff to hard clays and dense to very dense sands. In our opinion, the building may be supported on a mat foundation bearing on medium dense/very dense sands or very stiff clayey soils at the basement level. 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.

Based on the layout of the basement, the existing and previously backfilled UST's are expected to be located entirely within the footprint of the basement, however the exact depth of the UST's and/or previous tank backfill material is unknown. If loose/soft soils or fill related to the previous or existing UST's are encountered at the basement subgrade level, we will recommend that the fill and/or soft/loose soils be removed down to a more dense/stiff soil or reworked and compacted in order to provide a competent subgrade to support the basement mat. If extensive areas deeper fills are encountered below the basement subgrade level during excavation of the basement some supplemental recommendations may be required.

We note that portions of the clayey sand 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. On a preliminary basis, 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 20 to 30 pci). The mat should also be designed with sufficient depth 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. If desired, 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.

Lateral Loads for Basement Mat

Lateral loads may be resisted by friction between the bottom of the mat 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.

An ultimate coefficient of friction of 0.375 may be assumed for the mat bearing directly on native soil. An ultimate coefficient of friction of 0.45 may be assumed for the mat foundation bearing directly on a crushed rock section. 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, where appropriate. The ultimate passive soil resistance acting on the mat foundation should be limited to 3,000 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.

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.

Reinforced Concrete Piers

In our opinion, the raised driveway deck may be supported on a reinforced concrete pier foundation bearing in stiff native soil below the surface fill. Piers should have a minimum diameter of 16-inches and should extend at least 12 feet into native soil; or deeper as required by the structural engineer. The piers may be designed for an allowable skin friction in native soil of 425 pounds per square foot for dead plus live loads, with a one-third increase allowed when considering additional short-term wind or seismic loading. The uplift capacity of the piers may be based on a skin friction value of 350 pounds per square foot. Piers should have a center-to-center spacing of at least three pier diameters.

We recommend that relatively rigid grade beams be provided between piers supporting the improvements as required by the structural engineer. If installed, we recommend that the grade beams extend at least 8-inches below adjacent finished grade or slab subgrade elevation to help limit the infiltration of surface runoff under the driveway deck.

Pier drilling should be observed by our representative to confirm that the piers are bearing in competent material, extend the required minimum depth, and have been properly cleaned and dewatered. The minimum pier depths recommended above may require adjustment if differing conditions are encountered during drilling.

Concrete should be placed in the pier excavations as soon as practical after drilling. Ground water seepage may 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.

Lateral loads on the piers may be resisted by passive earth pressure based upon an equivalent fluid pressure of 350 pounds per cubic foot, acting on 2 times the projected area of the pier. The passive resistance of the upper 3 feet of the piers should be neglected.

Settlement

On a preliminary basis, 30-year post construction total settlement due to static loads is not expected approximately 1-inch across the mat foundation or pier supported improvements. We estimate post construction differential settlement of about ¾-inch between interior columns and perimeter basement walls 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. For improved performance, exterior slabs-on-grade, such as for patios, may 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.

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.

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 equivalent fluid pressure of 42 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. Although a deep ground water conditions is expected, if the basement walls will be designed as undrained, some provision should be made in basement wall design for at least locally undrained wall backfill conditions. To account for approximately 6 feet of perched ground water behind the basement walls, we recommend adding a line load surcharge of 680 pounds per lineal foot behind the basement walls. Since perched water conditions could develop at various depths behind the basement walls, we recommend the line load surcharge be applied at various depths to check the wall design for perched water conditions. 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 retaining walls that can yield may be simulated by a line load of $6H^2$ (in pounds per foot, where H is the wall height in feet). Seismic loads on walls that cannot yield may be subjected to a seismic load as high as about $12H^2$. 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 42 pounds per cubic foot).

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

We understand that stitch piers with wood lagging possibly with tie-backs, as needed, will be used for support of the temporary basement excavation. The following preliminary geotechnical design parameters are provided for the conventional concrete filled soldier beams and lagging 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, adjacent structures, and surface features.

In our opinion, the temporary stitch piers may be designed to support an active lateral soil pressure of at least 40 pounds per cubic foot across the entire vertical excavation cut. This design soil pressure assumes that drainage can occur between shimmed wood lagging resulting in a drained soil pressure on the shoring system. Where vehicle traffic or construction loads, will be applied on the soil surface behind the back of the shoring, a lateral surcharge pressure equal to 50 percent of the vertical surcharge pressure should be included in the shoring design.

Passive soil resistance of 375 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. 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.

