REPORT TO

INFINITE INVESTMENT REALTY CORP
SAN JOSE, CALIFORNIA

FOR

PROPOSED HOTEL
615 & 623 STOCKTON AVENUE
SAN JOSE, CALIFORNIA

GEOTECHNICAL INVESTIGATION

JULY 2018

PREPARED BY

SILICON VALLEY SOIL ENGINEERING
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File No. SV1795
July 17, 2018

Infinite Investment Realty Corp
1168 Park Avenue
San Jose, CA 95126

Attention: Mr. Alan Nguyen

Subject: Proposed Hotel
615 & 623 Stockton Avenue
San Jose, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Nguyen:

Pursuant to your request, we are pleased to present herein geotechnical investigation for the proposed hotel. The subject site is located at 615 and 623 Stockton Avenue in San Jose, California.

Our findings indicate that the site is suitable for the proposed development provided the recommendations contained in this report are carefully followed. Field reconnaissance, drilling, sampling, and laboratory testing of the surface and subsurface material evaluated the suitability of the site. The following report details our investigation, outlines our findings, and presents our conclusions based on those findings.

If you have any questions or require additional information, please feel free to contact our office at your convenience.

Very truly yours,

SILICON VALLEY SOIL ENGINEERING

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TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION — PAGE

LIST OF TABLES, FIGURES, AND APPENDICES
GEOTECHNICAL INVESTIGATION

TABLES

FIGURES

APPENDICES

INTRODUCTION

Per your authorization, Silicon Valley Soil Engineering (SVSE) conducted a geotechnical investigation. The purpose of this geotechnical investigation was to evaluate the nature of the surface and subsurface soil conditions at the subject site through field investigations and laboratory testing. This report presents an explanation of our investigative procedures, results of the testing program, our conclusions, and our recommendations for earthwork and foundation design to adapt the proposed development to the existing soil conditions.

SITE LOCATION AND DESCRIPTION

The subject site is located at 615 and 623 Stockton Avenue (APN 261-07-001 and 261-07-068) in San Jose, California (Figure 1). Stockton Avenue bounds the subject site to the northeast, Schiele Avenue to the southeast, residential development to the southwest, and an automotive shop to the northwest. At the time of this investigation, the subject site is an irregular, relatively flat parcels of land occupied by a one-story retail building and residential construction office with a paved parking lot at the eastern portion of the property. Based on the preliminary plans for the subject site, the proposed development will include the demolition of the existing structures and construction of a hotel building with a two-level underground basement parking garage and associated improvements. The approximate location of the proposed structure and our borings are shown on the Site Plan (Figure 2).

FIELD INVESTIGATION

After considering the nature of the proposed development and reviewing available data on the area, our geotechnical engineer conducted a field investigation at the subject site. It included a site reconnaissance to detect any unusual surface features, and the drilling of three exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on July 12,

2018. The approximate location of the borings is shown on the Site Plan (Figure 2). The borings were drilled to the depths of 10 feet to 60 feet below the existing ground surface. The borings were drilled with a truck mounted drill rig using 8-inch diameter hollow stem augers.

The soils encountered were logged continuously in the field during the drilling operation. Relatively undisturbed soil samples were obtained by hammering a 2.0-inch outside diameter (O.D.) split-tube sampler for a Standard Penetration Test (SPT), ASTM Standard D1586, into the ground at various depths. A 2.5-inch diameter split-tube sampler (Modified California) sampler was utilized to obtain soil sample for direct shear tests at the depths of 1.5 feet to 3 feet. A 140-pound hammer with a free fall of 30 inches was used to drive the sampler 18 inches into the ground. Blow counts were recorded on each 6-inch increment of the sampled interval. The blows required to advance the sampler the last 12 inches of the 18 inch sampled interval were recorded on the boring logs as penetration resistance. These values were also used to evaluate the liquefaction potential of the subsurface soils.

In addition, one disturbed bulk sample of the near-surface soil was collected for laboratory analyses. The Exploratory Boring Log, a graphic representation of the encountered soil profile which also shows the depths at which the relatively undisturbed soil samples were obtained, can be found in the Appendix at the end of this report.

LABORATORY INVESTIGATION

A laboratory-testing program was performed to determine the physical and engineering properties of the soils underlying the site.

1. Moisture content and dry density tests were performed on the relatively undisturbed soil samples in order to determine soil consistency and the moisture variation throughout the explored soil profile (Table I).

2. Atterberg Limits tests were performed on the sub-surface soil to assist in the classification of these soils and to obtain an evaluation of their expansion and shrinkage potential and liquefaction analysis (Figure 4 & Table I).

3. The strength parameters of the foundation soils were determined from direct shear tests that were performed on selected relatively undisturbed soil samples (Table I).

4. Laboratory compaction tests were performed on the near-surface material per the ASTM D1557 test procedure (Figure 5).

5. One R-Value test was performed on a near surface soil sample for pavement section design recommendations (Figure 6).

6. Two soil samples collected were submitted to Cooper Testing Lab for corrosivity analysis (Appendix).

The results of the laboratory-testing program are presented in the Tables and Figures at the end of this report.

SOIL CONDITIONS

In Boring B-1 (60 feet boring), the existing pavement surface consists of 2.5 inches of asphalt concrete (AC) over 6.0 inches of aggregate base (AB). Below the pavement surface to a depth of 3 feet, a black, moist, very stiff silty clay layer was encountered. A color change of medium gray was noted at a depth of 2 feet. From the depths of 3 feet to 10 feet, the soil became light olive brown, moist, very stiff sandy silt. From the depths of 10 feet to 32 feet, a medium brown, moist, stiff silty clay layer was encountered. A color change of medium olive brown was noted at a depth of 21 feet. From the depths of 32 feet to 38 feet, the soil became greenish gray, moist, dense sand. The sand was medium grained and poorly graded. A color change of tan brown was noted at a depth of 35 feet. From the depths of 38 feet to 42 feet, the soil became gray, wet, dense gravel. The gravel was 1.5 inches maximum diameter, sub-rounded and poorly graded. From the depths of 42 feet to 51 feet, the soil became tan brown, moist, dense sand. The sand was medium grained. From the depths of 51 feet to the end of the boring at 60 feet, the soil became bluish gray, moist, very stiff silty clay. Similar soil profiles were encountered in other borings.

Groundwater was initially encountered in Boring B-1 at the depth of 25 feet and rose a static level of 15 feet at the end of the drilling operation. It should be noted that the groundwater level would fluctuate as a result of seasonal changes and hydrogeological variations such as groundwater pumping and/or recharging. A graphic description of the explored soil profiles is presented in the Exploratory Boring Log contained in the Appendix.

GENERAL GEOLOGY

The site lies in the San Francisco Bay Region, which is part of the Coast Range province. The regional structure is dominated by the northwest trending Santa Cruz Mountains to the southwest and the Diablo Range to the northeast.

The Quaternary history of the region is recorded by sedimentary marine strata alternating with non-marine strata. The changes of the depositional environment are related to the fluctuation of sea level corresponding to the glacial and interglacial periods.

Late Quaternary deposits fill the center of the San Francisco Bay Region and most of the strata are of continental origin characterized as alluvial and fluvial materials. The project site is underlain by young alluvial fan deposits (Helley and Brabb, 1971, Rogers & Williams, 1974).

LIQUEFACTION ANALYSIS:

The site is located within the State of California Seismic Hazard Zone for liquefaction (CGS, 2001). Therefore, liquefaction analysis was performed.

A. GROUNDWATER

Groundwater was initially encountered in Boring B-1 at the depth of 25 feet and rose a static level of 15 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 058 (revised) [*Seismic Hazard Evaluation of the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California. 2002 (Updated 10/10/05).* Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 10 feet below ground elevation. Therefore, this depth of the groundwater table will be used for the liquefaction analysis.

B. SUSPECTED LIQUEFIABLE SOIL LAYERS

The site is located within the State of California Seismic Hazard Zone for liquefaction (CGS, 2001). The State Guidelines (CGS Special Publication 117A, revised 2008, Southern California Earthquake Center, 1999) were followed by this study. Based on recent studies (Bray and Sancio, 2006, Boulanger and Idriss, 2004), the "Chinese Criteria", previously used as the liquefaction screening (CGS SP 117, SCEC, 1999) is no longer valid indicator of liquefaction susceptibility. The revised screening criteria clearly stated that liquefaction is the transformation of loose saturated silts, sands, and clay with a Plasticity Index (PI) < 12 and moisture content (MC) $> 85\%$ of the liquid limits are susceptible to liquefaction and $12<PI<18$ and $MC>80\%$ of LL are moderately susceptible to liquefaction. This occurs under vibratory conditions such as those induced by a seismic event. To help evaluate liquefaction potential, samples of potentially liquefiable soil were obtained by hammering the split tube sampler into the ground. The number of blows required driving the sampler the last 12 inches of the 18 inch sampled interval were recorded on the log of test boring. The number of blows was recorded as a Standard Penetration Test (SPT), ASTM Standard D1586-92.

Suspected liquefiable soil layers were screened in Boring B-1 (60 feet deep).

BORING B-1: The results from our exploratory boring show that the subsurface soil material in Boring B-1 to the depth of 60 feet consists of very stiff silty clay to very stiff sandy silt to stiff silty clay to dense sand to dense gravel to dense sand to very stiff silty clay. The following is the determination of the liquefiable soil for each soil layer in Boring B-1.

1. The very stiff silty clay layer from the surface to the depth of 3.0 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).

2. The very stiff sandy silt layer from the depths of 3.0 feet to 10.0 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).

3. The stiff silty clay layer from the depths of 10.0 feet to 32.0 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

   - Sample No. 1-4 (15 feet) - [PI > 18; PI = 25 and MC = 36.6% < 80% LL = 37.6%; LL = 47]

   - Sample No. 1-6 (25 feet) - [PI > 18; PI = 23 and MC = 30.4% < 80% LL = 36.0%; LL = 45]

4. The dense sand layer from the depths of 32.0 feet to 38.0 feet is not liquefiable soil based on high blow counts.

5. The dense gravel layer from the depths of 38.0 feet to 42.0 feet is not liquefiable soil based on high blow counts.

6. The dense sand layer from the depths of 42.0 feet to 51.0 feet is not liquefiable soil based on high blow counts.

7. The very stiff silty clay layer from the depths of 51.0 feet to the end of the boring at 60.0 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

- Sample No. 1-12 (55 feet) - [PI > 18; PI = 22 and MC = 24.8% < 80% LL = 32.0%; LL = 40]
- Sample No. 1-13 (60 feet) - [PI > 18; PI = 21 and MC = 25.6% < 80% LL = 32.8%; LL = 41]

Based on the screening process performed for Boring B-1, there is no suspected liquefiable soil layer.

