REPORT TO

MR. DAN CUNNINGHAM
PALO ALTO, CALIFORNIA

FOR

PROPOSED OFFICE BUILDING

3241 PARK BOULEVARD
PALO ALTO, CALIFORNIA

GEOTECHNICAL INVESTIGATION

APRIL 2020

PREPARED BY

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

SILICON VALLEY SOIL ENGINEERING

GEOTECHNICAL CONSULTANTS

File No. SV1809
April 23, 2020

Mr. Dan Cunningham
3197 Park Boulevard
Palo Alto, CA 94306

Subject: Proposed Office Building
3241 Park Boulevard
Palo Alto, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Cunningham:

Pursuant to your request, we are pleased to transmit herein the results of our geotechnical investigation for the proposed office building. The subject site is located at 3241 Park Boulevard in Palo Alto, California.

Our findings indicate that the site is suitable for the proposed development provided the recommendations contained in this report are carefully followed. Our field reconnaissance, drilling, sampling, and laboratory testing of the surface and subsurface material evaluate 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

Sean Deivert
Project Manager

Vien Vo, P.E.

SV1809.GI2/Copies: 4 to Mr. Dan Cunningham

2391 Zanker Road, Suite 350 • San Jose, California 95131 • (408) 324-1400

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INTRODUCTION

Per your authorization, Silicon Valley Soil Engineering (SVSE) conducted a geotechnical investigation. The purpose of this 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 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.

PROJECT LOCATION AND DESCRIPTION

The project site is located at 3241 Park Boulevard in Palo Alto, California (Figure 1 – Vicinity Map). Park Boulevard bounds the subject site to the southwest, Matadero Creek concrete-lined channel with approximately 12 feet high retaining wall to the northwest and northeast, and existing City of Palo Alto electric power installation to the southeast. At the time of our investigation, the site was an irregular shaped, relatively flat parcel of land occupied by an automotive repair shop building. Based on the preliminary plan prepared by the Project Architect for the subject site, the proposed development will include the demolition of the existing structure and the construction of a two-story office building with underground parking lifts and associated improvements. Location of the proposed building and our exploratory soil borings are shown on the Figure 2 – Site Plan.

FIELD INVESTIGATION

After considering the nature of the proposed development and reviewing available data on the area, a field investigation was conducted at the project site under the direction of our geotechnical engineer. It included a site reconnaissance to detect any unusual surface features, and the drilling of four exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on August 28, 2018 to the depths of 10 feet to 50 feet below the existing ground surface elevation. The borings were drilled with a truck-mounted drill rig using 8-inch diameter hollow stem augers. The approximate boring locations are shown on Figure 2.

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 logs as penetration resistance.

In addition, disturbed bulk samples of the near-surface soil were collected for laboratory analyses. The Exploratory Boring Log contained in the Appendix are a graphic representation of the encountered soil profile; and also show the depths at which the relatively undisturbed soil samples were obtained.

LABORATORY INVESTIGATION

A laboratory-testing program was performed to determine the physical and engineering properties of the soils underlying the site. 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). The strength parameters of the foundation soils were determined from direct shear tests that were performed on selected relatively undisturbed soil samples (Table I). Atterberg Limits tests were also performed on the near-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). Laboratory compaction tests of the native soil material were performed to determine the maximum dry density per the ASTM D1557 test procedure (Figure 5). One R-Value test was performed on a near surface soil sample for pavement section design recommendations (Figure 6). One soil sample collected was submitted to Cooper Testing Lab for corrosivity analysis (Page 26 & 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, from the pavement surface consists of 10.0 inches of Asphalt Concrete (AC) over 12.0 inches of Aggregate Base (AB). Below the pavement surface to a depth of 3 feet, a medium brown, moist, very stiff sandy silty clay layer was encountered. This is the retaining wall backfill. From the depths of 3 feet to 7 feet, the soil became brown, moist, very stiff sandy silt. This is the retaining wall backfill. From the depths of 7 feet to 12 feet, a light olive brown, moist, very stiff silty clay layer was encountered. From the depths of 12 feet to 23 feet, the soil became tan brown, moist, very stiff clayey silt. From the depths of 23 feet to 40 feet, a tan brown, moist, very stiff sandy clay layer was encountered. From the depths of 40 feet to the end of the boring at 50 feet, a bluish gray, moist, very stiff clayey silt layer was encountered. Similar soil profiles were encountered in other borings.

Groundwater was initially encountered in Boring B-1 and B-4 at the depth of 21 feet and stabilized to static level of 17 feet and 18 feet respectively after the completion of the drilling operation. It should be noted that the groundwater table would fluctuate as a result of seasonal changes and hydrogeologic variations such as groundwater pumping and/or recharging. A detailed description of the soil profiles encountered is presented in Exploratory Boring Logs contained in the Appendix.

GENERAL GEOLOGY

The site lies in the Santa Clara Valley, which is part of the Coast Ranges geological province. The Santa Clara Valley occupies the structural trough formed by two northwest trending mountain ranges; the Santa Cruz Mountains to the southwest of the valley and the Diablo Range to the northeast. The Diablo Range is predominantly composed of Franciscan Formation, which is uppermost Jurassic to lower Upper Cretaceous eugosynclinal assemblage. The Santa Cruz Mountains are predominantly composed of material formed of Cenozoic shelf and slope deposits. A thick blanket of latest Cretaceous and Tertiary clastic sedimentary rocks and isolated intrusions of serpentine covers large parts of the province. Folds, thrust faults, steep reverse faults, and strikeslip faults developed as a consequence of Cenozoic deformations that occur very often within the province and some of them are continuing today (CDMG; 1966).

Sedimentary marine strata alternating with non-marine strata record the Quaternary history of the region. 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 Santa Clara Valley and most of the strata are of continental origin characterized as alluvial and fluvial materials. The project site is underlain by fluvial 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, a liquefaction analysis was performed.

A. GROUNDWATER

Groundwater was initially encountered in Boring B-1 at a depth of 21 feet and rose to a static level of 17 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 111 [Seismic Hazard Evaluation of the Palo Alto 7.5-Minute Quadrangle, San Mateo and Santa Clara Counties, California. 4/18/06. Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 15 feet below ground elevation. Therefore, this depth of the groundwater table will be used for the liquefaction analysis.

B. SUSPECTED LIQUEFIABLE SOIL LAYERS

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 (LL) 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.

The results from our exploratory boring show that the subsurface soil material in Boring B-1 to the depth of 50 feet consists of very stiff sandy silty clay to very stiff sandy silt to very stiff silty clay to very stiff clayey silt to very stiff sandy clay to very stiff clayey silt. The following is the determination of the liquefiable soil for each soil layer in Boring B-1.

1. The very stiff sandy silty clay layer from the surface to the depth of 3 feet is not liquefiable soil because it is above the highest expected groundwater table (15 feet).

2. The very stiff sandy silt layer from the depths of 3 feet to 7 feet is not liquefiable soil because it is above the highest expected groundwater table (15 feet).

3. The very stiff silty clay layer from the depths of 7 feet to 12 feet is not liquefiable soil because it is above the highest expected groundwater table (15 feet).

4. The very stiff clayey silt layer from the depths of 12 feet to 15 feet is not liquefiable soil because it is above the highest expected groundwater table (15 feet).

