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REPORT TO  
THE HUDSON COMPANIES  
CAMPBELL, CALIFORNIA

FOR

PROPOSED MIXED-USE DEVELOPMENT  
138 STOCKTON AVENUE  
SAN JOSE, CALIFORNIA  
GEOTECHNICAL INVESTIGATION  
OCTOBER 2016

PREPARED BY

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

# SILICON VALLEY SOIL ENGINEERING

GEOTECHNICAL CONSULTANTS

File No. SV1404  
October 13, 2016

The Hudson Companies  
1510 South Bascom Avenue, #7  
Campbell, CA 95008

Attention: Mr. Daniel Hudson

Subject: Proposed Mixed-Use Development  
138 Stockton Avenue  
San Jose, California  
**GEOTECHNICAL INVESTIGATION**

Dear Mr. Hudson:

Pursuant to your request, we are pleased to present herein geotechnical investigation for the proposed mixed-use development. The subject site is located at 138 Stockton Avenue in San Jose, 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.

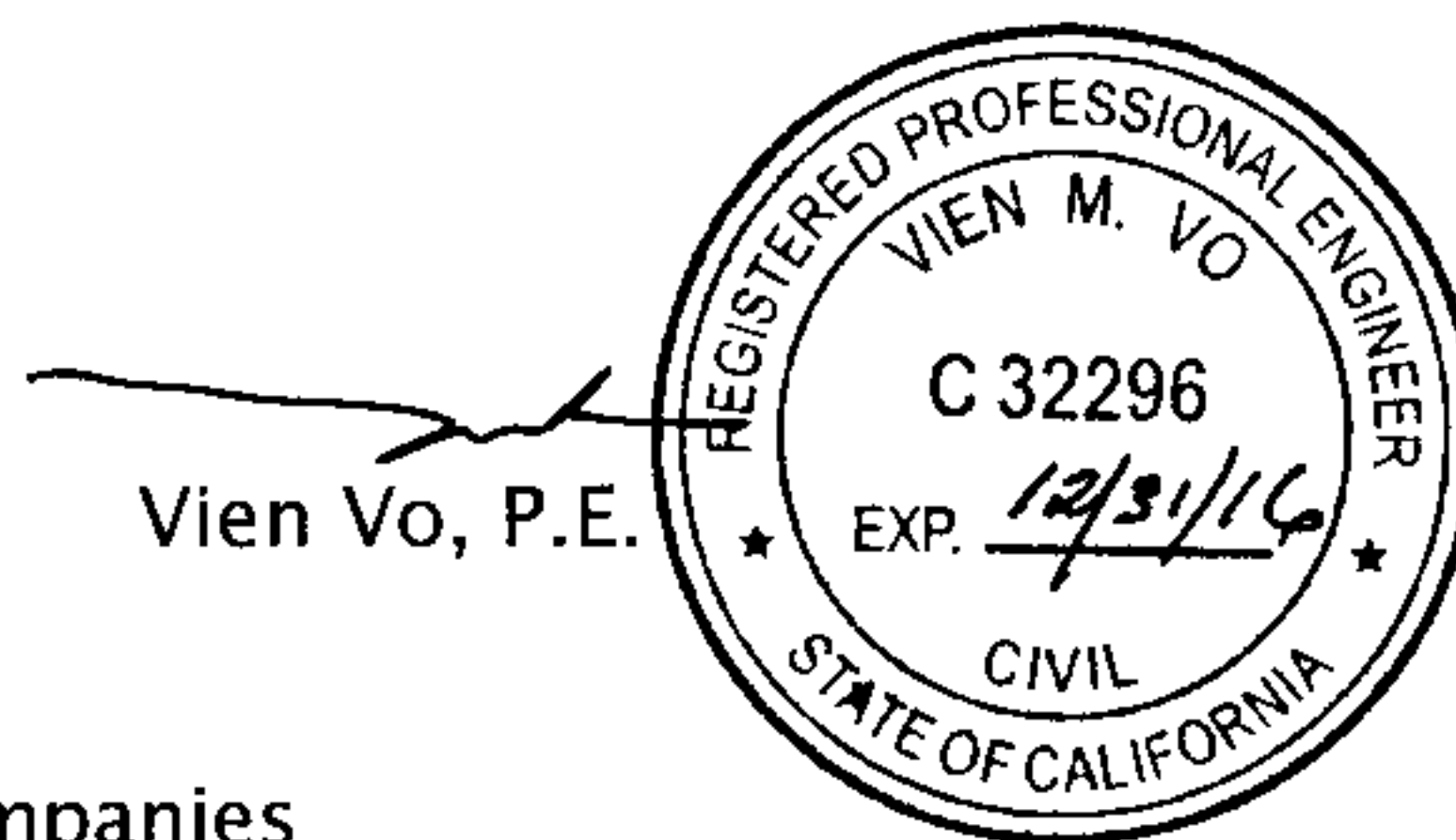
If you have any questions or require additional information, please feel free to contact our office at your convenience.

