Appendix D: Geotechnical Analysis

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REPORT TO
FREMONT MISSSION HILLS, LLC
FREMONT, CALIFORNIA

FOR

PROPOSED RESIDENTIAL DEVELOPMENT
10 EAST LAS PALMAS AVENUE
FREMONT, CALIFORNIA

GEOTECHNICAL INVESTIGATION AND PAVEMENT DESIGN

JULY 2012

PREPARED BY

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GEOTECHNICAL CONSULTANTS

File No. SV1059
July 2, 2012

Fremont Mission Hills, LLC
43951 Boscell Road
Fremont, CA 94538

Attention: Ms. Sheena Chang - Managing Member

Subject: Proposed Residential Development
The Club at Mission Hills
10 East Las Palmas Avenue
Fremont, California
**GEOTECHNICAL INVESTIGATION
AND PAVEMENT DESIGN**

Dear Ms. Chang:

We are pleased to transmit herein the results of our geotechnical investigation and pavement design for the proposed residential development. The subject site is the Club at Mission Hills located at 10 East Las Palmas Avenue in Fremont, 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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SV1059.GIPD/Copies: 4 to Fremont Mission Hills, LLC

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INTRODUCTION

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

SITE LOCATION AND DESCRIPTION

The subject site is located at 10 East Las Palmas Avenue in Fremont, California (Figure 1). Fremont East Las Palmas Avenue bound the subject site to the south and southeast, Almeria Avenue to the southwest and northwest, and Canyon Heights to the northeast. At the time of our investigation, the subject site is an irregular shaped, slightly sloped at the southwestern portion and moderately sloped at the northeastern portion with numerous low-lying valleys and high small hilltops. Mission Hills Tennis Club occupied the major portion of the site. Mission Hills Tennis Club is a private tennis and swimming club facilities consist of 13 hard surface tennis courts, one swimming pool, one-story office club building and a restroom building. Based on the available information for the subject site, the development will include the demolition of the six existing tennis courts located in the central portion of site and the construction of sixteen detached single-family residences at the central and northeastern portions of the site with associated improvements. The existing clubhouse, office building, swimming pool, restroom facility building, and six tennis courts will remain. The approximate location of the proposed structures and our borings is 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 six exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on June 27, 2012. The approximate location of the borings is shown on the Site Plan (Figure 2). The borings were drilled to the depths of 10 feet to 50 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 (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 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 D1557-91 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-2 (located in the low lying area - 50 feet deep boring), the surface soil consisted of 3 inches of organic. Below the organic material layer to the depth of 10 feet, a black, moist, firm silty clay layer was encountered. Color changes of dark brown and light brown were noted at the depths of 5 and 7 feet. It appears, this layer was fill soil material from previous grading or backfill. From the depths of 10 feet to 12 feet, the soil became tan brown, moist, medium dense silty sand. The sand was fined grained and poorly graded. From the depths of 12 feet to 16 feet, a medium brown, moist very stiff silt layer was encountered. From the depths of 16 feet to 30 feet, the soil became reddish brown, moist, very stiff to hard silty clay. From the depths of 30 feet to the end of the boring at 50 feet, a dark reddish brown, moist, hard sandy clay layer was encountered. Drilling became hard at the depth of 40 feet. In Boring B-1 (located in the southwestern portion of the site - 15 feet deep), the existing pavement surface consisted of 1.5 inches of asphalt concrete over 8 inches of aggregate base. Below the existing pavement to the end of the boring at 15 feet, a dark brown, damp, hard silt layer was encountered. Color changes of brown and light brown were noted at the depths of 5 feet and 11 feet. There are some small gravels encountered at the depth of 6 feet and an increase in gravel content at the depth of 10 feet. In Boring B-4 (located at the northeastern portion of the site - 10 feet deep), from the surface to the end of the boring at 10 feet, a black, damp, hard silty clay layer was encountered. A color change of brown was noted at the depth of 4 feet. There is an increase in gravel content at the depth of 6 feet. Similar soil profiles were encountered in other borings.

Groundwater was initially encountered in Boring B-2 at the depth of 30 feet and rose to static level of 27 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.

LIQUEFACTION ANALYSIS

A. GROUNDWATER

Groundwater was initially encountered in Boring B-2 at the depth of 30 feet and rose to a static level of 27 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 098 [*Seismic Hazard Evaluation of the Niles 7.5-Minute Quadrangle, Alameda County, California.* Open-File Report 2004. Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 25 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-2 to the depth of 50 feet consists of firm silty clay to medium dense silty sand to very stiff silt to very stiff silty clay to hard sandy clay. The following is the determination of the liquefiable soil for each soil layer in Boring B-2.

1. The firm silty clay layer from the surface to the depth of 10 feet <u>is not liquefiable</u> soil because it is above the groundwater table.

2. The medium dense silty sand layer from the depths of 10 feet to 12 feet <u>is not liquefiable</u> soil because it is above the groundwater table.

3. The very stiff silt layer from the depths of 12 feet to 16 feet <u>is not liquefiable</u> soil because it is above the groundwater table.

4. The very stiff clayey silt clay layer from the depths of 16 feet to 30 feet <u>is not liquefiable</u> soil because based on the Plasticity Index (PI) and moisture contents (MC):

   a. Sample No. 2-5 (20 feet) - [PI > 12; PI = 23 and MC = 24.2% < 85% LL; LL = 44]

   b. Sample No. 2-7 (30 feet) - [PI > 12; PI = 25 and MC = 20.1% < 85% LL; LL = 45]

5. The hard sandy clay layer from the depths of 30 feet to the end of the boring at 50 feet <u>is not liquefiable</u> soil based on the Plasticity Index (PI) and moisture contents (MC):

   a. Sample No. 2-8 (35 feet) - [PI > 12; PI = 20 and MC = 19.9% < 85% LL; LL = 41]

   b. Sample No. 2-10 (45 feet) - [PI > 12; PI = 21 and MC = 21.5% < 85% LL; LL = 43]

In summary, the subsurface soil to the depth of 50 feet at the subject site is not liquefiable soil.

C. CONCLUSION

Because no suspected liquefiable soil layer was identified at the subject site, we concluded that the potential of liquefaction at the subject site is very minimal.

D. DRY SETTLEMENT ANALYSIS

We estimated the earthquake-induced total settlements of the dry silty sand layer encountered from the depths of 10 feet to 12 feet (2 feet thick) using charts (Figures 7 and 8) developed by Tokimatsu and Seed in 1987 (Day, R.W. (2002). *Geotechnical Earthquake Engineering Handbook*, McGraw-Hill, pp. 7.13). By entering the $(N_1)_{60}$ value of 20 for the silty sand layer and the Cyclic Shear Strain value of 0.01%, we obtained the Volumetric Strain value of 0.02. This value translates into the total settlement of the dry sand layer is 0.5 inch. These values, in our opinion, are still within the range that could be tolerated by continuous perimeter foundation and isolated interior foundation.