C. CONCLUSIONS

Because no suspected liquefiable soil layer was found at Boring B-1, the potential of liquefaction at the site is minimal.

INUNDATION POTENTIAL

The subject site is located at 615 and 623 Stockton Avenue in San Jose, California. According to the Limerinos and others, 1973 report, the site is not located in an area that has potential for inundation as the result of a 100-year flood (Limerinos; 1973).

CONCLUSIONS

1. The site covered by this investigation is suitable for the proposed development provided the recommendations set forth in this report are carefully followed.

2. The proposed hotel building with a two-level underground basement garage should be supported on mat slab foundation.

3. Based on the laboratory testing results, the native surface soil at the subject site has been found to have a high expansion potential when subjected to fluctuations in moisture. The subgrade at the basement level has moderate expansion potential. Therefore, we recommend that the concrete slab including basement ramp should be underlain by a minimum of 18 inches non-expansive material including the rock section. The basement slab should be underlain with 12 inches of rock. During the construction of any building pad such as the basement ramp, highly expansive native soil should not be used as non-expansive engineered fill material.

4. Any imported non-expansive fill soils should be free of organic material and hazardous substances. All imported fill material to be used for engineered fill should be environmentally tested prior to be used at the site.

5. The highest expected groundwater table is at the depth of 10 feet below existing ground surface. Therefore, the basement grade should need to be dewatered.

6. The exterior of the proposed structure should be graded to promote proper drainage and diversion of water away from the building foundations.

7. We recommend a reference to our report should be stated in the grading and foundation plans that includes the geotechnical investigation file number and date.

8. On the basis of the engineering reconnaissance and exploratory borings, it is our opinion that trenches that will be excavated to depths less than 5 feet below the existing ground surface will not need shoring. However, for trenches and basement that will be excavated greater than 5 feet in depth, shoring will be required.

9. Specific recommendations are presented in the remainder of this report.

10. All earthwork including grading, backfilling, and shoring installation, foundation excavation and drilling shall be observed and inspected by a representative from Silicon Valley Soil Engineering (SVSE). Contact our office 48 hours prior to the commencement of any earthwork for inspection.

RECOMMENDATIONS:
GRADING

1. The placement of fill and control of any grading operations at the site should be performed in accordance with the recommendations of this report. These recommendations set forth the minimum standards to satisfy other requirements of this report.

2. All existing surface and subsurface structures, if any, that will not be incorporated in the final development shall be removed from the project site prior to any grading operations. These objects should be accurately located on the grading plans to assist the field engineer in establishing proper control over their removal. All utility lines in the new building pad area must be removed prior to any grading at the site.

3. The depressions left by the removal of subsurface structures should be cleaned of all debris, backfilled and compacted with clean, native or engineered fill soil. This backfill must be engineered fill and should be conducted under the supervision of a SVSE representative.

4. All organic surface material and debris shall be stripped prior to any other grading operations, and transported away from all areas that are to receive any surface structures or structural fills. Soil containing organic material may be stockpiled for later use in landscaping areas only.

5. After removing all the subsurface structures and existing gravel section and after stripping the organic material from the soil, the improved area at grade should be scarified by machine to a depth of 12 inches and thoroughly cleaned of vegetation and other deleterious matter.

6. After stripping, scarifying and cleaning operations, the existing surface subgrade soil should be moisture conditioned over 3% optimum moisture, compacted to not less than 90% relative maximum density using ASTM

D1557 procedure over the entire building pad, 5 feet beyond the perimeter of the pad and 3 beyond the edge of the parking area, as permitted.

7. All engineered fill or imported soil including baserock material should be placed in uniform horizontal lifts of not more than 8 inches in un-compacted thickness, and compacted to not less than 90% relative maximum density and 95% for baserock material. Before compaction begins, the fill shall be brought to a water content that will permit proper compaction by either; 1) aerating the material if it is too wet, or 2) spraying the material with water if it is too dry. Each lift shall be thoroughly mixed before compaction to assure a uniform distribution of water content.

8. The basement excavated grade should be moisture conditioned as necessary and compacted to 90% relative maximum density.

9. When fill material includes rocks, nesting of rocks will not be allowed and all voids must be carefully filled by proper compaction. Rocks larger than 4 inches in diameter should not be used for the final 2 feet of building pad.

10. Unstable (yielding) subgrade should be aerated or moisture conditioned as necessary. Yielding isolated area in the subgrade can be stabilized with an excavation of the subgrade to the depth of 12 to 18 inches, lined with stabilization fabric membrane (Mirafi 500X or equivalent) and backfilled with aggregate base.

11. SVSE should be notified at least two days prior to commencement of any grading operations so that our office may coordinate the work in the field with the contractor. All imported borrow must be approved by SVSE before being brought to the site. Import soil must have a plasticity index no greater than 15, an R-Value greater than 25 and environmentally clean (non-hazardous). The import soil should contain at least 30 percent fines

(particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate beneath structure.

12. All grading work shall be observed and approved by a representative from SVSE. The geotechnical engineer shall prepare a final report upon completion of the grading operations.

WATER WELLS

13. Any water wells and/or monitoring wells on the site which are to be abandoned, shall be capped according to the requirements of the Santa Clara Valley Water District. The final elevation of the top of the well casing must be a minimum of 3 feet below the adjacent grade prior to any grading operation.

FOUNDATION DESIGN CRITERIA

14. The basement subgrade has been found to have a moderate expansion potential when subjected to fluctuations in moisture. The proposed building structure with a two-level underground basement should be supported on mat foundation.

15. The mat foundation should have a minimum thickness of 24 inches with thickened edge at 30 inch depth and a contact pressure of 2,000 psf.

    - A value of 150 pci as the soil modulus of subgrade of reaction can be used in the design of the mat foundation.
    - The mat slab should be designed to resist a uniform vertical hydrostatic uplift pressure of 466 psf.
    - The mat slab should be underlain by a minimum of 18 inches of ¾-inch wash crushed rock.

- Mat slab should be waterproofed and protected with mud slab. A waterproof consultant should provide waterproofing recommendations.
- The subgrade soil should be compacted to not less than 90% relative maximum density.
- We estimate that post-construction differential settlement will be less than quarter inch settlement per 50 feet span.

16. The fore-mentioned bearing values are for dead plus live loads and may be increased by one-third for short term seismic and wind loads. The design of the structures and the foundations shall meet local building code requirements.

17. The ¾-inch washed crushed rock (recycled crushed asphalt concrete is not acceptable) should be placed on the finished subgrade pad elevation. The crushed rock should be compacted in-place with vibratory plate. The pad subgrade should be compacted prior to placement of the crushed rock and after installation of any under utility pipes and footing/thickened edge excavation with smooth drum roller and/or heavy vibratory plate equipment.

18. If subgrade unstable, the mat slab should be underlain with 18 inches to 24 of ¾ inch crushed rock over stabilization fabric membrane (Mirafi 500X or equivalent).

19. If portion of the structure would be located at-grade (near existing ground surface), the structure should be supported by conventional spread foundation.

20. Conventional spread foundation should be founded at a minimum depth of 24 inches below finished subgrade pad elevation. The allowable bearing capacity is 2,500 psf for both continuous perimeter and isolated interior spread footings.

21. Based on the laboratory testing results of the near-surface soil, the native soil on the site was found to have a high expansion potential when subjected to fluctuation in moisture. Therefore, the bottom of the footings should be underlain by a minimum of 12 inches of non-expansive fill soil material or control density fill (CDF). The non-expansive fill soil should be compacted to at least 90% relative maximum density

22. The footing bottoms and thickened edges should be compacted with jumping jack prior to rebar and form work placement and inspected.

23. The project structural engineer responsible for the foundation design shall determine the final design of the foundations and reinforcing required. We recommend that the foundation plans be reviewed by our office prior to submitting to the appropriate local agency and/or to construction.

2016 CBC SEISMIC VALUES

24. Chapter 16 of the 2016 California Building Code (CBC) outlines the procedure for seismic design. The site categorization and site coefficients are shown in the following table:

Classification/Coefficient	Design Value
Site Class (ASCE 7-10, Table 20.3-1; 2016 CBC, Section 1613A.3.2)	D
Risk Category	I,II,III
Site Latitude	37.339064° N.
Site Longitude	121.912203° W.
0.2-second Mapped Spectra Acceleration[1], S_S (Section 1613A.3.1)*	1.500g
1-second Mapped Spectra Acceleration[1], S_1 (Section 1613A.3.1)*	0.600g
Short-Period Site Coefficient, F_a Table 1613A.3.3(1)*	1.0
Long-Period Site Coefficient, F_V Table 1613A.3.3(2)*	1.5
0.2-second Period, Maximum considered Earthquake Spectral Response Acceleration, S_{MS} ($S_{MS} = F_aS_S$: Section 1613A.3.3)*	1.500g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration, S_{M1} ($S_{M1} = F_VS_1$: Section 1613A.3.3)*	0.900g
0.2-second Period, Designed Spectra Acceleration, S_{DS} ($S_{DS} = 2/3S_{MS}$: Section 1613A.3.4)*	1.000g
1-second Period, Designed Spectra Acceleration, S_{D1} ($S_{D1} = 2/3S_{M1}$: Section 1613A.3.4)*	0.600g

[1] For Site Class B, 5 percent damped.
*2016 CBC

CONCRETE SLAB-ON-GRADE CONSTRUCTION

25. Based on the laboratory testing results of the near-surface soil, the native surface soil at the project site has been found to have a high expansion potential when subjected to fluctuations in moisture.

26. The concrete slab, if any, should have a minimum thickness of 5 inches reinforced with No. 4 rebar at maximum spacing of 18 inches on-center both ways.

27. A minimum of 18 inches of ¾ inch crushed rock (recycled crushed asphalt concrete is not acceptable) and vapor barrier membrane (Stego 15 mil) should be placed between the finished grade and the concrete slab. The vapor barrier should be taped at the seams and/or mastic sealed at the protrusions. The subgrade should be moisture conditioned and compacted to 90% relative maximum density.