Some vertical and lateral deflection of the temporary shoring should be expected to occur in the planned cantilever shoring system which could result in ground settlement adjacent to the shoring. The amount of vertical and lateral deflection at the shoring face is typically on the order of ½ to 1½-percent of the total excavation depth (H) (reducing to ground settlement on the order of about $^1/_8$ to ¼ percent of H within a lateral distance of about twice 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 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 may 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.

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.

VEHICLE PAVEMENTS

Asphalt Concrete Pavements

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

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

Specifications, latest edition, except that compaction should be based on ASTM Test D1557.

Table 2. Pavement Sections
Hampton Inn
San Jose, California

Traffic Loading Condition	Design Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)	Total Thickness (inches)
Automobile Parking	4.0	3.0	6.0	9.0
Automobile Access	4.5	3.0	7.0	10.0
Light Truck Traffic	5.0	3.0	9.0	12.0
Moderate Truck Traffic	6.0	4.0	11.0	13.0
Heavy Truck Traffic	7.0	4.0	14.0	18.0

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

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

Portland Cement Concrete Pavements

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

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

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

EARTHWORK

Clearing and Subgrade Preparation

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

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
Hampton Inn
San Jose, California

	Relative Compaction*	Moisture Content*
General		
• 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.	92 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.

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 current OSHA excavation and trench safety standards.

Temporary excavations and slopes less than 4 feet deep excavated in the native soils should be capable of standing near-vertical for short construction periods with minimal bracing. Due to the potential for variation of the on-site soil, field modification of temporary cut slopes may be required. Unstable materials encountered on excavations and slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

Portions of the clayey sands encountered at the site were judged to have limited cohesion and will be prone to sloughing and/or caving if excavated near-vertical. This information should be considered by the contractor when establishing temporary shoring/sloping criteria for basement excavation.

Percolation Rates

The upper 5 to 10 feet of soils at the site were classified as primarily clayey soils that have low permeability. Based on the consistency and plasticity of the surface and near-surface soils, the rate of surface water infiltration into surface or near surface infiltration basins is estimated to be no greater than about 0.1 to 0.3 inches per hour. In Borings EB-1 and EB-2, we encountered a 3 to 5 foot deep strata of dense to very dense clayey sand below a depth of about 5 to 8.5 feet. If a deeper infiltration system is planned, the infiltration rate of the clayey sand strata may be estimated to be about 0.5 to 1.5 inches per hour. A site specific infiltration rate could be developed by performing an onsite field infiltration test at the specific location and depth of the planned infiltration areas, if requested. Drains should be provided for infiltration basins that direct water to an appropriate outlet as required by the civil engineer.

Surface Drainage

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

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, pier drilling, tie-back and/or soil nail installation, mat and/or slab subgrade preparation, utility trench backfill, basement wall drainage and backfill, pavement construction, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated April 18, 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) 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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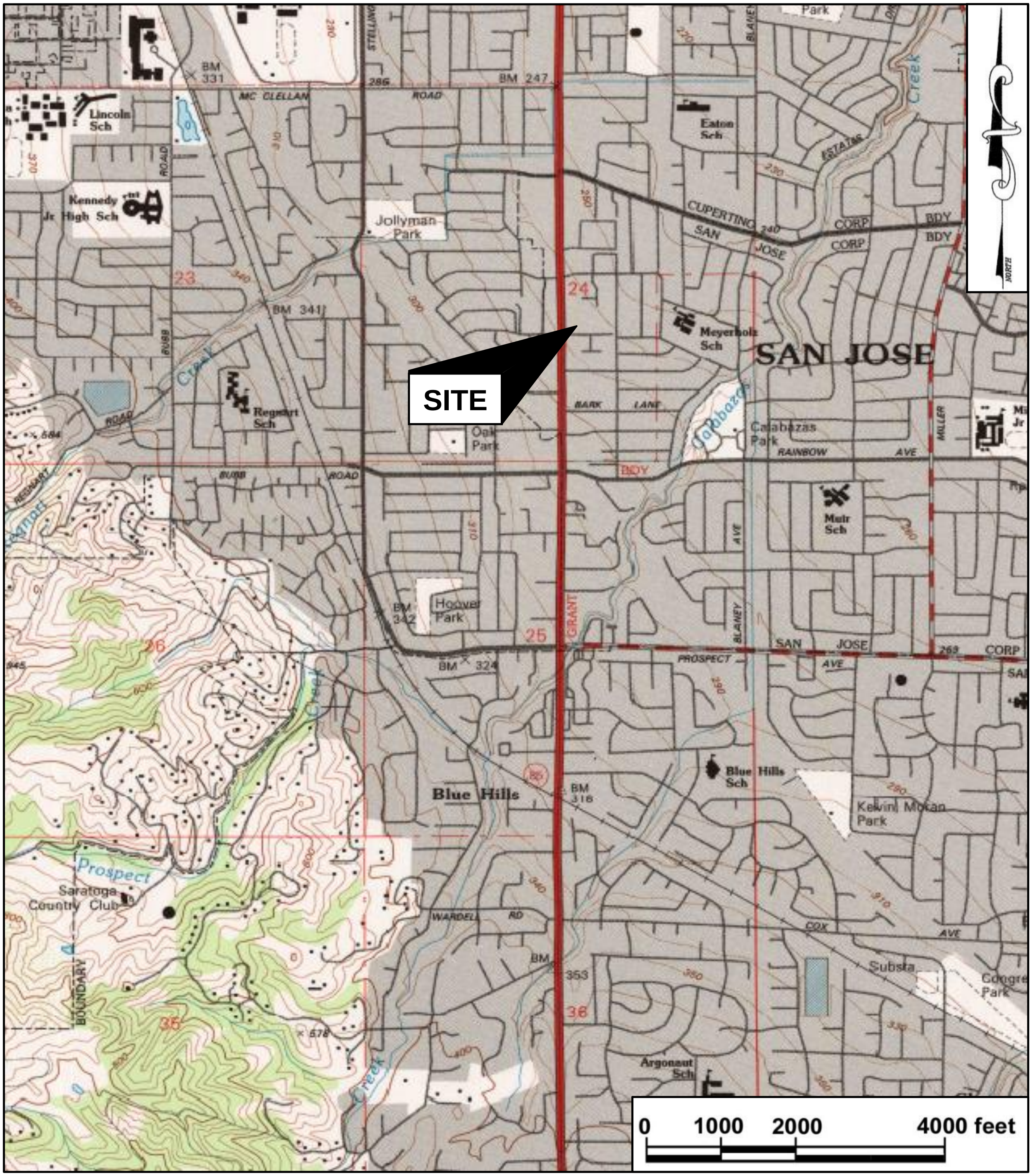