E. 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 Alameda County Flood Control Canal, a lateral spreading study was performed for the site. An empirical method developed by *Barlett and Youd (1995)* was used in this study to estimate the amount of lateral movement of the ground surface. The following equation was used:

$$\text{Log } D_H = -15.787 + 1.178M - 0.927 \log R - 0.031R + 0.429 \log S + 0.348 \log T + 4.527 \log (100 - F) - 0.922D_{50}$$

Where:

D_H = Horizontal ground displacement in meter

M = Earthquake magnitude

R = Distance to the nearest fault rupture in kilometer

T = Cumulative thickness of the liquefiable soil layer in meter

F = Percent finer than No. 200 sieve

D_{50} = Grain size corresponding to 50% fine of liquefiable soil layer in millimeter

S = Slope gradient of the ground surface

For this study, there were no liquefiable soil layers identified. Therefore, the lateral ground surface is very minimal.

INUNDATION POTENTIAL

The subject site is located at 10 East Las Palmas Avenue in Fremont, 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 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 garage pad be underlain by a minimum of 12 inches non-expansive fill layer. This layer should be compacted to at least 90% relative maximum density.

3. 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 by our office prior to be used at the site.

4. Previous fill soil material to the depth of 7 feet was discovered in the low - lying area of the site. The material should be removed, re-backfilled and compacted properly if the area is planned to receive a structure.

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

6. We recommend a reference to our report should be stated in the grading and foundation plans (this includes the geotechnical investigation file number and dates).

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

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

9. 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, if any, which 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, if any, 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, if any, 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, native soil should be re-compacted to not less than 90% relative maximum density using ASTM

D1557-91 test procedure over the entire building pad and 5 feet beyond the foundation.

7. All engineered fill or imported soil should be placed in uniform horizontal lifts of not more than 6 to 8 inches in un-compacted thickness, and compacted to not less than 90% relative maximum density using ASTM D1557-91 procedure. This should extend a minimum of 10 feet beyond the perimeter of the pad. 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. 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 12 and an R-Value greater than 25.

10. We recommend that the final grading plan should be reviewed by our office prior to submitting to the appropriate local agency and/or to construction.

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 Alameda County 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 new residences be supported on continuous perimeter foundation and isolated interior (post on pier) foundation. Recommendations are presented in the following paragraphs.

14. When continuous perimeter and isolated interior spread footings are used, they must be founded at a minimum depth of 24 inches below rough soil pad. Under these conditions, the recommended allowable bearing capacity is 2,500 p.s.f. for both continuous perimeter and isolated and interior spread footings. Both interior and perimeter foundations should be founded at the same elevation below pad grade with the exception of any soil retaining structure foundations.

15. Because of the high expansion potential of the 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.

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.

2010 CBC SEISMIC VALUES

18. Site Class: D (Table 1613.5.2 CBC 2010)

Mapped Spectra Acceleration for short periods S_S = 2.011g*

Mapped Spectra Acceleration for 1-second period S_1 = 0.785g*

Designed Spectra Acceleration for short periods S_{DS} = 1.341g*

Designed Spectra Acceleration for 1-second period S_{D1} = 0.785g*

(* USGS Seismic Hazard Curves and Uniform Hazard Response Spectra for 2010 CBC analysis)

Site Coefficient F_a = 1.0 (Table 1613.5.3(1) CBC 2010)

Site Coefficient F_V = 1.5 (Table 1613.5.3(2) CBC 2010)

Maximum considered earthquake spectral response accelerations for short period S_{MS} = 2.011g ($S_{MS} = F_a S_S$ - Equation 16-37 CBC 2010)

Maximum considered earthquake spectral response accelerations for 1-second period S_{M1} = 1.178g ($S_{M1} = F_V S_1$ - Equation 16-38 CBC 2010)

RETAINING WALLS

19. Any facilities that will retain a soil mass, such as retaining walls, shall be designed for a lateral earth pressure (active) equivalent to 40 pounds equivalent fluid pressure for horizontal backfill, 45 pounds equivalent fluid pressure for 3:1 sloped backfill, and 50 pounds for 2:1 sloped backfill. 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. The structural engineer shall discuss the surcharge loads with the geotechnical engineer prior to designing the retaining walls.

20. In designing for allowable resistive lateral earth pressure (passive), a value of 300 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 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 equivalent may be used behind the retaining wall. The Miradrain 2000 should extend from the base of the wall to within two feet of the ground surface. A perforated pipe should be placed at the base of the wall in direct contact with the Miradrain 2000. The pipe should be sloped to outfall to an appropriate discharge facility. The Miradrain fabric at the base of the Miradrain 2000 panel should be wrapped around the perforated pipe to prevent soil instruction into the pipe. Retaining walls associated with the structure should be waterproofed.

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

CONCRETE SLAB-ON-GRADE CONSTRUCTION (GARAGE)

26. Based on the laboratory testing results of the near-surface soil, the native soil on the site was found to have a high expansion potential when subjected to fluctuation in moisture. Therefore, we recommend the building pad be underlain by a minimum of 12 inches non-expansive fill layer. The original native soil grade prior to fill placement and fill soil should be compacted to at least 90% relative maximum density and 95% for the baserock material.

27. A minimum of 5 inches of Class II Baserock or ¾ inch crushed rock (recycled baserock and/or crushed asphalt concrete is not acceptable) and 2 inches of sand with vapor barrier membrane (15 mil) shall be used between the finished grade and the concrete slab. The baserock should be compacted to not less than 95% relative maximum density and 90% for the subgrade according to ASTM D1557-91.

28. In lieu of the non-expansive soil and rock section, a total of 18 inches of 3/4 inch crushed rock or Class II Baserock can be placed under the concrete slab.

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 baserock and the sand layers. If the slab would not receive a floor covering, the sand and 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. Minor difficulties due to soil conditions are anticipated in excavating the on-site material. However, 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. 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 structures. Rainwater discharge at downspouts should be directed onto pavement sections, splash blocks, or other acceptable facilities that 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 5 percent.

ON-SITE UTILITY TRENCHING

37. All on-site utility trenches must be backfilled with native on-site material or imported fill and compacted to at least 90% relative maximum density in accordance with ASTM D1557-91. 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.

38. 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.

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

PAVEMENT DESIGN

40. 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. Alternate pavement section designs, which satisfy the State of California Standard Design Criteria, and above traffic indices, are presented

in Table II. Rigid pavement section designs are presented in Table III. Due to the high expansion potential of the surface native soil, minor cracks in the pavement should be expected.

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.

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. 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.

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.

Day W. Robert (2002). *Geotechnical Earthquake Engineering Handbook*. McGraw-Hill, New York.

Hays, W.W., 1980 - Procedures for Estimating Earthquake Ground Motions, Geological Survey Professional Paper 1114.

