REPORT TO
LC ENGINEERING
SAN JOSE, CALIFORNIA

FOR

PROPOSED THREE-STORY RETAIL AND OFFICE BUILDING

SOUTHERN CORNER OF
S. KING ROAD AND STORY ROAD
SAN JOSE, CALIFORNIA

GEOTECHNICAL INVESTIGATION

JUNE 2014

PREPARED BY

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File No. SV1267
June 4, 2014

LC Engineering
1291 Oakland Road
San Jose, CA 95112

Attention: Mr. Ninh Le

Subject: Proposed Three-Story Retail and Office Building
Southern Corner of S. King Road & Story Road
San Jose, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Le:

We are pleased to transmit herein the results of our geotechnical investigation for the proposed Three-story retail and office building. The subject site is located on the southern corner of S. King Road and Story 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

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INTRODUCTION

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

SITE LOCATION AND DESCRIPTION

The subject site is located on the southern corner of S. King Road and Story Road in San Jose, California (Figure 1). S. King Road bound the subject site to the northeast and paved parking and driveways to the southeast, southwest, and northwest. At the time of this investigation, the subject site is an asphalt concrete paved parking stalls and driveways. Based on the available information for the subject site, the development will include the removal of the existing pavement section and the construction of a three-story retail and office building with associated improvements. The approximate location of the proposed building and our borings are shown on the Site Plan (Figure 2).

FIELD INVESTIGATION

After considering the nature of the proposed development and reviewing available data on the area, our geotechnical engineer conducted a field investigation at the project site. It included a site reconnaissance to detect any unusual surface features, and the drilling of two exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on May 28, 2014. The approximate location of the borings is shown on the Site Plan

(Figure 2). The borings were drilled to the depths of 11.5 feet and 51.5 feet below the existing pavement surface. The borings were drilled with a truck mounted drill rig using 8-inch diameter hollow stem augers.

The soils encountered were logged continuously in the field during the drilling operation. Relatively undisturbed soil samples were obtained by hammering a 2.0-inch outside diameter (O.D.) split-tube sampler for a Standard Penetration Test (S.P.T.), A.S.T.M. 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. After the completion of the drilling operation, the exploratory borings were backfilled from the bottom of the borehole to the surface with neat cement in accordance to the rules and regulations of the Santa Clara Valley Water District.

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.

3. The strength parameters of the foundation soils were determined from direct shear tests that were performed on selected relatively undisturbed soil samples. Laboratory compaction tests were performed on the near-surface material per the ASTM D1557-12 test procedure.

4. Grain size distribution analyses (sieve and hydrometer) were performed on suspected liquefiable soil to assist in their classification and gradation.

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

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 (51.5 feet boring), the existing pavement consists of 3 inches of asphalt concrete over 6 inches of aggregate base material. Below the pavement section to the depth of 8 feet, a black, moist, very stiff silty clay layer was encountered. A color change of brown was noted at the depth of 5 feet. From the depths of 8 feet to 13 feet, the soil became olive brown, moist, very stiff sandy clay/clayey sand. From the depths of 13 feet to 18 feet, an olive brown, moist, medium dense, silty clayey sand layer was encountered. The sand was medium grained and poorly graded. From the depths of 18 feet to 30 feet, the soil became bluish gray, moist, very stiff, silty clay. From the depths of 30 feet to 35 feet, an olive brown, moist, dense silty sand layer was encountered. The sand was medium grained and poorly graded. From the depths of 35 feet to the end of the boring at 51.5 feet, the soil became greenish gray, moist, very stiff silty clay. Similar soil profiles were encountered in other borings.

Groundwater was initially encountered in Boring B-1 at the depth of 14 feet and rose to a static level of 13 feet at the end of the drilling operation. It should be noted that the groundwater level would fluctuate as a result of seasonal changes and hydrogeological variations such as groundwater pumping and/or recharging. A graphic description of the explored soil profiles is presented in the Exploratory Boring Log contained in the Appendix.

GENERAL GEOLOGY

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

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

The Quaternary history of the region is recorded by sedimentary marine strata alternating with non-marine strata. The changes of the depositional environment are related to the fluctuation of sea level corresponding to the glacial and interglacial periods. Late Quaternary deposits fill the center of the San Francisco Bay Region and most of the strata are of continental origin characterized as alluvial and fluvial materials.

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

LIQUEFACTION ANALYSIS

A. GROUNDWATER

Groundwater was initially encountered in the borings at depths of 14 feet and rose to a static level ranging from 13 feet at the end of the drilling operation. 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 (Revised 01/17/2006).* Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 8 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. 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 (S.P.T.), A.S.T.M. Standard D1586-92.

The results from our exploratory boring show that the subsurface soil material in Boring B-1 to the depth of 51.5 feet consists of very stiff silty clay to very stiff sandy clay/clayey sand to medium dense silty clayey sand to very stiff silty clay to dense silty sand to very stiff silty clay. The following is the determination of the liquefiable soil for each soil layer in Boring B-1.

1. The very stiff silty clay layer from the surface to the depth of 8 feet <u>is not liquefiable</u> soil because it is above the groundwater table.

2. The very stiff sandy clay/clayey sand layer from the depth of 8 feet to 13 feet <u>is not liquefiable</u> soil because based on the Plasticity Index (PI) and moisture contents (MC):

   - Sample No. 1-3 (10 feet) - [PI > 12; PI = 17 and MC = 21.3% < 85% LL = 31.5% ; LL = 37]

3. The medium dense silty clayey sand layer from the depths of 13 feet to 18 feet <u>is liquefiable</u> soil based on the PI (PI<12).

4. The very stiff silty clay layer from the depth of 18 feet to 30 feet <u>is not liquefiable</u> soil because based on the Plasticity Index (PI) and moisture contents (MC):

- Sample No. 1-5 (20 feet) - [PI > 12; PI = 23 and MC = 24.6% < 85% LL = 40.0%; LL = 47]

- Sample No. 1-6 (25 feet) - [PI > 12; PI = 22 and MC = 26.5% < 85% LL = 40.8%; LL = 48]

5. The dense silty sand layer from the depth of 30 feet to 35 feet is not liquefiable soil because of the high blow counts.

6. The very stiff silty clay layer from the depth of 35 feet to the end of the boring at 51.5 feet is not liquefiable soil because based on the Plasticity Index (PI) and moisture contents (MC):

- Sample No. 1-8 (35 feet) - [PI > 12; PI = 19 and MC = 25.5 % < 85% LL = 37.4%; LL = 44]

- Sample No. 1-9 (40 feet) - [PI > 12; PI = 23 and MC = 21.6% < 85% LL = 38.3%; LL = 45]

- Sample No. 1-10 (45 feet) - [PI > 12; PI = 20 and MC = 27.1% < 85% LL = 36.6%; LL = 43]

- Sample No. 1-11 (50 feet) - [PI > 12; PI = 22 and MC = 29.9% < 85% LL = 38.3%; LL = 45]

In summary, there is one liquefiable soil layer underlying the subject site. This layer is the medium dense silty clayey sand layer from the depths of 13 feet to 18 feet (5 feet in thickness).

C. STANDARD PENETRATION TEST N_{60} VALUE CORRECTED FOR FIELD TESTING PROCEDURES

The measured standard penetration test (SPT) N value can be influenced by many factors including the soil types, groundwater conditions, and different field-testing procedures. The N values corrected for field-testing procedure are calculated using the following formula as suggested by *Skempton 1986*:

$N_{60} = 1.67\ E_m C_b C_r N$

N_{60} = standard penetration test N value corrected for field testing procedures

E_m = hammer efficiency ($E_m = 0.6$ for U.S. equipment)

C_b = bore hole diameter corrections ($C_b = 1.05$ for 150-mm diameter bore hole)

C_r = rod length correction:

$C_r = 0.75$ for rod length up to 13.1 feet

$C_r = 0.85$ for rod length between 13.1 to 19.7 feet

$C_r = 0.95$ for rod length between 19.7 to 32.8 feet

N = measured standard penetration test N value

The N_{60} values are shown in Table V.

D. STANDARD PENETRATION TEST $(N1)_{60}$ VALUE CORRECTED FOR FIELD TESTING PROCEDURES AND OVERBURDEN PRESSURE

For this liquefaction study, the standard penetration test N_{60} value is corrected for the vertical effective stress by the following formula:

$(N_1)_{60} = C_N N_{60} = (100/\sigma'_{ov})^{0.5} N_{60}$

$(N_1)_{60}$ = standard penetration test N value corrected for both field testing procedures and overburden pressure

C_N = correction factor to account for overburden pressure obtained from Figure 7 using the above data

N_{60} = standard penetration test N value corrected for field testing procedures

σ'_{ov} = vertical effective stress, also called the effective overburden pressure

The $(N_1)_{60}$ values for the suspected liquefiable silty clayey sand layer in Boring B-1 is shown in Table V.

E. PEAK GROUND ACCELERATION

The ground motion caused by earthquakes is generally characterizes in terms of ground surface displacement, velocity, and acceleration. For this liquefaction study, the measure of the cyclic ground motion is represented by the maximum horizontal acceleration at the ground surface, a_{max}. The maximum horizontal acceleration at ground surface is also called the peak horizontal ground acceleration. The value of peak ground acceleration is usually based on prior earthquake and faults studies because it is not possible to predict earthquakes. 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*. 2006 (revised 01/17/2006). Department of Conservation. Division of Mines and Geology], the peak ground acceleration is 0.61g.

F. CYCLIC STRESS RATIO (CSR) CAUSED BY THE EARTHQUAKE

The cyclic stress ratio (CSR) induced by the earthquake is a function of the depth of the liquefiable soil layer, peak ground acceleration, total vertical stress, effective vertical stress, and ground water depth. The CSR can be calculated using the following formula:

$CSR = 0.65\, r_d(\sigma_{ov}/(\sigma'_{ov})(\, a_{max}/g)$

CSR = cyclic stress ratio, also called seismic stress ratio

r_d = depth reduction factor, also called stress reduction coefficient

σ_{ov} = total vertical stress at a particular depth where the liquefaction analysis is performed

σ'_{ov} = vertical effective stress at the same depth in soil deposit where σ_{ov} was calculated

a_{max} = maximum horizontal acceleration at ground surface that is induced by the earthquake, also called peak ground acceleration

g = acceleration of gravity

The above formula was developed with the assumptions that the ground surface is level and the unit width and length soil column will move horizontally as a rigid body. The horizontal movement of the soil column is the result of the maximum horizontal accelerations, a_{max}, exerted by the earthquake at ground surface. The depth reduction factor was introduced to account for the fact that the soil column does not behave as a rigid body. The depth reduction factor was obtained from Figure 8, which was reproduced from *Andrus and Stokoe 2000.* The CSRs, which were computed using the above formula for the suspected liquefiable soil layer in Boring B-1 is shown in Table V.