5. The very stiff clayey silt layer from the depths of 15 feet to 23 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

    - Sample No. 1-5 (20 feet) - [PI > 18; PI = 19 and MC = 27.5% < 80% LL = 28.0%; LL = 35]

6. The very stiff sandy clay layer from the depths of 23 feet to 40 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

    - Sample No. 1-6 (25 feet) - [PI > 18; PI = 22 and MC = 18.9% < 80% LL = 32.8%; LL = 41]

- Sample No. 1-8 (35 feet) – [PI > 18; PI = 23 and MC = 25.4% < 80% LL = 33.6%; LL = 43]

7. The very stiff clayey silt layer from the depths of 40 feet to the end of the boring at 50 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

- Sample No. 1-10 (45 feet) – [PI > 18; PI = 26 and MC = 25.8% < 80% LL = 37.6%; LL = 47]

- Sample No. 1-11 (50 feet) – [PI > 18; PI = 27 and MC = 26.0% < 80% LL = 39.2%; LL = 49]

In summary, there is no liquefiable soil layer underlying the subject site.

C. CONCLUSION

Since there is no liquefiable soil layer underlying the subject site, the potential for liquefaction is minimal.

INUNDATION POTENTIAL

The subject site is located at 3241 Park Boulevard in Palo Alto, 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 years flood (Limerinos; 1973).

CONCLUSIONS

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

2. Since there is no liquefiable soil layer underlying the subject site, the potential for liquefaction is minimal.

3. The site is not located in a geologic hazard zone.

4. 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).

5. 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. Therefore, we recommend that the concrete slab-on-grade, if any, should be underlain by a minimum of 17 inches of non-expansive soil material including the rock section.

6. The building pad should be elevated above the adjacent ground surface to permit proper drainage and diversion of water away from the building foundation.

7. The proposed building should be supported on conventional spread foundation. The parking lift should be supported on structural slab with pier foundation and mat slab foundation. Any structures within 20 feet from the creek bank concrete retaining wall (setback zone) should be supported on pier foundation to prevent advert impact of the existing creek bank retaining wall, stability of the creek bank slope, and erosion.

8. Trees to be planted adjacent to existing creek bank retaining wall should not compromise the integrity of the retaining wall or drainage system behind the retaining wall. The roots from proposed trees should not be allowed to encroach behind the existing retaining wall. Therefore, a root barrier such as Typar BioBarrier Root Control System or equivalent approved by Arborist should be utilized.

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

10. Reference to our report should be stated in the grading and foundation plans that include the geotechnical investigation file number and date.

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

12. All earthwork including grading, backfilling, foundation excavation and pier 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 operations 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, which will not be incorporated in the final development shall be removed from the project site prior to any grading operations.

3. The depressions left by the removal of subsurface structures should be cleaned of all debris, backfilled and compacted with clean, native or acceptable on-site 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, if any, and after stripping the organic material and pavement section from the soil, the original site subgrade should be scarified by machine to a depth of 12 inches and thoroughly cleaned of vegetation and other deleterious matter. The aggregate base section can be re-used as non-expansive fill material.

6. After stripping, scarifying and cleaning operations, subgrade soil material should be compacted to not less than 90% relative maximum density using ASTM D1557 procedure with 3% over optimum moisture over the entire

building pad, 5 feet beyond the perimeter of the pad, and 3 feet beyond the edge of the driveway/parking 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. This should extend a minimum of 5 feet beyond the perimeter of the pad, if permitted. The baserock, if any, should be compacted to not less than 95% relative maximum density. 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. 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. 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).

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.

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 that are to be abandoned on the site 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 proposed office building should be supported on conventional continuous and isolated spread foundation.

15. The building foundation setback should be at least 20 feet from the adjacent creek concrete retaining wall. Any structure encroach into the setback zone should be supported on pier foundation – see Underground Parking Lift Foundation Design Criteria section.

16. Conventional spread footings should be founded at a minimum depth of 24 inches below finished subgrade pad elevation and the corresponding allowable bearing capacity is 2,500 psf.

17. The above 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.

18. 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.

UNDERGROUND PARKING LIFT FOUNDATION DESIGN CRITERIA

19. The proposed parking lift foundation should be supported by structural slab and additionally supported by skin friction drilled concrete piers if within 20 feet horizontal distance from existing retaining wall to prevent impact or compromise structural integrity of the existing creek bank retaining wall. Structural slab with pier foundation should be utilized to prevent piers from encroaching into the retaining wall foundation.

20. Skin friction piers shall have a minimum diameter of 18 inches and penetrate a minimum of 10 feet below bottom elevation of the existing retaining wall (14 feet minimum total from bottom of parking lift slab elevation) with 8 feet horizontal maximum spacing, on-center. These piers can be designed with an allowable skin friction value of 600 psf. The top 4 feet of the pier (elevation above the bottom of the existing retaining wall to parking lift foundation) should be neglected in the calculation of the allowable skin friction force and passive resistance.

21. 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.

22. All piers should be reinforced with at least four No. 5 rebars which shall run the entire length of the piers.

23. Due to shallow groundwater, the piers will be required to be tremied during concrete placement.

24. If the parking lift foundation is beyond 20 feet horizontal distance from existing retaining wall, the parking lift can be supported with structural or mat slab foundation without pier foundation.

25. The structural/mat slab should be a minimum of 8 inches in thickness with the allowable contact pressure of 2,000 psf. The modulus of subgrade reaction can be taken as 110 pci.

26. The mat slab should be underlain with 6 inches of ¾ inch wash crushed rock and the subgrade should be competent and compacted.

27. The parking lift concrete slab and retaining walls should be waterproofed.

28. The aforementioned bearing values are for dead plus live loads and may be increased by one-third for short term seismic and wind loads.

29. The project structural engineer responsible for the foundation design shall determine the final design of the foundations and reinforcing required. The design of the structure 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

30. 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	37.424357° N.
Site Longitude	122.134929° W.
Site Class (ASCE 7-16)	D
Risk Category	I,II,III
0.2-second Mapped Spectra Acceleration[1], S_s	1.728g
1-second Mapped Spectra Acceleration[1], S_1	0.614g
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} ($S_{MS} = F_aS_S$)	1.728g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration, S_{M1} ($S_{M1} = F_VS_1$)	1.044g
0.2-second Period, Designed Spectra Acceleration, S_{DS} ($S_{DS} = 2/3S_{MS}$)	1.152g
1-second Period, Designed Spectra Acceleration, S_{D1} ($S_{D1} = 2/3S_{M1}$)	0.696g

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

CONCRETE SLAB-ON-GRADE CONSTRUCTION

31. 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.

32. Therefore, we recommend that the concrete slab-on-grade should be underlain by a minimum of 17 inches of non-expansive soil material

including the rock section. The original subgrade and fill soil material should be compacted to at least 90% relative maximum density.

33. A minimum of 5 inches of ¾ inch crushed rock or Class II Baserock (recycled crushed asphalt concrete is not acceptable) should be placed on the compacted subgrade soil. The baserock material should be compacted to not less than 95% relative maximum density. The finished crushed rock grade should be compacted with vibratory plate.

34. The concrete slab, if any, should have a minimum thickness of 5 inches and reinforced with No. 4 rebar with maximum spacing of 18 inches on center both ways. If the concrete slab were to receive floor covering, a 15-mil vapor barrier should be placed between the rock section and concrete slab. The vapor barrier should be taped at the seams and/or mastic sealed at the protrusions.

35. Prior to placing the vapor barrier 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.

SITE RETAINING WALLS

36. Any facilities that will retain a soil mass should be designed for a lateral earth pressure (active) equivalent to 50 pounds equivalent fluid pressure for horizontal backfill. 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, to which should be added surcharge loads. The structural engineer should discuss the surcharge loads with the geotechnical engineer prior to designing the retaining walls.

37. In designing for allowable resistive lateral earth pressure (passive) of 250 pounds equivalent fluid pressure should be used with the resultant acting at the third point. The top foot of subgrade soil should be neglected for computation of passive resistance.

38. A friction coefficient of 0.3 should be used for retaining wall design. This can be increased by 1/3 for short term seismic and wind loads.