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 (2004). CGS Seismic Hazard Zone Report 098 [*Seismic Hazard Evaluation of the Niles 7.5-Minute Quadrangle, Alameda County, California*. Open-File Report 2004. Department Of Conservation. Division of Mines and Geology].

2010 (CBC) California Building Code, Title 24, Part 2.

USGS Seismic Hazard Curves and Uniform Hazard Response Spectra.

TABLES

TABLE I - SUMMARY OF MOISTURE/DENSITY, DIRECT SHEAR, PLASTICITY INDEX, AND LIQUID LIMITS

TABLE II - PROPOSED ALTERNATE PAVEMENT SECTIONS

TABLE III - PROPOSED RIGID PAVEMENT SECTIONS

TABLE I

SUMMARY OF MOISTURE/DENSITY, DIRECT SHEAR TEST, PLASTICITY INDEX, AND LIQUID LIMIT

Sample No.	Depth Ft.	In-Place Conditions: Dry Density p.c.f.	In-Place Conditions: Moisture Content % Dry Wt.	Direct Shear Testing: Angle of Internal Friction Degrees	Direct Shear Testing: Unit Cohesion k.s.f.	Plasticity Index (P.I.)	Liquid Limit (L.L.)
1-1	3	102.8	10.8	30	0.5		
1-2	5	96.6	11.1				
1-3	10	121.6	9.6				
1-4	15	122.2	9.1				
2-1	3	95.8	23.9				
2-2	5	110.0	20.4	20	0.9		
2-3	10	101.9	15.4				
2-4	15	110.5	19.4				
2-5	20	108.2	24.2			23	44
2-6	25	104.8	22.9				
2-7	30	107.5	20.1			25	45
2-8	35	108.6	19.9			20	41
2-9	40	109.3	20.4				
2-10	45	108.8	21.5			21	43
2-11	50	110.2	19.6				
3-1	3	112.9	11.5				
3-2	5	113.4	9.9				
3-3	10	110.7	10.6				

TABLE I (CONTINUED)

SUMMARY OF MOISTURE/DENSITY, DIRECT SHEAR TEST, PLASTICITY INDEX, AND LIQUID LIMIT

Sample No.	Depth Ft.	In-Place Conditions		Direct Shear Testing			
		Dry Density p.c.f.	Moisture Content % Dry Wt.	Angle of Internal Friction Degrees	Unit Cohesion k.s.f.	Plasticity Index (P.I.)	Liquid Limit (L.L.)
4-1	3	109.0	12.9				
4-2	5	92.1	13.6				
4-3	10	98.9	14.7				
5-1	3	108.1	12.8				
5-2	5	93.7	14.2				
5-3	10	100.1	14.4				
6-1	3	113.3	10.9				
6-2	5	114.2	9.0				
6-3	10	112.6	10.7				

TABLE II

PROPOSED ALTERNATE PAVEMENT SECTIONS

Location: Proposed Residential Development
The Club at Mission Hills
10 East Las Palmas Avenue
Fremont, 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 compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"

TABLE III

PROPOSED RIGID PAVEMENT SECTIONS

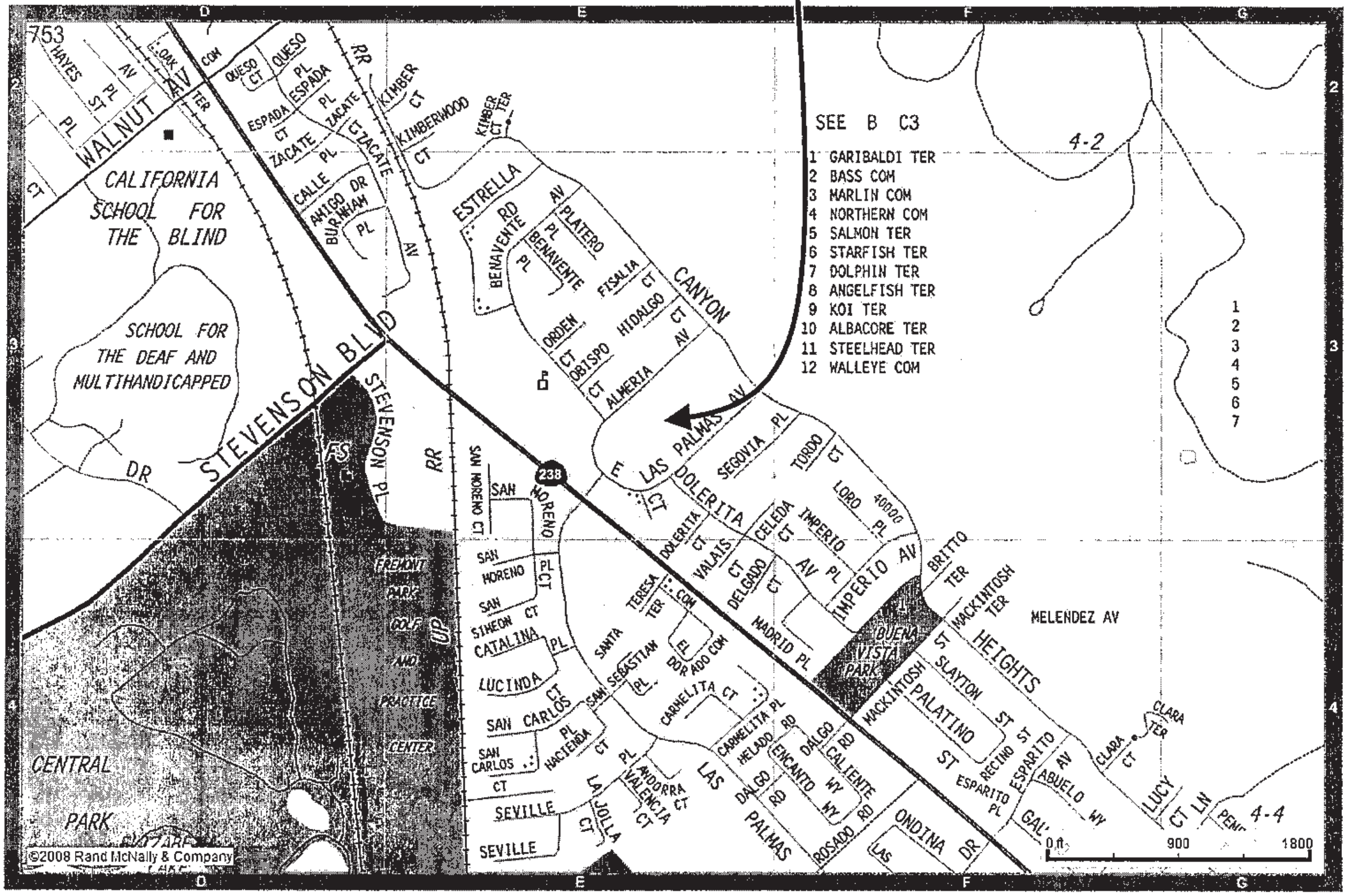
Location: Proposed Residential Development
The Club at Mission Hills
10 East Las Palmas Avenue
Fremont, California

