APPENDIX GEO

REPORT TO

DPM PROPERTY MANAGEMENT, INC.
REDWOOD CITY, CALIFORNIA

FOR

PROPOSED LINCOLN LANDING
MIXED-USE DEVELOPMENT
22301 FOOTHILL BOULEVARD
HAYWARD, CALIFORNIA

GEOTECHNICAL INVESTIGATION

APRIL 2015

PREPARED BY

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

SILICON VALLEY SOIL ENGINEERING
GEOTECHNICAL CONSULTANTS

File No. SV1302A
April 20, 2015

DPM Property Management, Inc.
555 Twin Dolphin Drive, Suite 600
Redwood City, CA 94065

Attention: Mr. Scott A. Athearn

Subject: Proposed Lincoln Landing
Mixed-Use Development
22301 Foothill Boulevard
Hayward, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Athearn:

Pursuant to your request, we are pleased to present herein geotechnical investigation for the proposed Lincoln Landing Mixed-Use Development. The subject site is located at 22301 Foothill Boulevard in Hayward, California.

Our findings indicate that the site is suitable for the development provided the recommendations contained in this report are carefully followed. Field reconnaissance, drilling, sampling, and laboratory testing of the surface and subsurface material evaluated the suitability of the site. The following report details our investigation, outlines our findings, and presents our conclusions based on those findings.

If you have any questions or require additional information, please feel free to contact our office at your convenience.

Very truly yours,

SILICON VALLEY SOIL ENGINEERING

Sean Deivert
Project Manager

Vien Vo, P.E.

REGISTERED PROFESSIONAL ENGINEER
VIEN M. VO
RCE 32296
EXP. 12/31/16
CIVIL
STATE OF CALIFORNIA

SV1302A.GI/Copies: 4 to DPM Property Management, Inc.

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associated improvements. The approximate location of the existing and proposed buildings and our borings are shown on the Site Plan (Figure 2).

PREVIOUS FIELD INVESTIGATION

In October 2014, our office performed a geotechnical investigation for a proposed building located at 22301 Foothill Boulevard in Hayward, California. This previous site was a small northwestern portion of the current site (APN 428 002606703). Two exploratory borings were drilled to the depths of 21.5 feet to 51.5 feet below existing pavement surface during the field investigation. The results of the investigation were presented in a report; File No. SV1302 dated October 3, 2014. The subsurface data contained from the investigation were reviewed and used for the preparation of this report. Copies of the previous exploratory borings are included at the end of the report.

CURRENT 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 three exploratory test borings and sounding of four CPT's (Cone Penetration Tests) per ASTM D5778 to determine the subsurface soil characteristics. The borings were drilled on April 1, 2015. The approximate location of the borings is shown on the Site Plan (Figure 2). The borings were drilled to the depths ranging from 20 feet to 80 feet below existing pavement surface. The borings were drilled with a truck mounted drill rig using 8-inch diameter hollow stem augers. The CPT's were advanced on the same day by Gregg Drilling & Testing, Inc. to the depths of 69 feet to 80 feet below existing pavement surface.

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 22301 Foothill Boulevard in Hayward, California (Figure 1). Elevations of the subject site range from 96 to 114 feet. Hazel Avenue and an existing gas service station bound the subject site to the northwest, Foothill Boulevard to the northeast, City Center Drive to the southeast, and San Lorenzo Creek to the south and southwest. The concrete lined creek flow line elevation range approximately from 73 to 78 feet. At the time of this investigation, the subject site consists of two parcels (APN 428 002606703 & 428 002606801) occupied by a one-story vacant building northwest of the subject site, a three-story vacant former Mervyns building and four-level parking structure to the southeast surrounded by paved parking and driveway area. Based on the available information for the subject site, the development will include the demolition of the existing buildings, removal of existing asphalt, and the construction of six-story mixed-use building and two one-story retail buildings. However, the four-level parking structure will remain. The six-story building will consist of various configurations with ground floor retail units and parking garage and include one level of parking and residential units above. In addition, the development will include a swimming pool at podium slab, public park and surrounding parking area with

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 (SPT), ASTM Standard D1586, into the ground at various depths. A 140-pound hammer with a free fall of 30 inches was used to drive the sampler 18 inches into the ground. Blow counts were recorded on each 6-inch increment of the sampled interval. The blows required to advance the sampler the last 12 inches of the 18 inch sampled interval were recorded on the boring logs as penetration resistance. These values were also used to evaluate the liquefaction potential of the subsurface soils. 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 Alameda County Public Works. A copy of the drilling permit is enclosed at the end of the report.

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. (Table I & Figure 4).

3. The strength parameters of the foundation soils were determined from direct shear tests that were performed on selected relatively undisturbed soil samples at various depths. The results were used in the vertical and lateral analysis of deep foundation (pre-stress, pre-cast concrete driven pile) recommendations (Table I).

4. Laboratory compaction tests were performed on the near-surface material per the ASTM D1557-12 test procedure (Figure 5).

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

6. One R-Value test was performed on a near surface soil sample for pavement section design recommendations (Figure 6).

The results of the laboratory-testing program are presented in the Tables and Figures at the end of this report.

SOIL CONDITIONS

In Boring B-3 (80 feet boring), the pavement surface soils consist of 3.0 inches of asphalt concrete over 9.0 inches of aggregate base. Below the pavement sections to the depth of 4 feet, a dark olive brown, moist, stiff sandy silt layer was encountered. From the depths of 4 feet to 20 feet, the soil became dark olive brown, damp, medium dense silty sand. The sand was medium grained and poorly graded. Color changes of brown and olive brown were noted at the depths of 10 feet and 17 feet. From the depths of 20 feet to 28 feet, a greenish gray, moist, firm clayey silt layer was encountered. From the depths of 28 feet to 38 feet, the soil became reddish brown, moist, dense sandy clayey gravel.

The gravel was 1.5 inches in maximum diameter, sub-rounded, and poorly graded. From the depths of 38 feet to 45 feet, a tan brown, moist, dense silty sand layer was encountered. The sand was medium grained and poorly graded. From the depths of 45 feet to the end of the boring at 80 feet, the soil became olive brown, moist, hard silty clay. A color change of greenish gray was noted at a depth of 59 feet. Similar soil profiles were encountered in other borings.

Groundwater was initially encountered in Boring B-1 at the depth of 22 feet and rose to a static level of 20 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 Boring B-3 at depths of 22 feet and rose to a static level ranging from 20 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 091 [*Seismic Hazard Evaluation of the Hayward 7.5-Minute Quadrangle, Alameda County, California. 2003 (Revised 10/10/2005).* Department Of Conservation. Division of Mines and Geology], the highest expected groundwater level is approximately 12 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 (SPT), ASTM Standard D1586-92.

The results from our exploratory boring show that the subsurface soil material in Boring B-3 to the depth of 80.0 feet consists stiff clayey sandy silt to medium dense silty sand to firm clayey silt to dense sandy clayey gravel to dense silty sand to hard silty clay. The following is the determination of the liquefiable soil for each soil layer in Boring B-3.

1. The stiff sandy silt layer from the surface to the depth of 4 feet <u>is not liquefiable</u> soil because it is above the highest expected groundwater table.
2. The medium dense silty sand from the depth of 4 feet to 12 feet <u>is not liquefiable</u> soil because it is above the highest expected groundwater table.
3. The medium dense silty sand layer from the depths of 12 feet to 20 feet <u>is liquefiable</u> soil based on the low blow counts and PI (PI<12).
4. The firm clayey silt layer from the depths of 20 feet to 28 feet <u>is not liquefiable</u> soil because based on the Plasticity Index (PI) and moisture contents (MC):
    - Sample No. 3-6 (25 feet) - [PI > 12; PI = 17 and MC = 36.4% < 85% LL = 38.3% ; LL = 45]

5. The dense sandy clayey gravel layer from the depths of 28 feet to 38 feet is not liquefiable soil based on the high blow counts.

6. The dense silty sand layer from the depths of 38 feet to 45 feet is not liquefiable soil based on the high blow counts.

7. The hard silty clay layer from the depth of 45 feet to the end of the boring at 80.0 feet is not liquefiable soil because based on the Plasticity Index (PI) and moisture contents (MC):

   - Sample No. 3-10 (50 feet) - [PI > 12; PI = 22 and MC = 30.9% < 85% LL = 35.7%; LL = 42]
   - Sample No. 3-11 (60 feet) - [PI > 12; PI = 23 and MC = 31.2% < 85% LL = 38.3%; LL = 45]
   - Sample No. 3-12 (70 feet) - [PI > 12; PI = 21 and MC = 23.7% < 85% LL = 34.0%; LL = 40]
   - Sample No. 3-11 (80 feet) - [PI > 12; PI =21 and MC = 22.6% < 85% LL = 34.9%; LL = 41]

In summary, there is one liquefiable soil layer underlying Boring B-3. This is the medium dense sand layer from the depths of 12 feet to 20 feet. There are two liquefiable soil layers underlying previous Boring B-1. CPT-01 through CPT-04 show liquefiable soil layers as yellow and tan color layers in Soil Behavior Type (SBT) columns. The CPT graphs were included at the end of the report.

C. 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 091 [*Seismic Hazard Evaluation of the Hayward 7.5-Minute Quadrangle, Alameda County, California*. 2003 (revised 10/10/2005). Department of Conservation. Division of Mines and Geology], the peak ground acceleration is 0.71g.

D. LIQUEFACTION ANALYSIS

The evaluation procedure is a semi-empirical method for a moment magnitude Mw7.9 earthquake, a peak ground acceleration of 0.71g, and highest expected groundwater table of 12 feet. A computer program named LiquefyPro Version 5.8n (CivilTech Corporation) was used in the liquefaction analysis for previous Boring B-1, current Boring B-3, and current CPT-01 through CPT-04. This program is based on the most recent publications of NCEER Workshop and procedure outline in SP117 Implementation. Based on our analysis, it is our opinion that the liquefaction potential of the sand layers are moderately high. The safety factor is less than 1.3. In addition, based on our analysis using Modified Robertson and Ishihara & Yosemine, we estimated maximum total settlements from liquefaction are approximately 8.6 inches and the maximum differential settlements are 5.7 inches. The results of the analysis including the liquefaction-induced settlements are enclosed at the end of the report.

E. 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 7. These figures were reproduced from *Kramer 1996, which was originally developed by Ishihara 1985*. In plotting the coordinates of the suspected liquefiable sand layers of previous Boring B-1 and current Boring B-3 in Figure 7, the thickness of surface non-liquefiable (H_1) soil layer and the thickness of the liquefiable (H_2) soil layer in previous Boring B-1 and current Boring B-3 was entered with a maximum peak acceleration of $a_{max} = 0.71g$. The following is the determination of H_1 and H_2.

Previous Boring B-1: H_1 = 4 meters; H_2 = 3.33 meters

Current Boring B-3: H_1 = 4 meters; H_2 = 3.33 meters

Based on the plotted coordinates of the suspected liquefiable soil layer of previous Boring B-1 and current Boring B-3 using the above data, we concluded that there is a moderate potential for liquefaction-induced ground surface damage to occur at the site.

F. 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 San Lorenzo Creek south and southwesterly 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:

$$\text{Log DH} = -16.213 + 1.532M - 1.406 \log R^* - 0.012R + 0.338 \log S + 0.540 \log T_{15} + 3.413 \log (100 - F_{15}) - 0.795 \log(D50_{15} + 0.1 \text{ mm})$$

Where:

DH = 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 = 1 kilometer fro Hayward Fault, R_0 = 84, R^* = 108

T_{15} = 0 meter, F_{15} = 0.1%, $D50_{15}$ = 1.5 millimeter, S = 2%

The lateral movement of the ground surface soil is calculated to be approximately 0.5 meters (1.5 feet or 18 inches) with respect to the Hayward Fault. Based on the magnitude of the lateral movement, we concluded that the liquefaction-induced lateral spreading is moderate.

G. CONCLUSIONS

The followings are the conclusions of this study.

Boring	Liquefaction-induced total max settlement (inch)	Liquefaction-induced differential settlement (inch)
B-1	4.92	3.25
B-3	3.23	2.14
CPT-01	0.59	0.39
CPT-02	4.90	0.39
CPT-03	8.60	5.68
CPT-04	6.08	4.01

- The liquefaction-induced total maximum settlement at the site is 8.6 inches.
- The liquefaction-induced differential settlement at the site is 5.7 inches.
- The potential of liquefaction-induced ground surface damage at the site is moderate.
- The liquefaction-induced lateral spreading is moderate.

INUNDATION POTENTIAL

The subject site is located at 22301 Foothill Boulevard in Hayward, 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 low expansion potential when subjected to fluctuations in moisture.

3. 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 pads and parking area rock section. The crushed baserock material for the building pads should be free of crushed asphalt concrete. Crushed cinder block, if any, can not be used as baserock material. The baserock material should be inspected and tested prior to final approval and use.

4. Because of the large liquefaction-induced settlements at the site, we recommend the one-story retail buildings should be supported on mat foundation. The six-story building should be supported on pre-cast, pre-stress concrete driven pile on perimeter grade beam for exterior walls and on pile cap for interior columns with structural concrete slab floor.

5. We recommend the exterior of the building pad be graded to permit proper drainage and diversion of water away from the building foundations.

6. Since the site is located in a low-lying area or adjacent to any creek or drainage channel, minor cracks and separations of the asphalt concrete pavement and/or curb and gutter should be expected.

7. 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).

8. 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.

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

10. 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 in the new building pad area must be removed prior to any grading at the site.

3. The depressions left by the removal of subsurface structures, if any, should be cleaned of all debris, backfilled and compacted with clean, native soil. This backfill must be engineered fill and should be conducted under the supervision of a SVSE representative.

4. All organic surface material and debris shall be stripped prior to any other grading operations, and transported away from all areas that are to receive structures or structural fills. Soil containing organic material may be stockpiled for later use in landscaping areas only.

5. After removing all the subsurface structures or 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 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

D1557-12 procedure over the entire building pad and 5 feet beyond the perimeter of the pad and 3 feet for the parking/driveway area.

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 D1557-12 procedure. The baserock, however, should be compacted to not less than 95% relative maximum density. Before compaction begins, the subgrade and/or fill material shall be brought to a water content that will permit proper compaction by either; 1) aerating the material if it is too wet, or 2) spraying the material with water if it is too dry. Each lift shall be thoroughly mixed before compaction to assure a uniform distribution of water content.

8. When fill material includes rocks, nesting of rocks will not be allowed and all voids must be carefully filled by proper compaction. Rocks larger than 4 inches in diameter should not be used for the final 2 feet of building pad.

9. Unstable (yielding) subgrade should be aerated or moisture conditioned as necessary. Yielding isolated area in the subgrade can be stabilized with an excavation of the subgrade to the depth of 12 to 18 inches, lined with stabilization fabric membrane (Mirafi 500X or equivalent) and backfilled with aggregate base.

10. 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 Alameda County Public Works. 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. Due to large liquefaction-induced total and differential settlements, the proposed one-story retail buildings should be supported on a mat foundation and the six-story mixed-use building should be supported on pre-cast, pre-stress concrete driven pile on perimeter grade beam for exterior walls and on pile cap for interior columns and structural concrete slab floor. Recommendations are presented in the following paragraphs.

14. The mat foundation should have a minimum thickness of 12 inches with 18 inch thickened edge. Under these conditions, the allowable contact pressure is 2,200 psf. The modulus of subgrade reaction can be taken as 150 pci in the design of the mat foundation.

15. The pile should be 14-inch square and terminated at a minimum depth of 65 feet below ground surface. The structural slab should have a minimum thickness of 12 inches and an allowable contact pressure of 1,200 psf.

16. A computer program ALLPile7 was used in the vertical and lateral analysis of the pile and soil interaction. The results are included in the figures and at the computer printouts are included at the end of the report.

17. VERTICAL ANALYSIS

- The ultimate vertical load carrying capacity and uplift capacity for 80 feet length pile are 1537 kips and 1131 kips respectively.
- The allowable vertical load carrying capacity and uplift capacity for 80 feet length pile are 987 kips and 582 kips respectively.
- The soil stress, side resistance, and axial force versus depth are shown in Figure 8.
- The vertical load versus total settlement are shown in Figure 9.
- The ultimate capacities versus pile depth are shown in Figure 10.
- The side resistance versus relative movement between soil and pile are shown in Figure 11.
- The tip resistance versus the tip moment are shown in Figure 12.
- The total settlement of the pile due to vertical loading is calculated to be 0.44 inch.

18. LATERAL ANALYSIS

- The pile deflection and force versus pile for free and fixed end conditions are shown in Figure 13 and Figure 18.
- The maximum allowable lateral shear force should be limited to 30 kips with the maximum allowable lateral deflection at the top of the pile is 0.249 inch for free end condition.

- The maximum allowable lateral shear force should be limited to 78 kips with the maximum allowable lateral deflection at the top of the pile is 0.247 inch for fixed end condition.

- The pile deflection versus loading for free and fixed end condition are shown in Figure 14 and Figure 19.

- The pile moment versus loading for free end and fixed end conditions are shown in Figure 15 and Figure 20.

- The soil resistance versus pile deflection for free and fixed end conditions are shown in Figure 16 and Figure 21.

- The lateral load versus deflection and maximum moment for free and fixed end conditions are shown in Figure 17 and Figure 22.

19. Additional lateral resistance can be mobilized by the pile caps and the soil in the form of passive resistance. A passive pressure of 250 pcf equivalent fluid pressure should be used. Passive pressure may be increased by one third for seismic loading.

20. The minimum pile spacing clearance should be 2.5 times the pile diameter (nominal dimension).

21. Pile specifications are included at the end of the report.

2013 CBC SEISMIC VALUES

22. 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	37.36700° N.
Site Longitude	122.90819° 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} ($S_{MS} = F_a S_S$ - Equation 11.4-1 CBC 2013)	1.500g*
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration S_{M1} ($S_{M1} = F_V S_1$ - Equation 11.4-2 CBC 2013)	0.900g*
0.2-second Period, Designed Spectra Acceleration, S_{DS} ($S_{DS} = 2/3 S_{MS}$ - Equation 11.4-3 CBC 2013)	1.000g*
1-second Period, Designed Spectra Acceleration, S_{D1} ($S_{D1} = 2/3 S_{M1}$ - 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

23. Any facilities that will retain a soil mass above grade 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.

24. 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.

25. 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.

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

27. 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.

28. As an alternative to the drain rock and fabric, Miradrain 2000 or approved equivalent drain mat may be used behind the retaining wall. The drain mat should extend from the base of the wall to the ground surface. A perforated pipe (subdrain system) should be placed at the base of the wall in direct contact with the drain mat. The pipe should be sloped to outfall to an appropriate discharge facility.

29. The elevator pit walls and associated building retaining walls, if any, should be waterproofed with Paraseal LG or equivalent.

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

CONCRETE SLAB CONSTRUCTION

31. 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 low expansion potential when subjected to fluctuations in moisture.

32. Concrete floor (mat and structural slab) shall be underlain by a minimum of 5 inches of Class II Baserock or 3/4 inch crushed rock with vapor barrier membrane (Stego 15 mil) and placed 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.

33. 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. The vapor barrier membrane should be overlapped, taped at seams and/or mastic applied for protrusions.

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

EXCAVATION

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

36. 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

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

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

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

40. 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.

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

42. Based on laboratory test results of the near surface soil at the subject site, we estimated that the infiltration rate is approximately 1 inch per hour. This rate can be used in the design of the bio-retention system for on-site storm drainage.

ABANDONMENT OF THE EXISTING UTILITY LINES

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

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

45. 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

46. All on-site utility trenches must be backfilled with native on-site material or import fill and compacted to at least 90% relative maximum density. Backfill should be placed in 8 to 12 inch lifts and compacted. Jetting of trench backfill is not recommended. An engineer from our firm should be notified at least 48 hours before the start of any utility trench backfilling operations.

47. 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.

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

PAVEMENT DESIGN

49. 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.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations presented herein are based on the soil conditions revealed by our test boring(s) and evaluated for the proposed construction planned at the present time. If any unusual soil conditions are encountered during the construction, or if the proposed construction will differ from that planned at the present time, Silicon Valley Soil Engineering (SVSE) should be notified for supplemental recommendations.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the necessary steps are taken to see that the contractor carries out the recommendations of this report in the field.

3. The findings of this report are valid, as of the present time. However, the passing of time will change the conditions of the existing property due to natural processes, works of man, from legislation or the broadening of knowledge. Therefore, this report is subjected to review and should not be relied upon after a period of three years.

4. The conclusions and recommendations presented in this report are professional opinions derived from current standards of geotechnical practice and no warranty is intended, expressed, or implied, is made or should be inferred.

5. The area of the boring(s) is/are very small compared to the site area. As a result, buried structures such as septic tanks, storage tanks, abandoned utilities, or etc. may not be revealed in the boring(s) during our field investigation. Therefore, if buried structures are encountered during grading or construction, our office should be notified immediately for proper disposal recommendations.

6. Standard maintenance should be expected after the initial construction has been completed. Should ownership of this property change hands, the prospective owner should be informed of this report and recommendations so as not to change the grading or block drainage facilities of this subject site.

7. 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.

Civil Tech Software (ALLPile 7)

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.

Risk Engineering, Inc., Seismic Hazard Analysis and Site Response Analysis Software (2009) (EZ-FRISK 7.62 Build 001)

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 (2002). CGS Seismic Hazard Zone Report 058 (Revised 10/10/2005) [*Seismic Hazard Evaluation of the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California. 2002 (Revised 10/10/2005).* Department Of Conservation. Division of Mines and Geology],

Youd T. Leslie, Hanson M. Corbett, and Barlett F. Steven, 2002 - *Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement*, Journal of Geotechnical and Geoenvironmental Engineering, December 2002.

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

TABLES

TABLE I

SUMMARY OF LABORATORY TESTS

Sample No.	Depth Ft.	In-Place Conditions		Direct Shear Testing		Atterberg Limits	
		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.
3-1	1.5	17.8	104.4	0	22		
3-2	3.5	18.8	106.0				
3-3	8.5	11.9	109.9	0	33		
3-4	13.5	21.6	102.5				<12
3-5	18.5	24.8	95.7				<12
3-6	23.5	36.4	87.1	0.5	18	45	17
3-7	28.5	17.5	118.6				
3-8	33.5	12.0	129.9				
3-9	38.5	27.2	99.4				
3-10	48.5	30.9	95.3	1.4	17	42	22
3-11	58.5	31.2	93.6	1.6	13	45	23
3-12	68.5	23.7	105.2	1.7	15	40	21
3-13	78.5	22.6	107.0	1.9	6	41	21
4-1	1.5	23.3	103.4				
4-2	3.5	20.2	102.5				
4-3	8.5	20.3	105.5				
4-4	13.5	28.3	97.1				
4-5	18.5	23.9	100.9				

TABLE I (CONTINUED)

SUMMARY OF LABORATORY TESTS

Sample No.	Depth Ft.	In-Place Conditions: Moisture Content % Dry Wt.	In-Place Conditions: Dry Density p.c.f.	Direct Shear Testing: Unit Cohesion k.s.f.	Direct Shear Testing: Angle of Internal Friction Degrees	Atterberg Limits: Liquid Limit L.L.	Atterberg Limits: Plasticity Index P.I.
5-1	1.5	17.0	113.1				
5-2	3.5	5.8	95.8				
5-3	8.5	9.2	94.0				
5-4	13.5	9.6	96.3				
5-5	18.5	6.0	94.2				

TABLE II

PROPOSED ALTERNATE PAVEMENT SECTIONS

Location: Proposed Lincoln Landing
Mixed-Use Development
22301 Foothill Boulevard
Hayward, 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"
Subgrade soil scarified and 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 Lincoln Landing
Mixed-Use Development
22301 Foothill Boulevard
Hayward, California

	DRIVEWAY *	SIDEWALK
Recommended Rigid Pavement Sections:		
P.C. Concrete	6.0"	4.0"
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	6.0"	4.0"
Subgrade soil scarified and compacted to at least 90% relative maximum density	12.0"	12.0"

* Including trash enclosures, stress slabs, valley gutters, and curb & gutters. Reinforcement provided by Structural Engineer. Maximum control joints at 10' x 10'.

TABLE IV

PROPOSED PAVER PAVEMENT SECTIONS

Location: Proposed Lincoln Landing
Mixed-Use Development
22301 Foothill Boulevard
Hayward, California

	DRIVEWAY/PARKING AREA			
Recommended Paver Pavement Sections:	1A*	1B*	2A	2B
Vehicular Rated Pavers	Min. 3.25" ± Permeable Paver Parking Stalls	Min. 3.25" ± Permeable Paver Driveway	Min. 3.25" ± Non-Permeable Paver Parking Stalls	Min. 3.25" ± Non-Permeable Paver Driveway
ASTM No. 8 Bedding Course & Paver Filler	2.0"	2.0"	2.0"	2.0"
3/4" Clean Crushed Rock or ASTM No. 57 Drain Stone or Class II Permeable Class II Baserock compacted to at least 92% relative maximum density	8.0"	11.0"	---	---
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	---	---	10.0"	13.0"
Subgrade soil scarified and compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"

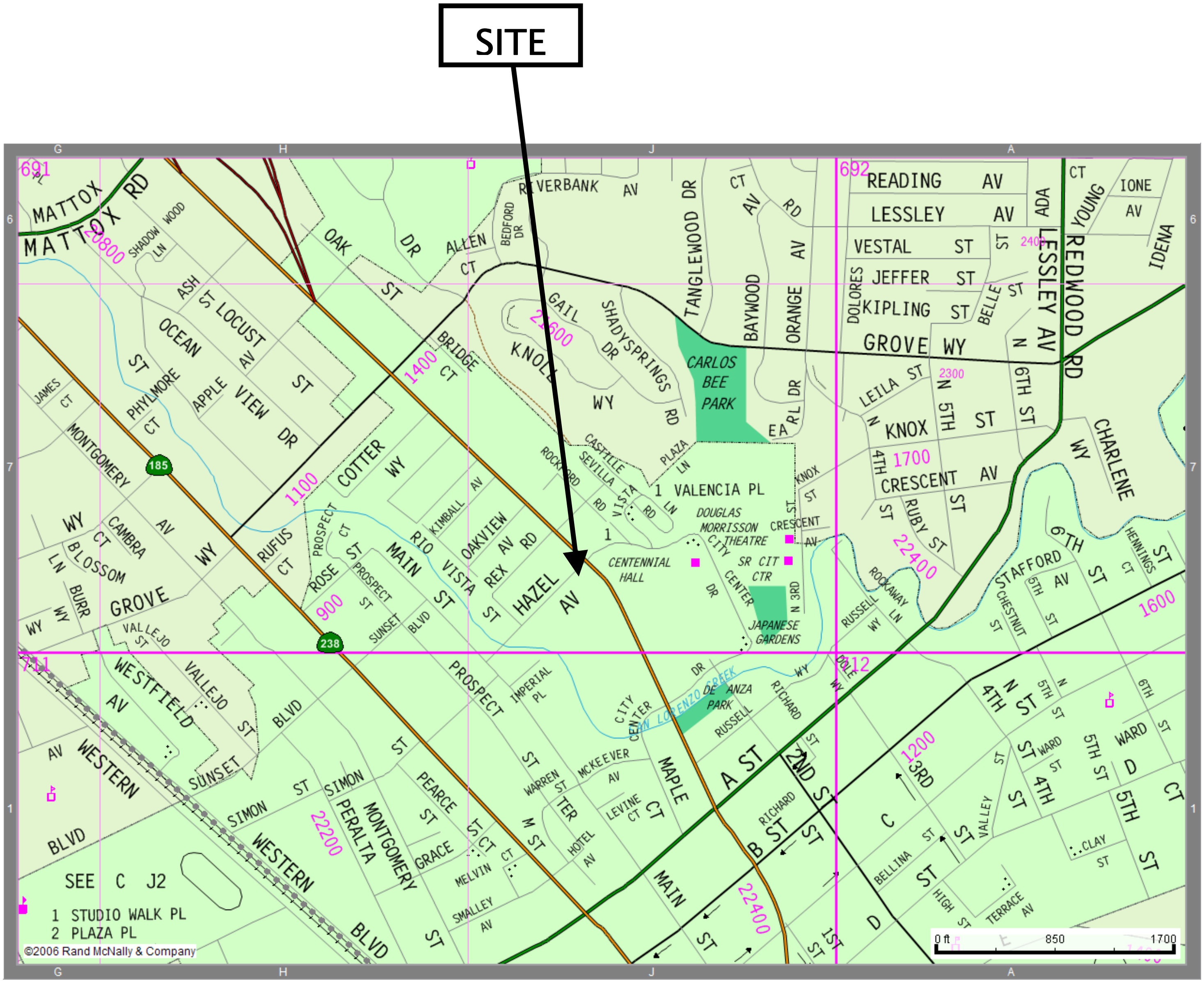
* (see next page)

* 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 Mirafi 500X should not cover subdrain system.

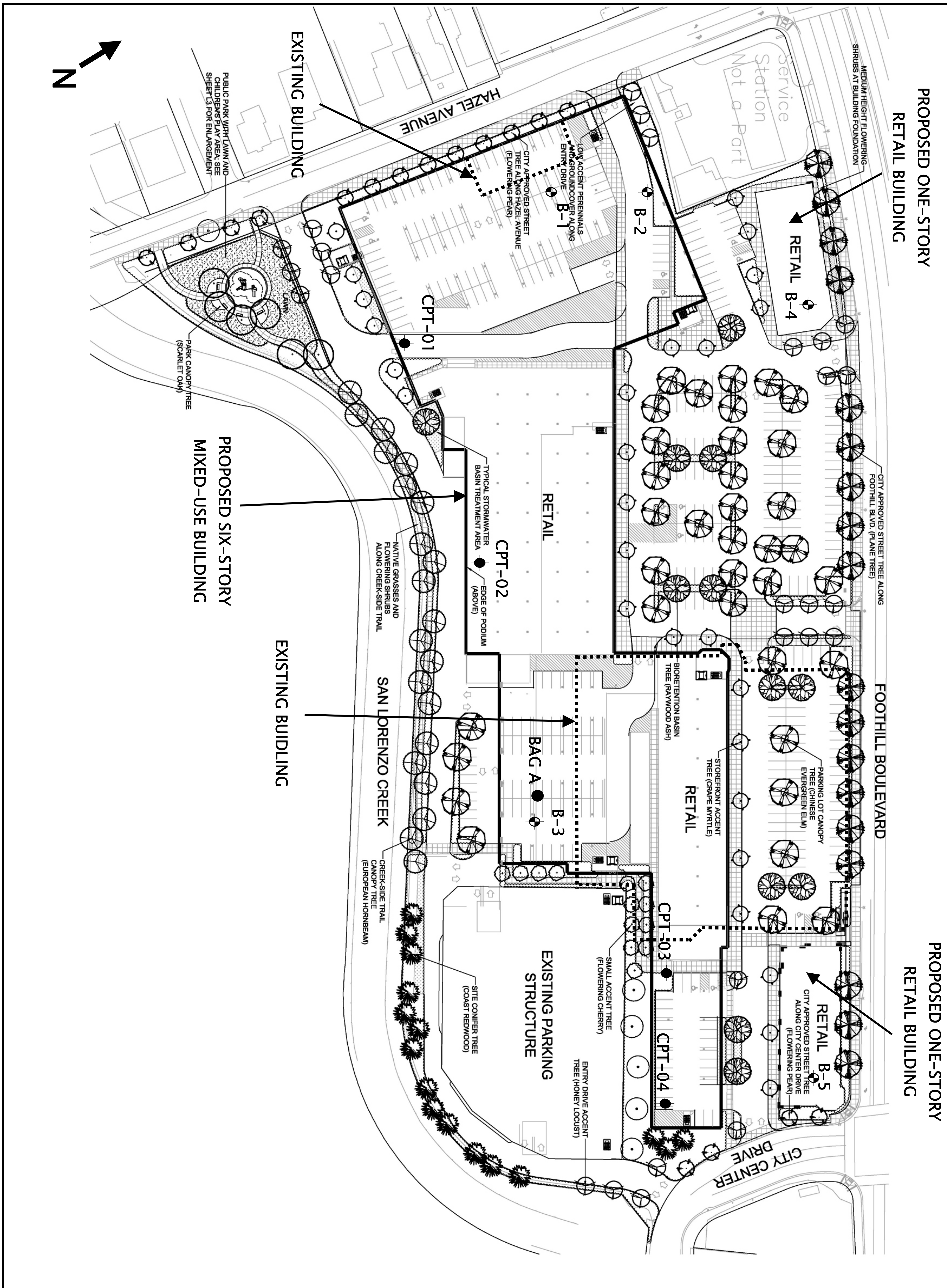
The subdrain system should consist of a 4-inch diameter perforated pipe surrounded by ¾ inch drain rock wrapped in a filter fabric. The drain rock wrapped in fabric should be at least 12 inches wide and 12 inches below the finished subgrade elevation. The drainage system should be sloped to outfall to a discharge facility.