28. Prior to placing the vapor membrane and/or pouring concrete, the slab grade shall be moistened with water to reduce the swell potential, if deemed necessary, by the field engineer at the time of construction.

EXCAVATION

29. No difficulties due to soil conditions are anticipated in excavating the on-site material. Conventional earth moving equipment will be adequate for this project.

30. Any vertical cuts deeper than 5 feet must be properly shored. The minimum cut slope for excavation to the desired elevation is one horizontal to one vertical (1:1). The cut slope should be increased to 2:1 if the excavation is conducted during the rainy season or when the soil is highly saturated with water.

BASEMENT EXCAVATION

31. It is our understanding that the excavation for the underground parking structure will be approximately 20 to 23 feet below the existing ground elevation. No difficulties due to soil conditions are anticipated in excavating the on-site material. Conventional earth moving equipment will be adequate for this project.

32. Any vertical cuts deeper than 5 feet must be properly shored. The temporary minimum cut slope for excavation to the desired elevation is

one horizontal to one vertical (1:1). The cut slope should be increased to 2:1 if the excavation is conducted during the rainy season or when the soil is highly saturated with water.

33. The bottom subgrade of the underground basement structure will be approximately 20 feet to 23 feet below ground surface elevation. Groundwater was initially encountered in Boring B-1 at the depth of 25 feet and rose a static level of 15 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 058 [*Seismic Hazard Evaluation of the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California. 2002 (Updated 10/10/05).* Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 10 feet below ground elevation. Therefore, dewatering may be required during basement excavation. A dewatering expert should be consulted for further design and recommendations.

34. The bottom subgrade of the basement excavation may be wet and soft due to the presence of groundwater. Therefore, the bottom subgrade should be stabilized with 18 to 24-inch layer of ¾-inch crushed rock compacted in-place over stabilization fabric membrane (Mirafi 500X or equivalent).

35. Standing groundwater at the bottom subgrade should be pumped out to provide a dry and stable working platform for the construction equipment.

36. If there are space constraints for open excavation, we recommend that the following procedure be implemented for shoring of the underground parking structure excavation.

SHORING SUPPORT FOR THE BASEMENT EXCAVATION

37. The basement will be excavated to the approximate depth of 20 to 23 feet below existing ground surface. Therefore, the excavation should be

supported with steel "H" beams and a 3 x 12 or 4 x 12 wood lagging. Prior to any excavation, the steel "H" beams should be placed in pre-drilled minimum 24-inch diameter holes to a minimum depth of 40 feet. The holes should be filled with concrete to one foot below the bottom of the excavation and concrete slurry (2 sack cement) for the remaining void to existing ground elevation. Groundwater will be encountered and should be displaced properly in the pier holes by the concrete via tremmie pipe or other methods approved by our office. At this point, excavation can begin. As the excavation operation proceeds, the wood lagging should be placed between the steel "H" beams. The "H" beams should be placed a maximum distance of 8 feet apart. There should be no voids between the soil wall excavation and wood lagging. However, if a void occurs, the void should be filled with sand slurry or pressure grouted especially at the area below each lagging bench (last lagging board). Proper attention should be considered during the construction. Introduction of any heavy equipment on the top of the vertical cut may damage the excavated slope. The lateral soil pressure acting on the shoring system is shown in Figure 7. The passive pressure of 250 pounds equivalent fluid pressure can be used for short-term shoring purposes. The shoring should be designed by the structural engineer or shoring design engineer and our office should review the shoring plan for approval.

38. Tie-backs can be utilized to reduce soldier beam depth.

BASEMENT RETAINING WALLS

39. The basement retaining walls should be design for seismic loading condition. The pseudo-static method by Seed and Whitman can be used (PE = $(3/8)(0.45\mathbf{a}_{max}/g)(\mathbf{H}^2)\mathbf{W}_t$ (where $\mathbf{a}_{max}$ = 0.50g; $\mathbf{H}$ = height of the retaining wall; $\mathbf{W}_t$ = total unit weight of retained soil, for this site $\mathbf{W}_t$ = 120

pcf). This pseudo-static pressure is inverted triangularly-distributed with the top value of 466 psf and 0 psf at the bottom. This pseudo-static pressure should be added to the active pressure for seismic loading condition.

40. The basement retaining wall shall be designed for active lateral earth pressure (static and seismic), hydrostatic lateral, and a surcharge value of 100 psf (vertically uniformed distributed down to 6 feet) as shown in Figure 8. This surcharge also include truck loading and any adjacent structures.

41. A friction coefficient of 0.3 shall be used for retaining wall design. This value may be increased by 1/3 for short-term seismic loads.

42. The basement walls should be waterproofed with Bitumen Waterproof Membrane, Paraseal LG or equivalent including pipes protruding through the basement concrete walls. A waterproofing consultant should provide waterproofing recommendations.

43. The basement walls should be designed to assume an un-drained condition. As a result, a subdrain system would not be required.

44. We recommend a thorough review by our office of all designs pertaining to facilities retaining a soil mass.

SITE RETAINING WALLS

45. Any facilities that will retain a soil mass near the existing ground surface shall be designed for a lateral earth pressure (active) equivalent to 50 pounds equivalent fluid pressure, plus surcharge loads. If the retaining walls are restrained from free movement at both ends, the walls shall be designed for the earth pressure resulting from 60 pounds equivalent fluid pressure, to which shall be added surcharge loads.

46. In designing for allowable resistive lateral earth pressure (passive), a value of 250 pounds equivalent fluid pressure may be used with the resultant acting at the third point. The top foot of native soil shall be neglected for computation of passive resistance.

47. A friction coefficient of 0.3 shall be used for retaining wall design. This value may be increased by 1/3 for short-term seismic loads.

48. The above values assume a drained condition and a moisture content compatible with those encountered during our investigation.

49. Drainage should be provided behind the retaining wall. The drainage system should consist of perforated pipe, Schedule 40 or equivalent, placed at the base of the retaining wall and surrounded by ¾ inch drain rock wrapped in a filter fabric, Mirafi 140N or equivalent. The drain rock wrapped in fabric (subdrain) should be at least 12 inches wide and extend from the base of the wall to within 1.5 feet of the ground surface. The upper 1.5 feet of backfill should consist of compacted native soil. The retaining wall drainage system should drain to an appropriate discharge facility.

50. As an alternative to the drain rock and fabric, Miradrain 2000, 6000, or approved equivalent drain mat may be used behind the retaining wall. The drain mat should extend from the base of the wall to the ground surface. A perforated pipe (subdrain system) should be placed at the base of the wall in direct contact with the drain mat. The retaining wall drainage system should drain to an appropriate discharge facility.

51. We recommend a thorough review by our office of all designs pertaining to facilities retaining a soil mass.

DRAINAGE

52. It is considered essential that positive drainage be provided during construction and be maintained throughout the life of the proposed structure.

53. The final exterior grade adjacent to the proposed structure should be such that the surface drainage will flow away from the structure. Rainwater discharge at downspouts should be directed onto pavement sections, splash blocks, or other acceptable facilities, which will prevent water from collecting in the soil adjacent to the foundations.

54. Utility lines that cross under the slab or through perimeter slab should be completely sealed to prevent moisture intrusion into the areas under the slab and/or perimeter. The utility trench backfill should be of impervious material and this material should be placed at least 4 feet on either side of the exterior perimeter.

55. Consideration should be given to collection and diversion of roof runoff and the elimination of planted areas or other surfaces which could retain water in areas adjoining the building. The grade adjacent to the foundation should be sloped away from the structure at a minimum of 3 percent.

56. Based on laboratory test results of the near surface soil at the subject site, we estimated that the infiltration rate is approximately 0.1 inch per hour ($K_{SAT} = 7.5x10^{-5}$ cm/sec). This rate can be used in the design of the retention system for on-site storm drainage.

ABANDONMENT OF THE EXISTING UTILITY LINES

57. All existing and abandoned utility lines located within the new building pad and basement area must be removed.

58. All abandoned utility lines within 2 feet from existing ground surface should be removed.

59. Removing the utility lines would require proper backfill and re-compaction of the excavation. Abandoning utility lines in-place would require to cap the abandoned portion of the pipe and all exposed pipe ends with concrete and the removal of any surface clean-outs, manhole or drain inlet structures.

ON-SITE UTILITY TRENCHING

60. All on-site utility trenches must be backfilled with approved bedding material around the pipe and native on-site material or import fill above the pipe. Backfill should be placed in 8 to 12 inch lifts and compacted to at least 90% relative maximum density. Jetting of trench backfill is not recommended. An engineer from our firm should be notified at least 48 hours before the start of any utility trench backfilling operations.

61. If utility trench excavation is to encounter groundwater, our office should be notified for dewatering recommendations.

PAVEMENT DESIGN

62. Due to the uniformity of the near-surface soil at the site, one R-Value Test was performed on a representative bulk sample. The result of the R-Value test is enclosed in this report. The following alternate asphalt sections are based on our laboratory resistance R-Value test of near-surface soil samples and traffic indices (T.I.) of 4.5 for parking stalls and 5.5 for parking area and driveway (travel way). Alternate asphalt pavement section designs, which satisfy the State of California Standard Design Criteria, and above traffic indices, are presented in Table II. Concrete and paver pavement section designs are presented in Table III

and IV. Due to the high expansion potential of the surface native soil, minor cracks in the pavement should be expected.

LIME TREATMENT ALTERNATIVES

63. Lime treatment of the subgrade soil can be considered as an option in order to reduce the high expansion potential of near-surface native soil and/or to weather proof (winterize) the subgrade soil during the winter construction of the building pad or parking structure basement area. The top 18 inches of the subgrade can be treated with a mixture of 5% of quick lime (High Calcium) and native soil by volume. If the lime treatment is used, minor cracks on the concrete slab and separation of the curb/gutter and pavement should be expected. In the building/basement pad area, if lime treatment would be implemented, the rock section could be reduced by one inch.

64. The lime-treated subgrade soil should not be exposed to the element for an extended period. If no improvements are planned for the immediate future, the lime-treated subgrade soil should be protected.

