

REPORT TO

CHARITIES HOUSING  
SAN JOSE, CALIFORNIA

FOR

PROPOSED BLOSSOM HILL DEVELOPMENT  
397 BLOSSOM HILL ROAD  
SAN JOSE, CALIFORNIA

GEOTECHNICAL INVESTIGATION  
FEBRUARY 2018

PREPARED BY

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GEOTECHNICAL CONSULTANTS

File No. SV1742  
February 28, 2018

Charities Housing  
1400 Parkmoor Ave. #190  
San Jose, CA 95126

Attention: Ms. Sandra Heredia, Project Manager

Subject: Proposed Blossom Hill Development  
397 Blossom Hill Road  
San Jose, California  
**GEOTECHNICAL INVESTIGATION**

Dear Ms. Heredia:

Pursuant to your request, we are pleased to present herein geotechnical investigation for the proposed Blossom Hill development. The subject site is located at 397 Blossom Hill Road 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

*Sean Deivert*  
Sean Deivert  
Project Manager

  
Vien Vo, P.E.



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**TABLE OF CONTENTS****GEOTECHNICAL INVESTIGATION****PAGE**

INTRODUCTION	1
SITE LOCATION AND DESCRIPTION	1
FIELD INVESTIGATION	1
LABORATORY INVESTIGATION	2
SOIL CONDITIONS	3
GENERAL GEOLOGY	4
LIQUEFACTION ANALYSIS	5
A. GROUNDWATER	5
B. SUSPECTED LIQUEFIABLE SOIL LAYERS	5
C. CONCLUSIONS	9
INUNDATION POTENTIAL	9
CONCLUSIONS	10
RECOMMENDATIONS	12
GRADING	12
WATER WELLS	14
FOUNDATION DESIGN CRITERIA	14
2016 CBC SEISMIC VALUES	16
EXCAVATION	16
BASEMENT EXCAVATION	17
SHORING SUPPORT FOR THE BASEMENT EXCAVATION	18
BASEMENT RETAINING WALLS	19
SITE RETAINING WALLS	20
DRAINAGE	21
ABANDONMENT OF THE EXISTING UTILITY LINES	22
ON-SITE UTILITY TRENCHING	23
PAVEMENT DESIGN	23
LIME TREATMENT ALTERNATIVES	24
LIMITATIONS AND UNIFORMITY OF CONDITIONS	25
REFERENCES	27

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## **LIST OF TABLES, FIGURES, AND APPENDICES** **GEOTECHNICAL INVESTIGATION**

### **TABLES**

TABLE I – SUMMARY OF LABORATORY TESTS

TABLE II – PROPOSED ASPHALT PAVEMENT SECTIONS

TABLE III – PROPOSED CONCRETE PAVEMENT SECTIONS

TABLE IV – PROPOSED PAVER PAVEMENT SECTIONS

### **FIGURES**

FIGURE 1 – VICINITY MAP

FIGURE 2A – SITE PLAN – STUDY 1

FIGURE 2B – SITE PLAN – STUDY 2

FIGURE 3 – FAULT LOCATION MAP

FIGURE 4 – PLASTICITY INDEX

FIGURE 5 – COMPACTION TEST A

FIGURE 6 – R-VALUE TEST

FIGURE 7 – LATERAL SOIL PRESSURES – SOLDIER PILE & WOOD LAGGING

FIGURE 8 – LATERAL SOIL PRESSURES – BASEMENT WALLS

### **APPENDICES**

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION CHART

KEY TO LOG OF BORING

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

SANTA CLARA VALLEY WATER DISTRICT DRILLING PERMIT

## **INTRODUCTION**

Per your authorization, Silicon Valley Soil Engineering (SVSE) conducted a geotechnical investigation. The purpose of this geotechnical investigation was to evaluate the nature of the surface and subsurface soil conditions at the 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 397 Blossom Hill Road in San Jose, California (Figure 1). Blossom Hill Road bounds the subject site to the south, existing Chase Bank building to the west, existing apartments to the north, and medical office building to the east. At the time of this investigation, the subject site is a rectangular, relatively flat lot occupied by ACCO furniture store building surrounded by paved parking lot and landscaping areas. Based on the preliminary plans for the subject site, the proposed development will include the demolition of the existing structure and the construction of a mixed-use development consisting of commercial and residential buildings with heights up to five stories with basement garage, community amenities and associated improvements. The approximate location of the proposed structures and our borings are shown on the Site Plans (Figure 2A & 2B).

## **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 four exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on January

29, 2018. The approximate location of the borings is shown on the Site Plans (Figure 2A & 2B). The borings were drilled to the depths of 20.0 feet to 50.0 feet below the existing ground surface. The borings were drilled with a truck mounted drill rig using 8-inch diameter hollow stem augers.

The soils encountered were logged continuously in the field during the drilling operation. Relatively undisturbed soil samples were obtained by hammering a 2-inch outside diameter (O.D.) split-tube sampler for a Standard Penetration Test (SPT), ASTM Standard D1586, into the ground at various depths. A 140-pound hammer with a free fall of 30 inches was used to drive the sampler 18 inches into the ground. Blow counts were recorded on each 6-inch increment of the sampled interval. The blows required to advance the sampler the last 12 inches of the 18 inch sampled interval were recorded on the boring logs as penetration resistance. These values were also used to evaluate the liquefaction potential of the subsurface soils.

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

## **LABORATORY INVESTIGATION**

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

1. Moisture content and dry density tests were performed on the relatively undisturbed soil samples in order to determine soil consistency and the moisture variation throughout the explored soil profile (Table I).
2. Atterberg Limits tests were performed on the sub-surface soil to assist in the classification of these soils and to obtain an evaluation of their

expansion and shrinkage potential and liquefaction analysis (Figure 4 & Table I).

3. The strength parameters of the foundation soils were determined from direct shear tests that were performed on selected relatively undisturbed soil samples (Table I).
4. Laboratory compaction tests were performed on the near-surface material per the ASTM D1557 test procedure (Figure 5).
5. One R-Value test was performed on a near surface soil sample for pavement section design recommendations (Figure 6).

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 feet boring), the pavement surface consists of 1.5 inches of Asphalt Concrete (AC) over 5.0 inches of Aggregate Base (AB). Below the pavement surface to the depth of 1.5 feet, a black, moist, stiff silty clay layer was encountered. From the depths of 1.5 feet to 13 feet, the soil became medium brown, moist, stiff silty clay. Color changes of light brown and medium brown were noted at the depths of 5 feet and 10 feet respectively. From the depths of 13 feet to 20 feet, a brown, moist, medium dense/stiff clayey sand/sandy clay layer was encountered. From the depths of 20 feet to 33 feet, the soil became olive brown, moist, stiff silty clay. From the depths of 33 feet to 38 feet, a dark gray, moist, stiff sandy silty clay layer was encountered. From the depths of 38 feet to 40 feet, the soil became dark olive brown, moist, medium dense/very stiff clayey silty sand/clayey sandy silt. From the depths of 40 feet to the end of the boring at 50 feet, a bluish gray, moist, very stiff silty clay layer was encountered. Similar soil profiles were encountered in Borings B-3 and B-4.

In Boring B-2 (60 feet boring), from the surface to the depth of 3 feet, a dark olive brown, moist, stiff silty clay layer was encountered. From the depths of 3 feet to 8 feet, the soil became light brown, damp, very stiff sandy silt. From the depths of 8 feet to 13 feet, a light tan, moist, very stiff clayey silt layer was encountered. From the depths of 13 feet to 40 feet, the soil became light brown, moist, stiff silty clay. Color changes of olive brown and brown were noted at a depth of 17 feet and 20 feet respectively. From the depths of 40 feet to the end of the boring at 60 feet, a bluish gray, moist, very stiff silty clay layer was encountered.

Groundwater was initially encountered in Boring B-1 and B-2 at the depth of 25 feet and 20 feet and rose to a static level of 18 feet and 15 feet at the end of the drilling operation. It should be noted that the groundwater level would fluctuate as a result of seasonal changes and hydrogeological variations such as groundwater pumping and/or recharging. A graphic description of the explored soil profiles is presented in the Exploratory Boring Log contained in the Appendix.

## **GENERAL GEOLOGY**

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

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

Late Quaternary deposits fill the center of the San Francisco Bay Region and most of the strata are of continental origin characterized as alluvial and fluvial



materials. The project site is underlain by young alluvial fan deposits (Helley and Brabb, 1971, Rogers & Williams, 1974).

### **LIQUEFACTION ANALYSIS:**

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

#### **A. GROUNDWATER**

Groundwater was initially encountered in Boring B-1 and B-2 at the depth of 25 feet and 20 feet and rose to a static level of 18 feet and 15 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 044 (revised) [*Seismic Hazard Evaluation of the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California. 2000 (Updated 1/17/06)*. Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 10 feet below ground elevation. Therefore, this depth of the groundwater table will be used for the liquefaction analysis.

#### **B. SUSPECTED LIQUEFIABLE SOIL LAYERS**

The site is located within the State of California Seismic Hazard Zone for liquefaction (CGS, 2001). The State Guidelines (CGS Special Publication 117A, revised 2008, Southern California Earthquake Center, 1999) were followed by this study. Based on recent studies (Bray and Sancio, 2006, Boulanger and Idriss, 2004), the "Chinese Criteria", previously used as the liquefaction screening (CGS SP 117, SCEC, 1999) is no longer valid indicator of liquefaction susceptibility. The revised screening criteria clearly stated that liquefaction is the transformation of loose saturated silts, sands, and clay with a Plasticity Index (PI) < 12 and moisture content (MC) > 85% of the liquid limits are susceptible to liquefaction and  $12 < PI < 18$  and  $MC > 80\%$  of LL are moderately susceptible to liquefaction. This occurs under vibratory conditions such as those induced by a seismic event.

To help evaluate liquefaction potential, samples of potentially liquefiable soil were obtained by hammering the split tube sampler into the ground. The number of blows required driving the sampler the last 12 inches of the 18 inch sampled interval were recorded on the log of test boring. The number of blows was recorded as a Standard Penetration Test (SPT), ASTM Standard D1586-92.

Suspected liquefiable soil layers were screened in Boring B-1 (50.0 feet deep).