G. CYCLIC RESISTANCE RATIO (CRR)

The cyclic resistance ratio (CRR) represents the liquefaction resistance of the in situ soil. The CRR of the suspected liquefiable soil layer was obtained from Figure 9. This figure was reproduced from *Seed et al. 1985.* In using Figure 9 the following data were entered:

1. *Standard penetration test* $(N_1)_{60}$ *value.*
2. *Percent fines.*
3. *Cyclic resistance ratio* (CRR) *for an anticipated magnitude 7.5 earthquake.*

4. *Correction for other magnitude earthquakes:* At higher magnitude values, the magnitude scaling factor can be determine from Figure 10. This figure was reproduced from *Andrus and Stokoe 2000*.

H. FACTOR OF SAFETY AGAINST LIQUEFACTION (FS)

The factor of safety against liquefaction is defined as follows:

FS = CRR/ CSR

The higher the safety factor, the more resistant the soil is to liquefaction. A factor of safety against liquefaction greater than 1.3 indicates that the level of risk associated with liquefaction hazard is acceptable. A factor of safety against liquefaction less than 1.3 indicates that an evaluation of the severity of the hazard associated with potential liquefaction of the suspected liquefiable soil layer should be performed. The severity of the hazard includes liquefaction-induced ground surface settlements and liquefaction-induced ground damage. Factors of safety against liquefaction, which were calculated for the suspected liquefiable silty clayey sand layer in exploratory Boring B-1, is shown in Table V. Based on the factor of safety against liquefaction (F.S. = 0.35 < 1.3), it is probable that during the anticipated earthquake, the silty clayey sand layer from the depths of 13 feet to 18 feet will liquefy.

I. SETTLEMENT VERSUS FACTOR OF SAFETY AGAINST LIQUEFACTION

As we have evaluated, the suspected liquefiable soil layer will liquefy. Accordingly, the settlement will occur as water flows from the soil in response to the earthquake-induced excess pore water pressures. To estimate the ground surface settlement of the suspected liquefiable soil layer, we used Figure 11. This figure was reproduced from *Kramer 1996, which was originally developed by Tokimatsu and Seed 1984*. In using Figure 11, the CSR and $(N_1)_{60}$

values of the suspected liquefiable soil layer in exploratory Boring B-3 were entered. Two types of settlements will be considered.

1. Total Settlement. The estimated total settlements were calculated from the resulting volumetric strain percentage applying to the suspected liquefiable soil layer thickness.

2. Differential Settlement. Because of variable soil conditions and structural loads, the earthquake-induced settlements are rarely uniform. As a result, the foundation will experience differential settlement. A common assumption as suggested by *Robert W. Day 2002 and The California Department of Conservation Division of Mines and Geology GUIDELINES* is that the maximum differential settlement of the foundation will be equal to 50 to 75 percent of the maximum total settlement.

The maximum total settlement is estimated to be 1.0 inch. Accordingly, the differential settlement is estimated to be 0.75 inch. The total and differential settlements are considered minimal.

J. LIQUEFACTION-INDUCED GROUND DAMAGE

In addition to the ground surface settlements, there could be also liquefaction-induced ground damage that causes settlement of structures. The ground damage may include sand boils and/or surface fissures. To evaluate liquefaction-induced ground damage, we use Figure 12. These figures were reproduced from *Kramer 1996, which was originally developed by Ishihara 1985*. In plotting the coordinates of the suspected liquefiable soil layer of Boring B-1 in Figure 12, the thickness of surface non-liquefiable (H_1){13 feet in thickness} soil layers and the thickness of the liquefiable (H_2){5 feet in thickness} layer in Boring B-1 were entered with a maximum peak acceleration of $a_{max} = 0.61g$. The following is the determination of H_1 and H_2 in Boring B-1.

Boring~~s~~ B-1: H_1 = 4.33 meters; H_2 = 1.67 meters ??

Based on the plotted coordinates of the suspected liquefiable soil layer of Boring B-1 using the above data, we concluded that there is a marginally minimal potential for liquefaction-induced ground damage to occur at the site.

K. LATERAL SPREADING

In addition to liquefaction-induced ground damage, the liquefaction may also cause lateral movement of the ground surface. The liquefaction-induced lateral spreading may damage the building foundation and underground utility lines. Due to the close proximity to the existing Coyote Creek located easterly of the site, a lateral spreading study was performed for the site. A revised empirical method developed by *Youd, Hansen and Barlett (2002)* was used in this study to estimate the amount of lateral movement of the ground surface. The following revised multi-linear regression equation was used for the gently sloping ground condition:

$$\text{Log } DH = -16.213 + 1.532M - 1.406 \log R^* - 0.012R + 0.338 \log S +$$

$$0.540 \log T15 + 3.413 \log (100 - F15) - 0.795\log(D5015 + 0.1 \text{ mm})$$

Where:

D_H = Horizontal ground displacement in meters

M = Earthquake magnitude

R = Distance to the nearest fault rupture in kilometers

T_{15} = Cumulative thickness of saturated granular layers with corrected blow counts, $(N_1)_{60}$, less than 15, in meters

F_{15} = Percent finer than No. 200 sieve for granular materials included within T_{15}

$D50_{15}$ = Average mean grain size for granular materials within T_{15} in millimeters

S = Slope gradient of the ground surface

$R^* = R + R_0$

$R_0 = 10^{(0.89M-5.64)}$

For this study:

M = 8.5, R = 28 kilometer from San Andreas Fault, R_0 = 84.14, R^* = 111

T_{15} = 1.67 meter, F_{15} = 98%, $D50_{15}$ = 0.5 millimeter, S = 2%

The lateral movement of the ground surface soil is calculated to be approximately 0.1 meters (0.3 feet or 2.9 inches) with respect to the San Andreas Fault. Based on the insignificant magnitude of the lateral movement, we concluded that the liquefaction-induced lateral spreading is very minimal.

L. CONCLUSIONS

The followings are the conclusions of this study.

- The liquefaction-induced total maximum settlements at the site is 1.0 inches
- The liquefaction-induced differential settlements at the site is 0.75 inch
- The potential of liquefaction-induced ground surface damage at the site is marginally minimal
- The liquefaction-induced lateral spreading is very minimal

INUNDATION POTENTIAL

The subject site is located on the southern corner of S. King Road and Story 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. Based on the laboratory testing results, 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 the building pad be underlain by a minimum of 12 inches non-expansive fill layer or 12 inches of lime-treated native soil material. During the construction of the building pad, any highly expansive native soil should not be used as non-expansive engineered fill material.

3. Due to excessive moisture of the native soil at the depth of 3 feet below existing pavement surface, we recommend the footing excavation for the four-story office building should be underlain by 12 inches of non-expansive fill, Class II Baserock or cement slurry if deemed necessary in the field by our representative from our office.

4. The existing asphalt concrete pavement can be crushed and mixed with the existing baserock and re-used as fill material. The existing concrete buildings can be crushed according to a Class II Baserock specification and re-used on the building pad and parking area rock section. The crushed baserock material for the building pad should be free of crushed asphalt concrete. The baserock material should be inspected and tested prior to final approval and use.

5. The 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.

6. The lime-treated subgrade soil, if any, 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.

7. We recommend the building pad be elevated above the adjacent ground surface to promote proper drainage and diversion of water away from the building foundations.

8. We recommended a reference to our report should be stated in the grading and foundation plans (this includes the Geotechnical Investigation File No. 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 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 and grading shall be observed and inspected by a representative from Silicon Valley Soil Engineering (SVSE). These operations are not limited to testing and inspection during grading.

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 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 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, including grass and weeds 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, existing pavement 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 below existing subgrade elevation and thoroughly cleaned of vegetation and other deleterious matter.

6. After stripping, scarifying and cleaning operations, native soil should be moisture conditioned to 3% over optimum moisture and re-compacted to 87% to 92% relative maximum density using ASTM D1557-12 procedure over the entire building pad and 5 feet beyond the perimeter of the pad.

7. All native engineered fill soil should be placed in uniform horizontal lifts of not more than 6 to 8 inches in un-compacted thickness, and compacted to 87% to 92% relative maximum density using ASTM D1557-12 procedure. The import soil should be compacted to not less than 90%. The baserock, however, 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. Silicon Valley Soil Engineering (SVSE), should be notified at least two days prior to commencement of any grading operations so that our office may coordinate the work in the field with the contractor. 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 and an R-Value greater than 25.

11. 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

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

FOUNDATION DESIGN CRITERIA

13. We recommend the proposed retail building be supported on continuous perimeter foundation and isolated interior spread footings. Recommendations are presented in the following paragraphs.

14. Conventional foundation should be founded at a minimum depth of 24 inches below finished subgrade elevation. Under these conditions, the recommended allowable bearing capacity is 2,800 p.s.f. for both continuous perimeter and isolated and interior spread footings.

15. Because of the high expansion potential of the near surface native soil, we recommend the footing excavation should be saturated with water (not overly saturated) and periodically after footing excavation and prior to concrete placement, if deemed necessary. If the footing bottoms are disturbed, a jumping jack should be used to compact the footing bottoms.

16. 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.

17. 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.

2013 CBC SEISMIC VALUES

18. The site categorization and site coefficients are shown in the following table.

Classification/Coefficient	Design Value
Site Class (Table 20.3-1 CBC 2013)	D
Risk Category	I,II,III
Site Latitude	37.338911° N.
Site Longitude	121.842553° W.
0.2-second Mapped Spectra Acceleration[1], S_S	1.500g*
1-second Mapped Spectra Acceleration[1], S_1	0.600g*
Short-Period Site Coefficient, F_a (Table 11.4-1 CBC 2013)	1.0
Long-Period Site Coefficient, F_V (Table 11.4-2 CBC 2013)	1.5
0.2-second Period, Maximum considered Earthquake Spectral Response Acceleration S_{MS} ($S_{MS} = F_aS_S$ - Equation 11.4-1 CBC 2013)	1.500g*
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration S_{M1} ($S_{M1} = F_VS_1$ - Equation 11.4-2 CBC 2013)	0.900g*
0.2-second Period, Designed Spectra Acceleration, S_{DS} ($S_{DS} = 2/3S_{MS}$ - Equation 11.4-3 CBC 2013)	1.000g*
1-second Period, Designed Spectra Acceleration, S_{D1} ($S_{D1} = 2/3S_{M1}$ - Equation 11.4-4 CBC 2013)	0.600g*

[1] For Site Class B, 5 percent damped.
* USGS Seismic Design Maps for 2013 CBC analysis.