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

FIGURES



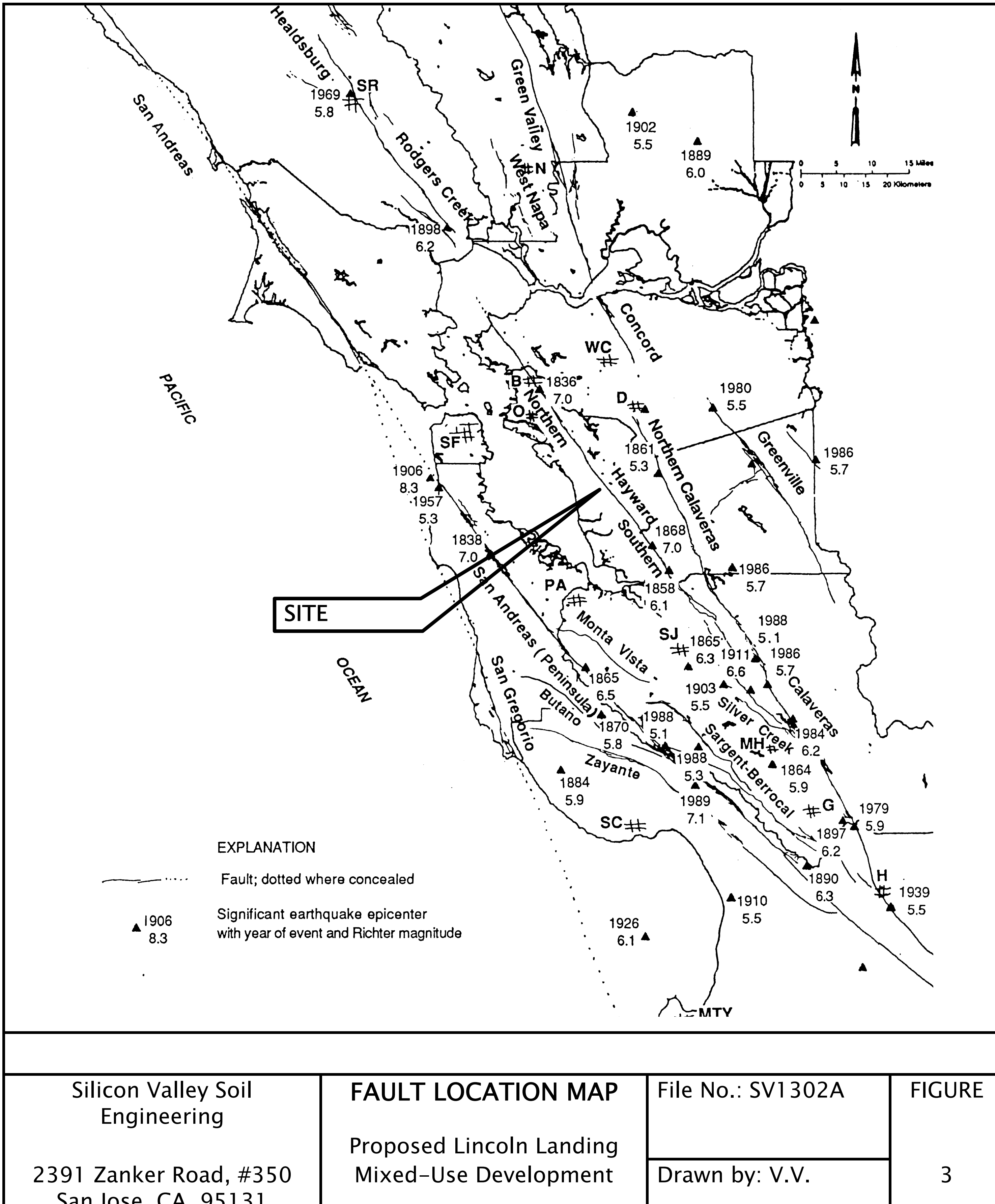
Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**VICINITY MAP** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A	FIGURE
		Drawn by: V.V.	1
		Scale: NOT TO SCALE	April 2015



NOTE: ⊕ DENOTES APPROXIMATE EXPLORATORY BORING LOCATION — DENOTES APPROXIMATE CPT SOUNDING LOCATION

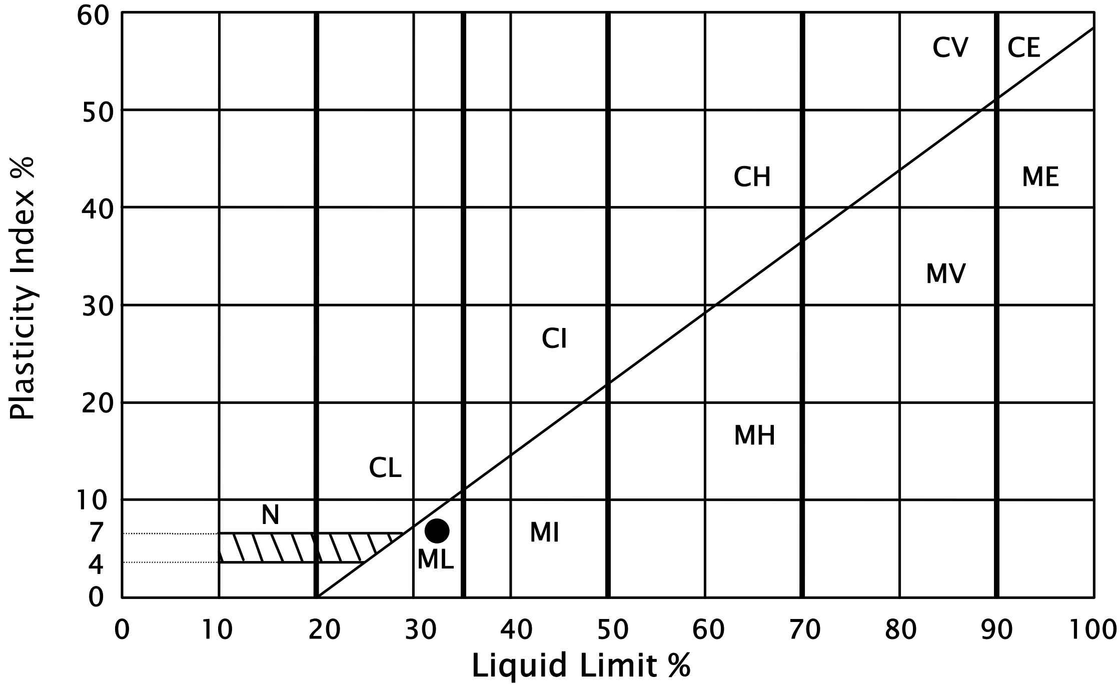
● DENOTES APPROXIMATE EXPLORATORY BAG SAMPLE LOCATION

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**SITE PLAN** Proposed Lincoln Landing Mixed-Used Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A	FIGURE
		Drawn by: V.V.	2
		Scale: 1 inch = 96 feet	April 2015



Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**FAULT LOCATION MAP** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 3 April 2015

PLASTICITY CHART

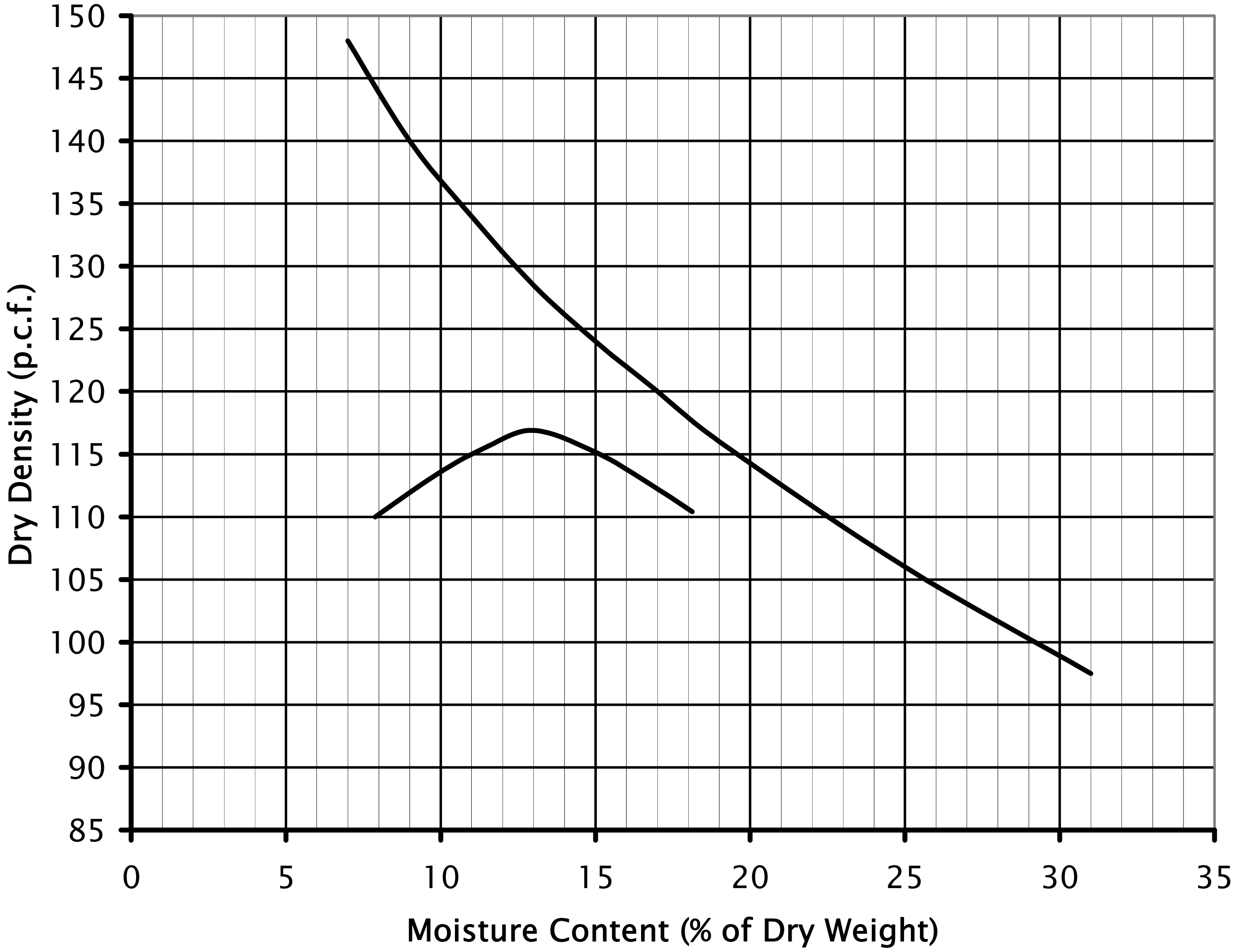


PLASTICITY DATA

Key Symbol	Hole No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	33	8	ML

*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 Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 4 April 2015



SAMPLE: A

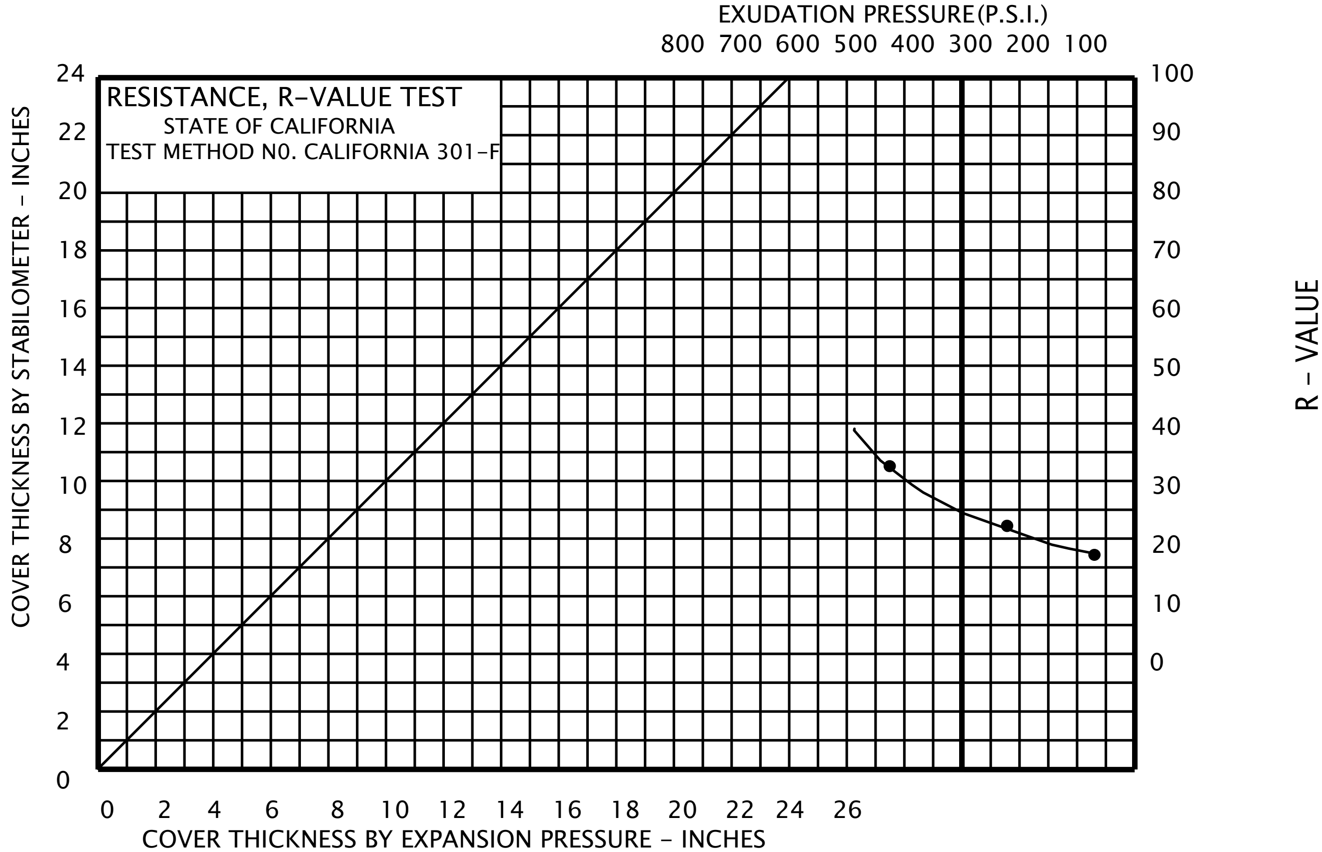
DESCRIPTION: Dark Olive Brown Sandy SILT

LABORATORY TEST PROCEDURE: ASTM D1557-12

MAXIMUM DRY DENSITY: 117.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 13.0 %

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**COMPACTION TEST A** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No. SV1302A	FIGURE
		Drawn by: V.V.	5
		Scale: NOT TO SCALE	April 2015

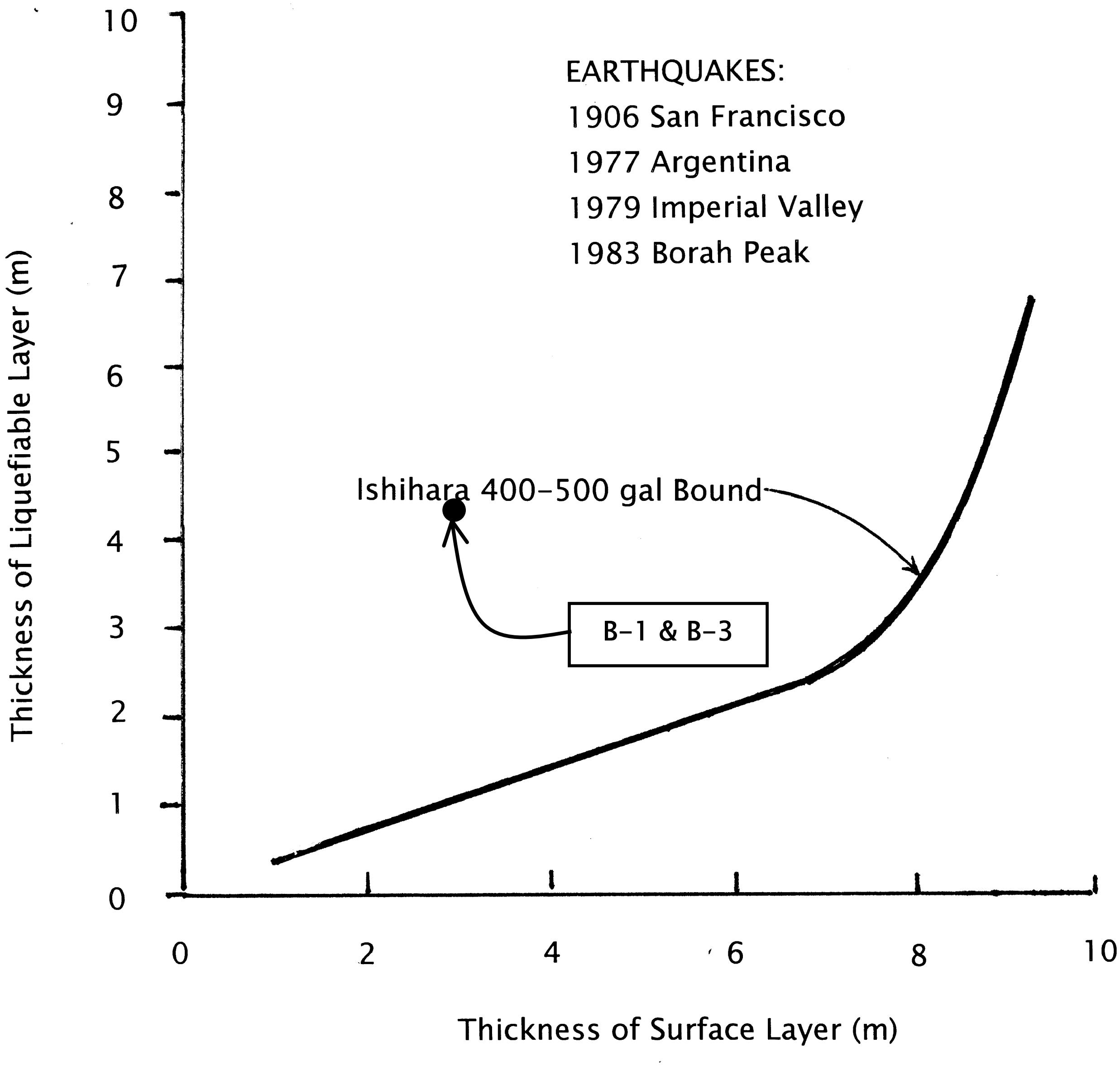


SAMPLE: A
DESCRIPTION: Dark Olive Brown Sandy SILT

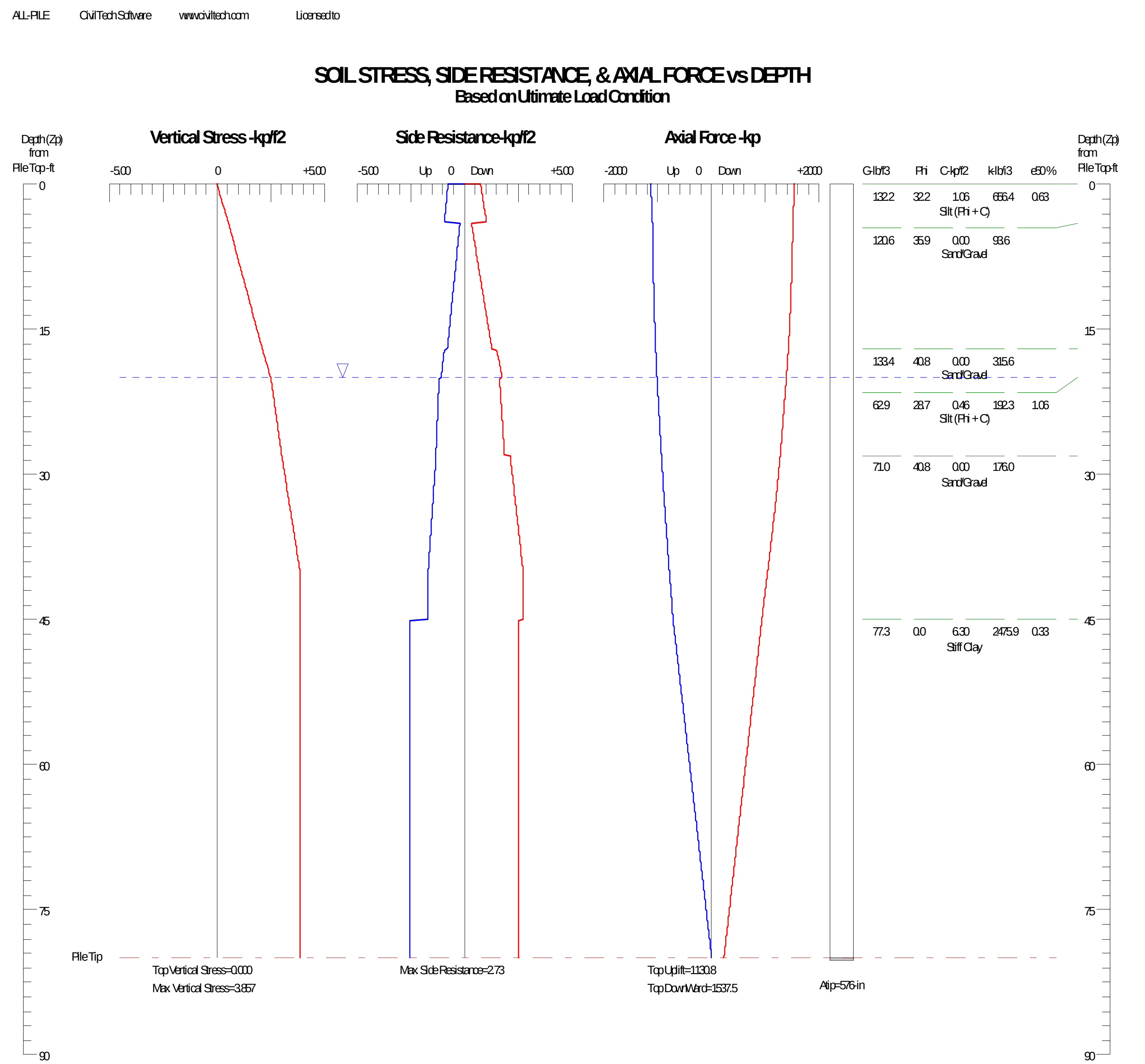
SPECIMEN	A	B	C
EXUDATION PRESSURE (P.S.I.)	105.0	251.0	449.0
EXPANSION DIAL (.0001")	9.0	14.0	20.0
EXPANSION PRESSURE (P.S.F.)	45.0	76.0	94.0
RESISTANCE VALUE, "R"	17.0	22.0	33.0
% MOISTURE AT TEST	16.2	14.7	13.6
DRY DENSITY AT TEST (P.C.F.)	116.7	118.5	121.2
R-VALUE AT 300 P.S.I. EXUDATION PRESSURE	= (24)		

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95054 (408) 988-2990	**R-VALUE TEST** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No. SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 6 April 2015

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 Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No. SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 7 April 2015



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SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 1

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**SOIL STRESS, SIDE RESISTANNCE, & AXIAL FORCE vs DEPTH** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 8 April 2015

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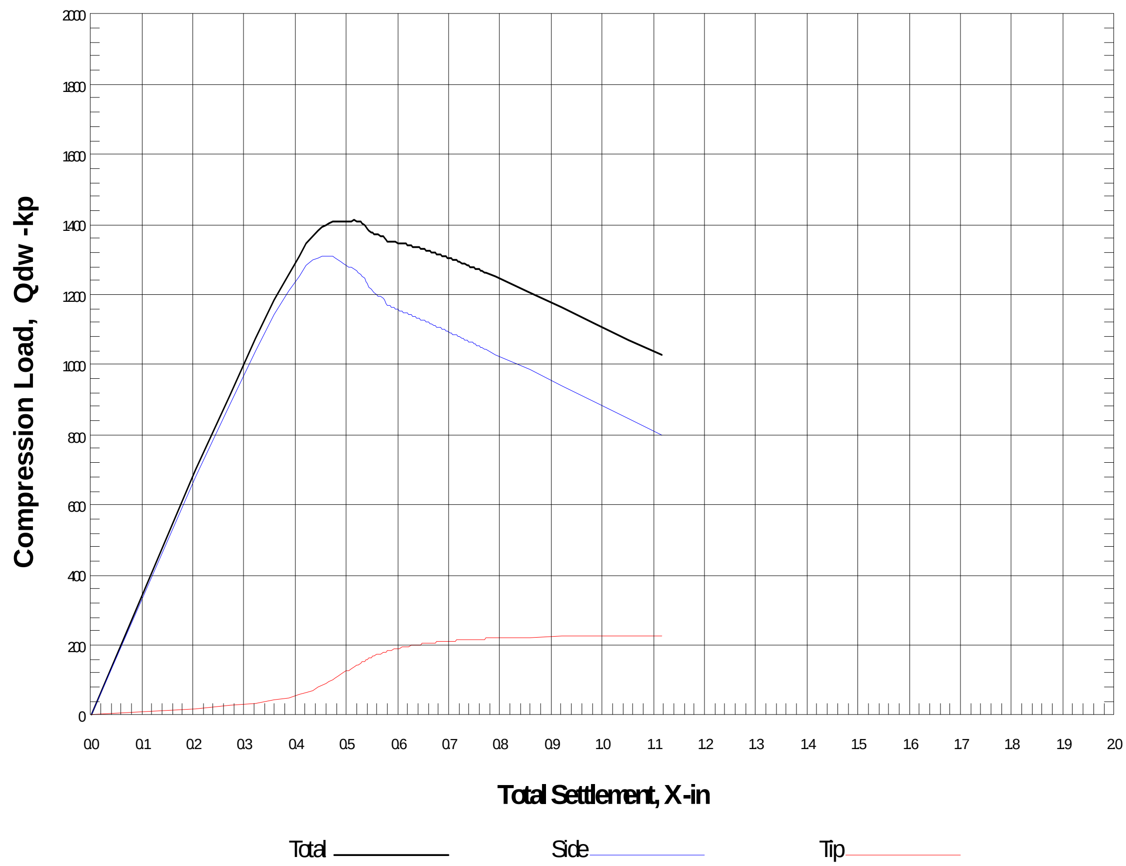
Vertical Load vs. Total Settlement



CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 1

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	VERTICAL LOAD vs TOTAL SETTLEMENTS Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 9 April 2015

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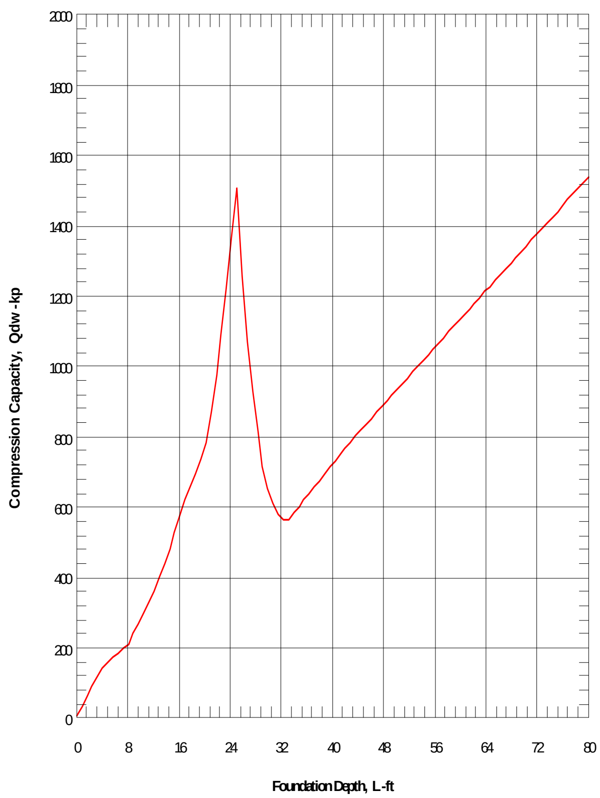
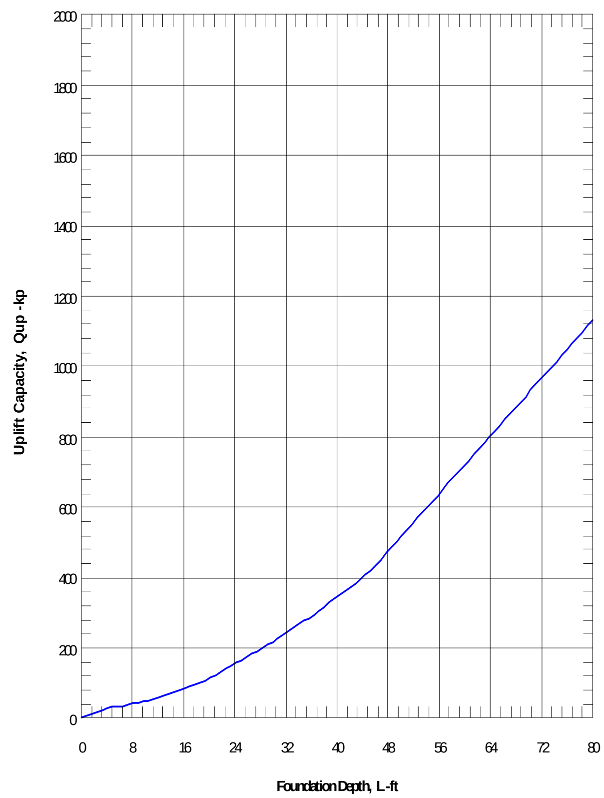
ULTIMATE CAPACITY vs FOUNDATION DEPTH





CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 1

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	ULTIMATE CAPACITY vs FOUNDATION DEPTH Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 10 April 2015

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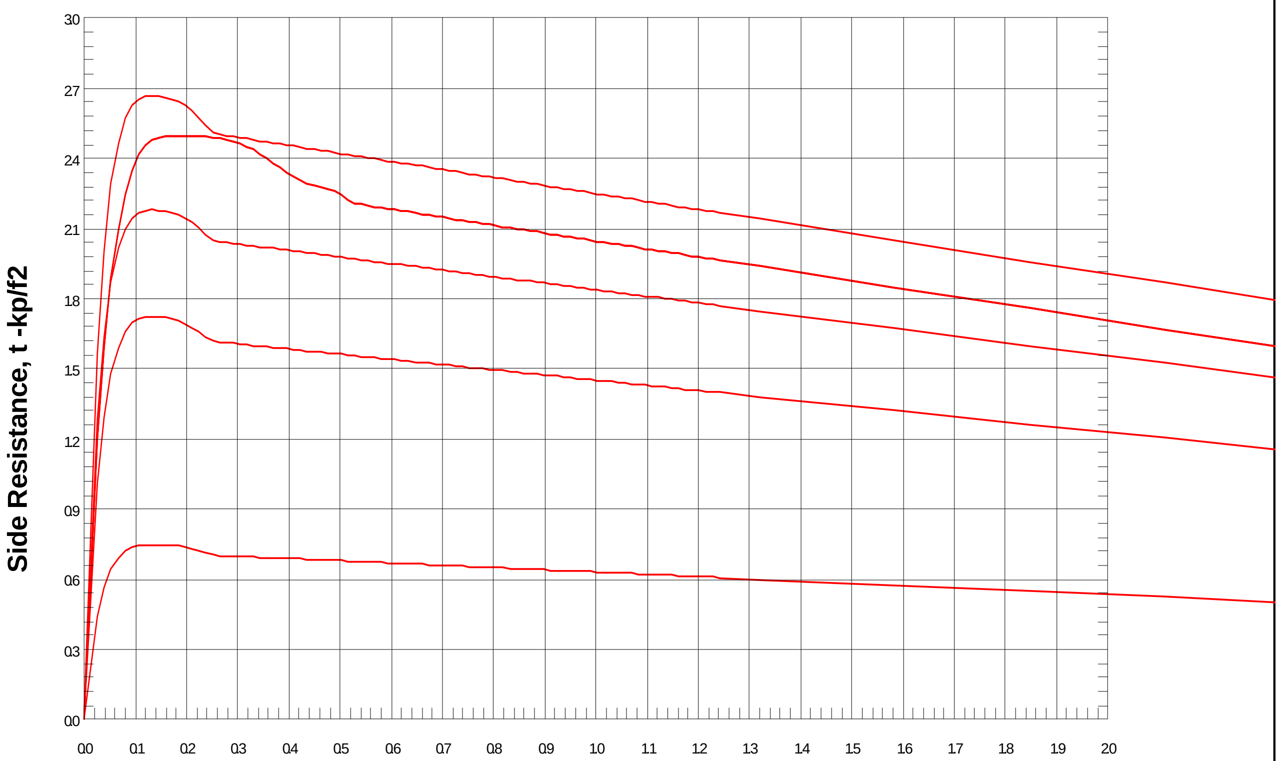
Side Resistance vs. Relative Movement between Soil and Shaft (t-z)



Soil Depth (Zs): 10.0, 20.0, 30.0, 40.0, 50.0, 60.0, 70.0 -ft

CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA **Figure 1**

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	SIDE RESISTANCE vs RELATIVE MOVEMENT BETWEEN SOIL AND SHAFT Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 11 April 2015

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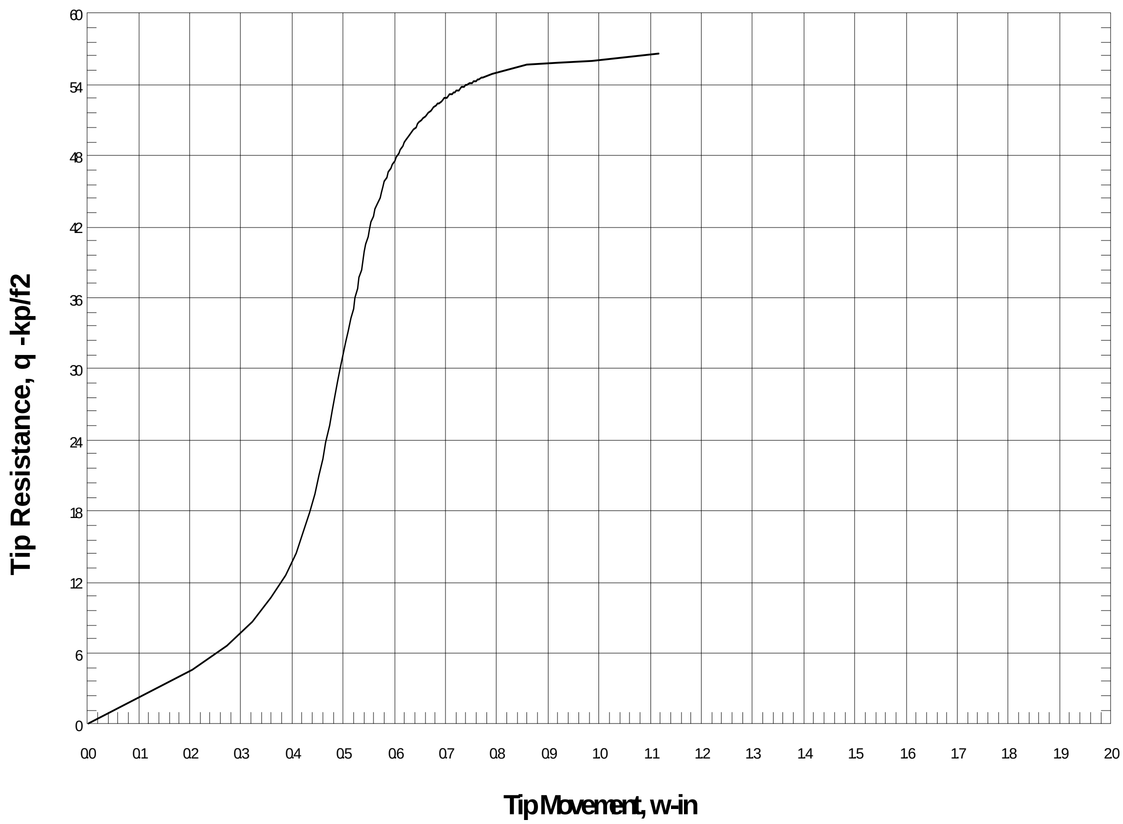
Tip Resistance vs. Tip Movement (q-w)



CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA **Figure 1**

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**TIP RESISTANCE VS TIP MOVEMENT** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No. SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 12 April 2015

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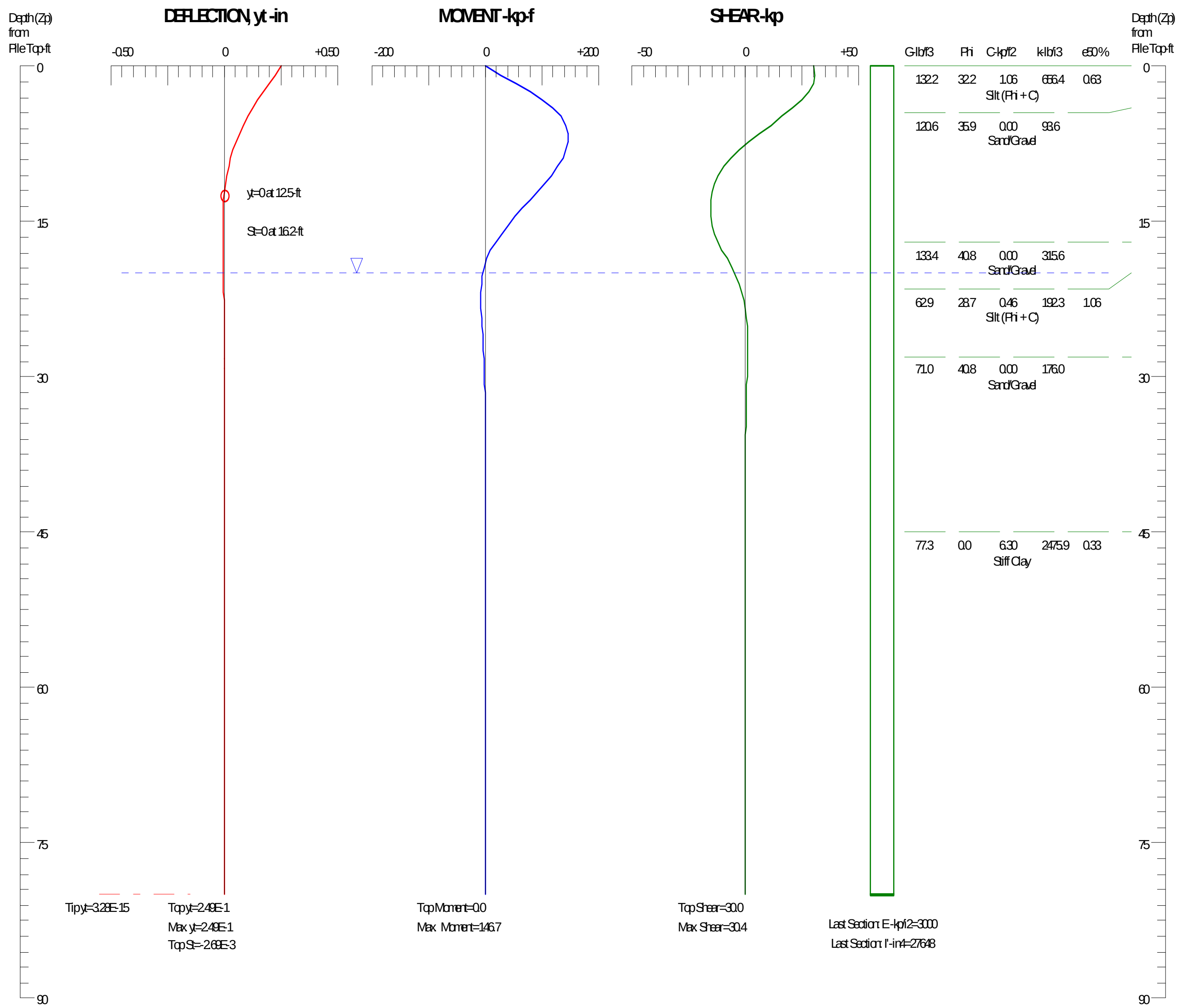
PILE DEFLECTION & FORCE vs DEPTH
Single Pile, Khead=2, Kbc=1



CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA

Figure 2

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	PILE DEFLECTION & FORCE vs DEPTH (FREE END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 13 April 2015