**BORING B-1.** The results from our exploratory boring show that the subsurface soil material in Boring B-1 to the depth of 50.0 feet consists of stiff silty clay to stiff silty clay to medium dense/stiff clayey sand/sandy clay to stiff silty clay to stiff sandy silty clay to medium dense/very stiff clayey silty sand/clayey sandy silt 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 1.5 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).
2. The stiff silty clay layer from the depths of 1.5 feet to 10 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).
3. The stiff silty clay layer from the depths of 10 feet to 13 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
  - Sample No. 1-3 (10 feet) - [PI < 18; PI = 19 and MC = 16.0% < 80% LL = 29.6%; LL = 37]
4. The medium dense/stiff clayey sand/sandy clay layer from the depths of 13 feet to 20 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

- Sample No. 1-4 (15 feet) – [PI < 18; PI = 19 and MC = 26.3% < 80% LL = 28.0%; LL = 35]
  - Sample No. 1-5 (20 feet) – [PI < 18; PI = 19 and MC = 30.6% > 80% LL = 31.2%; LL = 39]
5. The stiff silty clay layer from the depths of 20 feet to 33 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
- Sample No. 1-6 (25 feet) – [PI > 18; PI = 20 and MC = 27.8% < 80% LL = 33.6%; LL = 42]
  - Sample No. 1-7 (30 feet) – [PI > 18; PI = 22 and MC = 27.0% < 80% LL = 34.4%; LL = 43]
6. The stiff sandy silty clay layer from the depths of 33 feet to 38 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
- Sample No. 1-8 (35 feet) – [PI > 18; PI = 20 and MC = 22.9% < 80% LL = 33.6%; LL = 42]
7. The medium dense/very stiff clayey silty sand/clayey sandy silt layer from the depths of 38 feet to 40 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
- Sample No. 1-9 (40 feet) – [PI > 18; PI = 19 and MC = 22.9% < 80% LL = 28.0%; LL = 35]
8. The very stiff silty clay 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 = 25 and MC = 35.8% < 80% LL = 41.6%; LL = 52]

- Sample No. 1-11 (50 feet) - [PI > 18; PI = 28 and MC = 44.4% < 80% LL = 44.8%; LL = 56]

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

**BORING B-2.** The results from our exploratory boring show that the subsurface soil material in Boring B-2 to the depth of 60.0 feet consists of stiff silty clay to very stiff sandy silt to very stiff clayey silt to stiff silty clay 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 3 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).
2. The very stiff sandy silt layer from the depths of 3 feet to 8 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).
3. The very stiff clayey silt layer from the depths of 8 feet to 10 feet is not liquefiable soil because it is above the highest expected groundwater table (10 feet).
4. The very stiff clayey silt layer from the depths of 10 feet to 13 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
  - Sample No. 2-3 (10 feet) - [PI < 18; PI = 19 and MC = 4.7% < 80% LL = 27.2%; LL = 34]
5. The stiff silty clay layer from the depths of 13 feet to 40 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):

- Sample No. 2-5 (20 feet) – [PI < 18; PI = 22 and MC = 33.8% < 80% LL = 36.8%; LL = 46]
  - Sample No. 2-7 (30 feet) – [PI < 18; PI = 21 and MC = 24.7% < 80% LL = 36.0%; LL = 45]
  - Sample No. 2-9 (40 feet) – [PI < 18; PI = 23 and MC = 28.0% < 80% LL = 36.8%; LL = 46]
6. The very stiff silty clay layer from the depths of 40 feet to 60 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
- Sample No. 2-10 (45 feet) – [PI < 18; PI = 22 and MC = 34.4% < 80% LL = 37.6%; LL = 47]
  - Sample No. 2-12 (55 feet) – [PI < 18; PI = 24 and MC = 31.6% < 80% LL = 38.4%; LL = 48]

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

### **C. CONCLUSIONS**

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

### **INUNDATION POTENTIAL**

The subject site is located at 397 Blossom Hill Road in San Jose, California. According to the Limerinos and others, 1973 report, the site is not located in an area that has potential for inundation as the result of a 100-year flood (Limerinos; 1973).

## **CONCLUSIONS**

1. The site covered by this investigation is suitable for the proposed development provided the recommendations set forth in this report are carefully followed.
2. The proposed five-story mixed-use building with underground basement garage should be supported on mat slab foundation.
3. Based on the laboratory testing results, the native surface soil at the subject site has been found to have a high expansion potential and moderate expansion potential when subjected to fluctuations in moisture. Therefore, we recommend that the concrete slab should be underlain by a minimum of 18 inches non-expansive fill layer including 6 inches of rock below the slab or lime treatment. During the construction of the building pad, highly expansive native soil should not be used as non-expansive engineered fill material.
4. Any imported non-expansive fill soils should be free of organic material and hazardous substances. All imported fill material to be used for engineered fill should be environmentally tested prior to be used at the site.
5. The highest expected groundwater table is at the depth of 10 feet below existing ground surface. Therefore, the basement grade may need to be dewatered and stabilized with at least 18 inches of rock or lime treatment of the basement subgrade. Dewatering contractor should be consulted.
6. The basement walls and slab should be waterproofed with Bitumen Waterproof Membrane, Paraseal or equivalent including pipes protruding through the basement concrete walls. A waterproofing consultant should provide waterproofing recommendations.
7. The exterior of the building pad should be graded to promote proper drainage and diversion of water away from the building foundations.

8. 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.
9. On the basis of the engineering reconnaissance and exploratory borings, it is our opinion that trenches that will be excavated to depths less than 5 feet below the existing ground surface will not need shoring. However, for trenches and basement that will be excavated greater than 5 feet in depth, shoring will be required.
10. Specific recommendations are presented in the remainder of this report.
11. All earthwork including grading, backfilling, and shoring installation, foundation excavation and drilling shall be observed and inspected by a representative from Silicon Valley Soil Engineering (SVSE). Contact our office 48 hours prior to the commencement of any earthwork.

**RECOMMENDATIONS:****GRADING**

1. The placement of fill and control of any grading operations at the site should be performed in accordance with the recommendations of this report. These recommendations set forth the minimum standards to satisfy other requirements of this report.
2. All existing surface and subsurface structures, if any, that will not be incorporated in the final development shall be removed from the project site prior to any grading operations. These objects should be accurately located on the grading plans to assist the field engineer in establishing proper control over their removal. All utility lines in the new building pad area must be removed prior to any grading at the site.
3. The depressions left by the removal of subsurface structures should be cleaned of all debris, backfilled and compacted with clean, native 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 existing gravel section and after stripping the organic material from the soil, the building pad 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, the existing subgrade soil should be moisture conditioned over 3% optimum moisture, compacted to not less than 90% relative maximum density using ASTM



- D1557 procedure over the entire building pad, 5 feet beyond the perimeter of the pad and 3 beyond the edge of the parking area.
7. All engineered fill or imported soil including baserock material should be placed in uniform horizontal lifts of not more than 8 inches in uncompacted thickness, and compacted to not less than 90% 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. The basement excavated grade should be moisture conditioned as necessary and compacted to 90%.
  9. When fill material includes rocks, nesting of rocks will not be allowed and all voids must be carefully filled by proper compaction. Rocks larger than 4 inches in diameter should not be used for the final 2 feet of building pad.
  10. Unstable (yielding) subgrade should be aerated or moisture conditioned as necessary. Yielding isolated area in the subgrade can be stabilized with an excavation of the subgrade to the depth of 12 to 18 inches, lined with stabilization fabric membrane Mirafi 500X or equivalent) and backfilled with aggregate base.
  11. SVSE should be notified at least two days prior to commencement of any grading operations so that our office may coordinate the work in the field with the contractor. All imported borrow must be approved by SVSE before being brought to the site. Import soil must have a plasticity index no greater than 15, an R-Value greater than 25 and environmentally clean (non-hazardous). The import soil should contain at least 30 percent fines

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

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

### **WATER WELLS**

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

### **FOUNDATION DESIGN CRITERIA**

14. The proposed five-level building structure with an underground basement should be supported on mat foundation. Recommendations are presented in the following paragraphs.
15. The mat foundation should have a minimum thickness of 24 inches with thickened edge at 30 inch depth and a contact pressure of 2,000 psf.
  - A value of 150 pci as the soil modulus of subgrade of reaction can be used in the design of the mat foundation.
  - The mat slab should be underlain by a minimum of 12 inches of ¾-inch washed crushed rock. Note, Basement Excavation Section of this report recommend 18 inch layer of rock due to the possible presence of groundwater. Also, lime treatment can be considered to stabilize the basement grade.

- If the bottom of the basement slab would be below the highest expected groundwater table at 10 feet below existing ground surface, hydrostatic uplift pressure should be considered.
  - The subgrade soil should be compacted to not less than 90% relative maximum density.
  - Waterproof membrane (Paraseal or equivalent) should be placed between the basement concrete slab and rock section.
  - Mud slab of a minimum of 3 inches thick should be placed over the waterproofing membrane. The compressive strength of the mud slab should be a minimum of 75 psi.
  - We estimate that post-construction differential settlement will be less than quarter inch settlement per 50 feet span.
16. The fore-mentioned bearing values are for dead plus live loads and may be increased by one-third for short term seismic and wind loads. The design of the structures and the foundations shall meet local building code requirements.
17. The  $\frac{3}{4}$ -inch crushed rock (recycled crushed asphalt concrete is not acceptable) should be placed on the finished subgrade pad elevation. The  $\frac{3}{4}$ -inch crushed rock should be compacted in-place with vibratory plate. The pad subgrade should be compacted prior to placement of the crushed rock and after installation of any under utility pipes and footing/thickened edge excavation with smooth drum roller and/or heavy vibratory plate equipment.
18. The footing bottoms and thickened edges should be compacted with jumping jack prior to rebar and form work placement and inspected.
19. The project structural engineer responsible for the foundation design shall determine the final design of the foundations and reinforcing required. We

recommend that the foundation plans be reviewed by our office prior to submitting to the appropriate local agency and/or to construction.

## **2016 CBC SEISMIC VALUES**

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

<b>Classification/Coefficient</b>	<b>Design Value</b>
Site Class (ASCE 7-10, Table 20.3-1; 2016 CBC, Section 1613A.3.2)	D
Risk Category	I,II,III
Site Latitude	37.252139° N.
Site Longitude	121.827814° W.
0.2-second Mapped Spectra Acceleration <sup>1</sup> , $S_S$ (Section 1613A.3.1)*	1.607g
1-second Mapped Spectra Acceleration <sup>1</sup> , $S_I$ (Section 1613A.3.1)*	0.600g
Short-Period Site Coefficient, $F_a$ Table 1613A.3.3(1)*	1.0
Long-Period Site Coefficient, $F_V$ Table 1613A.3.3(2)*	1.5
0.2-second Period, Maximum considered Earthquake Spectral Response Acceleration, $S_{MS}$ ( $S_{MS} = F_a S_S$ ; Section 1613A.3.3)*	1.607g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration, $S_{M1}$ ( $S_{M1} = F_V S_I$ ; Section 1613A.3.3)*	0.900g
0.2-second Period, Designed Spectra Acceleration, $S_{DS}$ ( $S_{DS} = 2/3 S_{MS}$ ; Section 1613A.3.4)*	1.071g
1-second Period, Designed Spectra Acceleration, $S_{D1}$ ( $S_{D1} = 2/3 S_{M1}$ ; Section 1613A.3.4)*	0.600g

<sup>1</sup> For Site Class B, 5 percent damped.