RETAINING WALLS

19. Any facilities that will retain a soil mass such as loading dock walls 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, they shall be designed for the earth pressure resulting from 60 pounds equivalent fluid pressure, to which shall be added surcharge loads.

20. 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.

21. 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.

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

23. Drainage should be provided behind the retaining wall. The drainage system should consist of perforated (subdrain) pipe placed at the base of the retaining wall and surrounded by ¾ inch drain rock wrapped in a filter fabric. The drain rock wrapped in fabric should be at least 12 inches wide and extend from the base of the wall to within 1.5 feet of the ground surface. The upper 1.5 feet of backfill should consist of compacted native soil. The retaining wall drainage system should be sloped to outfall to a discharge facility.

24. As an alternative to the drain rock and fabric, Miradrain 2000 or approved drain mat equivalent may be used behind the retaining wall. The drain mat should extend from the base of the wall to within two feet of 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 be sloped to outfall to an appropriate discharge facility.

25. Any retaining walls associated with the building should be waterproofed such as elevator pit walls and slab bottom.

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

CONCRETE SLAB-ON-GRADE CONSTRUCTION

27. 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 the concrete slab be underlain by a minimum of 12 inches non-expansive fill layer. This layer should be compacted to at least 90% relative maximum density. The non-expansive fill or lime-treated native soil is not included in the rock section.

28. A minimum of 5 inches of ¾ inch crushed rock or Class II Baserock (recycled crushed asphalt concrete is not acceptable) and vapor barrier membrane (STEGO 15 mil) should be placed between the finished grade and the concrete slab. The vapor barrier should be taped at the seams and/or mastic sealed at the protrusions. The native subgrade and/or native engineered fill should be moisture conditioned to 3% over optimum moisture and compacted to 87% to 92% relative maximum density. The Class II Baserock should be compacted to not at least 95%.

29. Use of a vapor barrier membrane under the concrete slab is required if a floor covering would be applied. The membrane should be placed between the rock and the concrete slab. If the slab would not receive a floor covering, the vapor barrier membrane can be eliminated.

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

EXCAVATION

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

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

DRAINAGE

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

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

35. 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.

36. 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. In unpaved areas, it is recommended that protective slopes be stabilized adjoining perimeter building walls. These slopes should be extended to a minimum of 5 feet horizontally from building walls. They must have a minimum outfall of 2 percent.

37. 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.

ABANDONMENT OF THE EXISTING UTILITY LINES

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

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

40. 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

41. 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 in accordance with ASTM D1557-12 procedure. Backfill should be placed in 6 to 8 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.

42. 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.

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

PAVEMENT DESIGN

44. 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 pavement section designs, which satisfy the State of California Standard Design Criteria, and above traffic indices, are presented in Table II. Rigid and paver pavement section designs are presented in Table III and IV. Because of the high expansion potential of the surface native soil at the site, we provided alternative pavement section

(asphalt and baserock) recommendations for the parking area to be underlain by a minimum of 12 inches of non-expansive fill or lime-treated native material. The non-expansive fill soil and lime-treated native material should be compacted to at least 90% relative maximum density. These alternate pavement sections are presented in Table IIA, IIB and III. Due to the high expansion potential of the surface native soil, minor cracks in the pavement should be expected.

LIME TREATMENT ALTERNATIVES

47. 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 and driveway areas. The lime treatment process should extend a minimum of 3 feet beyond the building pad, curb and gutter, and/or any other improvements. 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. In the parking area, if lime treatment would be implemented, the baserock section could be reduced as shown in Table IIB.

48. The lime-treated subgrade soil should not be exposed to the element for an extended period. If no improvements are planed 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

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.

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 (Revised 01/17/2006)*. Department Of Conservation. Division of Mines and Geology],

2013 (CBC) California Building Code, Title 24, Part 2.

TABLES

TABLE I

SUMMARY OF MOISTURE/DENSITY, DIRECT SHEAR, PLASTICITY INDEX, & LIQUID LIMIT TESTS

Sample No.	Depth Ft.	In-Place Conditions		Direct Shear Testing			
		Moisture Content % Dry Wt.	Dry Density p.c.f.	Unit Cohesion k.s.f.	Angle of Internal Friction Degrees	Liquid Limit L.L.	Plasticity Index P.I.
1-1	3	35.8	81.2	0.8	11		
1-2	5	16.2	119.3			44	21
1-3	10	21.3	98.5			37	17
1-4	15	27.0	98.1				<12
1-5	20	24.6	104.6			47	23
1-6	25	26.5	99.4			48	22
1-7	30	19.7	105.8				<12
1-8	35	25.5	87.0			44	19
1-9	40	21.6	105.9			45	23
1-10	45	27.1	95.1			43	20
1-11	50	29.9	96.2			45	22
2-1	3	24.5	85.0				
2-2	5	17.5	79.3				
2-3	10	21.5	109.8				

TABLE II

PROPOSED ALTERNATE PAVEMENT SECTIONS

Location: Proposed 3-Story Retail and Office Building
Southern Corner of
S. King Road and Story Road
San Jose, California

	PARKING STALLS			DRIVEWAY		
Design R-Value	6.0			6.0		
Traffic Index	4.5			5.5		
Gravel Equivalent	17.0			20.0		
Recommended Alternate Pavement Sections:	1A	1B	1C	2A	2B	2C
Asphalt Concrete	3.0"	3.5"	4.0"	3.0"	3.5"	4.0"
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	9.0"	8.0"	7.0"	11.0"	10.0"	9.0"
Native soil scarified & compacted to at least 87% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"

TABLE IIA

PROPOSED NON-EXPANSIVE PAVEMENT SECTIONS

Location: Proposed 3-Story Retail and Office Building
Southern Corner of
S. King Road and Story Road
San Jose, California

	PARKING STALLS			DRIVEWAY		
Design R-Value	24.0			24.0		
Traffic Index	4.5			5.5		
Gravel Equivalent	14.0			16.0		
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	6.0"	5.0"	4.0"	9.0"	8.0"	7.0"
Non-expansive soil fill material compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"
Native soil scarified & compacted to at least 87% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"

TABLE IIB

PROPOSED LIME TREATMENT PAVEMENT SECTIONS

Location: Proposed 3-Story Retail and Office Building
Southern Corner of
S. King Road and Story Road
San Jose, California

	PARKING STALLS	DRIVEWAY		
Design R-Value	24.0	24.0		
Traffic Index	4.5	5.5		
Gravel Equivalent	14.0	16.0		
Recommended Alternate Pavement Sections:	1	2A	2B	2C
Asphalt Concrete	3.0"	3.0"	3.5"	4.0"
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	4.0"	7.0"	6.0"	5.0"
Lime-treated native soil material compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"

TABLE III

PROPOSED RIGID PAVEMENT SECTIONS

Location: Proposed 3-Story Retail and Office Building
Southern Corner of
S. King Road and Story Road
San Jose, California

	DRIVEWAY*			CURB & GUTTER			SIDEWALK		
Recommended Rigid Pavement Sections:	1A	1B	1C	2A	2B	2C	3A	3B	3C
P.C. Concrete*	6.0”	6.0”	6.0”	6.0”	6.0”	6.0”	4.0”	4.0”	4.0”
Class II Baserock (R=78 min.) compacted to at least 95% relative max. density	12.0”	6.0”	6.0”	8.0”	6.0”	6.0”	6.0”	4.0”	4.0”
Non-expansive soil fill material compacted to at least 90% relative max. density	---	12.0”	---	---	8.0”	---	---	8.0”	---
Lime-treated native soil material compacted to at least 90% relative max. density	---	---	12.0”	---	---	12.0”	---	---	12.0”
Native soil subgrade scarified & compacted to at least 87% relative max. density	12.0”	12.0”	---	12.0”	12.0”	---	12.0”	12.0”	---

* Including trash enclosures, stress pads, and valley gutters. Reinforcement 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 3-Story Retail and Office Building
Southern Corner of
S. King Road and Story Road
San Jose, 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	8.0"	12.0"	---	---
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	---	---	10.0"	14.0"
Non-expansive soil fill material compacted to at least 90% relative max. density, if any	---	---	---	---
Native soil scarified & compacted to at least 87% relative max. density	12.0"	12.0"	12.0"	12.0"

* (see next page)

* The subgrade should be lined with a geotextile membrane Mirafi 500X or equivalent. The liner should be place and overlapped properly for drainage. The subgrade should be sloped at a minimum of 2% towards 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 outfall to a discharge facility. The Mirafi 500X membrane should not be placed over the subdrain system.

The pavers should be bordered with a concrete curb/band. Typically, minor maintenance would be required during the life of the pavers.

TABLE V
LIQUEFACTION ANALYSIS

Boring	Liquefiable Soil Layer		N	N_{60}	$(N_1)_{60}$	σ_{ov} (k.s.f)	U (k.s.f.)	σ'_{ov} (k.s.f)	CSR	CRR	F.S.	Liquefaction -induced Settlements (Inch)
	Depth (Feet)	Thickness (Feet)										
B-1	13-18	5	16	14	17	2.19	0.62	1.57	0.54	0.19	0.35	1.0

NOTES:

N = measured standard penetration test N value

N_{60} = standard penetration test N value corrected for field testing procedures

$(N_1)_{60}$ = standard penetration test N value corrected for both field testing procedures and overburden pressure

σ_{ov} = total vertical stress in kip per square foot (k.s.f.) at a particular depth where the liquefaction analysis is performed

u = pore pressure in kip per square foot (k.s.f.) at a particular depth where the liquefaction analysis is performed