CORROSIVITY ANALYSIS

65. Two soil samples collected on July 12, 2018 at the depth of 3 feet to 5 feet (1-2) and 10 feet to 12 feet (1-3) below existing grade were submitted to Cooper Testing Lab. The sample was tested for Resistivity (100% Saturation), Conductivity, Chloride, Sulfate, pH, and Redox potential.

- The soil resistivity measurement for the near surface soil are 2,155 Ohm-cm to 3,041 Ohm-cm, which can be classified as "highly corrosive". Therefore, all buried iron, steel, cast iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the nature of the structure. In addition, all buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

- The chloride ion concentrations for the site soil are less than 2 mg/kg to 2 mg/kg. Because the chloride concentrations are 2 and less than 2, it is determined to be insufficient to attack steel embedded in a concrete mortar coating.

- The sulfate ion concentrations for the site soil are 67 mg/kg to 111 mg/kg. Therefore, the sulfate ion concentration in the soil is determined to be moderate to damage reinforced concrete structures and cement mortar-coated steel at the site.

- The type of cement for construction: Evaluation of soluble sulfate content of soil samples considered representative of the predominate material types on-site suggests that Type II or V cement is a requirement for use in construction.

- The soil pH for the near surface soil are 8.0, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

- The soil redox potential for the near surface soil are 470 mV to 488 mV, which is indicative of potentially "non-corrosive" soil resulting from anaerobic soil conditions.

A corrosivity consultant should be consulted if necessary such as for the cathodic protection design. The corrosive potential for each soil characteristic is summarized on the table below. The results of the corrosivity laboratory tests results are enclosed in the Appendix.

CORROSIVE POTENTIAL

Soil Characteristics	Range	Soil Sample 1-2	Soil Sample 1-3	Corrosive Potential
Resistivity (Ohm-cm)	>2000	2,155	3,041	Highly corrosive
Soil pH	>5.1	8.0	8.0	Non-corrosive
Chloride (mg/Kg)	<300	<2	2	Non-corrosive
Sulfate (mg/Kg)	>10	67	111	Moderately corrosive
Redox Potential (mV)	>100	470	488	Non-corrosive

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations presented herein are based on the soil conditions revealed by our test borings and evaluated for the proposed construction planned at the present time. If any unusual soil conditions are encountered during the construction, or if the proposed construction will differ from that planned at the present time, Silicon Valley Soil Engineering (SVSE) should be notified for supplemental recommendations.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the necessary steps are taken to see that the contractor carries out the recommendations of this report in the field.

3. The findings of this report are valid, as of the present time. However, the passing of time will change the conditions of the existing property due to natural processes, works of man, from legislation or the broadening of knowledge. Therefore, this report is subjected to review and should not be relied upon after a period of three years.

4. The conclusions and recommendations presented in this report are professional opinions derived from current standards of geotechnical practice and no warranty is intended, expressed, or implied, is made or should be inferred.

5. The area of the borings is very small compared to the site area. As a result, buried structures such as septic tanks, storage tanks, abandoned utilities, or etc. may not be revealed in the borings during our field investigation. Therefore, if buried structures are encountered during grading or construction, our office should be notified immediately for proper disposal recommendations.

6. Standard maintenance should be expected after the initial construction has been completed. Should ownership of this property change hands, the prospective owner should be informed of this report and recommendations so as not to change the grading or block drainage facilities of this subject site.

7. This report has been prepared solely for the purpose of geotechnical investigation and does not include investigations for toxic contamination studies of soil or groundwater of any type. If there are any environmental concerns, our firm can provide additional studies.

8. Any work related to grading and/or foundation operations during construction performed without direct observation from SVSE personnel will invalidate the recommendations of this report and, furthermore, if we are not retained for observation services during construction, SVSE will cease to be the Geotechnical Engineer of Record for this subject site.

REFERENCES

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USGS (March 19, 2018), U.S. Seismic Design Maps http://earthquake.usgs.gov/designmaps/us/application.php.

TABLES

TABLE I

SUMMARY OF LABORATORY TESTS

Sample No.	Depth (Feet)	In-Place Conditions: Moisture Content (% Dry Wt.)	In-Place Conditions: Dry Density (pcf)	Direct Shear Testing: Unit Cohesion (ksf)	Direct Shear Testing: Angle of Internal Friction (Degrees)	Atterberg Limits: Liquid Limit L.L.	Atterberg Limits: Plasticity Index P.I.
1-1	3	16.7	111.0	1.0	12		
1-2	5	14.7	106.3				
1-3	10	16.5	92.8				
1-4	15	36.6	84.4			47	25
1-5	20	34.7	87.5				
1-6	25	30.4	88.6			45	23
1-7	30	16.2	119.3				
1-8	35	12.9	126.9				
1-9	40	3.5	124.2				
1-10	45	16.6	117.1				
1-11	50	15.7	123.8				
1-12	55	24.8	105.0			40	22
1-13	60	25.6	103.1			41	21
2-1	3	16.4	113.5				
2-2	5	15.1	100.9				
2-3	10	20.4	105.8				
2-4	15	33.5	88.4				
2-5	20	35.7	84.1				

TABLE I (CONTINUED)

SUMMARY OF LABORATORY TESTS

Sample No.	Depth (Feet)	In-Place Conditions: Moisture Content (% Dry Wt.)	In-Place Conditions: Dry Density (pcf)	Direct Shear Testing: Unit Cohesion (ksf)	Direct Shear Testing: Angle of Internal Friction (Degrees)	Atterberg Limits: Liquid Limit L.L.	Atterberg Limits: Plasticity Index P.I.
3-1	3	16.9	110.1				
3-2	5	14.5	101.4				
3-3	10	22.9	103.6				

TABLE II

PROPOSED ASPHALT PAVEMENT SECTIONS

Location: Proposed Hotel
615 & 623 Stockton Avenue
San Jose, California

	PARKING STALLS			DRIVEWAY		
Design R-Value	6.0			6.0		
Traffic Index	4.5			5.5		
Gravel Equivalent	17.0			20.0		
Recommended Alternate Pavement Sections:	1A	1B	1C	2A	2B	2C
Asphalt Concrete	3.0"	3.5"	4.0"	3.0"	3.5"	4.0"
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	9.0"	8.0"	7.0"	11.0"	10.0"	9.0"
Subgrade soil scarified & compacted to at least 90% relative max. density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"

TABLE III

PROPOSED CONCRETE PAVEMENT SECTIONS

Location: Proposed Hotel
615 & 623 Stockton Avenue
San Jose, California

	DRIVEWAY*	CURB & GUTTER	SIDEWALK/PATIO**
Recommended Rigid Pavement Sections:			
P.C. Concrete*	6.0"	6.0"	4.0"
Class II Baserock (R=78 min.) compacted to at least 95% relative max. density	12.0"	8.0"	6.0"
Subgrade soil scarified & compacted to at least 90% relative max. density	12.0"	12.0"	12.0"

* Including trash enclosures, stress pads, and valley gutters. Minimum reinforcement: No. 4 rebar at maximum spacing, 18 inches on-center both ways or provided by Structural Engineer. Maximum control joints at 5 feet by 5 feet or as recommended by Structural Engineer. Vertical curbs should be keyed at least 3 inches into pavement subgrade.

** Minimum reinforcement: No. 3 rebar at maximum spacing, 18 inches on-center both ways or provided by Structural Engineer.

TABLE IV

PROPOSED PAVER PAVEMENT SECTIONS

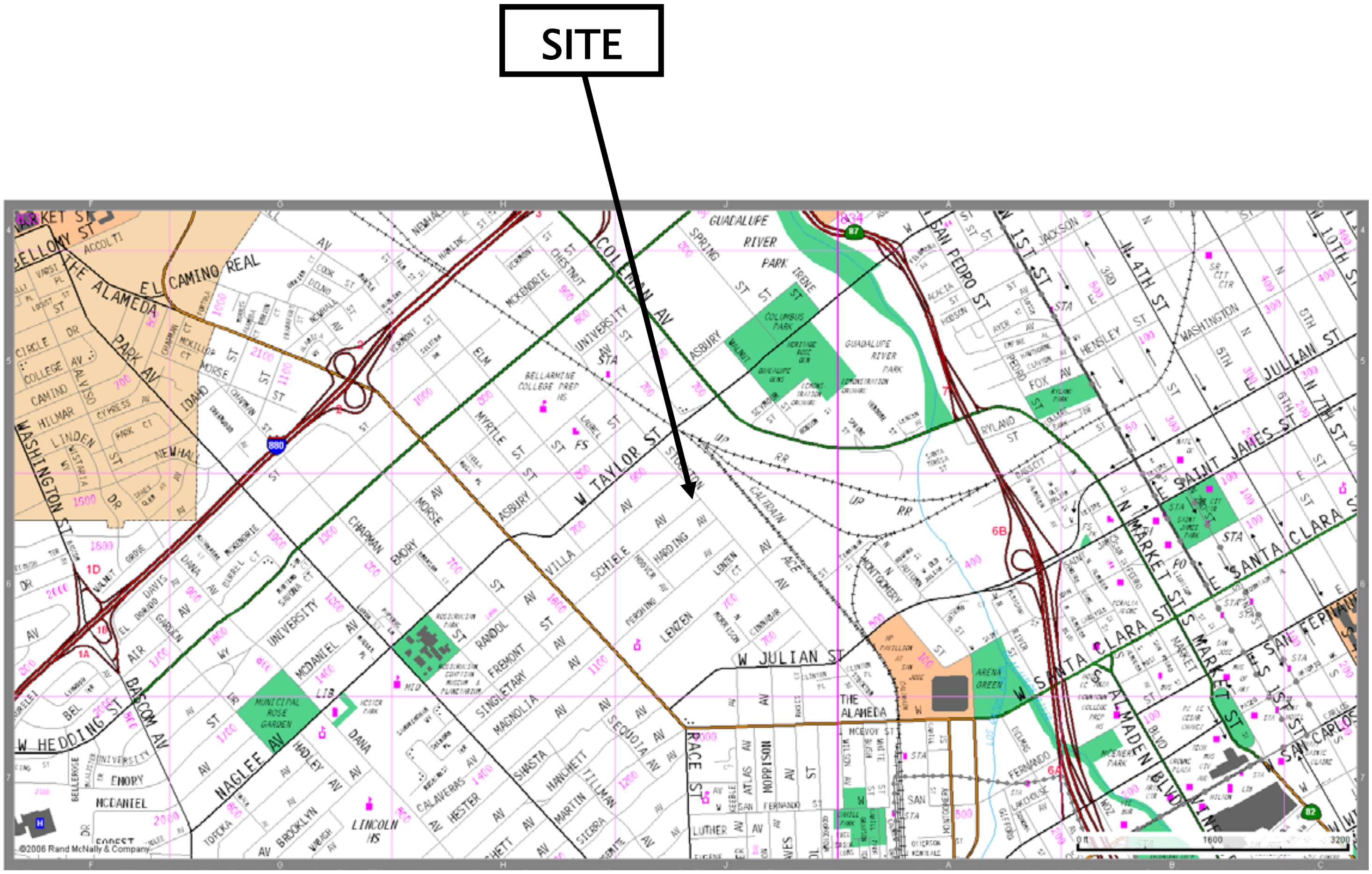
Location: Proposed Hotel
615 & 623 Stockton Avenue
San Jose, California