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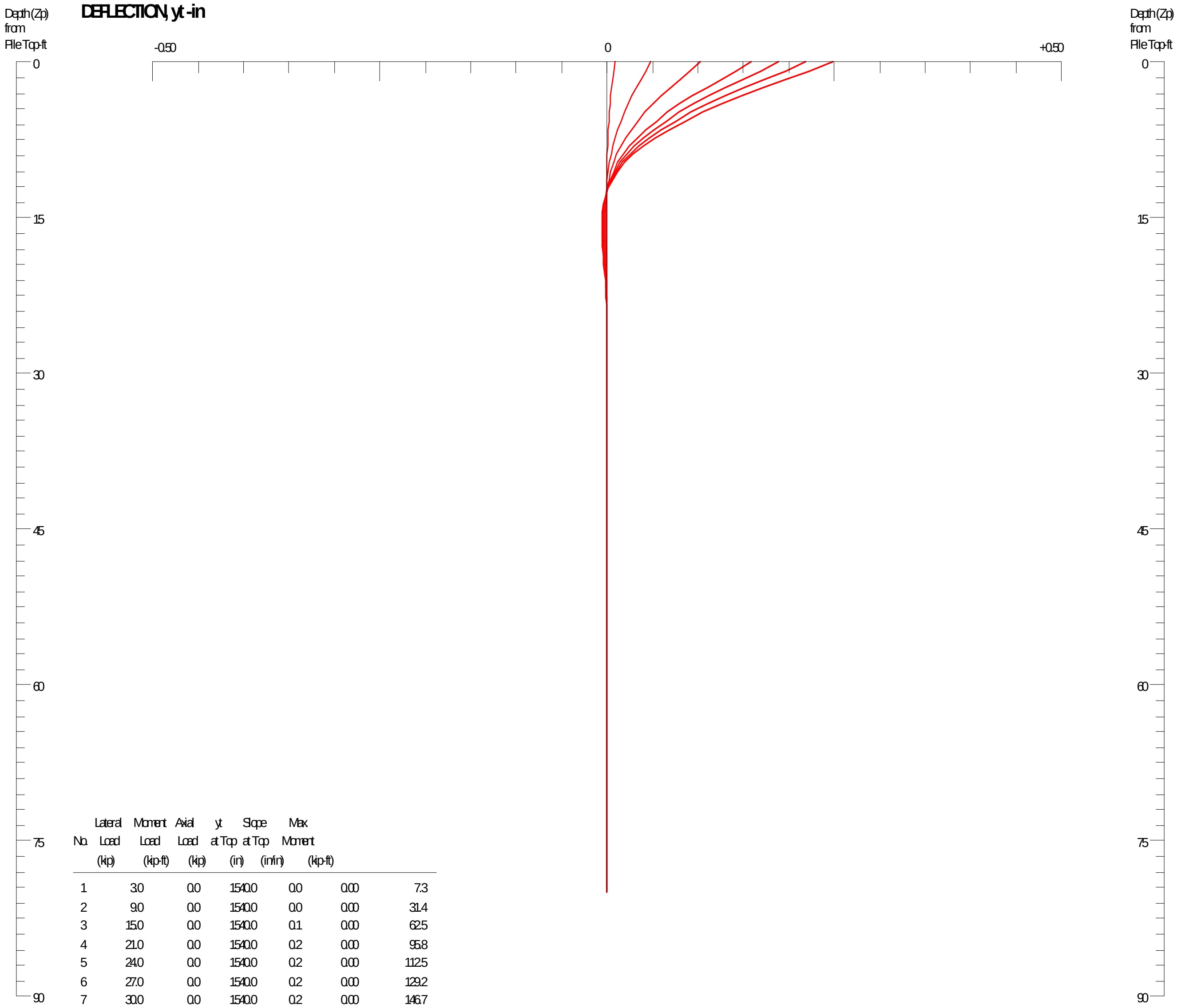
PILE DEFLECTION vs LOADING

Single Pile, Khead=2, Kbc=1



No.	Lateral Load (kip)	Moment Load (kip-ft)	Axial Load (kip)	yt at Top (in)	Slope at Top (in/in)	Max Moment (kip-ft)
1	3.0	0.0	1540.0	0.0	0.00	7.3
2	9.0	0.0	1540.0	0.0	0.00	31.4
3	15.0	0.0	1540.0	0.1	0.00	62.5
4	21.0	0.0	1540.0	0.2	0.00	95.8
5	24.0	0.0	1540.0	0.2	0.00	112.5
6	27.0	0.0	1540.0	0.2	0.00	129.2
7	30.0	0.0	1540.0	0.2	0.00	146.7

CivilTech Software

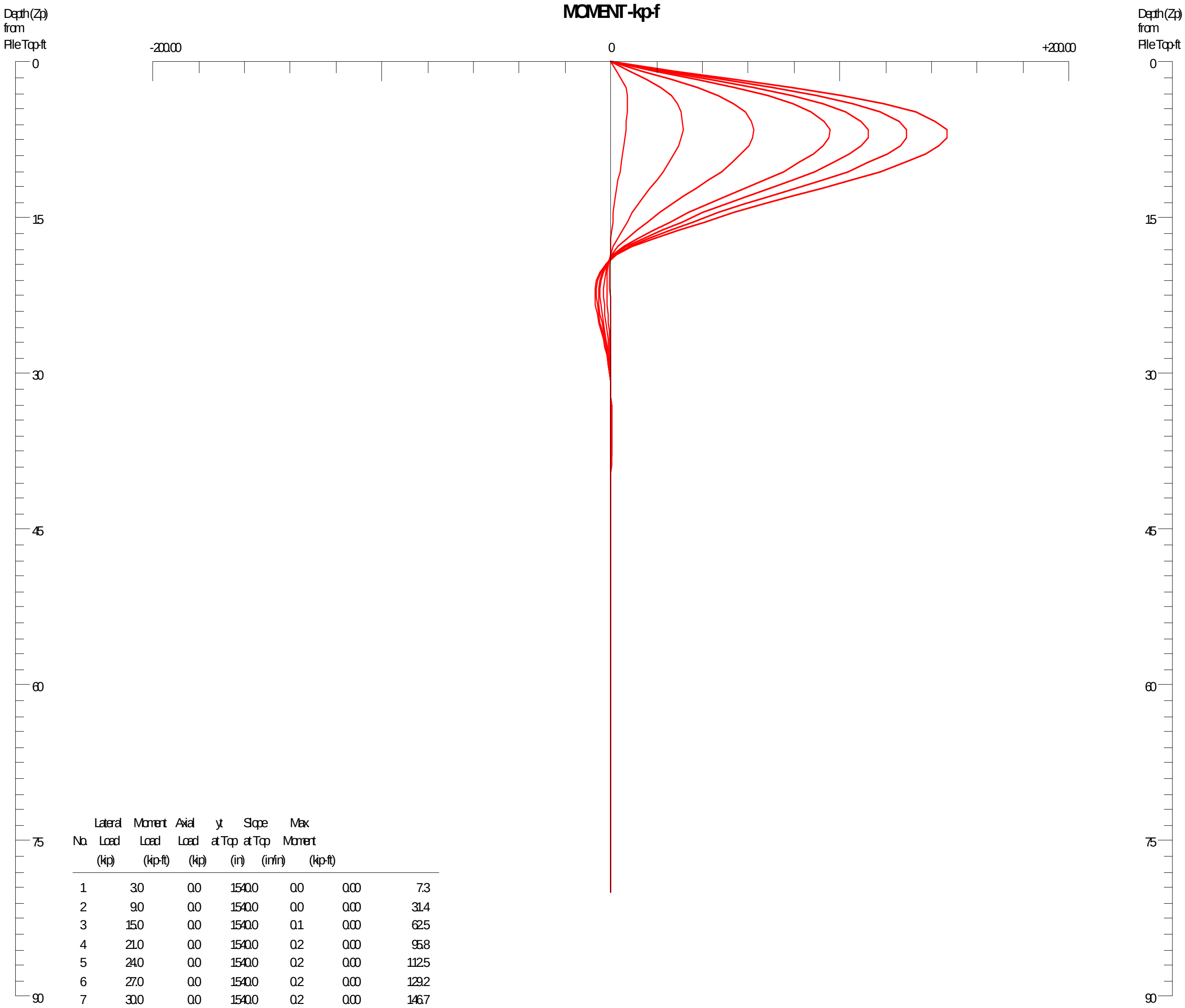
SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA   **Figure 2**

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**PILE DEFLECTION vs LOADING (FREE END)** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 14 April 2015

ALL-PILE CivilTech Software www.civiltech.com Licensed to

PILE MOMENT vs LOADING

Single Pile, Khead=2, Kbc=1



No.	Lateral Load (kip)	Moment Load (kip-ft)	Axial Load (kip)	yt at Top (in)	Slope at Top (in/in)	Max Moment (kip-ft)
1	3.0	0.0	1540.0	0.0	0.00	7.3
2	9.0	0.0	1540.0	0.0	0.00	31.4
3	15.0	0.0	1540.0	0.1	0.00	62.5
4	21.0	0.0	1540.0	0.2	0.00	95.8
5	24.0	0.0	1540.0	0.2	0.00	112.5
6	27.0	0.0	1540.0	0.2	0.00	129.2
7	30.0	0.0	1540.0	0.2	0.00	146.7

CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 2

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	PILE MOMENT vs LOADING (FREE END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 15 April 2015

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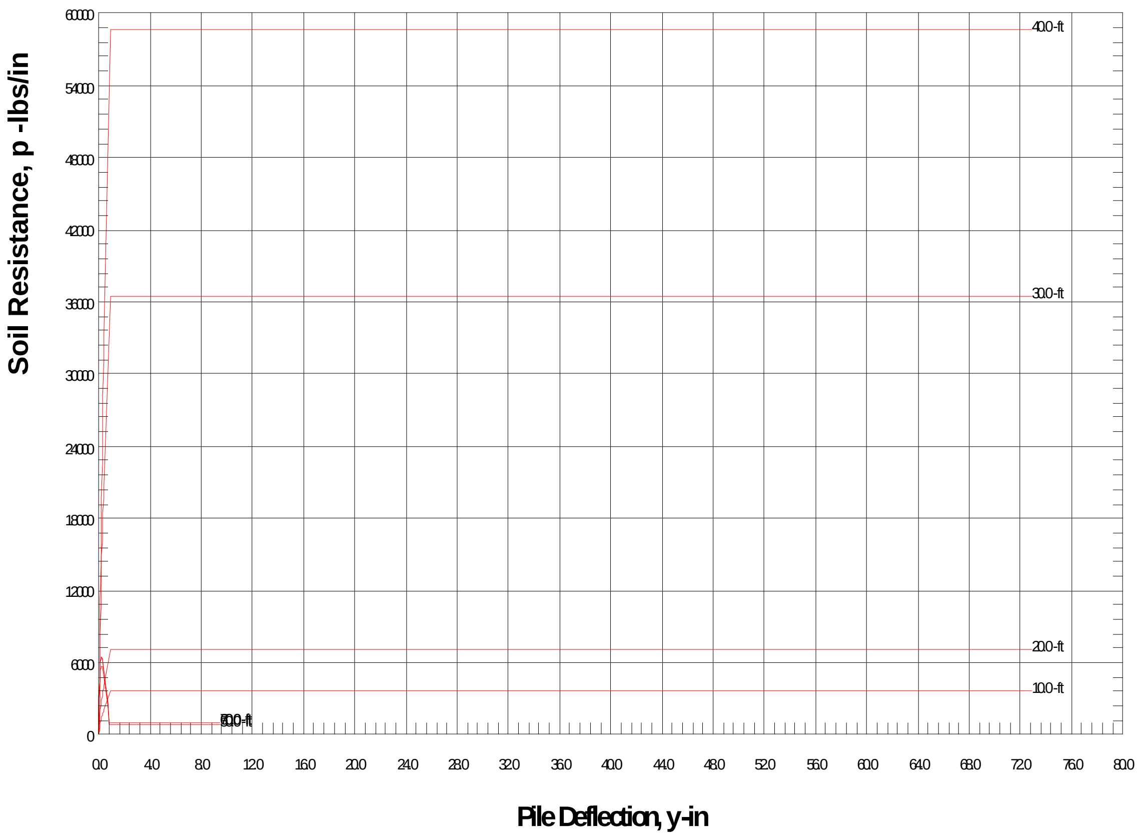
Soil Resistance vs. Pile Deflection (p-y)



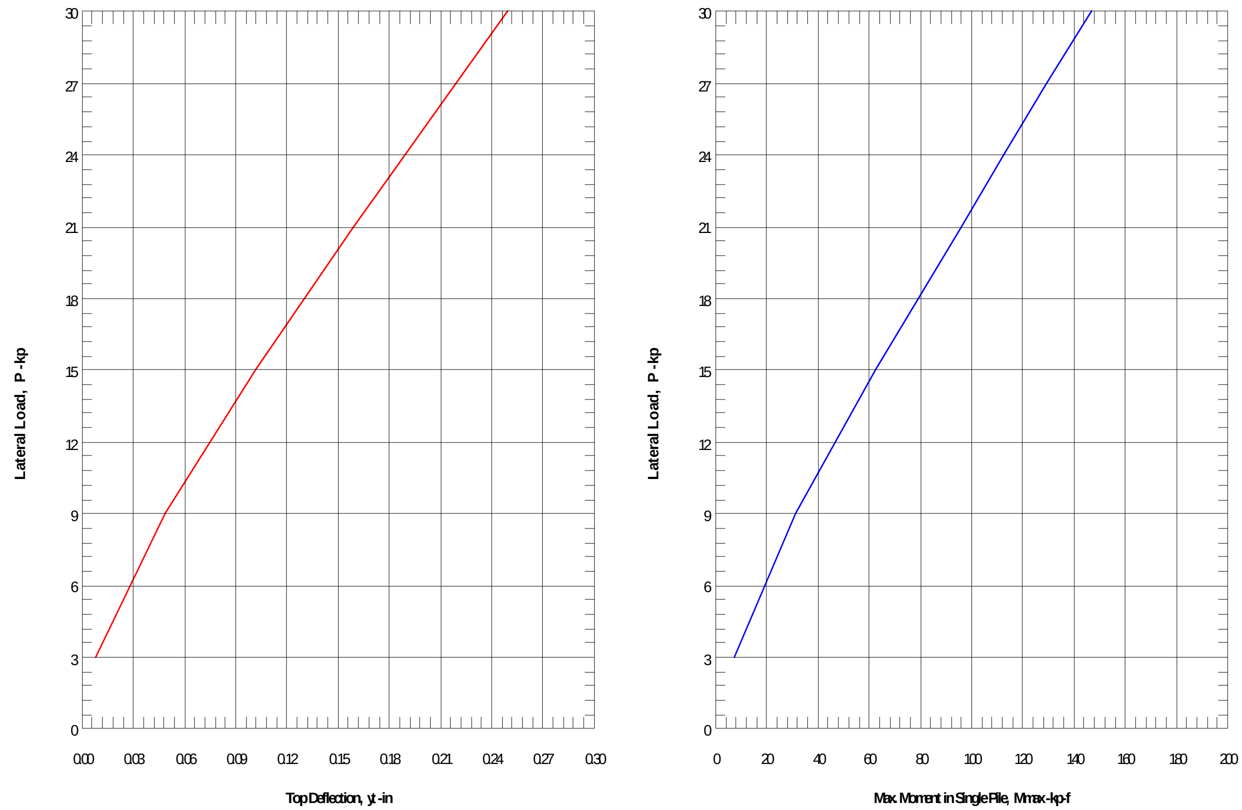
Soil Depth (Zs): 10.0, 20.0, 30.0, 40.0, 50.0, 60.0, 70.0 -ft

CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA   **Figure 2**

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	SOIL RESISTANCE VS PILE DEFLECTION (FREE END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A	FIGURE
		Drawn by: V.V.	16
		Scale: NOT TO SCALE	April 2015

LATERAL LOAD vs DEFLECTION & MAX. MOMENT



CivilTech Software

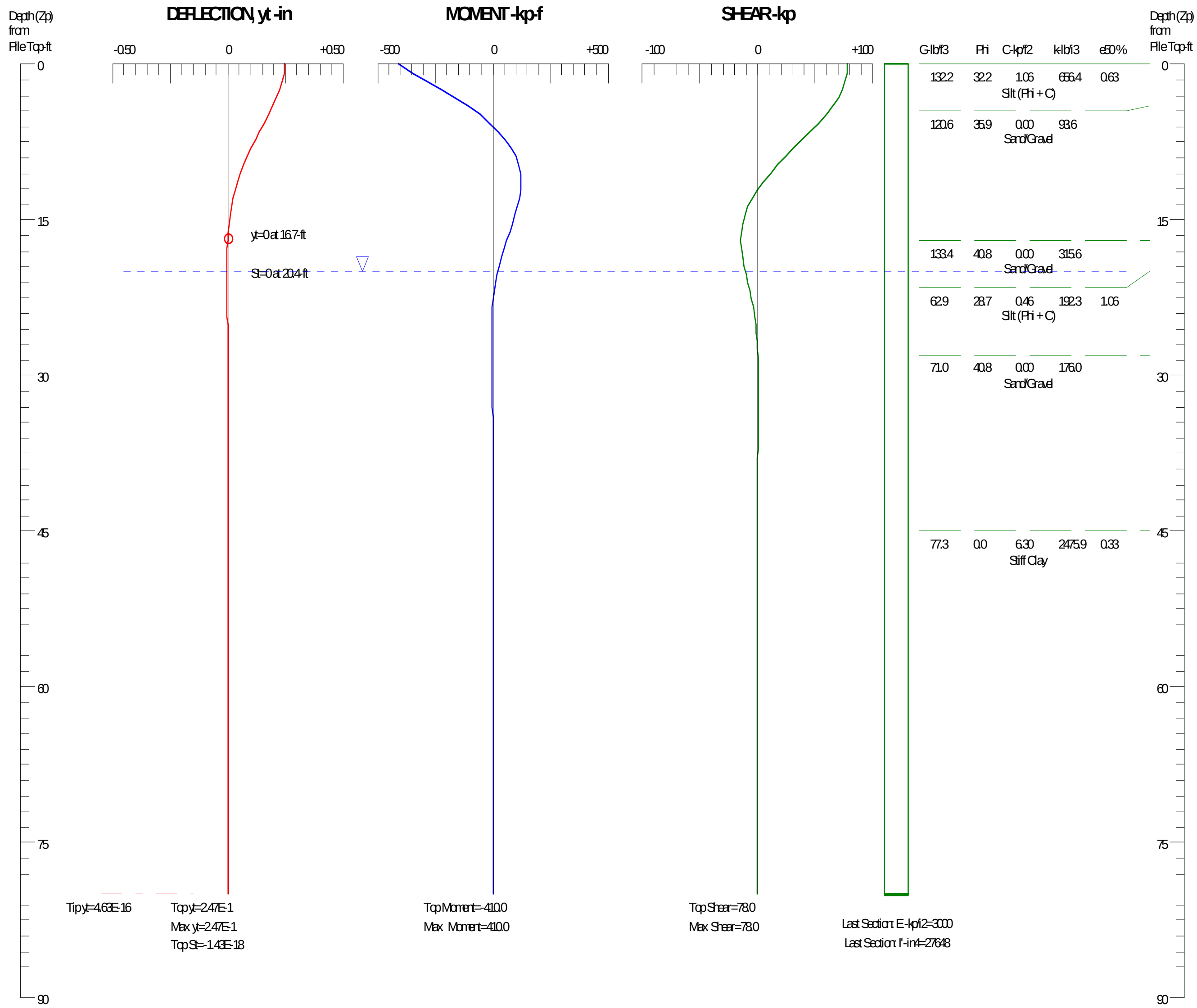
SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 2

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	LATERAL LOAD vs DEFLECTION & MAX. MOMENT (FREE END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No. SV1302A	FIGURE
		Drawn by: V.V.	17
		Scale: NOT TO SCALE	April 2015

ALL-PILE CivilTech Software www.civiltech.com Licensed to

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=5, Kbc=2



CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA **Figure 2**

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	PILE DEFLECTION & FORCE vs DEPTH (FIXED END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 18 April 2015

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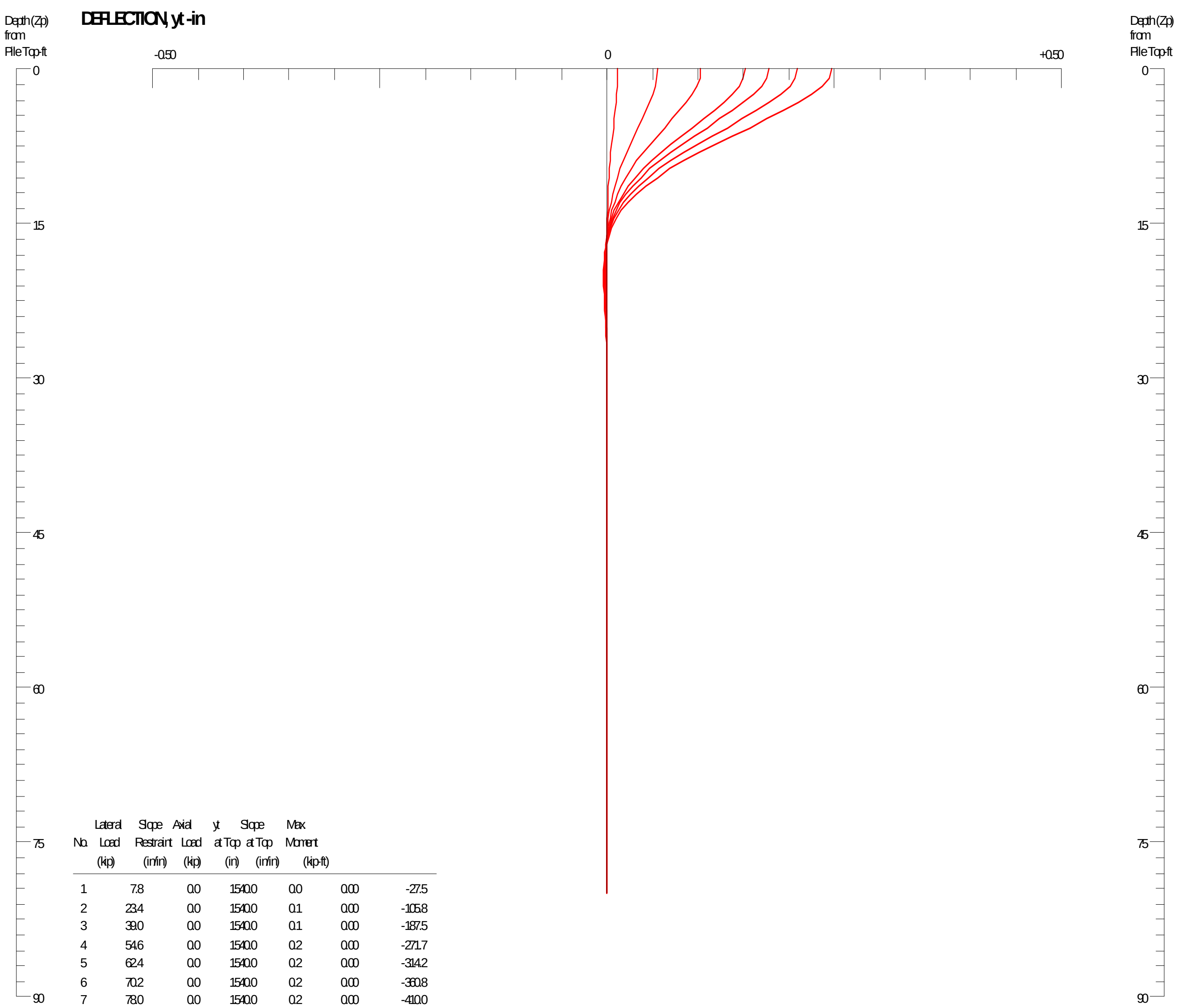
PILE DEFLECTION vs LOADING

Single Pile, Khead=5, Kbc=2



No.	Lateral Load (kip)	Slope Restraint (in/in)	Axial Load (kip)	yt at Top (in)	Slope at Top (in/in)	Max Moment (kip-ft)
1	7.8	0.0	1540.0	0.0	0.00	-27.5
2	23.4	0.0	1540.0	0.1	0.00	-105.8
3	39.0	0.0	1540.0	0.1	0.00	-187.5
4	54.6	0.0	1540.0	0.2	0.00	-271.7
5	62.4	0.0	1540.0	0.2	0.00	-314.2
6	70.2	0.0	1540.0	0.2	0.00	-360.8
7	78.0	0.0	1540.0	0.2	0.00	-410.0

CivilTech Software

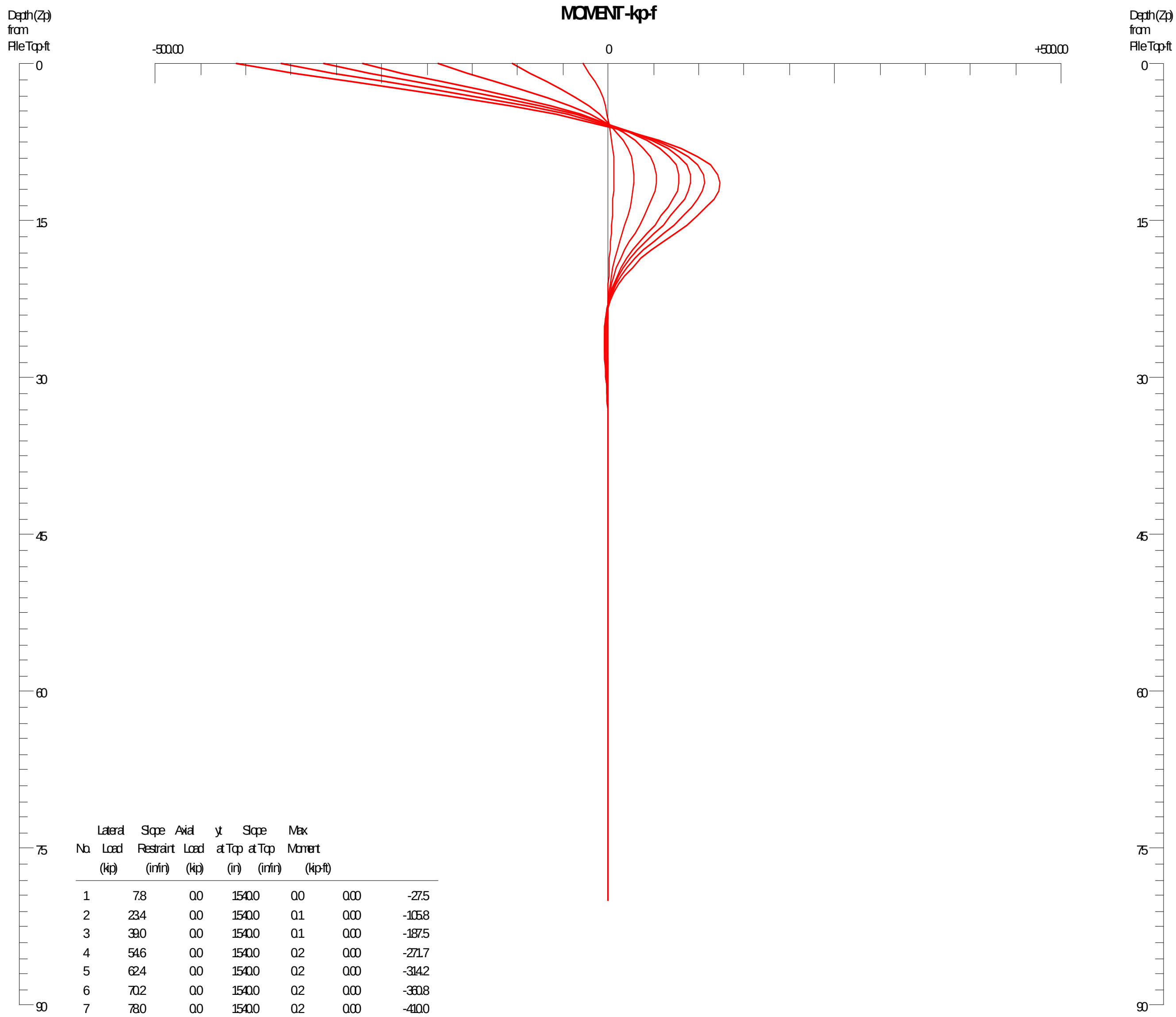
SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA

Figure 2

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	**PILE DEFLECTION vs LOADING (FIXED END)** Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 19 April 2015

ALL-PILE CivilTech Software www.civiltech.com Licensed to

PILE MOMENT vs LOADING
Single Pile, Khead=5, Kbc=2



No.	Lateral Load (kip)	Slope Restraint (in/in)	Axial Load (kip)	yt at Top (in)	Slope at Top (in/in)	Max Moment (kip-ft)
1	7.8	0.0	1540.0	0.0	0.00	-27.5
2	23.4	0.0	1540.0	0.1	0.00	-105.8
3	39.0	0.0	1540.0	0.1	0.00	-187.5
4	54.6	0.0	1540.0	0.2	0.00	-271.7
5	62.4	0.0	1540.0	0.2	0.00	-314.2
6	70.2	0.0	1540.0	0.2	0.00	-360.8
7	78.0	0.0	1540.0	0.2	0.00	-410.0

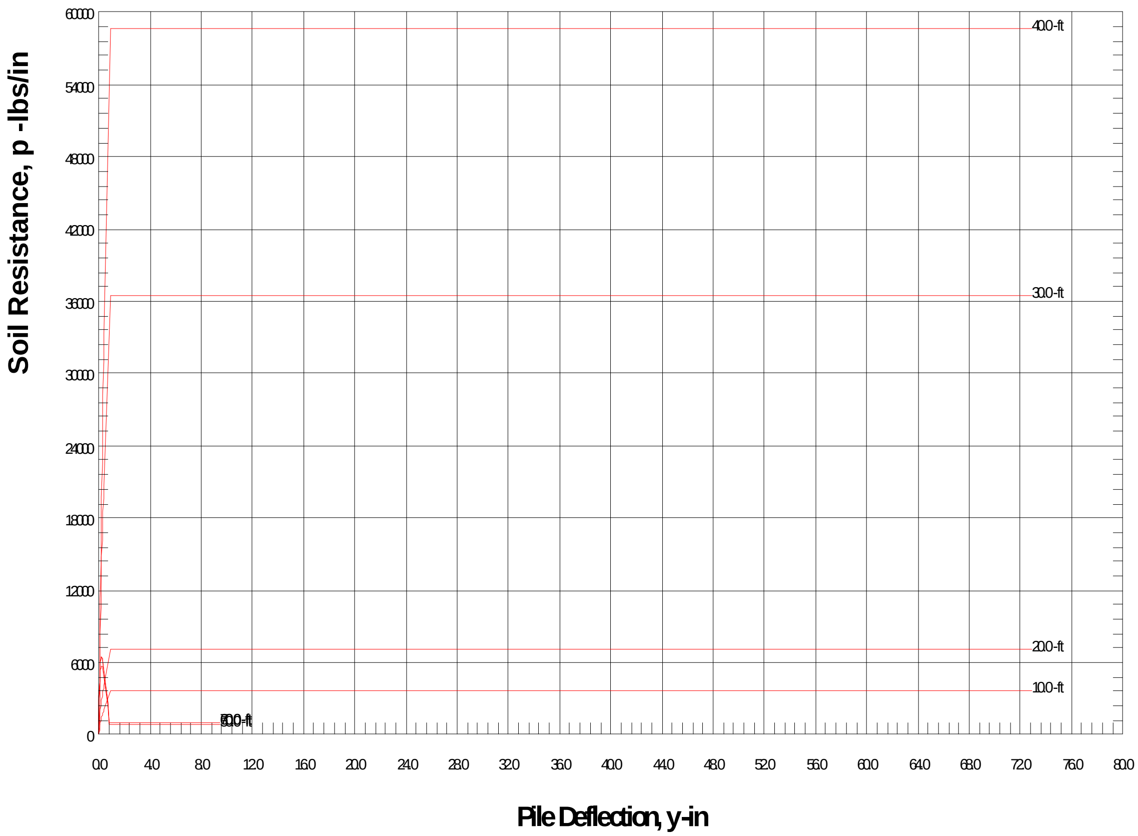
CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 2

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	PILE MOMENT vs LOADING (FIXED END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 20 April 2015

ALL-PILE CivilTech Software www.civiltech.com Licensed to

Soil Resistance vs. Pile Deflection (p-y)



Soil Depth (Zs): 10.0, 20.0, 30.0, 40.0, 50.0, 60.0, 70.0-ft

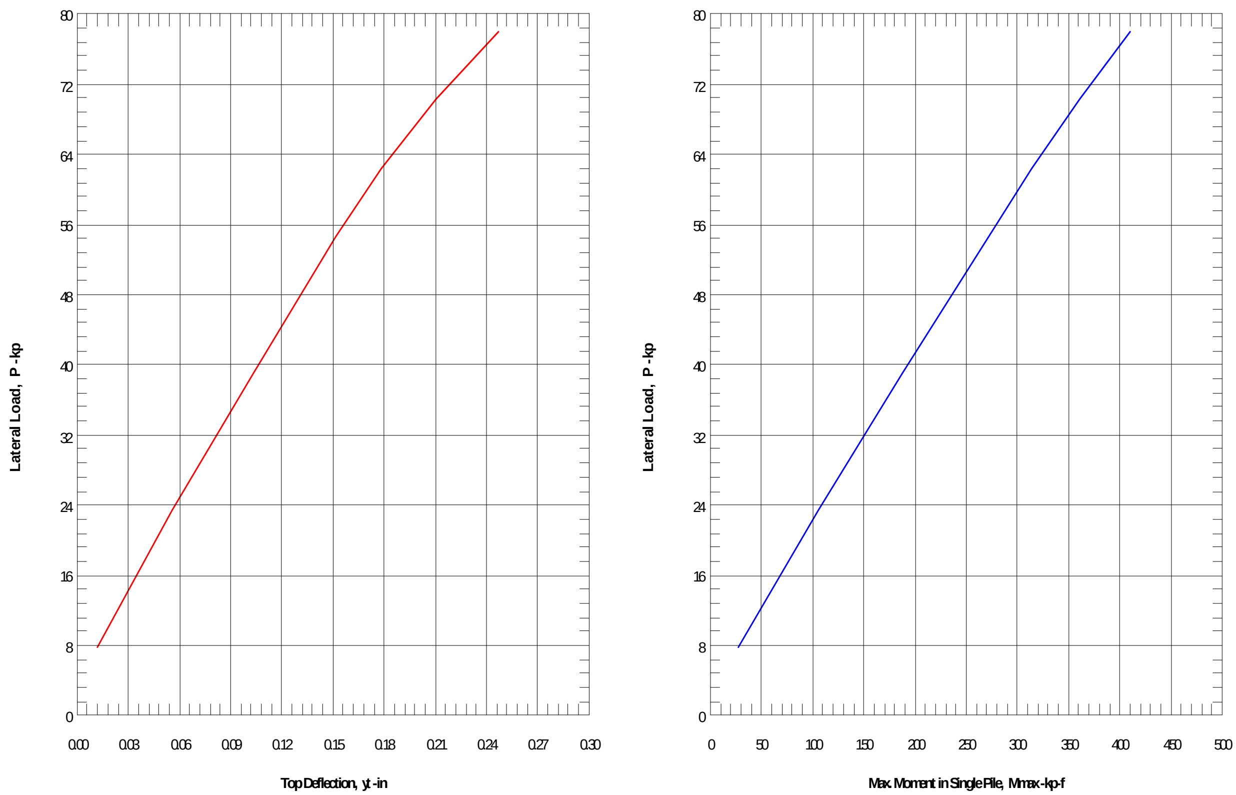
CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA **Figure 2**

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	SOIL RESISTANCE VS PILE DEFLECTION (FIXED END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No.: SV1302A	FIGURE
		Drawn by: V.V.	21
		Scale: NOT TO SCALE	April 2015

ALL-PILE CivilTech Software www.civiltech.com Licensed to

LATERAL LOAD vs DEFLECTION & MAX. MOMENT



CivilTech Software

SV1302A - Proposed Lincoln Landing Mixed-Use Development
22301 Foothill Blvd., Hayward, CA Figure 2

Silicon Valley Soil Engineering 2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	LATERAL LOAD vs DEFLECTION & MAX. MOMENT (FIXED END) Proposed Lincoln Landing Mixed-Use Development 22301 Foothill Boulevard Hayward, California	File No. SV1302A Drawn by: V.V. Scale: NOT TO SCALE	FIGURE 22 April 2015

APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

KEY TO LOG OF BORING

EXPLORATORY BORING LOGS (B-1 THROUGH B-5)

CONE PENETRATION TESTS (CPT-01 THROUGH CPT-04)

CONE PENETRATION TEST PROCEDURE

ALAMEDA COUNTY PUBLIC WORKS DRILLING PERMIT

LIQUEFACTION ANALYSIS

COMPUTER PRINTOUTS FOR PILE VERTICAL AND LATERAL ANALYSIS (ALLPILE 7)

PILE SPECIFICATIONS

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.