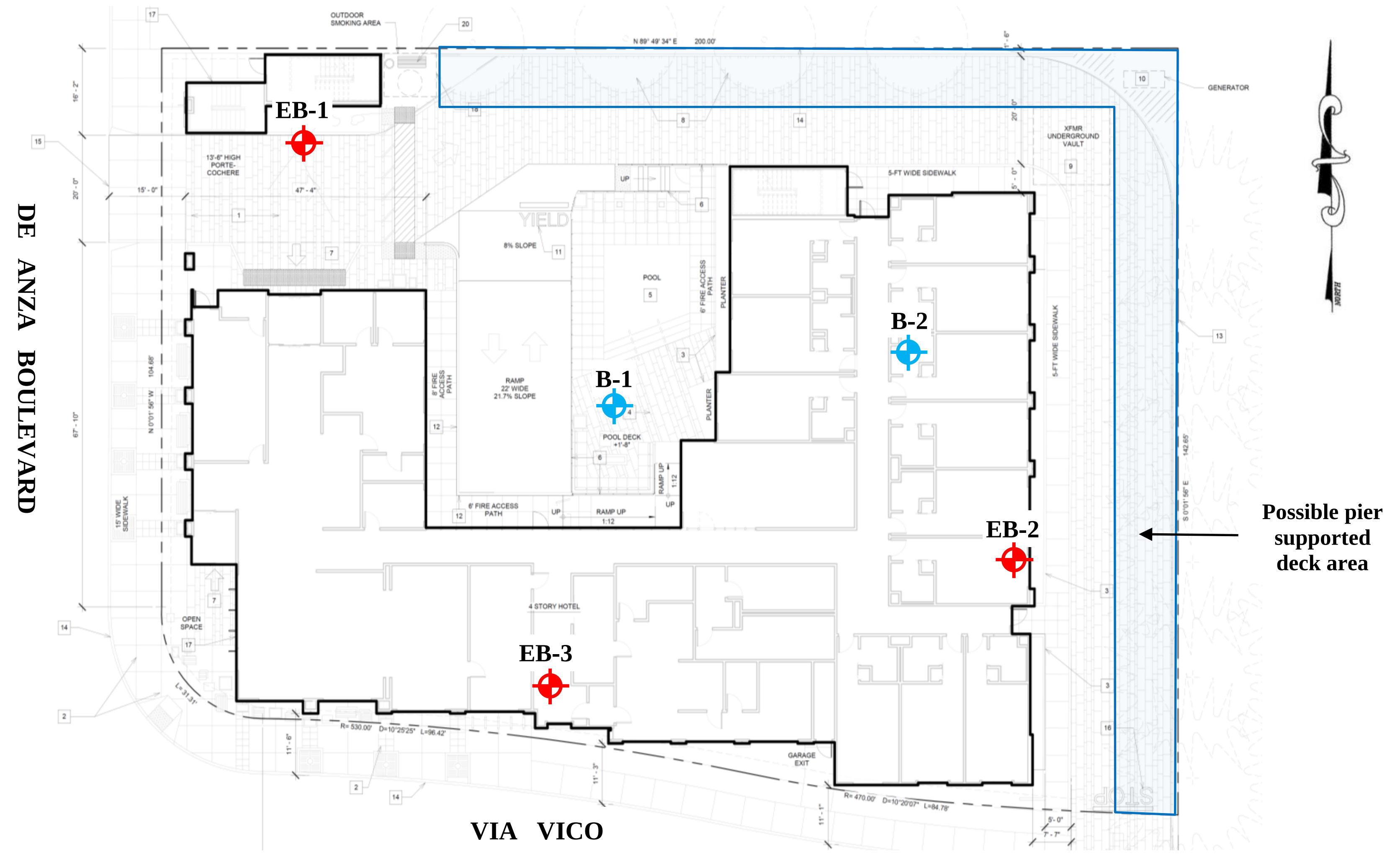
Scale: 1 inch = 2000 feet

Base is United States Geological Survey Cupertino 7.5 Minute Quadrangle, dated 1991.

VICINITY MAP
HAMPTON INN
SAN JOSE, CALIFORNIA

FIGURE 1
APRIL 2017
PROJECT NO. 4017-1

