GEOTECHNICAL INVESTIGATION
FOR
4-STORY HOTEL BUILDING
1036 N. 4TH STREET
SAN JOSE, CALIFORNIA 95112

May 2016

Prepared for

Mr. Anil Patel
1036 N. 4th Street
San Jose, California 95112

Project No. 3745-1A

ROMIG ENGINEERS, INC.
GEOTECHNICAL & ENVIRONMENTAL SERVICES

May 17, 2016
3745-1A

Mr. Anil Patel
1036 N. 4th Street
San Jose, California 95112

RE: GEOTECHNICAL INVESTIGATION
4-STORY HOTEL BUILDING
1036 N. 4TH STREET
SAN JOSE, CALIFORNIA

Dear Mr. Patel:

As requested, we have performed a geotechnical investigation for the proposed hotel building to be constructed at 1036 N. 4th Street 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.

Lucas J. Ottoboni, P.E.

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GEOTECHNICAL INVESTIGATION
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SAN JOSE, CALIFORNIA 95112

PREPARED FOR:

MR. ANIL PATEL

1036 N. 4TH STREET

SAN JOSE, CALIFORNIA 95112

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MAY 2016

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4-STORY HOTEL BUILDING
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SAN JOSE, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed hotel building to be constructed at 1036 N. 4^{th} Street in 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 four-story hotel building over one level of underground parking at your property in San Jose. The underground parking will extend beneath nearly the entire site, will be approximately 15,200 square feet in plan dimension and will include a ramp down at the southeast side. We assume that the basement elevation will be at a depth of approximately 10 to 13 feet below existing grade. In addition, elevator shafts as well as garage lifts may also be included and would extend deeper. The four-story portion of the structure is expected to be centrally located above the underground parking. The existing motel will be demolished prior to construction.

Scope of Work

Our scope of work for this investigation was presented in our agreement with Mr. Anil Patel dated March 25, 2016. 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.
- Subsurface exploration consisting of drilling, sampling, and logging two exploratory borings in the area of the proposed hotel 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 Mr. Anil Patel for specific application to developing geotechnical design criteria for the proposed hotel building to be constructed at 1036 N. 4th Street in San Jose, California. We make no warranty, expressed or implied, for the services 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 planned improvements; review of readily available reports relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes occur, we should be advised so that we can review our report in light of those changes.

SITE EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on April 29, 2016. Subsurface exploration was performed using a Mobile B-40 truck-mounted drill equipped with 8-inch diameter hollow-stem augers. Two exploratory borings were advanced to depths of 35 and 50 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 northeast side of N. 4th Street. At the time of our investigation, the site was occupied by a U-shaped single-story motel building that had a two-story office at the front. Covered carports were located adjacent to each room. Asphalt concrete parking and driveway areas were located at the center of the site. A concrete handicap parking space and walkway were located near the front and a concrete pad was located at the north corner. The relatively flat site included lawn grass and small trees.

Based on the age, we expect that the building is supported on a conventional shallow foundation system, although the depth and width of the foundations are unknown. The exterior stem wall was generally covered by the stucco siding and was not visible. The surrounding asphalt concrete parking lot appeared to be in adequate condition with some hairline to 1-inch wide cracks observed. The concrete walkway and handicap parking space at the front appeared to be in good condition. The concrete pad at the north corner of the lot appeared to be in poor condition with cracks up to 1/2-inch wide and up to 1/2-inch out of level. Roof downspouts appeared to discharge adjacent to the perimeter foundations.

Subsurface Conditions

At the location of our borings, we encountered approximately 13 feet of stiff to very stiff sandy lean clay of low to moderate plasticity underlain by stiff to very stiff fat clay of high plasticity to the maximum depth explored of 35 feet in Boring EB-2 and to a depth of about 43.5 feet in Boring EB-1. Beneath the fat clay in Boring EB-1, we encountered very stiff lean clay of low plasticity to the maximum depth explored of 50 feet.

We note that in Boring EB-1 we encountered approximately 4 feet of medium dense to stiff silty sand to sandy silt between depths of 9.5 and 13.5 feet. The medium dense sandy silt/silty sand stratum encountered in Boring EB-1 at or below the basement parking level may be prone to liquefaction during a moderate to strong earthquake. Details of our dynamic settlement evaluation are included in the section titled "Liquefaction Evaluation."

A Liquid Limit of 41 and a Plasticity Index of 17 were measured on a sample of near-surface soil in Boring EB-1, indicating a low to moderate potential for expansion. Liquid Limits of 55, 50, and 51 were measured on three samples between depths of approximately 13.5 to 40 feet in Borings EB-1 and EB-2 indicating the clayey soils in the noted depth range have a high plasticity. A Liquid Limit of 31 was measured on a sample at a depth of approximately 45 feet in Boring EB-1, indicating a decrease to low plasticity.

Ground Water

Free ground water was measured at a depth of approximately 13.5 feet during the drilling of Boring EB-1. The boring was backfilled with grout immediately following drilling; therefore a stabilized ground water depth may not have been obtained. Information in Seismic Hazard Zone Report 058 for the San Jose West Quadrangle (California Geological Survey, 2002) indicates the historical high ground water level to be approximately 8 feet below the ground surface in the area of the site. 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.

We also reviewed groundwater monitoring data for three nearby environmental sites listed on the State's Geotracker website. The highest historical readings from the three sites reviewed in detail were 5.7 feet, 7.5 feet, and 9.2 feet for sites at 1120 North 1st Street, 1091 North 10th Street, and 1170 North 4th Street, respectively. The highest historical readings were measured in 2005 at each of the three sites.

Based on our experience, it is possible for ground water to rise up higher than the historical highest ground water level. In our opinion, a design ground water level of about 5 feet below the existing ground surface may be used for the proposed basement design.

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 that the site is underlain by Holocene-age flood plain deposits, Qhfp (Wentworth, Blake, McLaughlin, Graymer, 1999). These deposits are generally expected to consist of medium to dark gray, dense, sandy to silty clay. Lenses of coarser material (silt, sand, and pebbles) may be locally present. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

The Seismic Hazard Zones Map of the San Jose West Quadrangle (California Geological Survey, 2002) indicates that the site is located in an area that is potentially susceptible to liquefaction during a major earthquake. The potential for liquefaction of the soils encountered at the site is discussed later in this report.