File No. SV1302A

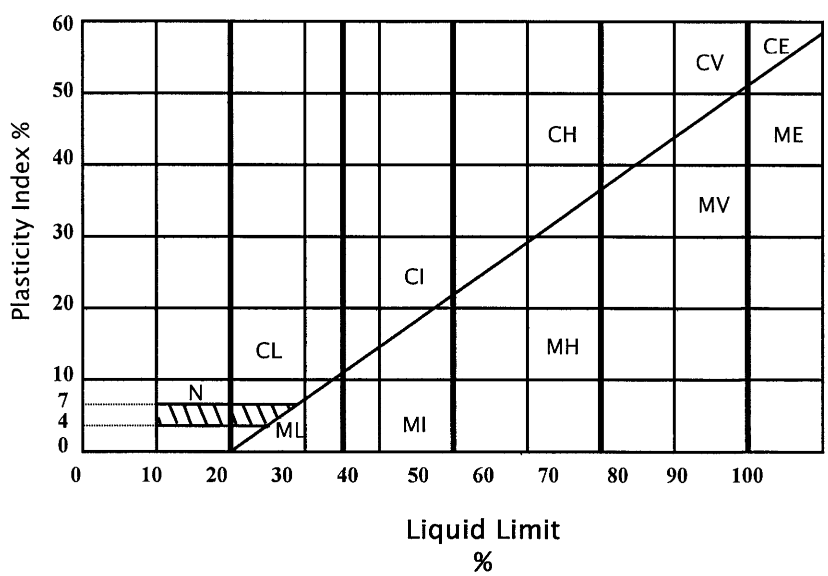
METHOD OF SOIL CLASSIFICATION CHART

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



Project: Proposed Lincoln Landing Mixed-Use Dvelopment **Project Location:** 22301 Foothill Boulevard Hayward, California **Project Number:** SV1302A	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	# Key to Log of Boring **Sheet 1 of 1**

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

COLUMN DESCRIPTIONS

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

FIELD AND LABORATORY TEST ABBREVIATIONS

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

MATERIAL GRAPHIC SYMBOLS

Asphaltic Concrete (AC)
Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
SILTY CLAY (CL-ML)
Poorly graded GRAVEL (GP)
Aggregate Base (AB)
SILT, SILT w/SAND, SANDY SILT (ML)
Poorly graded SAND (SP)

TYPICAL SAMPLER GRAPHIC SYMBOLS

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

OTHER GRAPHIC SYMBOLS

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

GENERAL NOTES

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

Project: Proposed Building **Project Location:** 22301 Foothill Boulevard Hayward, California **Project Number:** SV1302	**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 **09/26/14**	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 **23 feet (09/26/14)**	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 – 0.3				Asphalt		4.0 inches of Asphalt Concrete (AC)						
0.3 – 1						8.0 inches of Aggregate Base (AB)						
1 – 3		1-1	7	CL-ML		Medium Brown Clayey SILT/Silty CLAY Moist, stiff	19.0	97.6	0.8	25		
3 – 4				SP		Tan Brown Silty SAND Damp, loose SAND: Medium grained, poorly graded						
4 – 8		1-2	15	ML		Medium Olive Brown Clayey SILT Moist, stiff	17.8	101.6				
8 – 17				SP		Tan Brown SAND Damp, loose SAND: Medium grained, poorly graded						
10		1-3	9				9.1	94.4				<12
15		1-4	15				7.0	102.1				<12
17 – 22				SP		Medium Brown Gravelly SAND Damp, medium dense SAND: Medium grained, poorly graded						
20		1-5	17				7.4	108.1				<12
22 – 27				SC-CL		Bluish Gray Clayey SAND/Sandy CLAY Moist, stiff Stabilized after drilling completion						
25		1-6	12				24.4	102.7			41	19
27 – 30				SP		Brown Gravelly SAND Wet, dense SAND: Medium grained, poorly graded First encountered						

Project: Proposed Building **Project Location:** 22301 Foothill Boulevard Hayward, California **Project Number:** SV1302	**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	48	SP		Brown Gravelly SAND Wet, dense SAND: Medium grained, poorly graded	8.3	102.5				
32				CL		Light Olive Brown Silty CLAY Moist, very stiff						
35		1-8	29				28.8	93.1			43	21
40		1-9	47				23.9	103.0			41	20
45		1-10	20				34.4	88.7			44	22
50		1-11	31				31.8	90.2			40	21
51.5						Boring terminated at 51.5 feet						
55												
60												
65												

Project: Proposed Building **Project Location:** 22301 Foothill Boulevard Hayward, California **Project Number:** SV1302	**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 **09/26/14**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **21.5 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				Asphalt		4.0 inches of Asphalt Concrete (AC)						
						10.0 inches of Aggregate Base (AB)						
				CL-ML		Medium Brown Clayey SILT/Silty CLAY Moist, stiff						
4		2-1	9				18.7	98.1				
				ML		Medium Olive Brown Clayey SILT Moist, stiff						
5		1-2	16				17.5	102.3				
8				SP		Tan Brown SAND Damp, loose SAND: Medium grained, poorly graded						
10		1-3	11				8.8	95.2				
15		2-4	14				7.4	104.6				
				SP		Medium Brown Gravelly SAND Damp, medium dense SAND: Medium grained, poorly graded						
20		2-5	20				7.9	106.9				
						Boring terminated at 21.5 feet						
25												
30												

Project: Proposed Lincoln Landing Mixed-Use Dvelopment Project Location: 22301 Foothill Boulevard Hayward, California Project Number: SV1302A	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-3 **Sheet 1 of 3**

Date(s) Drilled **04/01/15**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **80.0 feet**
		Approximate Surface Elevation **103 feet**
Groundwater Level and Date Measured **20 feet (04/01/15)**	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0				Asphalt		3.0 inches Asphalt Concrete (AC)						
0.25						9.0 inches of Agrregate Base (AB)						
1				ML		Dark Olive Brown Sandy SILT Moist, stiff						
		3-1	17				17.8	104.4	0	22		
4		3-2	15	SP		Dark Olive Brown Silty SAND Damp, medium dense Medium grained, poorly graded	18.8	106.0				
5												
		3-3	23				11.9	109.9	0	33		
10						Color changed to brown						
		3-4	15				21.6	102.5				<12
15						Color changed to olive brown						
		3-5	55+			Stabilized after drilling completion	24.8	95.7				<12
20				CL-ML		Greenish Gray Clayey SILT Moist, firm First encountered						
		3-6	7				36.4	87.1	0.5	18	45	17
25												
28				GP		Reddish Brown Sandy Clayey GRAVEL Moist, Dense						
		3-7	36				17.5	118.6				
30												

Project: Proposed Lincoln Landing Mixed-Use Dvelopment
Project Location: 22301 Foothill Boulevard Hayward, California
Project Number: SV1302A

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

Log of Boring B-3

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				GP		Reddish Brown Sandy Clayey GRAVEL Moist, Dense GRAVEL: 1-1/2 inches maximum diameter Sub-rounded, poorly graded						
		3-8	55+				12.0	129.9				
35												
38				SP		Tan Brown Silty SAND Moist, dense SAND: Medium grained, poorly graded						
		3-9	55+				27.2	99.4				
40												
45				CL		Olive Brown Silty CLAY Moist, hard						
		3-10	55+				30.9	95.3	1.4	17	42	22
50												
55												
		3-11	51			Color changed to greenish gray	31.2	93.6	1.6	13	45	23
60												
65												

Project: Proposed Lincoln Landing Mixed-Use Dvelopment **Project Location:** 22301 Foothill Boulevard Hayward, California **Project Number:** SV1302A	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	Log of Boring B-3 **Sheet 3 of 3**

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, %
65				CL		Greenish Gray Silty CLAY Moist, hard						
70		3-12	51				23.7	105.2	1.7	15	40	21
75												
80		3-13	71				22.6	107.0	1.9	6	41	21
						Boring terminated at 80.0 feet						
85												
90												
95												
100												

Project: Proposed Lincoln Landing Mixed-Use Dvelopment Project Location: 22301 Foothill Boulevard Hayward, California Project Number: SV1302A	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	# Log of Boring B-4 **Sheet 1 of 1**
Date(s) Drilled **04/01/15**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **20.0 feet**
		Approximate Surface Elevation **112 feet**
Groundwater Level and Date Measured	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0				Asphalt		3.0 inches Asphalt Concrete (AC)						
1						9 inches of Agrregate Base (AB)						
		4-1	18	CH		Dark Gray to Black Silty CLAY Moist, stiff	23.3	103.4				
		4-2	22				20.2	102.5				
5						Color changed to brown						
8				CL-ML		Tan Brown Clayey SILT Moist, stiff						
		4-3	26				20.3	105.5				
10												
13				ML		Tan Brown Sandy SILT Moist, stiff						
		4-4	24				28.3	97.1				
15												
18				CL-ML		Light Olive Brown Clayey SILT Moist, stiff						
		4-5	38				23.9	100.9				
20						Boring terminated at 20.0 feet						
25												
30												

Project: Proposed Lincoln Landing Mixed-Use Dvelopment **Project Location:** 22301 Foothill Boulevard Hayward, California **Project Number:** SV1302A	**Silicon Valley Soil Engineering** **2391 Zanker Road, Suite 350** **San Jose, CA 95131** **(408) 324-1400**	# Log of Boring B-5 **Sheet 1 of 1**

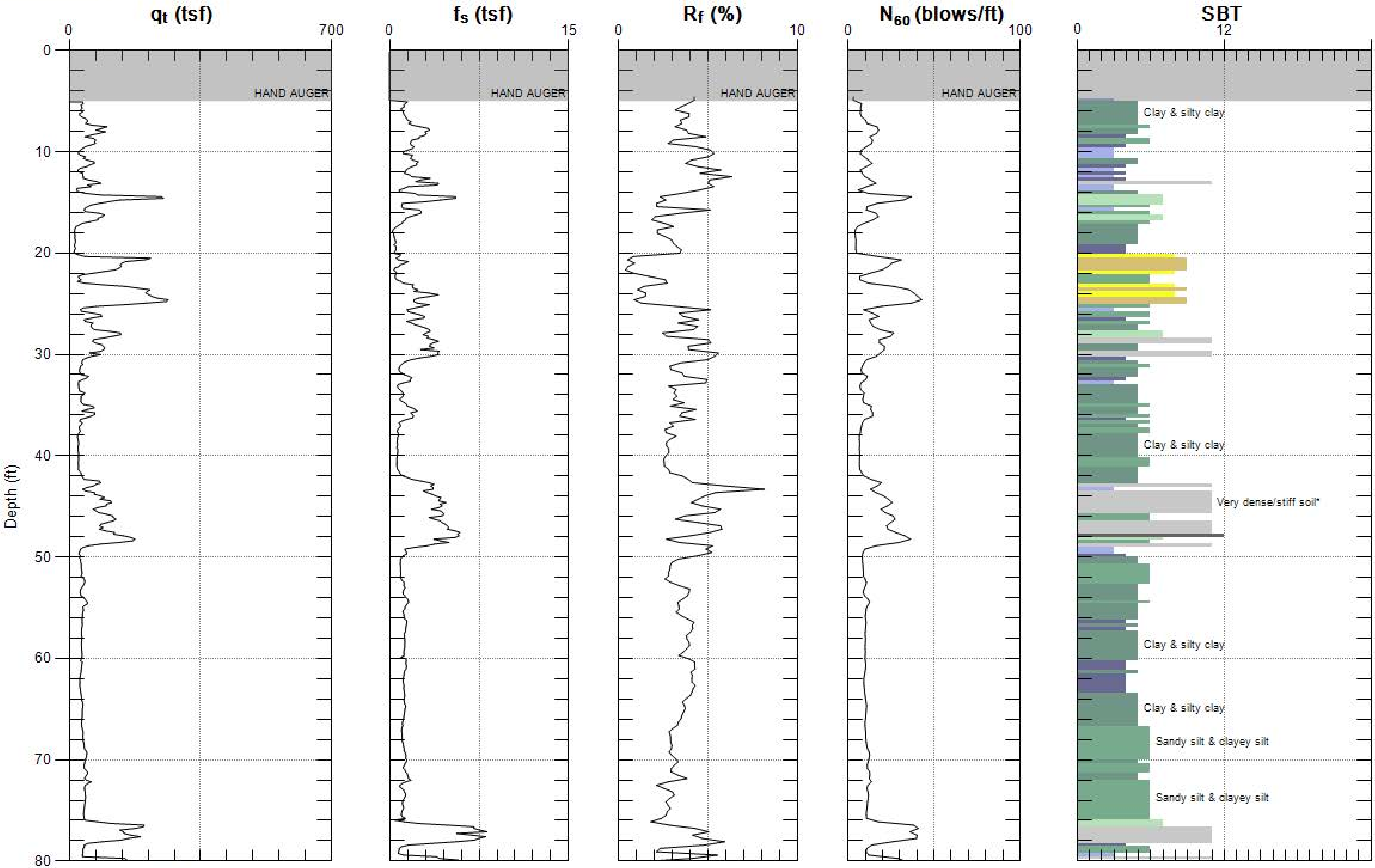
Date(s) Drilled **04/01/15**	Logged By **V.V.**	Checked By
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch**	Total Depth of Borehole **20.0 feet**
		Approximate Surface Elevation **119 feet**
Groundwater Level and Date Measured	Sampling Method(s) **SPT**	Hammer Data **140 lbs**
Borehole Backfill **Grout**	Location	

Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Direct Shear Test - Cohesion in ksf	Direct Shear Test - Internal Friction Angle in degrees	Liquid Limit - LL, %	Plasticity Index - PI, %
0				Asphalt		3.0 inches Asphalt Concrete (AC)						
						9 inches of Agrregate Base (AB)						
		5-1	27	CL		Brown Silty CLAY Moist, very stiff	17.0	113.1				
		5-2	55+	ML		Brown Gravelly Sandy SILTD Damp, hard	5.8	95.8				
5												
		5-3	21				9.2	94.0				
10												
		5-4	28	SP		Brown Silty SAND Damp, medium dense	9.6	96.3				
15												
		5-5	32				6.0	94.2				
20						Boring terminated at 20.0 feet						
25												
30												

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD. Engineer: V.VO

Sounding: CPT-01 Date: 4/1/2015 08:52



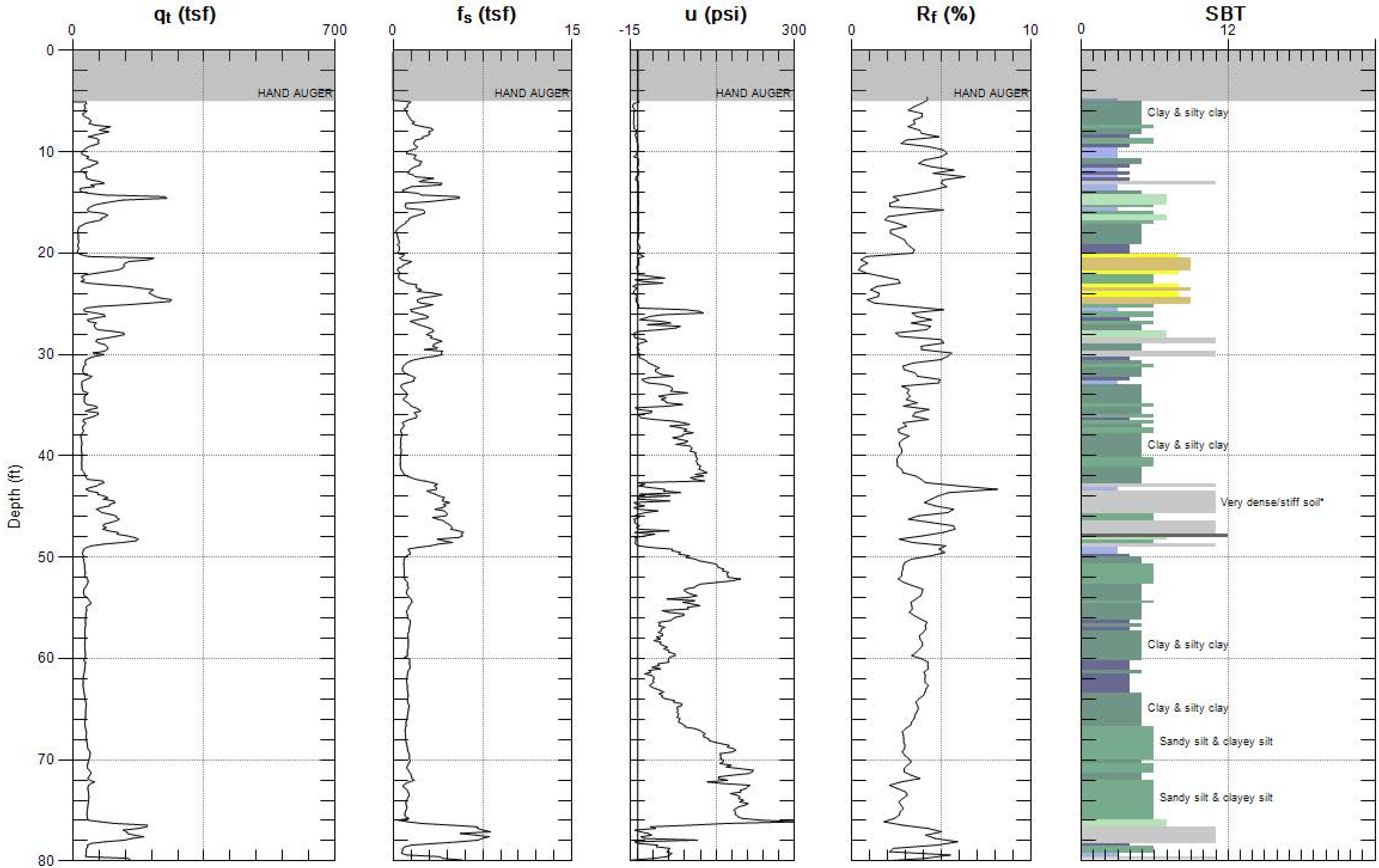
Max. Depth: 80.217 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD. Engineer: V.VO
Sounding: CPT-01 Date: 4/1/2015 08:52



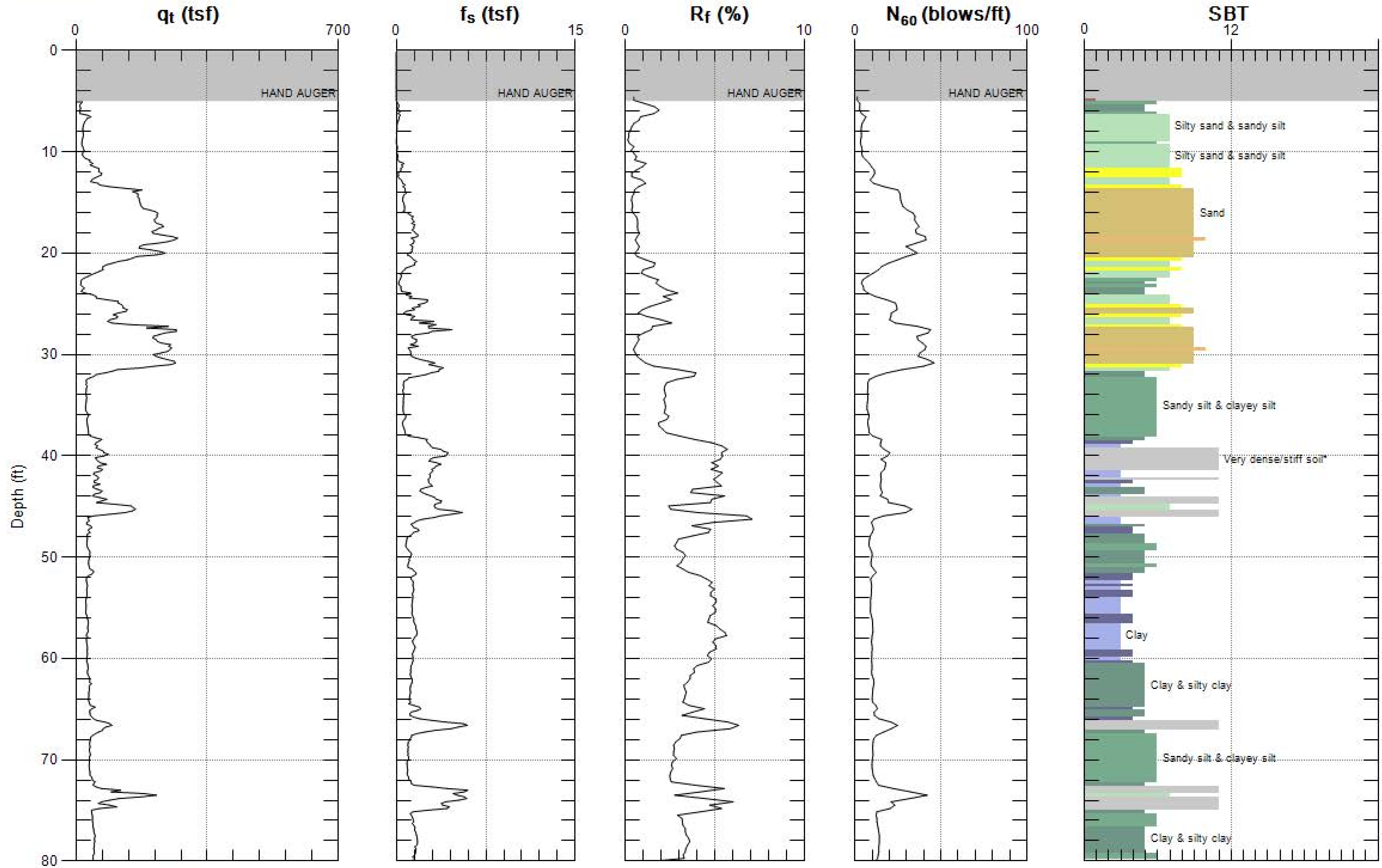
Max. Depth: 80.217 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **SILICON VALLEY SOIL ENG.**

Site: 22301 FOOTHILL BLVD. **Engineer: V.VO**

Sounding: CPT-02 **Date: 4/1/2015 10:50**



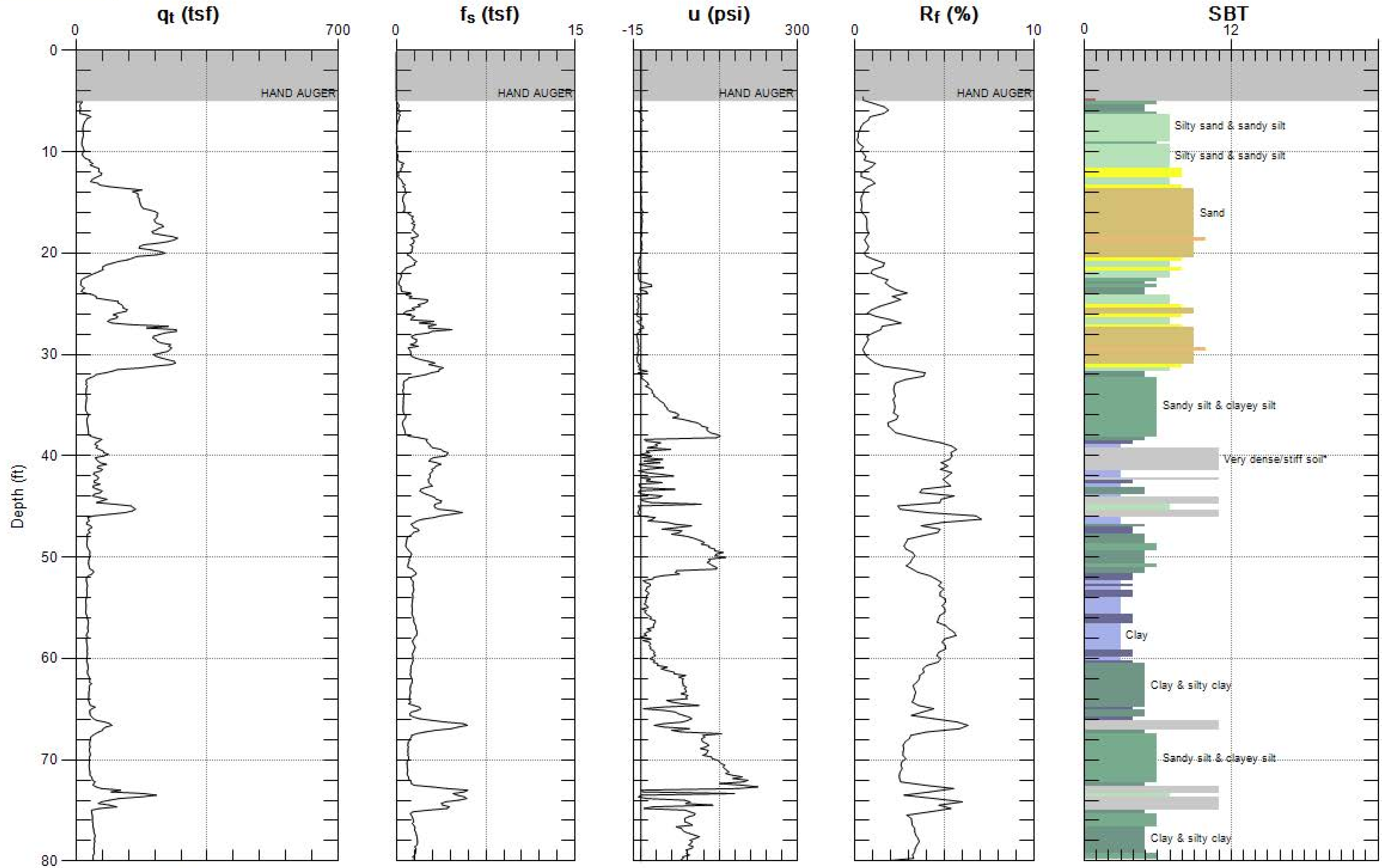
Max. Depth: 80.217 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD.
Engineer: V.VO
Sounding: CPT-02
Date: 4/1/2015 10:50



Max. Depth: 80.217 (ft)
Avg. Interval: 0.328 (ft)

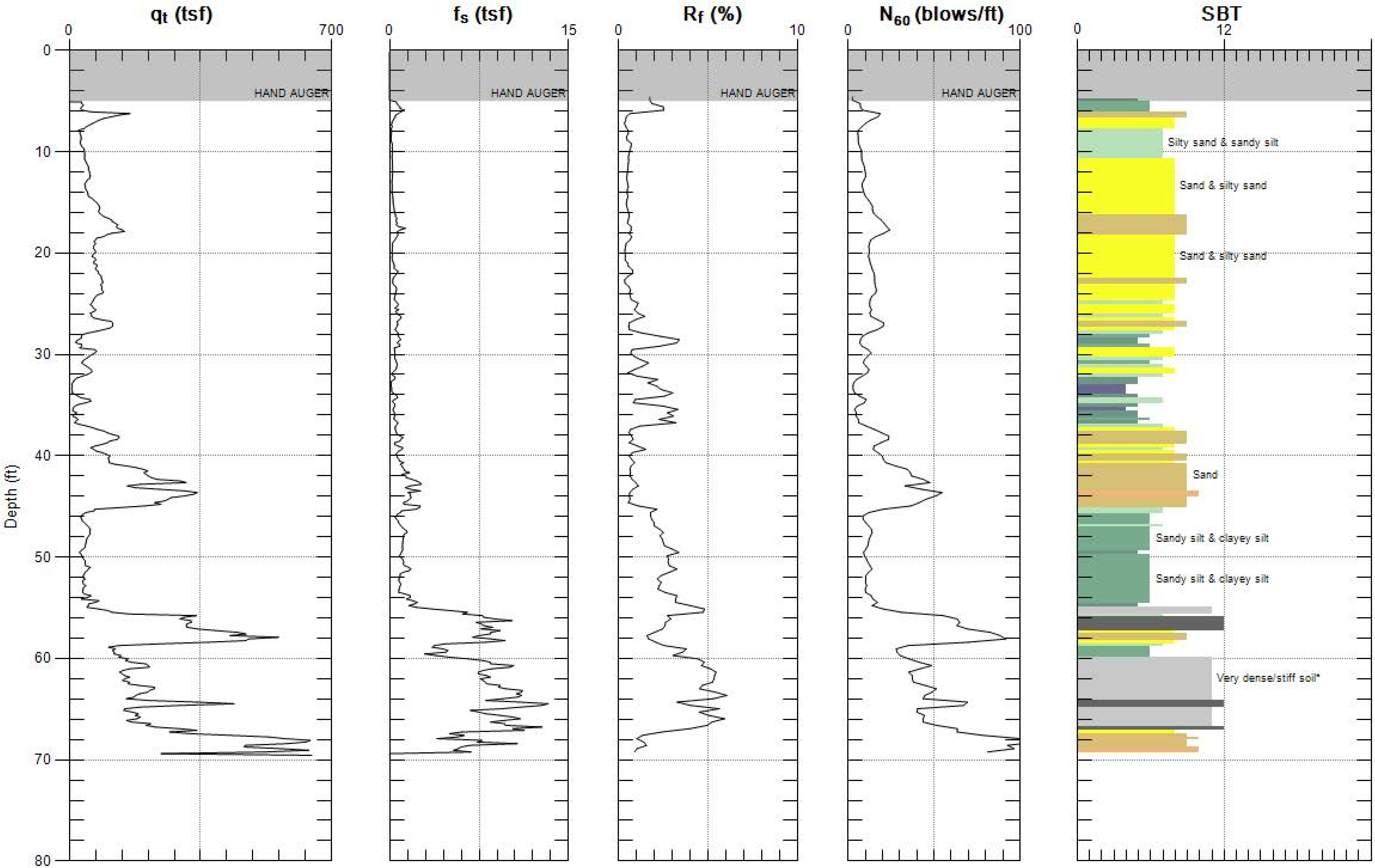
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD. Engineer: V.VO

Sounding: CPT-03 Date: 4/1/2015 12:20



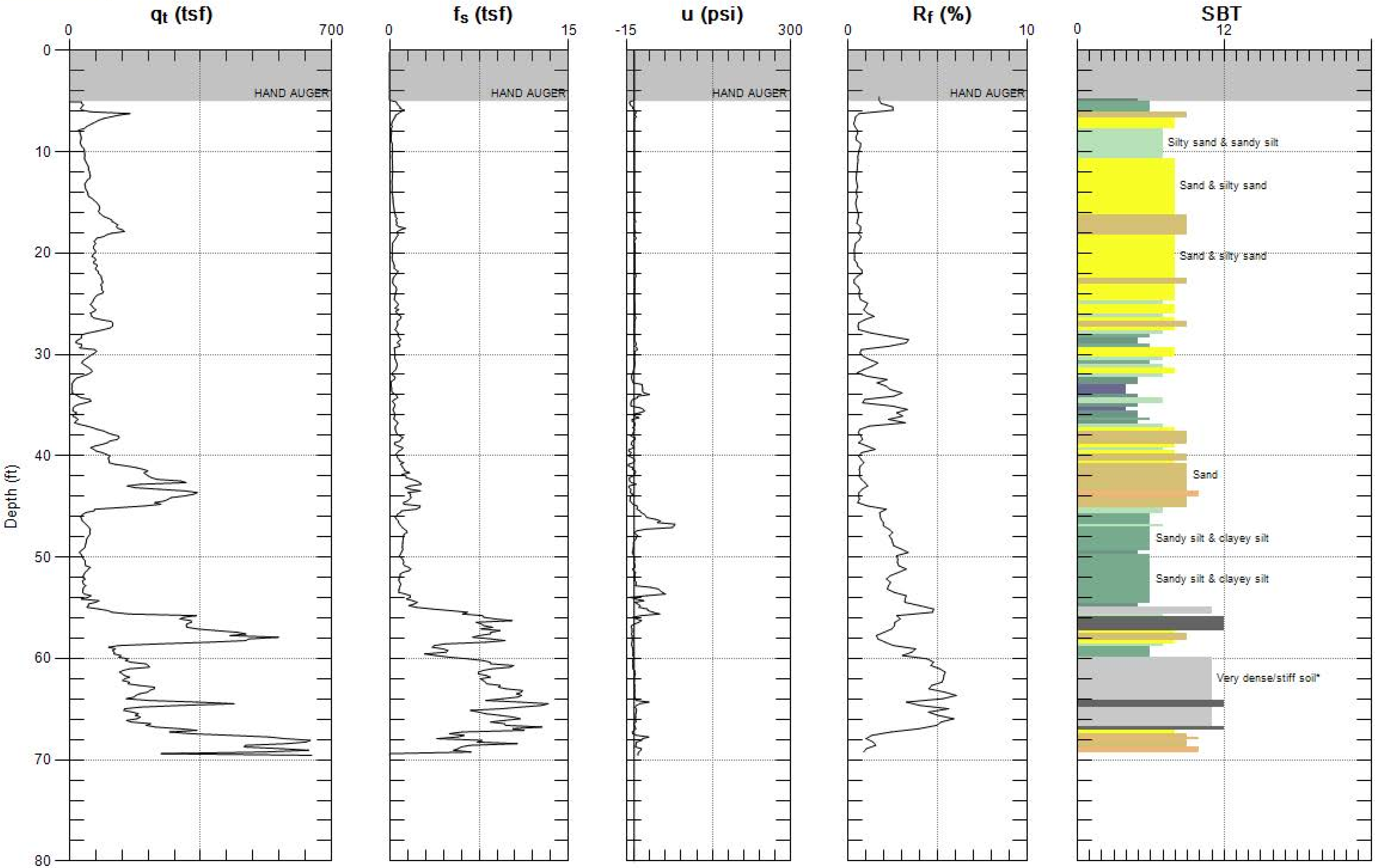
Max. Depth: 69.554 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD.
Sounding: CPT-03

Engineer: V.VO
Date: 4/1/2015 12:20



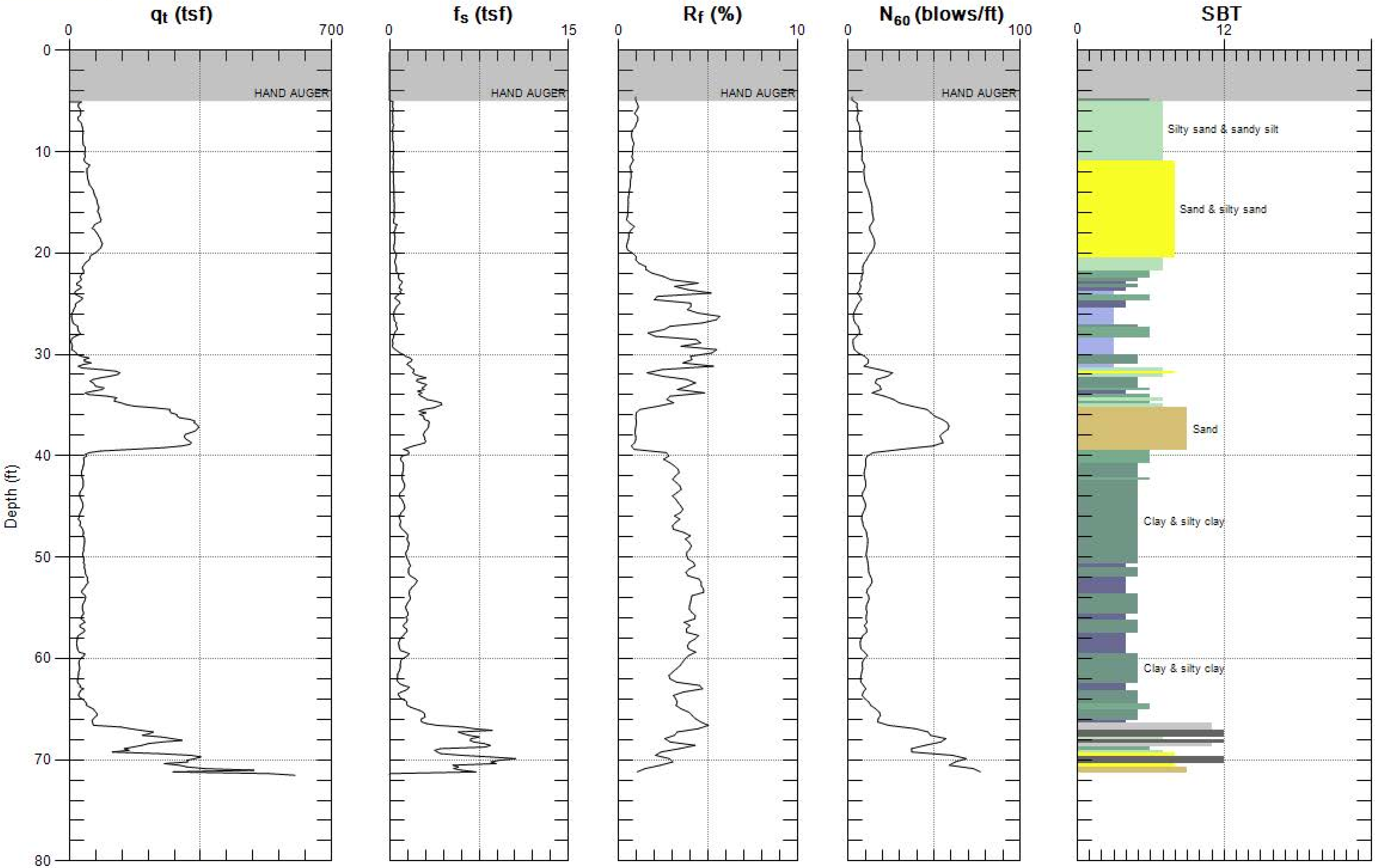
Max. Depth: 69.554 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD.
Engineer: V.VO
Sounding: CPT-04
Date: 4/1/2015 02:24



Max. Depth: 71.522 (ft)
Avg. Interval: 0.328 (ft)

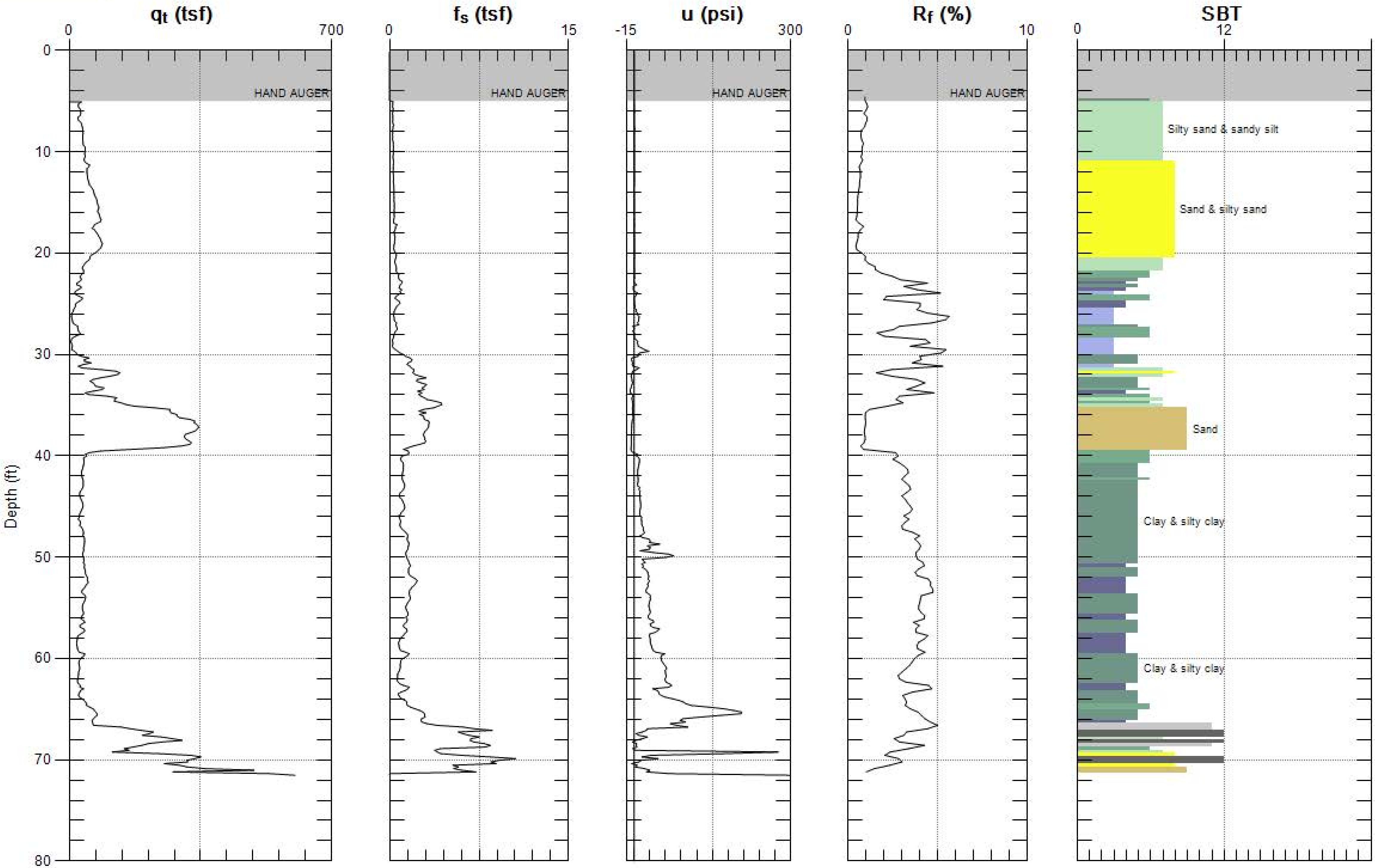
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SILICON VALLEY SOIL ENG.