39. The fore-mentioned values assume a drained condition and a moisture content compatible with those encountered during our investigation.

40. Drainage should be provided behind the retaining wall. The drainage (subdrain) system should consist of perforated 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.

41. As an alternative to the drain rock and fabric backfill, 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.

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

EXCAVATION

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

44. Any vertical cuts deeper than 5 feet must be properly shored or excavate per OSHA. 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.

UNDERGROUND PARKING LIFT EXCAVATION

45. It is our understanding that the excavation for the underground parking lift may be approximately 8 to 10 feet below the existing ground elevation. 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.

46. 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.

47. The bottom subgrade of the underground parking lift structure may be approximately 8 to 10 feet below ground surface elevation. Groundwater was first encountered in Boring B-1 at 21 feet and stabilized at 17 feet after the drilling completion. Based on the State guidelines and CGS Seismic Hazard Zone Report 111 [Seismic Hazard Evaluation of the Palo Alto 7.5-Minute Quadrangle, San Mateo and Santa Clara Counties,

California. 2006 (Preliminary Release 04/18/06). Department of Conservation. Division of Mines and Geology], the highest expected groundwater level is 15 feet below ground elevation. Therefore, dewatering will not be required during underground parking lift excavation.

48. If seepage water is encountered, contact our office for recommendations.

49. If there are space constraints for open excavation due to neighboring structure, driveway or property boundary, we recommend that the following procedure be implemented for shoring of the underground parking structure excavation.

SHORING SUPPORT FOR THE PARKING LIFT EXCAVATION

50. The parking lift will be excavated to the approximate depth of 8 to 10 feet below existing ground surface. Therefore, the excavation can 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 18-inch diameter holes to a minimum depth of 18 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. 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.

51. The shoring should be designed by the structural engineer or shoring design engineer and our office should review the shoring plan for approval.

UNDERGROUND PARKING LIFT RETAINING WALLS

52. The underground parking lift retaining walls should be design for seismic loading condition. The pseudo-static method by Seed and Whitman can be used (PE = (3/8)(0.45a_{max}/g)(H2)Wt (where a_{max} = 0.711g; H = height of the retaining wall; Wt = total unit weight of retained soil, for this site Wt = 120 pcf). This pseudo-static pressure is inverted triangularly distributed with the top value of 288 psf and 0 psf at the bottom. This pseudo-static pressure should be added to the active pressure for seismic loading condition.

53. The parking lift retaining wall shall be designed for active lateral earth pressure (static and seismic) and a surcharge value of 100 psf (vertically uniformed distributed down to 6 feet) as shown in Figure 8. This surcharge also includes truck loading and any adjacent structures. The parking lift walls can be designed as an undrain condition - no subdrain system.

54. If the parking lift walls would be designed for drained condition, see Site Retaining Wall section for subdrain system recommendation.

55. 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.

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

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

DRAINAGE

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

59. The final exterior grade adjacent to the proposed structures should be such that the surface drainage will flow away from the structures and away from adjacent existing retaining wall. 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.

60. Utility lines that cross under or through perimeter footings should be completely sealed to prevent moisture intrusion into the areas under the slab and/or footings. 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 footings.

61. 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 structures at a minimum of 5 percent.

62. 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} = 7.0 x 10^{-5} cm/sec). This rate can be used in the design of the bio-retention system for on-site storm drainage.

ON-SITE UTILITY TRENCHING

63. All on-site utility trenches must be backfilled with bedding material such as pre-gravel or approved material around the pipes and native on-site material or imported fill above the pipes. 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.

64. 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 (75 psi minimum compressive strength).

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

PAVEMENT DESIGN

66. 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

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 driveway (travel way). Flexible asphalt pavement section designs, which satisfy the State of California Standard Design Criteria, and above traffic indices, are presented in Table II. Rigid 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.

CORROSIVITY ANALYSIS

67. One soil sample collected on August 30, 2018 at the depth of 3 feet (2-1) 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 is 807 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 concentration for the near surface soil is 27 mg/kg. Because the chloride concentration is less than 100 mg/kg, it is determined to be insufficient to attack steel embedded in a concrete mortar coating.

- The sulfate ion concentration for the near surface soil is 150 mg/kg < 1,000 mg/Kg. Therefore, the sulfate ion concentration in the soil is determined to be insufficient 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 no special type cement is a requirement for use in construction.

- The soil pH for the near surface soil is 7.6, 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 is 536 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 following table. The results of the corrosivity laboratory tests results are enclosed in the Appendix.

CORROSIVE POTENTIAL

Soil Characteristics	**Range**	**Soil Sample 2-1**	**Corrosive Potential**
Resistivity (Ohm-cm)	<1,000	807	Highly corrosive
Soil pH	<8.5 >5.1	7.6	Non-corrosive
Chloride (mg/Kg)	<300	27	Non-corrosive
Sulfate (mg/Kg)	<1,000	150	Non-corrosive
Redox Potential (mV)	>100	536	Non-corrosive

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 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, structural, foundation design, 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; United States Geological Survey 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 (2006). CGS Seismic Hazard Zone Report 111 (Revised) *Seismic Hazard Evaluation for the Palo Alto 7.5-Minute Quadrangle, San Mateo and Santa Clara Counties, California*. (Preliminary release 04/18/2006). 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.

<u>TABLES</u>

TABLE I

SUMMARY OF LABORATORY TESTS

Sample No.	Depth (Ft.)	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	12.1	92.2	0.8	13		
1-2	5	8.7	103.8				
1-3	10	16.5	108.4				
1-4	15	24.4	100.5				
1-5	20	27.5	100.9			35	19
1-6	25	18.9	113.4			41	22
1-7	30	23.3	106.6				
1-8	35	25.4	105.9			43	23
1-9	40	31.2	96.7				
1-10	45	25.8	100.2			47	26
1-11	50	26.0	101.4			49	27
2-1	3	5.8	104.7				
2-2	5	18.5	109.7				
2-3	10	14.5	119.5				
3-1	3	25.8	94.8				
3-2	5	28.1	93.4				
3-3	10	26.5	99.9				

TABLE I (continued)

SUMMARY OF LABORATORY TESTS

Sample No.	Depth (Ft.)	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.
4-1	3	24.6	95.3				
4-2	5	27.2	94.5				
4-3	10	26.7	100.1				
4-4	15	23.8	101.4				
4-5	20	26.0	102.2				
4-6	30	22.9	105.7				

TABLE II

PROPOSED ASPHALT PAVEMENT SECTIONS

Location: Proposed Office Building
3241 Park Boulevard
Palo Alto, 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 maximum density	12.0”	12.0”	12.0”	12.0”	12.0”	12.0”

TABLE III

PROPOSED CONCRETE PAVEMENT SECTIONS

Location: Proposed Office Building
3241 Park Boulevard
Palo Alto, 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”	8.0”
Subgrade soil scarified & compacted to at least 90% relative maximum density	12.0”	12.0”	12.0”

* Including trash enclosures, stress pads, and valley gutters. Minimum reinforcement: #4 rebar at max. spacing, 18" on-center both ways or provided by Structural Engineer. Maximum control joints at 5’ by 5’ or as recommended by Structural Engineer. Vertical curbs should be keyed at least 3 inches into pavement subgrade.

