APPENDIX E

GEOTECHNICAL INVESTIGATION

REPORT TO

DESIGN TRENDS, INC.
SAN JOSE, CALIFORNIA

FOR

PROPOSED OFFICE BUILDING

200-210 NORTH BASCOM AVENUE
SAN JOSE, CALIFORNIA

GEOTECHNICAL INVESTIGATION

JANUARY 2020

PREPARED BY

SILICON VALLEY SOIL ENGINEERING
2391 ZANKER ROAD, SUITE 350
SAN JOSE, CALIFORNIA

SILICON VALLEY SOIL ENGINEERING
GEOTECHNICAL CONSULTANTS

File No. SV2017
January 23, 2020

Design Trends, Inc.
1528 Padres Drive
San Jose, CA 95125

Attention: Mr. Shishu Bedi, President & CEO

Subject: Proposed Office Building
200-210 North Bascom Avenue
San Jose, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Bedi:

Pursuant to your request, we are pleased to present herein geotechnical investigation for the proposed office building. The subject site is located at 200-210 North Bascom 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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INTRODUCTION

Per your authorization, Silicon Valley Soil Engineering (SVSE) conducted a geotechnical investigation. The purpose of this geotechnical investigation was to determine the nature of the surface and subsurface soil conditions at the project 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 200-210 North Bascom Avenue in San Jose, California (Figure 1). North Bascom Avenue bounds the subject site to the west, commercial retail building to the north, residential building to the east, and Forest Avenue to the south. At the time of this investigation, the subject site is a rectangular shaped, relatively flat parcel occupied by a commercial/office building. Based on the preliminary plans, the proposed development will include the demolition of the existing structure and the construction of a four-story medical office building with ground floor parking and associated improvements. The approximate location of the proposed building and our exploratory soil 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 two exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on January 16, 2020. The approximate location of the borings is shown on the Site Plan (Figure 2). The borings were drilled to the depth of 50 feet below existing ground surface 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 operations. 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 for advancing the sampler the last 12 inches of the 18 inch sampled interval were recorded on the boring log as penetration resistance.

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

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

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).

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 (50.0 foot boring), the existing pavement surface consists of 3.0 inches of asphalt concrete (AC) over 5.0 inches of aggregate base (AB). Below the pavement surface to the depth of 2 feet, a dark olive brown, moist, stiff silty clay layer was encountered. From the depths of 2 feet to 5 feet, the soil became olive brown, moist, very stiff gravelly sandy silty clay. From the depths of 5 feet to 9.5 feet, a tan yellow brown, moist, very stiff sandy silt layer was encountered. A color change of dark olive gray was noted at a depth of 9 feet. From the depths of 9.5 feet to 35 feet, the soil became dark olive gray mottled with brown, moist, very stiff sandy silty clay. Color changes of tan and gray and tan brown were noted at a depth of 14 feet and 20 feet respectively. From the depths of 35 feet to 38 feet, a brown, wet, dense sand layer was encountered. The sand was medium-grained and poorly graded. From the depths of 38 feet to 43 feet, the soil became olive gray, moist, very stiff sandy silty clay. From the depths of 43 feet to 47 feet, a gray, wet, dense sandy gravel layer was encountered. The gravel was 1.5 inches maximum diameter, sub-rounded and poorly graded. From the depths of 47 feet to the end of the boring at 50 feet, the soil became dark gray, moist, very stiff silty clay. A similar soil profile was encountered in Boring B-2.

Groundwater was initially encountered in Boring B-1 and B-2 at the depth of 35 feet and rose a static level of 34 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 across the bay to the northeast.

The site lies on the east flank of the Santa Cruz Mountains on a thin layer of Holocene alluvial deposits overlying the Merced formation, Lower Pleistocene and Upper Pliocene marine deposits. The Santa Cruz Mountains consists of two entirely different, incompatible core complexes, lying side by side and separated from each other by large faults. These two core complexes are Early Cretaceous Granitic intrusions, and an Upper Jurassic to Lower Cretaceous eugosynclinal assemblage - the Franciscan formation. These core complexes are blanketed by thick layers of Eocene to Pleistocene marine deposits. Some Miocene volcanic intrusions are also present in the Santa Cruz Mountains southwest of the subject site. The core complex of the Diablo Range to the northeast of the subject site is comprised of Franciscan formation, predominantly covered with Upper Cretaceous and Lower to Middle Pliocene marine deposits.

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.

Folds, thrust faults, steep reverse faults, and strike-slip faults developed as a consequence of Cenozoic deformations that occur very often within the province and are continuing today. Regional faults are shown on Figure 3.

LIQUEFACTION ANALYSIS:

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

A. GROUNDWATER

Groundwater was initially encountered in Boring B-1 at the depth of 35 feet and rose a static level of 34 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 30 feet of less below ground elevation. Therefore, the depth of the groundwater table at 30 feet 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 & USGS). 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 and B-2 (both are 50.0 foot deep).

BORING B-1: The results from our exploratory boring show that the subsurface soil material in Boring B-1 to the depth of 50.0 feet consists of stiff silty clay to very stiff gravelly sandy silty clay to very stiff sandy silt to very stiff sandy silty clay to dense sand to very stiff sandy silty clay to dense sandy gravel to very stiff silty clay. The following is the determination of the liquefiable soil for each soil layer in Boring B-1.

1. The stiff silty clay layer from the surface to the depth of 2.0 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

2. The very stiff gravelly sandy silty clay layer from the depths of 2.0 feet to 5.0 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

3. The very stiff sandy silt layer from the depths of 5.0 feet to 9.5 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

4. The very stiff sandy silty clay layer from the depths of 9.5 feet to 30.0 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

5. The very stiff sandy silty clay layer from the depths of 30.0 feet to 35.0 feet is not liquefiable based on the Plasticity Index (PI) and Moisture Content (MC):

   - Sample No. 1-8 (35 feet) - [PI > 18; PI = 25 and MC = 35.7% < 80% LL = 36.8%; LL = 46]

6. The dense sand layer from the depths of 35.0 feet to 38.0 feet is not liquefiable soil based on high blow count.

7. The very stiff sandy silty clay layer from the depths of 38.0 feet to 43.0 feet is not liquefiable based on the Plasticity Index (PI) and Moisture Content (MC):

   - Sample No. 1-9 (40 feet) - [PI > 18; PI = 24 and MC = 32.3% < 80% LL = 35.2%; LL = 44]

8. The dense sandy gravel layer from the depths of 43.0 feet to 47.0 feet is not liquefiable soil based on high blow count.