Site: 22301 FOOTHILL BLVD. Engineer: V.VO

Sounding: CPT-04 Date: 4/1/2015 02:24



Max. Depth: 71.522 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

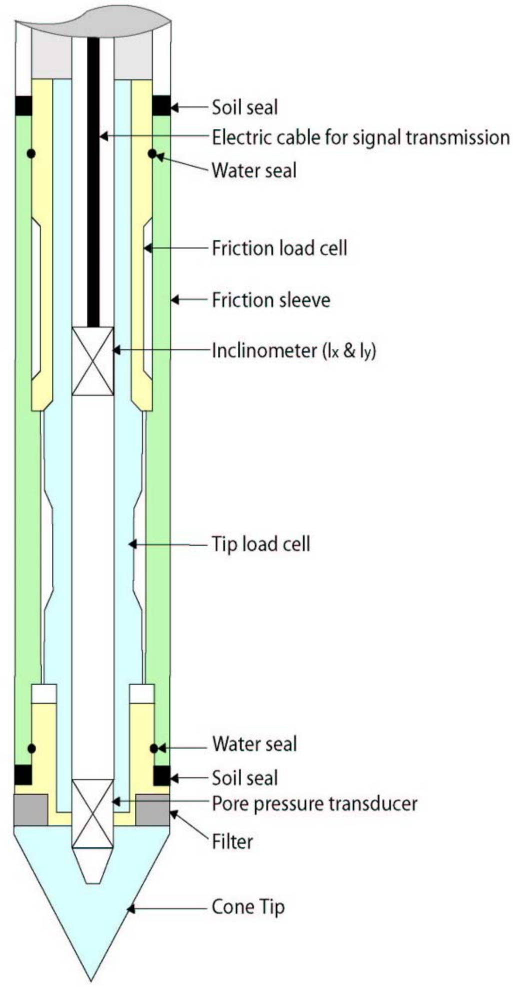


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

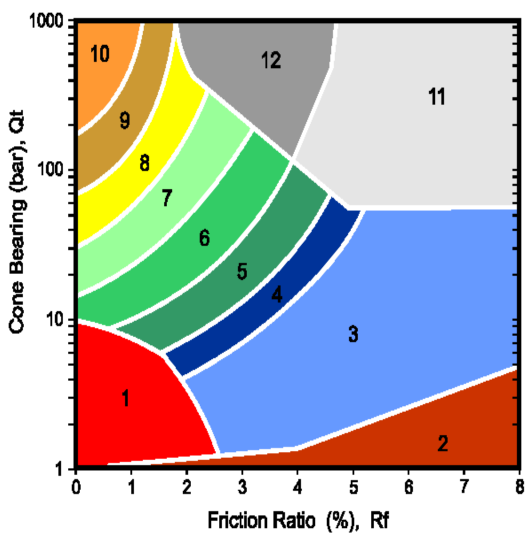
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1. Units for display (Imperial or metric) (atm. pressure, p_a = 0.96 tsf or 0.1 MPa)
2. Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3. Elevation of ground surface (ft or m)
4. Depth to water table, z_w (ft or m) – input required
5. Net area ratio for cone, a (default to 0.80)
6. Relative Density constant, C_{Dr} (default to 350)
7. Young's modulus number for sands, α (default to 5)
8. Small strain shear modulus number
   a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
   b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9. Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10. Over Consolidation ratio number, k_{ocr} (default to 0.3)
11. Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1. Depth, z, (m) – CPT data is collected in meters
2. Depth (ft)
3. Cone resistance, q_c (tsf or MPa)
4. Sleeve resistance, f_s (tsf or MPa)
5. Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6. Other – any additional data
7. Total cone resistance, q_t (tsf or MPa) $\quad q_t = q_c + u\,(1-a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma\ z$
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\ (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, n = 0.5 (clean sand)
When $I_c > 3.30$, n = 1.0 (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\,C_N$,
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, φ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$

Only SBT_n 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go

a. $G_o = S_G\,(q_t\ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*

b. $G_o = C_G\,q_t$ *For SBT_n 1, 2, 3& 4*

Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\,Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones		**SBT_n Zones**	
1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u*) with time is measured behind the tip of the cone and recorded.

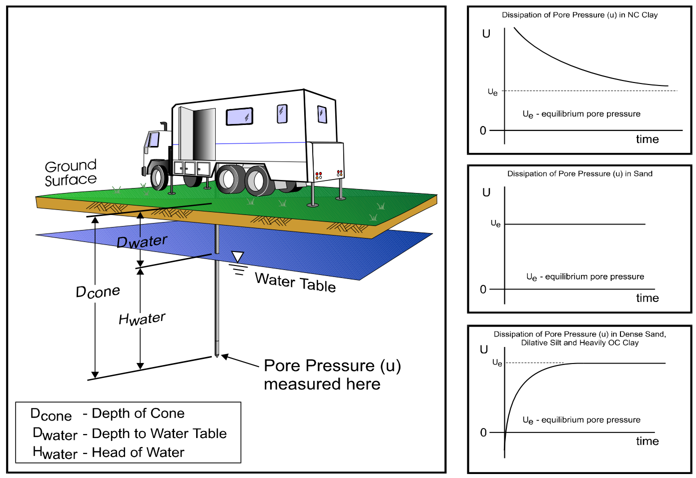
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d/\Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

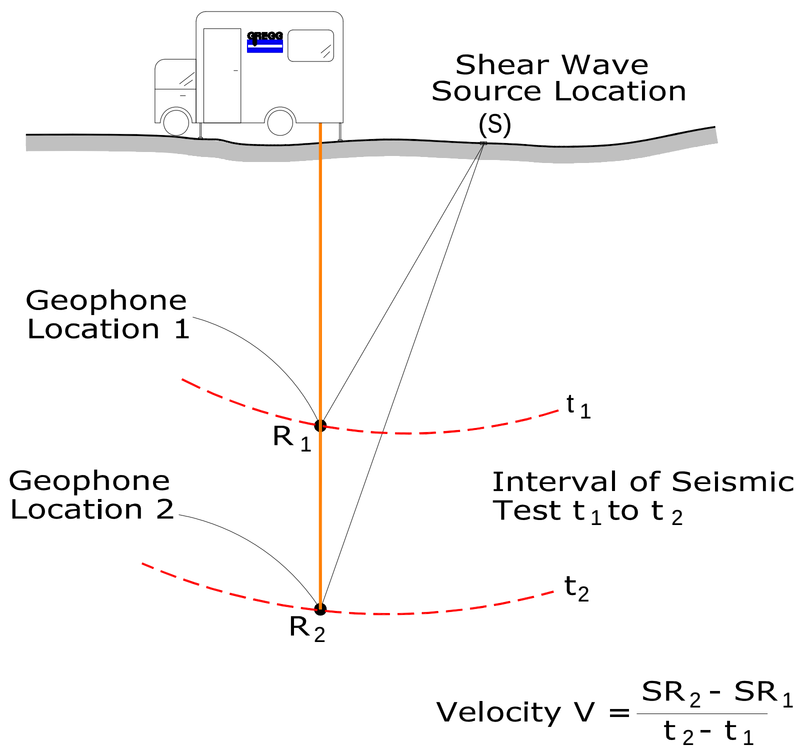


Figure SCPT

Groundwater Sampling

Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

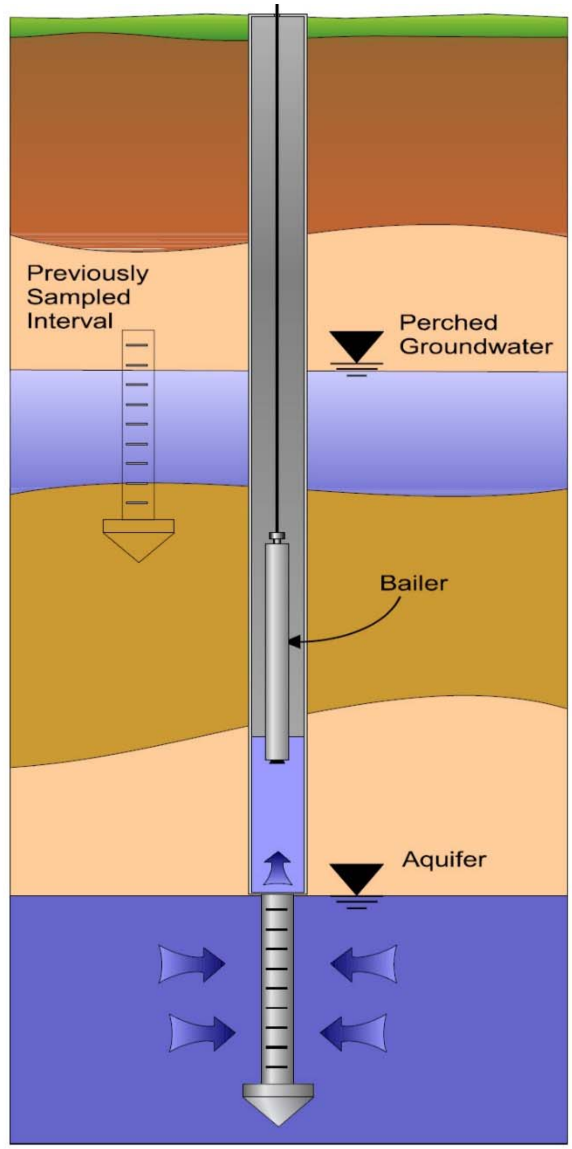


Figure GWS

Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

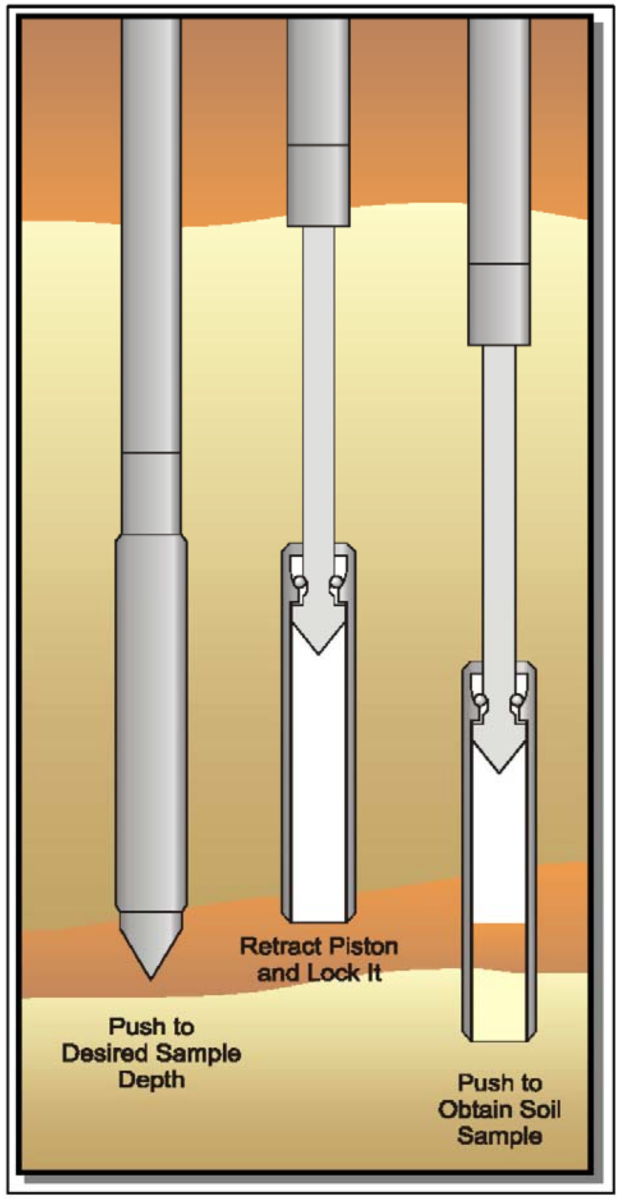


Figure SS

Dakota Technologies UVOST Log Reference

2008-12-12

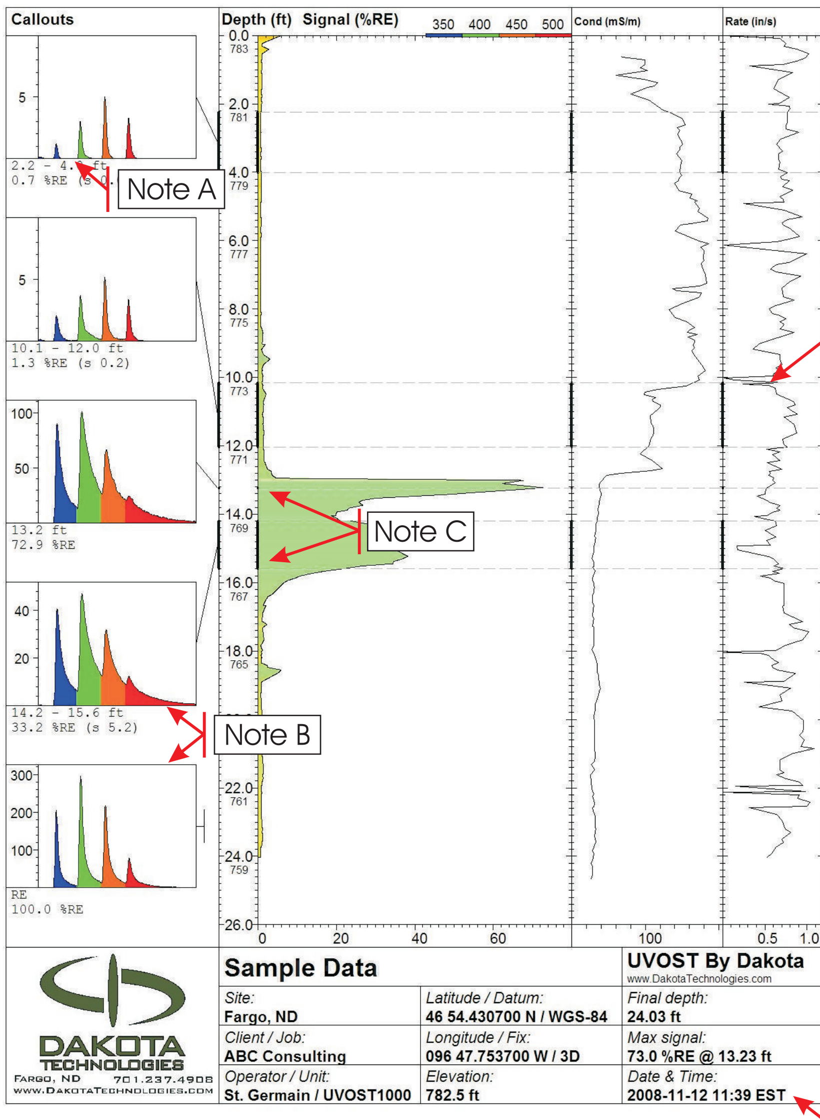
Main Plot :
Signal (total fluorescence) versus depth where signal is relative to the Reference Emitter (RE). The total area of the waveform is divided by the total area of the Reference Emitter yielding the %RE. This %RE scales with the NAPL fluorescence. The fill color is based on relative contribution of each channel's area to the total waveform area (see callout waveform). The channel-to-color relationship and corresponding wavelengths are given in the upper right corner of the main plot.

Callouts :
Waveforms from selected depths or depth ranges showing the multi-wavelength waveform for that depth.

The four peaks are due to fluorescence at four wavelengths and referred to as "channels". Each channel is assigned a color.

Various NAPLs will have a unique waveform "fingerprint" due to the relative amplitude of the four channels and/or broadening of one or more channels.

Basic waveform statistics and any operator notes are given below the callout.



Conductivity Plot :
The Electrical Conductivity (EC) of the soil can be logged simultaneously with the UVOST data. EC often provides insight into the stratigraphy.
Note the drop in EC from 10 - 13 ft, indicating a shift from consolidated to unconsolidated stratigraphy. This correlates with the observed NAPL distribution.

Rate Plot :
The rate of probe advancement. ~ 0.8in (2cm) per second is preferred.

A noticeable decrease in the rate of advancement may be indicative of difficult probing conditions (gravel, angular sands, etc.) such as that seen here at ~5 ft.

Notice that this log was terminated arbitrarily, not due to "refusal", which would have been indicated by a sudden rate drop at final depth.

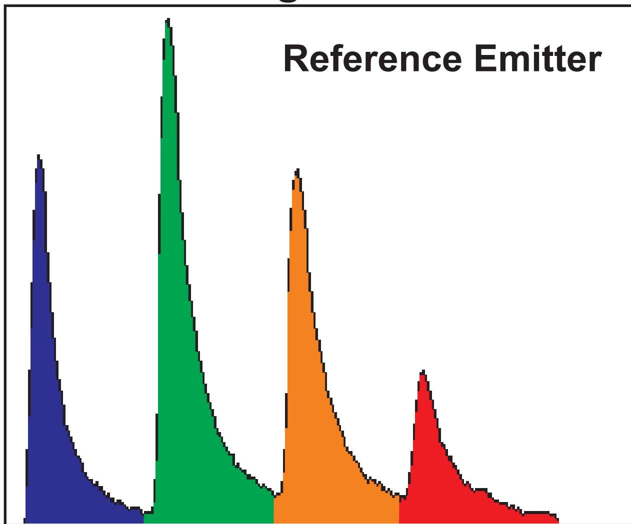
Info Box :
Contains pertinent log info including name and location.

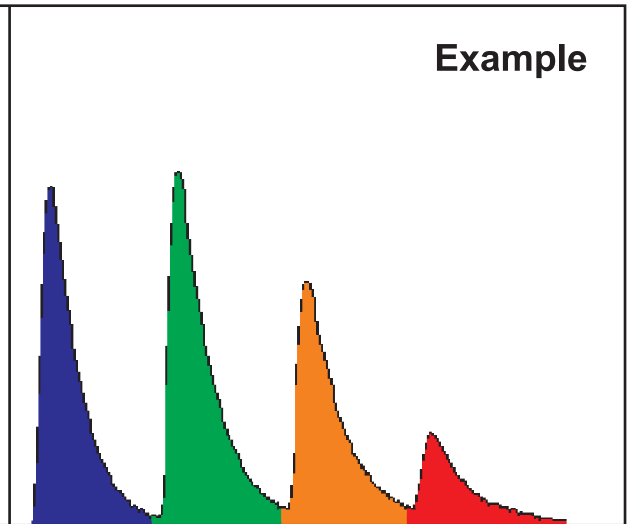
Note A :
Time is along the x axis. No scale is given, but it is a consistent 320ns wide.
The y axis is in mV and directly corresponds to the amount of light striking the photodetector.

Note B :
These two waveforms are clearly different. The first is weathered diesel from the log itself while the second is the Reference Emitter (a blend of NAPLs) always taken before each log for calibration.

Note C :
Callouts can be a single depth (see 3rd callout) or a range (see 4th callout). The range is noted on the depth axis by a bold line. When the callout is a range, the average and standard deviation in %RE is given below the callout.

Waveform Signal Calculation



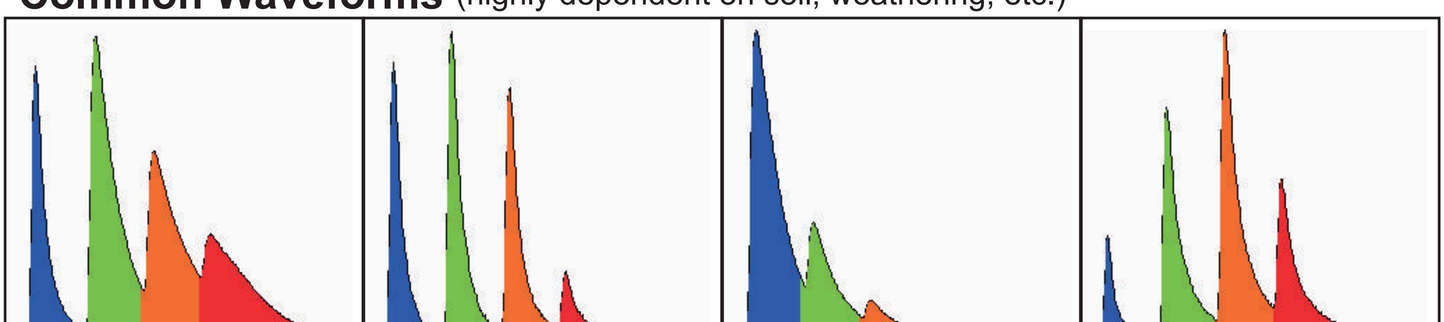


CH1		CH2		CH3		CH4		Total	Channel	CH1		CH2		CH3		CH4		Total
4820	+	8108	+	6249	+	2984	=	**22161**	***Area (pVs)***	4923	+	5743	+	4166	+	1735	=	**16587**
21.7		36.6		28.2		13.5		**100%**	***Percent RE***	22.3		25.9		18.8		7.8		**75%**

Data Files

***.lif.raw.bin**	Raw data file. Header is ASCII format and contains information stored when the file was initially written (e.g. date, total depth, max signal, gps, etc., and any information entered by the operator). All raw waveforms are appended to the bottom of the file in a binary format.
***.lif.plt**	Stores the plot scheme history (e.g. callout depths) for associated Raw file. Transfer along with the Raw file in order to recall previous plots.
***.lif.jpg**	A jpg image of the OST log including the main signal vs. depth plot, callouts, information, etc.
***.lif.dat.txt**	Data export of a single Raw file. ASCII tab delimited format. No string header is provided for the columns (to make importing into other programs easier). Each row is a unique depth reading. The columns are: Depth, Total Signal (%RE), Ch1%, Ch2%, Ch3%, Ch4%, Rate, Conductivity Depth, Conductivity Signal, Hammer Rate. Summing channels 1 to 4 yields the Total Signal.
***.lif.sum.txt**	A summary file for a number of Raw files. ASCII tab delimited format. The file contains a string header. The summary includes one row for each Raw file and contains information for each file including: the file name, gps coordinates, max depth, max signal, and depth at which the max signal occured.
***.lif.log.txt**	An activity log generated automatically located in the OST application directory in the 'log' subfolder. Each OST unit the computer operates will generate a separate log file per month. A log file contains much of the header information contained within each separate Raw file, including: date, total depth, max signal, etc.

Common Waveforms (highly dependent on soil, weathering, etc.)



Diesel	Gas	Kerosene	Motor Oil

Ultra-Violet Induced Fluorescence (UVOST)

Gregg Drilling conducts Laser Induced Fluorescence (LIF) Cone Penetration Tests using a UVOST module that is located behind the standard piezocone, *Figure UVOST*. The laser induced fluorescence cone works on the principle that polycyclic aromatic hydrocarbons (PAH's), mixed with soil and/or groundwater, fluoresce when irradiated by ultra violet light. Therefore, by measuring the intensity of fluorescence, the lateral and vertical extent of hydrocarbon contamination in the ground can be estimated.

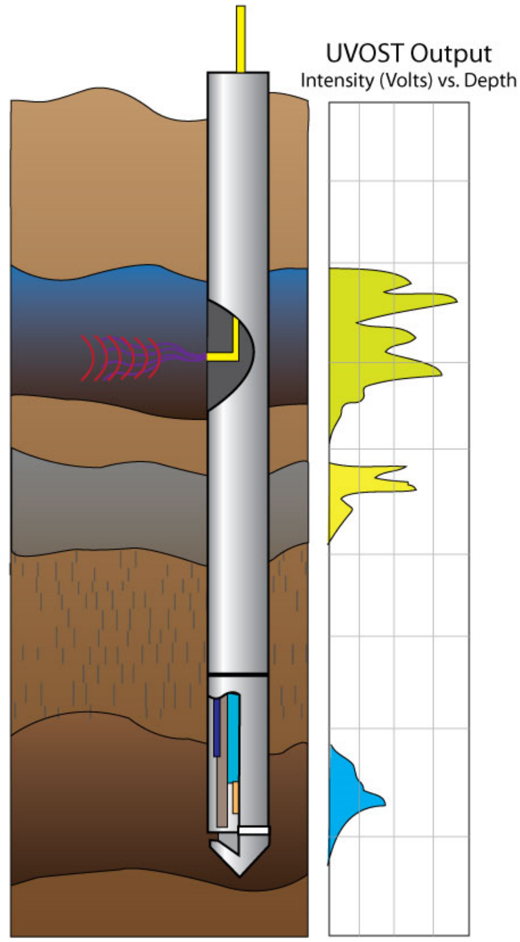


Figure UVOST

The UVOST module uses principles of fluorescence spectrometry by irradiating the soil with ultra violet light produced by a laser and transmitted to the cone through fiber optic cables. The UV light passes through a small window in the side of the cone into the soil. Any hydrocarbon molecules present in the soil absorb the light energy during radiation and immediately re-emit the light at a longer wavelength. This re-emission is termed fluorescence. The UVOST system also measures the emission decay with time at four different wavelengths (350nm, 400nm, 450nm, and 500nm). This allows the software to determine a product "signature" at each data point. This process provides a method to evaluate the type of contaminant. A sample output from the UVOST system is shown in *Figure Output*. In general, the typical detection limit for the UVOST system is <100 ppm and it will operate effectively above and below the saturated zone.

With the capability to push up to 200m (600ft) per day, laser induced fluorescence offers a fast and efficient means for delineating PAH contaminant plumes. Color coded logs offer qualitative information in a quick glance and can be produced in the field for real-time decision making. Coupled with the data provided by the CPT, a complete site assessment can be completed with no samples or cuttings, saving laboratory costs as well as site and environmental impact.

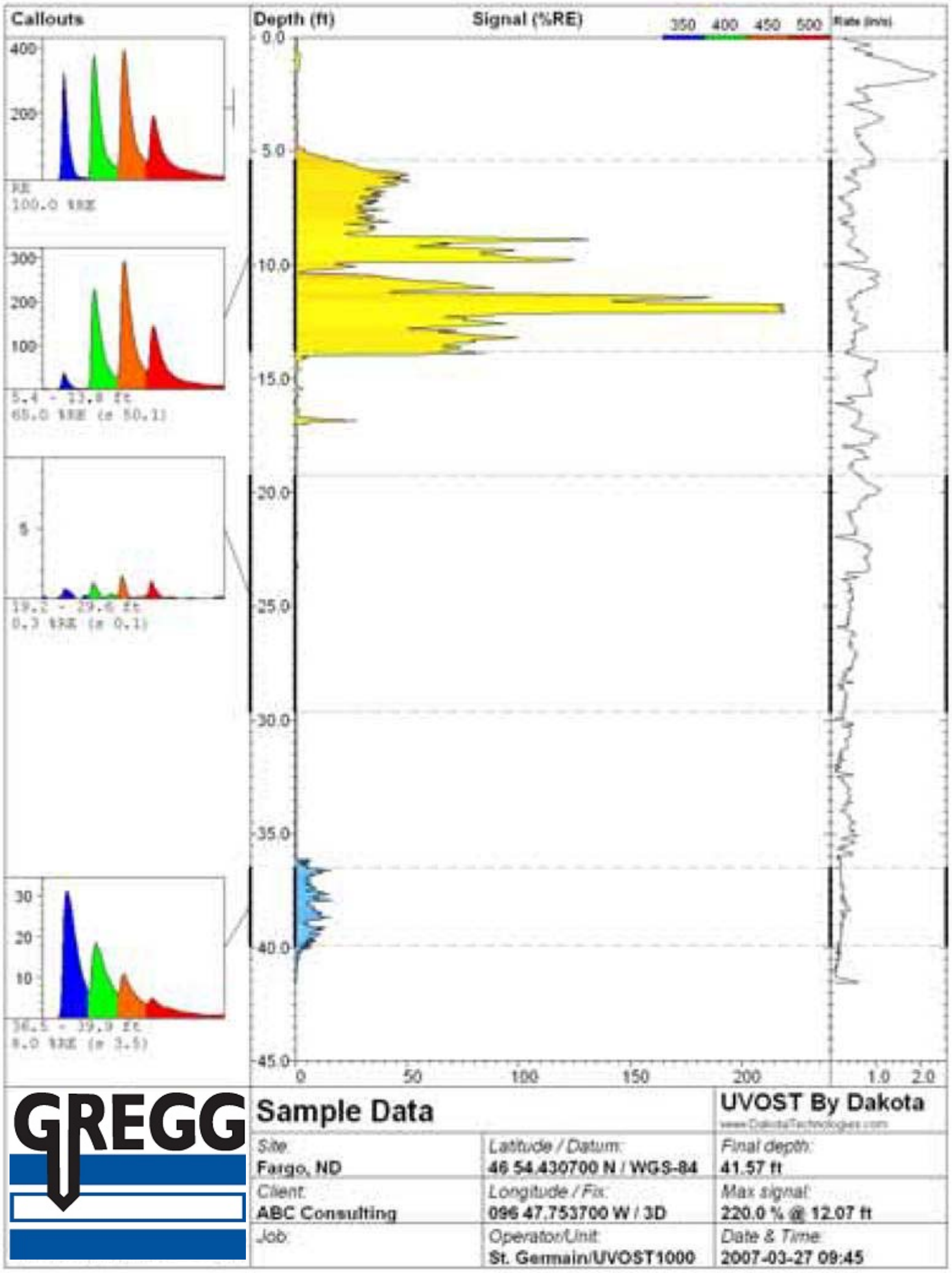


Figure Output

Hydrocarbons detected with UVOST

- Gasoline
- Diesel
- Jet (Kerasene)
- Motor Oil
- Cutting fluids
- Hydraulic fluids
- Crude Oil

Hydrocarbons rarely detected using UVOST

- Extremely weathered gasoline
- Coal tar
- Creosote
- Bunker Oil
- Polychlorinated bi-phenols (PCB's)
- Chlorinated solvent DNAPL
- Dissolved phase (aqueous) PAH's

Potential False Positives (fluorescence observed)

- Sea-shells (weak-medium)
- Paper (medium-strong depending on color)
- Peat/meadow mat (weak)
- Calcite/calcareous sands (weak)
- Tree roots (weak-medium)
- Sewer lines (medium-strong)

Potential False Negatives (do not fluoresce)

- Extremely weathered fuels (especially gasoline)
- Aviation gasoline (weak)
- "Dry" PAHs such as aqueous phase, lamp black, purifier chips
- Creosotes (most)
- Coal tars (most) gasoline (weak)
- Most chlorinated solvents
- Benzene, toluene, zylenes (relatively pure)

Alameda County Public Works Agency - Water Resources Well Permit

Public Works Agency
Alameda County

399 Elmhurst Street
Hayward, CA 94544-1395
Telephone: (510)670-6633 Fax:(510)782-1939

Application Approved on: 03/30/2015 By jamesy

Permit Numbers: W2015-0274
Permits Valid from 04/01/2015 to 04/01/2015

Application Id: 1427734109772
City of Project Site: Hayward
Site Location: 22301 Foothill Blvd, Hayward, CA
Project Start Date: 04/01/2015
Completion Date: 04/01/2015
Assigned Inspector: Contact Steve Miller at (510) 670-5517 or stevem@acpwa.org

Applicant: Silicon Va Soil Engineering - Sean Deivert
2391 Zanker Road, #350, San Jose, CA 95131
Phone: 408-324-1400
Property Owner: 22301 Foothill Hayward LLC Chavez Mgmt. Group
1860 El Camino Real #250, Burlingame, CA 94101
Phone: --
Client: ** same as Property Owner **

Total Due: $265.00
Receipt Number: WR2015-0153 **Total Amount Paid:** $265.00
Payer Name : Silicon Valley Soil Engineering Paid By: CHECK **PAID IN FULL**

Works Requesting Permits:

Borehole(s) for Investigation-Geotechnical Study/CPT's - 8 Boreholes
Driller: Gregg Drilling and Exploration Geoservices-484288 - Lic #: 485165 - **Work Total: $265.00**
Method: other

Specifications

Permit Number	Issued Dt	Expire Dt	# Boreholes	Hole Diam	Max Depth
W2015-0274	03/30/2015	06/30/2015	8	8.00 in.	80.00 ft

Specific Work Permit Conditions

1. Backfill bore hole by tremie with cement grout or cement grout/sand mixture. Upper two-three feet replaced in kind or with compacted cuttings. All cuttings remaining or unused shall be containerized and hauled off site.