\*2016 CBC

## **EXCAVATION**

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

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

### **BASEMENT EXCAVATION**

23. It is our understanding that the excavation for the underground parking structure will be approximately 12 to 13 feet below the existing ground elevation. No difficulties due to soil conditions are anticipated in excavating the on-site material. Conventional earth moving equipment will be adequate for this project.
24. 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.
25. The bottom subgrade of the underground basement structure will be approximately 12 to 13 feet below ground surface elevation. The groundwater table at the time of our investigation was encountered at a depth of 25 feet during the drilling operation and stabilized at a depth of 15 feet. Based on the State guidelines and CGS Seismic Hazard Zone Report 044 [*Seismic Hazard Evaluation of the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California. 2000 (Updated 1/17/06).* Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 10 feet below ground elevation. Therefore, dewatering maybe require during basement excavation. Based on the time of the year and precipitation accumulation, a

- dewatering expert should be consulted for further design and recommendations.
26. The bottom subgrade of the basement excavation may be wet and soft due to the presence of groundwater or nearby groundwater and soft clayey sand/sandy clay below 13 feet. Therefore, the bottom subgrade should be stabilized with a 3-inch concrete mud slab over 18-inch layer of 3/4-inch crushed rock compacted in-place over stabilization fabric membrane (Mirafi 500X or equivalent).
  27. Standing groundwater at the bottom subgrade should be pumped out to provide a dry and stable working platform for the construction equipment.
  28. If there are space constraints for open excavation, we recommend that the following procedure be implemented for shoring of the underground parking structure excavation.

### **SHORING SUPPORT FOR THE BASEMENT EXCAVATION**

29. The basement will be excavated to the approximate depth of 12 to 13 feet below existing ground surface. Therefore, the excavation should be supported with steel "H" beams and a 3 x 12 or 4 x 12 wood lagging. Prior to any excavation, the steel "H" beams should be placed in pre-drilled minimum 24-inch diameter holes to a minimum depth of 24 feet. The holes should be filled with concrete to one foot below the bottom of the excavation and concrete slurry (2 sack cement) for the remaining void to existing ground elevation. Groundwater will be encountered and should be displaced properly in the pier holes by the concrete via tremmie pipe or other methods approved by our office. At this point, excavation can begin. As the excavation operation proceeds, the wood lagging should be placed between the steel "H" beams. The "H" beams should be placed a maximum distance of 8 feet apart. There should be no voids between the soil wall

excavation and wood lagging. However, if a void occurs, the void should be filled with sand slurry or pressure grouted especially at the area below each lagging bench (last lagging board). Proper attention should be considered during the construction. Introduction of any heavy equipment on the top of the vertical cut may damage the excavated slope. The lateral soil pressure acting on the shoring system is shown in Figure 7. The passive pressure of 250 pounds equivalent fluid pressure can be used for short-term shoring purposes. The shoring should be designed by the structural engineer or shoring design engineer and our office should review the shoring plan for approval.

### **BASEMENT RETAINING WALLS**

30. The basement retaining walls should be design for seismic loading condition. The pseudo-static method by Seed and Whitman can be used ( $PE = (3/8)(0.45a_{max}/g)(H^2)W_t$  (where  $a_{max} = 0.56g$ ;  $H$  = height of the retaining wall;  $W_t$  = total unit weight of retained soil, for this site  $W_t = 120$  pcf). This pseudo-static pressure is inverted triangularly-distributed with the top value of 360 psf and 0 psf at the bottom. This pseudo-static pressure should be added to the active pressure for seismic loading condition.
31. The basement retaining wall shall be designed for active lateral earth pressure (static and seismic), hydrostatic lateral, and a surcharge value of 100 psf (vertically uniformed distributed down to 6 feet) as shown in Figure 8. This surcharge also include truck loading and any adjacent structures.
32. 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.

33. The basement walls should be waterproofed with Bitumen Waterproof Membrane, Paraseal LG or equivalent including pipes protruding through the basement concrete walls. A waterproofing consultant should provide waterproofing recommendations.
34. The basement walls should be designed to assume an un-drained condition. As a result, a subdrain system would not be required.
35. We recommend a thorough review by our office of all designs pertaining to facilities retaining a soil mass.

### **SITE RETAINING WALLS**

36. Any facilities that will retain a soil mass near the existing ground surface shall be designed for a lateral earth pressure (active) equivalent to 50 pounds equivalent fluid pressure, plus surcharge loads. If the retaining walls are restrained from free movement at both ends, the walls shall be designed for the earth pressure resulting from 60 pounds equivalent fluid pressure, to which shall be added surcharge loads.
37. In designing for allowable resistive lateral earth pressure (passive), a value of 250 pounds equivalent fluid pressure may be used with the resultant acting at the third point. The top foot of native soil shall be neglected for computation of passive resistance.
38. 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.
39. The above 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 system should consist of perforated pipe, Schedule 40 or equivalent, placed at the base of the retaining wall and surrounded by  $\frac{3}{4}$  inch drain



rock wrapped in a filter fabric, Mirafi 140N or equivalent. The drain rock wrapped in fabric (subdrain) should be at least 12 inches wide and extend from the base of the wall to within 1.5 feet of the ground surface. The upper 1.5 feet of backfill should consist of compacted native soil. The retaining wall drainage system should drain to an appropriate discharge facility.

41. As an alternative to the drain rock and fabric, Miradrain 2000, 6000, or approved equivalent drain mat may be used behind the retaining wall. The drain mat should extend from the base of the wall to the ground surface. A perforated pipe (subdrain system) should be placed at the base of the wall in direct contact with the drain mat. The retaining wall drainage system should drain to an appropriate discharge facility.
42. We recommend a thorough review by our office of all designs pertaining to facilities retaining a soil mass.

## **DRAINAGE**

43. It is considered essential that positive drainage be provided during construction and be maintained throughout the life of the proposed structures.
44. The final exterior grade adjacent to the proposed structures should be such that the surface drainage will flow away from the structures. 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.
45. Utility lines that cross under the slab or through perimeter slab should be completely sealed to prevent moisture intrusion into the areas under the slab and/or perimeter. The utility trench backfill should be of impervious

material and this material should be placed at least 4 feet on either side of the exterior perimeter.

46. Consideration should be given to collection and diversion of roof runoff and the elimination of planted areas or other surfaces which could retain water in areas adjoining the building. The grade adjacent to the foundation should be sloped away from the structure at a minimum of 2 percent.
47. 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 foundation or concrete walkway, a moisture cut-off barrier should be provided.
48. 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 ( $K_{SAT} = 7.5 \times 10^{-5}$  cm/sec). This rate can be used in the design of the retention system for on-site storm drainage.

### **ABANDONMENT OF THE EXISTING UTILITY LINES**

49. All existing and abandoned utility lines located within the new building pad and basement area must be removed.
50. All abandoned utility lines within 2 feet from existing ground surface should be removed.
51. Removing the utility lines would require proper backfill and re-compaction of the excavation. Abandoning utility lines in-place would require to cap the abandoned portion of the pipe and all exposed pipe ends with concrete and the removal of any surface clean-outs, manhole or drain inlet structures.

## **ON-SITE UTILITY TRENCHING**

52. All on-site utility trenches must be backfilled with native on-site material or import fill and compacted to at least 90% relative maximum density. Backfill should be placed in 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.
53. 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 with minimum compression strength of 75 psi.
54. If utility trench excavation is to encounter groundwater, our office should be notified for dewatering recommendations.

## **PAVEMENT DESIGN**

55. 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 sections are based on our laboratory resistance R-Value test of near-surface soil samples and traffic indices (T.I.) of 4.5 for parking stalls and 5.5 for parking area and driveway (travel way). Alternate asphalt pavement section designs, which satisfy the State of California Standard Design Criteria, and above traffic indices, are presented in Table II. Concrete and paver pavement section designs are presented in Table III and IV. Due to

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

### **LIME TREATMENT ALTERNATIVES**

56. Lime treatment of the subgrade soil can be considered as an option in order to reduce the high expansion potential of near-surface native soil and/or to weather proof (winterize) the subgrade soil during the winter construction of the building pad or parking structure basement area. The top 12 inches of the subgrade can be treated with a mixture of 5% of quick lime (High Calcium) and native soil by volume. If the lime treatment is used, minor cracks on the concrete slab and separation of the curb/gutter and pavement should be expected. In the building pad area, if lime treatment would be implemented, the rock section could be reduced by one inch.
57. The lime-treated subgrade soil should not be exposed to the element for an extended period. If no improvements are planned for the immediate future, the lime-treated subgrade soil should be protected.

## LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations presented herein are based on the soil conditions revealed by our test borings and evaluated for the proposed construction planned at the present time. If any unusual soil conditions are encountered during the construction, or if the proposed construction will differ from that planned at the present time, Silicon Valley Soil Engineering (SVSE) should be notified for supplemental recommendations.
2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the necessary steps are taken to see that the contractor carries out the recommendations of this report in the field.
3. The findings of this report are valid, as of the present time. However, the passing of time will change the conditions of the existing property due to natural processes, works of man, from legislation or the broadening of knowledge. Therefore, this report is subjected to review and should not be relied upon after a period of three years.
4. The conclusions and recommendations presented in this report are professional opinions derived from current standards of geotechnical practice and no warranty is intended, expressed, or implied, is made or should be inferred.
5. The area of the borings is very small compared to the site area. As a result, buried structures such as septic tanks, storage tanks, abandoned utilities, or etc. may not be revealed in the borings during our field investigation. Therefore, if buried structures are encountered during grading or construction, our office should be notified immediately for proper disposal recommendations.

6. Standard maintenance should be expected after the initial construction has been completed. Should ownership of this property change hands, the prospective owner should be informed of this report and recommendations so as not to change the grading or block drainage facilities of this subject site.
7. This report has been prepared solely for the purpose of geotechnical investigation and does not include investigations for toxic contamination studies of soil or groundwater of any type. If there are any environmental concerns, our firm can provide additional studies.
8. Any work related to grading and/or foundation operations during construction performed without direct observation from SVSE personnel will invalidate the recommendations of this report and, furthermore, if we are not retained for observation services during construction, SVSE will cease to be the Geotechnical Engineer of Record for this subject site.

## REFERENCES

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- California Building Code (CBC, 2016) Title 24, Part 2.
- Helley E.J., Brabb, E.E., 1971 – Geologic map of Late Cenozoic deposits, Santa Clara County, California, U.S.G.S. MFS No. 335, Basic Data Contribution No. 27.
- 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 (1997). *Guidelines for Evaluating and Mitigating Seismic Hazards in California*. Special Publication 117. Department Of Conservation. Division of Mines and Geology.
- USGS (2000). CGS Seismic Hazard Zone Report 044 [*Seismic Hazard Evaluation of the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California. 2000 (Updated 1/17/06)*]. Department Of Conservation. Division of Mines and Geology].
- USGS (July 27, 2017), U.S. Seismic Design Maps <http://earthquake.usgs.gov/designmaps/us/application.php>.