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

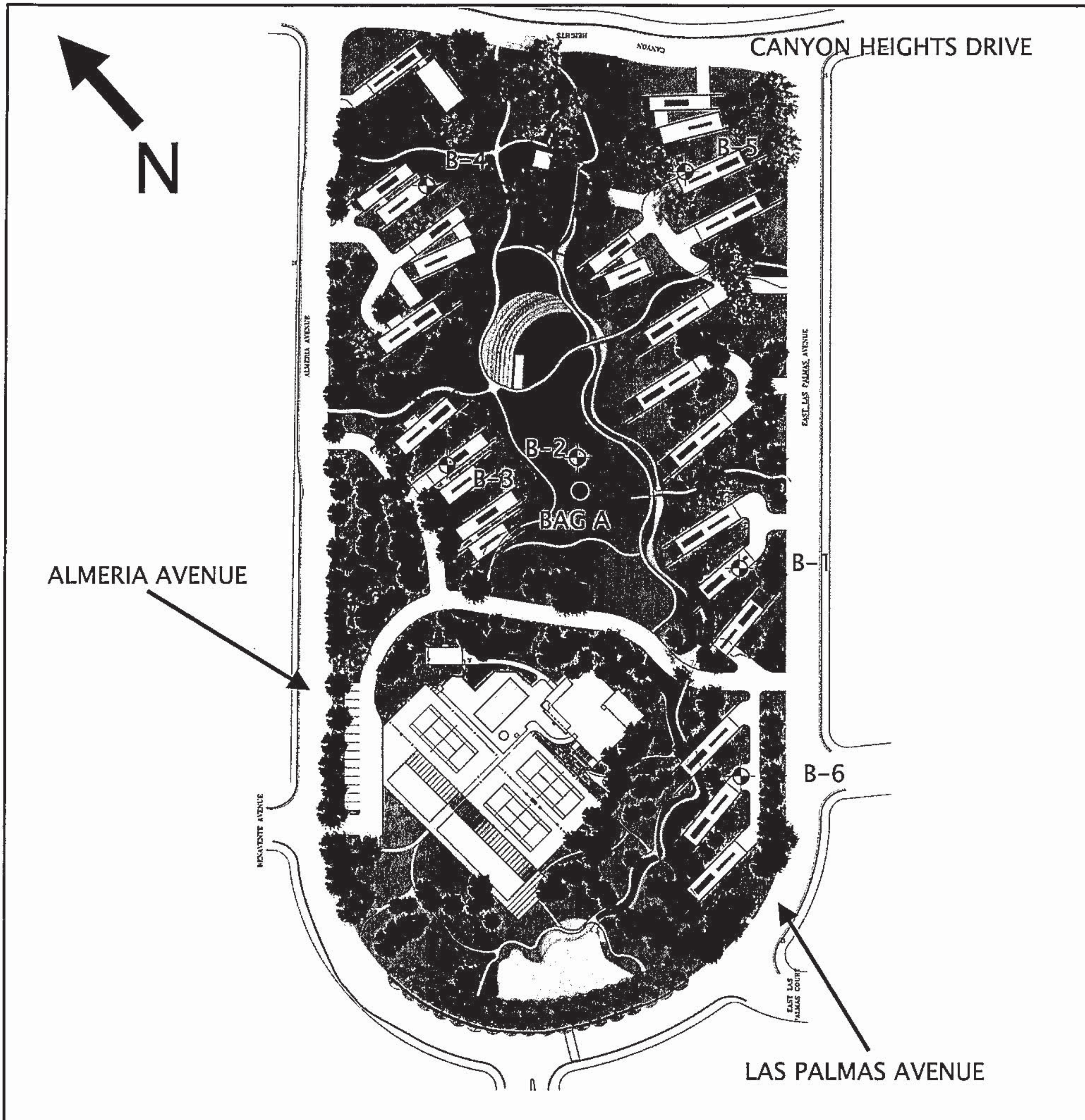
*Including trash enclosures, stress pads and valley gutters.