Very truly yours,

SILICON VALLEY SOIL ENGINEERING

*Sean Deivert, P.E.*  
Sean Deivert  
Project Manager

SV1404.GI/Copies: 4 to The Hudson Companies



Vien Vo, P.E.

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SANTA CLARA VALLEY WATER DISTRICT DRILLING PERMIT



## **INTRODUCTION**

Per your authorization, Silicon Valley Soil Engineering (SVSE) conducted a geotechnical investigation. The purpose of this geotechnical investigation was to 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 138 Stockton Avenue in San Jose, California (Figure 1). Stockton Avenue bounds the subject site to the southwest, existing office building/warehouse to the northwest, CALTRAIN railroad tracks to the northeast, and existing automotive body shop to the southeast. At the time of this investigation, the subject site is irregular shaped, occupied by several structures on three parcels (APN 259-28-003, 004, & 005) surrounded by paved parking lot. Based on the preliminary plans for the subject site, the proposed development will include the demolition of the existing structures and the construction of a seven-story mixed-use building with associated improvements. The building will consist of a two-story main structure over the majority of the subject site with residential towers above the main two-story structure. The ground and second floor will consist of parking and commercial units. The third floor will consist of commercial and residential units with a swimming pool and courtyard on podium slab. The remaining levels will consist of residential units. The approximate location of the proposed structure and our borings are shown on the Site Plan (Figure 2).

## **FIELD INVESTIGATION**

After considering the nature of the proposed development and reviewing available data on the area, our geotechnical engineer conducted a field investigation at the project site. It included a site reconnaissance to detect any unusual surface features, and the drilling of five exploratory test borings to determine the subsurface soil characteristics. The borings were drilled on June 17 and 30, 2015. The approximate location of the borings is shown on the Site Plan (Figure 2). The borings were drilled to the depths ranging from 21.5 feet to 81.5 feet below existing ground surface. The borings were drilled with a truck mounted drill rig using 4-inch diameter solid stem and 8-inch diameter hollow stem augers.

The soils encountered were logged continuously in the field during the drilling operation. Relatively undisturbed soil samples were obtained by hammering a 2-inch outside diameter (O.D.) split-tube sampler for a Standard Penetration Test (S.P.T.), A.S.T.M. Standard D1586, into the ground at various depths. A 140-pound hammer with a free fall of 30 inches was used to drive the sampler 18 inches into the ground. Blow counts were recorded on each 6-inch increment of the sampled interval. The blows required to advance the sampler the last 12 inches of the 18 inch sampled interval were recorded on the boring logs as penetration resistance. These values were also used to evaluate the liquefaction potential of the subsurface soils. After the completion of the drilling operation, the exploratory borings were backfilled from the bottom of the borehole to the surface with neat cement in accordance to the rules and regulations of the Santa Clara Valley Water District. 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 surface and 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 the depths of 3, 10, 20, 30, 40, and 50 feet. 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-1 (81.5 foot boring), the pavement surface soils consist of 3.0 inches of asphalt concrete over 4.0 inches of aggregate base. Below the pavement sections to the depth of 7 feet, a black, moist, very stiff silty clay was encountered. Color change of dark brown was noted at the depth of 4 feet. From the depths of 7 feet to 13 feet, the soil became olive brown, moist, stiff clayey silt. Color change of light olive brown was noted at the depth of 11 feet. From the depths of 13 feet to 17 feet, a light olive brown, moist, medium dense silty sand layer was encountered. The sand was medium grained and poorly graded. From the depths of 17 feet to 40 feet, the soil became medium bluish gray, moist, very stiff silty clay layer was encountered. Color changes of medium olive brown to bluish gray were noted at the depths of 30 feet and 35 feet respectively. From the depths of 40 feet to 48 feet, the soil became bluish gray, moist, very stiff clayey silt layer was encountered. From the depths of 48 feet to 58 feet, the soil became brown, moist, dense gravelly sand layer was encountered. The sand was medium grained and poorly graded. From the depths of 58 feet to 63 feet, the soil became bluish, gray, moist, very stiff sandy clay layer was encountered. From the depths of 63 feet to 75 feet, the soil became brown, wet, dense sandy gravel layer was encountered. The gravel was  $\frac{1}{4}$  inch maximum diameter, sub-rounded, and poorly graded. From the depths of 75 feet to the end of the boring at 81.5 feet, a brown, wet, dense gravelly sand layer was encountered. The sand was medium grained and poorly graded. Similar soil profiles were encountered in other borings.