## TABLES

**TABLE I – SUMMARY OF LABORATORY TESTS**

**TABLE II – PROPOSED ASPHALT PAVEMENT SECTIONS**

**TABLE III – PROPOSED CONCRETE PAVEMENT SECTIONS**

**TABLE IV – PROPOSED PAVER PAVEMENT SECTIONS**



**TABLE I****SUMMARY OF LABORATORY TESTS**

Sample No.	Depth (Feet)	In-Place Conditions		Direct Shear Testing		Atterberg Limits	
		Moisture Content (% Dry Wt.)	Dry Density (pcf)	Unit Cohesion (ksf)	Angle of Internal Friction (Degrees)	Liquid Limit L.L.	Plasticity Index P.I.
1-1	3	16.1	108.0				
1-2	5	21.8	100.9				
1-3	10	16.0	113.9			37	19
1-4	15	26.3	90.2			35	19
1-5	20	30.6	92.2			39	19
1-6	25	27.8	93.9			42	20
1-7	30	27.0	99.7			43	22
1-8	35	22.9	105.0			42	20
1-9	40	22.9	107.5			35	19
1-10	45	35.8	87.3			52	25
1-11	50	44.4	71.3			56	28
2-1	3	11.0	98.8	0.8	12		
2-2	5	8.0	101.4				
2-3	10	4.7	94.3	0.6	22	34	19
2-4	15	23.0	108.0				
2-5	20	33.8	86.3			46	22
2-6	25	28.7	94.7				
2-7	30	24.7	101.3			45	21

**TABLE I (CONTINUED)****SUMMARY OF LABORATORY TESTS**

Sample No.	Depth (Feet)	In-Place Conditions		Direct Shear Testing		Atterberg Limits	
		Moisture Content (% Dry Wt.)	Dry Density (pcf)	Unit Cohesion (ksf)	Angle of Internal Friction (Degrees)	Liquid Limit L.L.	Plasticity Index P.I.
2-8	35	29.2	91.1				
2-9	40	28.0	98.0			46	23
2-10	45	34.4	88.6			47	22
2-11	50	32.1	91.4				
2-12	55	31.6	93.2			48	24
2-13	60	33.4	90.1				
3-1	3	15.4	107.3				
3-2	5	22.6	102.8				
3-3	10	15.1	114.4				
3-4	15	25.9	93.7				
3-5	20	31.2	93.6				
4-1	3	12.2	99.3				
4-2	5	8.8	100.7				
4-3	10	5.5	97.8				
4-4	15	25.3	107.6				
4-5	20	32.1	85.9				

**TABLE II**

**PROPOSED ASPHALT PAVEMENT SECTIONS**

Location: Proposed Blossom Hill Development  
 397 Blossom Hill Road  
 San Jose, California

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

**TABLE III**

**PROPOSED CONCRETE PAVEMENT SECTIONS**

Location: Proposed Blossom Hill Development  
 397 Blossom Hill Road  
 San Jose, California

	<u>DRIVEWAY*</u>	<u>CURB &amp; GUTTER</u>	<u>SIDEWALK/PATIO**</u>
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 max. density	12.0"	12.0"	12.0"

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

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

**TABLE IV**

**PROPOSED PAVER PAVEMENT SECTIONS**

Location: Proposed Blossom Hill Development  
 397 Blossom Hill Road  
 San Jose, California

	<b><u>DRIVEWAY/PARKING AREA</u></b>			
<b>Recommended Paver Pavement Sections:</b>	<b>1A*</b>	<b>1B*</b>	<b>2A</b>	<b>2B</b>
<b>Vehicular Rated Pavers</b>	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
<b>ASTM No. 8 Bedding Course &amp; Paver Filler</b>	2.0"	2.0"	2.0"	2.0"
<b>3/4" Clean Crushed Rock or ASTM No. 57 Drain Stone</b>	12.0"	16.0"	---	---
<b>Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density</b>	---	---	12.0"	14.0"
<b>Subgrade soil scarified &amp; compacted to at least 90% relative max. density</b>	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 placed 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.

## FIGURES

FIGURE 1 – VICINITY MAP

FIGURE 2A – SITE PLAN – STUDY 1

FIGURE 2B – SITE PLAN – STUDY 2

FIGURE 3 – FAULT LOCATION MAP

FIGURE 4 – PLASTICITY INDEX

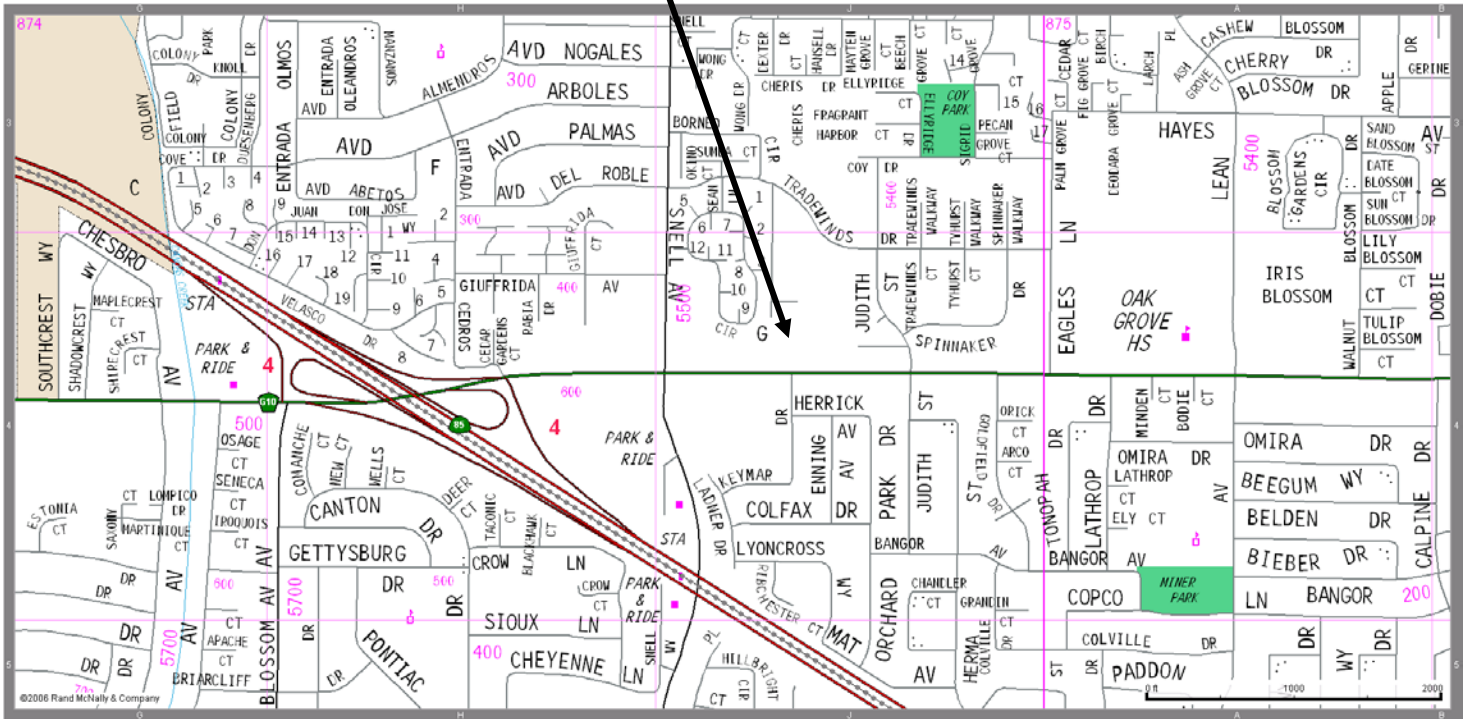
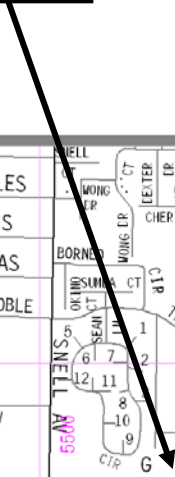
FIGURE 5 – COMPACTION TEST A