	DRIVEWAY/PARKING AREA			
Recommended Paver Pavement Sections:	1A*	2A**	1B*	2B**
Vehicular Rated Pavers	Min. 3.25" ± Permeable Paver Parking Stalls	Min. 3.25" ± Permeable Paver Parking Stalls	Min. 3.25" ± Permeable Paver Driveway	Min. 3.25" ± Permeable Paver Driveway
ASTM No. 8 Bedding Course & Paver Filler	2.0"	2.0"	2.0"	2.0"
3/4" Clean Crushed Rock or ASTM No. 57 Stone	13.0"	4.0"	17.0"	8.0"
ASTM No. 2 Stone	---	12.0"	---	12.0"
Subgrade soil scarified and compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"

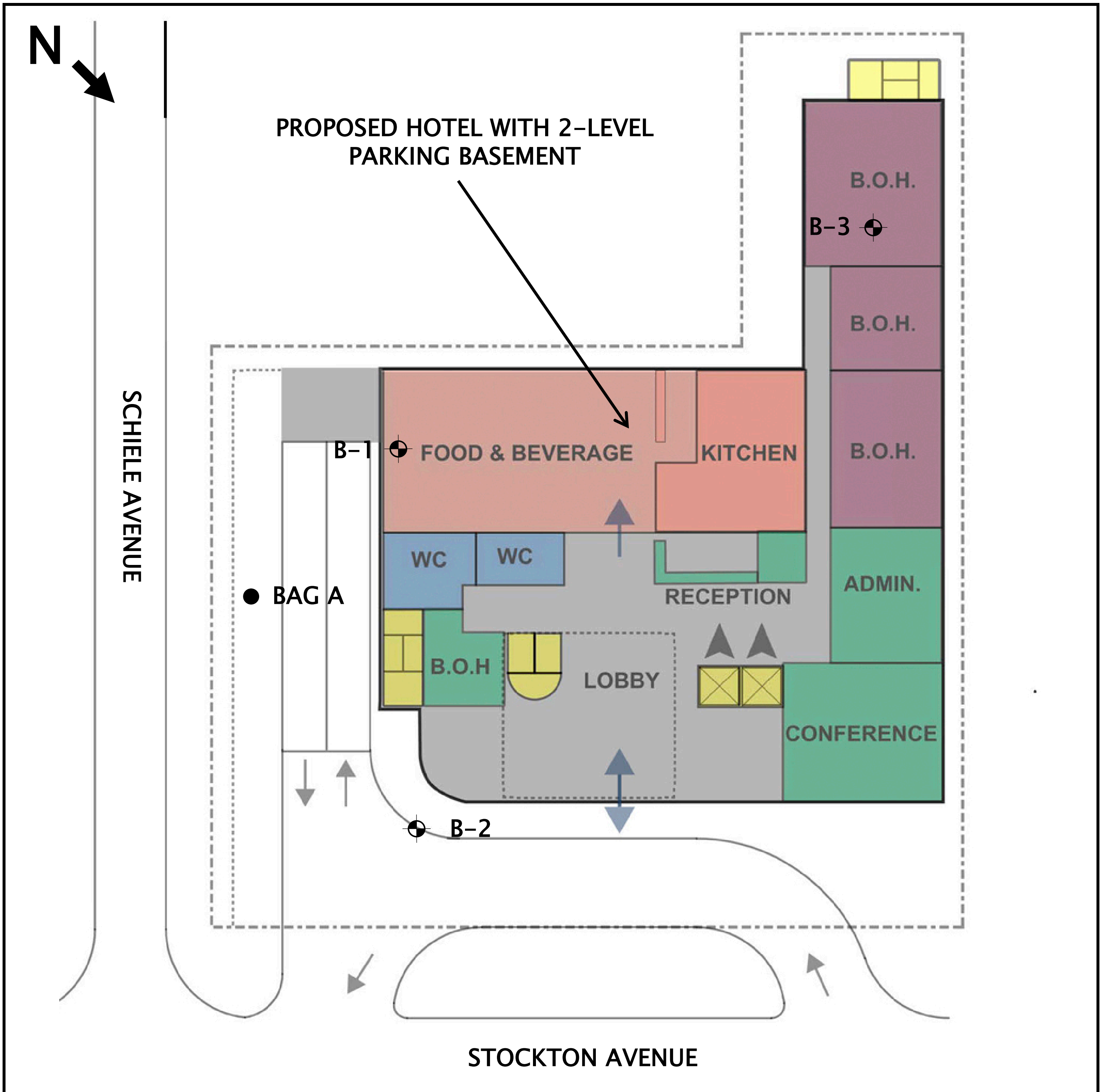
* The subgrade should be lined with a geotextile membrane, Geogrid or equivalent. The subgrade should be sloped at a minimum of 2% towards the subdrain system and away from building foundation. The subdrain system should consist of a 4-inch diameter perforated pipe surrounded by ¾ inch drain rock wrapped in a filter fabric. The drain rock wrapped in fabric should be at least 12 inches wide and 12 inches below the finished subgrade elevation. The drainage system should be sloped to a discharge facility. The pavers should be bordered with a concrete curb/band. Typically, minor maintenance would be required during the life of the pavers.

** The subgrade should be lined with a geotextile membrane Geogrid. The section should have an overflow output and subgrade should be sloped at a minimum of 2% away from building foundation. The pavers should be bordered with a concrete curb/band. Typically, minor maintenance would be required during the life of the pavers.

FIGURES

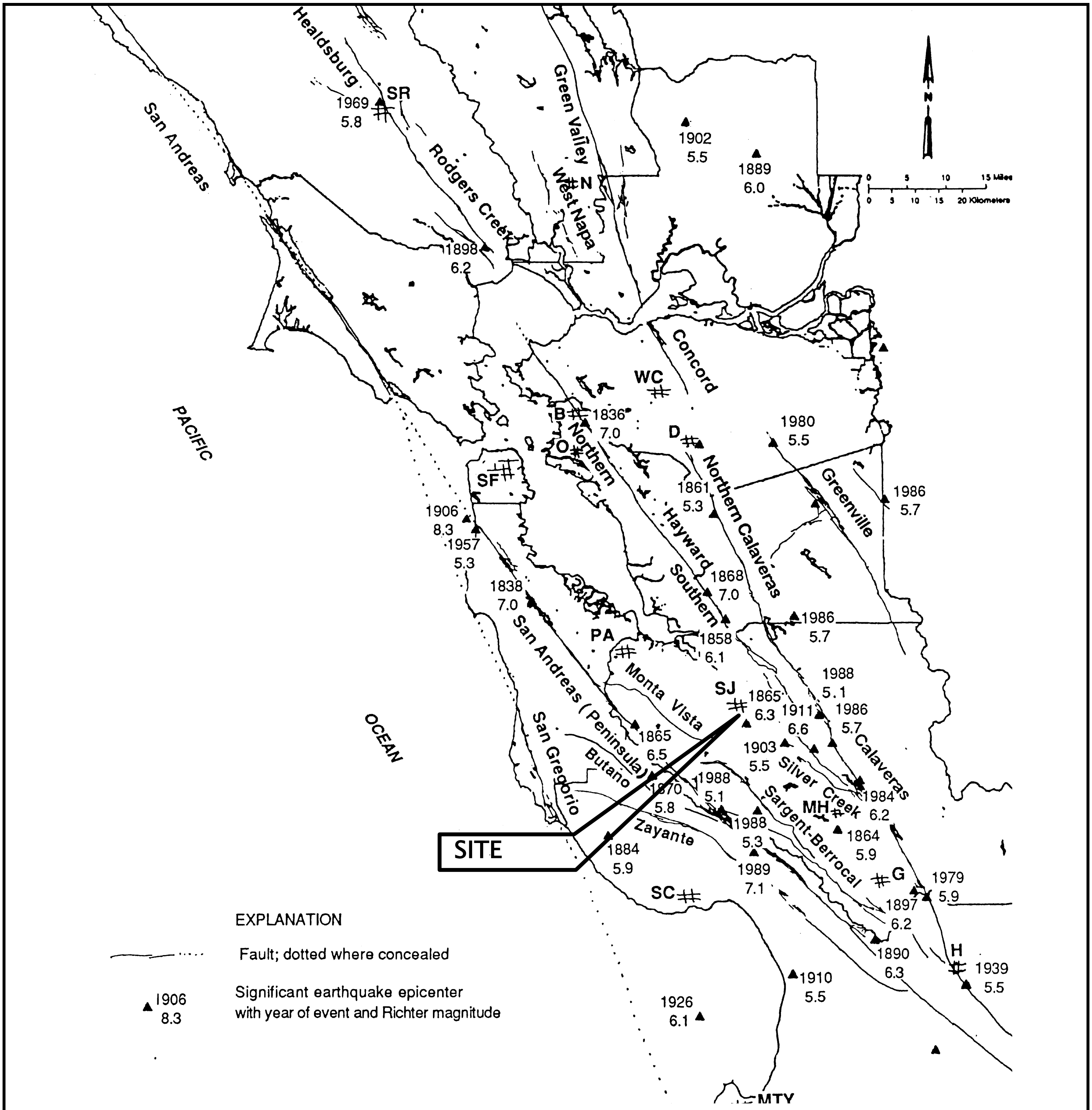


Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**VICINITY MAP** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No.: SV1795	FIGURE
		Drawn by: V.V.	1
		Scale: NOT TO SCALE	July 2018