FIGURES

SITE

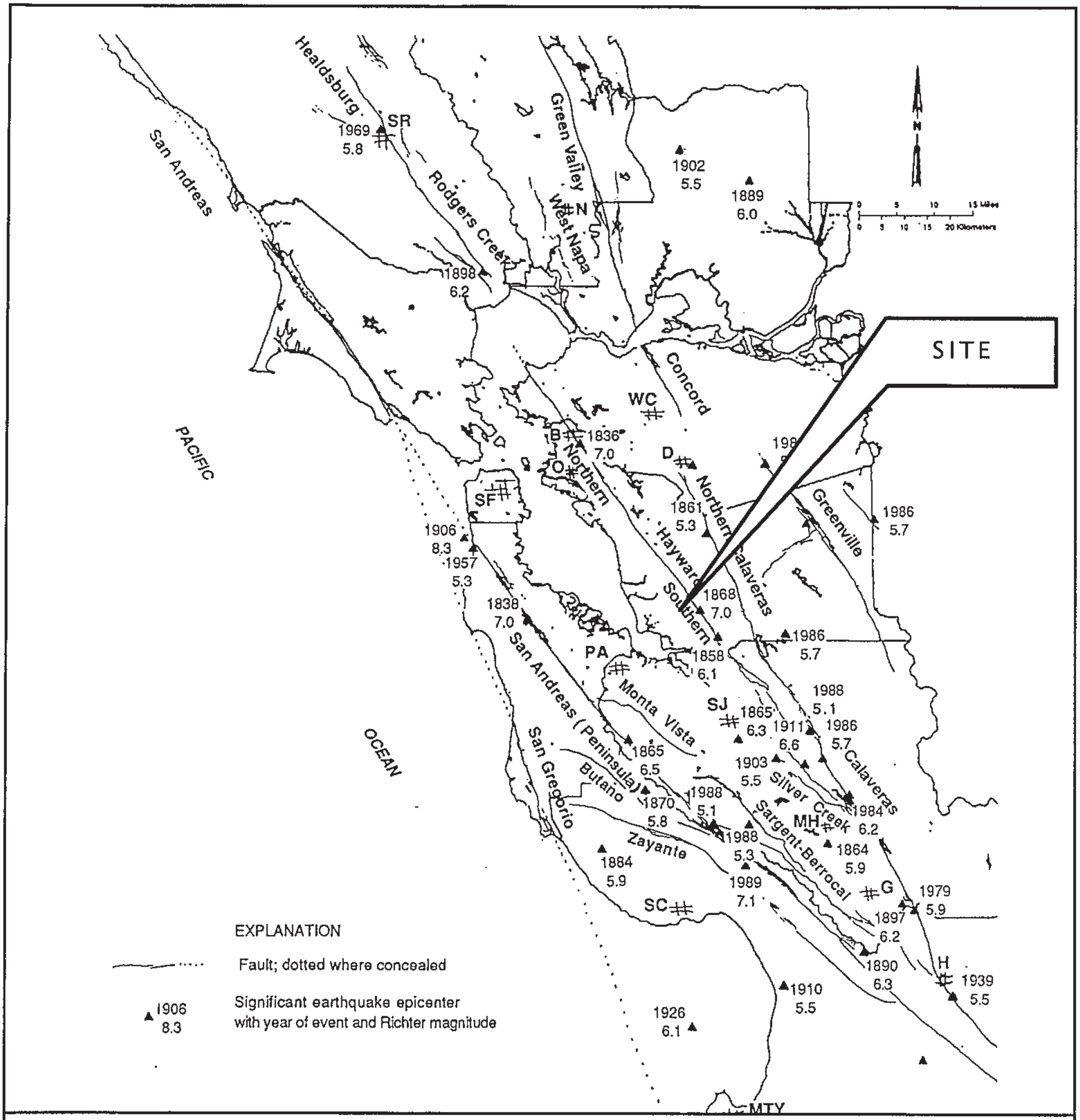


Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**VICINITY MAP** Proposed Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 1 July 2012

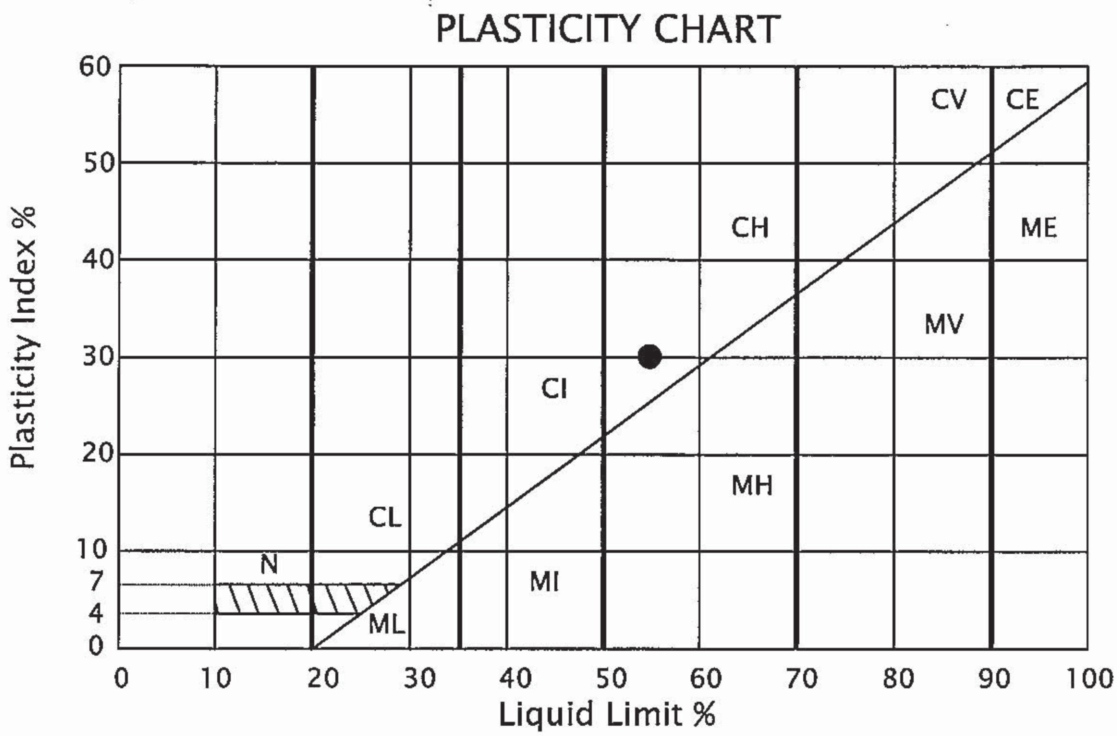


NOTE: DENOTES APPROXIMATE EXPLORATORY BORING LOCATION
DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	SITE PLAN Proposed Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 2 July 2012



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**FAULT LOCATION MAP** Proposed Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 3 July 2012