Groundwater was initially encountered in Boring B-1 at the depth of 33 feet and rose to a static level of 30 feet at the end of the drilling operation. It should be noted that the groundwater level would fluctuate as a result of seasonal changes and hydrogeological variations such as groundwater pumping and/or recharging. A graphic description of the explored soil profiles is presented in the Exploratory Boring Log contained in the Appendix.

## GENERAL GEOLOGY

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

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

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

Bay Region and most of the strata are of continental origin characterized as alluvial and fluvial materials.

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

## **LIQUEFACTION ANALYSIS**

### **A. GROUNDWATER**

Groundwater was initially encountered in the borings at depths of 33 feet and rose to a static level ranging from 30 feet at the end of the drilling operation. Based on the State guidelines and CGS Seismic Hazard Zone Report 058 [*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], the highest expected groundwater level is approximately 13 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-1 to the depth of 81.5 feet consists of very stiff silty clay to stiff clayey silt to medium dense silty sand to very stiff silty clay to very stiff clayey silt to dense gravelly sand to very stiff sandy clay to dense sandy gravel to dense gravelly sand. The following is the determination of the liquefiable soil for each soil layer in Boring B-1.

1. The very stiff silty clay layer from the surface to the depth of 7 feet is not liquefiable soil because it is above the highest expected groundwater table.
2. The stiff clayey silt layer from the depths of 7 feet to 13 feet is not liquefiable soil because it is above the highest expected groundwater table.
3. The medium dense silty sand layer from the depths of 13 feet to 17 feet is liquefiable soil based on the low blow counts and Plasticity Index (PI) [PI < 12] .
4. The very stiff silty clay layer from the depths of 17 feet to 40 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
  - Sample No. 1-5 (20 feet) - [PI = 19 > 12 and MC = 32.0% < 34.9% = 85% LL; LL = 41]
  - Sample No. 1-6 (25 feet) - [PI = 20 > 12 and MC = 23.0% < 34.0% = 85% LL; LL = 40]
  - Sample No. 1-7 (30 feet) - [PI = 21 > 12 and MC = 31.7% < 37.4% = 85% LL; LL = 44]



- Sample No. 1-8 (35 feet) - [PI = 20 > 12 and MC = 31.7% < 36.6% = 85% LL; LL = 43]
5. The very stiff clayey silt layer from the depths of 40 feet to 48 feet is not liquefiable soil based on the Plasticity Index (PI) and Moisture Content (MC):
- Sample No. 1-9 (40 feet) - [PI = 17 > 12 and MC = 25.5% < 29.8% = 85% LL; LL = 35 ]
  - Sample No. 1-10 (45 feet) - [PI = 18 > 12 and MC = 24.7% < 31.5% = 85% LL; LL = 37]
6. The dense gravelly sand layer from the depths of 48 feet to 58 feet is not liquefiable soil based on high blow counts.

In summary, there is one liquefiable soil layer underlying the subject site. This is the medium dense silty sand layer from the depths of 13 feet to 17 feet (4 feet thick).