The lot and the immediate site vicinity are located in an area that slopes very gently to the north (approximately 10 feet vertically per 1800 feet laterally, although locally the topography may be steeper). The site is located at an elevation of approximately 60 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 southeast extension of the Hayward fault, located approximately 5.1 miles northwest of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is low.

The San Francisco Bay Area is, however, an active seismic region. Earthquakes in the region result from strain energy constantly accumulating because of the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906 and 1989. The faults considered most likely to produce large earthquakes in the area include the San Andreas, San Gregorio, Hayward, and Calaveras faults. The Calaveras and San Andreas faults are located approximately 8.0 and 13 miles northeast and southwest of the site, respectively. The San Gregorio fault is located approximately 27 miles southwest of the site. These faults and significant earthquakes that have been documented in the Bay Area are listed below in Table 1.

Table 1. Earthquake Magnitudes and Historical Earthquakes
4-Story Hotel Building
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 2013 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 2013 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 (37.3577) and longitude (-121.9018) and Site Class D, SDs = 1.000 and SD1 = 0.600.

Liquefaction Evaluation

Severe ground shaking during an earthquake can cause loose to medium dense granular soils to densify. If the granular soils are below ground water, their densification can cause increases in pore water pressure, which can lead to soil softening, liquefaction, and ground deformation. Soils most prone to liquefaction are saturated, loose to medium dense, silty sands and sandy silts with limited drainage, and in some cases, sands and gravels that are interbedded with or that contain seams or layers of impermeable soil. Soils with normalized standard penetration test, $(N_1)_{60}$, greater than 30 blows per feet were considered too dense to liquefy.

The silty sand to sandy silt stratum encountered in Boring EB-1 between depths of 9.5 and 13.5 feet, which is below the design ground water depth estimated to be at about 5 feet below ground surface, was considered in our liquefaction analysis. A peak ground acceleration (PGA) of 0.50g, the PGA_M for maximum considered earthquake based on ASCE 7-10, was also considered in our evaluation.

We evaluated the potential for earthquake-induced liquefaction of the clayey strata encountered below the historical high ground water level at the site using the guidelines described in CDMG Special Publication 117 (1997) and the methods described in the 2006 publication by Bray and Sancio titled "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils." According to Bray and Sancio (2006), fine grained soils need to satisfy the following criteria in order to be considered potentially susceptible to severe strength loss and liquefaction during an earthquake:

$PI < 12$:	$Wc/LL > 0.85$
$12 < PI < 18$:	$Wc/LL > 0.80$
$PI \geq 18$:	Not susceptible to liquefaction

The results of our laboratory tests and liquefaction evaluation of the sandy silt, and clay of moderate to high plasticity encountered at the site are presented on Table 2 below.

Table 2: Results of Liquefaction Evaluation of On-site Soils
4-Story Hotel Building
San Jose, California

Boring No.	Strata Depth (ft)	Soil Type	N1(60)cs (blow/ft)*	Liquid Limit (%)	Plasticity Index (%)	Water Content (%)	Wc/LL	Potentially Susceptible To Liquefaction
EB-1	9.5 – 13.5	SM/ML	17	--	--	23	--	Yes
EB-1	23.5-25	CH	14	55	--	27	0.49	No
EB-1	33.5-35	CH	14	50	--	28	0.56	No
EB-1	43.5-45	CL	18	31	--	24	0.77	No
EB-2	13.5-15	CH	14	52	--	28	0.54	No

* Normalized standard penetration test corrected to an equivalent clean sand value.

Based on our analyses on the boring data summarized in the above table, the medium dense to stiff silty sand to sandy silt encountered in Boring EB-1 between depths of 9.5 and 13.5 feet could be considered potentially susceptible to liquefaction during strong seismic shaking. Total settlement at the ground surface that could occur as a result of liquefaction in this stratum from the design-level earthquake is estimated to be up to about 1.25 inches. This settlement was estimated based on the entire 4-foot thick stratum of silt starting at 9.5 feet. Since we expect the basement excavation and thickness of surface compacted subgrade will be deeper than 9.5 feet (although the final excavation depth has not yet been determined), in our opinion, the estimated settlement at the bottom of the basement mat due to liquefaction within the silt strata will decrease at a rate of about 0.3 inches of settlement per foot of excavation/compaction extending below a depth of 9.5 feet. Since this silty stratum was not encountered in Boring EB-2 and may not be continuous, the total settlement should be considered the amount of differential settlement and should be considered in the structural design. As a worst case, the basement mat should be designed for differential settlement of about 1.25 inches over a horizontal span of 50 feet.

In addition, as shown above, the clay strata beneath 13.5 feet were determined to be highly plastic fat clay or lean clay which did not appear to be liquefiable.

Geologic Hazards

We briefly reviewed the potential for geologic hazards other than liquefaction to impact the site, considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below.

- Fault Rupture - The site is not located in an Earthquake Fault Zone or area where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site, and the potential for fault rupture to occur at the site is considered low, in our opinion.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Strong ground shaking should therefore be expected several times during the design life of the development, as is typical for sites throughout the Bay Area. The building should be designed and constructed in accordance with current earthquake resistance standards.

- Differential Compaction - Differential compaction occurs during moderate and large earthquakes when soft or loose, natural or fill soils are densified and settle, often unevenly across a site. Since the materials encountered in our borings above the design ground water table were generally stiff to hard clay, in our opinion, the likelihood of significant differential compaction of these materials is low. The potential for seismic related settlement below the design ground water table was addressed in the Liquefaction section of our report.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed 4-story hotel building provided the recommendations presented in this report are followed during design and construction. Specific geotechnical recommendations for the project are presented in the following sections of this report.

The primary geotechnical concerns for the proposed construction are the relatively high historical ground water level, the need for shoring at the property lines, the presence of the medium dense silty sand to sandy silt stratum encountered near the bottom of the assumed basement elevation that may be susceptible to liquefaction during strong seismic shaking, and the potential for severe ground shaking at the site during a major earthquake. Based on the expected basement elevation, the basement foundation is expected to bear in stiff to clays or stiff to medium dense sandy silt to silty sand. In our opinion, the building may be supported on a mat foundation bearing on stiff clayey and medium dense silty soils at the basement level. At least the upper 6 inches of the basement subgrade soils should be scarified and surface compacted.