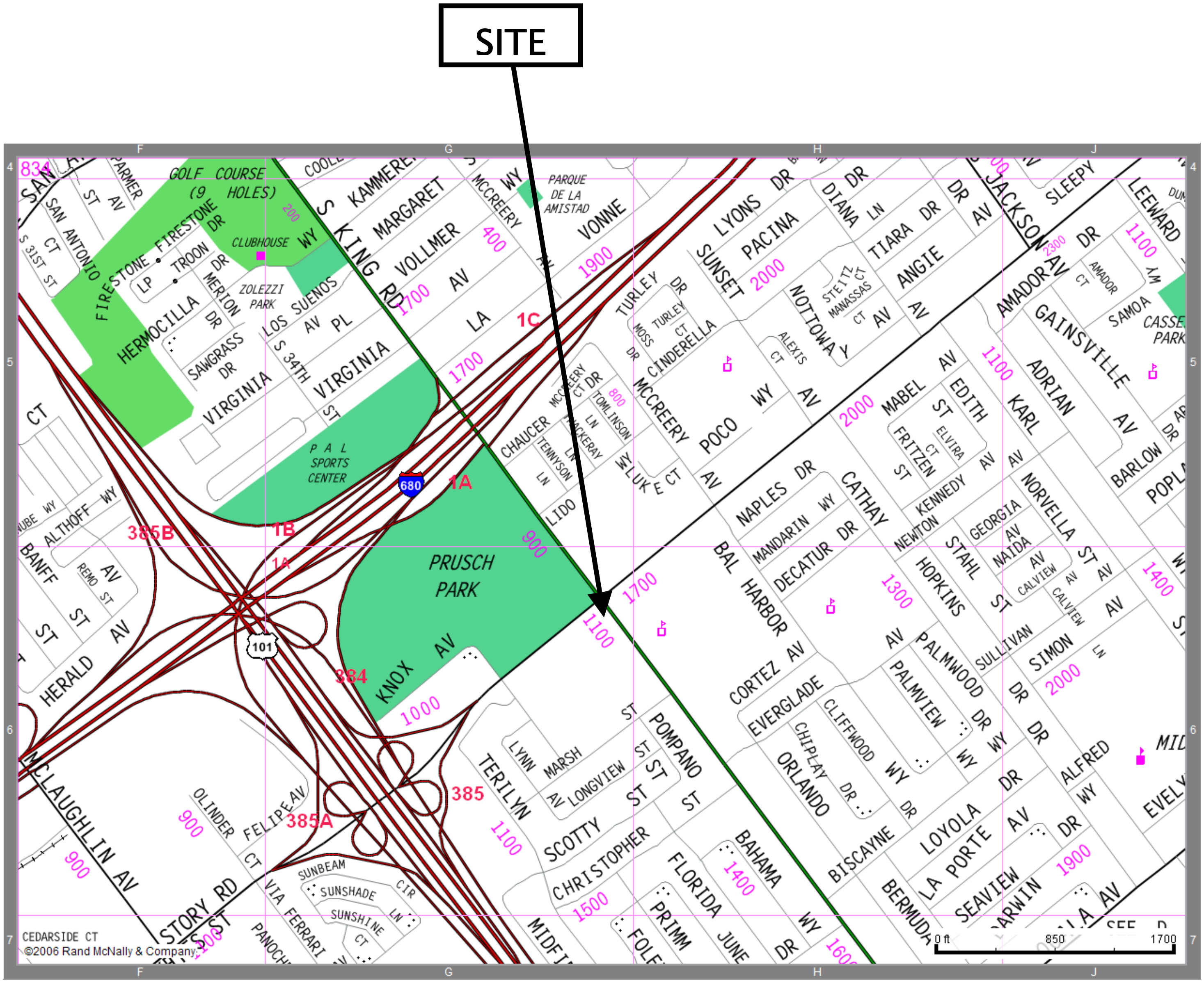
σ'_{ov} = vertical effective stress in kip per square foot (k.s.f.); $\sigma'_{ov} = \sigma_{ov} - u$, also called the effective overburden pressure

CSR = cyclic stress ratio, also called seismic stress ratio

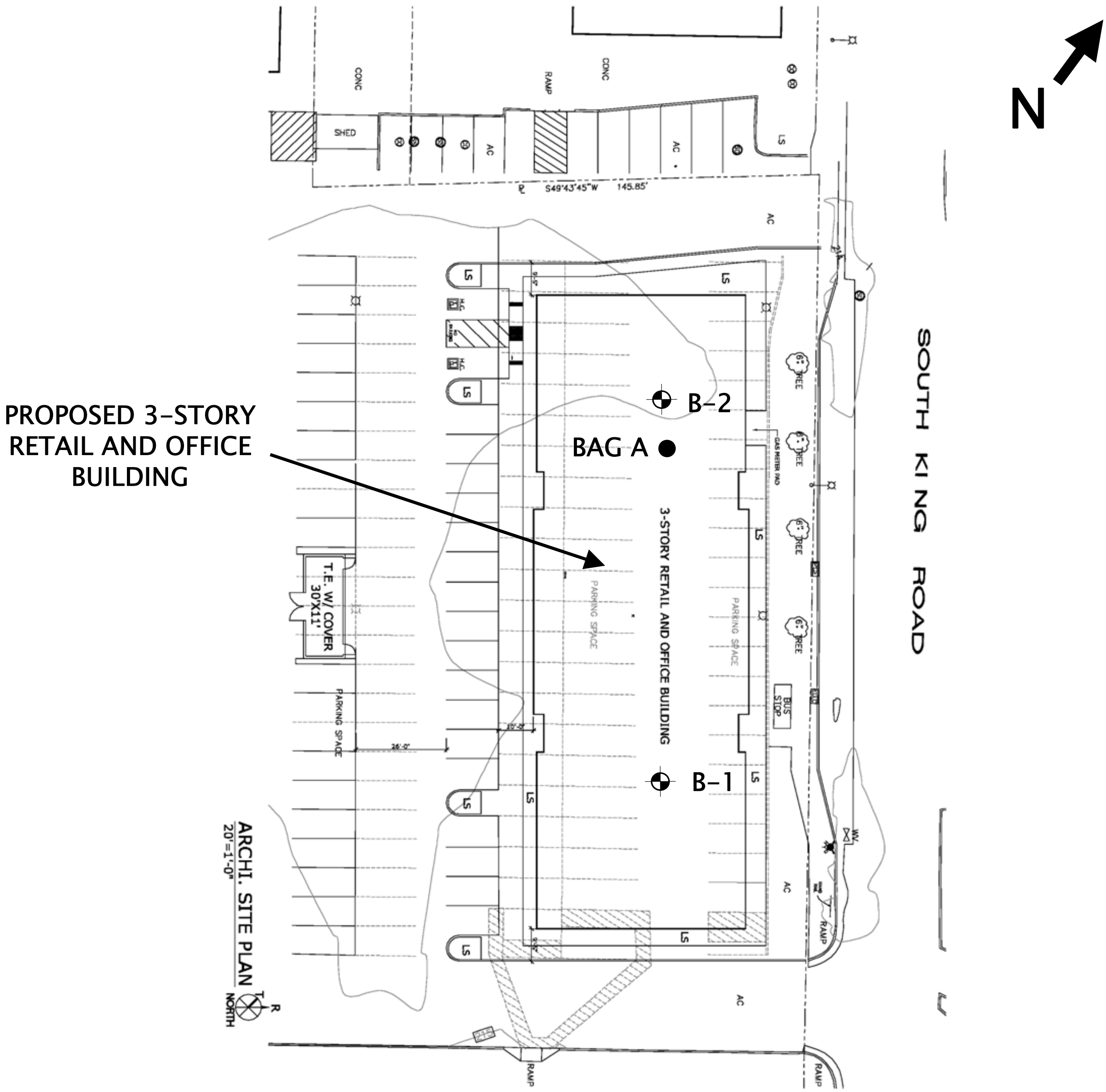
CRR = cyclic resistance ratio obtained from Figure 7

F.S. = CRR/ CSR = Factor of Safety against liquefaction

FIGURES



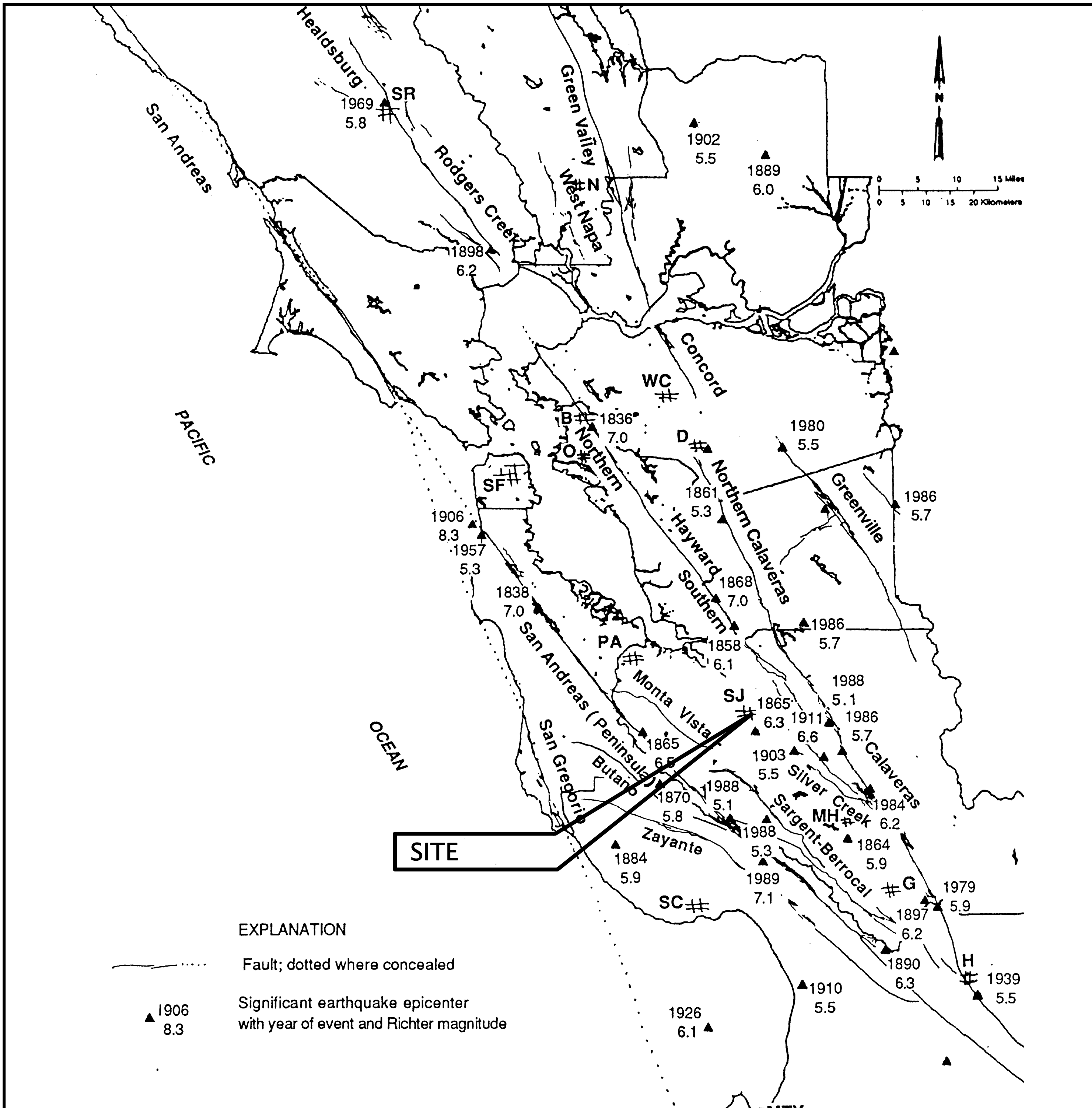
Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**VICINITY MAP** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No.: SV1267	FIGURE
		Drawn by: V.V.	1
		Scale: NOT TO SCALE	June 2014