### 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 058 [*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], the peak ground acceleration is 0.55g.

#### **D. LIQUEFACTION ANALYSIS**

The evaluation procedure is a semi-empirical method for a moment magnitude Mw7.9 earthquake, a peak ground acceleration of 0.55g, and highest expected groundwater table of 13 feet. A computer program named LiquefyPro Version 5.8n (CivilTech Corporation) was used in the liquefaction analysis. 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 of the medium dense silty sand layer is low. 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 0.39 inch and the maximum differential settlements are 0.26 inch. 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 soil layer of Boring B-1 in Figure 7, the thickness of surface non-liquefiable ( $H_1$ ) [14 feet or 4.7 meters] soil layer and the thickness of the liquefiable ( $H_2$ ) [4 feet or 1.33 meter] soil layer in Boring B-1 was entered with a maximum peak acceleration of  $a_{max} = 0.55g$ . The following is the determination of  $H_1$  and  $H_2$  in Boring B-1.

**Boring B-1:**  $H_1 = 4.7$  meters;  $H_2 = 1.33$  meters

Based on the plotted coordinates of the suspected liquefiable soil layer of Boring B-1 using the above data, we concluded that there is a minimal 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 Guadalupe River northeasterly 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:

$M = 8.5$ ,  $R = 19$  kilometer from San Andreas Fault,  $R_0 = 84$ ,  $R^* = 103$

$T_{15} = 0$  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 0.2 meters (0.6 feet or 7.2 inches) with respect to the Hayward Fault. Based on the magnitude of the lateral movement, we concluded that the liquefaction-induced lateral spreading is minimal.

## **G. CONCLUSIONS**

The followings are the conclusions of this study.

- The liquefaction-induced total maximum settlement at the site is 0.39 inches
- The liquefaction-induced differential settlement at the site is 0.26 inches
- The potential of liquefaction-induced ground surface damage at the site is minimal
- The liquefaction-induced lateral spreading is minimal

## **INUNDATION POTENTIAL**

The subject site is located at 138 Stockton Avenue in San Jose, California. According to the Limerinos and others, 1973 report, the site is not located in an area that has potential for inundation as the result of a 100-year flood (Limerinos; 1973).

## CONCLUSIONS

1. The site covered by this investigation is suitable for the proposed development provided the recommendations set forth in this report are carefully followed.
2. Based on the laboratory testing results, the native surface soil at the project site has been found to have a high expansion potential when subjected to fluctuations in moisture. Therefore, we recommend the building pad be underlain by a minimum of 12 inches non-expansive fill layer or 12 inches of lime-treated native soil material. During the construction of the building pad, any highly expansive native soil should not be used as non-expansive engineered fill material.
3. 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. The baserock material should be inspected and tested prior to final approval and use. Cinder block (light weight concrete) can not be crushed and reused.
4. The imported non-expansive fill soils should be free of organic material and hazardous substances. All imported fill material to be used for engineered fill should be environmentally tested prior to be used at the site.
5. The lime-treated subgrade soil, if any, should not be exposed to the element for an extended period. If no improvements are planned for the immediate future, the lime-treated subgrade soil should be protected.
6. We recommend the exterior of the building pad be graded to permit proper drainage and diversion of water away from the building foundations.



7. The two-story/seven-story structures should be supported on either convention spread footings and conventional spread footings with geopiers, if required, or driven pre-stress pre-cast concrete pile foundation.
8. If the site is located in a low-lying area or adjacent to any creek or drainage channel, minor cracks and separations of the concrete slab-on-grade, asphalt concrete pavement and/or curb and gutter should be expected.
9. 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).
10. 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.
11. Specific recommendations are presented in the remainder of this report.
12. All earthwork and grading shall be observed and inspected by a representative from Silicon Valley Soil Engineering (SVSE). These operations are not limited to testing and inspection during grading.