Based on our field investigation and experience with other sites in the area, the highest projected ground water level at the site is estimated to be approximately 5 feet below the existing ground surface. In our opinion, because of the potential for relatively high ground water, the basement mat should be designed for hydrostatic conditions from a depth of approximately 5 feet or a fail-safe method for potentially continuous dewatering of the basement mat and retaining walls should be designed to prevent the basement from becoming buoyant. As an alternative, a subsurface drain pipe could be set about 2 feet above the finish floor elevation closer to the ground surface to reduce the possibility of continuous pumping of ground water. If this option is selected, the basement should be designed for hydrostatic pressures starting at the elevation/depth of the perforated pipe.

Since the basement will occupy the entire lot, it will likely not be possible to slope back the excavation and therefore shoring will be needed along the property lines. The contractor should be responsible for staging the required cuts and wall construction and the design of temporary cut slopes and required shoring. Protection of structures and slopes near cuts should also be the responsibility of the contractor.

As indicated in the above section, up to about 1.25 inches of total settlement and differential settlement over a horizontal distance of about 50 feet are estimated at the ground surface and at the basement level from liquefaction during the design earthquake event. If the basement excavation and thickness of surface compacted soil extend below a depth of 9.5 feet, the amount of differential settlement to be considered beneath the basement mat may be reduced as indicated in the section above titled “Liquefaction Evaluation.” The structural engineer should consider the potential seismic related settlement during building design.

In addition, construction dewatering may be required depending on the depth of basement excavation and the ground water level at the time of excavation. It should be noted that providing water proofing recommendations is outside of our scope of services and expertise. We note however, that providing adequate waterproofing of the basement mat and walls is essential for the success of the basement.

We also note that portions of the medium dense sandy silt to silty sand encountered in Boring EB-1 between depths of approximately 9.5 to 12 feet 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/slope 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 may be supported on a mat foundation at the basement level. The mat may be designed for an average allowable bearing pressure of 2,000 pounds per square foot for combined dead plus live loads, with maximum localized bearing pressures of 3,500 pounds per square foot at column or wall loads. These pressures may be increased by one-third for total loads including wind or seismic forces. These pressures are net values; the weight of the mat may be neglected in design.

Preferably, the mat foundation should be designed to resist hydrostatic uplift pressure resulting from the highest projected ground water level of 5 feet below the existing ground surface or a fail-safe method should be designed and constructed for dewatering or reducing hydrostatic pressure on the basement mat as discussed previously.

A water-proofing system designed by others should be installed below and around the edges of the mat foundation (and behind the basement walls).

A modulus of subgrade reaction (Kv1) of 100 pounds per cubic inch may be assumed for the basement subgrade. This value is based on a 1-foot square bearing area and should be scaled to account for mat foundation size effects. Alternatively, when the building loads are known, we could be contacted to provide a modulus of subgrade reaction based on the anticipated building load and differential settlement, (Kv).

The bottom of the excavation for the basement mat should be cleaned of all loose or soft soil and debris. At least the upper 6 inches of the basement subgrade should be scarified and compaceted to a relative compaction of at least 90 percent (ASTM D1557).

Lateral Loads

Lateral loads may be resisted by friction between the bottom of the damp-proofing membrane (or vapor barrier) and the supporting subgrade. The structural engineer should consult with the damp-proofing manufacturer for coefficient of friction to be assumed for design. We recommend assuming an equivalent fluid pressure of 300 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the mat foundation is not covered and protected by a concrete slab or pavement.

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. The damp-proofing system below the basement mat may be placed directly on the prepared subgrade soils, a 4- to 8- inch section of crushed rock or on a thin working slab, as determined by the water-proofing consultant and design team.

Settlement

At this time, the column layout and structural loads on the basement mat are not available. On a preliminary basis, we estimate that the 30 year post-construction differential settlement due to static loads is not expected to exceed about 3/4-inch across the building supported on a mat foundation, provided that the foundations are designed and constructed as recommended.

In addition, as indicated in the above section, up to about 1.25 inches of differential settlement over a horizontal distance of about 50 feet is estimated at the ground surface and potentially at the basement level from liquefaction during the design earthquake event. If the basement excavation and scarified subgrade extends below a depth of 9.5 feet, the amount of differential settlement to be considered beneath the basement mat may be reduced as indicated in the section above titled “Liquefaction Evaluation.” The structural engineer should consider the potential seismic related settlement during building design.

The settlement estimates should be updated during the design when structural loads are available.

BASEMENT RETAINING WALLS

Basement retaining walls should be designed to resist lateral pressures from the adjacent native soil and backfill. Drained retaining walls with level backfill that are not free to deflect or rotate, such as basement walls, should be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot (where H is the height of the wall in feet). Drained retaining walls with level backfill that are free to rotate and able to rotate freely may be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot. Where the walls will be subjected to surcharge loads, such as from foundations, vehicular traffic, or construction loading, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Where basement retaining walls are assumed as undrained and subject to ground water pressure below the design ground water depth of 5 feet, 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.

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 $7H^2$ (in pounds per foot, where H is the wall height in feet). Seismic loads on walls that cannot yield, such as the basement retaining walls, may be subjected to a seismic load as high as about $13H^2$. This seismic surcharge line load should be assumed to act at 1/3H above the base of the wall (in addition to the active wall design pressure of 45 pounds per cubic foot).

To prevent buildup of water pressure from surface water infiltration or ground water, a subsurface drainage system could be installed behind the basement walls. The drainage system may consist of a conventional gravel backdrain or an approved drainage fabric.

If a gravel backdrain is used, a 4-inch diameter perforated pipe (perforations placed down) should be 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 free-draining outlet or sump that pumps to a suitable location. Damp-proofing of the basement walls should be included in areas where wall dampness and efflorescence would be undesirable. A diagrammatic section illustrating a typical drainage system for the basement is shown on Figure 4.

