January 19, 2018
4257-1

Temple San Jose LLC
527 Simas Drive
Milpitas, California 95035

RE: GEOTECHNICAL INVESTIGATION
RESIDENCE INN
459 PIERCY ROAD
SAN JOSE, CALIFORNIA

Attention: Mr. Bob Desai

Gentlemen:

As requested, we have performed a geotechnical investigation for the proposed Residence Inn to be constructed at 459 Piercy Road 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, P.E.

Glenn A. Romig, P.E., G.E.

Copies: Addressee (4)
Temple Hospitality South San Jose LLC (via email)
Attn: Mr. Vijay Meher

GAR:TWP:dr

GEOTECHNICAL INVESTIGATION
RESIDENCE INN
459 PIERCY ROAD
SAN JOSE, CALIFORNIA 95138

PREPARED FOR:
TEMPLE SAN JOSE LLC
527 SIMAS DRIVE
MILPITAS, CALIFORNIA 95035

PREPARED BY:
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JANUARY 2018

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GEOTECHNICAL INVESTIGATION
FOR
RESIDENCE INN
459 PIERCY ROAD
SAN JOSE, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed 112 room Residence Inn to be constructed at 459 Piercy Road 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 five-story hotel building at the approximately 2-acre site in San Jose. The building will be located along the northeast (rear) side of the property. A swimming pool and outdoor courtyard are planned at a courtyard located along the southwest side of the building. Paved parking and drive aisles will located between the hotel and Hellyer Avenue. The site is currently a relatively flat undeveloped lot. Structural loads are expected to be moderate to high as is typical for this type of construction.

Scope of Work

Our scope of work for this investigation was presented in our agreement with Temple San Jose LLC dated November 1, 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.
- 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 Temple San Jose LLC for specific application to developing geotechnical design criteria for the proposed Residence Inn to be constructed at 459 Piercy Road 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 EXPLORATION AND RECONNAISSANCE

Site reconnaissance and subsurface exploration were performed on December 20, 2017. Subsurface exploration was performed using a Mobile B-61 truck-mounted drill equipped with 8-inch diameter hollow-stem augers. Three exploratory borings were advanced to depths ranging between 35 to 49.9 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 area at the northeast side of Hellyer Avenue. At the time of our investigation, the site was a relatively flat, undeveloped lot partially surrounded by wood fencing. The northwest portion of the site appeared to be underlain by several feet of fill possibly placed during grading work during development of the adjacent site. The site was vegetated with native grass and a few small shrubs.

Subsurface Conditions

At the location of Borings EB-1 and EB-2, we encountered approximately 2 to 2.5 feet of fill which consisted of very stiff to hard sandy lean clay of low plasticity. These surface soils appeared to be a possible surface fill or disturbed native soil. The surface soils were underlain by stiff to hard sandy lean clay of low to high plasticity which extended to the maximum depths explored of 44.9 and 49.9 feet.

In Boring EB-3, we encountered approximately 23 feet of stiff to hard sandy lean clay of low to moderate plasticity underlain by medium dense to very dense poorly graded sand which extended to the maximum depth explored of 35 feet.

A Liquid Limit of 49 and Plasticity Index of 25 were measured on a sample of native soil obtained from Boring EB-1. These test results indicate that at least portions of the near surface native soils have moderate to high plasticity and a moderate to high potential for expansion.

Ground Water

Ground water was measured at a depth of about 42 feet in Boring EB-1, at a depth of about 24 feet in Boring EB-2, and at a depth of about 23 feet in Boring EB-3, shortly after drilling and sampling was completed. The borings were backfilled with grout shortly after drilling, therefore a stabilized ground water level may not have been obtained. Information presented in Seismic Hazard Zone Report 044 for the San Jose East Quadrangle (California Geological Survey, 2000) indicates the historical high ground water level in the area of the site is expected to be present at an average depth of approximately 20 to 30 feet below the ground surface. Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, landscaping, surface and subsurface drainage patterns, and other factors.

GEOLOGIC SETTING

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. The information reviewed indicates that the site is underlain by Holocene age older alluvial fan deposits, Qhf2 (Blake, Graymer, McLaughlin and Wentworth, 1999). The unit is generally described as brown or tan, medium dense gravelly sand or sandy gravel that transitions upward to sandy or silty clay. The geology within the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

The property and the immediate vicinity are located in an area that slopes very gently toward the southwest (approximately 10 feet vertically per 3,000 feet laterally, although locally the topography may be steeper). The site is located at an elevation of approximately 200 feet above sea level.