9. The very stiff silty clay layer from the depths of 47.0 feet to the end of boring at 50.0 feet is not liquefiable based on the Plasticity Index (PI) and Moisture Content (MC):

   - Sample No. 1-11 (50 feet) – [PI > 18; PI = 24 and MC = 34.4% < 80% LL = 36.0%; LL = 45]

BORING B-2: The results from our exploratory boring show that the subsurface soil material in Boring B-2 to the depth of 50.0 feet consists of stiff silty clay to very stiff gravelly sandy silty clay to very stiff sandy silty clay to medium dense sand to very stiff sandy silty clay to dense sandy gravel to very stiff silty clay. The following is the determination of the liquefiable soil for each soil layer in Boring B-2.

1. The stiff silty clay layer from the surface to the depth of 5 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

2. The very stiff gravelly sandy silty clay layer from the depths of 5 feet to 9 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

3. The very stiff sandy silty clay layer from the depths of 9 feet to 30 feet is not liquefiable soil because it is above the highest expected groundwater table (30 feet).

4. The very stiff sandy silty clay layer from the depths of 30 feet to 35 feet is not liquefiable based on the Plasticity Index (PI) and Moisture Content (MC):

   - Sample No. 2-8 (35 feet) – [PI > 18; PI = 25 and MC = 33.6% < 80% LL = 36.0%; LL = 45]

5. The medium dense sand layer from the depths of 35 feet to 38 feet <u>is not liquefiable</u> soil based on high blow count.

6. The very stiff sandy silty clay layer from the depths of 38 feet to 43 feet <u>is not liquefiable</u> based on the Plasticity Index (PI) and Moisture Content (MC):

- Sample No. 2-9 (40 feet) - [PI > 18; PI = 24 and MC = 30.8% < 80% LL = 34.4%; LL = 43]

7. The dense sandy gravel layer from the depths of 43 feet to 47 feet <u>is not liquefiable</u> soil based on high blow count.

8. The very stiff silty clay layer from the depths of 47 feet to the end of boring at 50 feet <u>is not liquefiable</u> based on the Plasticity Index (PI) and Moisture Content (MC):

- Sample No. 2-11 (50 feet) - [PI > 18; PI = 25 and MC = 28.9% < 80% LL = 35.2%; LL = 44]

Based on the screening process performed for Boring B-1 and B-2, there is no suspected liquefiable soil layer underlying the site.

C. LIQUEFACTION CONCLUSIONS

Because no suspected liquefiable soil layer was found at Boring B-1 and B-2, the potential of liquefaction at the site is minimal.

INUNDATION POTENTIAL

The subject site is located at 200-210 North Bascom Avenue in San Jose, California. According to the Limerinos and others, 1973 report, the proposed building 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. Based on the laboratory testing results, the native surface soil at the project site has been found to have a low expansion potential when subjected to fluctuations in moisture.

3. The exterior of the building pad should be graded to permit proper drainage and diversion of water away from the building foundation.

4. The proposed building should be supported on conventional spread foundation with concrete slab-on-grade.

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

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

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

8. All earthwork including grading, backfill, and foundation excavation 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 to schedule 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 should be removed from the subject 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 building 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 soil or approved 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 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 removing existing pavement aggregate base from the surface, the building pad and parking/driveway area (improved area) 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, native soil should be compacted to not less than 90% relative maximum density and 95% in parking/driveway using ASTM D1557 procedure over the entire improved area, 5 feet beyond the perimeter of the building pad, parking/driveway area, and 3 feet beyond parking/driveway area.

7. All engineered fill or imported soil 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. The Class II Baserock (baserock) should be compacted to not less than 95% relative maximum density. Before compaction begins, the subgrade and/or fill material 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. 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.

9. 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.

10. Parking/driveway asphalt pavement section designs are presented in Table II. Rigid concrete and paver pavement section designs are presented in Table III and IV.

11. All imported soil 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).

12. Silicon Valley Soil Engineering (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.

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

WATER WELLS

14. Any water wells and/or monitoring wells on the site which are to be abandoned, should be capped according to the requirements of the Santa Clara Valley Water District (Valley Water). 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

15. The proposed four-story office building should be supported on conventional continuous perimeter and isolated interior spread foundation.

16. Conventional spread foundation should be founded at a minimum depth of 42 inches below finished subgrade pad elevation with 24 inch minimum width. Under these conditions, the allowable bearing capacity is 3,500 psf for all continuous perimeter and isolated spread footings. Trash enclosure footings can be founded at a minimum depth of 18 inches with allowable bearing capacity of 2,500 psf.

17. The footing bottoms should be compacted with jumping jack prior to rebar and form work placement.

18. Skin friction piers for light poles, boundary wall or associated improvements should be embedded at a minimum depth of 6 feet below the lowest adjacent grade with minimum diameter of 18 inches. The allowable skin friction value for this type of foundation is 500 psf.

19. The top 1 foot of the pier should be neglected in the calculation of the allowable skin friction force and passive resistance. In designing for allowable resistive lateral earth pressure (passive) of 250 pounds equivalent fluid pressure may be used with the resultant acting at the third point with form factor of 2.

20. A friction coefficient of 0.3 should be used. The aforementioned bearing values are for dead plus live loads and may be increased by one-third for short term seismic and wind loads.

21. We estimate that post-construction differential settlement will be less than quarter inch per 50 feet span.

22. We highly recommend that a representative from our office be present to make any field adjustments during the excavation of footings and/or drilling of the piers.

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

2019 CBC SEISMIC VALUES

24. Chapter 16 of the 2019 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 Latitude	37.327277° N.
Site Longitude	121.931692° W.
Site Class (ASCE 7-16)	D
Risk Category	I,II,III
0.2-second Mapped Spectra Acceleration[1], S_S	1.500g
1-second Mapped Spectra Acceleration[1], S_1	0.600g
Short-Period Site Coefficient, F_a	1.0
Long-Period Site Coefficient, F_V	1.7
0.2-second Period, Maximum considered Earthquake Spectral Response Acceleration, S_{MS} ($S_{MS} = F_a S_S$)	1.500g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration, S_{M1} ($S_{M1} = F_V S_1$)	1.020g
0.2-second Period, Designed Spectra Acceleration, S_{DS} ($S_{DS} = 2/3 S_{MS}$)	1.000g
1-second Period, Designed Spectra Acceleration, S_{D1} ($S_{D1} = 2/3 S_{M1}$)	0.680g

[1] For Site Class B, 5 percent damped.
*2019 CBC

CONCRETE SLAB-ON-GRADE CONSTRUCTION

25. Based on the laboratory testing results of the near-surface soil, the native soil on the site was found to have a low expansion potential when subjected to fluctuation in moisture. The native subgrade should be moisture conditioned and compacted to at least 90% relative maximum density.