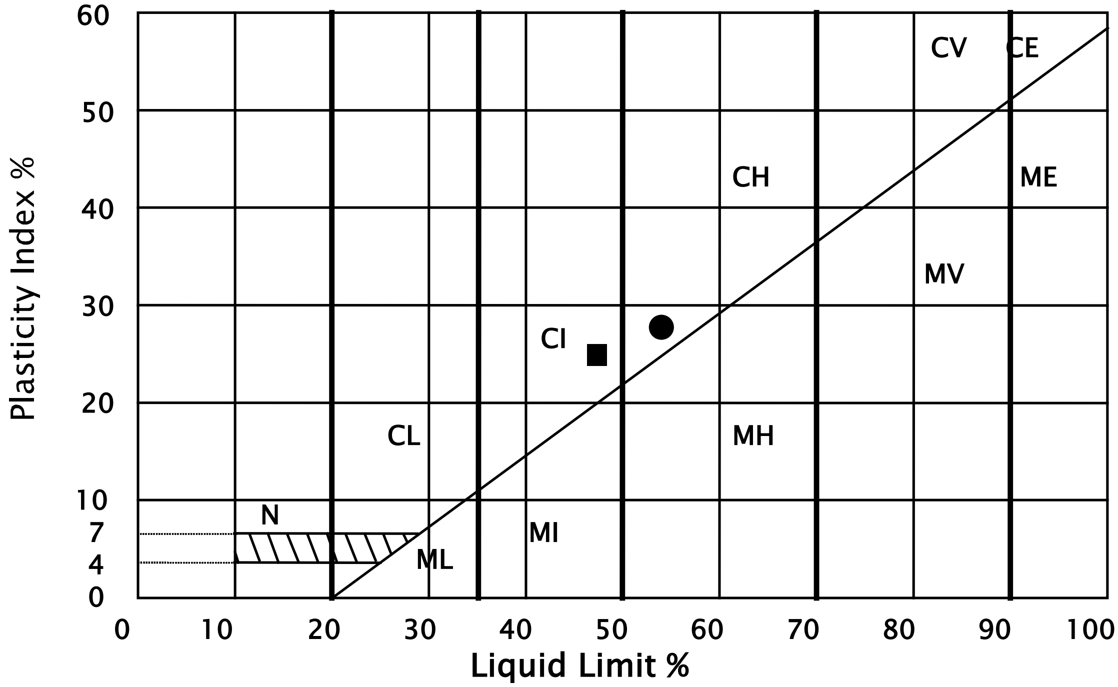
NOTE: ⊕ DENOTES APPROXIMATE EXPLORATORY BORING LOCATION
● DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**SITE PLAN** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No.: SV1795	FIGURE
		Drawn by: V.V.	2
		Scale: NOT TO SCALE	July 2018



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**FAULT LOCATION MAP** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No.: SV1795	FIGURE
		Drawn by: V.V.	3
		Scale: NOT TO SCALE	July 2018

PLASTICITY CHART

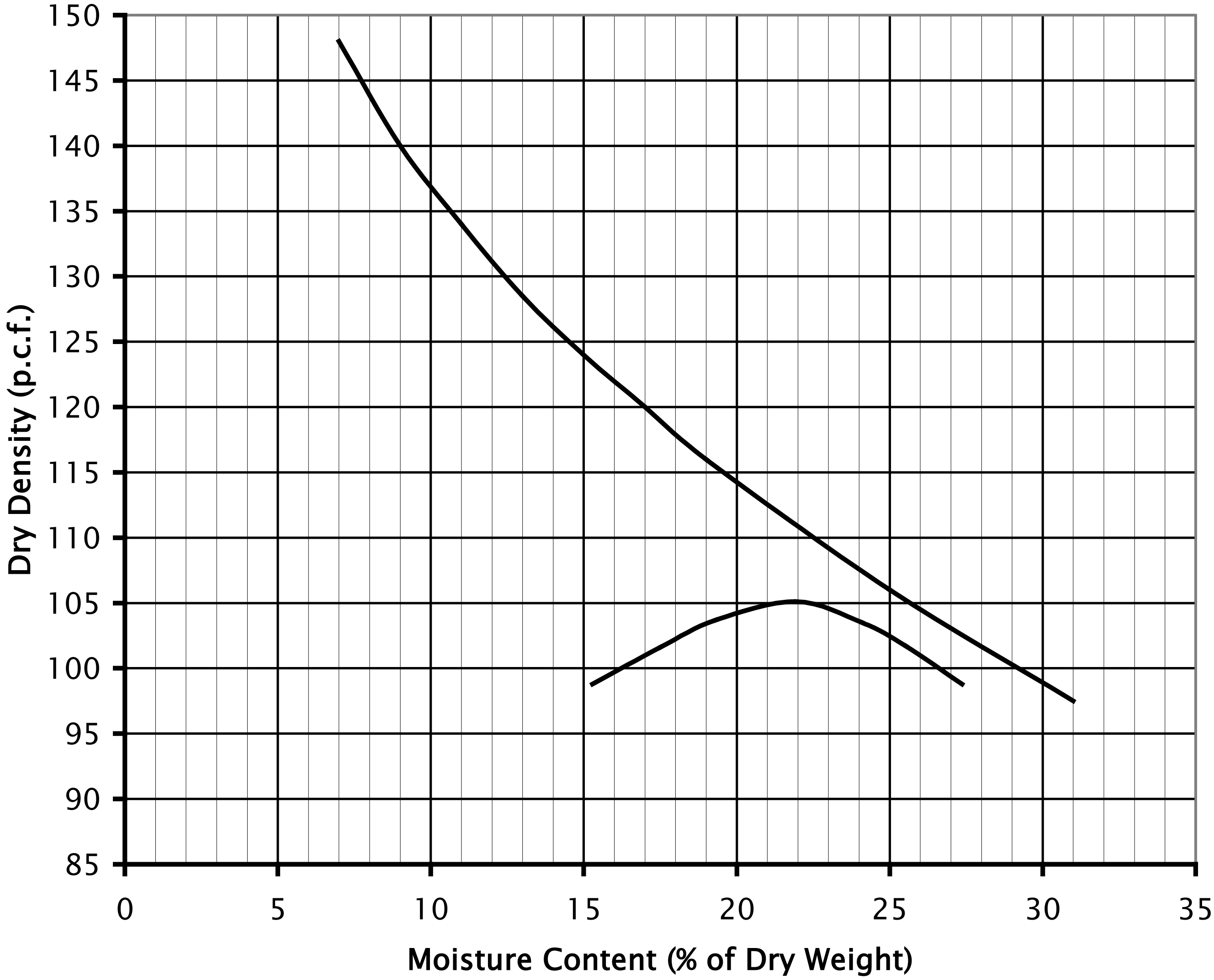


PLASTICITY DATA

Key Symbol	Hole No.	Depth Feet	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	55	28	CH
■	1-4	14-15	47	25	CI

*Soil type classification Based on British suggested revisions
to Unified Soil Classification System

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**PLASTICITY INDEX** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No.: SV1795 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 4 July 2018



SAMPLE: A

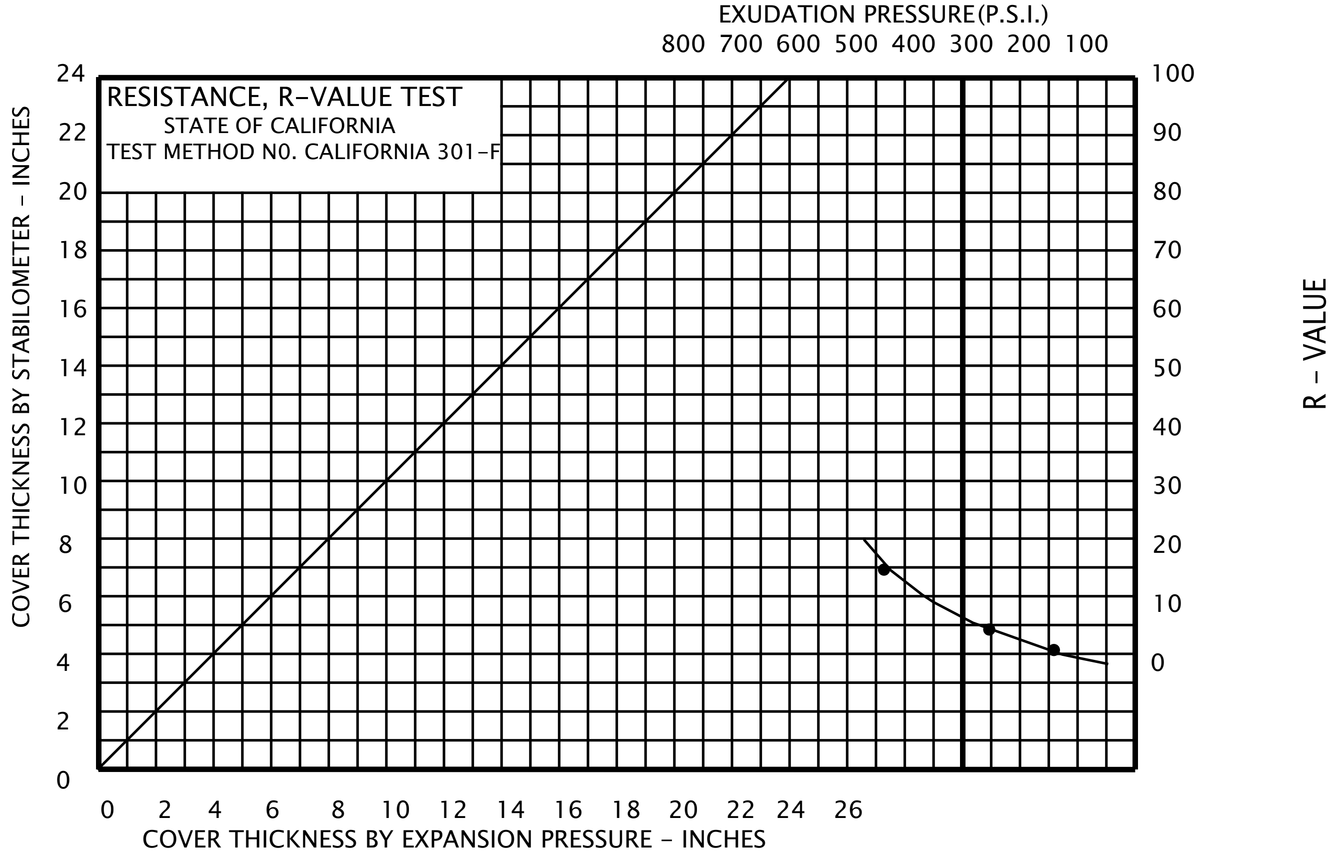
DESCRIPTION: Black Silty CLAY

LABORATORY TEST PROCEDURE: ASTM D1557

MAXIMUM DRY DENSITY: 105.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 22.0 %