ROMIG
ENGINEERS



LEGEND

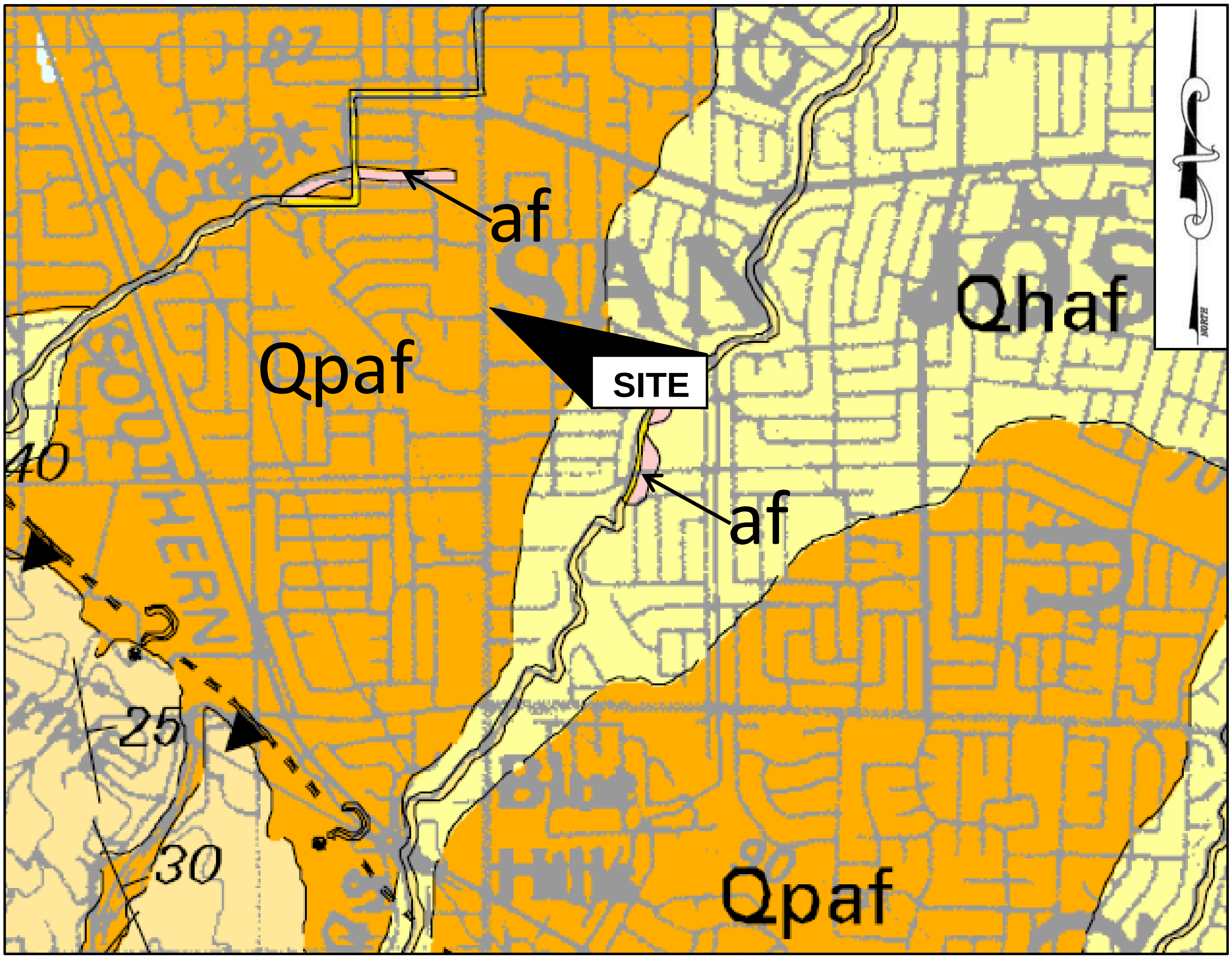
EB-3	🔴	Approximate Location of Exploratory Boring by Romig Engineers (2017).
B-2	🔵	Approximate Locations of Exploratory Borings (Earth Systems Pacific, 2014).