26. A minimum of 5 inches of Class II Baserock or ¾ inch crushed rock should be placed on the compacted finished subgrade for the office building. The ¾ inch crushed rock (recycled crushed asphalt concrete is not acceptable) should be compacted in-place with vibratory plate. The baserock material should be compacted to at least 95% relative maximum density.

27. The concrete slab should have a minimum thickness of 5 inches, 6 inches thick for vehicle ground floor parking and reinforced with minimum of No. 4 rebar with maximum spacing of 18 inches on-center both ways. Structural engineer should verify and detail reinforcement. If the concrete slab would receive a floor covering or sealant, a Stego 15-mil vapor barrier between the rock layer and concrete slab should be used. The vapor barrier should be taped at the seams and/or mastic sealed at the protrusions if sand is not used.

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

RETAINING WALLS

29. Any facilities that will retain a soil mass above grade should 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 should be designed for the earth pressure resulting from 60 pounds equivalent fluid pressure.

30. 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 subgrade soil should be neglected for computation of passive resistance.

31. 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.

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

33. Any retaining walls or elevator pit walls associated with the building should be waterproofed.

34. Drainage should be provided behind the retaining wall. The drainage system should consist of perforated (subdrain) pipe placed at the base of the retaining wall and 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 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.

35. As an alternative to the drain rock and fabric, Miradrain 2000 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 pipe should drain to an appropriate discharge facility.

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

EXCAVATION

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

38. 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.

DRAINAGE

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

40. The final exterior grade adjacent to the 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.

41. Utility lines that cross under or through slab, footings, or walls should be completely sealed as necessary, to prevent moisture intrusion into the areas under the slab and footings.

42. 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 landscape grade adjacent to

the foundation should be sloped away from the structure at a minimum of 5 percent.

43. If the subgrade in the landscaping area is moderately to highly expansive, proper drainage should be provided in the landscaping area adjacent to the building foundation. A drip irrigation system is preferable. If the sprinkler system is located adjacent to the building perimeter or concrete walkway, a moisture cut-off barrier should be provided.

44. Based on laboratory test results of the near surface soil at the subject site, we estimated that the infiltration rate is approximately 0.5 inch per hour ($K_{SAT} = 3.5x10^{-4}$ cm/sec). This rate can be used in the design of the bioretention system for on-site storm drainage.

45. Bioretention systems should not be located within 10 feet from building foundation or 5 feet from parking/driveway edge and should not undermine parking curbs. If the bioretention is no more than 3 feet from parking/driveway curb, the bioretention should be lined with impermeable liner (15 mil plastic or thicker) to above the overflow elevation and waterproofed if adjacent to building. Concrete curbs shall be deepened for proper support. Biosoil mix and gravel should not be used for calculation to support curbs.

ON-SITE UTILITY TRENCHING

46. Utility trenches within the public right-of-way should be excavated, bedded, and backfilled in accordance with local or governing jurisdiction requirements.

47. All utility lines including plumbing should be bedded with at least 6 inches over the pipe or conduit with 1/4, 3/8 or 3/4 inch crushed rock or

well graded sand conforming to pipe manufacture's requirements. Sand and gravel should be compacted in-place.

48. The remaining excavated area should be backfilled with native on-site material or approved imported fill and compacted to at least 90% relative maximum density. Backfill should be placed in uniform 8 to 12 inch lifts and compacted. 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.

49. The utility trenches running parallel to the building foundation should not be located in an influence zone that will undermine the stability of the foundation. The influence zone is defined as the imaginary line extending at the outer edge of the footing at a downward slope of 1:1 (one unit horizontal distance to one unit vertical distance). If the utility trenches were encroaching the influence zone, the encroached area should be stabilized with cement sand slurry (minimum of 75 psi compressive strength).

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

PAVEMENT DESIGN

51. 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 shown on Figure 6. The following alternate pavement 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).

52. Alternate pavement section designs, which satisfy the State of California Standard Design Criteria, and above traffic indices, are presented in Table II. Rigid and paver pavement section designs are presented in Table III and IV.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations presented herein are based on the soil conditions revealed by our test boring(s) 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 boring(s) is/are 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 boring(s) 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. Stormwater management and calculations are not part of our investigation or scope.

8. 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.

9. 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

Borcherdt R.D., Gibbs J. F., Lajoie K.R., 1977 – Maps showing maximum earthquake intensity predicted in the southern San Francisco Bay Region, California, for large earthquakes on the San Andreas and Hayward faults. U.S.G.S. MF-709.

Limerinos J.T., Lee K.W., Lugo P.E., 1973 – Flood Prone Areas in the San Francisco Bay Region, California U.S.G.S. Open file report.

Rogers T.H., and Williams J.W., 1974 – Potential seismic hazards in Santa Clara County, California Special Report, No. 107, California Division of Mines and Geology.

USGS (2002). 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].

OSHPD, U.S. Seismic Design Maps, https://seismicmaps.org.

2019 (CBC) California Building Code, Title 24, Part 2.

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)	Liquid Limit L.L.	Plasticity Index P.I.
1-1	3	21.4	93.5	0.8	12		
1-2	5	19.2	93.9				
1-3	10	21.5	101.4				
1-4	15	20.1	101.4				
1-5	20	20.5	107.5				
1-6	25	13.9	96.3				
1-7	30	35.4	85.6				
1-8	35	35.7	84.0			46	25
1-9	40	32.3	91.3			44	24
1-10	45	13.1	134.8				
1-11	50	34.4	87.4			45	24
2-1	3	22.7	94.3				
2-2	5	26.3	93.4				
2-3	10	27.6	95.5				
2-4	15	18.9	100.6				
2-5	20	20.0	96.4				
2-6	25	20.6	106.0				
2-7	30	29.9	90.0				
2-8	35	33.6	86.4			45	25
2-9	40	30.8	95.2			43	24
2-10	45	14.4	130.1				
2-11	50	28.9	101.3			44	25

TABLE II

PROPOSED ASPHALT PAVEMENT SECTIONS

Location: Proposed Building
200-210 North Bascom Avenue
San Jose, California

	PARKING STALLS			DRIVEWAY		
Design R-Value	10.0			10.0		
Traffic Index	4.5			5.5		
Gravel Equivalent	16.0			19.0		
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	8.0"	7.0"	6.0"	11.0"	10.0"	9.0"
Subgrade soil scarified & compacted to at least 95% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"

TABLE III

PROPOSED CONCRETE PAVEMENT SECTIONS

Location: Proposed Building
200-210 North Bascom Avenue
San Jose, California

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

* Including trash enclosures, stress pads, and valley gutters. Rebar No. 4 at 18" maximum spacing on-center both ways or as recommended by Structural Engineer. Maximum control joints at 10' by 10' or as recommended by Structural Engineer. Vertical curbs should be keyed at least 3 inches into pavement subgrade.