Miradrain, Enkadrain or other drainage fabrics approved by our office may 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 down to the drain pipe at the base of the wall. 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 placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used for compaction of wall backfill, the walls should be temporarily braced.

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

SLABS-ON-GRADE

General Slab Considerations

The surface and near surface soils at this site have a moderate potential for expansion. 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 native or fill soil subgrade should be kept moist up until the time the non-expansive fill and/or aggregate base is placed. Slab subgrades and non expansive fill should be prepared and compacted as recommended in the section of this report titled "Earthwork." Exterior flatwork should be underlain by a layer of non expansive fill as discussed below. The non expansive fill should consist of aggregate base rock, granular soil, or a clayey soil with a plasticity index of 15 or less.

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

Exterior Flatwork

Near surface concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 6 inches of Class 2 aggregate base. It would be preferable for exterior slabs-on-grade, such as for patios, to 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. Where used, the thickened edges should be at least 8 inches wide and should extend at least 2 inches below the bottom of the underlying aggregate base layer.

Interior Slabs

Concrete slab-on-grade floors should be constructed on a layer of non-expansive fill at least 6 inches thick. In areas where dampness of concrete floor slabs would be undesirable, such as within the building interior, concrete slabs should be underlain by at least 6 inches of clean gravel, such as ½- to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used for this capillary break material. The crushed rock layer should be densified and leveled with vibratory compaction equipment.

To reduce vapor transmission up through concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor barrier meeting the minimum ASTM E 1745, Class C requirements or better. Preferably, the vapor barrier should be placed directly below the floor slab. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations. The crushed rock section may be considered as the non-expansive fill layer recommended above.

Please note that the below-grade portion of the mat should be underlain by a high-quality water-proofing membrane. The membrane should be selected by your water proofing consultant.

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

Basement Mat Drainage

If the basement and basement mat will be designed for hydrostatic conditions from the design ground water level, a drainage system beneath the basement mat need not be considered. However, if it is desired to reduce the possibility of water pressure developing below the basement mat and floor damp-proofing system, a subsurface drain system may be installed below the basement mat. If installed, the 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 4- to 8-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 4.

As an alternative to a basement drainage pipe below the basement mat, as shown on Figure 4, a subsurface drain pipe could be set about 2 feet above the finish floor elevation of the basement to reduce the possibility of continuous pumping of ground water. If this option is selected, the basement should be designed for hydrostatic pressures starting at the elevation/depth of the perforated pipe and with the crushed rock layer left in place below the mat foundation and behind the basement retaining walls. We could be contacted during design to provide input regarding the elevation of the drainage pipe.

PORTLAND CEMENT CONCRETE PAVEMENT

If the driveway and/or entrance ramp to the lower parking level will be constructed with Portland cement concrete (PCC), we recommend the driveway pavement consist of at least 5 inches of PCC on at least 8 inches of Class 2 aggregate base. Un-reinforced concrete for the 5-inch-thick driveway pavement should have a 28-day compressive strength of at least 3,500 psi. PCC pavements should be laterally constrained with curbs or shoulders and sufficient control joints should be incorporated in the design and construction to limit and control cracking.

The soil subgrade and aggregate base below the pavement section should be prepared and compacted as recommended below. The use of a moisture cut-off or thickened edge along the edges of the driveway would be desirable in order to reduce water seepage below the edges of the driveway and into the underlying aggregate base and subgrade, which can lead to premature pavement distress.

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as existing pavements, existing foundations, 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 and test during the scarification and compaction of the basement excavation subgrade.

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.

Temporary Slopes, Excavations and/or Shoring

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, state and federal safety regulations, including the current OSHA excavation and trench safety standards. Due to the limited space between the basement excavation and the surrounding roadways and structures on adjacent properties, unsupported cut slopes may not be feasible during the basement excavation so that shoring or bracing in accordance with OSHA standards will likely be required. This information should be considered by the contractor when establishing temporary shoring/slope criteria for the basement excavation and other temporary slopes and cuts. In addition, protection of the roadways and structures near cuts and excavations should also be the responsibility of the contractor.

Portions of the medium dense sandy silt to silty sand encountered in Boring EB-1 between depths of approximately 9.5 to 12 feet 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/slope criteria for basement excavation, as needed.

Protection of structures and slopes near cuts should also be the responsibility of the contractor. In our experience, a preconstruction survey is generally performed to document existing conditions prior to construction, with intermittent monitoring of the structures during construction. The contractor should be responsible for staging the required cuts and wall construction and the design of temporary cut slopes and/or required shoring.

Finished Slopes

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

Basement Excavation Support

Based on the assumed finished floor elevation of the basement, temporary excavations up to approximately 12 to 14 feet deep (depending on the finished floor elevation and foundation depth) or deeper for elevator pits and/or garage lifts will be required in order to construct the basement. 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. 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.

Temporary Dewatering For Basement Excavation

Ground water was measured at approximately 13.5 feet below surface grade and historic high ground water levels are as high as about 6 feet below grade. Therefore, temporary dewatering may be necessary during construction. Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions, they should be aware that modifications to the dewatering system may be required during construction depending on the conditions encountered.

We recommend that any dewatering of the site be carried out in such a manner as to maintain the ground water a minimum of 3 feet below the bottom of the mass excavation. The contractor should design a system to achieve this criteria. Additionally, the ground water should be maintained at least 2 feet below all local excavations for deepened foundations, utilities or other structures.