Approximate Scale: 1 inch = 30 feet.
Base is site plan prepared by RYS Architects, Inc., dated August 9, 2016

SITE PLAN
HAMPTON INN
SAN JOSE, CALIFORNIA

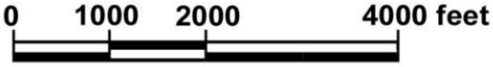
FIGURE 2
APRIL 2017
PROJECT NO. 4017-1

ROMIG ENGINEERS



LEGEND

Qhaf	Alluvial fan and fluvial deposits		Geologic Contact - dashed where approximate, dotted where inferred.
Qpaf	Alluvial fan and fluvial deposits		Fault - dashed where approximate, dotted where inferred.
af	Artificial fill		Fault - thrust or reverse

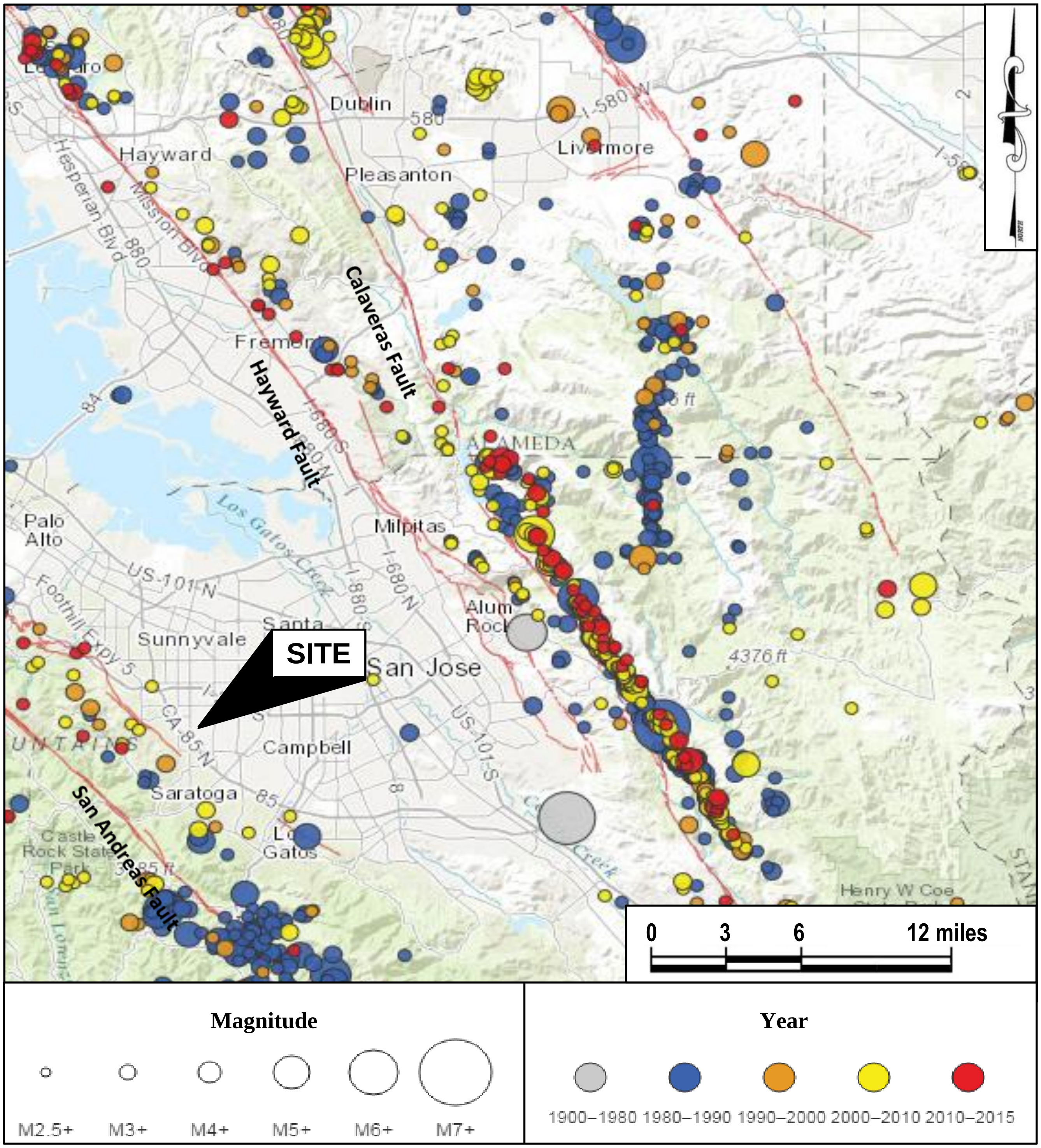


Scale: 1 inch = 2000 feet
Base is Geologic Map of Palo Alto 30 x 60 Minute Quadrangle, (Brabb, Graymer, and Jones, 2000).

VICINITY GEOLOGIC MAP
HAMPTON INN
SAN JOSE, CALIFORNIA

FIGURE 3
APRIL 2017
PROJECT NO. 4017-1

ROMIG ENGINEERS



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
HAMPTON INN
SAN JOSE, CALIFORNIA

FIGURE 4
APRIL 2017
PROJECT NO. 4017-1

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
HAMPTON INN
SAN JOSE, CALIFORNIA

FIGURE A-1
APRIL 2017
PROJECT NO. 4017-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 3/16/17

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)*
7 inches of reinforced concrete.				0					
Fill: Dark brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine subangular to subrounded gravel, low plasticity.	Stiff	CL				16	11		>4.5
Brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine subangular to subrounded gravel, low plasticity, tan and orange mottling. ■ Liquid Limit = 32, Plasticity Index = 15.	Very Stiff	CL		5	■	29	12	0.4	1.5
Brown, Clayey Sand, moist, fine to medium grained sand. ● 26% Passing No. 200 Sieve.	Very Dense	SC			●	50/3"	12		
Brown, Sandy Lean Clay, moist, fine grained sand, low plasticity. ◆ Dry Density = 134 pcf.	Hard	CL		10	◆	42	16		3.0
				15		60	14	1.5	4.3
Silt content, low to moderate plasticity.				20		37	19	0.9	4.5
Continued on Next Page									

EXPLORATORY BORING LOG EB-1