** Rebar No. 3 at 18" maximum spacing on-center both ways.

TABLE IV

PROPOSED PAVER PAVEMENT SECTIONS

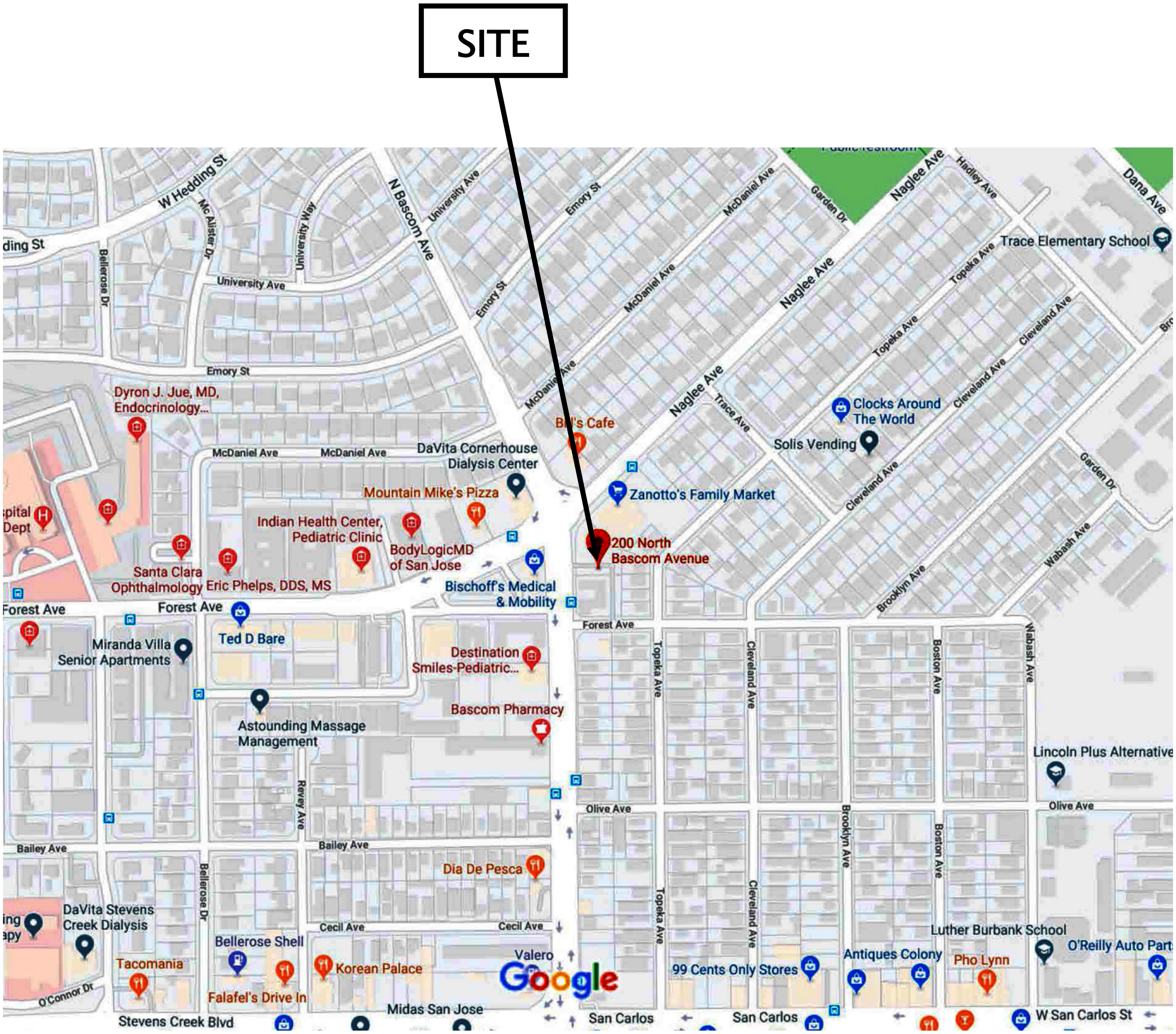
Location: Proposed Building
200-210 North Bascom Avenue
San Jose, California

	DRIVEWAY/PARKING AREA*			
Recommended Paver Pavement Sections:	1A	1B*	2A**	2B**
Vehicular Rated Pavers	Min. 3.25" ± Permeable Paver Parking Stalls With subdrain	Min. 3.25" ± Permeable Paver Parking Stalls	Min. 3.25" ± Permeable Paver Driveway With subdrain	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	12.0"	4.0"	16.0"	6.0"
ASTM No. 2 Stone	---	12.0"	---	12.0"
Subgrade scarified & compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"