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

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
4-Story Hotel Building
San Jose, California

	Relative Compaction*	Moisture Content*
General		
• Scarified subgrade in areas to receive structural fill.	88-93 percent	At least 2 percent above optimum
• Structural fill composed of native soil.	88-93 percent	At least 2 percent above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
• Structural fill below a depth of 5 feet.	92 percent	At least 2 percent above optimum
Pavement Areas		
• Upper 6-inches of soil below aggregate base.	95 percent	2 percent above optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soil.	88 to 93 percent	At least 2 percent above optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Surface Drainage

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

Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during the first two years following construction. We recommend preparing an as-built plan showing the locations of surface and subsurface drain lines and clean-outs. The drainage facilities should be periodically checked to verify that they are continuing to function properly. It is likely the drainage facilities will need to be periodically cleaned of silt 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, slab subgrade and non-expansive fill preparation, foundation and slab construction, subslab drainage system installation, retaining wall drainage and backfilling, utility trench backfilling, tieback/soil nail installation and testing, shoring pier installation, pavement subgrade and aggregate base construction and site drainage should be performed as recommended in the geotechnical report, dated May 17, 2016, prepared by Romig Engineers, Inc. Romig Engineers should be notified at least 48 hours in advance of any earthwork and 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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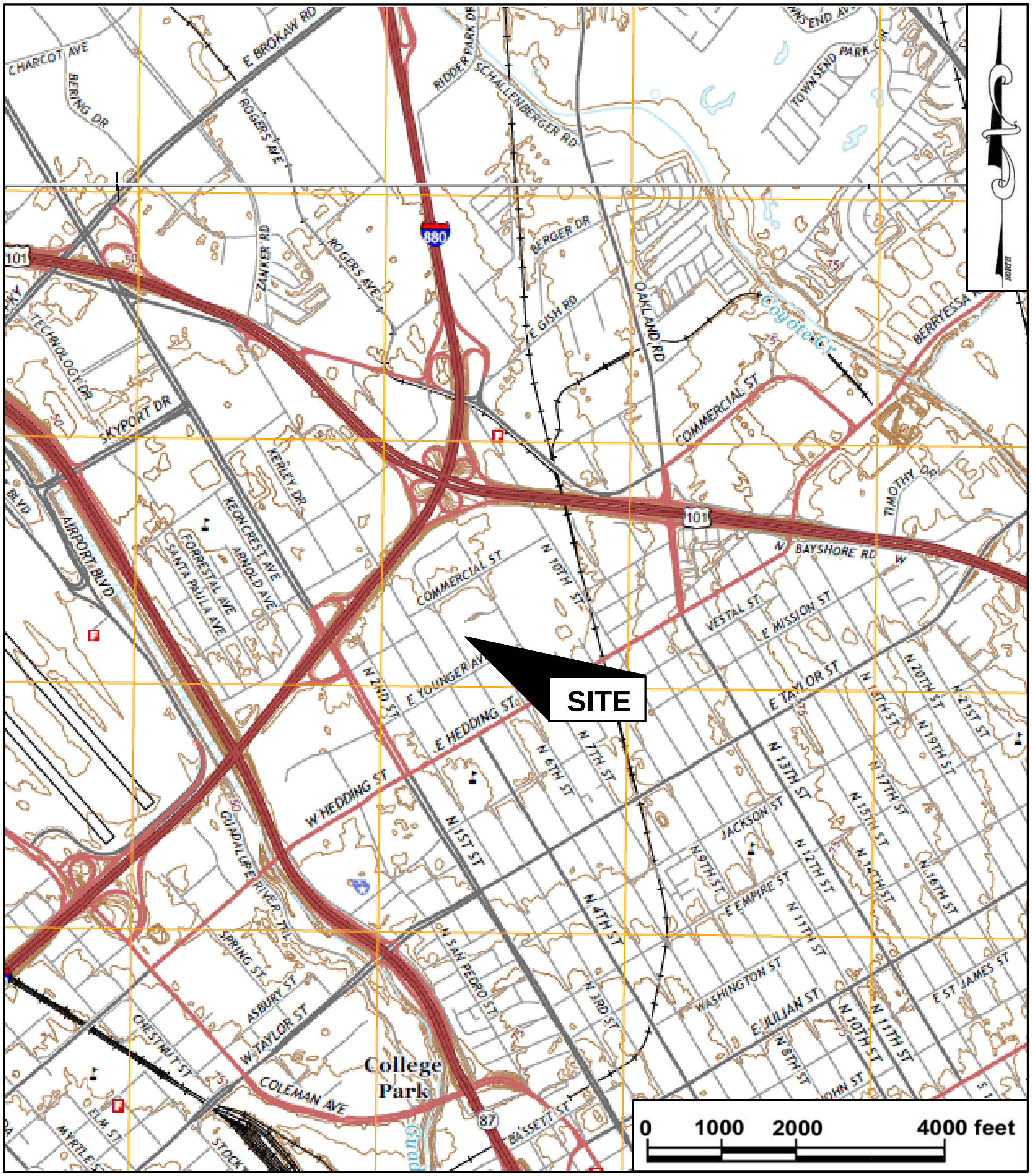
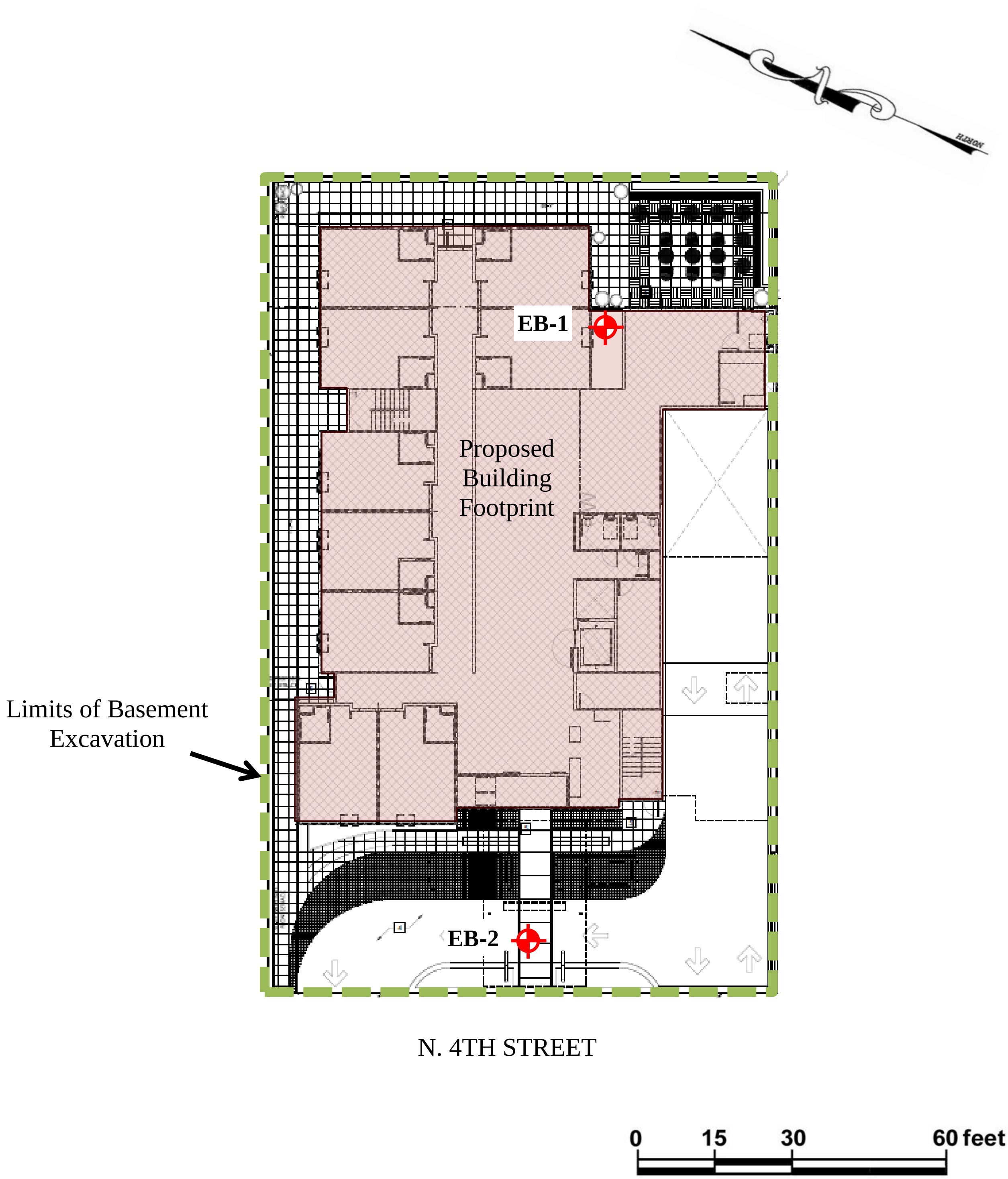