HAMPTON INN
SAN JOSE, CALIFORNIA

BORING EB-1
PAGE 1 OF 2
APRIL 2017
PROJECT NO. 4017-1

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 3/16/17

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)*
Dark brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine subangular to subrounded gravel, low plasticity.	Very Stiff	CL		**20**					
				25		30	17	0.2	2.5
Fine to medium grained sand, fine subangular gravel.									
				30		31	14		
Light brown, Clayey Sand, moist, fine to coarse grained sand, fine subangular to subrounded gravel, low plasticity fines, red and orange mottling.	Dense to Very Dense	SC							
				35		50	9		
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. ● 29% Passing No. 200 Sieve.									
				40	●	34	9		
Bottom of Boring at 40 feet.									

EXPLORATORY BORING LOG EB-1

HAMPTON INN

SAN JOSE, CALIFORNIA

BORING EB-1

PAGE 2 OF 2

APRIL 2017

PROJECT NO. 4017-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 3/16/17

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)*
7 inches of reinforced concrete.				0					
Fill: Grayish brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine subangular gravel, low plasticity, dark orange and tan mottling.	Stiff	CL				14	13		3.8
Brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine subangular gravel, low plasticity, dark orange and tan mottling.	Hard	CL		5		40	15		>4.5
◆ Dry Density = 143 pcf.					◆	50/6"	7		>4.5
Light brown, Clayey Sand, moist, fine to coarse grained sand, fine subangular to subrounded gravel, low plasticity fines.	Dense	SC		10	●	45	8		
● 24% Passing No. 200 Sieve.									
Brown, Sandy Lean Clay/Clayey Sand, fine to coarse grained sand, fine subrounded gravel, low plasticity fines.	Hard/ Very Dense	CL/ SC		15	◆	50/6"	11		2.5
◆ Dry Density = 133 pcf.									
Light brown, Clayey Sand, moist, fine to coarse grained sand, fine subangular to subrounded gravel.	Very Dense	SC		20		70	9		