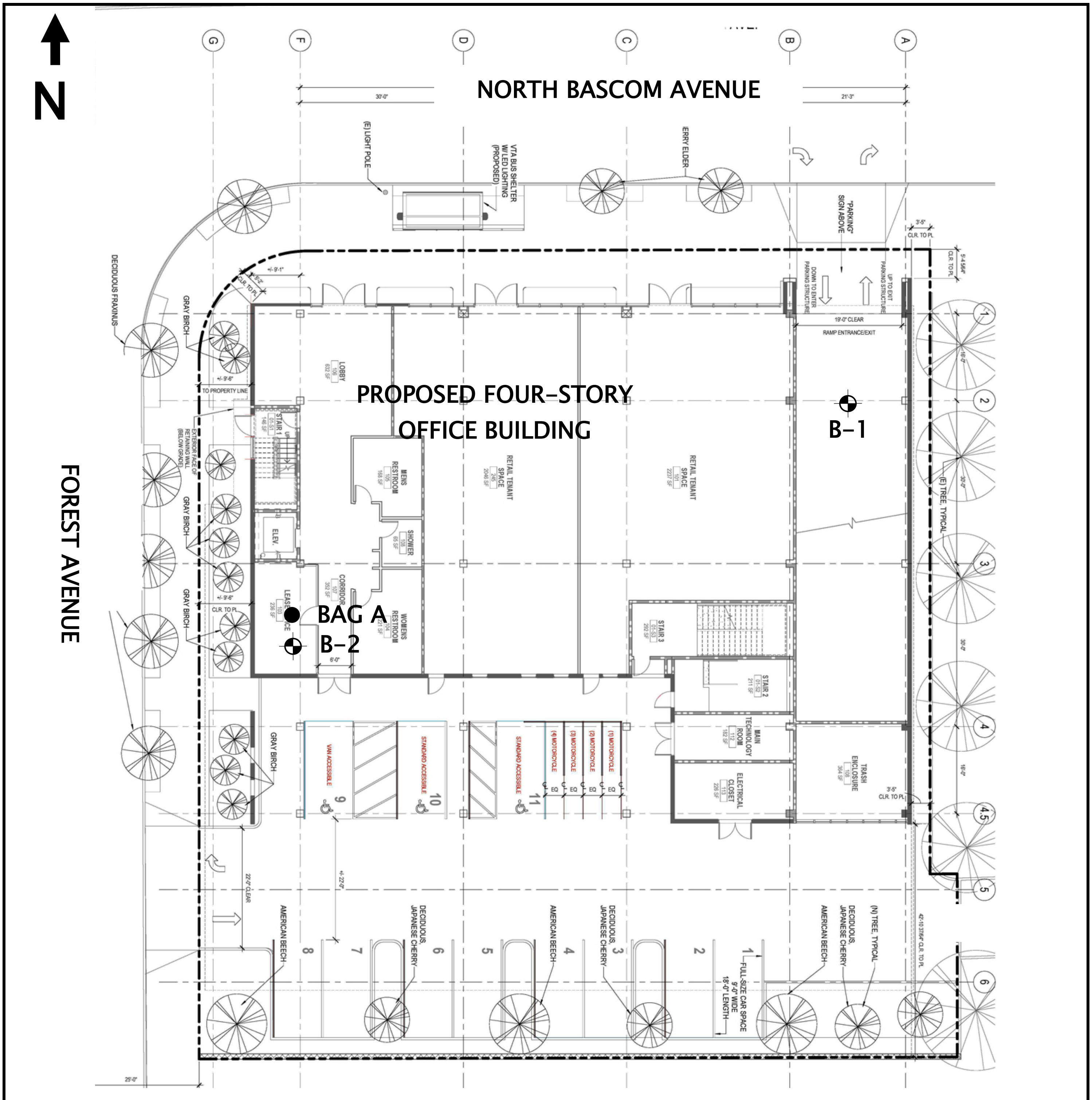
* The subgrade should be lined with Mirafi 140N Filter Fabric and Tensar BX1100 Geogrid or equivalent. Subgrade should be sloped at a minimum of 2% towards the subdrain system, if necessary. If subdrain is allowed to be notched in the subgrade, the subdrain trench should be at least 12 inches wide and 6 inches below the finished subgrade elevation and the walls and bottom should be lined with filter fabric. The subdrain system should consist of a 4-inch diameter perforated pipe schedule 40 or equivalent surrounded by ASTM No. 57 Stone (¾ inch drain rock). Or, the subdrain pipe may be required to be within the ASTM No. 2 Stone section. The drainage system should drain to a discharge facility. The pavers should be bordered with a concrete curb/band to avoid water infiltration into non-permeable parking areas. Typically, minor maintenance would be required during the life of the pavers.