2. Boreholes shall not be left open for a period of more than 24 hours. All boreholes left open more than 24 hours will need approval from Alameda County Public Works Agency, Water Resources Section. All boreholes shall be backfilled according to permit destruction requirements and all concrete material and asphalt material shall be to Caltrans Spec or County/City Codes. No borehole(s) shall be left in a manner to act as a conduit at any time.

3. Permittee shall assume entire responsibility for all activities and uses under this permit and shall indemnify, defend and save the Alameda County Public Works Agency, its officers, agents, and employees free and harmless from any and all expense, cost, liability in connection with or resulting from the exercise of this Permit including, but not limited to, properly damage, personal injury and wrongful death.

4. Prior to any drilling activities, it shall be the applicant's responsibility to contact and coordinate an Underground Service Alert (USA), obtain encroachment permit(s), excavation permit(s) or any other permits or agreements required for that Federal, State, County or City, and follow all City or County Ordinances. No work shall begin until all the permits and requirements have been approved or obtained. It shall also be the applicants responsibilities to provide to the Cities or to Alameda County an Traffic Safety Plan for any lane closures or detours planned. No work shall begin until all the permits and requirements have been approved or obtained.

5. Applicant shall contact assigned inspector listed on the top of the permit at least five (5) working days prior to starting, once the permit has been approved. Confirm the scheduled date(s) at least 24 hours prior to drilling.

6. Permittee, permittee's contractors, consultants or agents shall be responsible to assure that all material or waters generated during drilling, boring destruction, and/or other activities associated with this Permit will be safely handled, properly managed, and disposed of according to all applicable federal, state, and local statutes regulating such. In no case shall these materials and/or waters be allowed to enter, or potentially enter, on or off-site storm sewers, dry wells, or waterways or be allowed to move off the property where work is being completed.

7. Cuttings may also be left on site or spread out as long as the applicants has approval from the property owner and the cuttings will not violate the State and County Clean Water laws (NPDES).

8. Copy of approved drilling permit must be on site at all times. Failure to present or show proof of the approved permit application on site shall result in a fine of $500.00.

9. Prior to any drilling activities onto any public right-of-ways, it shall be the applicants responsibilities to contact and coordinate a Underground Service Alert (USA), obtain encroachment permit(s), excavation permit(s) or any other permits required for that City or to the County and follow all City or County Ordinances. It shall also be the applicants responsibilities to provide to the Cities or to Alameda County a Traffic Safety Plan for any lane closures or detours planned. No work shall begin until all the permits and requirements have been approved or obtained.

10. Permit is valid only for the purpose specified herein. No changes in construction procedures, as described on this permit application. Boreholes shall not be converted to monitoring wells, without a permit application process.

****

LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV (1300-1309)\SV1302 - Foothill Blvd. - Hayward\SV1302.GI\SV1302.LA.liq
Title: SV1302 - Proposed City Sport
Subtitle: 22301 Foothill Blvd, Hayward, CA

Surface Elev.=100
Hole No.=B-1
Depth of Hole= 51.50 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 23.00 ft
Max. Acceleration= 0.71 g
Earthquake Magnitude= 7.90

Input Data:
Surface Elev.=100
Hole No.=B-1
Depth of Hole=51.50 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 23.00 ft
Max. Acceleration=0.71 g
Earthquake Magnitude=7.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: Post Liquefaction
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 0.88
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	7.00	116.00	NoLiq
3.00	15.00	120.00	NoLiq
8.00	9.00	103.00	15.00
12.00	15.00	109.00	15.00
17.00	17.00	106.00	15.00
22.00	12.00	128.00	NoLiq
27.00	48.00	111.00	15.00
32.00	29.00	120.00	NoLiq
37.00	47.00	128.00	NoLiq
42.00	20.00	119.00	NoLiq

47.00	31.00	119.00	NoLiq

Output Results:
Settlement of Saturated Sands=2.88 in.
Settlement of Unsaturated Sands=2.04 in.
Total Settlement of Saturated and Unsaturated Sands=4.92 in.
Differential Settlement=2.461 to 3.248 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.60	5.00	2.88	2.04	4.92
1.00	2.00	0.60	5.00	2.88	2.04	4.92
2.00	2.00	0.60	5.00	2.88	2.04	4.92
3.00	2.00	0.60	5.00	2.88	2.04	4.92
4.00	0.31	0.59	5.00	2.88	1.91	4.79
5.00	0.27	0.59	5.00	2.88	1.85	4.73
6.00	0.25	0.59	5.00	2.88	1.67	4.55
7.00	0.21	0.59	5.00	2.88	1.40	4.28
8.00	0.14	0.59	5.00	2.88	1.04	3.92
9.00	0.17	0.59	5.00	2.88	0.67	3.55
10.00	0.18	0.59	5.00	2.88	0.35	3.23
11.00	0.20	0.58	5.00	2.88	0.17	3.05
12.00	0.21	0.58	0.36*	2.88	0.00	2.88
13.00	0.21	0.61	0.34*	2.63	0.00	2.63
14.00	0.20	0.63	0.32*	2.37	0.00	2.37
15.00	0.23	0.65	0.35*	2.12	0.00	2.12
16.00	0.23	0.67	0.34*	1.88	0.00	1.88
17.00	0.22	0.69	0.32*	1.64	0.00	1.64
18.00	0.26	0.71	0.36*	1.40	0.00	1.40
19.00	0.24	0.72	0.33*	1.16	0.00	1.16
20.00	0.22	0.74	0.30*	0.91	0.00	0.91
21.00	0.20	0.75	0.27*	0.65	0.00	0.65
22.00	0.19	0.76	0.25*	0.39	0.00	0.39
23.00	0.28	0.77	0.37*	0.17	0.00	0.17
24.00	0.44	0.78	0.56*	0.01	0.00	0.01
25.00	0.44	0.79	0.55*	0.00	0.00	0.00
26.00	0.44	0.80	0.55*	0.00	0.00	0.00
27.00	0.44	0.81	0.54*	0.00	0.00	0.00
28.00	0.44	0.82	0.53*	0.00	0.00	0.00
29.00	0.44	0.83	0.53*	0.00	0.00	0.00
30.00	0.44	0.83	0.52*	0.00	0.00	0.00
31.00	0.44	0.83	0.52*	0.00	0.00	0.00
32.00	0.44	0.83	0.52*	0.00	0.00	0.00
33.00	2.00	0.83	5.00	0.00	0.00	0.00
34.00	2.00	0.83	5.00	0.00	0.00	0.00
35.00	2.00	0.83	5.00	0.00	0.00	0.00
36.00	2.00	0.83	5.00	0.00	0.00	0.00
37.00	2.00	0.83	5.00	0.00	0.00	0.00
38.00	2.00	0.82	5.00	0.00	0.00	0.00
39.00	2.00	0.82	5.00	0.00	0.00	0.00
40.00	2.00	0.82	5.00	0.00	0.00	0.00
41.00	2.00	0.81	5.00	0.00	0.00	0.00
42.00	2.00	0.81	5.00	0.00	0.00	0.00
43.00	2.00	0.81	5.00	0.00	0.00	0.00
44.00	2.00	0.80	5.00	0.00	0.00	0.00
45.00	2.00	0.80	5.00	0.00	0.00	0.00
46.00	2.00	0.80	5.00	0.00	0.00	0.00
47.00	2.00	0.79	5.00	0.00	0.00	0.00
48.00	2.00	0.79	5.00	0.00	0.00	0.00
49.00	2.00	0.78	5.00	0.00	0.00	0.00
50.00	2.00	0.78	5.00	0.00	0.00	0.00

51.00	2.00	0.77	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

---

1 atm (atmosphere) = 1 tsf (ton/ft2)

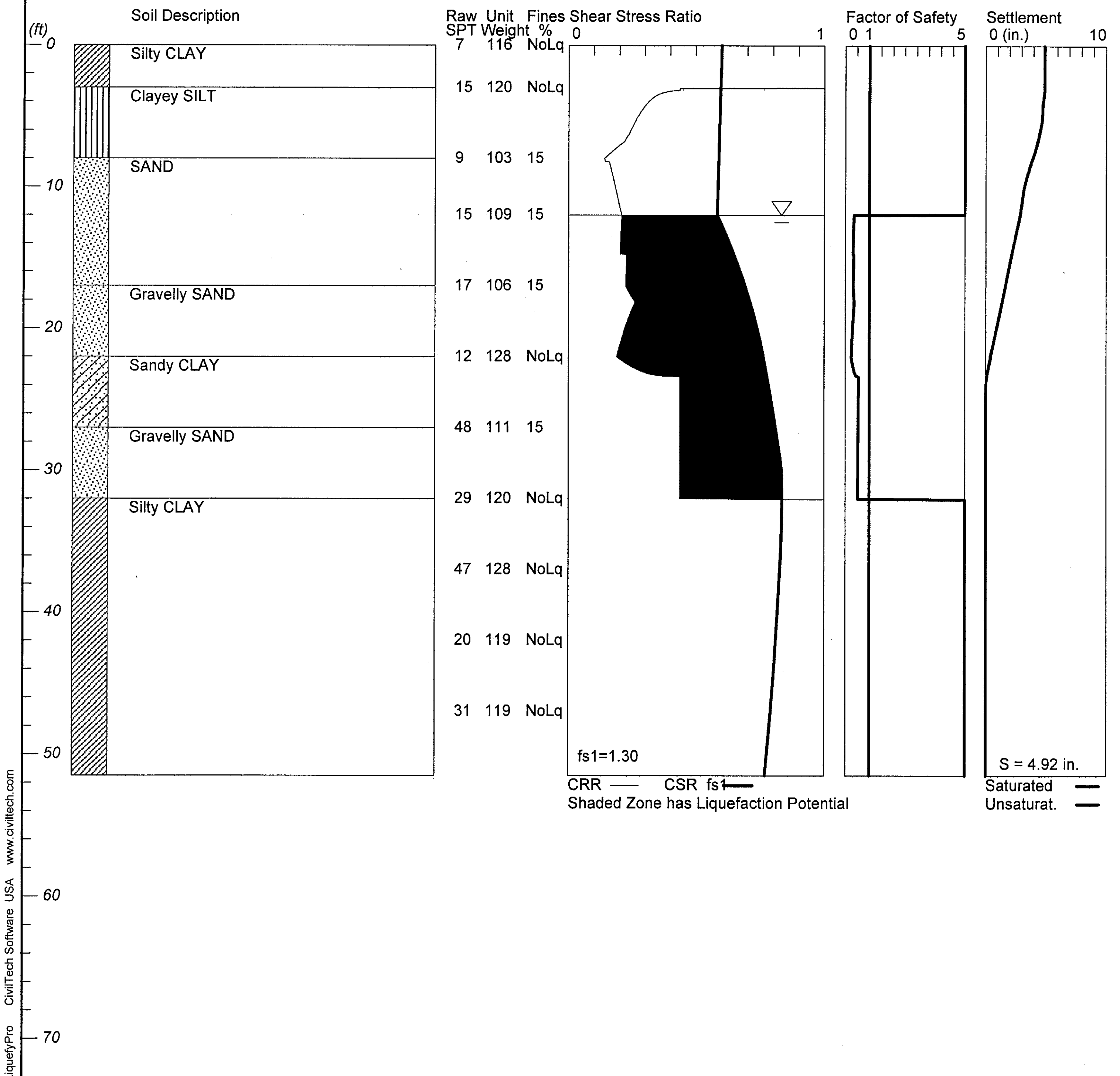
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

SV1302 - Proposed City Sport

Hole No.=B-1   Water Depth=12 ft   Surface Elev.=100

Magnitude=7.9
Acceleration=0.71g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV (1300-1309)\SV1302 - Foothill Blvd. - Hayward\SV1302A.GI\SV1302A. Lique. Boring B-3.liq
Title: SV1302A - Proposed Lincoln Landing
Subtitle: 22301 Foothill Blvd., Hayward, CA

Surface Elev.=100
Hole No.=B-3
Depth of Hole= 80.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.71 g
Earthquake Magnitude= 7.90

Input Data:
Surface Elev.=100
Hole No.=B-3
Depth of Hole=80.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.71 g
Earthquake Magnitude=7.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 0.88
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	17.00	123.00	40.00
4.00	15.00	125.90	5.00
7.00	23.00	122.90	5.00
12.00	15.00	124.60	5.00
17.00	55.00	119.40	5.00
20.00	7.00	118.80	40.00
28.00	36.00	139.40	5.00
32.00	55.00	145.50	5.00
38.00	55.00	126.40	5.00

45.00	55.00	124.70	NoLiq
55.00	51.00	122.80	NoLiq
65.00	51.00	130.10	NoLiq
75.00	71.00	131.10	NoLiq

Output Results:
Settlement of Saturated Sands=2.02 in.
Settlement of Unsaturated Sands=1.22 in.
Total Settlement of Saturated and Unsaturated Sands=3.23 in.
Differential Settlement=1.617 to 2.135 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	0.44	0.60	5.00	2.02	1.22	3.23
1.00	0.44	0.60	5.00	2.02	1.22	3.23
2.00	0.33	0.60	5.00	2.02	1.21	3.23
3.00	0.26	0.60	5.00	2.02	1.18	3.19
4.00	0.22	0.59	5.00	2.02	0.99	3.00
5.00	0.29	0.59	5.00	2.02	0.94	2.96
6.00	0.44	0.59	5.00	2.02	0.88	2.89
7.00	0.44	0.59	5.00	2.02	0.76	2.77
8.00	0.31	0.59	5.00	2.02	0.61	2.62
9.00	0.30	0.59	5.00	2.02	0.45	2.46
10.00	0.24	0.59	5.00	2.02	0.36	2.38
11.00	0.20	0.58	5.00	2.02	0.24	2.26
12.00	0.17	0.58	0.30*	2.02	0.00	2.02
13.00	0.28	0.61	0.47*	1.78	0.00	1.78
14.00	0.44	0.63	0.70*	1.69	0.00	1.69
15.00	0.44	0.64	0.68*	1.69	0.00	1.69
16.00	0.44	0.66	0.66*	1.69	0.00	1.69
17.00	0.44	0.68	0.65*	1.69	0.00	1.69
18.00	0.44	0.69	0.63*	1.69	0.00	1.69
19.00	0.44	0.71	0.62*	1.68	0.00	1.68
20.00	0.14	0.72	0.19*	1.46	0.00	1.46
21.00	0.17	0.73	0.24*	1.15	0.00	1.15
22.00	0.20	0.74	0.27*	0.89	0.00	0.89
23.00	0.23	0.75	0.31*	0.65	0.00	0.65
24.00	0.26	0.76	0.35*	0.44	0.00	0.44
25.00	0.31	0.77	0.41*	0.25	0.00	0.25
26.00	0.44	0.77	0.57*	0.10	0.00	0.10
27.00	0.44	0.78	0.56*	0.01	0.00	0.01
28.00	0.44	0.79	0.56*	0.00	0.00	0.00
29.00	0.44	0.79	0.55*	0.00	0.00	0.00
30.00	0.44	0.79	0.55*	0.00	0.00	0.00
31.00	0.44	0.79	0.55*	0.00	0.00	0.00
32.00	0.44	0.79	0.55*	0.00	0.00	0.00
33.00	0.44	0.79	0.55*	0.00	0.00	0.00
34.00	0.44	0.79	0.55*	0.00	0.00	0.00
35.00	0.43	0.78	0.55*	0.00	0.00	0.00
36.00	0.43	0.78	0.55*	0.00	0.00	0.00
37.00	0.43	0.78	0.55*	0.00	0.00	0.00
38.00	0.43	0.78	0.55*	0.00	0.00	0.00
39.00	0.43	0.77	0.55*	0.00	0.00	0.00
40.00	0.43	0.77	0.55*	0.00	0.00	0.00
41.00	0.43	0.77	0.55*	0.00	0.00	0.00
42.00	0.42	0.77	0.55*	0.00	0.00	0.00
43.00	0.42	0.76	0.56*	0.00	0.00	0.00
44.00	0.42	0.76	0.56*	0.00	0.00	0.00
45.00	0.42	0.75	0.56*	0.00	0.00	0.00
46.00	2.00	0.75	5.00	0.00	0.00	0.00
47.00	2.00	0.75	5.00	0.00	0.00	0.00

48.00	2.00	0.74	5.00	0.00	0.00	0.00
49.00	2.00	0.74	5.00	0.00	0.00	0.00
50.00	2.00	0.73	5.00	0.00	0.00	0.00
51.00	2.00	0.73	5.00	0.00	0.00	0.00
52.00	2.00	0.72	5.00	0.00	0.00	0.00
53.00	2.00	0.72	5.00	0.00	0.00	0.00
54.00	2.00	0.71	5.00	0.00	0.00	0.00
55.00	2.00	0.71	5.00	0.00	0.00	0.00
56.00	2.00	0.70	5.00	0.00	0.00	0.00
57.00	2.00	0.70	5.00	0.00	0.00	0.00
58.00	2.00	0.69	5.00	0.00	0.00	0.00
59.00	2.00	0.68	5.00	0.00	0.00	0.00
60.00	2.00	0.68	5.00	0.00	0.00	0.00
61.00	2.00	0.67	5.00	0.00	0.00	0.00
62.00	2.00	0.67	5.00	0.00	0.00	0.00
63.00	2.00	0.66	5.00	0.00	0.00	0.00
64.00	2.00	0.65	5.00	0.00	0.00	0.00
65.00	2.00	0.65	5.00	0.00	0.00	0.00
66.00	2.00	0.64	5.00	0.00	0.00	0.00
67.00	2.00	0.63	5.00	0.00	0.00	0.00
68.00	2.00	0.62	5.00	0.00	0.00	0.00
69.00	2.00	0.62	5.00	0.00	0.00	0.00
70.00	2.00	0.61	5.00	0.00	0.00	0.00
71.00	2.00	0.60	5.00	0.00	0.00	0.00
72.00	2.00	0.60	5.00	0.00	0.00	0.00
73.00	2.00	0.59	5.00	0.00	0.00	0.00
74.00	2.00	0.58	5.00	0.00	0.00	0.00
75.00	2.00	0.57	5.00	0.00	0.00	0.00
76.00	2.00	0.57	5.00	0.00	0.00	0.00
77.00	2.00	0.57	5.00	0.00	0.00	0.00
78.00	2.00	0.57	5.00	0.00	0.00	0.00
79.00	2.00	0.56	5.00	0.00	0.00	0.00
80.00	2.00	0.56	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

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1 atm (atmosphere) = 1 tsf (ton/ft2)

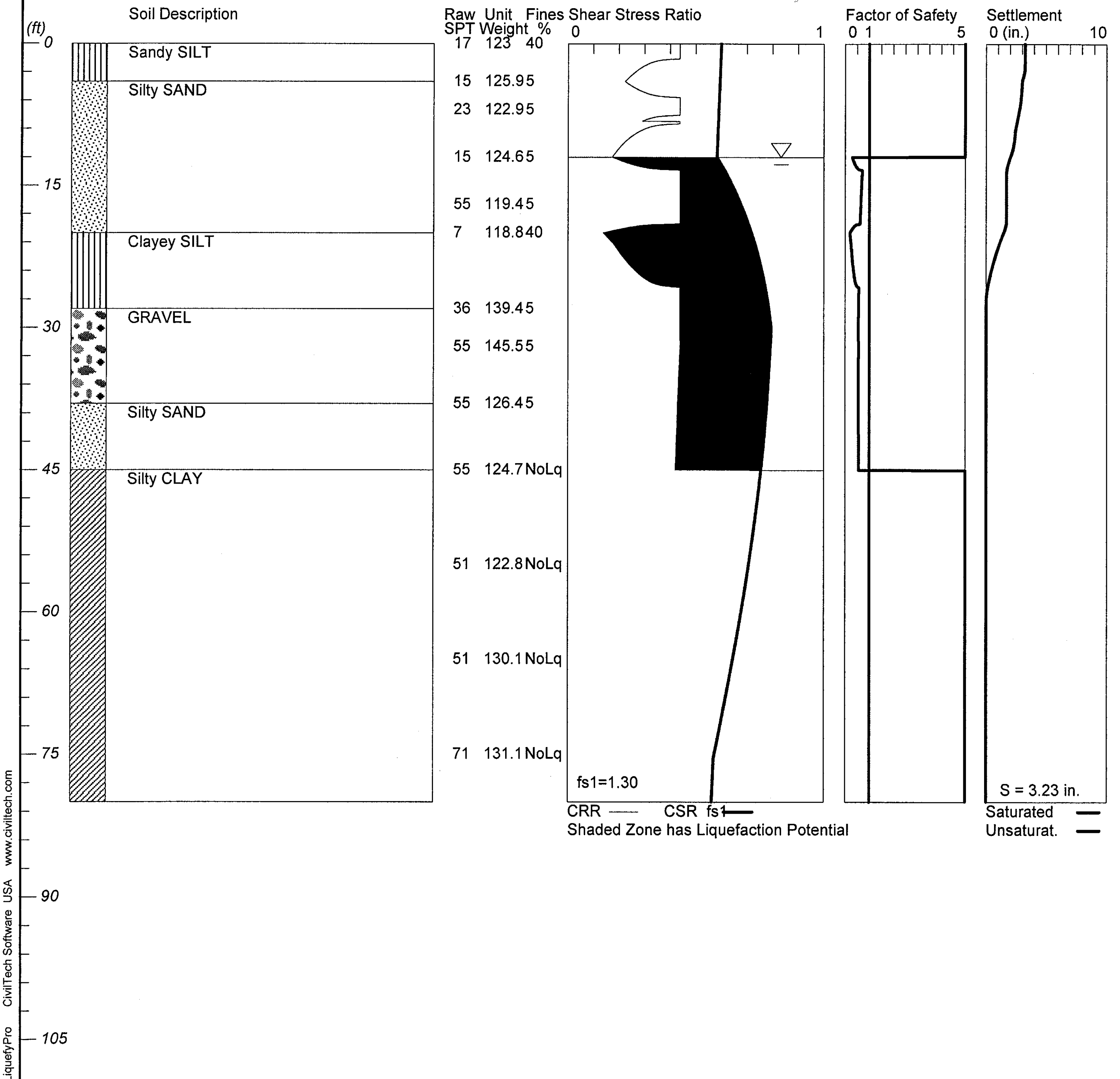
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

SV1302A - Proposed Lincoln Landing

Hole No.=B-3   Water Depth=12 ft   Surface Elev.=100

Magnitude=7.9
Acceleration=0.71g



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LIQUEFACTION ANALYSIS SUMMARY

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Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 4/27/2015 5:58:49 PM

Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV (1300-1309)\SV1302 - Foothill Blvd. - Hayward\SV1302A.GI\SV1302A. Lique. CPT-01.liq
Title: SV1302A - Proposed Lincoln Landing
Subtitle: 22031 Foothill Blvd., Hayward, CA

Surface Elev.=100
Hole No.=CPT-01
Depth of Hole= 80.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.71 g
Earthquake Magnitude= 7.90

Input Data:
Surface Elev.=100
Hole No.=CPT-01
Depth of Hole=80.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.71 g
Earthquake Magnitude=7.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. CPT Calculation Method: Modify Robertson*
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
9. User request factor of safety (apply to CSR) , User= 1.3
Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	qc atm	fs atm	Rf pcf	gamma %	Fines mm	D50
0.00	11.78	0.50	4.22	111.00	NoLiq	0.50
4.92	33.17	1.04	3.14	115.00	NoLiq	0.50
5.91	46.31	1.58	3.42	115.00	NoLiq	0.50
6.89	84.66	3.20	3.78	115.00	NoLiq	0.50
7.87	67.34	2.05	3.04	115.00	NoLiq	0.50
8.86	30.02	1.54	5.12	111.00	NoLiq	0.50
9.84	50.51	2.04	4.04	115.00	NoLiq	0.50
10.82	25.13	1.43	5.69	111.00	NoLiq	0.50
11.81	53.88	2.73	5.06	115.00	NoLiq	0.50
12.79	21.00	0.95	4.51	111.00	NoLiq	0.50
13.78	163.40	4.28	2.62	118.00	NoLiq	0.50
14.76	41.29	2.11	5.10	115.00	NoLiq	0.50
15.74	69.34	1.27	1.84	118.00	NoLiq	0.50

16.73	14.81	0.32	2.17	115.00	NoLiq	0.50
17.71	15.73	0.46	2.94	115.00	NoLiq	0.50
18.70	14.67	0.51	3.50	115.00	NoLiq	0.50
19.68	189.20	0.95	0.50	124.00	NoLiq	0.50
20.66	64.45	0.53	0.82	124.00	0.00	0.50
21.98	49.90	1.35	2.71	115.00	0.00	0.50
22.96	208.10	3.13	1.51	124.00	0.00	0.50
23.95	204.60	2.56	1.25	124.00	0.00	0.50
24.93	58.75	2.02	3.44	115.00	0.00	0.50
25.91	53.90	1.81	3.36	115.00	0.00	0.50
26.90	126.90	3.09	2.44	115.00	NoLiq	0.50
27.88	72.21	3.71	5.14	131.00	NoLiq	0.50
28.87	73.88	4.11	5.56	131.00	NoLiq	0.50
29.85	34.21	1.19	3.47	115.00	NoLiq	0.50
30.84	28.41	1.02	3.60	115.00	NoLiq	0.50
31.82	24.50	1.21	4.94	111.00	NoLiq	0.50
32.80	34.62	1.10	3.18	115.00	NoLiq	0.50
33.79	32.90	1.23	3.75	115.00	NoLiq	0.50
34.77	56.57	2.00	3.54	115.00	NoLiq	0.50
35.76	30.27	0.90	2.97	115.00	NoLiq	0.50
36.74	24.97	0.69	2.78	115.00	NoLiq	0.50
37.73	23.42	0.65	2.78	115.00	NoLiq	0.50
38.71	22.80	0.68	2.98	115.00	NoLiq	0.50
39.69	22.68	0.61	2.71	115.00	NoLiq	0.50
40.68	28.08	0.85	3.02	115.00	NoLiq	0.50
41.66	77.69	3.21	4.13	115.00	NoLiq	0.50
42.65	58.35	3.19	5.46	131.00	NoLiq	0.50
43.63	107.10	4.35	4.06	131.00	NoLiq	0.50
44.61	82.50	4.43	5.37	131.00	NoLiq	0.50
45.60	100.60	4.44	4.42	115.00	NoLiq	0.50
46.58	111.20	5.69	5.11	131.00	NoLiq	0.50
47.57	128.40	4.34	3.38	115.00	NoLiq	0.50
48.55	25.92	1.40	5.39	111.00	NoLiq	0.50
49.54	28.73	0.91	3.16	115.00	NoLiq	0.50
50.52	31.24	0.94	3.00	115.00	NoLiq	0.50
51.50	38.30	1.17	3.07	115.00	NoLiq	0.50
52.49	27.87	1.15	4.11	115.00	NoLiq	0.50
53.47	45.47	1.54	3.39	115.00	NoLiq	0.50
54.46	35.25	1.16	3.29	115.00	NoLiq	0.50
55.44	33.67	1.44	4.27	115.00	NoLiq	0.50
56.43	31.85	1.29	4.06	115.00	NoLiq	0.50
57.41	32.13	1.28	3.99	115.00	NoLiq	0.50
58.39	32.69	1.21	3.70	115.00	NoLiq	0.50
59.38	31.98	1.39	4.34	115.00	NoLiq	0.50
60.36	30.72	1.25	4.06	115.00	NoLiq	0.50
61.35	27.69	1.15	4.15	115.00	NoLiq	0.50
62.33	29.64	1.24	4.17	115.00	NoLiq	0.50
63.32	33.73	1.24	3.68	115.00	NoLiq	0.50
64.30	33.70	1.26	3.73	115.00	NoLiq	0.50
65.28	31.75	1.10	3.46	115.00	NoLiq	0.50
66.27	34.66	1.02	2.96	115.00	NoLiq	0.50
67.25	35.78	1.10	3.07	115.00	NoLiq	0.50
68.24	43.13	1.28	2.97	115.00	NoLiq	0.50
69.22	39.30	1.38	3.50	115.00	NoLiq	0.50
70.21	44.04	1.37	3.12	115.00	NoLiq	0.50
71.19	49.08	1.41	2.87	115.00	NoLiq	0.50
72.17	39.15	1.26	3.21	115.00	NoLiq	0.50
73.16	39.13	1.10	2.81	115.00	NoLiq	0.50
74.14	35.91	1.04	2.89	115.00	NoLiq	0.50
75.13	72.39	1.36	1.88	115.00	NoLiq	0.50
76.11	41.57	7.10	17.09	115.00	NoLiq	0.50
77.10	91.64	5.44	5.93	131.00	NoLiq	0.50
78.08	35.34	0.76	2.16	115.00	NoLiq	0.50

79.06	165.20	1.93	1.17	124.00	NoLiq	0.50

Modify Robertson method generates Fines from qc/fs. Inputted Fines are not relevant.

Output Results:
Settlement of Saturated Sands=0.59 in.
Settlement of Unsaturated Sands=0.00 in.
Total Settlement of Saturated and Unsaturated Sands=0.59 in.
Differential Settlement=0.293 to 0.387 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.60	5.00	0.59	0.00	0.59
1.00	2.00	0.60	5.00	0.59	0.00	0.59
2.00	2.00	0.60	5.00	0.59	0.00	0.59
3.00	2.00	0.60	5.00	0.59	0.00	0.59
4.00	2.00	0.59	5.00	0.59	0.00	0.59
5.00	2.00	0.59	5.00	0.59	0.00	0.59
6.00	2.00	0.59	5.00	0.59	0.00	0.59
7.00	2.00	0.59	5.00	0.59	0.00	0.59
8.00	2.00	0.59	5.00	0.59	0.00	0.59
9.00	2.00	0.59	5.00	0.59	0.00	0.59
10.00	2.00	0.59	5.00	0.59	0.00	0.59
11.00	2.00	0.58	5.00	0.59	0.00	0.59
12.00	2.00	0.58	5.00	0.59	0.00	0.59
13.00	2.00	0.61	5.00	0.59	0.00	0.59
14.00	2.00	0.63	5.00	0.59	0.00	0.59
15.00	2.00	0.65	5.00	0.59	0.00	0.59
16.00	2.00	0.67	5.00	0.59	0.00	0.59
17.00	2.00	0.69	5.00	0.59	0.00	0.59
18.00	2.00	0.70	5.00	0.59	0.00	0.59
19.00	2.00	0.72	5.00	0.59	0.00	0.59
20.00	2.00	0.73	5.00	0.59	0.00	0.59
21.00	0.12	0.74	0.17*	0.34	0.00	0.34
22.00	0.22	0.76	0.29*	0.12	0.00	0.12
23.00	0.87	0.77	1.14	0.04	0.00	0.04
24.00	0.69	0.78	0.89*	0.04	0.00	0.04
25.00	0.39	0.78	0.50*	0.00	0.00	0.00
26.00	0.33	0.79	0.41*	0.00	0.00	0.00
27.00	2.00	0.80	5.00	0.00	0.00	0.00
28.00	2.00	0.81	5.00	0.00	0.00	0.00
29.00	2.00	0.82	5.00	0.00	0.00	0.00
30.00	2.00	0.82	5.00	0.00	0.00	0.00
31.00	2.00	0.82	5.00	0.00	0.00	0.00
32.00	2.00	0.82	5.00	0.00	0.00	0.00
33.00	2.00	0.82	5.00	0.00	0.00	0.00
34.00	2.00	0.82	5.00	0.00	0.00	0.00
35.00	2.00	0.82	5.00	0.00	0.00	0.00
36.00	2.00	0.82	5.00	0.00	0.00	0.00
37.00	2.00	0.82	5.00	0.00	0.00	0.00
38.00	2.00	0.82	5.00	0.00	0.00	0.00
39.00	2.00	0.82	5.00	0.00	0.00	0.00
40.00	2.00	0.82	5.00	0.00	0.00	0.00
41.00	2.00	0.81	5.00	0.00	0.00	0.00
42.00	2.00	0.81	5.00	0.00	0.00	0.00
43.00	2.00	0.81	5.00	0.00	0.00	0.00
44.00	2.00	0.80	5.00	0.00	0.00	0.00
45.00	2.00	0.80	5.00	0.00	0.00	0.00
46.00	2.00	0.79	5.00	0.00	0.00	0.00
47.00	2.00	0.79	5.00	0.00	0.00	0.00
48.00	2.00	0.78	5.00	0.00	0.00	0.00

49.00	2.00	0.78	5.00	0.00	0.00	0.00
50.00	2.00	0.77	5.00	0.00	0.00	0.00
51.00	2.00	0.77	5.00	0.00	0.00	0.00
52.00	2.00	0.76	5.00	0.00	0.00	0.00
53.00	2.00	0.76	5.00	0.00	0.00	0.00
54.00	2.00	0.75	5.00	0.00	0.00	0.00
55.00	2.00	0.75	5.00	0.00	0.00	0.00
56.00	2.00	0.74	5.00	0.00	0.00	0.00
57.00	2.00	0.74	5.00	0.00	0.00	0.00
58.00	2.00	0.73	5.00	0.00	0.00	0.00
59.00	2.00	0.72	5.00	0.00	0.00	0.00
60.00	2.00	0.72	5.00	0.00	0.00	0.00
61.00	2.00	0.71	5.00	0.00	0.00	0.00
62.00	2.00	0.71	5.00	0.00	0.00	0.00
63.00	2.00	0.70	5.00	0.00	0.00	0.00
64.00	2.00	0.69	5.00	0.00	0.00	0.00
65.00	2.00	0.69	5.00	0.00	0.00	0.00
66.00	2.00	0.68	5.00	0.00	0.00	0.00
67.00	2.00	0.67	5.00	0.00	0.00	0.00
68.00	2.00	0.67	5.00	0.00	0.00	0.00
69.00	2.00	0.66	5.00	0.00	0.00	0.00
70.00	2.00	0.65	5.00	0.00	0.00	0.00
71.00	2.00	0.64	5.00	0.00	0.00	0.00
72.00	2.00	0.64	5.00	0.00	0.00	0.00
73.00	2.00	0.63	5.00	0.00	0.00	0.00
74.00	2.00	0.62	5.00	0.00	0.00	0.00
75.00	2.00	0.62	5.00	0.00	0.00	0.00
76.00	2.00	0.61	5.00	0.00	0.00	0.00
77.00	2.00	0.61	5.00	0.00	0.00	0.00
78.00	2.00	0.61	5.00	0.00	0.00	0.00
79.00	2.00	0.61	5.00	0.00	0.00	0.00
80.00	0.20	0.60	0.34*	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

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1 atm (atmosphere) = 1 tsf (ton/ft2)