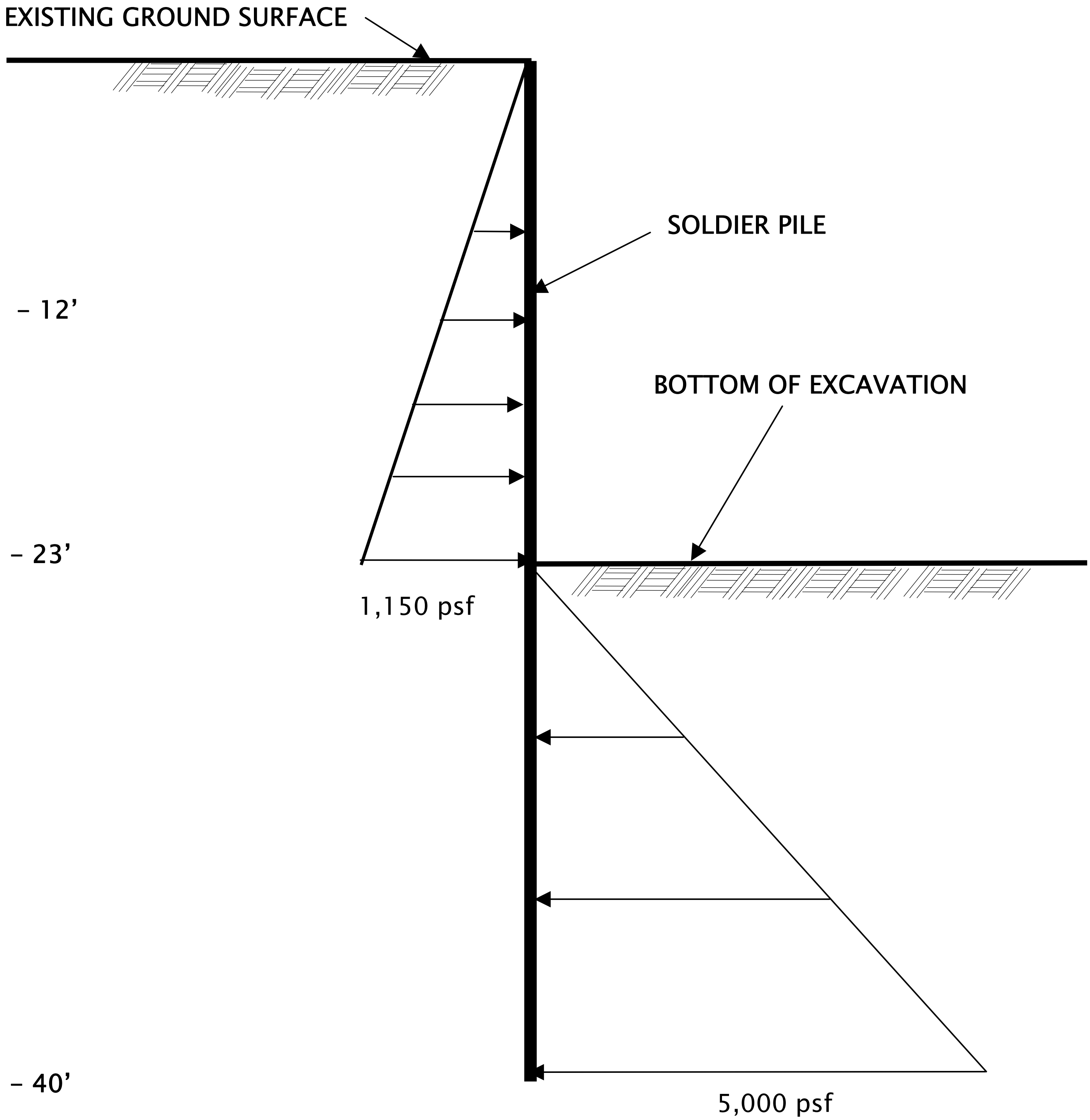
Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**COMPACTION TEST A** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No. SV1795 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 5 July 2018



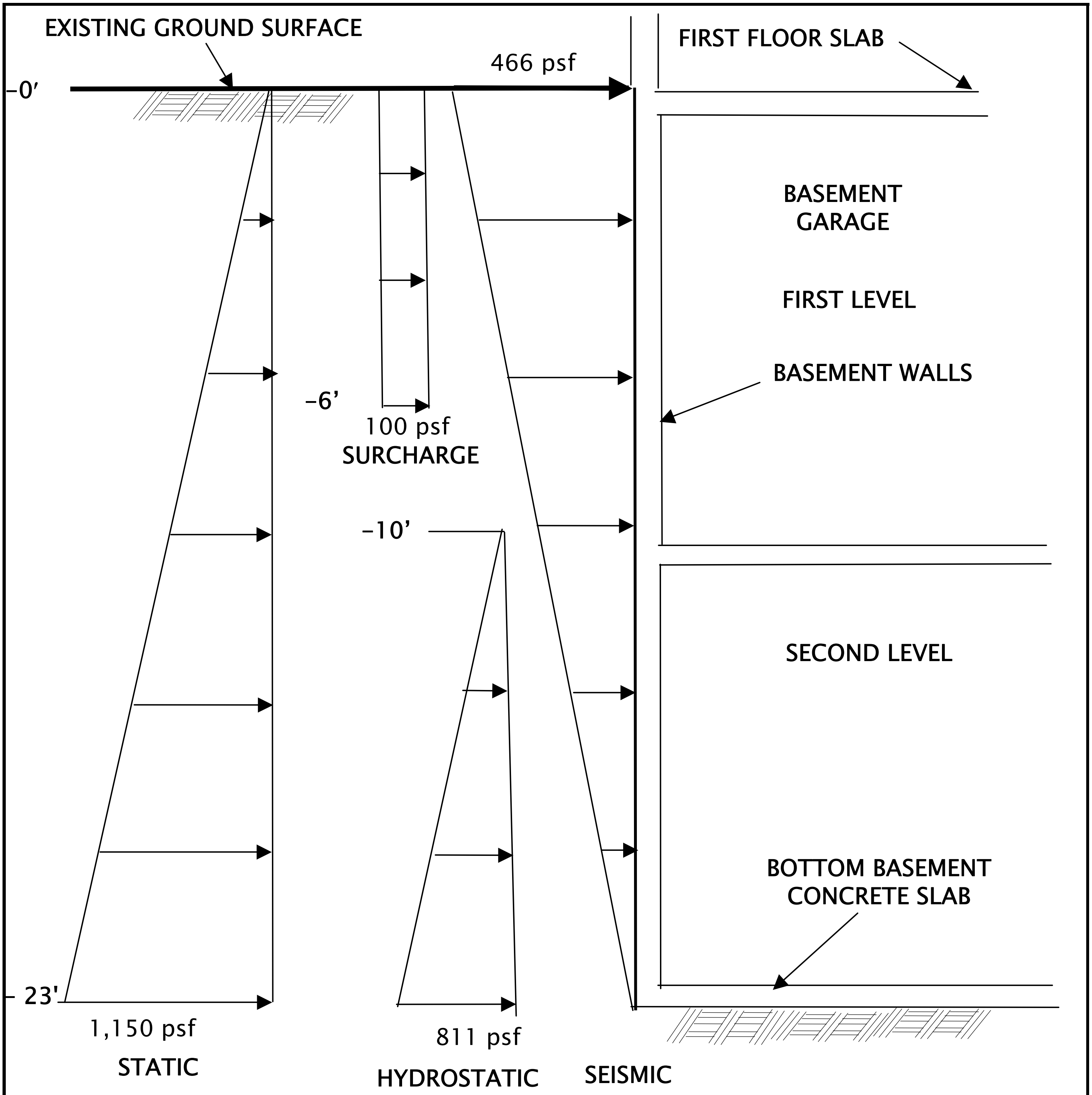
SAMPLE: A
DESCRIPTION: Black Silty CLAY

SPECIMEN	A	B	C
EXUDATION PRESSURE (P.S.I.)	149.0	251.0	449.0
EXPANSION DIAL (.0001")	9.0	14.0	20.0
EXPANSION PRESSURE (P.S.F.)	45.0	76.0	94.0
RESISTANCE VALUE, "R"	1.0	4.0	15.0
% MOISTURE AT TEST	20.7	18.0	17.6
DRY DENSITY AT TEST (P.C.F.)	106.7	108.5	111.2
R-VALUE AT 300 P.S.I. EXUDATION PRESSURE	= (6)		

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**R-VALUE TEST** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No. SV1795	FIGURE 6
		Drawn by: V.V.	
		Scale: NOT TO SCALE	July 2018



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**LATERAL SOIL PRESSURES** **SOLDIER PILE & WOOD LAGGING** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No.: SV1795 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 7 July 2018



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**LATERAL SOIL PRESSURES BASEMENT WALLS** Proposed Hotel 615 & 623 Stockton Avenue San Jose, California	File No.: SV1795	FIGURE
		Drawn by: V.V.	8
		Scale: NOT TO SCALE	July 2018

APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

KEY TO LOG OF BORING

EXPLORATORY BORING LOGS (B-1 THROUGH B-3)

CORROSIVITY TESTS SUMMARY

GENERAL COMPARISON BETWEEN EARTHQUAKE MAGNITUDE AND THE EARTHQUAKE EFFECTS DUE TO GROUND SHAKING

Earthquake Category	Richter Magnitude	Modified Mercalli Intensity Scale* (After Housner, 1970)	Damage to Structure
		I – Detected only by sensitive instruments.	
	2.0	II – Felt by few persons at rest, especially on upper floors; delicate suspended objects may swing.	
	3.0	III – Felt noticeably indoors, but not always recognized as an earthquake; standing cars rock slightly, vibration like passing truck.	No Damage
Minor		IV – Felt indoors by many, outdoors by a few; at night some awaken; dishes, windows, doors disturbed; cars rock noticeably.	
	4.0	V – Felt by most people; some breakage of dishes, windows, and plaster; disturbance of tall objects.	Architec-tural Damage
		VI – Felt by all; many are frightened and run outdoors; falling plaster and chimneys; damage small.	
5.3	5.0	VII – Everybody runs outdoors. Damage to building varies, depending on quality of construction; noticed by drivers of cars.	
Moderate	6.0	VIII – Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected; drivers of cars disturbed.	
6.9		IX – Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked, underground pipes broken; serious damage to reservoirs and embankments.	Structural Damage
Major	7.0	X – Most masonry and frame structures destroyed; ground cracked; rail bent slightly; landslides.	
7.7		XI – Few structures remain standing; bridges destroyed; fissures in ground; pipes broken; landslides; rails bent.	
Great	8.0	XII – Damage total; waves seen on ground surface; lines of sight and level distorted; objects thrown into the air; large rock masses displaced.	Near Total Destruction

*Intensity is a subject measure of the effect of the ground shaking, and is not engineering measure of the ground acceleration.