## RECOMMENDATIONS:

### GRADING

1. The placement of fill and control of any grading operations at the site should be performed in accordance with the recommendations of this report. These recommendations set forth the minimum standards to satisfy other requirements of this report.
2. All existing surface and subsurface structures, if any, that will not be incorporated in the final development shall be removed from the project site prior to any grading operations. These objects should be accurately located on the grading plans to assist the field engineer in establishing proper control over their removal. All utility lines in the new building pad area must be removed prior to any grading at the site.
3. The depressions left by the removal of subsurface structures should be cleaned of all debris, backfilled and compacted with clean, native soil. This backfill must be engineered fill and should be conducted under the supervision of a SVSE representative.
4. All organic surface material and debris shall be stripped prior to any other grading operations, and transported away from all areas that are to receive structures or structural fills. Soil containing organic material may be stockpiled for later use in landscaping areas only.
5. After removing all the subsurface structures 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 moisture conditioned to 3% over optimum moisture.
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 Santa Clara Valley Water District. The final elevation of the top of the well casing must be a minimum of 3 feet below the adjacent grade prior to any grading operation.

### **FOUNDATION DESIGN CRITERIA**

13. We recommend that the two-story/seven-story structures should be supported on conventional spread footings and conventional spread footings with geopiers, if required, or driven pre-stress pre-cast concrete pile foundation. Recommendations are presented in the following paragraphs.
14. The proposed structures can be supported on conventional spread footings:
  - The allowable bearing capacity at footing depth of 48 inches is 3,700 psf
  - The allowable bearing capacity at footing depth of 60 inches is 4,200 psf
15. The proposed structures with heavy column loads can be supported on conventional spread footings with geopiers:

- The allowable bearing capacity at footing depth of 48 inches with geopier 24 inches in diameter and 13 feet deep below the bottom of the foundation is 8,000 psf
  - The allowable bearing capacity at footing depth of 60 inches with geopier 24 inches in diameter and 10 feet deep below the bottom of the foundation is 8,000 psf
  - The maximum differential settlement per 40 feet span is estimated to be 1/8 of an inch.
  - To provide proper support of the concrete slab-on-grade floor, the 24-inch or the 36-inch subgrade soil layer above the top of the footing should be properly backfilled and re-compacted as stated in the **GRADING** section.
16. The above bearing values are for dead plus live loads, and may be increased by one-third for short term seismic and wind loads. The design of the structures and the foundations shall meet local building code requirements.
17. The proposed structures can 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. The pile should be 14-inch square and terminated at a minimum depth of 55 feet below ground surface. The structural slab should have a minimum thickness of 12 inches and an allowable contact pressure of 1,200 psf.
18. 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.



## 19. VERTICAL ANALYSIS

- The ultimate vertical load carrying capacity and uplift capacity for 55 feet length pile are 372 kips and 280 kips respectively.
- The allowable vertical load carrying capacity and uplift capacity for 55 feet length pile are 238.5 kips and 144.6 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 0.15 inch.

## 20. 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 23 kips with the maximum allowable lateral deflection at the top of the pile is 0.257 inch for free end condition.
- The maximum allowable lateral shear force should be limited to 45 kips with the maximum allowable lateral deflection at the top of the pile is 0.249 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.
21. 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.
  22. The minimum pile spacing clearance should be 2.5 times the pile diameter.
  23. Pile specifications are included at the end of the report.
  24. Passive pressure can be developed at the side face of the footings or grade beams. This value is 250 pounds equivalent fluid pressure. The coefficient of friction is 0.3.
  25. The above bearing values are for dead plus live loads, and may be increased by one-third for short term seismic and wind loads. The design of the structures and the foundations shall meet local building code requirements.
  26. The project structural engineer responsible for the foundation design should determine the final design of the foundations and reinforcing required. We recommend that the foundation plans be reviewed by our office prior to submitting to the appropriate local agency and/or to construction.

## 2013 CBC SEISMIC VALUES

27. 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	37.332967° N.
Site Longitude	121.904453° W.
0.2-second Mapped Spectra Acceleration <sup>1</sup> , $S_S$	1.500g*
1-second Mapped Spectra Acceleration <sup>1</sup> , $S_I$	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 $S_{M1}$ ( $S_{M1} = F_v S_I$ - 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*

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

\* USGS Seismic Design Maps for 2013 CBC analysis.

## RETAINING WALLS

28. 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.