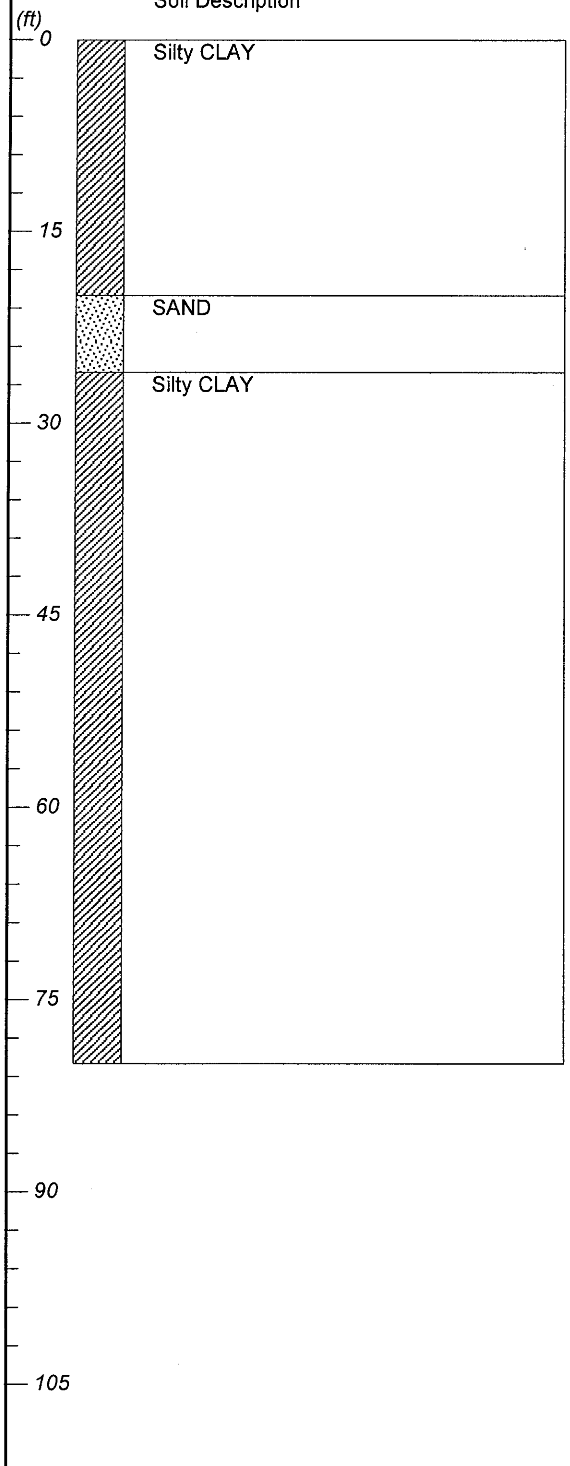
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

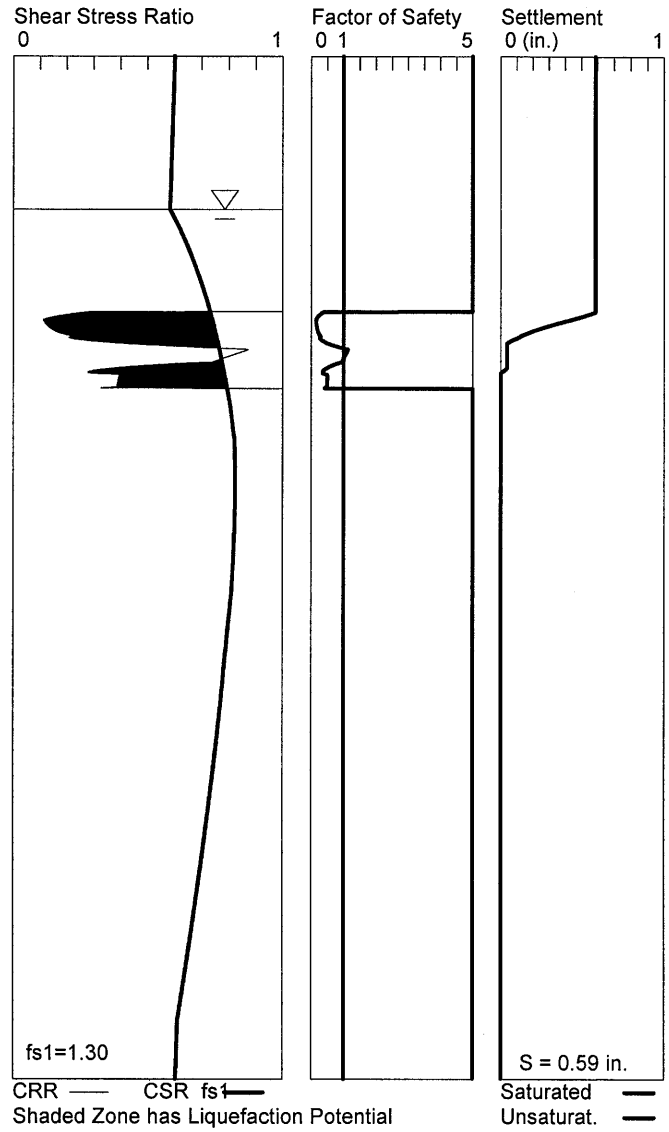
SV1302A - Proposed Lincoln Landing

Hole No.=CPT-01 Water Depth=12 ft Surface Elev.=100

Magnitude=7.9
Acceleration=0.71g



Raw qc	fc	Unit Weight	Fines %
11.78	0.497	111	NoLq
33.17	1.042	115	NoLq
46.31	1.583	115	NoLq
84.66	3.196	115	NoLq
67.34	2.050	115	NoLq
30.02	1.537	111	NoLq
50.51	2.042	115	NoLq
25.13	1.431	111	NoLq
53.88	2.728	115	NoLq
21.00	0.947	111	NoLq
163.4	4.276	118	NoLq
41.29	2.105	115	NoLq
69.34	1.274	118	NoLq
14.81	0.321	115	NoLq
15.73	0.463	115	NoLq
14.67	0.514	115	NoLq
189.2	0.949	124	NoLq
64.45	0.531	124	5
49.90	1.351	115	101
208.1	3.134	124	5
204.6	2.561	124	5
58.75	2.02	115	101
53.90	1.81	115	101
126.9	3.094	115	NoLq
72.21	3.712	131	NoLq
73.88	4.106	131	NoLq
34.21	1.188	115	NoLq
28.41	1.022	115	NoLq
24.50	1.21	111	NoLq
34.62	1.1	115	NoLq
32.90	1.233	115	NoLq
56.57	2.0	115	NoLq
30.27	0.898	115	NoLq
24.97	0.695	115	NoLq
23.42	0.652	115	NoLq
22.80	0.679	115	NoLq
22.68	0.614	115	NoLq
28.08	0.849	115	NoLq
77.69	3.212	115	NoLq
58.35	3.187	131	NoLq
107.1	4.352	131	NoLq
82.50	4.431	131	NoLq
100.6	4.444	115	NoLq
111.2	5.686	131	NoLq
128.4	4.34	115	NoLq
25.92	1.398	111	NoLq
28.73	0.908	115	NoLq
31.24	0.937	115	NoLq
38.30	1.175	115	NoLq
27.87	1.146	115	NoLq
45.47	1.543	115	NoLq
35.25	1.161	115	NoLq
33.67	1.439	115	NoLq
31.85	1.294	115	NoLq
32.13	1.281	115	NoLq
32.69	1.209	115	NoLq
31.98	1.389	115	NoLq
30.72	1.246	115	NoLq
27.69	1.148	115	NoLq
29.64	1.235	115	NoLq
33.73	1.241	115	NoLq
33.70	1.257	115	NoLq
31.75	1.097	115	NoLq
34.66	1.025	115	NoLq
35.78	1.097	115	NoLq
43.13	1.281	115	NoLq
39.30	1.377	115	NoLq
44.04	1.372	115	NoLq
49.08	1.411	115	NoLq
39.15	1.257	115	NoLq
39.13	1.1	115	NoLq
35.91	1.039	115	NoLq
72.39	1.359	115	NoLq
41.57	7.104	115	NoLq
91.64	5.438	131	NoLq
35.34	0.762	115	NoLq
165.2	1.926	124	NoLq



LiquefyPro CivilTech Software USA www.civiltech.com

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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV (1300-1309)\SV1302 - Foothill Blvd. - Hayward\SV1302A.GI\SV1302A. Lique. CPT-02.liq
Title: SV1302A - Proposed Lincoln Landing
Subtitle: 22031 Foothill Blvd., Hayward, CA

Surface Elev.=100
Hole No.=CPT-02
Depth of Hole= 80.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.71 g
Earthquake Magnitude= 7.90

Input Data:
Surface Elev.=100
Hole No.=CPT-02
Depth of Hole=80.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.71 g
Earthquake Magnitude=7.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. CPT Calculation Method: Modify Robertson*
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	qc atm	fs atm	Rf pcf	gamma %	Fines mm	D50
0.00	5.72	0.03	0.45	115.00	0.00	0.50
4.92	11.60	0.22	1.87	115.00	0.00	0.50
5.91	26.87	0.20	0.74	118.00	0.00	0.50
6.89	16.91	0.02	0.12	118.00	0.00	0.50
8.86	21.28	0.13	0.62	118.00	0.00	0.50
10.49	48.98	0.46	0.94	118.00	0.00	0.50
11.48	94.53	0.69	0.73	124.00	0.00	0.50
13.45	171.30	0.63	0.37	124.00	0.00	0.50
15.09	210.80	1.39	0.66	124.00	0.00	0.50
16.73	251.90	1.63	0.65	124.00	0.00	0.50
18.37	225.90	1.11	0.49	124.00	0.00	0.50
20.01	68.64	0.67	0.98	121.00	0.00	0.50
21.65	17.24	0.34	1.96	115.00	0.00	0.50

23.29	115.60	2.27	1.97	118.00	0.00	0.50
24.93	95.19	1.89	1.98	118.00	0.00	0.50
26.57	211.30	1.42	0.67	124.00	0.00	0.50
28.21	226.20	1.22	0.54	124.00	NoLiq	0.50
29.85	127.00	3.70	2.91	115.00	NoLiq	0.50
31.49	28.25	0.62	2.20	115.00	NoLiq	0.50
33.13	26.58	0.60	2.24	115.00	NoLiq	0.50
34.77	28.52	0.69	2.40	115.00	NoLiq	0.50
36.41	39.18	1.25	3.19	115.00	NoLiq	0.50
38.05	77.79	4.17	5.36	131.00	NoLiq	0.50
39.68	64.65	3.09	4.78	131.00	NoLiq	0.50
41.33	52.23	2.79	5.33	131.00	NoLiq	0.50
42.97	75.19	3.59	4.77	131.00	NoLiq	0.50
44.61	33.33	2.37	7.11	115.00	NoLiq	0.50
46.26	29.23	1.14	3.90	115.00	NoLiq	0.50
47.90	34.79	1.19	3.41	115.00	NoLiq	0.50
49.54	38.29	1.28	3.34	115.00	NoLiq	0.50
51.18	30.05	1.41	4.70	115.00	NoLiq	0.50
52.82	26.49	1.34	5.07	115.00	NoLiq	0.50
54.46	30.97	1.46	4.70	115.00	NoLiq	0.50
56.10	29.25	1.65	5.65	111.00	NoLiq	0.50
57.74	29.88	1.41	4.74	111.00	NoLiq	0.50
59.38	31.42	1.22	3.87	115.00	NoLiq	0.50
61.02	36.61	1.23	3.35	115.00	NoLiq	0.50
62.66	34.13	1.12	3.27	115.00	NoLiq	0.50
64.30	47.03	2.13	4.53	115.00	NoLiq	0.50
65.94	43.80	1.42	3.25	115.00	NoLiq	0.50
67.58	35.06	0.99	2.82	115.00	NoLiq	0.50
69.22	33.50	0.94	2.81	115.00	NoLiq	0.50
70.86	46.44	1.92	4.14	115.00	NoLiq	0.50
72.50	71.61	4.38	6.11	131.00	NoLiq	0.50
74.14	42.97	1.39	3.23	115.00	NoLiq	0.50
75.78	48.70	1.70	3.50	115.00	NoLiq	0.50
77.42	47.48	1.62	3.42	115.00	NoLiq	0.50
79.06	43.92	0.54	1.22	115.00	NoLiq	0.50

Modify Robertson method generates Fines from qc/fs. Inputted Fines are not relevant.

Output Results:

Settlement of Saturated Sands=1.56 in.
Settlement of Unsaturated Sands=3.34 in.
Total Settlement of Saturated and Unsaturated Sands=4.90 in.
Differential Settlement=2.450 to 3.234 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	1.82	0.60	5.00	1.56	3.34	4.90
1.00	0.09	0.60	5.00	1.56	3.33	4.89
2.00	0.10	0.60	5.00	1.56	3.31	4.87
3.00	0.12	0.60	5.00	1.56	3.21	4.77
4.00	0.16	0.59	5.00	1.56	3.01	4.57
5.00	0.13	0.59	5.00	1.56	2.95	4.51
6.00	0.09	0.59	5.00	1.56	2.61	4.17
7.00	0.07	0.59	5.00	1.56	2.00	3.56
8.00	0.08	0.59	5.00	1.56	1.28	2.84
9.00	0.08	0.59	5.00	1.56	0.71	2.27
10.00	0.11	0.59	5.00	1.56	0.32	1.88
11.00	0.17	0.58	5.00	1.56	0.11	1.67
12.00	0.30	0.58	0.52*	1.56	0.00	1.56
13.00	0.55	0.61	0.91*	1.42	0.00	1.42
14.00	0.81	0.63	1.29	1.41	0.00	1.41

15.00	1.03	0.65	1.59	1.41	0.00	1.41
16.00	1.29	0.66	1.94	1.41	0.00	1.41
17.00	1.39	0.68	2.04	1.41	0.00	1.41
18.00	1.06	0.70	1.52	1.41	0.00	1.41
19.00	0.40	0.71	0.57*	1.39	0.00	1.39
20.00	0.12	0.72	0.17*	1.16	0.00	1.16
21.00	0.10	0.73	0.14*	0.87	0.00	0.87
22.00	0.14	0.75	0.19*	0.72	0.00	0.72
23.00	0.26	0.76	0.35*	0.51	0.00	0.51
24.00	0.29	0.77	0.38*	0.39	0.00	0.39
25.00	0.25	0.78	0.32*	0.23	0.00	0.23
26.00	0.40	0.78	0.51*	0.06	0.00	0.06
27.00	0.61	0.79	0.76*	0.01	0.00	0.01
28.00	0.66	0.80	0.82*	0.00	0.00	0.00
29.00	2.00	0.81	5.00	0.00	0.00	0.00
30.00	2.00	0.81	5.00	0.00	0.00	0.00
31.00	2.00	0.81	5.00	0.00	0.00	0.00
32.00	2.00	0.81	5.00	0.00	0.00	0.00
33.00	2.00	0.81	5.00	0.00	0.00	0.00
34.00	2.00	0.81	5.00	0.00	0.00	0.00
35.00	2.00	0.81	5.00	0.00	0.00	0.00
36.00	2.00	0.81	5.00	0.00	0.00	0.00
37.00	2.00	0.81	5.00	0.00	0.00	0.00
38.00	2.00	0.81	5.00	0.00	0.00	0.00
39.00	2.00	0.81	5.00	0.00	0.00	0.00
40.00	2.00	0.80	5.00	0.00	0.00	0.00
41.00	2.00	0.80	5.00	0.00	0.00	0.00
42.00	2.00	0.79	5.00	0.00	0.00	0.00
43.00	2.00	0.79	5.00	0.00	0.00	0.00
44.00	2.00	0.79	5.00	0.00	0.00	0.00
45.00	2.00	0.78	5.00	0.00	0.00	0.00
46.00	2.00	0.78	5.00	0.00	0.00	0.00
47.00	2.00	0.77	5.00	0.00	0.00	0.00
48.00	2.00	0.77	5.00	0.00	0.00	0.00
49.00	2.00	0.77	5.00	0.00	0.00	0.00
50.00	2.00	0.76	5.00	0.00	0.00	0.00
51.00	2.00	0.76	5.00	0.00	0.00	0.00
52.00	2.00	0.75	5.00	0.00	0.00	0.00
53.00	2.00	0.75	5.00	0.00	0.00	0.00
54.00	2.00	0.74	5.00	0.00	0.00	0.00
55.00	2.00	0.74	5.00	0.00	0.00	0.00
56.00	2.00	0.73	5.00	0.00	0.00	0.00
57.00	2.00	0.73	5.00	0.00	0.00	0.00
58.00	2.00	0.72	5.00	0.00	0.00	0.00
59.00	2.00	0.72	5.00	0.00	0.00	0.00
60.00	2.00	0.71	5.00	0.00	0.00	0.00
61.00	2.00	0.70	5.00	0.00	0.00	0.00
62.00	2.00	0.70	5.00	0.00	0.00	0.00
63.00	2.00	0.69	5.00	0.00	0.00	0.00
64.00	2.00	0.68	5.00	0.00	0.00	0.00
65.00	2.00	0.68	5.00	0.00	0.00	0.00
66.00	2.00	0.67	5.00	0.00	0.00	0.00
67.00	2.00	0.66	5.00	0.00	0.00	0.00
68.00	2.00	0.66	5.00	0.00	0.00	0.00
69.00	2.00	0.65	5.00	0.00	0.00	0.00
70.00	2.00	0.64	5.00	0.00	0.00	0.00
71.00	2.00	0.64	5.00	0.00	0.00	0.00
72.00	2.00	0.63	5.00	0.00	0.00	0.00
73.00	2.00	0.62	5.00	0.00	0.00	0.00
74.00	2.00	0.61	5.00	0.00	0.00	0.00
75.00	2.00	0.61	5.00	0.00	0.00	0.00
76.00	2.00	0.60	5.00	0.00	0.00	0.00
77.00	2.00	0.60	5.00	0.00	0.00	0.00

78.00	2.00	0.60	5.00	0.00	0.00	0.00
79.00	2.00	0.60	5.00	0.00	0.00	0.00
80.00	2.00	0.60	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

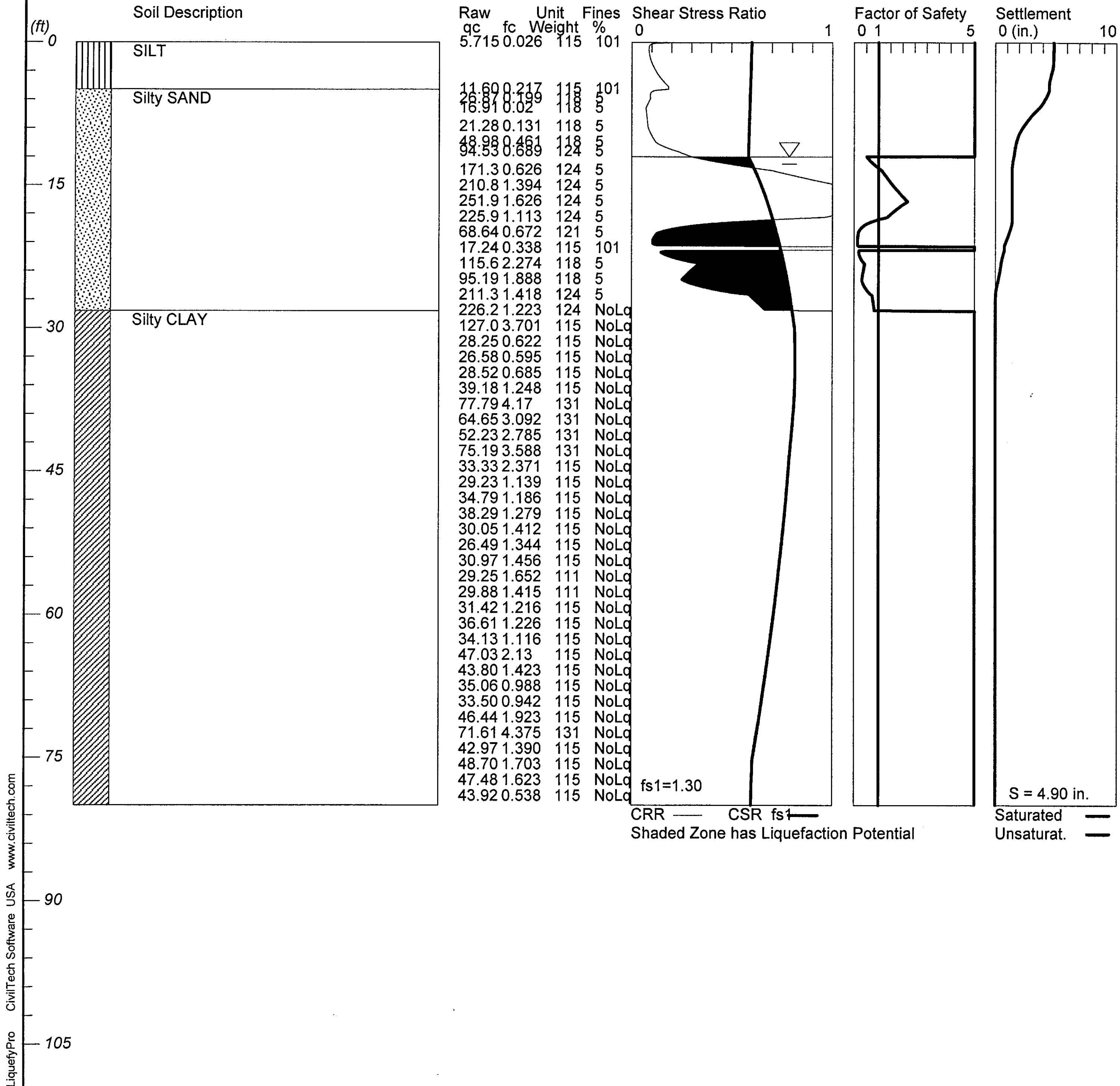
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

SV1302A - Proposed Lincoln Landing

Hole No.=CPT-02   Water Depth=12 ft   Surface Elev.=100

Magnitude=7.9
Acceleration=0.71g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV (1300-1309)\SV1302 - Foothill Blvd. - Hayward\SV1302A.GI\SV1302A. Lique. CPT-03.liq
Title: SV1302A - Proposed Lincoln Landing
Subtitle: 22031 Foothill Blvd., Hayward, CA

Surface Elev.=100
Hole No.=CPT-03
Depth of Hole= 69.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.71 g
Earthquake Magnitude= 7.90

Input Data:
Surface Elev.=100
Hole No.=CPT-03
Depth of Hole=69.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.71 g
Earthquake Magnitude=7.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. CPT Calculation Method: Modify Robertson*
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	qc atm	fs atm	Rf pcf	gamma %	Fines mm	D50
0.00	10.91	0.19	1.72	115.00	0.00	0.50
4.92	117.28	0.73	0.62	124.00	0.00	0.50
6.23	31.19	0.16	0.51	121.00	0.00	0.50
7.87	34.67	0.24	0.68	118.00	0.00	0.50
9.51	46.49	0.25	0.53	121.00	0.00	0.50
11.15	49.16	0.22	0.45	121.00	0.00	0.50
12.80	54.09	0.28	0.51	121.00	0.00	0.50
14.44	81.80	0.45	0.55	121.00	0.00	0.50
16.08	139.26	0.97	0.69	124.00	0.00	0.50
17.72	65.53	0.25	0.38	121.00	0.00	0.50
19.36	68.17	0.34	0.49	121.00	0.00	0.50
21.00	87.18	0.28	0.33	124.00	0.00	0.50
22.64	71.50	0.45	0.63	121.00	0.00	0.50

24.28	60.32	0.63	1.05	121.00	0.00	0.50
25.92	90.57	0.54	0.59	124.00	0.00	0.50
27.56	31.24	0.71	2.26	115.00	0.00	0.50
29.20	36.02	0.60	1.67	121.00	NoLiq	0.50
30.84	15.99	0.35	2.16	115.00	0.00	0.50
32.48	23.29	0.60	2.58	115.00	0.00	0.50
34.12	14.12	0.39	2.73	115.00	NoLiq	0.50
35.76	76.71	0.50	0.65	121.00	NoLiq	0.50
37.40	71.40	0.75	1.05	124.00	0.00	0.50
39.04	111.56	0.98	0.88	124.00	0.00	0.50
40.68	249.86	1.85	0.74	124.00	0.00	0.50
42.32	303.54	1.77	0.58	127.00	0.00	0.50
43.96	48.67	0.86	1.77	124.00	0.00	0.50
45.60	54.23	1.25	2.30	115.00	0.00	0.50
47.24	42.57	1.09	2.55	115.00	NoLiq	0.50
48.88	42.26	1.16	2.75	115.00	NoLiq	0.50
50.53	37.69	0.81	2.14	115.00	NoLiq	0.50
52.17	49.10	1.60	3.26	115.00	NoLiq	0.50
53.81	125.03	5.91	4.73	131.00	NoLiq	0.50
55.45	348.56	8.52	2.44	121.00	NoLiq	0.50
57.09	158.59	3.99	2.52	124.00	NoLiq	0.50
58.73	172.67	3.99	2.31	131.00	NoLiq	0.50
60.37	150.00	7.96	5.31	131.00	NoLiq	0.50
62.01	179.23	10.80	6.03	131.00	NoLiq	0.50
63.65	166.09	7.44	4.48	131.00	NoLiq	0.50
65.29	287.58	14.45	5.02	131.00	NoLiq	0.50
66.63	505.35	7.80	1.54	124.00	NoLiq	0.50
68.57	467.13	4.06	0.87	127.00	0.00	0.50

Modify Robertson method generates Fines from qc/fs. Inputted Fines are not relevant.

Output Results:
Settlement of Saturated Sands=5.99 in.
Settlement of Unsaturated Sands=2.61 in.
Total Settlement of Saturated and Unsaturated Sands=8.60 in.
Differential Settlement=4.301 to 5.678 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.60	5.00	5.99	2.61	8.60
1.00	0.34	0.60	5.00	5.99	2.61	8.60
2.00	0.44	0.60	5.00	5.99	2.60	8.60
3.00	0.59	0.60	5.00	5.99	2.59	8.59
4.00	0.78	0.59	5.00	5.99	2.55	8.54
5.00	0.83	0.59	5.00	5.99	2.53	8.52
6.00	0.13	0.59	5.00	5.99	2.40	8.40
7.00	0.09	0.59	5.00	5.99	1.99	7.98
8.00	0.10	0.59	5.00	5.99	1.58	7.57
9.00	0.10	0.59	5.00	5.99	1.18	7.17
10.00	0.10	0.59	5.00	5.99	0.81	6.80
11.00	0.10	0.58	5.00	5.99	0.41	6.40
12.00	0.10	0.58	0.18*	5.99	0.00	5.99
13.00	0.11	0.61	0.18*	5.67	0.00	5.67
14.00	0.13	0.63	0.21*	5.36	0.00	5.36
15.00	0.20	0.65	0.31*	5.10	0.00	5.10
16.00	0.33	0.66	0.49*	4.88	0.00	4.88
17.00	0.17	0.68	0.25*	4.67	0.00	4.67
18.00	0.10	0.69	0.15*	4.36	0.00	4.36
19.00	0.10	0.71	0.15*	4.03	0.00	4.03
20.00	0.11	0.72	0.15*	3.71	0.00	3.71
21.00	0.12	0.73	0.16*	3.39	0.00	3.39

22.00	0.11	0.74	0.15*	3.07	0.00	3.07
23.00	0.11	0.75	0.15*	2.76	0.00	2.76
24.00	0.11	0.76	0.14*	2.46	0.00	2.46
25.00	0.12	0.77	0.15*	2.16	0.00	2.16
26.00	0.13	0.78	0.16*	1.87	0.00	1.87
27.00	0.11	0.79	0.14*	1.57	0.00	1.57
28.00	2.00	0.80	5.00	1.45	0.00	1.45
29.00	2.00	0.80	5.00	1.45	0.00	1.45
30.00	2.00	0.81	5.00	1.45	0.00	1.45
31.00	2.00	0.81	5.00	1.45	0.00	1.45
32.00	2.00	0.81	5.00	1.45	0.00	1.45
33.00	2.00	0.81	5.00	1.45	0.00	1.45
34.00	2.00	0.81	5.00	1.45	0.00	1.45
35.00	2.00	0.81	5.00	1.45	0.00	1.45
36.00	2.00	0.81	5.00	1.45	0.00	1.45
37.00	2.00	0.81	5.00	1.45	0.00	1.45
38.00	0.13	0.80	0.16*	1.19	0.00	1.19
39.00	0.16	0.80	0.20*	0.93	0.00	0.93
40.00	0.36	0.80	0.45*	0.71	0.00	0.71
41.00	0.70	0.80	0.88*	0.65	0.00	0.65
42.00	0.94	0.79	1.19	0.65	0.00	0.65
43.00	0.35	0.79	0.44*	0.62	0.00	0.62
44.00	0.13	0.78	0.16*	0.39	0.00	0.39
45.00	0.16	0.78	0.20*	0.17	0.00	0.17
46.00	2.00	0.78	5.00	0.00	0.00	0.00
47.00	2.00	0.77	5.00	0.00	0.00	0.00
48.00	2.00	0.77	5.00	0.00	0.00	0.00
49.00	2.00	0.76	5.00	0.00	0.00	0.00
50.00	2.00	0.76	5.00	0.00	0.00	0.00
51.00	2.00	0.76	5.00	0.00	0.00	0.00
52.00	2.00	0.75	5.00	0.00	0.00	0.00
53.00	2.00	0.75	5.00	0.00	0.00	0.00
54.00	2.00	0.74	5.00	0.00	0.00	0.00
55.00	2.00	0.73	5.00	0.00	0.00	0.00
56.00	2.00	0.73	5.00	0.00	0.00	0.00
57.00	2.00	0.72	5.00	0.00	0.00	0.00
58.00	2.00	0.71	5.00	0.00	0.00	0.00
59.00	2.00	0.71	5.00	0.00	0.00	0.00
60.00	2.00	0.70	5.00	0.00	0.00	0.00
61.00	2.00	0.69	5.00	0.00	0.00	0.00
62.00	2.00	0.69	5.00	0.00	0.00	0.00
63.00	2.00	0.68	5.00	0.00	0.00	0.00
64.00	2.00	0.67	5.00	0.00	0.00	0.00
65.00	2.00	0.67	5.00	0.00	0.00	0.00
66.00	2.00	0.66	5.00	0.00	0.00	0.00
67.00	2.00	0.65	5.00	0.00	0.00	0.00
68.00	1.67	0.64	2.59	0.00	0.00	0.00
69.00	1.66	0.64	2.61	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

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1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils
CSRsf Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S. Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat Settlement from saturated sands
S_dry Settlement from Unsaturated Sands

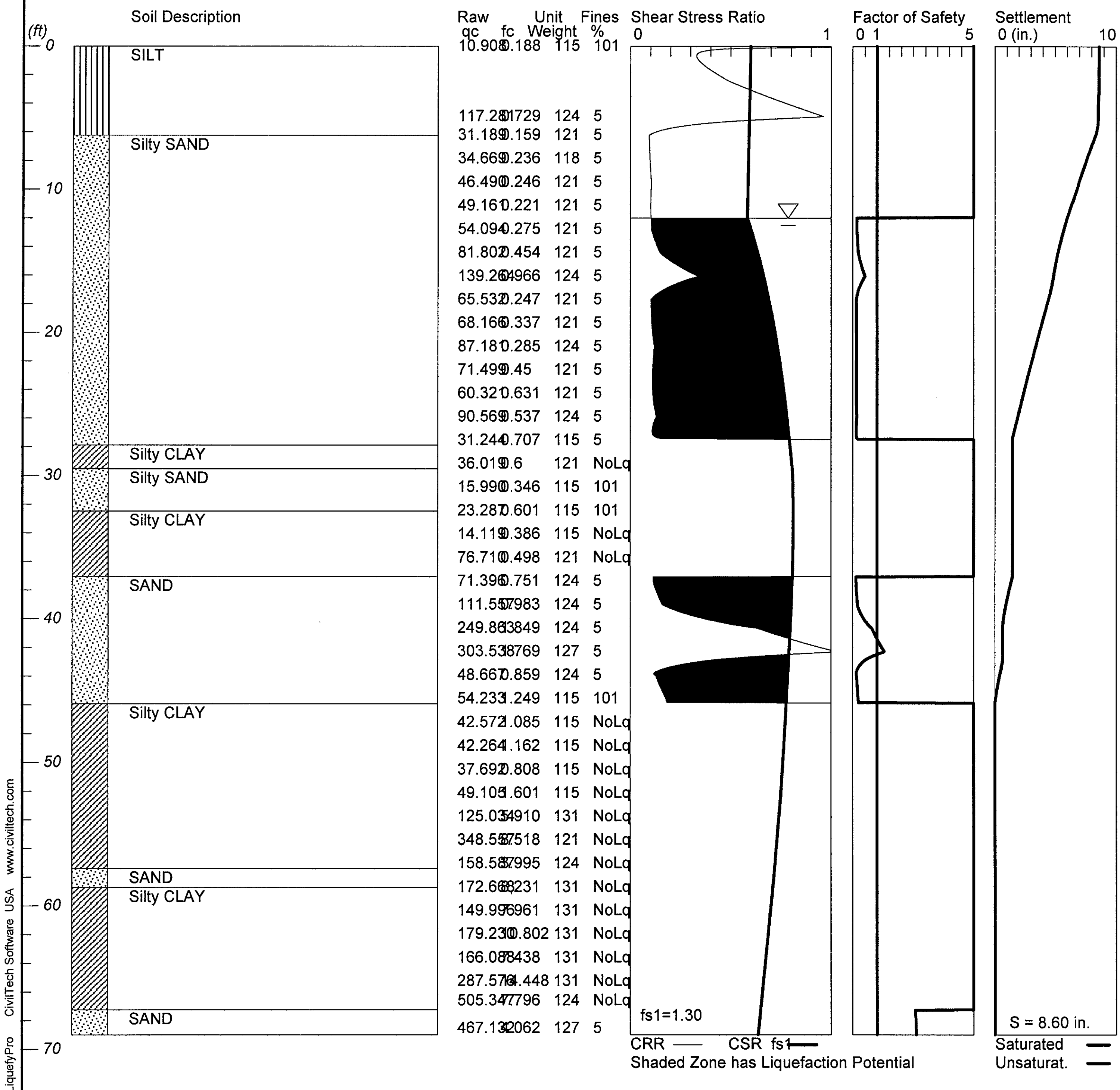
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

SV1302A - Proposed Lincoln Landing

Hole No.=CPT-03 Water Depth=12 ft Surface Elev.=100

Magnitude=7.9
Acceleration=0.71g



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LIQUEFACTION ANALYSIS SUMMARY

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Font: Courier New, Regular, Size 8 is recommended for this report.
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Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV (1300-1309)\SV1302 - Foothill Blvd. - Hayward\SV1302A.GI\SV1302A. Lique. CPT-04.liq
Title: SV1302A - Proposed Lincoln Landing
Subtitle: 22031 Foothill Blvd., Hayward, CA

Surface Elev.=100
Hole No.=CPT-04
Depth of Hole= 70.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.71 g
Earthquake Magnitude= 7.90

Input Data:
Surface Elev.=100
Hole No.=CPT-04
Depth of Hole=70.00 ft
Water Table during Earthquake= 12.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.71 g
Earthquake Magnitude=7.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. CPT Calculation Method: Modify Robertson*
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	qc atm	fs atm	Rf pcf	gamma %	Fines mm	D50
0.00	10.78	0.10	0.95	115.00	0.00	0.50
4.92	25.60	0.27	1.05	118.00	0.00	0.50
6.56	36.40	0.26	0.71	118.00	0.00	0.50
8.20	42.26	0.32	0.77	118.00	0.00	0.50
9.84	51.35	0.31	0.61	121.00	0.00	0.50
11.48	52.66	0.34	0.64	121.00	0.00	0.50
13.12	73.05	0.38	0.52	121.00	0.00	0.50
14.76	81.33	0.39	0.48	121.00	0.00	0.50
16.40	73.67	0.40	0.55	121.00	0.00	0.50
18.04	78.45	0.38	0.49	121.00	0.00	0.50
19.68	36.86	0.55	1.49	118.00	0.00	0.50
21.32	22.60	1.00	4.43	115.00	NoLiq	0.50
22.96	31.44	0.62	1.98	115.00	NoLiq	0.50

24.60	6.30	0.36	5.75	111.00	NoLiq	0.50
26.24	26.60	0.43	1.62	115.00	NoLiq	0.50
27.88	8.92	0.50	5.64	111.00	NoLiq	0.50
29.52	31.52	1.67	5.29	115.00	NoLiq	0.50
31.16	62.27	2.66	4.28	115.00	0.00	0.50
32.80	122.80	3.29	2.68	118.00	0.00	0.50
34.44	289.30	2.75	0.95	124.00	0.00	0.50
36.08	322.10	2.91	0.90	124.00	0.00	0.50
37.73	166.50	1.44	0.86	124.00	0.00	0.50
39.37	37.09	1.16	3.12	115.00	NoLiq	0.50
41.01	36.88	1.19	3.22	115.00	NoLiq	0.50
42.65	31.09	1.00	3.20	115.00	NoLiq	0.50
44.29	26.54	0.83	3.13	115.00	NoLiq	0.50
45.93	37.18	1.36	3.64	115.00	NoLiq	0.50
47.57	37.93	1.51	3.99	115.00	NoLiq	0.50
49.21	39.88	1.70	4.27	115.00	NoLiq	0.50
50.85	48.36	2.24	4.64	115.00	NoLiq	0.50
52.49	41.91	1.71	4.08	115.00	NoLiq	0.50
54.13	34.25	1.48	4.32	115.00	NoLiq	0.50
55.77	35.88	1.38	3.84	115.00	NoLiq	0.50
57.41	21.69	0.86	3.96	115.00	NoLiq	0.50
59.05	25.66	0.91	3.53	115.00	NoLiq	0.50
60.69	21.94	0.74	3.35	115.00	NoLiq	0.50
62.33	32.35	1.06	3.27	115.00	NoLiq	0.50
63.97	69.82	2.95	4.23	115.00	NoLiq	0.50
65.61	209.10	6.83	3.27	121.00	NoLiq	0.50
67.25	158.80	5.36	3.38	121.00	0.00	0.50
68.89	286.90	6.67	2.33	121.00	0.00	0.50

Modify Robertson method generates Fines from qc/fs. Inputted Fines are not relevant.