Continued on Next Page

EXPLORATORY BORING LOG EB-2
HAMPTON INN
SAN JOSE, CALIFORNIA

BORING EB-2

APRIL 2017
PROJECT NO. 4017-1

ROMIG ENGINEERS

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 3/16/17

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 brown, Clayey Sand, moist, fine to coarse grained sand, fine subangular to subrounded gravel.	Very Dense	SC		**20**					
Brown, Sandy Lean Clay, moist, fien to coarse grained sand, fine subrounded gravel, low plasticity.	Hard	CL		**25**		38	18		>4.5
Light brown, Clayey Sand, moist, fine to coarse grained sand, fine subangular to subrounded gravel. ● 40% Passing No. 200 Sieve.	Dense	SC		**30**	●	36	13		
Brown, Clayey Sand, fine to coarse grained sand, fine subrounded gravel, low plasticity.	Very Stiff	CL		**35**		20	16		3.8
Bottom of Boring at 35 feet.				**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
HAMPTON INN
SAN JOSE, CALIFORNIA

BORING EB-2
PAGE 2 OF 2
APRIL 2017
PROJECT NO. 4017-1

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

DEPTH TO GROUND WATER: Not Encountered **SURFACE ELEVATION:** NA **DATE DRILLED:** 3/16/17

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)*
7 inches of reinforced concrete.				0					
Fill: Dark brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine subangular to subrounded gravel, low to moderate plasticity, red and orange mottling.	Very Stiff to Hard	CL				21	11		4.0
Brown, Sandy Lean Clay, low plasticity, fine to medium grained sand.	Very Stiff to Hard	CL				31	17		1.8
				5		43	15	0.9	4.3
Coarse subangular to subrounded gravel.						50/6"	15		
						50/5"	19	0.5	4.0
				10		40	18		>4.5
Bottom of Boring at 10 feet.									
				15					
				20					

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

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-3
HAMPTON INN
SAN JOSE, CALIFORNIA

BORING EB-3
APRIL 2017
PROJECT NO. 4017-1

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 one sample of soil in accordance with ASTM D4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The results of this test are presented in Figure B-1 and on the log of Boring EB-1 at the appropriate sample depth.

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

The amount of Dry Density was determined on three samples of soil in accordance with ASTM D7263. The results of these tests are presented on the boring logs at the appropriate sample depths.

❖ ❖ ❖ ❖ ❖

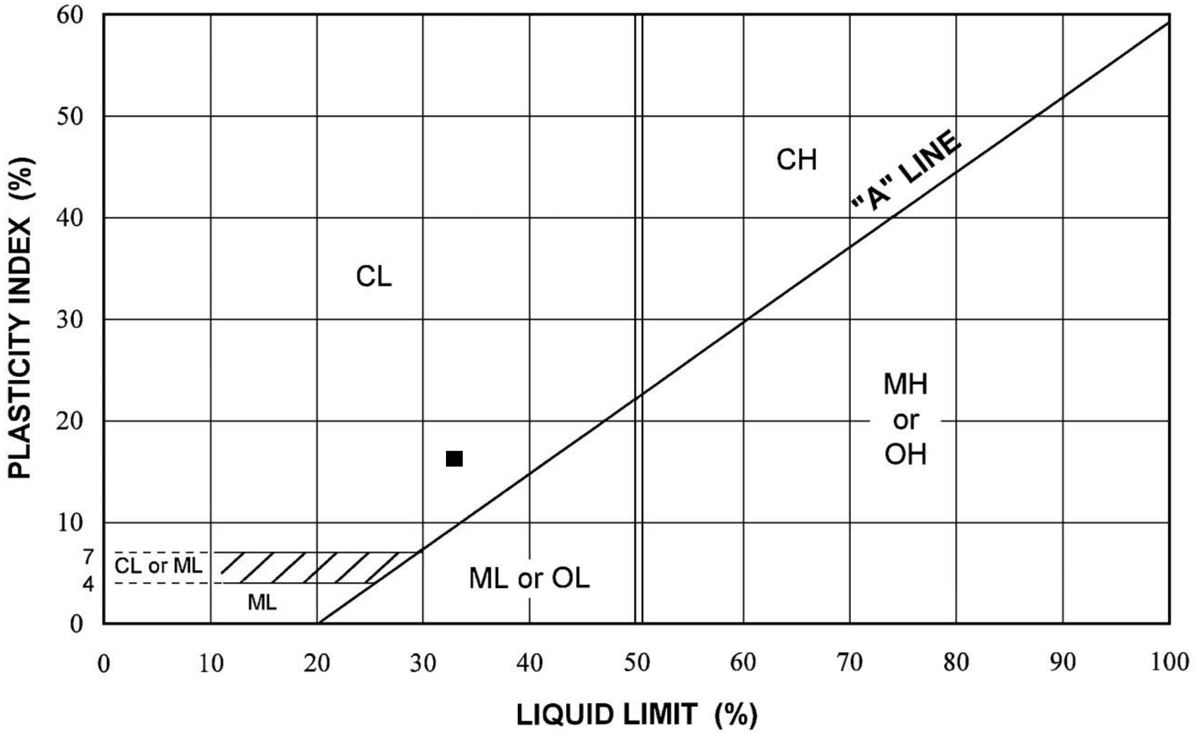


Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	3-4.5	12	32	15	-33		CL

PLASTICITY CHART
HAMPTON INN
SAN JOSE, CALIFORNIA

FIGURE B-1
APRIL 2017
PROJECT NO. 4017-1

APPENDIX C

PREVIOUS EXPLORATION LOGS

Boring Logs B1 and B2 (Earth Systems Pacific, 2014)