Scale: 1 inch = 2000 feet
Base is United States Geological Milpitas and San Jose West 7.5 Minute Quadrangle, dated 2015.

VICINITY MAP
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

FIGURE 1
MAY 2016
PROJECT NO. 3745-1A



LEGEND

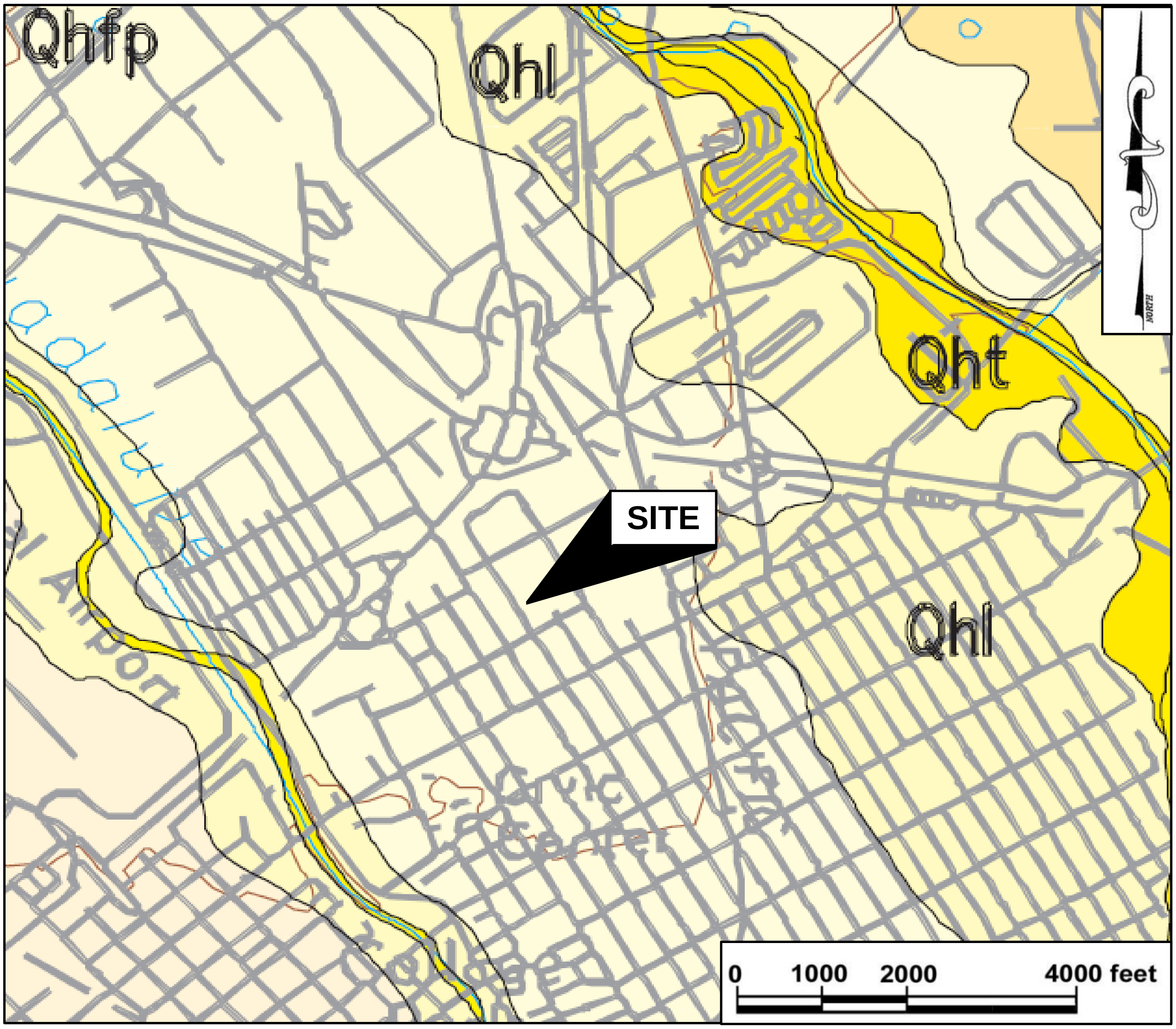
EB-2 Approximate Location of Exploratory Borings.