Output Results:

Settlement of Saturated Sands=2.80 in.
Settlement of Unsaturated Sands=3.28 in.
Total Settlement of Saturated and Unsaturated Sands=6.08 in.
Differential Settlement=3.040 to 4.012 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.60	5.00	2.80	3.28	6.08
1.00	0.11	0.60	5.00	2.80	3.28	6.07
2.00	0.11	0.60	5.00	2.80	3.26	6.06
3.00	0.10	0.60	5.00	2.80	3.01	5.80
4.00	0.10	0.59	5.00	2.80	2.67	5.46
5.00	0.11	0.59	5.00	2.80	2.36	5.15
6.00	0.11	0.59	5.00	2.80	2.01	4.81
7.00	0.11	0.59	5.00	2.80	1.65	4.45
8.00	0.11	0.59	5.00	2.80	1.31	4.10
9.00	0.11	0.59	5.00	2.80	0.97	3.76
10.00	0.11	0.59	5.00	2.80	0.65	3.45
11.00	0.11	0.58	5.00	2.80	0.35	3.14
12.00	0.12	0.58	0.20*	2.80	0.00	2.80
13.00	0.13	0.61	0.22*	2.50	0.00	2.50
14.00	0.14	0.63	0.22*	2.22	0.00	2.22
15.00	0.14	0.65	0.21*	1.93	0.00	1.93
16.00	0.13	0.66	0.19*	1.64	0.00	1.64
17.00	0.12	0.68	0.18*	1.35	0.00	1.35
18.00	0.12	0.70	0.18*	1.05	0.00	1.05
19.00	0.10	0.71	0.15*	0.74	0.00	0.74
20.00	0.14	0.72	0.19*	0.45	0.00	0.45
21.00	2.00	0.74	5.00	0.39	0.00	0.39

22.00	2.00	0.75	5.00	0.39	0.00	0.39
23.00	2.00	0.76	5.00	0.39	0.00	0.39
24.00	2.00	0.77	5.00	0.39	0.00	0.39
25.00	2.00	0.78	5.00	0.39	0.00	0.39
26.00	2.00	0.79	5.00	0.39	0.00	0.39
27.00	2.00	0.80	5.00	0.39	0.00	0.39
28.00	2.00	0.80	5.00	0.39	0.00	0.39
29.00	2.00	0.81	5.00	0.39	0.00	0.39
30.00	2.00	0.82	5.00	0.39	0.00	0.39
31.00	2.00	0.82	5.00	0.39	0.00	0.39
32.00	0.40	0.82	0.49*	0.39	0.00	0.39
33.00	0.46	0.82	0.56*	0.39	0.00	0.39
34.00	0.85	0.82	1.04	0.39	0.00	0.39
35.00	1.27	0.82	1.55	0.39	0.00	0.39
36.00	1.48	0.82	1.81	0.39	0.00	0.39
37.00	0.65	0.82	0.79*	0.39	0.00	0.39
38.00	0.25	0.81	0.31*	0.28	0.00	0.28
39.00	0.15	0.81	0.18*	0.05	0.00	0.05
40.00	2.00	0.81	5.00	0.00	0.00	0.00
41.00	2.00	0.81	5.00	0.00	0.00	0.00
42.00	2.00	0.80	5.00	0.00	0.00	0.00
43.00	2.00	0.80	5.00	0.00	0.00	0.00
44.00	2.00	0.80	5.00	0.00	0.00	0.00
45.00	2.00	0.79	5.00	0.00	0.00	0.00
46.00	2.00	0.79	5.00	0.00	0.00	0.00
47.00	2.00	0.79	5.00	0.00	0.00	0.00
48.00	2.00	0.78	5.00	0.00	0.00	0.00
49.00	2.00	0.78	5.00	0.00	0.00	0.00
50.00	2.00	0.77	5.00	0.00	0.00	0.00
51.00	2.00	0.77	5.00	0.00	0.00	0.00
52.00	2.00	0.76	5.00	0.00	0.00	0.00
53.00	2.00	0.76	5.00	0.00	0.00	0.00
54.00	2.00	0.75	5.00	0.00	0.00	0.00
55.00	2.00	0.75	5.00	0.00	0.00	0.00
56.00	2.00	0.74	5.00	0.00	0.00	0.00
57.00	2.00	0.74	5.00	0.00	0.00	0.00
58.00	2.00	0.73	5.00	0.00	0.00	0.00
59.00	2.00	0.72	5.00	0.00	0.00	0.00
60.00	2.00	0.72	5.00	0.00	0.00	0.00
61.00	2.00	0.71	5.00	0.00	0.00	0.00
62.00	2.00	0.71	5.00	0.00	0.00	0.00
63.00	2.00	0.70	5.00	0.00	0.00	0.00
64.00	2.00	0.69	5.00	0.00	0.00	0.00
65.00	2.00	0.69	5.00	0.00	0.00	0.00
66.00	2.00	0.68	5.00	0.00	0.00	0.00
67.00	2.00	0.67	5.00	0.00	0.00	0.00
68.00	0.72	0.66	1.08	0.00	0.00	0.00
69.00	1.04	0.66	1.59	0.00	0.00	0.00
70.00	1.03	0.65	1.59	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

---

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm — Cyclic resistance ratio from soils
CSRsf — Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S. — Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat — Settlement from saturated sands

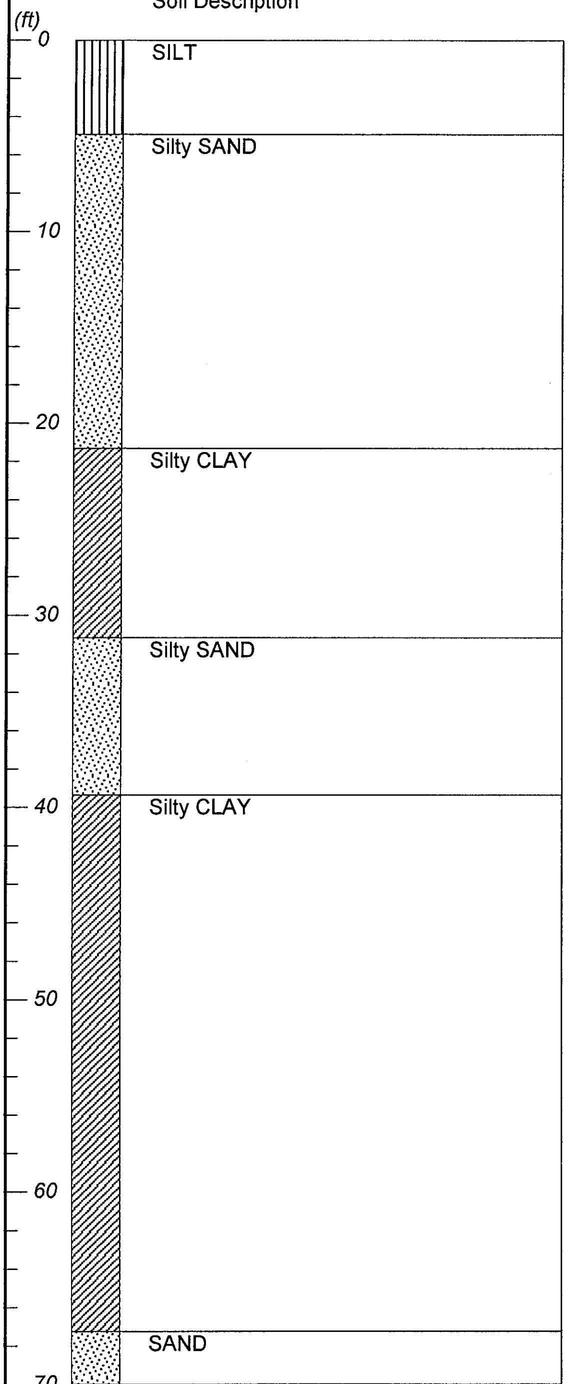
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

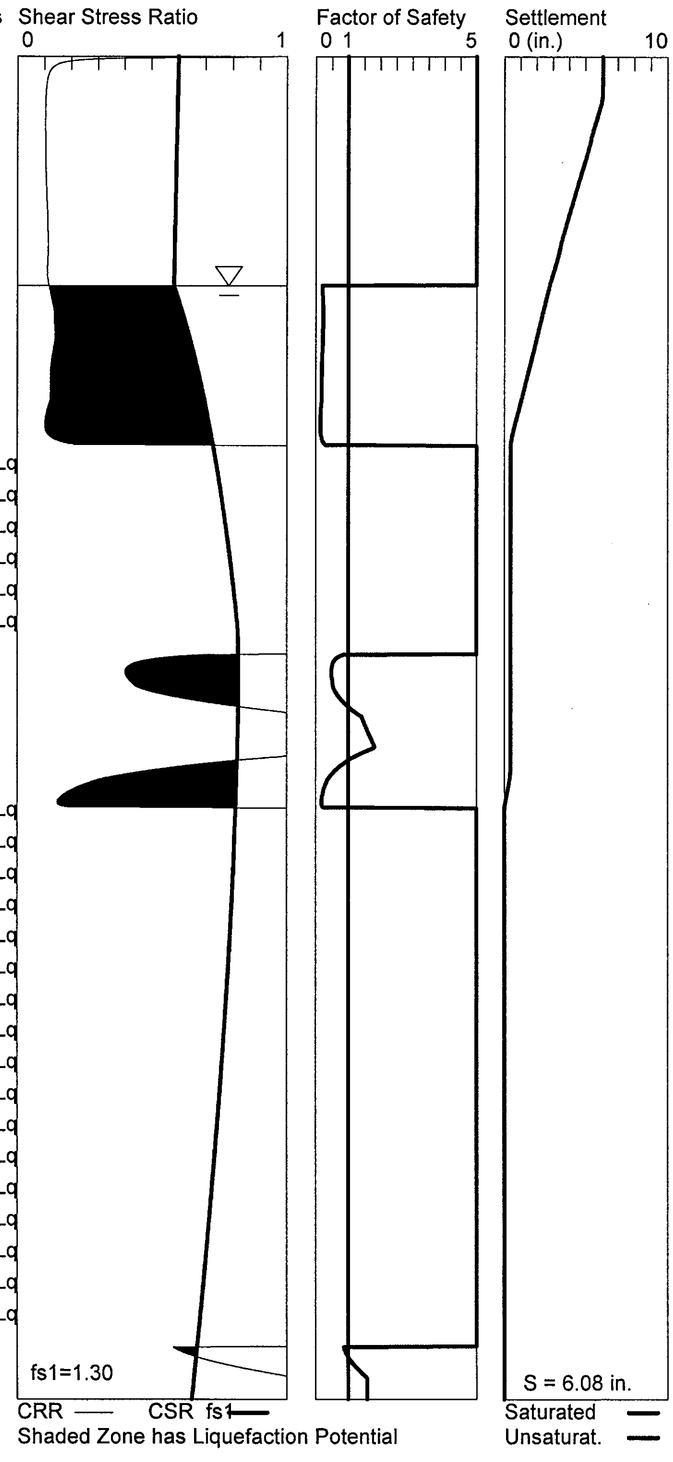
SV1302A - Proposed Lincoln Landing

Hole No.=CPT-04   Water Depth=12 ft   Surface Elev.=100

Magnitude=7.9
Acceleration=0.71g



Raw qc	fc	Unit Weight	Fines %
10.78	0.102	115	101
25.60	.270	118	5
36.40	0.259	118	5
42.26	0.325	118	5
51.35	0.315	121	5
52.66	0.339	121	5
73.05	0.377	121	5
81.33	0.391	121	5
73.67	0.402	121	5
78.45	0.382	121	5
36.86	0.549	118	5
22.60	1.001	115	NoLq
31.44	0.622	115	NoLq
6.301	0.362	111	NoLq
26.6	0.430	115	NoLq
8.916	0.503	111	NoLq
31.52	1.667	115	NoLq
62.27	2.665	115	101
122.8	3.291	118	5
289.3	2.751	124	5
322.1	2.907	124	5
166.5	1.438	124	5
37.09	1.159	115	NoLq
36.88	1.189	115	NoLq
31.09	0.995	115	NoLq
26.54	0.831	115	NoLq
37.18	1.355	115	NoLq
37.93	1.514	115	NoLq
39.88	1.704	115	NoLq
48.36	2.244	115	NoLq
41.91	1.708	115	NoLq
34.25	1.481	115	NoLq
35.88	1.377	115	NoLq
21.69	0.860	115	NoLq
25.66	0.906	115	NoLq
21.94	0.735	115	NoLq
32.35	1.059	115	NoLq
69.82	2.955	115	NoLq
209.1	6.831	121	NoLq
158.8	5.364	121	5
286.9	6.673	121	5



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Osummary

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*****************************************************************
              ALLPILE 7
      VERTICAL ANALYSIS SUMMARY OUTPUT
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*****************************************************************
TOTAL LOADS:
        Vertical Load, Q: 1537.0 -kp
        Vertical Load with Load Factor, Q: 1537.0 -kp
        Vertical Load with Load factor and Pile Cap, Q= 1537.0 -kp
          Load Factor for Vertical Load and Torsion= 1.0
          Vertical Loads Supported by Pile Cap: 0 %
          Load Factor for Vertical Loads: 1.0

PILE PROFILE:
        Pile Length, L= 80.0 -ft
        Top Height, H= 0 -ft
        Slope Angle, As= 0
        Batter Angle, Ab= 0.00    Batter Factor, Kbat= 1.00

SINGLE PILE:
          Kdown= 1.3      Kup= 0.8        Ka= 0.71

Single Pile Vertical Analysis:
        Total Ultimate Capacity (Down)= 1537.502-kp  Total Ultimate Capacity (Up)=
1130.790-kp
        Total Allowable Capacity (Down)= 987.201-kp  Total Allowable Capacity (Up)=
581.912-kp

        Weight above Ground= 0.00       Total Weight= 33.03-kp      *Soil Weight is
not included
        Side Resistance (Down)= 1310.702-kp  Side Resistance (Up)= 1097.755-kp
        Tip Resistance (Down)= 226.800-kp  Tip Resistance (Up)= 0.000-kp
        Negative Friction, Qneg= 0.000-kp, which has been subtracted from Total
Ultimate Capacity (Down)
        Negative friction does not affect Total Ultimate Capacity (Up)

        At Work Load= 1537.00-kp,  Settlement= 99999.00000-in
        At Work Load= 1537.00-kp,  Secant Stiffness Kqx= 99999.00-kp/-in
        At Allowable Settlement= 1.000000-in,  Capacity= 1106.02-kp
        !!! Work Load, 1537.00-kp, Exceeds the Capacity at Allowable Settlement=
1.00000-in, Capacity (Down)= 1106.02-kp
        !!! Work Load, 1537.00-kp, Exceeds the Allowable Capacity (Down)= 987.20-kp

_____________________________________________
FACTOR OF SAFETY:
        FSside  FStip   FSuplif FSweight
        1.5     2.0     2.0     1.0

Note:  If the program cannot find a result or the result exceeds the upper limit.
The result will be displayed as 99999.
1       1       1       1       1
```

****
ALLPILE 7
LATERAL ANALYSIS SUMMARY OUTPUT

****

```
FACTORS AND CONDITIONS:
        Load Factor for Vertical Loads: 1.0
        Load Factor for Lateral Loads: 1.0
        Loads Supported by Pile Cap: 0 %
        Shear Condition: Static

SINGLE PILE:
        (with Load Factor)
        Vertical Load= 1537.00 -kp
        Shear= 30.00 -kp
        Moment= 0.00 -kp-f

Results:
        Top Deflection, yt= 0.24900-in
        Max. Moment, M= 146.67-kp-f
        Top Deflection Slope, St= -0.00269

        Top Deflection, 0.2490-in, OK with the Allowable Deflection= 1.00-in

Note:  If the program cannot find a result or the result exceeds the upper limit.
The result will be displayed as 99999.
```

```
Notes:
 Q - Vertical Load at pile top
 P - Lateral Shear Load at pile top
 M - Moment at pile top
 Xtop - Pile top total settlement
 yt - Pile top deflection
 St - Pile top deflection slope (deflection/unit length)


The Max. Moment calculated by program is an internal moment of shaft due to the
loading. Egineers
have to check whether the pile has enough moment capacity to resist the Max. Moment
with adequate
factor of safety.  If not, the pile may be damaged under the loading.
1       1       1       1       1
```

```
*******************************************************************
                 ALLPILE 7
      LATERAL ANALYSIS SUMMARY OUTPUT
       Copyright by CivilTech Software
              www.civiltech.com
*******************************************************************
FACTORS AND CONDITIONS:
        Load Factor for Vertical Loads: 1.0
        Load Factor for Lateral Loads: 1.0
        Loads Supported by Pile Cap: 0 %
        Shear Condition: Static

SINGLE PILE:
        (with Load Factor)
        Vertical Load= 1537.00 -kp
        Shear= 78.00 -kp
        Slope Restrain, St= 0.00 -in/-in

Results:
        Top Deflection, yt= 0.24700-in
        Max. Moment, M= -410.00-kp-f
        Top Deflection Slope, St= 0.00000

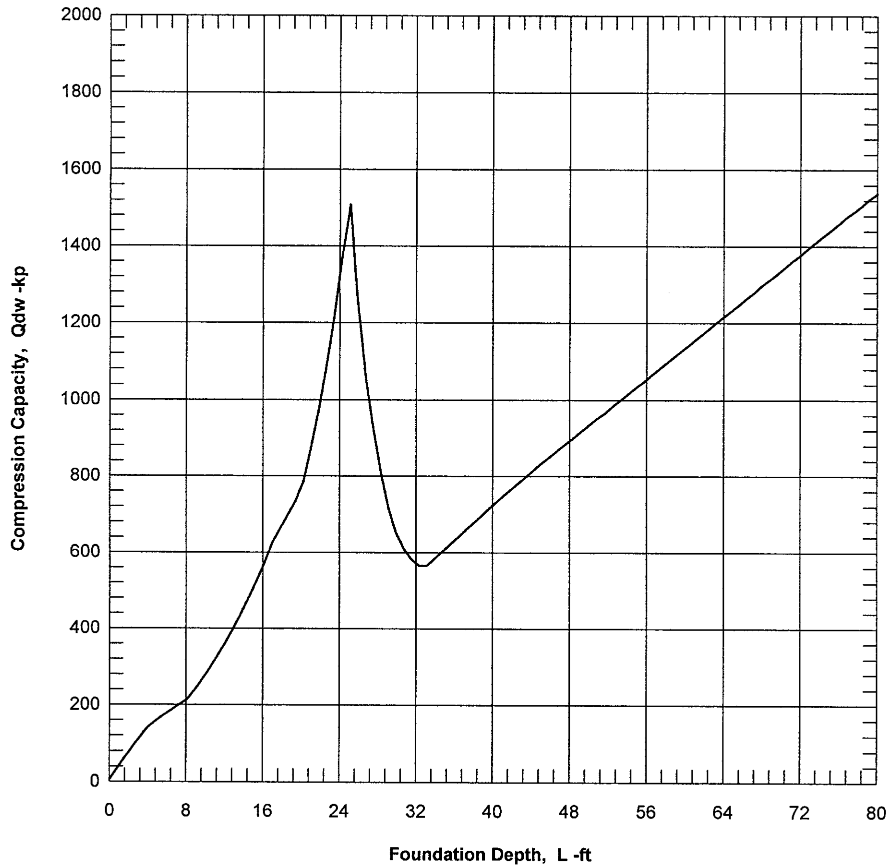
        Top Deflection, 0.2470-in, OK with the Allowable Deflection= 1.00-in

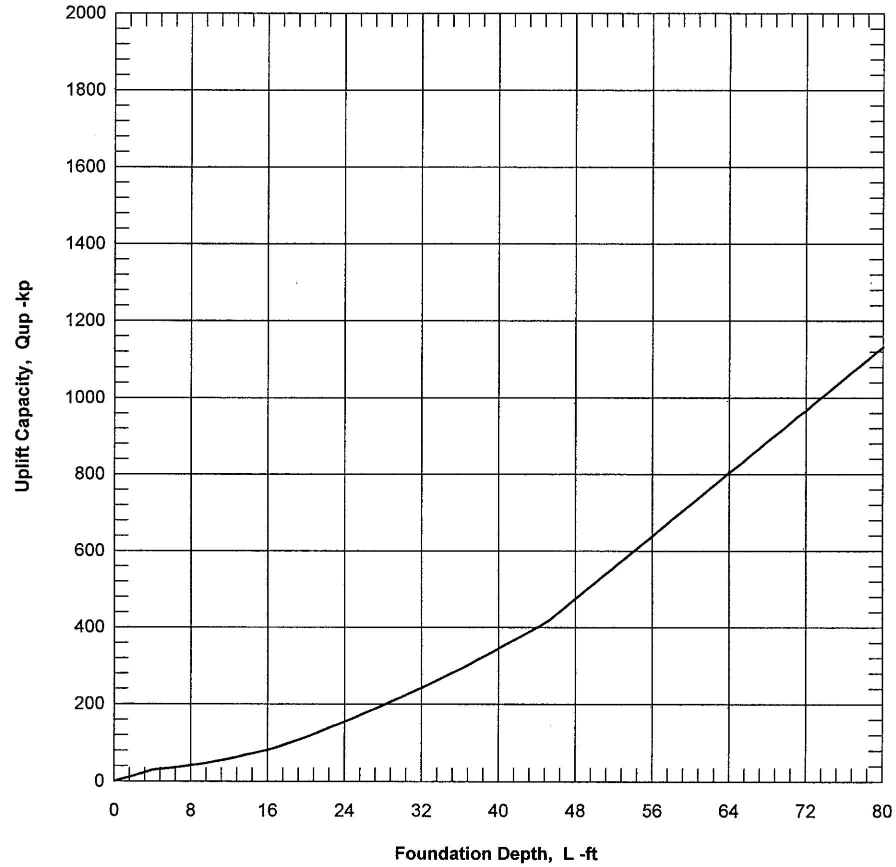
Note:  If the program cannot find a result or the result exceeds the upper limit.
The result will be displayed as 99999.
____________________________________

Notes:
 Q - Vertical Load at pile top
 P - Lateral Shear Load at pile top
 M - Moment at pile top
 Xtop - Pile top total settlement
 yt - Pile top deflection
 St - Pile top deflection slope (deflection/unit length)


The Max. Moment calculated by program is an internal moment of shaft due to the
loading. Egineers
have to check whether the pile has enough moment capacity to resist the Max. Moment
with adequate
factor of safety.  If not, the pile may be damaged under the loading.
1       1       1       1       1
```

ULTIMATE CAPACITY vs FOUNDATION DEPTH





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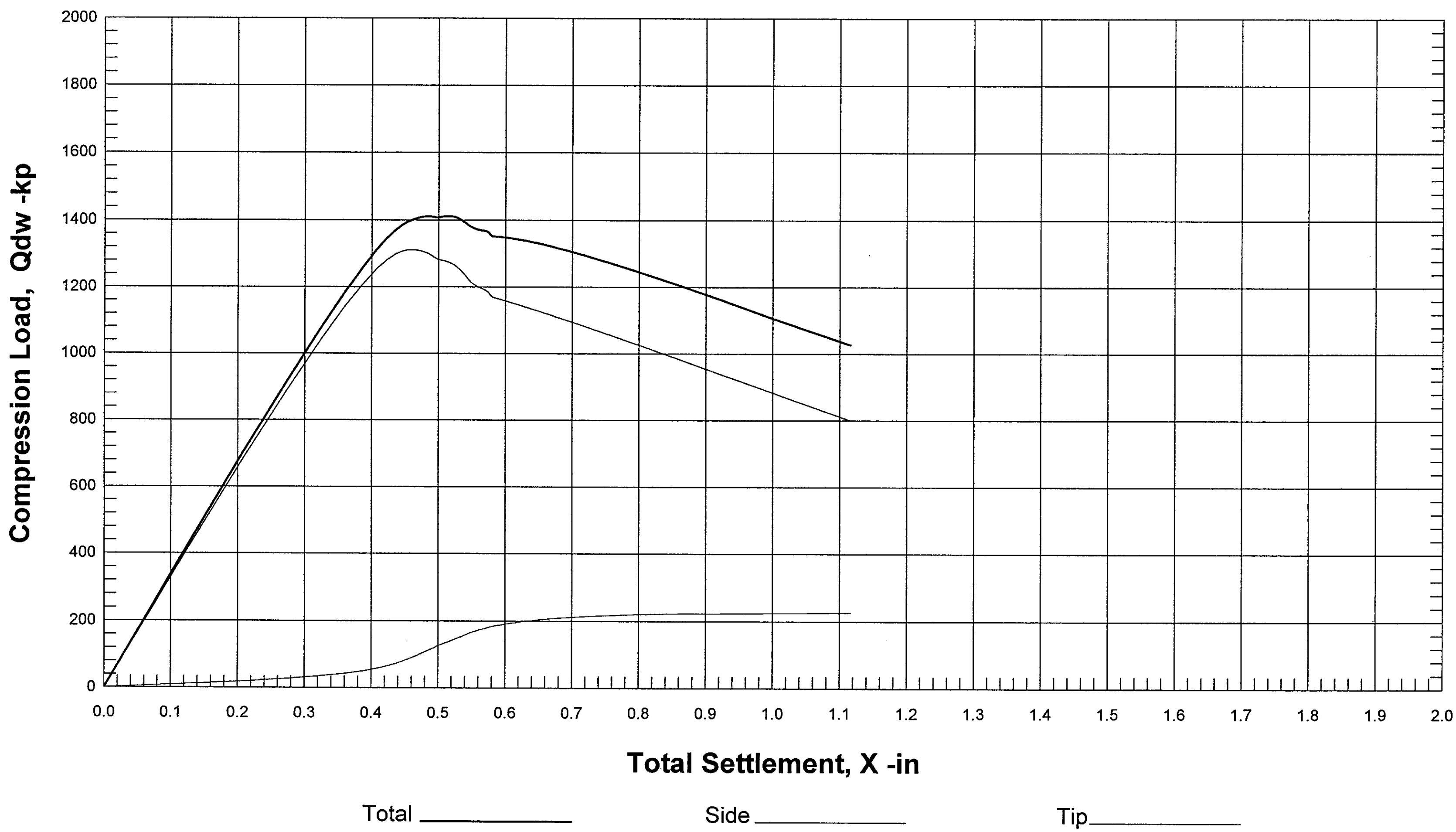
22301 Foothill Blvd., Hayward, CA

Figure 1

Vertical Load vs. Total Settlement



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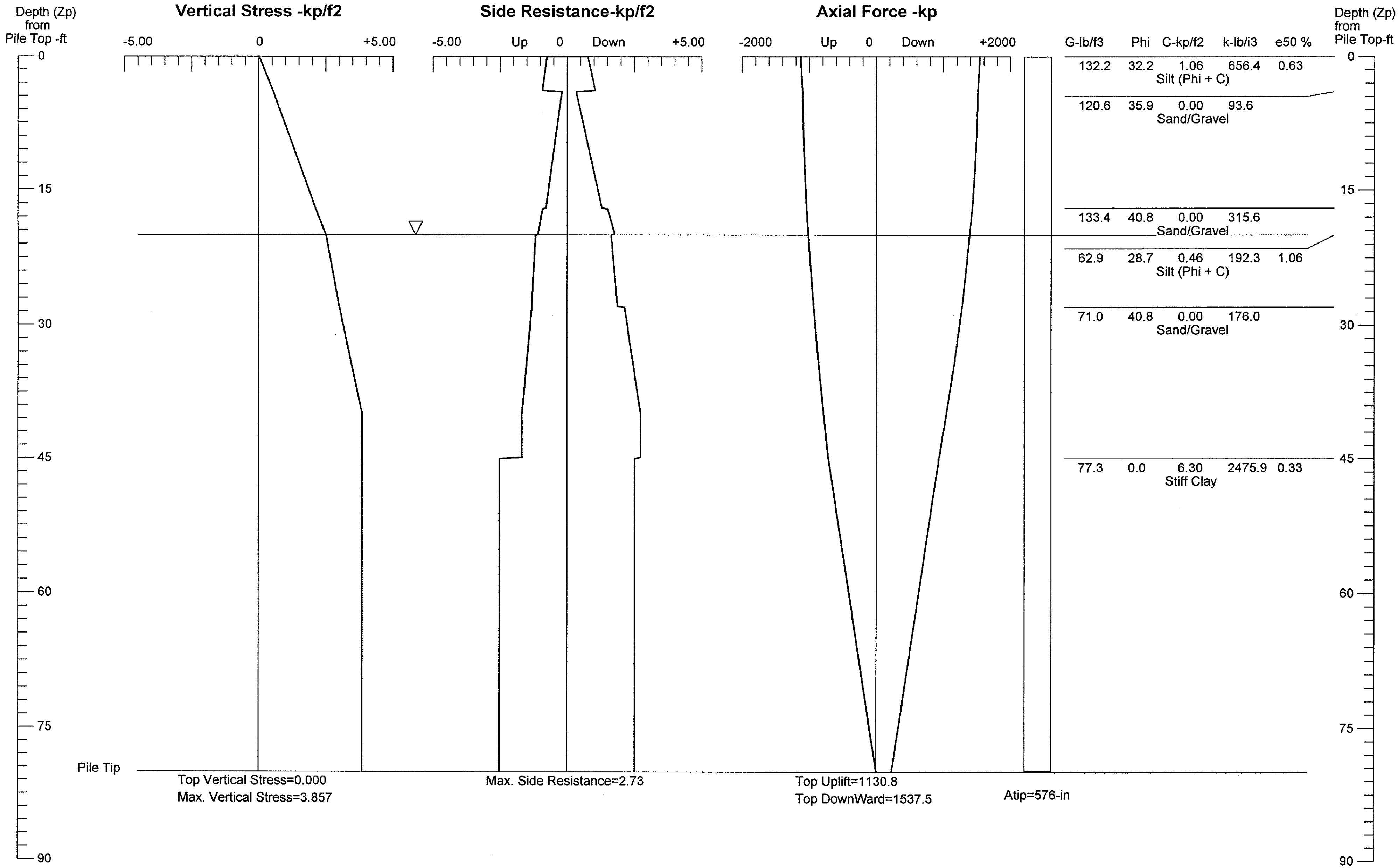
22301 Foothill Blvd., Hayward, CA

Figure 1

SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE vs DEPTH
Based on Ultimate Load Condition



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SV1302A - Proposed Lincoln Landing Mixed-Use Developme
22301 Foothill Blvd., Hayward, CA

Figure 1

PRE-CAST PRE-STRESS SKIN-FRICTION PILE SPECIFICATIONS

FOR

PROPOSED LINCOLN LANDING MIXED-USE DEVELOPMENT

22301 FOOTHILL BOULEVARD
HAYWARD, CALIFORNIA

I. GENERAL SPECIFICATIONS

a. Qualifications

Piling subcontractor shall be qualified and experienced in this type of work.

b. Responsibility

Owners shall accept no responsibility for driveability of piles as shown and specified.

c. Grading

Necessary clearing, excavation and filling shall be done by the contractor.

d. Pile Locations

Civil engineer will stake out pile locations. Cost for replacing moved and damage stakes shall be borne by the contractor.

e. Available Data

Records of the borings made at these sites are included in the contract drawings available from the civil engineer. These records pertain to conditions at the boring locations. Contractors are expected to make a personal inspection of the sites and to otherwise satisfy themselves as to the conditions affecting the work. No claims for extra compensation or extension of time shall be allowed on account of near subsurface conditions inconsistent with the data given.

f. Pile Depth

All piles shall be given to minimum depths as indicated on the plans and shall meet the requirements set in the Standard Specifications.

g. Inspection

The soil engineer will inspect the driving of all piles. At least one week's notice shall be given before the first pile is to be driven.

II. PILE TYPES

a. Type 1

Pre-cast, pre-stress pile (Alternate X - Class 70).

b. Type 2

Pre-cast pile (Alternate X - Class 70).

c. Type 3

Concrete casing filled with Class "A" P.C.C.

III. PILE MATERIALS

Piles should meet the requirements of standard specifications set by the State of California Department of Public Works.

IV. HANDLING OF PILES

All piles shall be handled with care to avoid damage. Damage to any pile to driving shall be cause for immediate rejection.

V. INSTALLATION

a. General

After the first pile row is driven, the driving criteria will be reviewed and if necessary modified by the engineer. Each pile should be driven without interruption, except for splicing, only by written permission shall deviation from this procedure be allowed. Under no condition will a pile be started if it cannot be finished the same day.

b. Record of Driving

Kept by soil engineer

1. Reference

   All piles recorded with an appropriate numbering system.

2. Dimensions

   Include elevations of tip and butt before and after cutting.

3. Driving resistance

   Complete record with number of blows required to drive each foot for full length of each pile.

4. Time

   Include time of starting, completion, interruptions (if any), and condition of pile after driving.

c. Minimum Spacing

All piles shall have a minimum clear spacing between outside equal to 3 times the pile butt's greatest dimension, or 4 feet, whichever is greater.

d. Alignment

Do not exceed 2 percent maximum deviation from vertical on any section of length. Keep pile center at cutoff within 3 inches of design location. Pulling piles into position shall not be permitted. The contractor shall provide substitute piles where driven piles exceed specified tolerances; all correction costs under this section, including any structural redesign, additional materials, and labor, shall be paid by the contractor.

e. Damaged Piles

1. General

   Any pile driven into previously driven pile automatically rejects both piles. Replace whose handling or driving record indicates possible damage or defect; replace as directed with a substitute pile at no expense to owner. Do not drive piles damaged or suspected damage until inspected and approved.

2. Driving Damage

- Type "X" and "Y" (Pre-cast, pre-stress piles). Development of tension cracks, spall or chips in the concrete within the pay length shall be cause for rejection.
- Type "W" (concrete casing filled by P.C.C.). General criteria as for type "X" and type "Y" piling applies. In addition, any crimping or buckling within the pay length due to excessive hard driving, shall be cause for rejection.

f. Driving Equipment

Use approved type as generally used in standard pile driving practice. Use driving hammers of such size and type able to consistently deliver effective dynamic energy suitable to piles and materials which they are driving; operate at manufacturer's recommended speeds and pressures. Swing leads not permitted; use fixed leads or other suitable means for holding pile firmly in position and alignment with the hammer. Pile shall be plumb before driving. Take special precautions to insure against leading away of pile from plumb to true position. Care shall be taken during driving to prevent and correct any tendency of piles to twist, rotate, or walk.

VI. DRIVING CRITERIA

a. Driving Energy

Use hammers developing minimum driving energies for the various classes of piles as follows:

Pile Type	Minimum Rated Hammer Energy
Class I	24,000 ft-lbs.
Class II	19,000 ft-lbs.

Hammers developing greater or lesser energies, or sonic hammers, may be used upon written authorization of the engineer.

b. Reduction of Hammer Energy

When piles have settled into the ground under their own weight and the weight of the hammer, and the point of the pile is passing through soft soil so that there is little resistance, there is a possibility that longitudinal tensile stress will be set up in the pile. For such driving conditions, the first hammer blows delivered to the pile shall have a lesser energy by reducing the stroke of the hammer to approximately 24 inches. In no case shall the stroke of the hammer exceed 42 inches.

c. Driving Criteria

Estimated termination of pile penetrations is given in the Recommendation section of this report. Actual pile tip elevation shall be determined, at time of driving, by the soil engineer in the field.

VII. PILE TYPES NOT SPECIFIED

a. General

Consideration will be given to pile types other than those shown or specified. If the contractor proposes to use a type other than those shown, he shall submit to the owner or the structural engineer for review a description of the pile and shall demonstrate by calculations and other corroborating evidence the ability of the pile to sustain required loads.

b. Prequalification

Review proposed foundation pile plans at no cost to owner; plans to be prepared and stamped by licensed civil engineer. Comply with all local jurisdictional codes.

c. <u>Engineering Design</u>

Prepare revised foundation pile plans at no cost to owner; plans to be prepared and stamped by licensed civil engineer. Comply with all local jurisdictional codes.

d. <u>Pile Tests</u>

If, in the opinion of the owner or his representative, pile load tests are required to confirm the load bearing capacity, the costs of such tests shall be borne by the contractors.