The Geologic Hazard Zone Map (2012) prepared by the County of Santa Clara and the State Seismic Hazard Zones Map of the San Jose East Quadrangle (California Geological Survey, 2001) indicates the site is located in an area that may be underlain by soils that have the potential to liquefy during a major earthquake. The potential for liquefaction of the soils encountered at the site is discussed later in this report.

Faulting and Seismicity

The County Hazard map indicates the site is located in a fault rupture hazard zone possibly related to the Silver Creek fault located to the northeast. The City of San Jose map (1983) indicates a splay fault, shown as the Evergreen fault splay mapped immediately to the northeast of the site. However, we understand that recent fault trenching work conducted between the site and the mapped faults did not encounter traces of faulting and that the City has indicated that a fault study is not required for project approval.

The site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the Hayward fault, located approximately 3.6 miles northeast of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is low.

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 Hayward, Calaveras, San Andreas, and San Gregorio faults. The San Andreas and Calaveras faults are located approximately 12 miles southwest and 13 miles northeast of the site, respectively. The San Gregorio fault is located approximately 30 miles southwest of the site. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 1 and are shown on the Regional Fault and Seismicity Map, Figure 4.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Residence 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 Response Acceleration parameters and site coefficients may be taken directly from the U.S.G.S. website based on the longitude and latitude of the site. For site latitude (37.2601), longitude (-121.7819) and Site Class D, design parameters are presented on Table 2.

Table 2. 2016 CBC Seismic Design Criteria
Residence Inn
San Jose, California

Spectral Response Acceleration Parameters			Design Value
Mapped Value for Short Period	-	S_S	1.50
Mapped Value for 1-sec Period	-	S_1	0.60
Site Coefficient	-	F_a	1.0
Site Coefficient	-	F_v	1.5
Adjusted for Site Class	-	S_{MS}	1.500
Adjusted for Site Class	-	S_{M1}	0.90
Value for Design Earthquake	-	S_{DS}	1.00
Value for Design Earthquake	-	S_{D1}	0.60

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.

The clayey sand encountered at the site below the highest projected ground water depth, which is estimated to be about 23 feet below the ground surface, was considered in our liquefaction analysis. Soils with normalized standard penetration test, $(N_1)_{60}$, greater than 30 blows per feet were considered too dense to liquefy.

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

Potentially liquefiable soils were encountered in Boring EB-3 between depths of approximately 27 to 32 feet. These clayey sands and gravelly sands are potentially prone to liquefaction when subjected to the maximum considered earthquake acceleration (PGA_M) of 0.50g based on the Probabilistic Seismic Hazards Mapping Ground Motion Page (CGS, 2016). Based on the results of our analysis of these sand and gravel layers, we estimate that total and differential settlement of less than ¼-inch could occur within these sand strata due to severe ground shaking caused by a major earthquake. This seismic settlement would not be expected to significantly affect the proposed building designed and constructed in accordance with the recommendations presented in this report.

Geologic Hazards

As part of our investigation, we reviewed the potential for geologic hazards, other than liquefaction which was discussed above, 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.

- 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 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 Residence 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.

The primary geotechnical concerns for the proposed improvements are the presence of up to 3 feet of existing fill material encountered along the northwest portion of the site and the potential for variable support conditions across the building foundation depending on the final grading scheme. We understand that it is unknown if site grades will be raised and a final grading plan for the building pad has not been developed at this time. Depending on the proposed pad elevation, the building foundation may potentially bear on stiff to hard native soils and/or on areas of existing fill.

In our opinion, the proposed building may be supported on mat or conventional spread footing foundation bearing in stiff native soils below any existing fill, or on a properly compacted structural fill pad across the entire building footprint. Once a grading plan has been developed, we should be contacted to verify if supplemental recommendations are required.

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.

In our opinion, any existing fill not removed during grading for the building pad should be excavated and recompacted below the building, exterior flatwork, pavements, and any other site improvements during site preparation. The reworking of the fill and subgrade preparation should proceed as recommended in the section of this report titled "Earthwork." If documentation regarding the compaction of the existing fill can be obtained, it may be possible to utilize some of the existing fill, provided that the fill was compacted to current engineering standards.

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

Spread Footing Foundations

In our opinion, the building and other minor site improvements such as privacy and trash enclosure walls may be supported on a conventional spread footing foundation system bearing on stiff native soils or properly compacted structural fill. All continuous footings should have a width of at least 15 inches and should extend at least 30 inches below exterior grade and at least 24 inches below the bottom of concrete slabs-on-grade, whichever is deeper. On a preliminary basis, continuous footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 3,000 pounds per square foot for dead loads, 4,000 pounds per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading.