FIGURE 6 – R-VALUE TEST

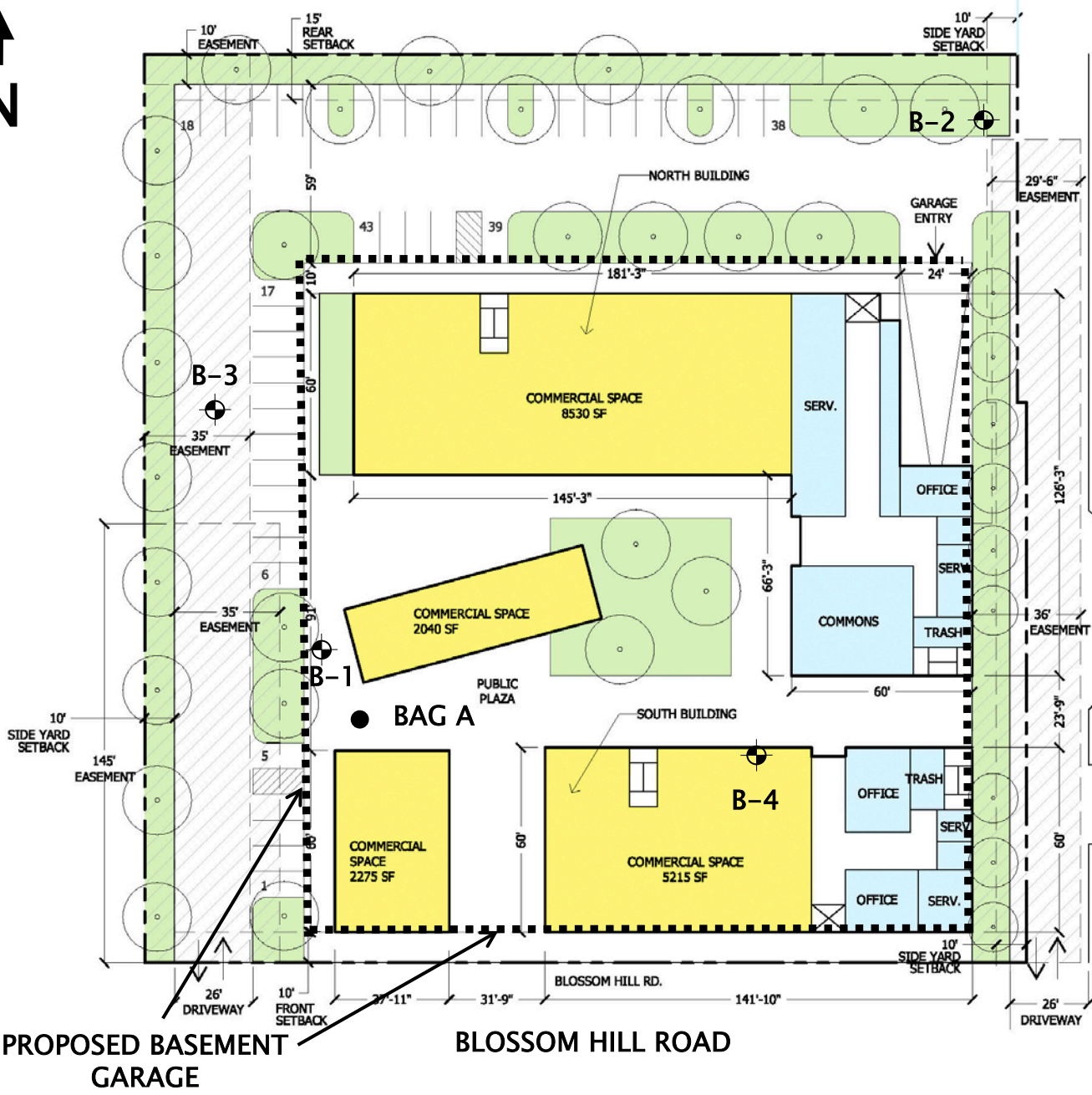
FIGURE 7 – LATERAL SOIL PRESSURES – SOLDIER PILE & WOOD  
LAGGING



FIGURE 8 – LATERAL SOIL PRESSURES – BASEMENT WALLS

**SITE**



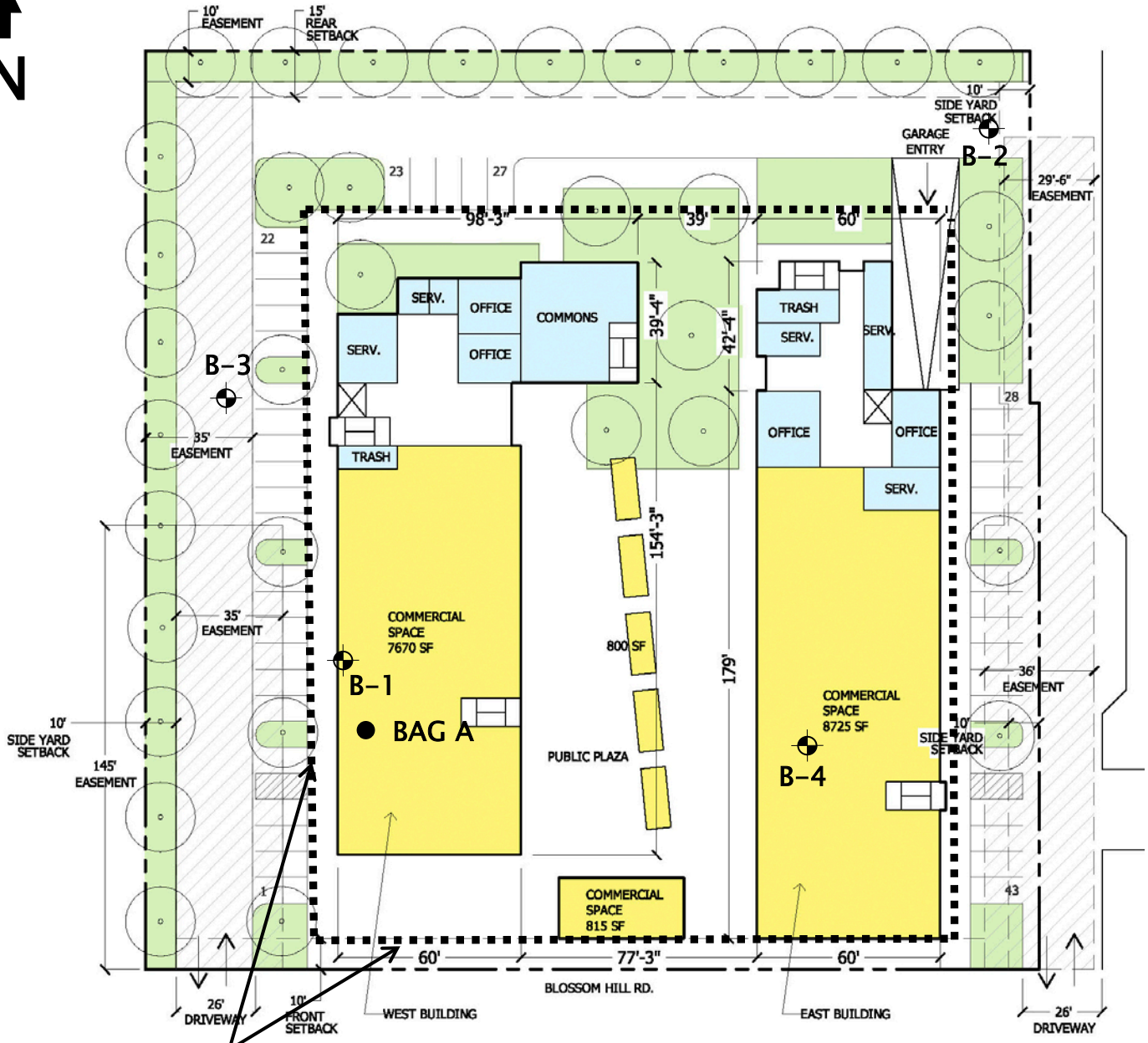
<p>Silicon Valley Soil Engineering</p> <p>2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400</p>	<p><b>VICINITY MAP</b></p> <p>Proposed Blossom Hill Development</p> <p>397 Blossom Hill Road San Jose, California</p>	<p>File No.: SV1742</p>	<p>FIGURE</p> <p>1</p>
		<p>Drawn by: V.V.</p>	
		<p>Scale: NOT TO SCALE</p>	<p>February 2018</p>



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	<b>SITE PLAN – STUDY 1</b>  Proposed Blossom Hill Development  397 Blossom Hill Road San Jose, California		File No.: SV1742	FIGURE
			Drawn by: V.V.	2A
			Scale: NOT TO SCALE	February 2018





**PROPOSED BASEMENT GARAGE**

**BLOSSOM HILL ROAD**

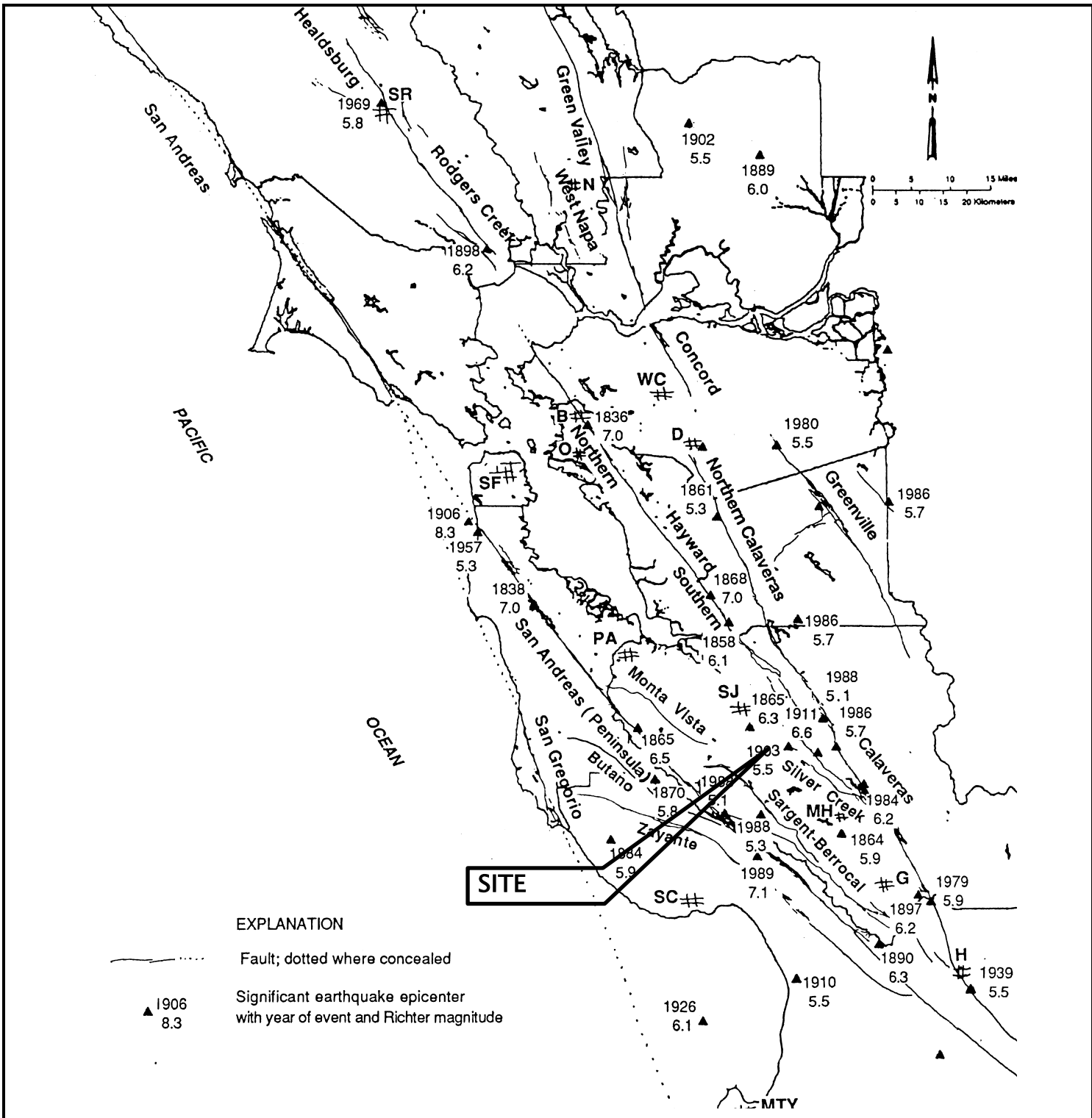
NOTE: DENOTES APPROXIMATE EXPLORATORY BORING LOCATION  
 DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