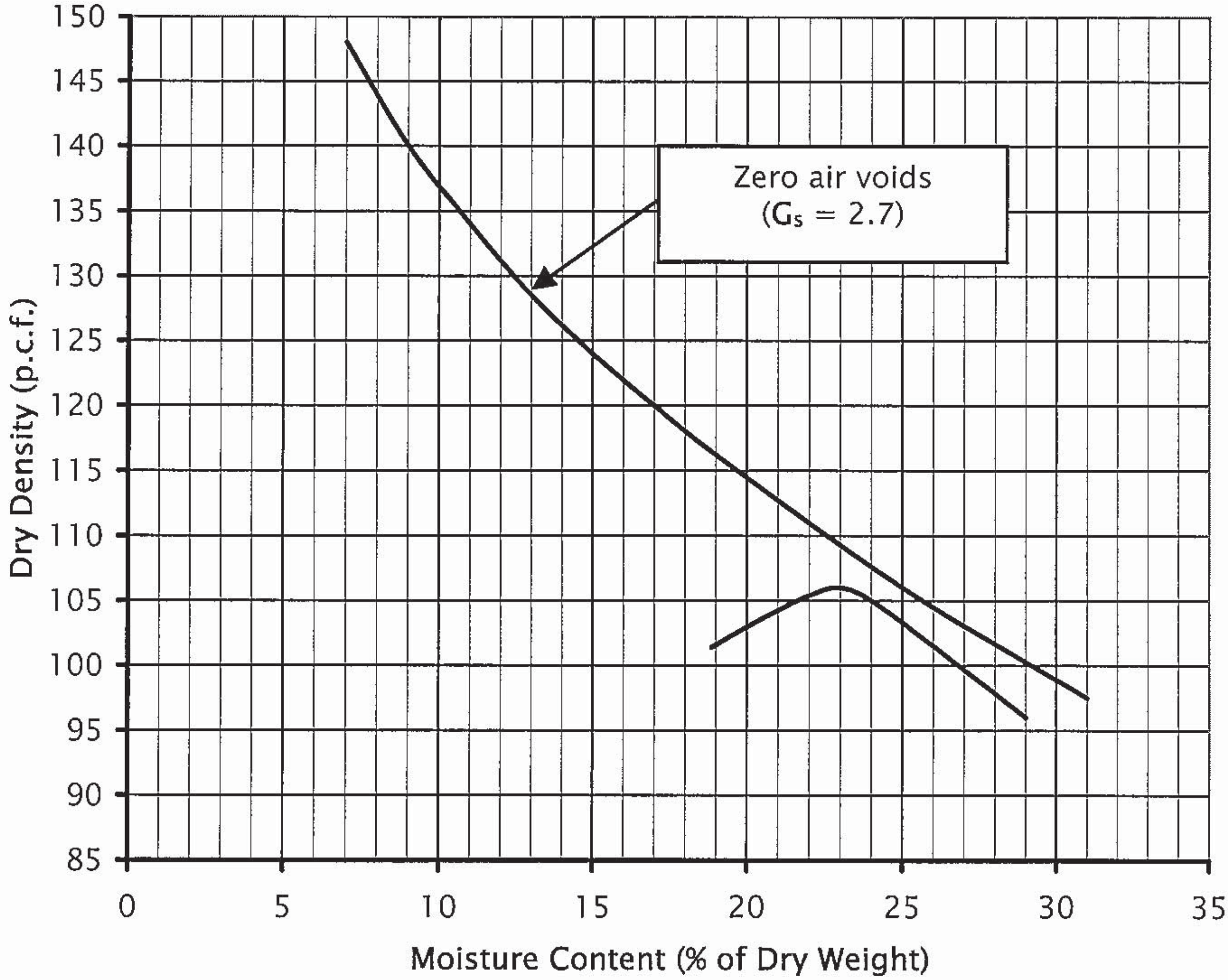


PLASTICITY DATA

Key Symbol	Hole No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	55	30	CH

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

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**PLASTICITY INDEX** Proposed Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 4 July 2012



SAMPLE: A

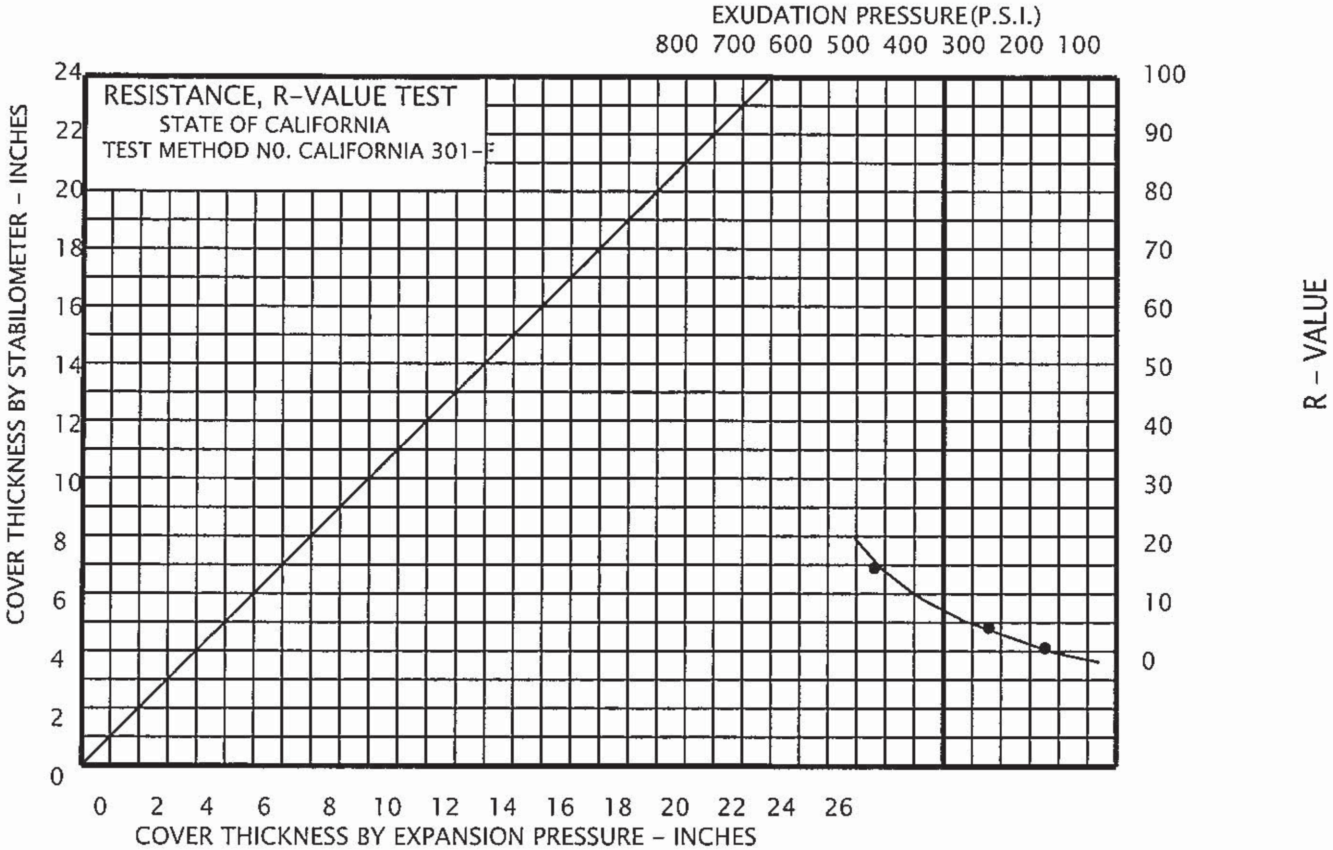
DESCRIPTION: Black Silty CLAY

LABORATORY TEST PROCEDURE: ASTM D1557-91

MAXIMUM DRY DENSITY: 106.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 23.0 %

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**COMPACTION TEST A** Proposed Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059	FIGURE
		Drawn by: V.V.	5
		Scale: NOT TO SCALE	July 2012



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	105.5	104.2
R-VALUE AT 300 P.S.I. EXUDATION PRESSURE	= (6)		

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	R-VALUE TEST Proposed Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 6 July 2012