❖ ❖ ❖ ❖ ❖

Earth Systems Pacific

LOGGED BY: X Mejia
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

Boring No. 1
PAGE 1 OF 1
JOB NO.: SH-12410-SA
DATE: 4/1/14

Car Wash Convenience Store Addition
1090 S. De Anza Blvd
San Jose, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA						
				INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
0 - 1	CL	▨	Lawn Brown lean CLAY, very moist, medium stiff							
2 - 3	CL	▨	Gray and dark orange-brown mottle lean CLAY, very moist, stiff	1.0-2.5	1-1	▬	115.5	18.9	7 8 9	1.5
4 - 5		▨	-more reddish brown with gray streaks, moist, hard; trace coarse sunrounded gravel	3.5-5.0	1-2	▬	109.8	19.5	7 13 25	2.25
9 - 10		▨	-less gray streaks, increase fine and coarse subrounded gravels	8.5-10.0	1-3	▬	116.6	13.8	22 21 38	3.5
11 - 13		▨	-more brown							
14 - 15	SC	▩	Reddish brown clayey SAND, moist, dense; trace gravels	13.5-15.0	1-4	●	-	11.5	10 13 22	-
15 - 26			End of boring at 15 feet Groundwater not encountered							

LEGEND: ▬ 2.5" Mod Cal Sample ○ Bulk Sample ▭ 2.0" Mod Cal Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 2

PAGE 1 OF 1
JOB NO.: SH-12410-SA
DATE: 4/1/14

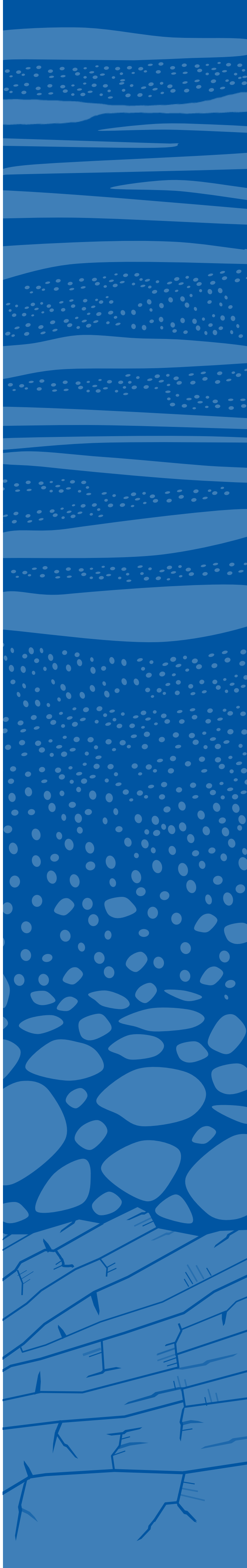
LOGGED BY: X Mejia
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

Car Wash Convenience Store Addition
1090 S. De Anza Blvd
San Jose, California

				SAMPLE DATA						
DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
0	CL		Lawn Brown lean CLAY, very moist, medium stiff							
1				0.0-2.0	A	○				
2	CL		Gray and dark orange-brown mottle lean CLAY, very moist, stiff							
3				2.0-3.5	2-1	▬	108.5	19.8	6 9 11	1.25
4										
5										
6	CL		Reddish brown to brown sandy lean CLAY, moist, hard; trace subrounded gravels and sandstone gravels	5.0-6.5	2-2	▬	117.7	15.0	9 21 31	4.0
7										
8										
9	SC		Reddish brown clayey SAND, moist, dense; medium sand, fine to coarse gravels							
10				8.5-10.0	2-3	▬	116.2	12.3	15 30 50	-
11										
12										
13										
14										
15				13.5-15.0	2-4	●	-	9.1	15 23 25	-
16			End of boring at 15 feet Groundwater not encountered							
17										
18										
19										
20										
21										
22										
23										
24										
25										
26										

LEGEND: ▬ 2.5" Mod Cal Sample ○ Bulk Sample ▭ 2.0" Mod Cal Sample ● SPT

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.



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