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

**SITE PLAN – STUDY 2**  
 Proposed Blossom Hill Development  
 397 Blossom Hill Road  
 San Jose, California

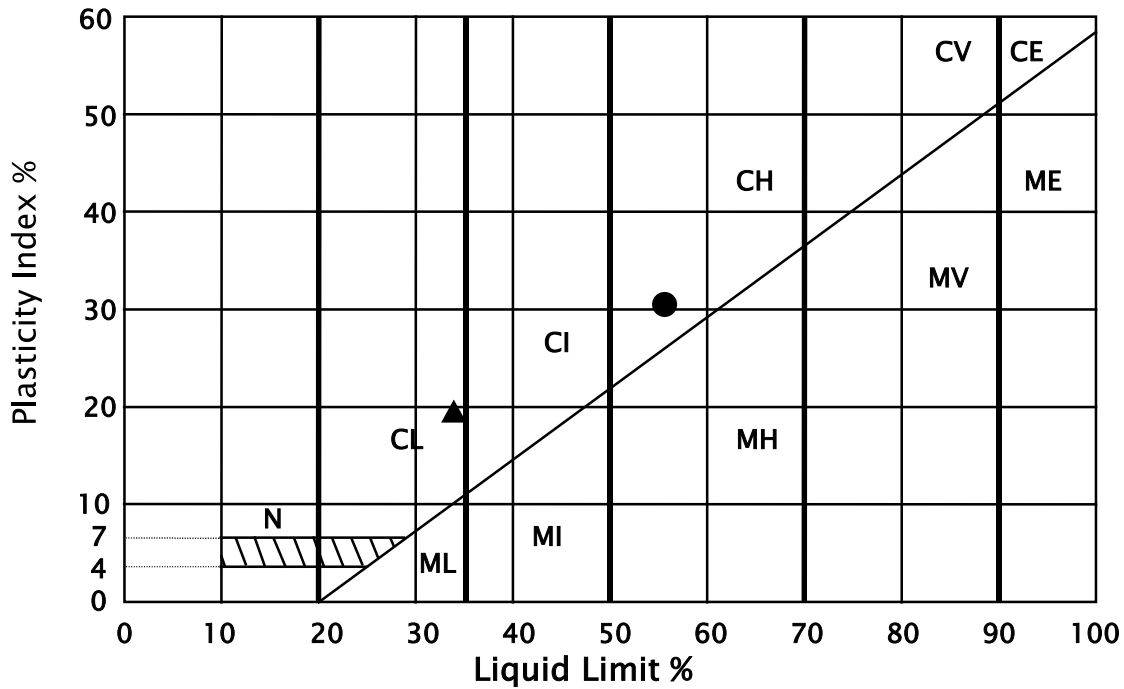
File No.: SV1742  
 Drawn by: V.V.  
 Scale: NOT TO SCALE

FIGURE  
 2B  
 February 2018



Silicon Valley Soil Engineering  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>FAULT LOCATION MAP</b>		File No.: SV1742	FIGURE  3
	Proposed Blossom Hill Development		Drawn by: V.V.	
	397 Blossom Hill Road San Jose, California		Scale: NOT TO SCALE	February 2018

### PLASTICITY CHART

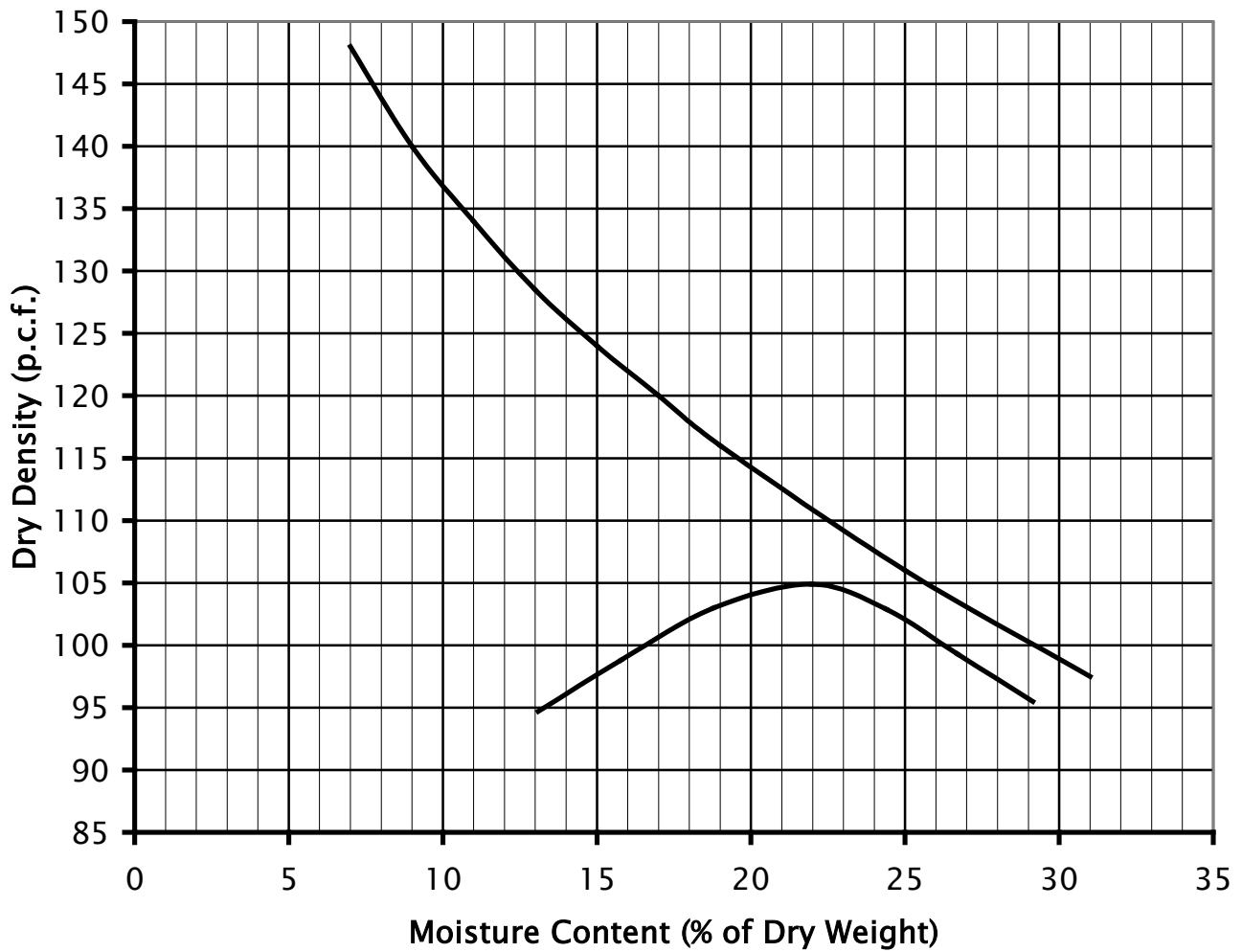


### PLASTICITY DATA

Key Symbol	Hole No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	55	30	CH
▲	1-4	13.5-15.0	35	19	CL

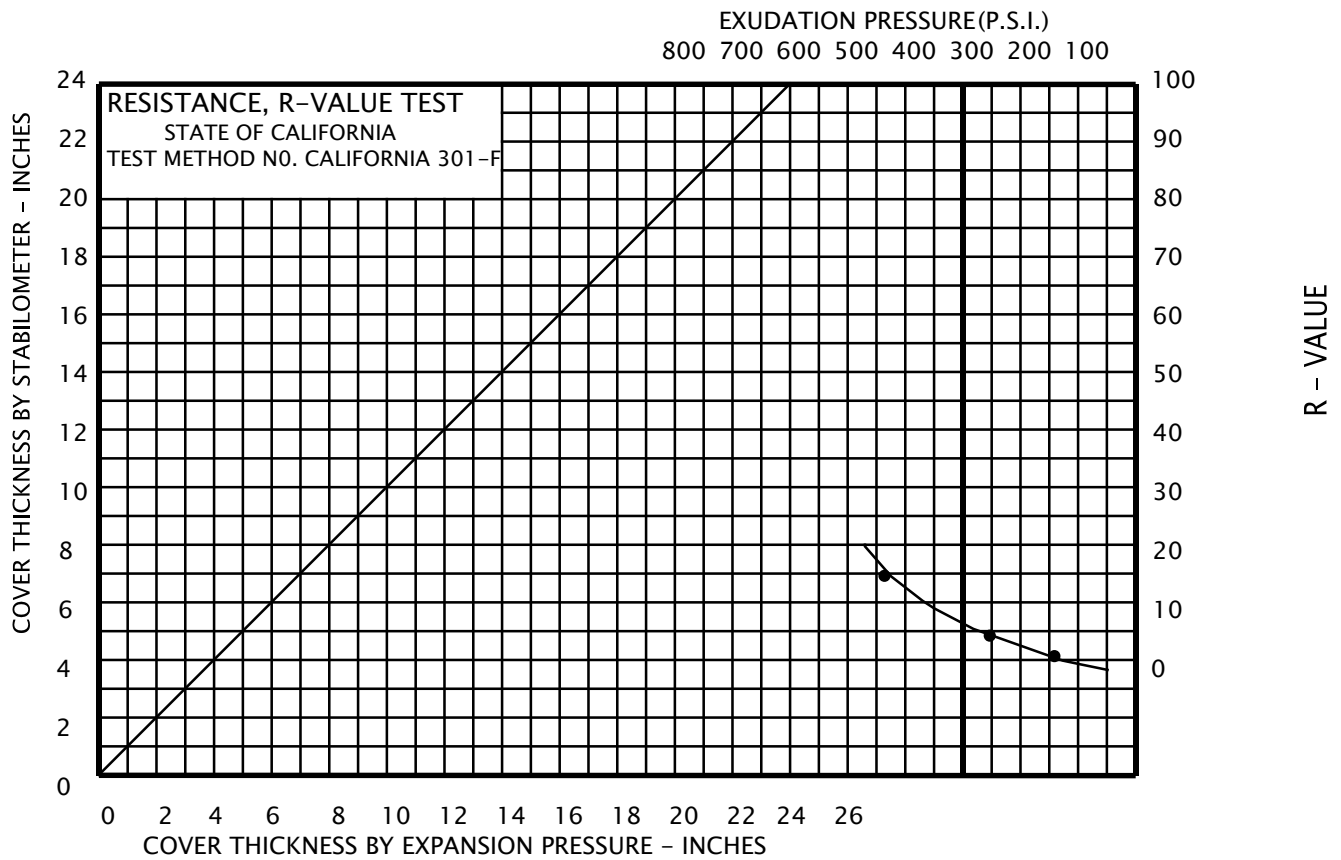
\*Soil type classification Based on British suggested revisions to Unified Soil Classification System

Silicon Valley Soil Engineering  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>PLASTICITY INDEX</b>  Proposed Blossom Hill Development  397 Blossom Hill Road San Jose, California	File No.: SV1742	FIGURE
		Drawn by: V.V.	4
		Scale: NOT TO SCALE	February 2018



**SAMPLE:** A  
**DESCRIPTION:** Black Silty CLAY  
**LABORATORY TEST PROCEDURE:** ASTM D1557  
**MAXIMUM DRY DENSITY:** 105.0 p.c.f.  
**OPTIMUM MOISTURE CONTENT:** 22.0 %

Silicon Valley Soil Engineering  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>COMPACTION TEST A</b>  Proposed Blossom Hill Development  397 Blossom Hill Road San Jose, California	File No. SV1742	FIGURE  5
		Drawn by: V.V.	
		Scale: NOT TO SCALE	February 2018