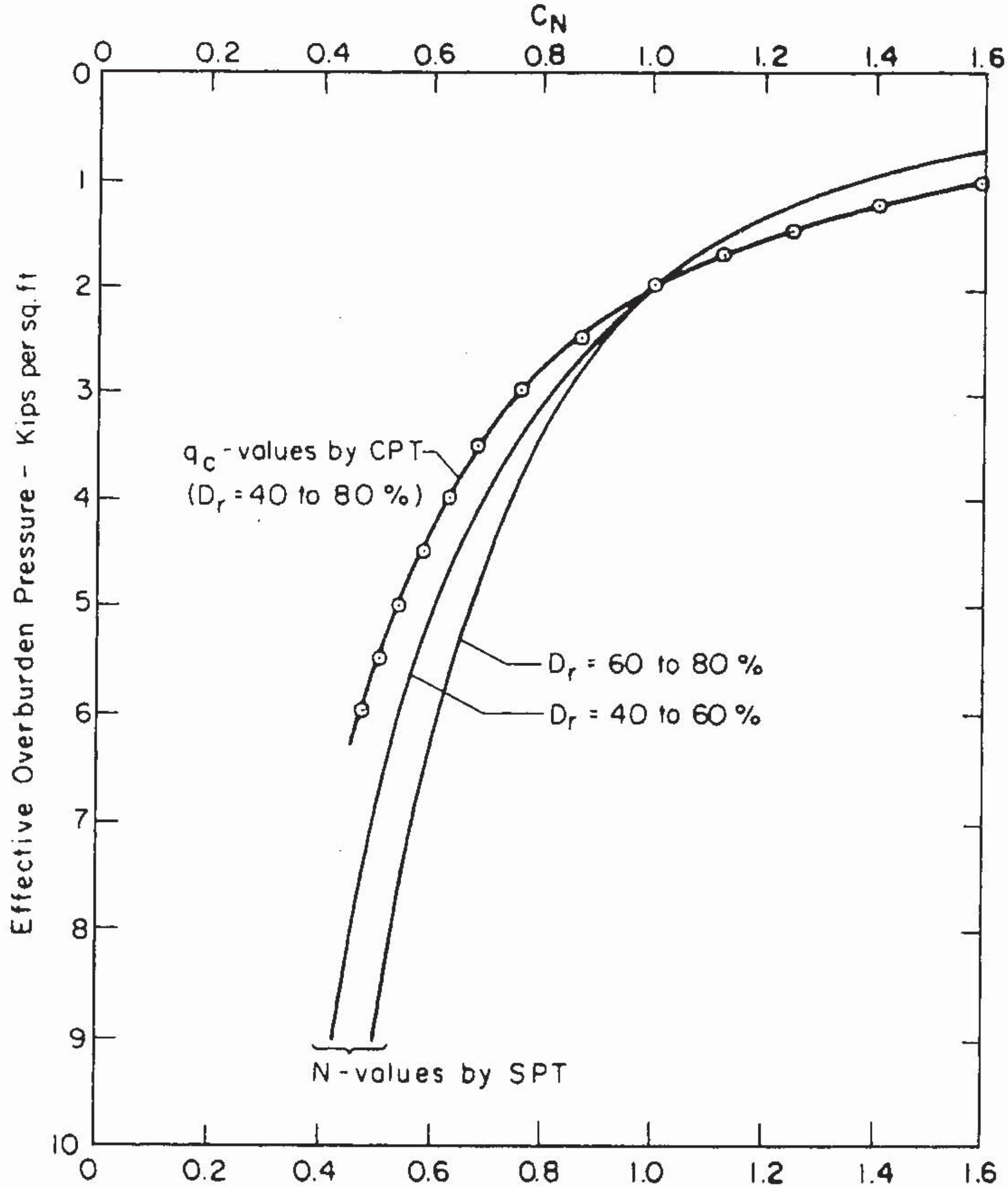


FIGURE 5.11 Correction factor C_N used to adjust the standard penetration test N value and cone penetration test q_c value for the effective overburden pressure. The symbol D_r refers to the relative density of the sand. (*Reproduced from Seed et al. 1983, with permission from the American Society of 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 Residential Development 10 E. Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V, Scale: NOT TO SCALE	FIGURE 7 July 2012

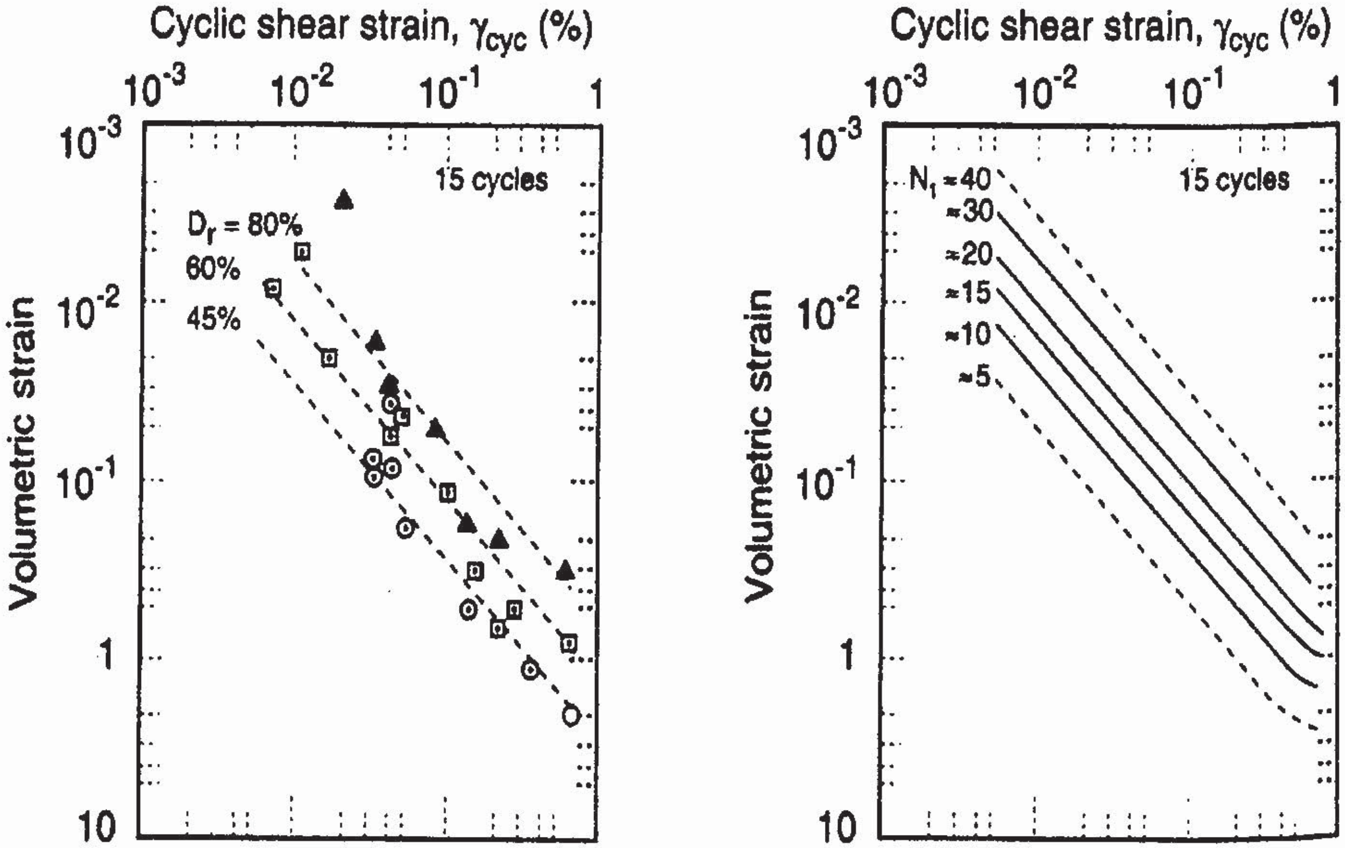


FIGURE 7.9 Plots that can be used to estimate the volumetric strain ε_v based on the cyclic shear strain γ_{cyc} and relative density D_r or N_1 value. Assume that N_1 in this figure refers to the $(N_1)_{60}$ values from Eq. (5.2). (*Reproduced from Tokimatsu and Seed 1987, with permission from the American Society of Civil Engineers.*)

Silicon Valley Soil Engineering 231 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	EARTHQUAKE-INDUCED SETTLEMENT FOR DRY SAND Proposed Residential Development 10 East Las Palmas Avenue Fremont, California	File No. SV1059 Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 8 July 2012

APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

EXPLANATION OF BORING LOG SYMBOLS

EXPLORATORY BORING LOGS (B-1 THROUGH B-6)

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.