** Support fire apparatus of 75,000 lbs.

FIGURES

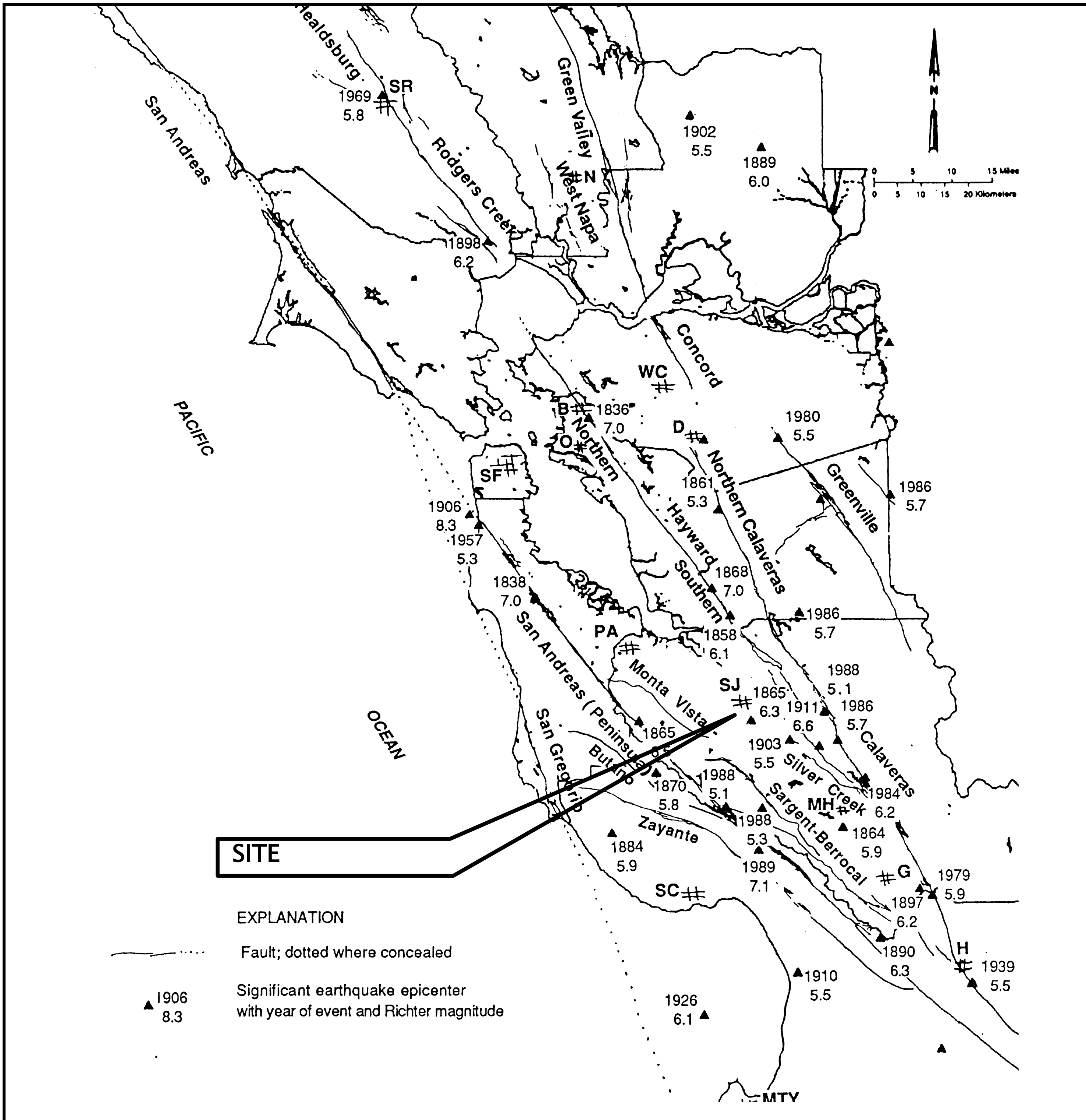


Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**VICINITY MAP** Proposed Building 200-210 North Bascom Ave. San Jose, California	File No.: SV2017 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 1 January 2020



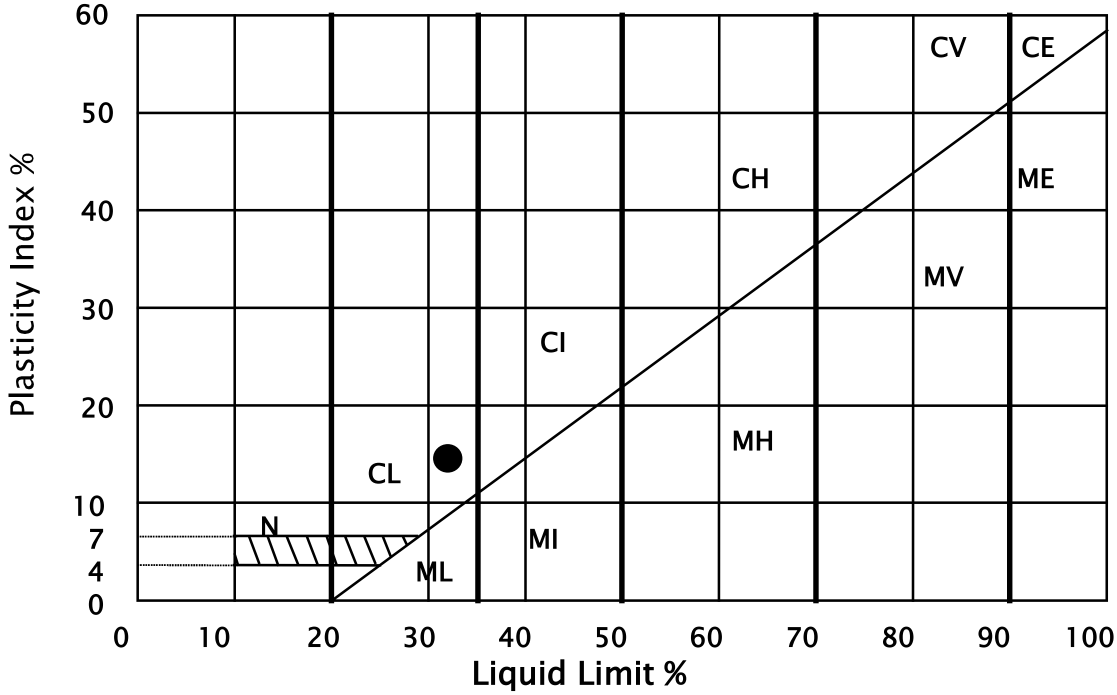
NOTE: ⊕ DENOTES APPROXIMATE EXPLORATORY BORING LOCATION
● DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**SITE PLAN** Proposed Building 200-210 North Bascom Ave. San Jose, California	File No.: SV2017 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 2 January 2020



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**FAULT LOCATION MAP** Proposed Building 200-210 North Bascom Ave. San Jose, California	File No.: SV2017 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 3 January 2020