TABLE IV

PROPOSED PAVER PAVEMENT SECTIONS

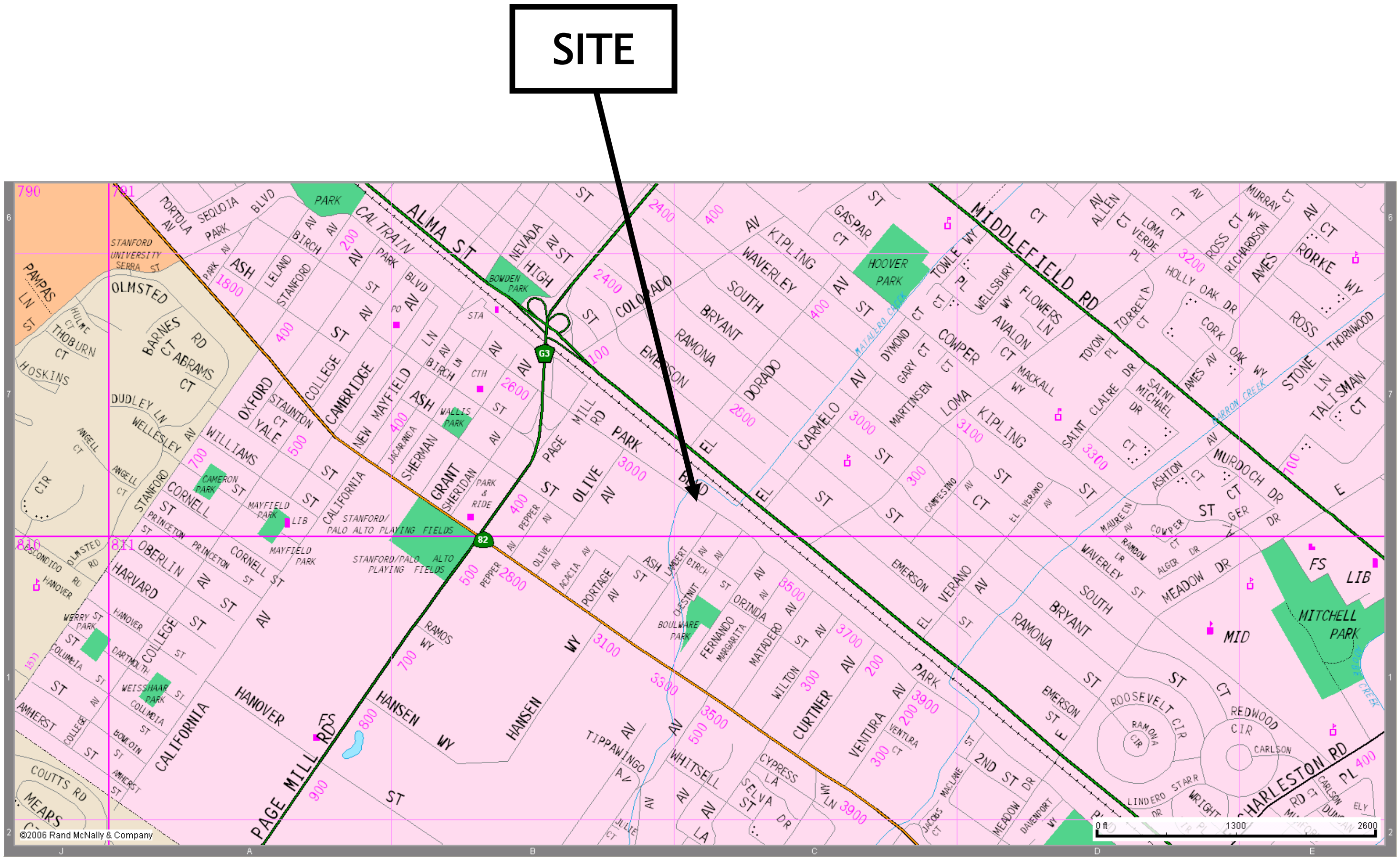
Location: Proposed Office Building
3241 Park Boulevard
Palo Alto, California

	DRIVEWAY/PARKING AREA*			
Recommended Paver Pavement Sections:	1A*	1B*	2A	2B
Vehicular Rated Pavers	Min. 3.25" ± Permeable Paver Parking Stalls	Min. 3.25" ± Permeable Paver Driveway	Min. 3.25" ± Non-Permeable Paver Parking Stalls	Min. 3.25" ± Non-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 Drain Stone or Class II Permeable Baserock compacted to at least 92% relative maximum density	10.0"	16.0"	---	---
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	---	---	10.0"	14.0"
Subgrade soil scarified & compacted to at least 90% relative max. density	12.0"	12.0"	12.0"	12.0"

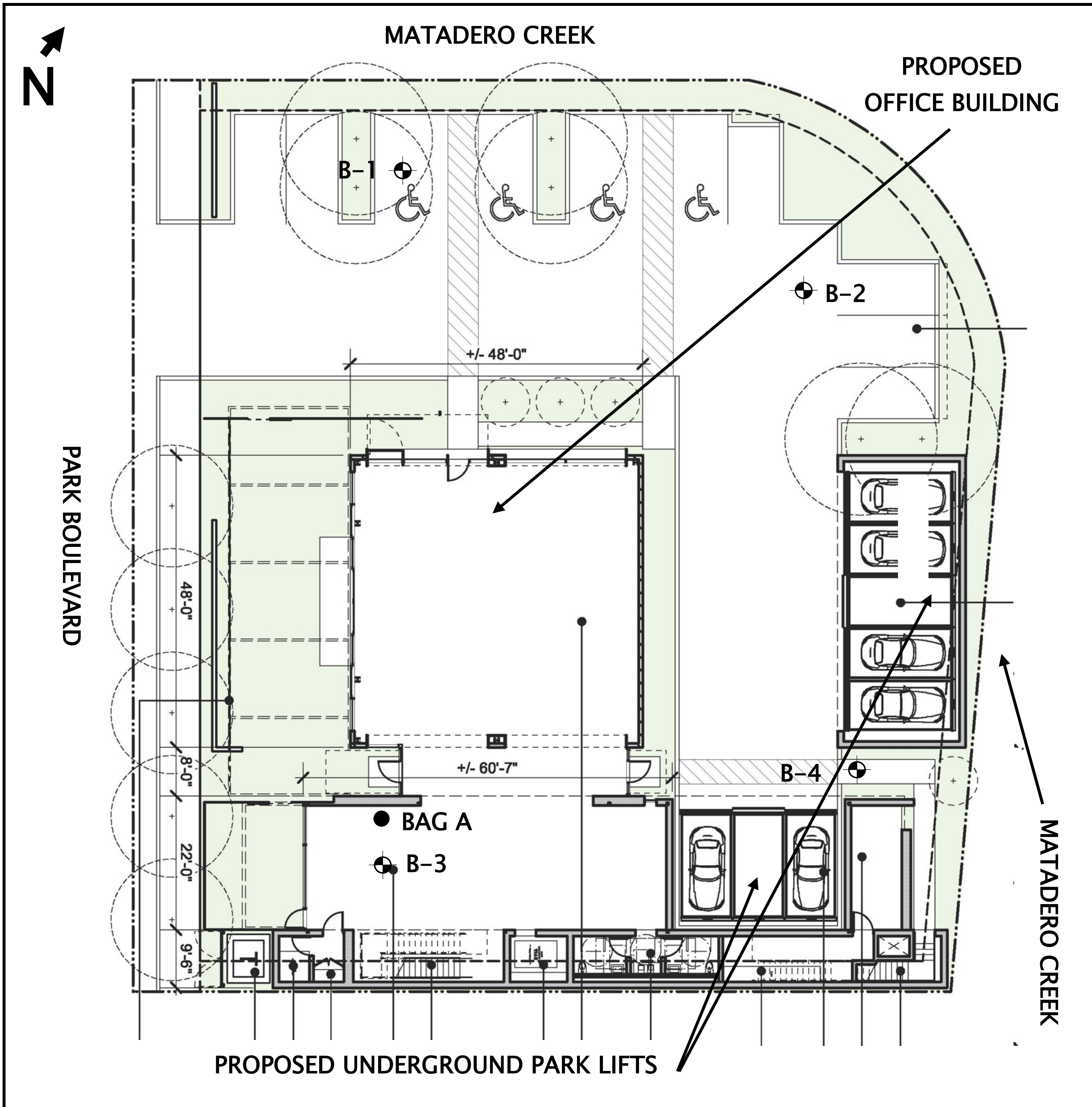
* The subgrade should be lined with a geotextile membrane Mirafi 500X, Geogrid, or equivalent. The membrane should be place and overlapped properly for drainage. The subgrade should be sloped at a minimum of 2% towards the subdrain system away from building foundation. The Mirafi 500X should not be placed over the subdrain system.