⊕ DENOTES APPROXIMATE CURRENT EXPLORATORY BORING LOCATION (BY SVSE)

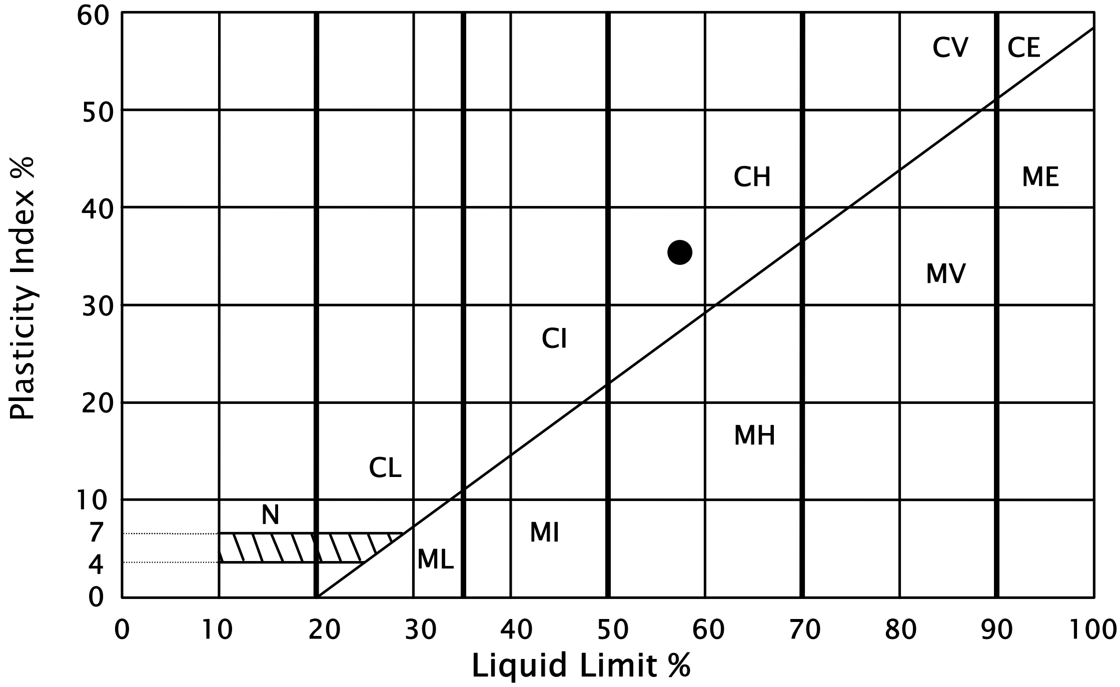
● DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**SITE PLAN** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No.: SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 2 June 2014



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**FAULT LOCATION MAP** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No.: SV1267	FIGURE
		Drawn by: V.V.	3
		Scale: NOT TO SCALE	June 2014

PLASTICITY CHART

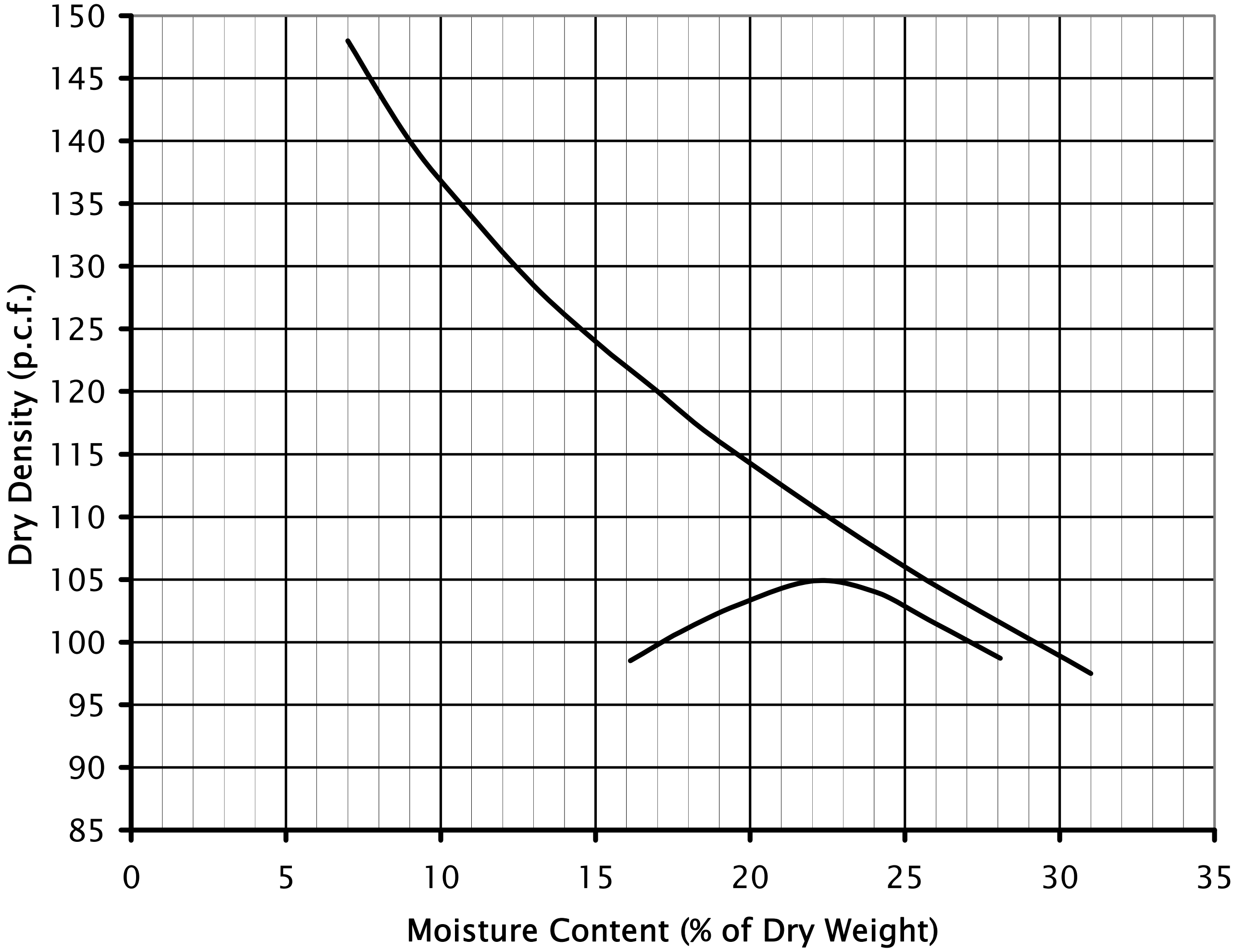


PLASTICITY DATA

Key Symbol	Hole No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	35	57	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 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No.: SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 4 June 2014



SAMPLE: A

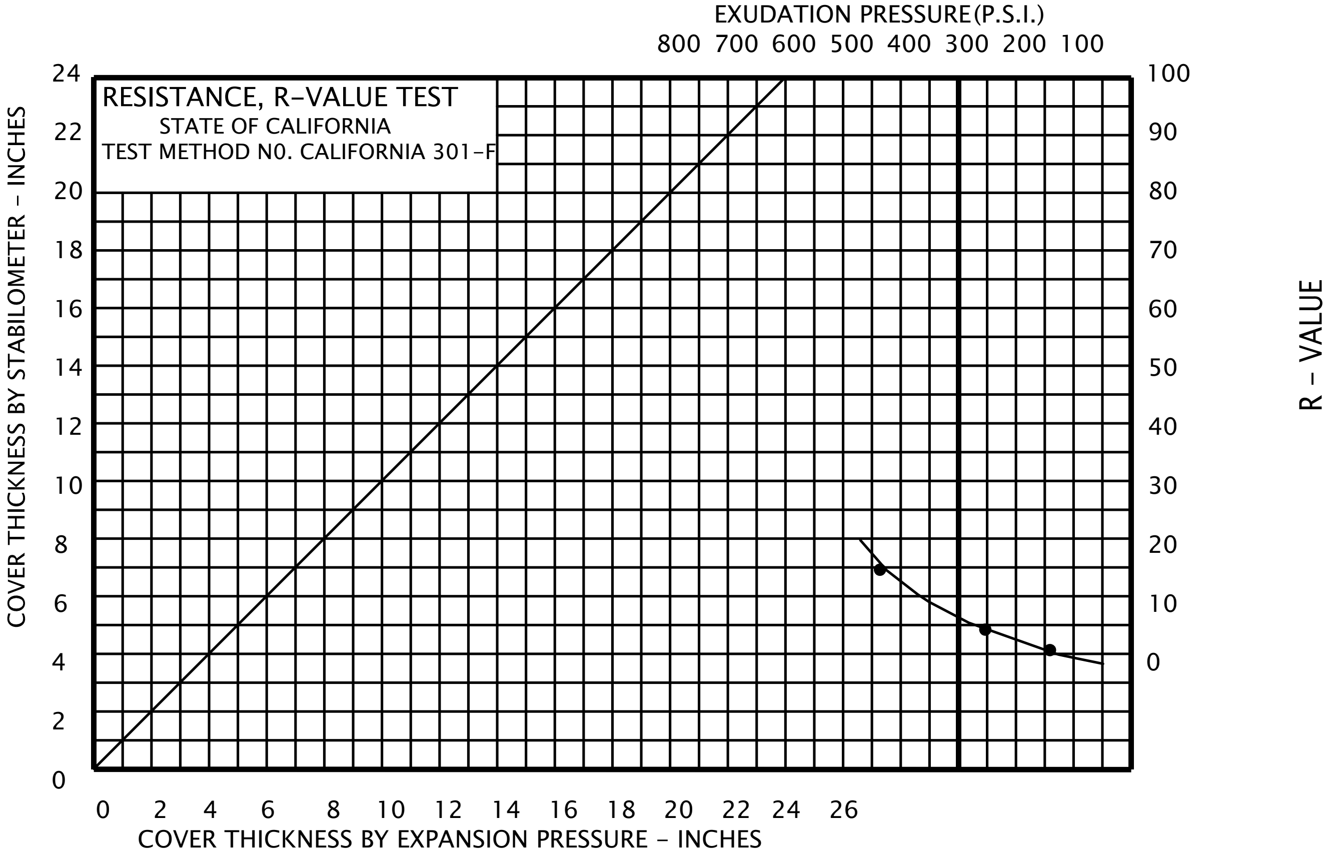
DESCRIPTION: Black Silty CLAY

LABORATORY TEST PROCEDURE: ASTM D1557-12

MAXIMUM DRY DENSITY: 105.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 22.0 %