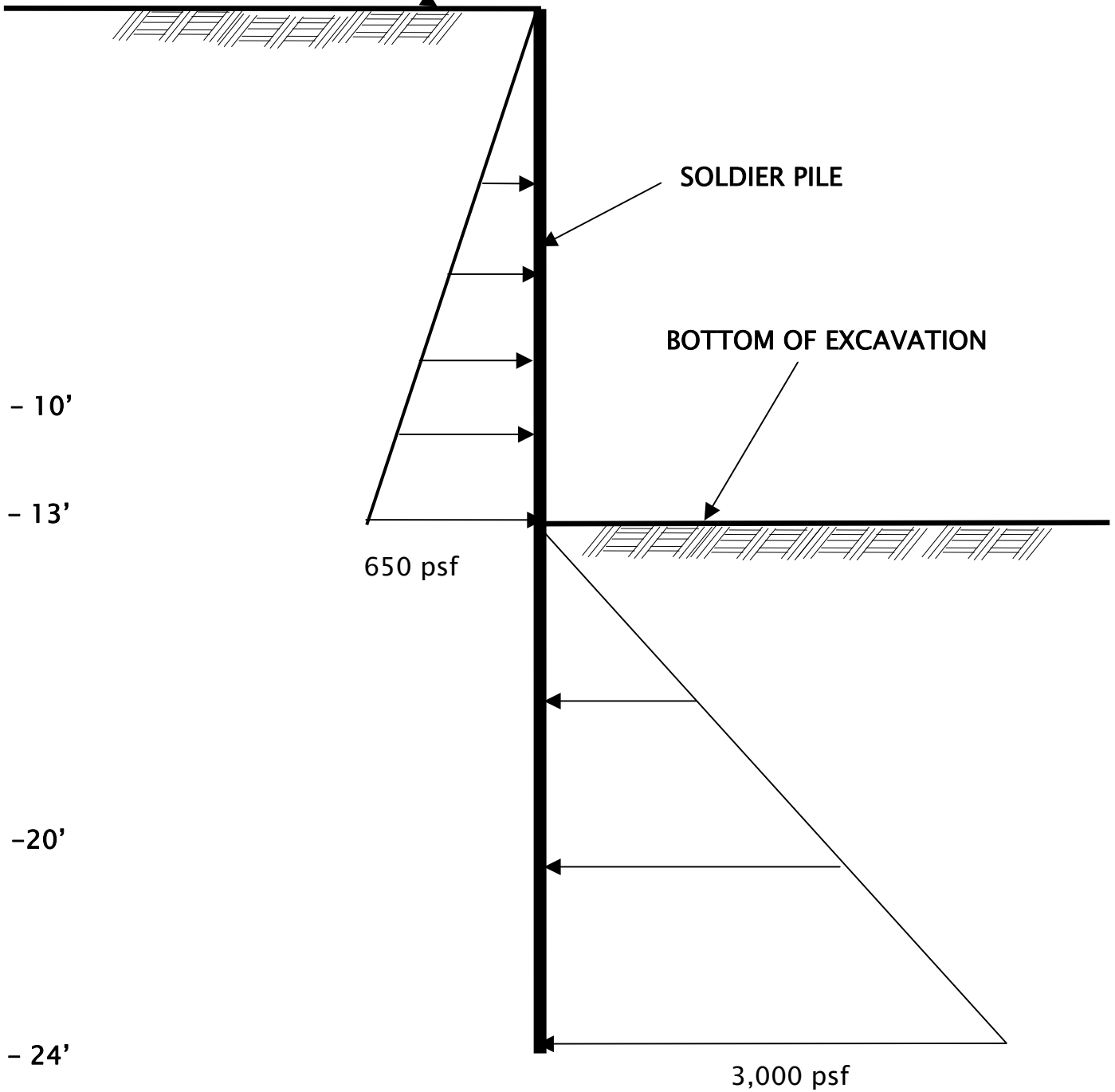


SAMPLE:           A  
DESCRIPTION:    Black Silty CLAY

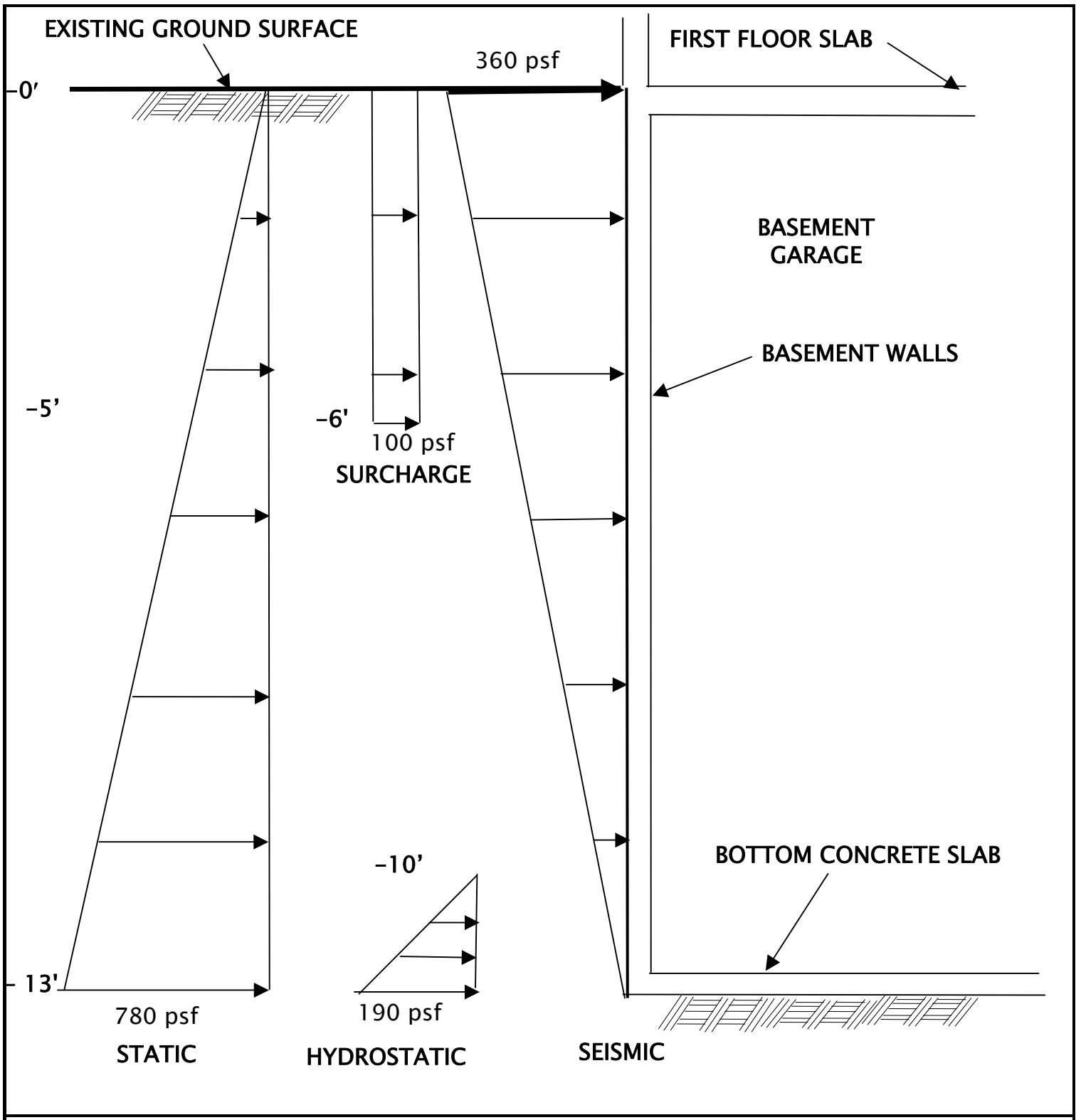
SPECIMEN	A	B	C
EXUDATION PRESSURE (P.S.I.)	149.0	251.0	449.0
EXPANSION DIAL (.0001")	9.0	14.0	20.0
EXPANSION PRESSURE (P.S.F.)	45.0	76.0	94.0
RESISTANCE VALUE, "R"	1.0	4.0	15.0
% MOISTURE AT TEST	20.7	18.0	17.6
DRY DENSITY AT TEST (P.C.F.)	106.7	108.5	111.2
R-VALUE AT 300 P.S.I. EXUDATION PRESSURE	=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

Silicon Valley Soil Engineering  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>R-VALUE TEST</b>	File No. SV1742	FIGURE
	Proposed Blossom Hill Development	Drawn by: V.V.	6
	397 Blossom Hill Road San Jose, California	Scale: NOT TO SCALE	February 2018

EXISTING GROUND SURFACE



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>LATERAL SOIL PRESSURES SOLDIER PILE &amp; WOOD LAGGING</b> Proposed Blossom Hill Development  397 Blossom Hill Road San Jose, California	File No.: SV1742	FIGURE
		Drawn by: V.V.	7
		Scale: NOT TO SCALE	February 2018



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>LATERAL SOIL PRESSURES          BASEMENT WALLS</b> Proposed Blossom Hill Development  397 Blossom Hill Road San Jose, California	File No.: SV1742	FIGURE
		Drawn by: V.V.	8
		Scale: NOT TO SCALE	February 2018

**APPENDICES**

**MODIFIED MERCALLI SCALE**

**METHOD OF SOIL CLASSIFICATION**

**KEY TO LOG OF BORING**

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

**SANTA CLARA VALLEY WATER DISTRICT DRILLING PERMIT**



**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.	Architectural 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.

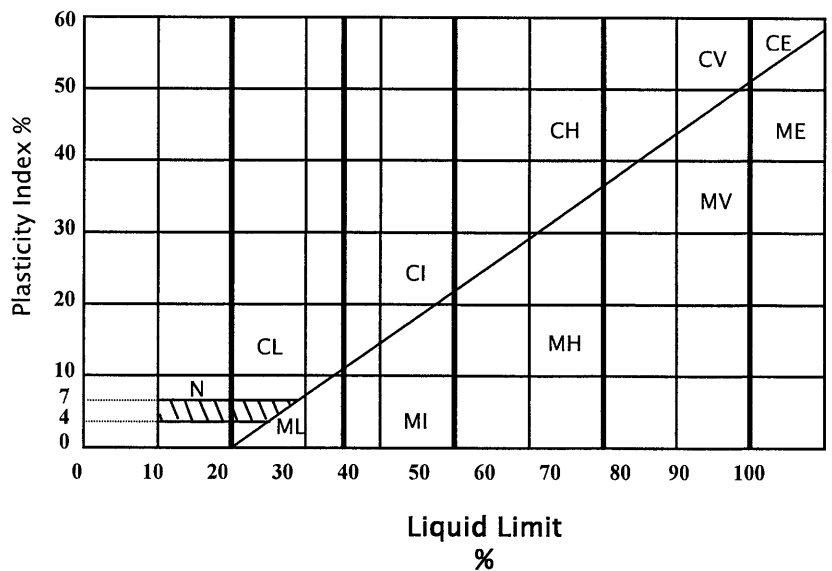
## METHOD OF SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > no. 200 sieve size)	<u>GRAVELS</u>	GW	Well graded gravel or gravel-sand mixtures, little or no fines
	(More than 1/2 of coarse fraction > no. 4 sieve size)	GP	Poorly graded gravel or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey Gravels, gravel-sand-clay mixtures
		<u>SANDS</u>	SW
	(More than 1/2 of coarse fraction < no. 4 sieve size)	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)	<u>SILTS &amp; CLAYS</u>	ML	Inorganic silts and very fine sand, rock, flour, silty or clayey fine sand or clayey silt/slight plasticity
	<u>LL &lt; 50</u>	CL	Inorganic clay of low to medium plasticity, gravelly clays, sandy clay, silty clay, lean clays
		OL	Organic silts and organic silty clay of low plasticity
		<u>SILTS &amp; CLAYS</u>	MH
	<u>LL &gt; 50</u>	CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
<u>HIGHLY ORGANIC SOIL</u>		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	3" to No. 4	76.2 to 4.76
	3" to 3/4"	76.2 to 19.1
	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
	No. 4 to No. 10	4.76 to 2.00
	No.10 to No. 40	2.00 to 0.420
	No.40 to No. 200	0.420 to 0.074
SILT AND CLAY	Below No. 200	Below 0.074