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.

<u>FIGURES</u>

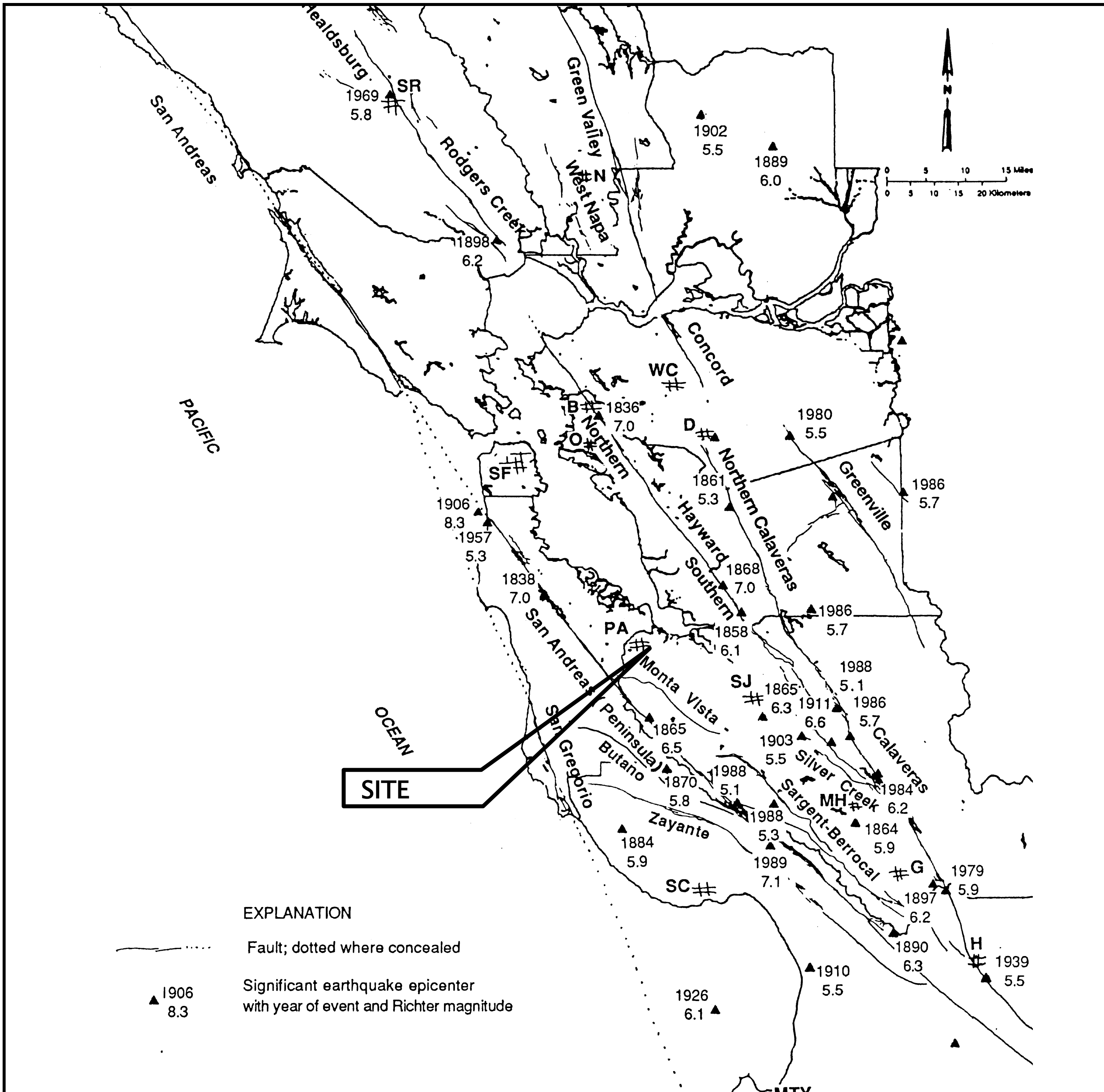


Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**VICINITY MAP** Proposed Office Building 3241 Park Boulevard Palo Alto, California	File No.: SV1809 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 1 April 2020



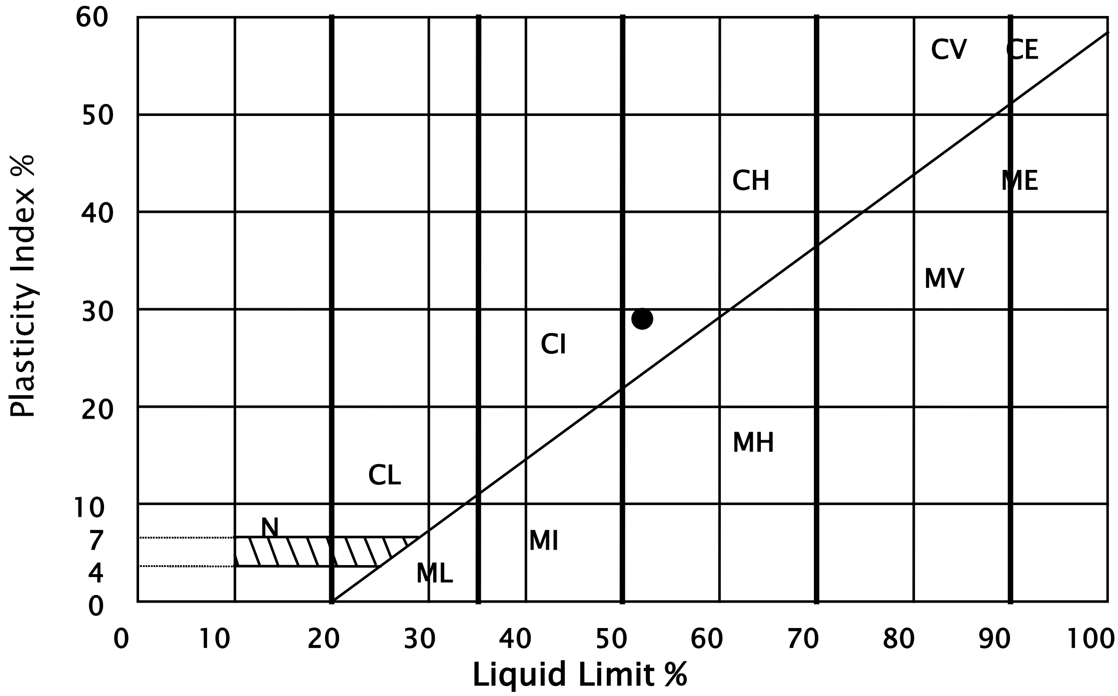
NOTE: ⊕ DENOTES APPROXIMATE EXPLORATORY BORING LOCATION
● DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**SITE PLAN** Proposed Office Building 3241 Park Boulevard Palo Alto, California	File No.: SV1809 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 2 April 2020



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**FAULT LOCATION MAP** Proposed Office Building 3241 Park Boulevard Palo Alto, California	File No.: SV1809 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 3 April 2020