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**COMPACTION TEST A** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 5 June 2014



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 95054 (408) 988-2990	**R-VALUE TEST** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 6 June 2014

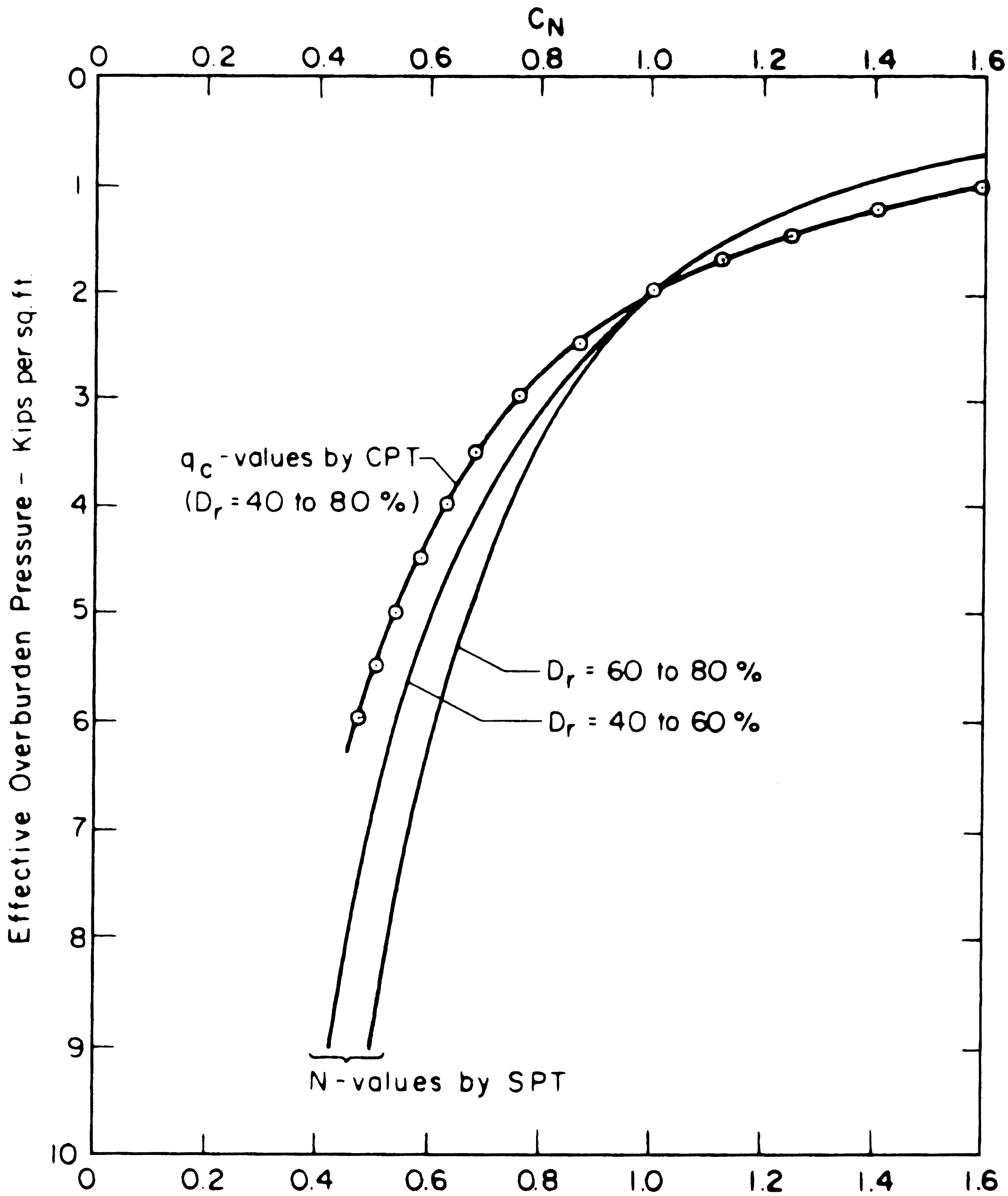


FIGURE 5.11 Correction factor C_N used to adjust the standard penetration test N value and cone penetration test q_c value for the effective overburden pressure. The symbol D_r refers to the relative density of the sand. (*Reproduced from Seed et al. 1983, with permission from the American Society of Civil Engineers.*)

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**CORRECTION FACTOR C_N FOR EFFECTIVE OVERBURDEN PRESSURE** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267	FIGURE
		Drawn by: V.V.	7
		Scale: NOT TO SCALE	June 2014

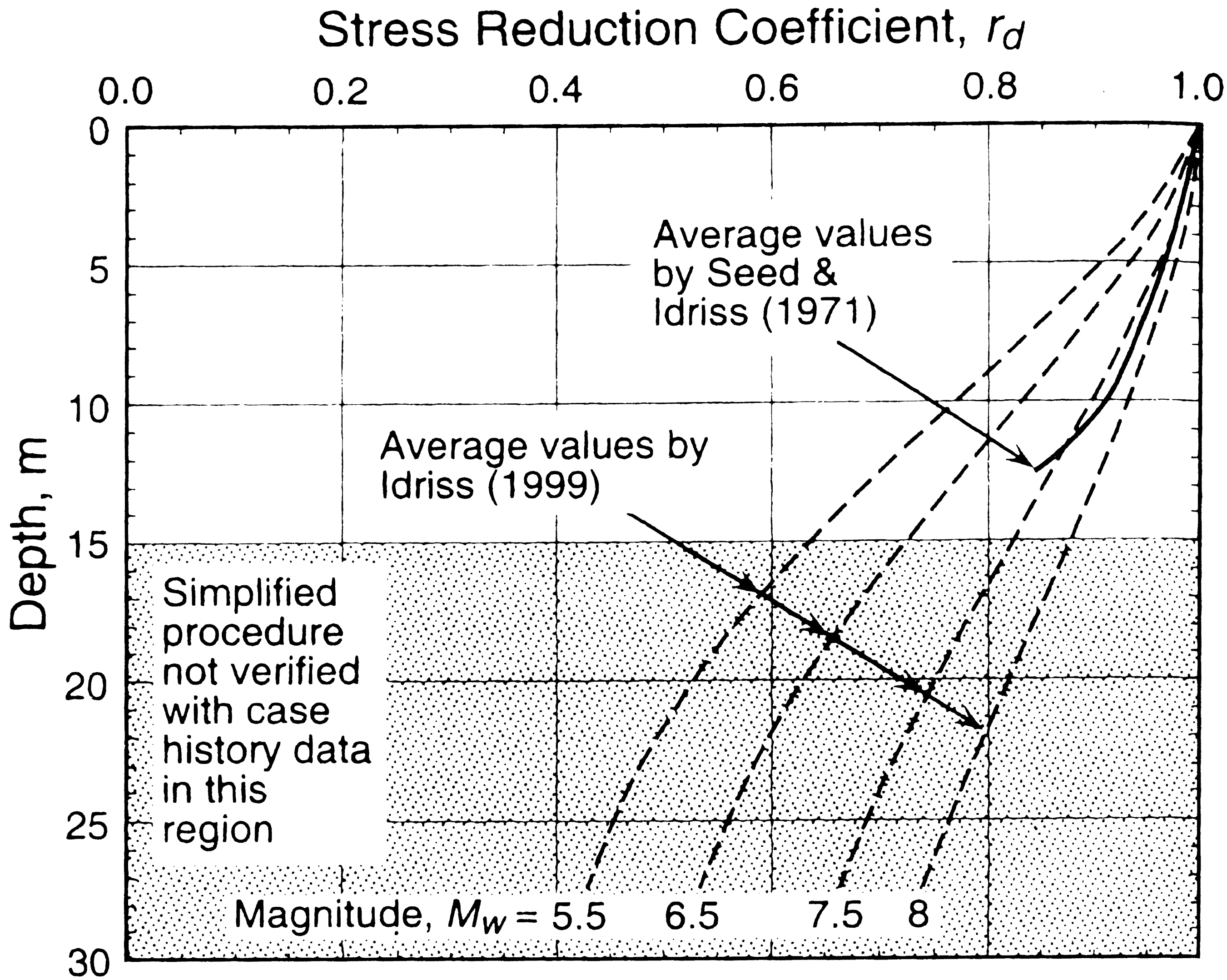


FIGURE 6.5 Reduction factor r_d versus depth below level or gently sloping ground surfaces. (*From Andrus and Stokoe 2000, reproduced with permission from the American Society of Civil Engineers.*)

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**REDUCTION FACTOR *(R_D)*** Proposed Retail Building Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267	FIGURE
		Drawn by: V.V.	8
		Scale: NOT TO SCALE	June 2014