Approximate Scale: 1 inch = 30 feet.

Base is site plan prepared by Studio S Squared, dated March 18, 2016.

SITE PLAN
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

FIGURE 2
MAY 2016
PROJECT NO. 3745-1A



Legend:

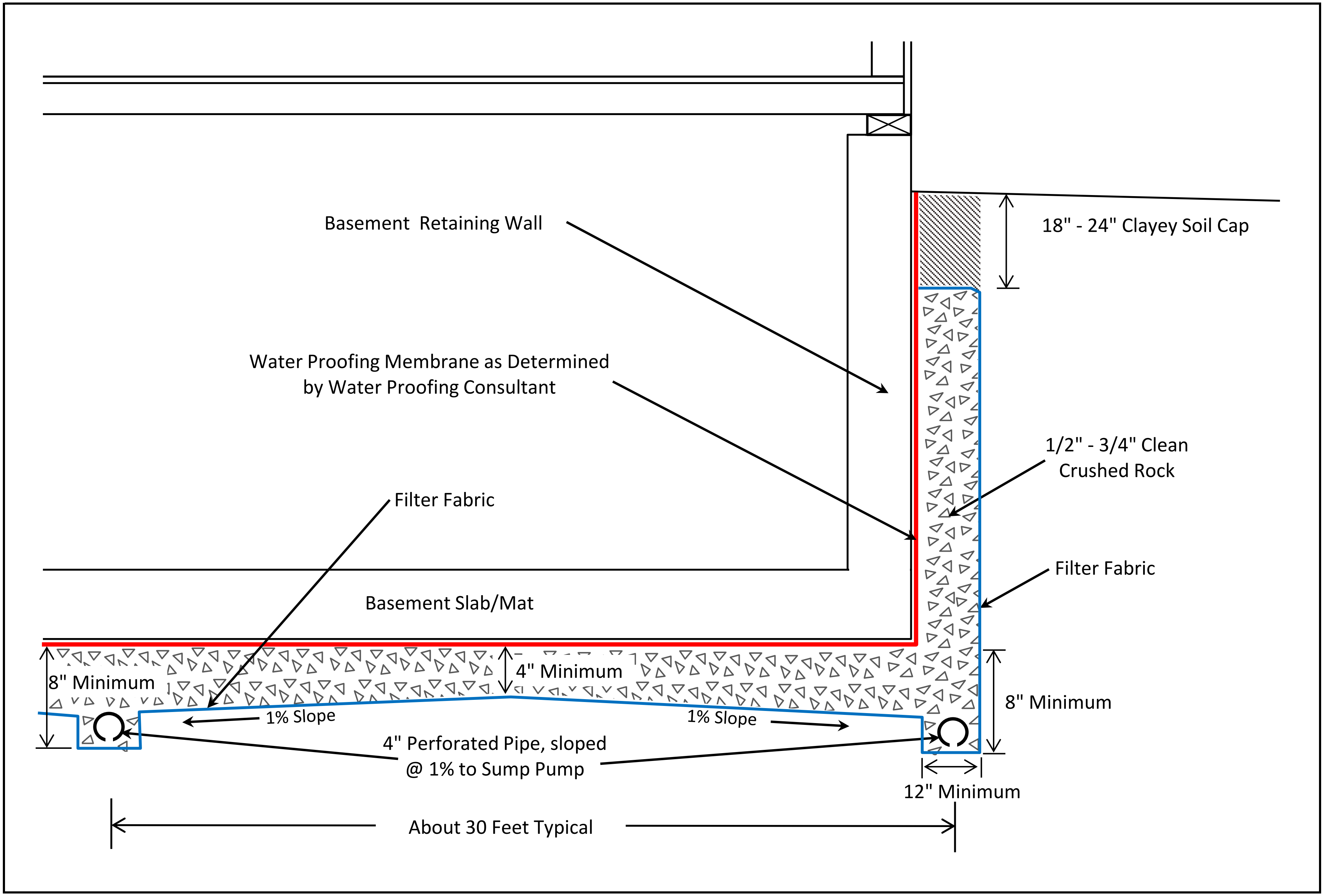
Qhfp	Flood plain deposits
Qhl	Levee deposits
Qht	Stream terrace deposits

Contact: dashed where approximately located, dotted where concealed

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

VICINITY GEOLOGIC MAP
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

FIGURE 3
MAY 2016
PROJECT NO. 3745-1A



SUBSLAB DRAINAGE DETAIL
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

FIGURE 4
MAY 2016
PROJECT NO. 3745-1A

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 prepared by Studio S Squared Architecture, dated March 18, 2016 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 **FIGURE A-1**

PATEL 4-STORY HOTEL BUIDLING MAY 2016

SAN JOSE, CALIFORNIA PROJECT NO. 3745-1A

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 13.5 Feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 04/29/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
3 inches asphaltic concrete, 2 inches clay and gravel.		AC		0					
Dark Brown, Lean Clay, moist, fine grained sand, trace fine subrounded gravel, moderate plasticity.	Stiff to Very Stiff	CL							
■ Liquid Limit = 41, Plasticity Index = 17.					■	13	24	0.8	3.0
				5		29	25	1.1	4.5
						18	24	0.9	3.0
● 81% Passing No. 200 Sieve.					●		20		
Gray, Silty Sand to Sandy Silt, very moist, low plasticity fines.	Stiff	SM/ ML		10	●	12	23	0.5	3.3
● 50% Passing No. 200 Sieve.									
▼ Ground water encountered during drilling at 13.5 feet.				▼					
Grayish brown, Fat Clay, very moist, trace coarse grained sand, high plasticity. ● 96% Passing No. 200 Sieve.	Stiff to Very Stiff	CH		15	●	13	29	0.8	1.8
				20		24	30	0.9	1.3
Continued on next page.									

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 13.5 Feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 02/01/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Grayish brown, Fat Clay, very moist, trace coarse grained sand, high plasticity.	Stiff to Very Stiff	CH		**20**					
■ Liquid Limit = 55				**25**		10	27	0.6	1.8
				30		12			
						17	44		
■ Liquid Limit = 50				**35**		11	28	0.6	1.5
Decrease in plasticity.				**40**		22	31	0.5	2.0
Continued on next page.									