PLASTICITY INDEX CHART



**Project:** Proposed Blossom Hill Development  
**Project Location:** 397 Blossom Hill Rd. San Jose, California  
**Project Number:** SV1742

**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**

1	2	3	4	5	6	7	8	9	10	11	12	13
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, %








**COLUMN DESCRIPTIONS**

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







**FIELD AND LABORATORY TEST ABBREVIATIONS**

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






**MATERIAL GRAPHIC SYMBOLS**

-  Asphaltic Concrete (AC)
-  Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)
-  Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
-  Lean-Fat CLAY, CLAY w/SAND, SANDY CLAY (CL-CH)
-  Aggregate Base (AB)
-  Clayey SAND to Sandy CLAY (SC-CL)
-  Poorly graded SAND with Clay (SP-SC)

**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

**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.





<b>Project:</b> Proposed Blossom Hill Development <b>Project Location:</b> 397 Blossom Hill Rd. San Jose, California <b>Project Number:</b> SV1742	<b>Silicon Valley Soil Engineering</b> 2391 Zanker Road, Suite 350 San Jose, CA 95131 (408) 324-1400	<b>Log of Boring B-2</b> <b>Sheet 1 of 2</b>
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Date(s) Drilled: <b>01/29/18</b>	Logged By: <b>V.V.</b>	Checked By:
Drilling Method: <b>Hollow Stem Auger</b>	Drill Bit Size/Type: <b>8-inch</b>	Total Depth of Borehole: <b>60.0 feet</b>
Groundwater Level and Date Measured: <b>15 feet (01/29/18)</b>	Sampling Method(s): <b>SPT</b>	Approximate Surface Elevation: <b>feet</b>
Borehole Backfill: <b>Grout</b>	Location:	Hammer Data: <b>140 lbs</b>

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				CL		Dark Olive Brown Silty CLAY Moist, stiff						
2-1		14	14			Color changed to medium brown	11.0	98.8	0.8	12		
2-2		21	21	ML		Light Brown Sandy SILT Damp, very stiff	8.0	101.4				
2-3		23	23	CL-ML		Light Tan Clayey SILT Moist, very stiff	4.7	94.3	0.6	22	34	19
2-4		8	8	CL		Light Brown Silty CLAY Moist stiff	23.0	108.0				
15						Stabilized at drilling completion						
2-5		7	7			Color changed to olive brown Color changed to brown	33.8	86.3			46	22
2-6		8	8			First encountered	28.7	94.7				
2-7		11	11				24.7	101.3			45	21






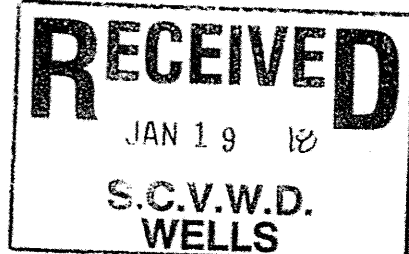
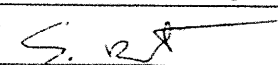
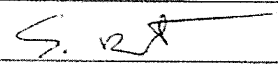
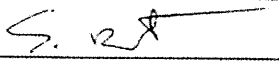
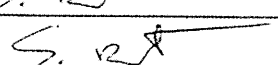




SV1742

Santa Clara Valley Water District | 5750 Almaden Expressway  
San Jose, CA 95118-3686  
(408) 265-2600

**APPLICATION TO DRILL EXPLORATORY BORINGS**  
FC 285 (03-26-15)  
Page 1 of 2

Date Issued: 1-26-18		Expiration Date: 1-26-19		District Permit No.: E2018012600	
Client (if different from property owner): Blossom Hill, L.P.		Property Owner: Blossom Hill, L.P.		Name of Business/Residence at Site: Proposed Blossom Hill Development	
Client's Address: 1400 Parkmoor Avenue, #190 City, State, Zip: San Jose, CA 95126		Property Owner's Address: 1400 Parkmoor Avenue, #190 City, State, Zip: San Jose, CA 95126		Address of Site: 397 Blossom Hill Road City, State, Zip: San Jose, California 95123	
Telephone No.: 408-550-8308 Sandra		Telephone No.: 408-550-8308 Sandra-Charitites Housing		Assessor's Parcel No. of Site: Book 690 Page 25 Parcel 021	
Consulting Company Name: Silicon Valleys Soil Engineering			Drilling Company Name: Exploration Geoservices, Inc.		
Address: 2391 Zanker Road, Suite 350 City, State, Zip: San Jose, CA 95131			Address: 1535 Industrial Avenue City, State, Zip: San Jose, CA 95112		
Telephone No.: 408-324-1400		Telephone No.: 408-280-6822		C-57/C-61 License No.: 484288	
<input type="checkbox"/> Check if address or phone number has changed			<input type="checkbox"/> Check if address or phone number has changed		
<p>In space at right, sketch location of proposed boring(s) in sufficient detail to identify location. In addition to distances to nearest street and intersection, show distances to any existing structures, landmarks, or topographic features.</p> <p>How many borings will be installed on parcel? 2</p> <p><input type="checkbox"/> Proposed borings on District property/easement (See General Condition F, page 2.)</p> <p><input type="checkbox"/> Within 50 feet of the top of a creek bank or District facility</p> <p>Proposed depth of boring(s):</p> <p><input checked="" type="checkbox"/> 45 to 150 feet</p> <p><input type="checkbox"/> 151 to 300 feet</p> <p><input type="checkbox"/> Over 300 feet</p> <p><b>NOTE: No permit is required for borings under 45 feet deep.</b></p>			<p><b>SITE PLAN</b> (Please draw accurately)</p> <p>See Attached</p> <div style="text-align: right;">  </div> <div style="text-align: center;">  </div>		
<p>Boring Type:</p> <p><input checked="" type="checkbox"/> Hollow stem</p> <p><input type="checkbox"/> Rotary</p> <p><input type="checkbox"/> CPT</p> <p><input type="checkbox"/> Hydropunch</p> <p><input type="checkbox"/> Other:</p>		<p>Boring Use:</p> <p><input checked="" type="checkbox"/> Geotechnical Investigation</p> <p><input type="checkbox"/> Environmental Investigation</p> <p><input type="checkbox"/> Material Emplacement</p> <p><input type="checkbox"/> Sensor Emplacement</p> <p><input type="checkbox"/> Other:</p>			
<b>SIGNATURES</b>					
I understand and agree that all work associated with this permit is required to be done in accordance with Santa Clara Valley Water District (District) Well Ordinance 90-1, the District Well Standards, and conditions of this permit (see page 2). I certify that the information given in this permit is correct to the best of my knowledge and that the signature below, whether original, electronic, or photocopied, is authorized and valid, and is affixed with the intent to be enforceable. I also certify that a right of entry/encroachment agreement has been formalized between the well owner and property owner, if parties differ.					
Signature of Property Owner/Agent: 		Print/Type Name: Sean Deivert		Date: 1/18/18	
Signature of Client/Agent: 		Print/Type Name: Sean Deivert		Date: 1/18/18	
Signature of Driller/Agent: 		Print/Type Name: Sean Deivert		Date: 1/18/18	
Signature of Consultant/Agent: 		Print/Type Name: Sean Deivert		Date: 1/18/18	



5750 Almaden Expressway  
 San Jose, CA 95118-3686  
 (408) 265-2600

# APPLICATION TO DRILL EXPLORATORY BORINGS

FC 285 (03-26-15)  
 Page 2 of 2

**IMPORTANT: A minimum 24-hour notice must be given to Santa Clara Valley Water District Well Inspection Department prior to installing the annular seal. Call (408) 265-2607, ext. 2660. Please allow 10 working days to process permit application.**

**GENERAL CONDITIONS**

- A. District (telephone 408-630-2660) **must be notified a minimum of one working day before the exploratory boring is backfilled.** An authorized District representative must be on site to witness the sealing operation. This requirement may be waived by an authorized District representative. If the District waives the inspection requirement, the District may request the permittee(s) to furnish certification under penalty of perjury that the seal was constructed in accordance with the District Well Standards.
- B. This permit is valid only for the purpose specified herein. Boring destruction methods authorized under this permit may not be changed except by written approval of an authorized District representative, and only if the District believes that such a change will result in equal or superior compliance with the District and State Well Standards (e.g., if the District representative finds that site conditions warrant such a change).
- C. This permit is only valid for the Assessor's Parcel No. indicated on it.
- D. This permit may be voided if it contains incorrect information.
- E. Borings shall be sealed within 24 hours following completion of testing or sampling activities. Borings shall not be left in such a condition as to allow for the introduction of surface waters or foreign materials into them. Borings shall be secured such that they do not endanger public health.
- F. If any work associated with this permit will take place on District property/easement, an encroachment or construction permit must be granted by the District's Community Projects Review Unit (telephone 408-630-2350, -2217, or -2253).
- G. The permittee(s) shall assume entire responsibility for all activities and uses under this permit and shall indemnify, defend, and hold the District, its officers, agents, and employees, free and harmless from any and all expense, cost, and liability in connection with or resulting from the granting or exercise of this permit including, but not limited to, property damage, personal injury, and wrongful death.
- H. Permittees are required to be in full compliance with Cal/OSHA California Labor Code Section 6300.
- I. A current C-57 or C-61 Contractor's License is required for work associated with this permit.
- J. Permittee, permittee's contractors, consultants, or agents shall be responsible to assure that all materials or waters generated during drilling, boring destruction, and/or other activities associated with this permit will be safely handled, properly managed, and disposed of according to all applicable federal, state, and local statues regulating such. In no case shall these materials and/or waters be allowed to enter, or potentially enter, on- or off-site storm sewers, dry wells, or waterways or be allowed to move off the property where the work is being completed.
- K. The driller and consultants (if applicable) shall have an active copy of their Worker's Compensation Insurance on file with District.
- L. This permit shall expire if not exercised within 180 calendar days of its approval, unless an extension of the permit expiration date is granted by an authorized District representative.
- M. This permit shall be kept on site during all activities associated with it and shall immediately be presented to an authorized District representative upon request.
- N. Permittee shall notify Underground Service Alert (USA) at 1-800-227-2600 or 811 prior to any digging.

Permit Approved by:

*[Handwritten Signature]*

Date:

1-26-18

**Please allow 10 working days to process this application.**