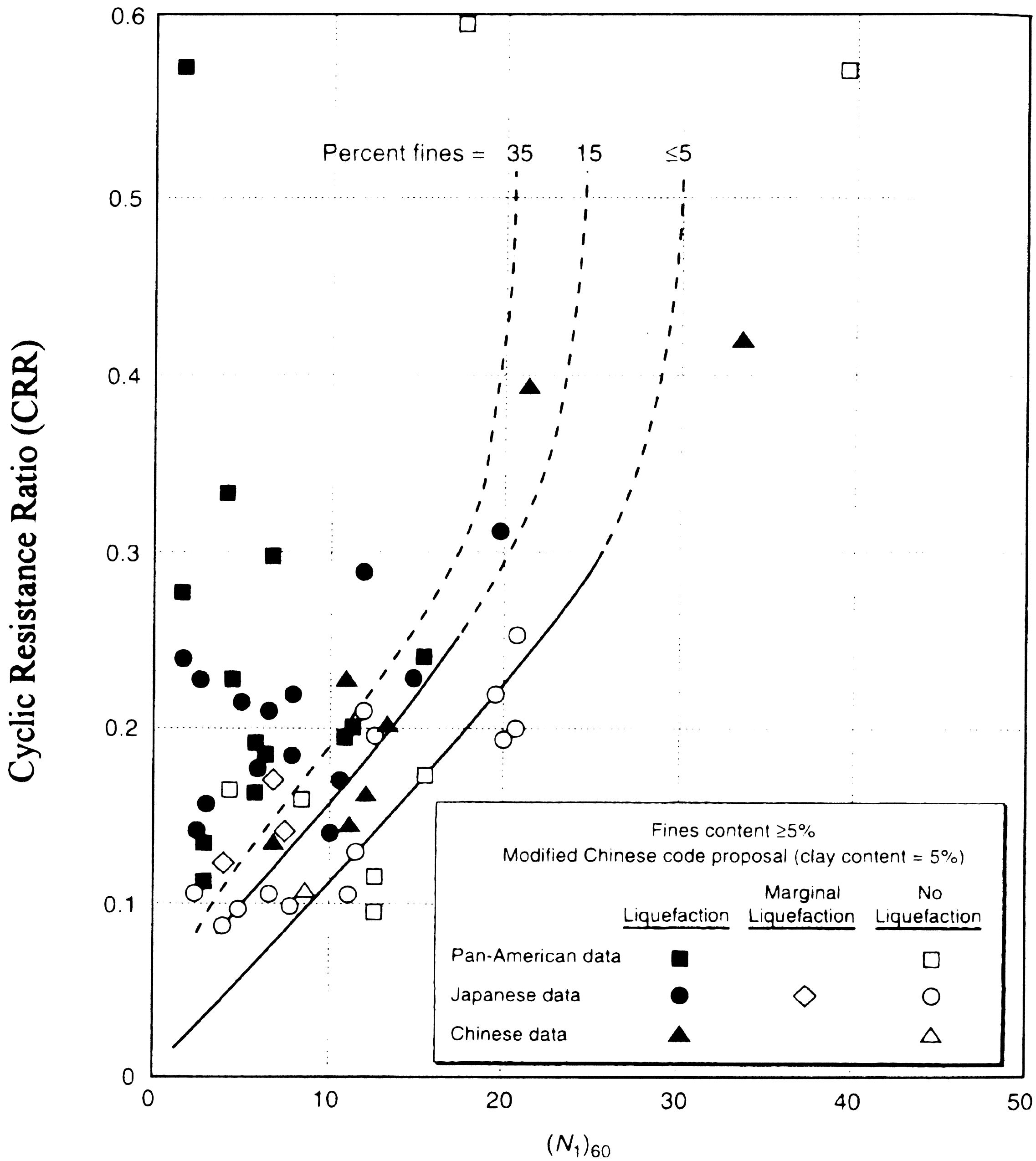


FIGURE 6.6 Plot used to determine the cyclic resistance ratio for clean and silty sands for $M = 7.5$ earthquakes. (*After Seed et al. 1985, reprinted with permission of the American Society of Civil Engineers.*)

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**CYCLIC RESISTANCE RATIO (CRR)** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 9 June 2014

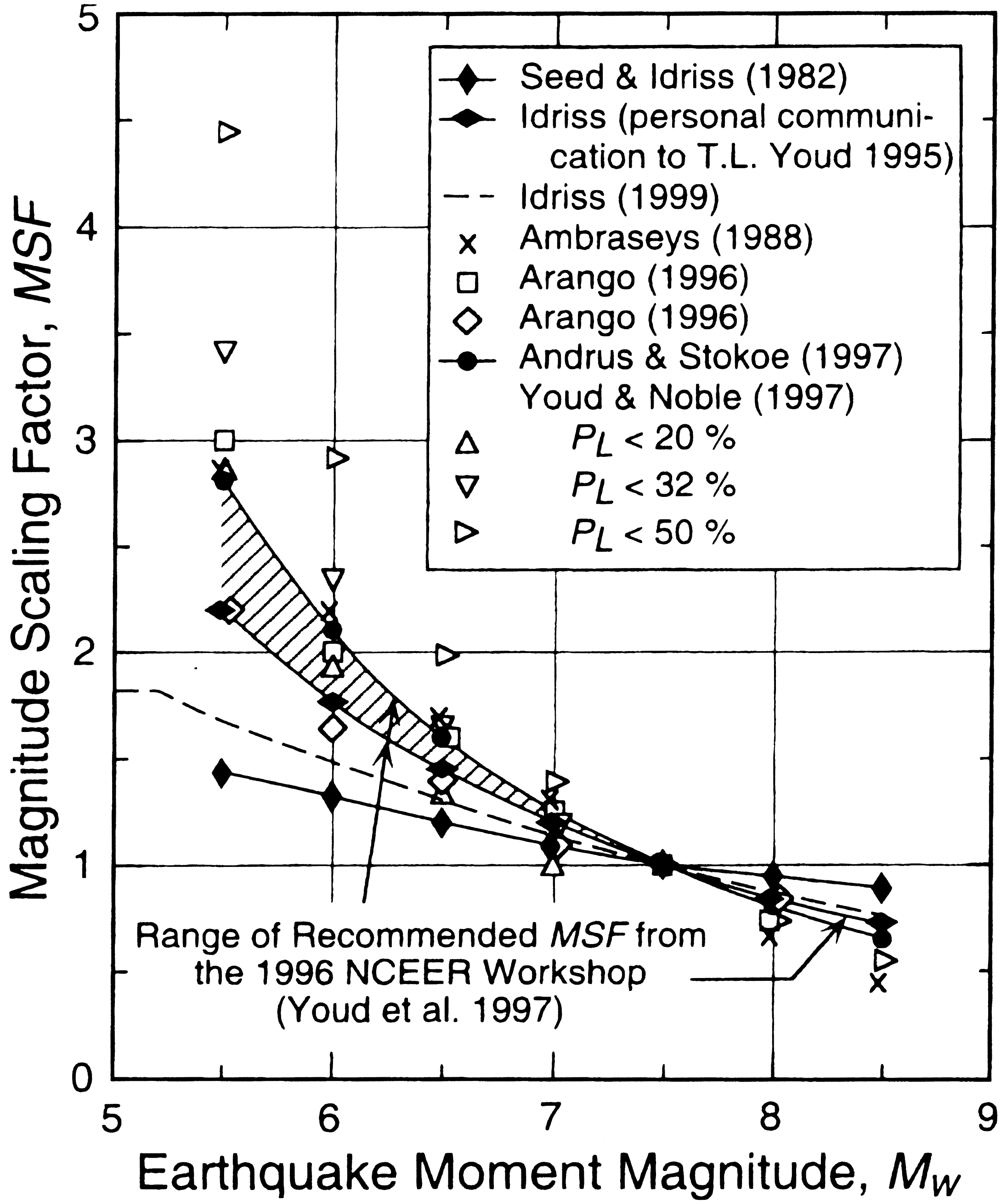


FIGURE 6.7 Magnitude scaling factors derived by various investigators. (*From Andrus and Stokoe 2000, reprinted with permission of the American Society of Civil Engineers.*)

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**MAGNITUDE SCALING FACTORS** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 10 June 2014