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

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

Structural Mat Foundation

As an alternative to the spread footing foundation, the building may be supported on a reinforced concrete mat foundation bearing on non-expansive fill or a properly prepared and compacted native soil subgrade. On a preliminary basis, the mat may be designed for an average allowable bearing pressure of 3,000 pounds per square foot for combined dead plus live loads, with maximum localized bearing pressures of 4,000 pounds per square foot at column or wall loads. These pressures may be increased by one-third when considering additional short-term wind or seismic loading. The weight of the mat may be neglected in design. The mat foundation should be designed with a thickened perimeter edge. The thickened perimeter edge should have a width of at least 12 inches, should extend at least 24 inches below exterior grade, and at least 12 inches below the bottom of the mat, whichever is deeper.

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 15 to 35 pci). The mat should also be designed with sufficient depth and reinforcing to span over localized weak compressible or expansive fill areas.

In our opinion, the mat should be underlain by a layer of non-expansive fill at least 10-inches thick that is placed and compacted on a properly prepared and compacted soil subgrade as discussed in the "Slabs-On-Grade" section below. Where a capillary barrier system is placed below the mat, the crushed rock may be considered at the upper portion of the non expansive fill recommended above.

Prior to mat construction, the mat subgrade should be proof-rolled to provide a smooth firm surface for mat support. In areas where floor dampness is not desired, a capillary barrier system should be installed below the mat in accordance with the slab-on-grade recommendations in this report, or other waterproofing measures should be taken as appropriate considering the floor surface finishes planned. Non-expansive fill is not needed below the mat foundation as long the mat subgrade surface is properly scarified and moisture conditioned prior to mat construction.

Lateral Loads

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

Lateral loads may be resisted by friction between the bottom of the footings and the supporting subgrade. A coefficient of friction of 0.30 may be assumed for footing design. In addition, lateral resistance may be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations or backfilled with compacted structural fill. We recommend assuming an equivalent fluid pressure of 350 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing or mat will be landscaped or subject to softening from rainfall and/or surface water runoff.

Settlement

On a preliminary basis, the 30-year post-construction differential movement due to static loads is not expected to exceed about 1-inch across the proposed building, provided the building foundations are designed and constructed as recommended. 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 movement.

SLABS-ON-GRADE

General Slab Considerations

At least portions of the near surface soils at the site have a moderate to high expansion potential. Expansive soils have a tendency to expand due to increases in moisture content and shrink as they dry. This can result in some slab cracking and heave regardless of the geotechnical measures implemented.

Our recommendations below will help mitigate the impacts of the expansive soils beneath slabs-on-grade, but will not eliminate the risk entirely. To reduce the potential for movement of the slab subgrade, at least the upper 6-inches of expansive soil should be scarified and compacted at a moisture content at least 3 percent above the laboratory optimum. The native soil subgrade should be kept moist up until the time the non-expansive fill, crushed rock and vapor barrier, and/or aggregate base is placed. Slab subgrades and non expansive fill should be prepared and compacted as recommended in the section of this report titled “Earthwork.” Exterior flatwork and interior slabs-on-grade should be underlain by a layer of non expansive fill as discussed below. The non expansive fill should consist of aggregate base rock or a clayey soil with a plasticity index of 15 or less.

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

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 10 inches of Class 2 aggregate base. The potential for distress to exterior slabs due to expansive soil movements could be reduced by placing and compacting 4 inches of non-expansive fill, or aggregate base, below the minimum 10-inch thick layer of aggregate base recommended above. To improve performance, exterior slabs-on-grade may be constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs.

Interior Slabs

Concrete slab-on-grade floors for the building (other than a structural mat slab) should be constructed on a layer of non-expansive fill at least 14-inches thick and constructed on a properly prepared and compacted soil subgrade.

Structural Mat

Due to the moderately to highly expansive soils, if the building is supported on a structural mat bearing on native soils, the mat should be underlain by a layer of non-expansive fill at least 10-inches thick that is placed and compacted on a properly prepared and compacted soil subgrade. If the building will be underlain by a compacted structural fill pad of non-expansive material, the non-expansive fill section recommended above may be reduced or eliminated. Once a grading plan has been developed, we can provide additional input regarding the mat construction.

Moisture Considerations

In areas where dampness of concrete floor slabs or mat would be undesirable, such as within building interiors, concrete slabs and mat should be underlain by at least 4 inches of clean, free-draining gravel, such as ½-inch to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used. The crushed rock should be compacted with vibratory equipment and may be considered at the upper portion of the non expansive fill recommended above.