EXPLORATORY BORING LOG EB-1
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

BORING EB-1
PAGE 2 OF 3
MAY 2016
PROJECT NO. 3745-1A

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: 13.5 Feet **SURFACE ELEVATION:** N/A **DATE DRILLED:** 02/01/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Grayish brown, Fat Clay, very moist, trace coarse grained sand, high plasticity.	Stiff to Very Stiff	CH		40					
Brown, Gray, Lean Clay, moist, low to moderate plasticity, brown to orange mottling. ■ Liquid Limit = 31	Very Stiff	CL		45		18	24	0.4	1.0
				50		28	29	0.4	1.0
Bottom of Boring at 50 feet.				55					
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual. *Measured using Torvane and Pocket Penetrometer devices.				60					

EXPLORATORY BORING LOG EB-1
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

BORING EB-1

MAY 2016
PROJECT NO. 3745-1A

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: Not Measured. **SURFACE ELEVATION:** N/A **DATE DRILLED:** 04/29/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
2 inches asphaltic concrete, 4 inches clay and gravel.		AC		0					
Dark Brown to brown, Lean Clay, moist, fine grained sand, moderate plasticity.	Stiff to Very Stiff	CL				11	26	0.5	1.5
						14	26	1.9	1.7
				5					
Brown to gray, some orange to brown mottling.						30	23	0.8	3.0
Increased orange to brown mottling, increased fine grained sand.				10		11	20		3.0
Grayish brown, Fat Clay, very moist, trace coarse grained sand, high plasticity. ■ Liquid Limit = 52	Stiff	CH		15		14	28	0.8	2.0
				20		14	30		1.5
Continued on next page.									

EXPLORATORY BORING LOG EB-2
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

BORING EB-2

MAY 2016

DRILL TYPE: Mobile Drill B-40 with 8" Hollow Stem Auger **LOGGED BY:** CT

DEPTH TO GROUND WATER: Not Measured. **SURFACE ELEVATION:** N/A **DATE DRILLED:** 04/29/2016

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Grayish brown, Fat Clay, very moist, trace coarse grained sand, high plasticity.	Stiff	CH		20					
				25		10	34	0.4	0.5
				30		10	41		0.5
				35		10	38	0.9	1.3
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
PATEL 4-STORY HOTEL BUIDLING
SAN JOSE, CALIFORNIA

BORING EB-2
PAGE 2 OF 2
MAY 2016
PROJECT NO. 3745-1A

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 in accordance with ASTM D4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The result of this test is 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 three samples of soil in accordance with ASTM D422. The results of these tests are presented on Boring EB-1 at the appropriate sample depths.

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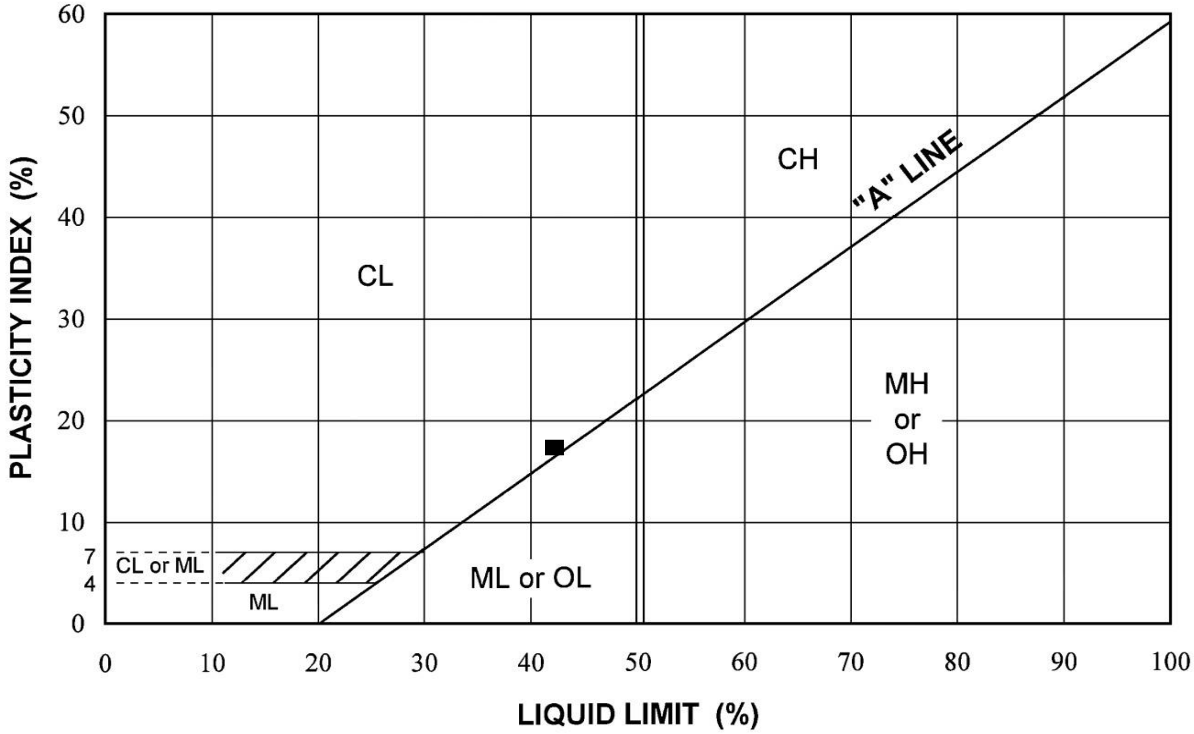

Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	1-2.5	24	41	17	0		CL
-	EB-1	23.5-25	27	55				CH
-	EB-1	33.5-35	28	50				CH
-	EB-1	43.5-45	24	31				CL
-	EB-2	13.5-15	28	52				CH

PLASTICITY CHART **FIGURE B-1**
PATEL 4-STORY HOTEL BUIDLING MAY 2016
SAN JOSE, CALIFORNIA PROJECT NO. 3745-1A