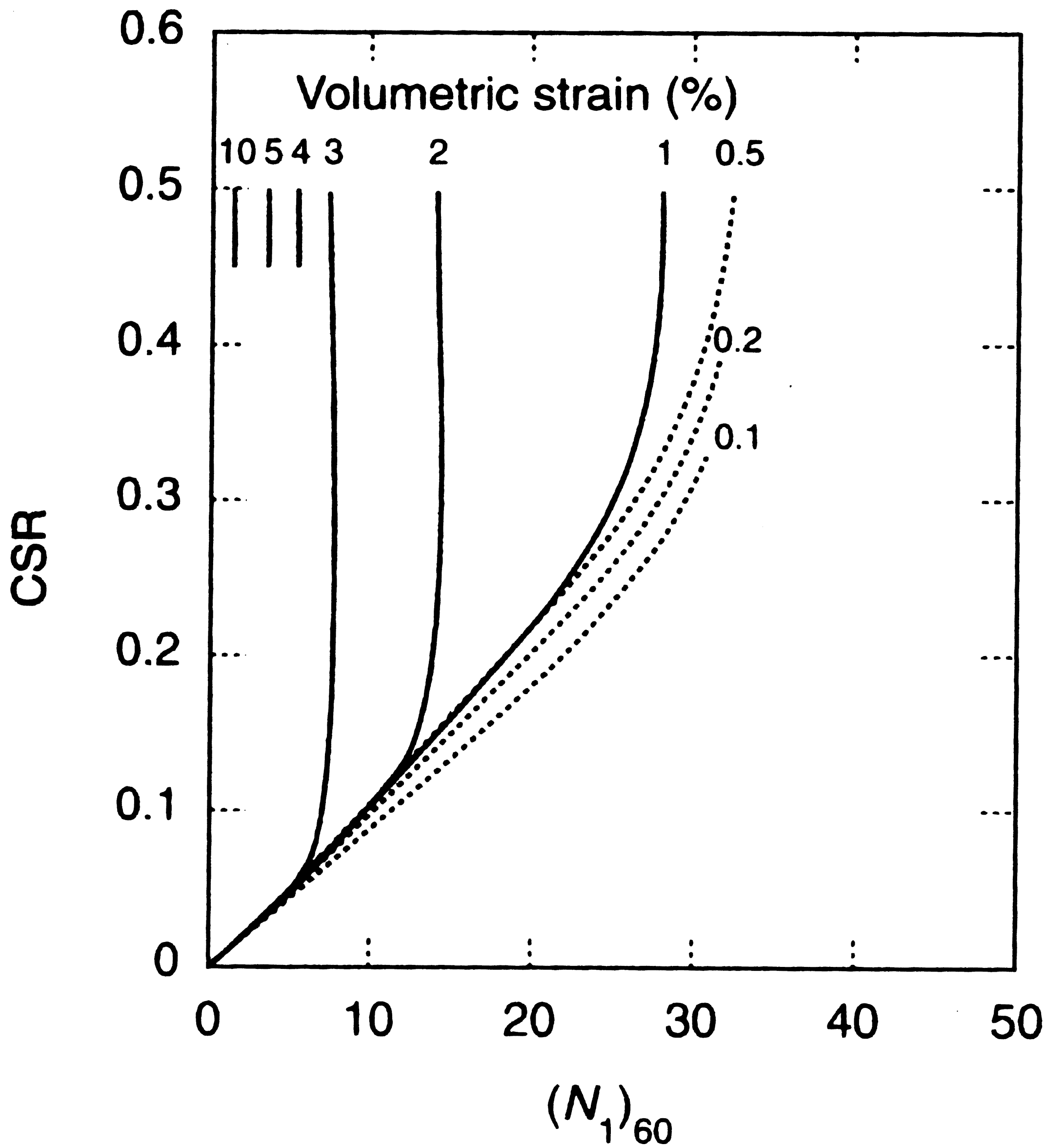
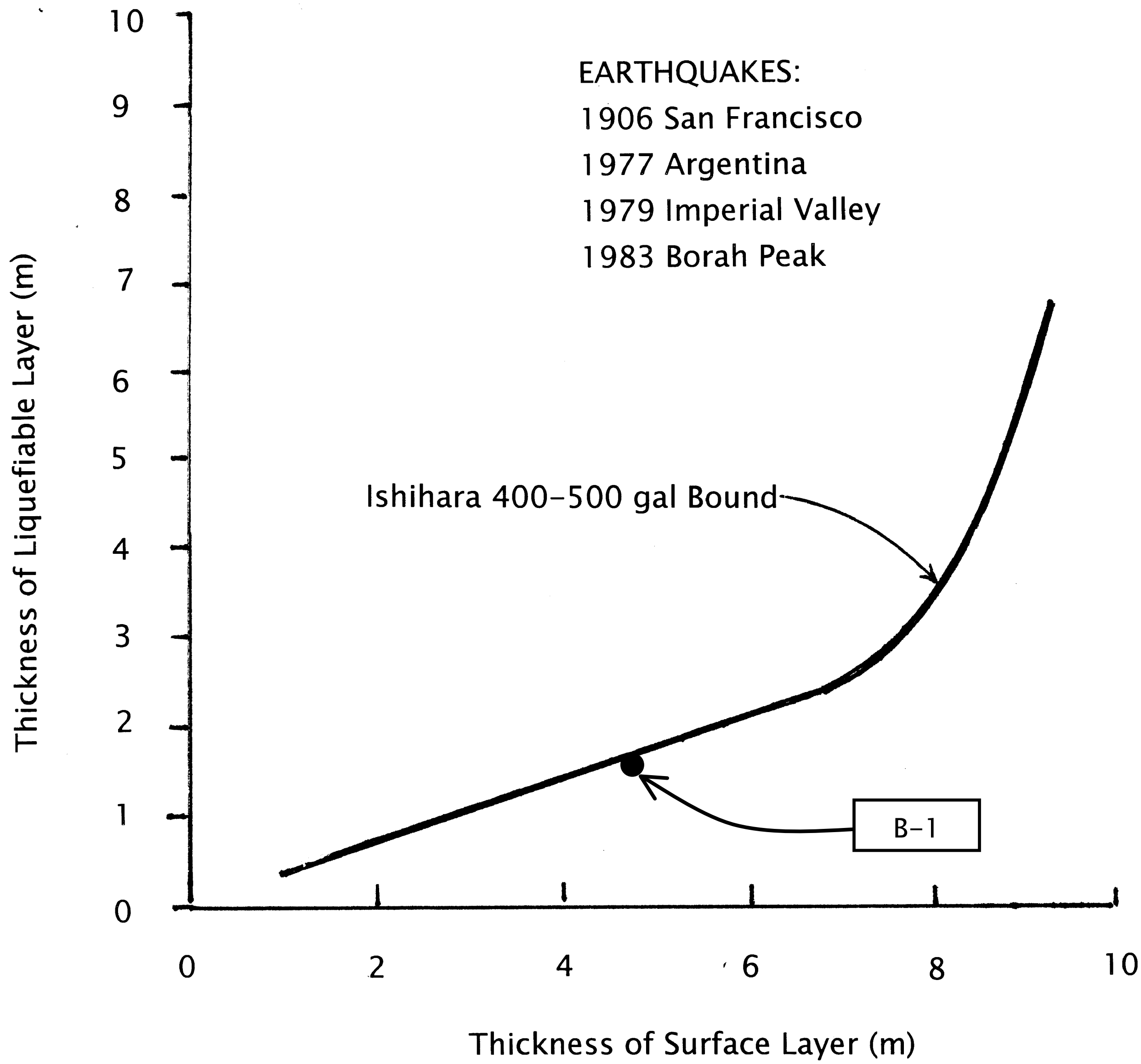


FIGURE 7.2 Chart for estimating the ground surface settlement of clean sand for factor of safety against liquefaction less than or equal to 1.0 (solid lines) and greater than 1.0 (dashed lines). To use this figure, the cyclic stress ratio from Eq. (6.6) and the $(N_1)_{60}$ value from Eq. (5.2) must be determined. (*Reproduced from Kramer 1996, originally developed by Tokimatsu and Seed 1984.*)

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	LIQUEFACTION-INDUCED GROUND SURFACE SETTLEMENT Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 11 June 2014

Magnitude Range: 5.9 to 8.0 M_w
Acceleration Range: 0.56 to 0.78g



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**LIQUEFACTION-INDUCED GROUND DAMAGE** Proposed 3-Story Retail and Office Building Southern Corner of S. King Street & Story Road San Jose, California	File No. SV1267	FIGURE
		Drawn by: V.V.	12
		Scale: NOT TO SCALE	June 2014

APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

KEY TO LOG OF BORING

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

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

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

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

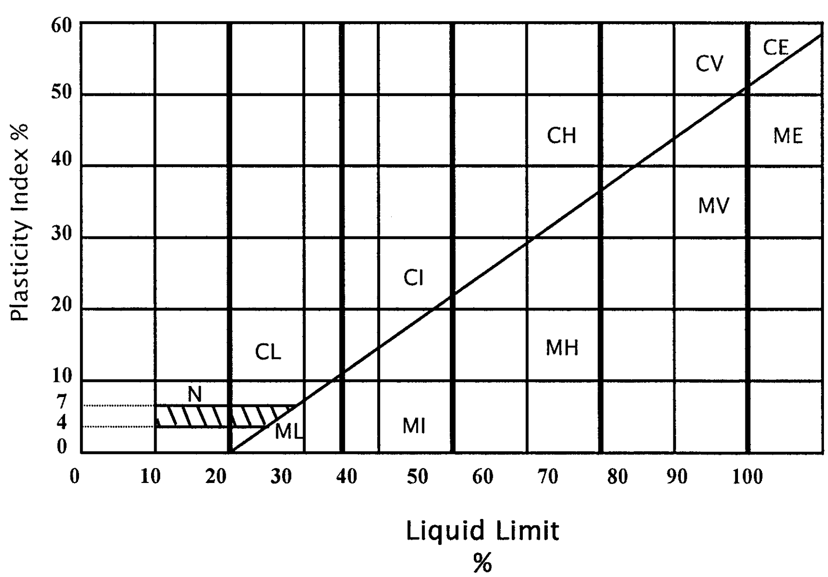
METHOD OF SOIL CLASSIFICATION CHART

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	RANGE OF GRAIN SIZES: 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
Coarse	3" to 3/4"	76.2 to 19.1
Fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
Coarse	No. 4 to No. 10	4.76 to 2.00
Medium	No.10 to No. 40	2.00 to 0.420
Fine	No.40 to No. 200	0.420 to 0.074
SILT AND CLAY	Below No. 200	Below 0.074

PLASTICITY INDEX CHART



Project: Proposed 3-Story Retail and Office Building **Project Location:** Southwest Coner of King Road & Story Road - San Jose, California **Project Number:** SV1267	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Key to Log of Boring **Sheet 1 of 1**

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

COLUMN DESCRIPTIONS

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

FIELD AND LABORATORY TEST ABBREVIATIONS

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

MATERIAL GRAPHIC SYMBOLS

Asphaltic Concrete (AC)
Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)
Aggregate Base (AB)
Clayey SAND to Sandy CLAY (SC-CL)

TYPICAL SAMPLER GRAPHIC SYMBOLS

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

OTHER GRAPHIC SYMBOLS

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

GENERAL NOTES

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

Project: Proposed 3-Story Retail and Office Building **Project Location:** Southwest Coner of King Road & Story Road - San Jose, California **Project Number:** SV1267	**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 **05/28/14**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **51.5 feet**
		Approximate Surface Elevation **100 feet**
Groundwater Level and Date Measured **13 feet (05/28/14)**	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0 – 0.25				Asphalt		3.0 inches of Asphalt Concrete (AC)						
0.25 – 0.75						6.0 inches of Aggregate Base (AB)						
0.75				CH		Black Silty CLAY Moist, very stiff						
		1-1	29				35.8	81.2	0.8	11		
						Color changed to brown						
5		1-2	29				16.2	119.3			44	21
8				SC-CL		Olive Brown Sandy CLAY/Clayey SAND Moist, very stiff						
10		1-3	14				21.3	98.5			37	17
13				SP		Stabilized after drilling completion Olive Brown Silty Clayey SAND Moist, medium dense SAND: Medium grained, poorly graded First encountered						
15		1-4	16				27.0	98.1				<12
18				CL		Bluish Gray Silty CLAY Moist, very stiff						
20		1-5	36				24.6	104.6			47	23
						Drilling became very stiff						
25		1-6	27				26.5	99.4			48	22
30												

Project: Proposed 3-Story Retail and Office Building **Project Location:** Southwest Coner of King Road & Story Road - San Jose, California **Project Number:** SV1267	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-1 **Sheet 2 of 2**

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
30		1-7	40	SP		Olive Brown Silty SAND Moist, dense SAND: Medium grained, poorly graded	19.7	105.8				<12
35		1-8	22	CL		Greenish Gray Silty CLAY Moist, very stiff	25.5	87.0			44	19
40		1-9	15				21.6	105.9			45	23
45		1-10	20				27.1	95.1			43	20
50		1-11	18				29.9	96.2			45	22
51.5						Boring terminated at 51.5 feet						
55												
60												
65												

Project: Proposed 3-Story Retail and Office Building **Project Location:** Southwest Coner of King Road & Story Road - San Jose, California **Project Number:** SV1267	**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 **05/28/14**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **11.5 feet**
		Approximate Surface Elevation **100 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												
0.25				Asphalt		3.0 inches of Asphalt Concrete (AC)						
0.75				CH		6.0 inches of Aggregate Base (AB) Black Silty CLAY Moist, very stiff						
		2-1	27				24.5	85.0				
5						Color changed to brown						
		2-2	30				17.5	79.3				
8				SC-CL		Olive Brown Sandy CLAY/Clayey SAND Moist, very stiff						
10												
		2-3	15				21.5	109.8				
11.5						Boring terminated at 11.5 feet						
15												
20												
25												
30												