PLASTICITY CHART

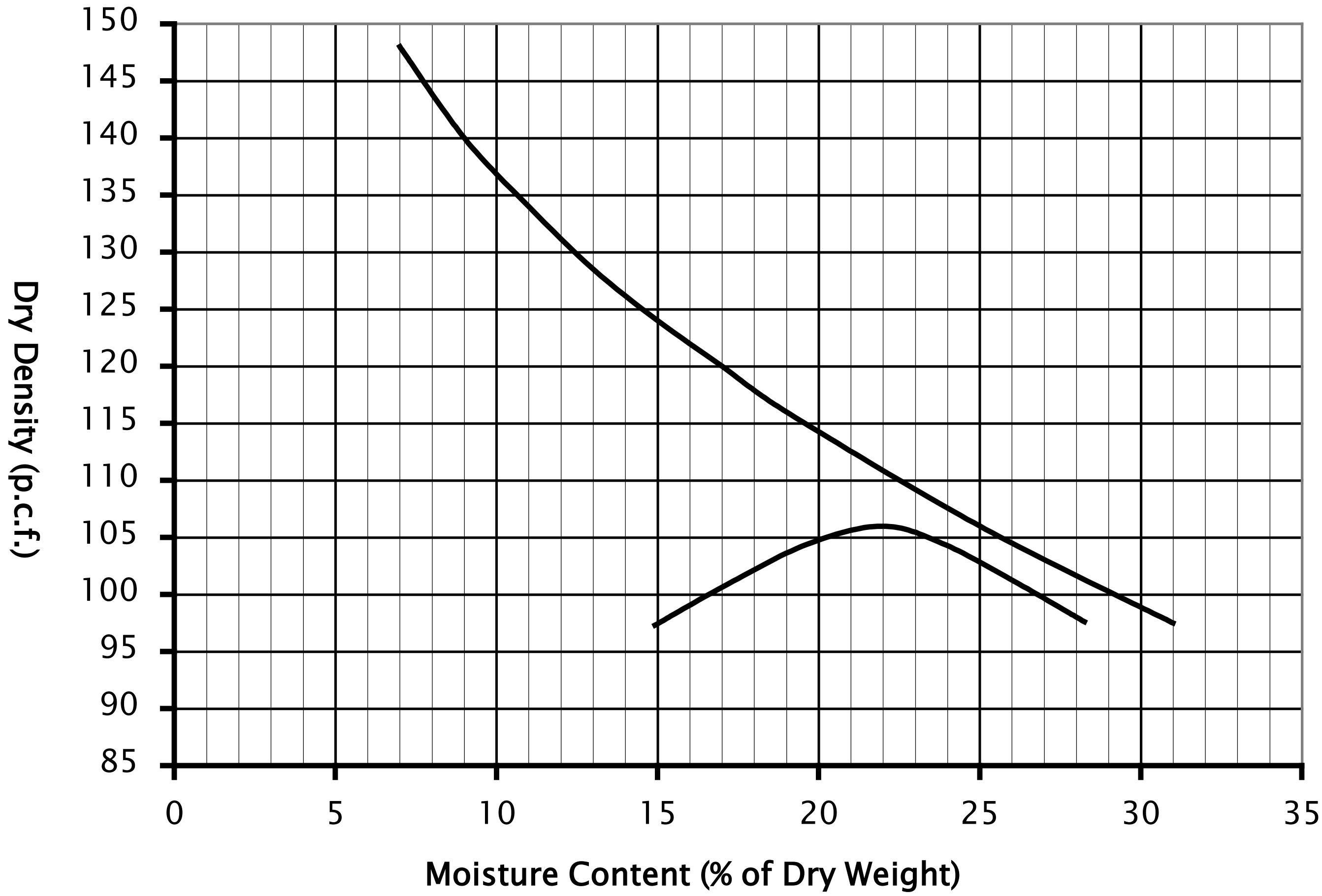


PLASTICITY DATA

Key Symbol	Sample No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	52	30	CH

*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 Office Building 3241 Park Boulevard Palo Alto, California	File No.: SV1809 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 4 April 2020



SAMPLE: A

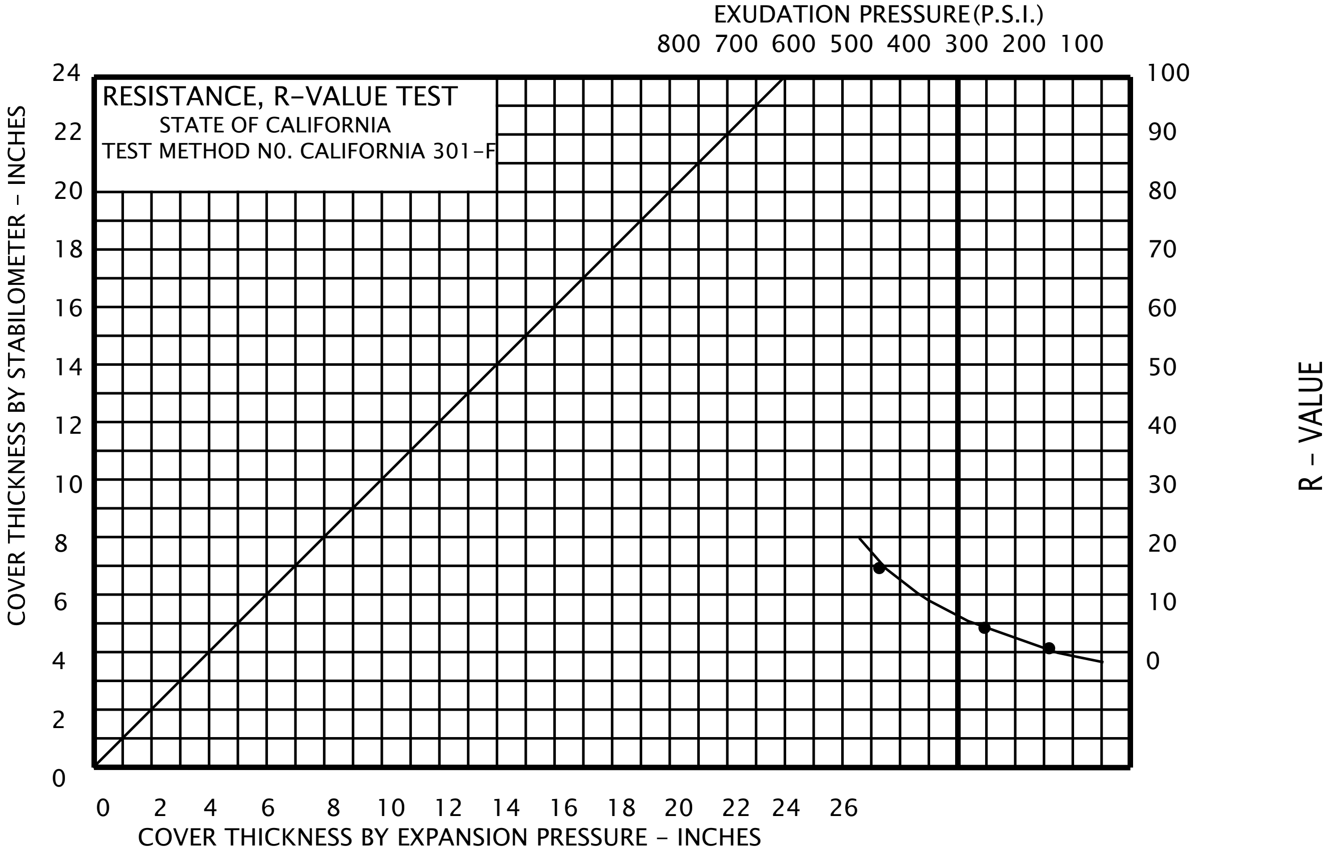
DESCRIPTION: Black Silty CLAY

LABORATORY TEST PROCEDURE: ASTM D1557

MAXIMUM DRY DENSITY: 106.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 22.0 %