File No. SV1795

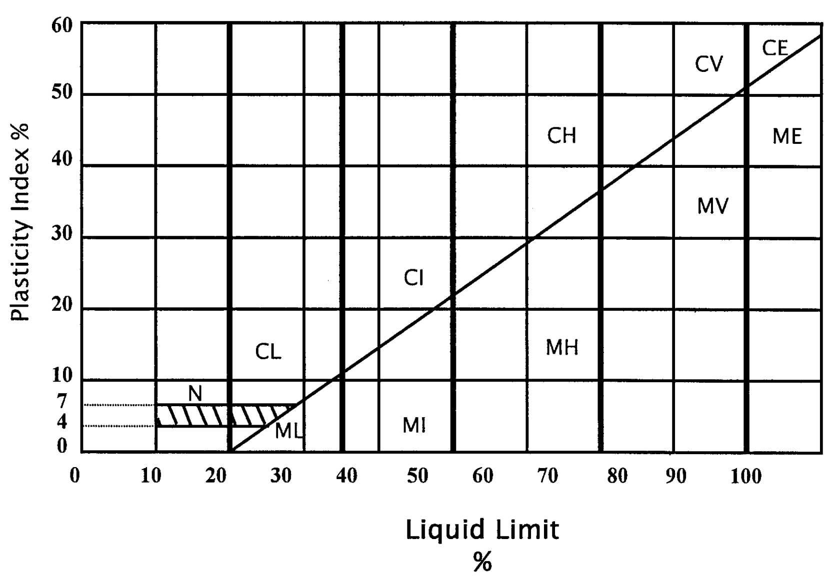
METHOD OF SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS	(More than 1/2 of soil > no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	GW	Well graded gravel or gravel-sand mixtures, little or no fines
			GP	Poorly graded gravel or gravel-sand moistures, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey Gravels, gravel-sand-clay mixtures
		SANDS (More than 1/2 of coarse fraction < no. 4 sieve size	SW	Well graded sands or gravelly sands, no fines
			SP	Poorly graded sands or gravelly sands, no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS	(More than 1/2 of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML	Inorganic silts and very fine sand, rock, flour, silty or clayey fine sand or clayey silt/slight plasticity
			CL	Inorganic clay of low to medium plasticity, gravelly clayes, sandy clay, silty clay, lean clays
			OL	Organic siltys and organic silty clay of low plasticity
		SILTS & CLAYS LL > 50	MH	Inorganic silts, micaceous or diatocaceous fine sandy, or silty soils, elastic silt
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOIL			PT	Peat and other highly organic soils

CLASSIFICATION CHART - UNIFIED SOIL CLASSIFICATION SYSTEM

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size In Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVELS Coarse Fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND Coarse Medium Fine	No. 4 to No. 200 No. 4 to No. 10 No.10 to No. 40 No.40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT AND CLAY	Below No. 200	Below 0.074

PLASTICITY INDEX CHART



Project: Proposed Hotel **Project Location:** 615 & 623 Stockton Ave. San Jose, California **Project Number:** SV1795	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Key to Log of Boring **Sheet 1 of 1**

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
1	2	3	4	5	6	7	8	9	10	11	12	13

COLUMN DESCRIPTIONS

1 Depth (feet): Depth in feet below the ground surface.
2 Sample Type: Type of soil sample collected at the depth interval shown.
3 Sample Number: Sample identification number.
4 Sampling Resistance, blows/ft: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.
5 Material Type: Type of material encountered.
6 Graphic Log: Graphic depiction of the subsurface material encountered.
7 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
8 Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.
9 Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.
10 Direct Shear Test - Cohesion in ksf: Cohesion is the y-axis intercept of the failure envelope tangent to the Mohr circles.
11 Direct Shear Test - Internal Friction Angle in degrees: The internal friction angle (Phi) is the angle inclination of the failure envelope.
12 Liquid Limit - LL, %: Liquid Limit, expressed as a water content.
13 Plasticity Index - PI, %: Plasticity Index, expressed as a water content.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Asphaltic Concrete (AC)
Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)
Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
Lean-Fat CLAY, CLAY w/SAND, SANDY CLAY (CL-CH)
Poorly graded GRAVEL (GP)
Aggregate Base (AB)
SILT, SILT w/SAND, SANDY SILT (ML)
Poorly graded SAND (SP)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Auger sampler
Bulk Sample
3-inch-OD California w/ brass rings
CME Sampler
Grab Sample
2.5-inch-OD Modified California w/ brass liners
Pitcher Sample
2-inch-OD unlined split spoon (SPT)
Shelby Tube (Thin-walled, fixed head)

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)
Water level (after waiting)
Minor change in material properties within a stratum
Inferred/gradational contact between strata
—?— Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Project: Proposed Hotel Project Location: 615 & 623 Stockton Ave. San Jose, California Project Number: SV1795	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	# Log of Boring B-1 **Sheet 1 of 2**

Date(s) Drilled **07/12/18**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **60.0 feet**
		Approximate Surface Elevation **feet**
Groundwater Level and Date Measured **15 feet (07/12/18)**	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0–0.21				Asphalt		2.5 inches of Asphalt Concrete (AC)						
0.21–0.71						6.0 inches of Aggregate Base (AB)						
0.71–3		1-1	50	CH		Black Silty CLAY Moist, very stiff	16.7	111.0	1.0	12		
3						Color changed to medium gray						
3–10		1-2	40	ML		Light Olive Brown Sandy SILT Moist, very stiff	14.7	106.3				
		1-3	29				16.5	92.8				
10–30				CL		Medium Brown Silty CLAY Moist, stiff						
		1-4	16				36.6	84.4			47	25
15						Stabilized at drilling completion						
		1-5	32				34.7	87.5				
20						Color changed to medium olive brown						
		1-6	43				30.4	88.6			45	23
25						First encountered						
		1-7	42				16.2	119.3				
30												

Project: Proposed Hotel
Project Location: 615 & 623 Stockton Ave.
San Jose, California
Project Number: SV1795

Silicon Valley Soil Engineering
2391 Zanker Road, Suite 350
San Jose, CA 95131
(408) 324-1400

Log of Boring B-1

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
30				CL		Greenish Gray Silty CLAY Moist, very stiff						
32				SP		Greenish Gray SAND Moist, dense SAND: medium-grained, poorly graded						
		1-8	55+				12.9	126.9				
35						Color changed to tan brown						
38				GP		Gray GRAVEL Wet, dense GRAVEL: 1.5 inches maximum diameter Sub-rounded, poorly graded						
		1-9	55+				3.5	124.2				
40												
42				SP		Tan Brown SAND Moist, dense Medium grained						
		1-10	55+				18.6	117.1				
45												
		1-11	55+				15.7	123.8				
50												
51				CL-CH		Bluish Gray Silty CLAY Moist, very stiff						
		1-12	37				24.8	105.0			40	22
55												
		1-13	32				25.6	103.1			41	21
60						Boring terminated at 60.0 feet						
65												

Project: Proposed Hotel **Project Location:** 615 & 623 Stockton Ave. San Jose, California **Project Number:** SV1795	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-2 **Sheet 1 of 1**

Date(s) Drilled **07/12/18**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **20.0 feet**
		Approximate Surface Elevation **feet**
Groundwater Level and Date Measured	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0				Asphalt		2.5 inches of Asphalt Concrete (AC)						
0.71				CH		6.0 inches of Aggregate Base (AB)						
		2-1	59			Black Silty CLAY Moist, very stiff	16.4	113.5				
3				ML		Color changed to medium gray						
		2-2	27			Tan Brown Sandy SILT Moist, stiff	15.1	100.9				
5												
		2-3	27				20.4	105.8				
10				CL		Medium Brown Silty CLAY Moist, stiff						
		2-4	24				33.5	88.4				
15												
		2-5	39				35.7	84.1				
20						Boring terminated at 20.0 feet						
25												
30												

Project: Proposed Hotel **Project Location:** 615 & 623 Stockton Ave. San Jose, California **Project Number:** SV1795	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	# Log of Boring B-3 **Sheet 1 of 1**

Date(s) Drilled **07/12/18**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **10.0 feet**
		Approximate Surface Elevation **feet**
Groundwater Level and Date Measured	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0						2.0 inches of gravel						
0.17				CH		Black Silty CLAY Moist, very stiff						
		3-1	37			Color changed to medium gray	16.9	110.1				
				ML		Tan Brown Sandy SILT Moist, stiff						
		3-2	32				14.5	101.4				
5												
		3-3	28				22.9	103.6				
10						Boring terminated at 10.0 feet						
15												
20												
25												
30												

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 768-061 | Date: 7/25/2018 | Tested By: PJ | Checked: PJ

Client: Silicon Valley Soil Engineering | Project: SV1795 | Proj. No: 615 & 623 Stockton Ave, San Jose

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B-1-1	-	3	-	-	2,155	<2	67	0.0067	8.0	470	24	-	1.1	Black Sandy CLAY
B-1-3	-	10	-	-	3,041	2	111	0.0111	8.0	488	24	-	0.9	Dark Olive Brown Clayey SAND