PLASTICITY CHART

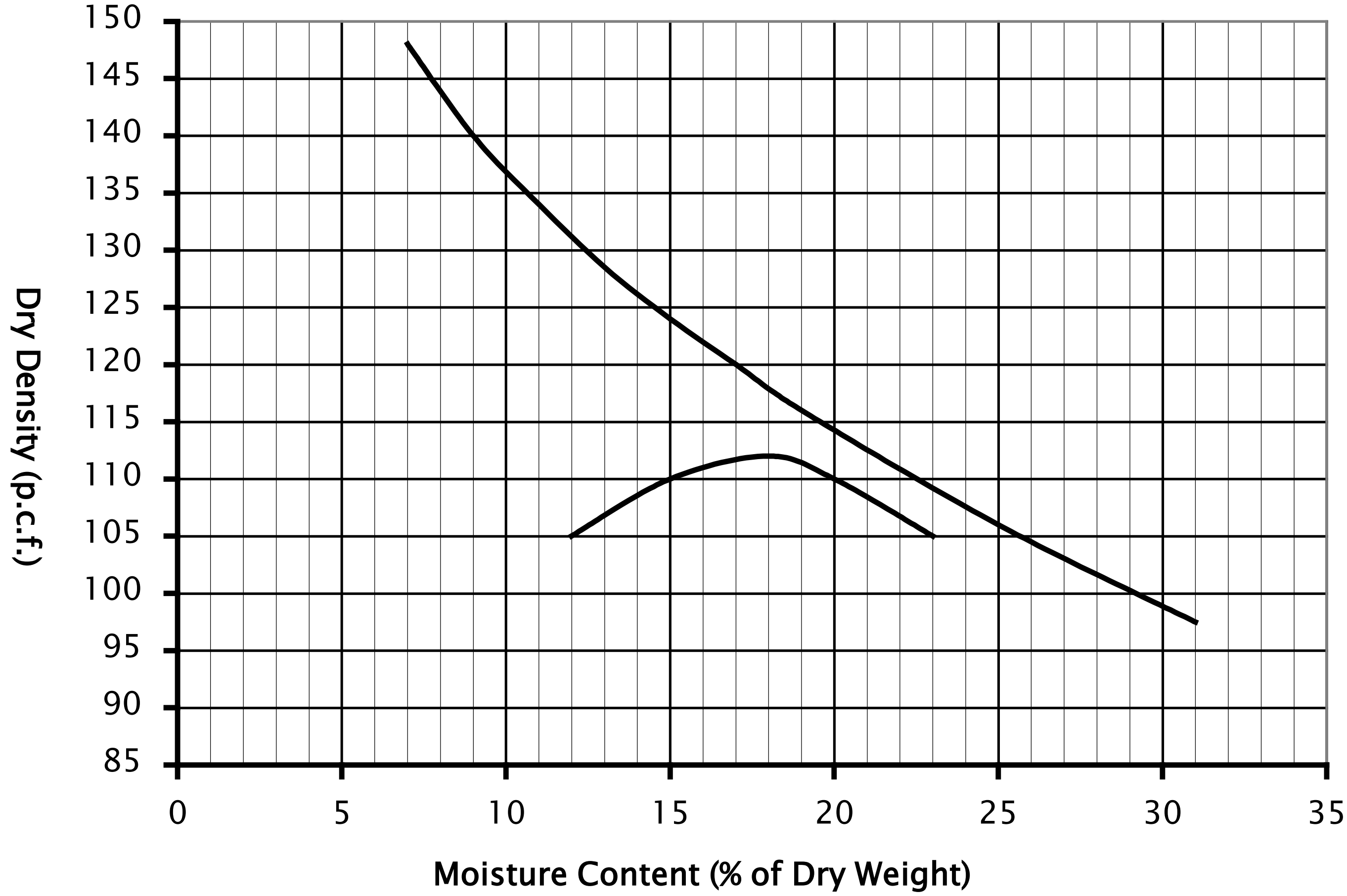


PLASTICITY DATA

Key Symbol	Sample No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	32	15	CL

*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 Building 200-210 North Bascom Ave. San Jose, California	File No.: SV2017 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 4 January 2020



SAMPLE: A

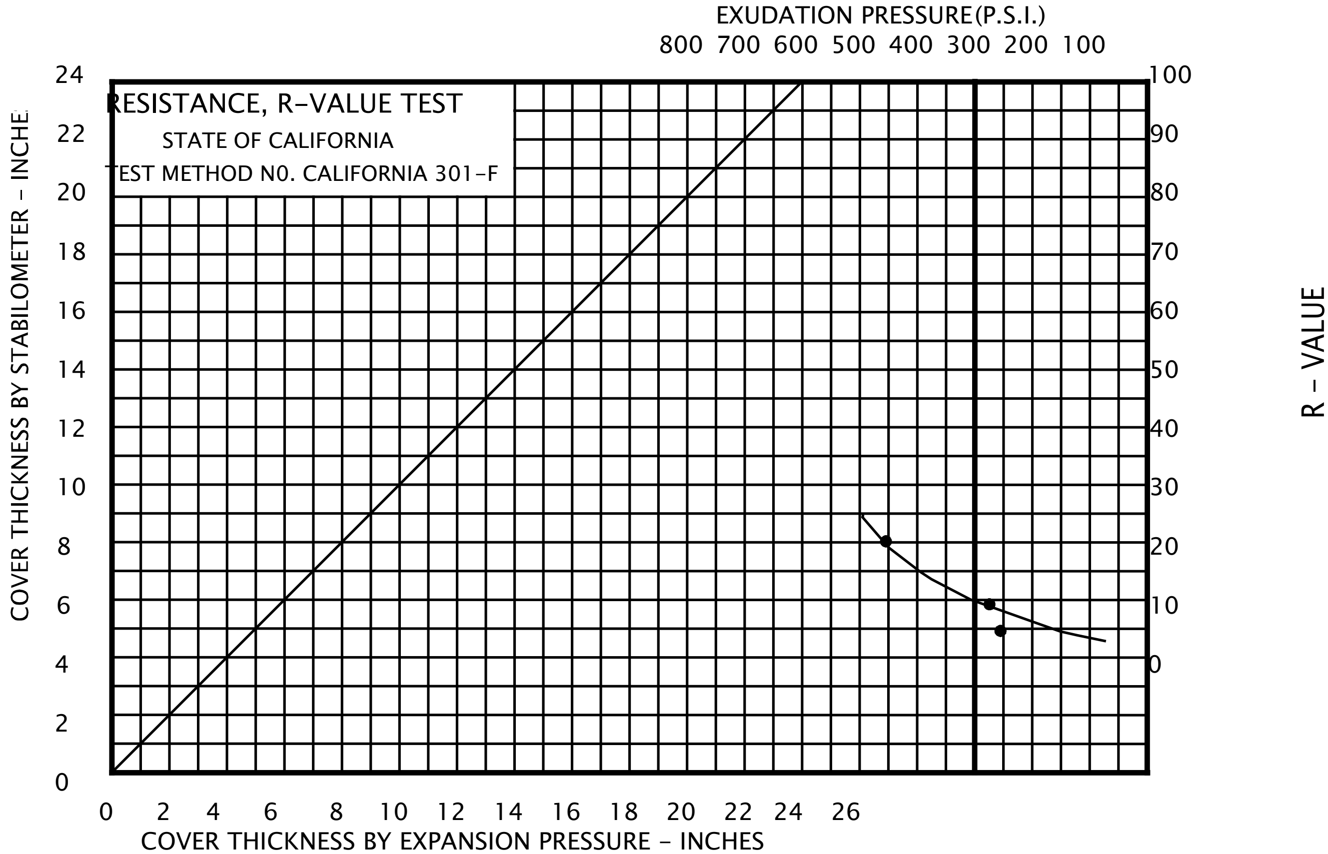
DESCRIPTION: Dark Olive Brown Silty CLAY

LABORATORY TEST PROCEDURE: ASTM D1557

MAXIMUM DRY DENSITY: 112.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 18.0 %

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**COMPACTION TEST A** Proposed Building 200-210 North Bascom Ave. San Jose, California	File No. SV2017 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 5 January 2020



SAMPLE: A
DESCRIPTION: Dark Olive Brown Silty CLAY

SPECIMEN	A	B	C
EXUDATION PRESSURE (P.S.I.)	149.0	275.0	452.0
EXPANSION DIAL (.0001")	9.0	14.0	20.0
EXPANSION PRESSURE (P.S.F.)	45.0	76.0	94.0
RESISTANCE VALUE, "R"	5.0	8.0	20.0
% 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	= (10)		