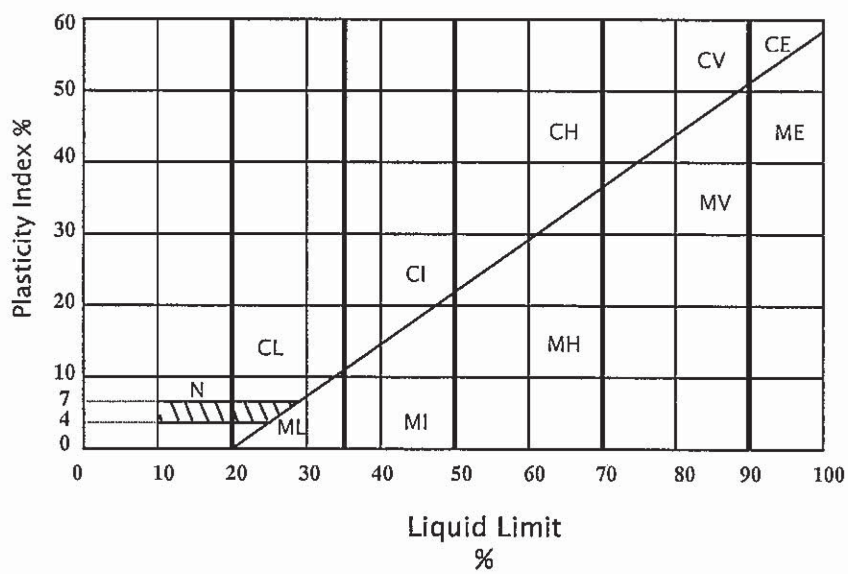
METHOD OF SOIL CLASSIFICATION CHART

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

CLASSIFICATION CHART - UNIFIED SOIL CLASSIFICATION SYSTEM

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

PLASTICITY INDEX CHART



Logged By:	EXPLANATION OF TEST BORING SYMBOLS	Boring No.
Date Drilled:		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									STATIC GROUNDWATER
									GROUNDWATER FIRST NOTED
									CLAY
									SILT
									SAND
									GRAVEL
									clayey
									silty
									sandy
									gravelly
						1-1			Sample Taken with sample number and lab results given
									Sample Attempt - Unsuccessful No Sample Number
		**							Refusal noted in Remarks
									CLAYSTONE/SILTSTONE BEDROCK

Remarks:

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No. B-1
Date Drilled: 06/27/12		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (P.I.)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										1.5" Asphalt Concrete/8" Aggregate Base Dark Brown SILT (ML)
102.8	10.8	65*	0.5	30			1-1	3		Damp, hard
96.6	11.1	78*					1-2	5		Color changed to brown Some small gravels
121.6	9.6	✠✠					1-3	10		Increasing gravel content Color changed to light brown
122.2	9.1	✠✠					1-4	15		
										Boring terminated at 15 feet

Remarks: ✠ Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586

✠✠ 50 blows for 6 inches

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No. B-2
Date Drilled: 06/27/12		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (P.I.)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										2 Inches of Organic Material Black Silty CLAY (CH)
95.8	23.9	9*					2-1	3		Moist, firm
110.0	20.4	10*	0.9	20			2-2	5		Color changed to dark brown
										Color changed to light brown
101.9	15.4	20*					2-3	10		
										Tan Brown Silty SAND (SP) Moist, medium dense**
										Medium Brown SILT (ML) Moist very stiff
110.5	19.4	36*					2-4	15		
										Reddish Brown Silty CLAY (CL) Moist, very stiff
108.2	24.2	33*			23	44	2-5	20		
104.8	22.9	47*					2-6	25		
107.5	20.1	67*			25	45	2-7	30		

Remarks: * Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586
** Fine grained, poorly graded

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No. B-2 (continued)
Date Drilled: 06/27/12		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (P.I.)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										Dark Reddish Brown Silty Sandy CLAY (SC) Moist, hard
108.6	19.9	✠✠			20	41	2-8	35		
109.3	20.4	✠✠					2-9	40		Hard drilling
108.8	21.5	✠✠			21	43	2-10	45		
110.2	19.6	✠✠					2-5	50		
										Boring terminated at 50 feet

Remarks: ✠ Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586
✠✠ 50 Blows for 6 inches

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No. B-3
Date Drilled: 06/27/12		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (P.I)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										1.5" Asphalt Concrete/7" Aggregate Base
										Medium Brown Sandy SILT (ML)
112.9	11.5	✠✠					3-1	3		Damp, hard
113.4	9.9	✠✠					3-2	5		Increasing gravel content
110.7	10.6	✠✠					3-3	10		
										Boring terminated at 10 feet

Remarks: ✠ Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586

✠✠ 50 blows for 6 inches

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No. B-4
Date Drilled: 06/27/12		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (p.I.)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										3 inches organic material
										Black Silty CLAY (CH)
109.0	12.9	50✠					4-1	3		Most, hard
										Color changed to brown
92.1	13.6	✠✠					4-2	5		
										Increasing gravel content
98.9	14.7	✠✠					4-3	10		
										Boring terminated at 10 feet

Remarks: ✠ Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586

✠✠ 50 blows for 6 inches

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No.
Date Drilled: 06/27/12		B-5

File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (p.I.)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										3 inches organic material
										Black Silty CLAY (CH)
108.1	12.8	46*					5-1	3		Most, hard
										Color changed to brown
93.7	14.2	**					5-2	5		
										Increasing gravel content
100.1	14.4	**					5-3	10		
										Boring terminated at 10 feet

Remarks: * Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586

** 50 blows for 6 inches

Logged By: V.V.	EXPLORATORY BORING LOG	Boring No. B-6
Date Drilled: 06/27/12		File No. SV1059

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Plasticity Index (p.I.)	Liquid Limit (L.L.)	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
										2" Asphalt Concrete/8" Aggregate Base
										Medium Brown Sandy SILT (ML)
113.3	10.9	47*					6-1	3		Damp, hard
114.2	9.0	**					6-2	5		Increasing gravel content
112.6	10.7	**					6-3	10		
										Boring terminated at 10 feet

Remarks: * Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586

** 50 blows for 6 inches