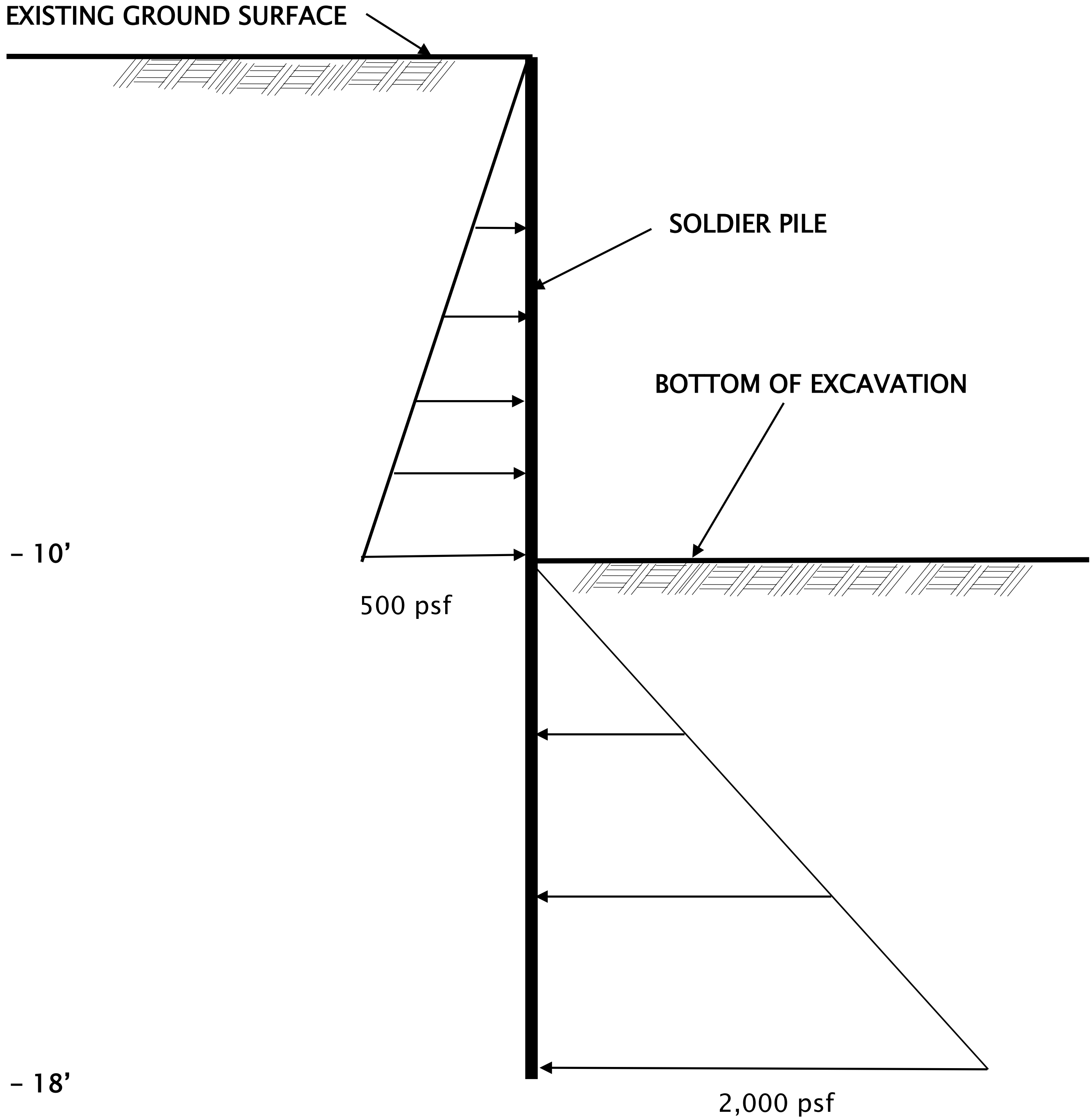
Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	COMPACTION TEST A Proposed Office Building 3241 Park Boulevard Palo Alto, California	File No. SV1809 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 5 April 2020



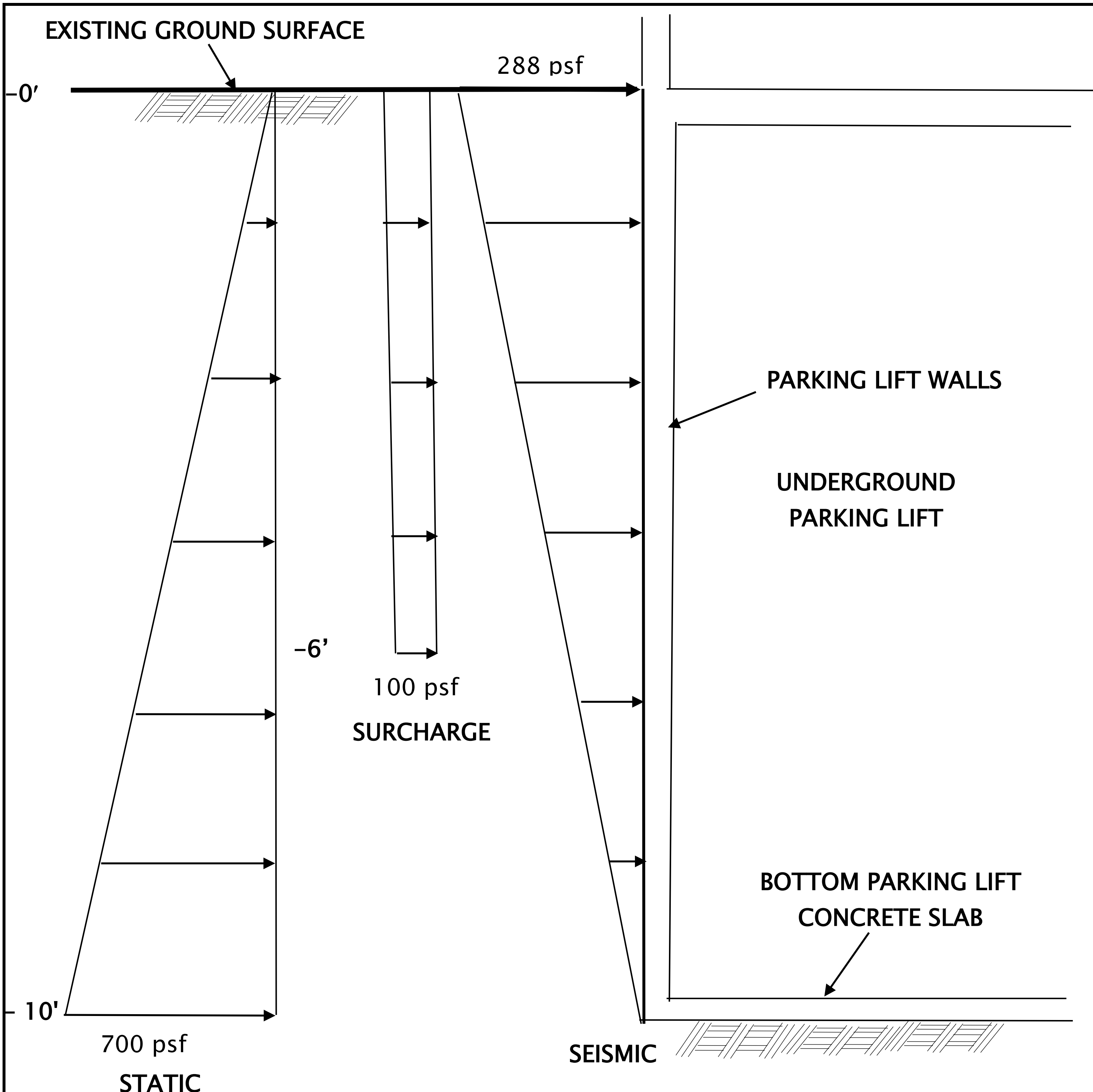
SAMPLE: A
DESCRIPTION: Black Silty CLAY

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 Office Building 3241 Park Boulevard Palo Alto, California	File No. SV1809	FIGURE
		Drawn by: V.V.	6
		Scale: NOT TO SCALE	April 2020



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**LATERAL SOIL PRESSURES SOLDIER PILE & WOOD LAGGING** Proposed Office Building 3241 Park Boulevard Palo Alto, California	File No.: SV1809	FIGURE
		Drawn by: V.V.	7
		Scale: NOT TO SCALE	April 2020



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**LATERAL SOIL PRESSURES** **PARKING LIFT WALLS** Proposed Office Building 3241 Park Boulevard Palo Alto, California	File No.: SV1809 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 8 April 2020

APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

KEY TO LOG OF BORING

EXPLORATORY BORING LOGS (B-1 THROUGH B-4)

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.