To reduce vapor transmission up through at-grade concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor retarder meeting the minimum ASTM E 1745, Class C requirements or better. If moisture-sensitive floor coverings are proposed and/or additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms (such as 15-mil thick “Stego Wrap Class A”) may be used rather than a Class C vapor retarder. The vapor retarder or barrier should be placed directly below the concrete slab. Sand above the vapor retarder/barrier is not recommended. The vapor retarder/barrier should be installed in accordance with ASTM E 1643. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer’s recommendations.

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

RETAINING WALLS

Retaining walls should be designed to resist lateral pressures from the adjacent native and fill soils and backfill. We recommend retaining walls with level backfill that are not free to deflect or rotate, 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. Retaining walls with level backfill that are free to rotate may be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot. Retaining walls with backfill that slopes at about 2:1 (horizontal:vertical) should be designed to resist an equivalent fluid pressure of 65 pounds per cubic foot for walls free to rotate, with 8H added as recommended above for walls not free to rotate. Wherever walls will be subjected to surcharge loads, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge load for restrained walls and one-third of the surcharge load for unrestrained walls.

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 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 or 65 pounds per cubic foot).

To prevent buildup of water pressure from surface water infiltration, a subsurface drainage system should be installed behind the walls. 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 free-draining outlet or 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 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 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. The backfill behind the walls should be placed on level benches, rather than directly on the sloping grade.

Site retaining walls may be supported on a conventional spread footing foundation as presented previously.

SWIMMING POOL

In our opinion, the swimming pool walls should be designed to resist a lateral equivalent fluid pressure of 65 pounds per cubic foot. The pool walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge pressure applied at the surface, such as from foundations. In addition, a pressure relief valve(s) should be placed in the bottom of the pool to limit damage from hydrostatic pressure that may develop when the pool is emptied for maintenance.

To allow ground water to flow to the pressure relief valve(s), 4-inches of clean, 1/2- to 3/4-inch crushed rock should be placed beneath the pool. If installed, filter fabric should be used to separate the crushed rock from the subgrade soils. If desired, drainage pipes could be provided from the gravel to a sump that could pump temporarily when the pool is empty or to daylight. If the crushed rock section is not placed below the pool, the pool bottom may need to be perforated at several locations as a buoyancy prevention measure when the pool is emptied for maintenance.

In our opinion, due to the potential for differential movement from the expansive soil and possible variable support conditions, the swimming pool should be designed with additional stiffening elements incorporated within the pool bottom and walls to improve performance should some differential movement occur. Additional steel reinforcement and thickness within the pool bottom, coping and walls are typically used as pool stiffening elements. In addition, the steel reinforcement and thickness should be uniform across the pool shell, specifically between the shallow and deep ends of the pool.

Proper surface drainage should be provided about the pool decks to divert water to catch basins and other inlets for water to be carried away in closed drainpipes. Also, flexible bituminous caulking or equivalent should be applied at the juncture of the pool and decks to limit infiltration of surface water into the native soils. Recommendations for swimming pool deck construction are presented in the "Slabs-on-Grade" section above.

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 3 below based on Procedure 630 of the Caltrans Highway Design Manual.

Table 3. Pavement Sections
Residence 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	7.0	10.0
Automobile Access	4.5	3.0	8.0	11.0
Light Truck Traffic	5.0	3.0	9.0	12.0
Moderate Truck Traffic	6.0	4.0	11.0	15.0
Heavy Truck Traffic	7.0	4.0	14.0	18.0

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

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

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

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 8 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, swimming pool, and utility trench excavations, and slab and pavement subgrades should be kept in a moist condition throughout the construction period to help mitigate the potential effects of the expansive on-site soils on the proposed surface improvements.

Building Pad Recommendations

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

Material For Fill

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

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 4 on the following page. The relative compaction and moisture content recommended in Table 4 is relative to ASTM Test D1557, latest edition.

Table 4. Compaction Recommendations
Residence Inn
San Jose, California

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

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.

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

Finished Slopes

We recommend that finished slopes be cut or filled to an inclination no steeper than 2:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion, which could require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted with erosion-resistant vegetation.

Surface Drainage

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

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

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

FUTURE SERVICES

Plan Review

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations presented in this report. We should be provided with these plans as soon as possible upon their completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review. 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, mat and/or slab subgrade preparation, swimming pool construction, utility trench backfill, pavement construction, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated January 19, 2018. 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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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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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 two samples of soil in accordance with ASTM D422. The results are presented on the boring logs at the appropriate sample depths.

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