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**R-VALUE TEST** Proposed Building 200-210 North Bascom Ave. San Jose, California	File No. SV2017	FIGURE
		Drawn by: V.V.	6
		Scale: NOT TO SCALE	January 2020

APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

KEY TO LOG OF BORING

EXPLORATORY BORING LOGS (B-1 AND B-2)

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. SV2017

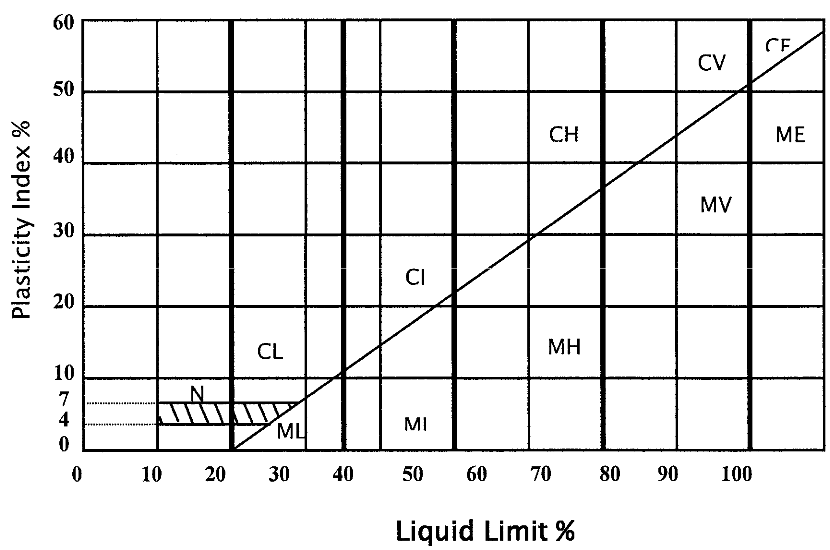
METHOD OF SOIL CLASSIFICATION CHART

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 Building **Project Location:** 200-210 N. Bascom Avenue, San Jose, California **Project Number:** SV2017	**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)
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)
Clayey SAND to Sandy CLAY (SC-CL)
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 Building **Project Location:** 200-210 N. Bascom Avenue, San Jose, California **Project Number:** SV2017	**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 **01/16/20**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **50.0 feet**
		Approximate Surface Elevation **feet**
Groundwater Level and Date Measured **34 feet (01/16/20)**	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												
0.25				Asphalt		3.0 inches of Asphalt Concrete (AC)						
0.67				CL		5.0 inches of Aggregate Base (AB)						
2		1-1	36	SC-CL		Dark Olive Brown Silty CLAY Moist, stiff	21.4	93.5	0.8	12		
		1-2	51			Olive Brown Gravelly Sandy Silty CLAY Moist, very stiff	19.2	93.9				
5				ML		Tan Yellow Brown Sandy SILT Moist, very stiff						
9.5		1-3	68			Color changed to dark olive gray	21.5	101.4				
10				SC-CL		Dark Olive Gray Mottled with Brown Sandy Silty CLAY Moist, very stiff						
		1-4	80			Color changed to tan and gray	20.1	101.4				
15												
		1-5	65				20.5	107.5				
20						Color changed to tan brown						
		1-6	38				13.9	96.3				
25												
		1-7	19				35.4	85.6				
30												

Project: Proposed Building
Project Location: 200-210 N. Bascom Avenue, San Jose, California
Project Number: SV2017

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				SC-CL		Tan Brown Mottled with Olive Gray Sandy Silty CLAY Moist, very stiff Stabilized at drilling completion ▼ First encountered ▽						
		1-8	33				35.7	84.0			46	25
35				SP		Brown SAND Wet, dense SAND: medium-grained, poorly graded						
38				CL		Olive Gray Sandy Silty CLAY Moist, very stiff						
40		1-9	37				32.3	91.3			44	24
43				GP		Gray Sandy GRAVEL Wet, dense GRAVEL: 1.5 inches maximum diameter Sub-rounded, poorly graded						
45		1-10	51				13.1	134.8				
47				CL-CH		Dark Gray Silty CLAY Moist, very stiff						
50		1-11	35				34.4	87.4			45	24
						Boring terminated at 50.0 feet						
55												
60												
65												

Project: Proposed Building **Project Location:** 200-210 N. Bascom Avenue, San Jose, California **Project Number:** SV2017	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-2 **Sheet 1 of 2**

Date(s) Drilled **01/16/20**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **50.0 feet**
		Approximate Surface Elevation **feet**
Groundwater Level and Date Measured **34 feet (01/16/20)**	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		3.0 inches of Asphalt Concrete (AC)						
						3.0 inches of Aggregate Base (AB)						
				CL		Dark Olive Brown Silty CLAY Moist, stiff						
		2-1	56				22.7	94.3				
		2-2	24				26.3	93.4				
5				SC-CL		Olive Brown Gravelly Sandy Silty CLAY Moist, very stiff						
9		2-3	26	SC-CL		Dark Olive Gray Mottled with Brown Sandy Silty CLAY Moist, very stiff	27.6	95.5				
10												
						Color changed to tan and gray						
		2-4	81				18.9	100.6				
15												
		2-5	68				20.0	96.4				
20						Color changed to tan brown						
		2-6	43				20.6	106.0				
25												
		2-7	22				29.9	90.0				
30												

Project: Proposed Building **Project Location:** 200-210 N. Bascom Avenue, San Jose, California **Project Number:** SV2017	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-2 **Sheet 2 of 2**

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				SC-CL		Tan Brown Mottled with Olive Gray Sandy Silty CLAY Moist, very stiff						
		2-8	34			Stabilized at drilling completion ▼ First encountered ▽	33.6	86.4			45	25
35				SP		Brown SAND Wet, medium dense SAND: medium-grained, poorly graded						
		2-9	35	CL		Olive Gray Sandy Silty CLAY Moist, very stiff	30.8	95.2			43	24
40												
		2-10	55	GP		Gray Sandy GRAVEL Wet, dense GRAVEL: 1.5 inches maximum diameter Sub-rounded, poorly graded	14.4	130.1				
45												
		2-11	46	CL-CH		Dark Gray Silty CLAY Moist, very stiff	28.9	101.3			44	25
50						Boring terminated at 50.0 feet						
55												
60												
65												