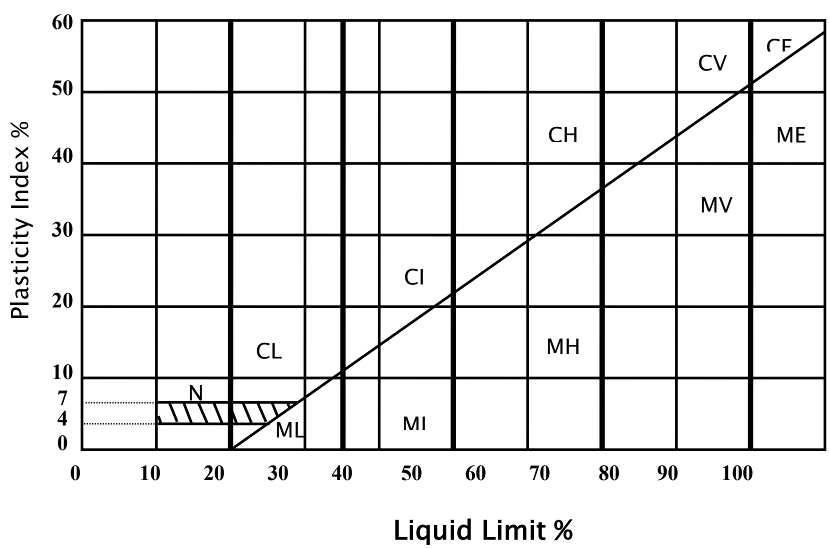
METHOD OF SOIL CLASSIFICATION CHART

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 Office Building **Project Location:** 3241 Park Boulevard Palo Alto, California **Project Number:** SV1809	**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)
Aggregate Base (AB)
Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
SILT, SILT w/SAND, SANDY SILT (ML)
SILTY CLAY (CL-ML)
Clayey SAND to Sandy CLAY (SC-CL)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Auger sampler
CME Sampler
Pitcher Sample
Bulk Sample
Grab Sample
2-inch-OD unlined split spoon (SPT)
3-inch-OD California w/ brass rings
2.5-inch-OD Modified California w/ brass liners
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 Office Building **Project Location:** 3241 Park Boulevard Palo Alto, California **Project Number:** SV1809	**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 **08/28/18**	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 **28 feet**
Groundwater Level and Date Measured **17 feet (08/28/18)**	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				Asphalt		10.0 inches of Asphalt Concrete (AC)						
0.83						12.0 inches of Aggregate Base (AB)						
1.83		1-1	33	CL		Medium Brown Sandy Silty CLAY Moist, very stiff (RETAINING WALL BACKFILL)	12.1	92.2	0.8	13		
3		1-2	36	ML		Brown Sandy SILT Moist, very stiff (RETAINING WALL BACKFILL)	8.7	103.8				
5												
7		1-3	65	CL		Light Olive Brown Silty CLAY Moist, very stiff	16.5	108.4				
10												
12		1-4	37	CL-ML		Tan Brown Clayey SILT Moist, very stiff	24.4	100.5				
15												
						Stabilized at drilling completion ▼						
		1-5	28				27.5	100.9			35	19
20												
						First encountered ▽						
23		1-6	30	SC-CL		Tan Brown Sandy CLAY Moist, very stiff	18.9	113.4			41	22
25												
		1-7	43				23.3	106.6				
30												

Project: Proposed Office Building
Project Location: 3241 Park Boulevard
Palo Alto, California
Project Number: SV1809

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 Sandy CLAY Moist, very stiff						
		1-8	42				25.4	105.9			43	23
35												
		1-9	55+				31.2	96.7				
40				CL-ML		Bluish Gray Clayey SILT Moist, very stiff						
		1-10	41				25.8	100.2			47	26
45												
		1-11	46				26.0	101.4			49	27
50						Boring terminated at 50.0 feet						
55												
60												
65												

Project: Proposed Office Building **Project Location:** 3241 Park Boulevard Palo Alto, California **Project Number:** SV1809	**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 **08/28/18**	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 **31 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		8.0 inches of Asphalt Concrete (AC)						
						6.0 inches of Aggregate Base (AB)						
1.17		2-1	25	CH		Black Silty CLAY Moist, very stiff	5.8	104.7				
3		2-2	40	CL		Light Tan Silty CLAY Moist, very stiff	18.5	109.7				
5						Color changed to dark brown						
		2-3	61				14.5	119.5				
10						Boring terminated at 10.0 feet						
15												
20												
25												
30												

Project: Proposed Office Building **Project Location:** 3241 Park Boulevard Palo Alto, California **Project Number:** SV1809	**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 **08/28/18**	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 **30 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		8.0 inches of Asphalt Concrete (AC)						
0.67						6.0 inches of Aggregate Base (AB)						
1.17				CH		Black Silty CLAY Moist, very stiff						
		3-1	36				25.8	94.8				
		3-2	33				28.1	93.4				
5				CL		Dark Gray Silty CLAY Moist, very stiff						
		3-3	39				26.5	99.9				
10						Boring terminated at 10.0 feet						
15												
20												
25												
30												

Project: Proposed Office Building **Project Location:** 3241 Park Boulevard Palo Alto, California **Project Number:** SV1809	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-4 **Sheet 1 of 1**

Date(s) Drilled **08/28/18**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **30.0 feet**
		Approximate Surface Elevation **31 feet**
Groundwater Level and Date Measured **18 feet (08/28/18)**	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				Asphalt		3.0 inches of Asphalt Concrete (AC)						
0.25						6.0 inches of Aggregate Base (AB)						
0.75				CH		Black Silty CLAY Moist, very stiff						
		4-1	34				24.6	95.3				
		4-2	30				27.2	94.5				
5				CL		Dark Gray Silty CLAY Moist, very stiff Color changed to olive brown						
		4-3	41				26.7	100.1				
10												
12				CL-ML		Tan Brown Clayey SILT Moist, very stiff						
		4-4	35				23.8	101.4				
15												
						Stabilized at drilling completion ▼						
		4-5	30				26.0	102.2				
20												
						First encountered ▽						
24				SC-CL		Tan Brown Sandy CLAY Moist, very stiff						
25												
		4-6	41				22.9	105.7				
30						Boring terminated at 30.0 feet						

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 768-065 **Date:** 9/6/2018 **Tested By:** PJ **Checked:** PJ

Client: Silicon Valley Soil Engineering **Project:** 3241 Park Blvd, Palo Alto, CA **Proj. No:** SV1809

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	
2-1	-	3	-	-	807	27	150	0.0150	7.6	536	23	-	25.2	Dark Olive Gray Clayey SAND w/ Gravel