29. 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.
30. 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.
31. The above values assume a drained condition, and a moisture content compatible with those encountered during our investigation.
32. 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  $\frac{3}{4}$  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.
33. 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.
34. We recommend a thorough review by our office of all designs pertaining to facilities retaining a soil mass.

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



foundation should be extended to a minimum of 5 feet horizontally from building walls at a minimum slope of 2 percent away from structure.

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 0.5 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. Because of the high expansion potential of the surface native soil at the site, we provided alternative pavement section (asphalt and baserock) recommendations for the parking area to be underlain by a minimum of 12 inches of non-expansive fill or lime-treated native material. The non-expansive fill soil and lime-treated native material should be compacted to at least 90% relative maximum density. These alternate pavement

sections are presented in Table IIA, IIB and III. Due to the high expansion potential of the surface native soil, minor cracks in the pavement should be expected.

### **LIME TREATMENT ALTERNATIVES**

50. Lime treatment of the subgrade soil can be considered as an option in order to reduce the high expansion potential of near-surface native soil and/or to weather proof (winterize) the subgrade soil during the winter construction of the building pad or parking and driveway areas. The lime treatment process should extend a minimum of 3 feet beyond the building pad, curb and gutter, and/or any other improvements. The top 12 inches of the subgrade can be treated with a mixture of 5% of quick lime (High Calcium) and native soil by volume. If the lime treatment is used, minor cracks on the concrete slab and separation of the curb/gutter and pavement should be expected. In the building pad area, if lime treatment would be implemented, the rock section could be reduced by one inch. In the parking area, if lime treatment would be implemented, the baserock section could be reduced as shown in Table IIB.
51. The lime-treated subgrade soil should not be exposed to the element for an extended period. If no improvements are planed for the immediate future, the lime-treated subgrade soil should be protected.

## LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations presented herein are based on the soil conditions revealed by our test 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.



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## TABLES

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

TABLE II – PROPOSED ALTERNATE PAVEMENT SECTIONS

TABLE IIA – PROPOSED NON-EXPANSIVE PAVEMENT SECTIONS

TABLE IIB – PROPOSED LIME TREATMENT PAVEMENT SECTIONS

TABLE III – PROPOSED RIGID PAVEMENT SECTIONS

TABLE IV – PROPOSED PAVER PAVEMENT SECTIONS

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

Sample No.	Depth Ft.	In-Place Conditions		Direct Shear Testing			
		Moisture Content % Dry Wt.	Dry Density p.c.f.	Unit Cohesion k.s.f.	Angle of Internal Friction Degrees	Liquid Limit L.L.	Plasticity Index P.I.
1-1	3	25.9	89.5	1.0	10		
1-2	5	27.3	87.8				
1-3	10	20.9	105.6	0.5	22		
1-4	15	15.9	110.3				<12
1-5	20	32.0	90.6	0.8	10	41	19
1-6	25	23.0	101.6			40	20
1-7	30	31.7	89.8	0.7	11	44	21
1-8	35	23.3	103.8			43	20
1-9	40	25.5	98.7	0.5	20	35	17
1-10	45	24.7	101.2			37	18
1-11	50	14.7	105.5	0	33		<12
1-12	55	14.1	106.3				<12
1-13	60	25.9	100.4				
1-14	65	8.8	110.7				
1-15	70	9.6	108.3				
1-16	75	10.3	104.5				
1-17	80	9.9	105.2				

**TABLE I (CONTINUED)****SUMMARY OF MOISTURE/DENSITY, DIRECT SHEAR,  
PLASTICITY INDEX, & LIQUID LIMIT TESTS**

Sample No.	Depth Ft.	In-Place Conditions		Direct Shear Testing		Liquid Limit L.L.	Plasticity Index P.I.
		Moisture Content % Dry Wt.	Dry Density p.c.f.	Unit Cohesion k.s.f.	Angle of Internal Friction Degrees		
2-1	3	16.6	94.6				
2-2	5	16.9	106.3				
2-3	10	15.1	113.5				
2-4	15	16.9	113.8				
2-5	20	19.4	110.5				
3-1	3	18.6	89.8				
3-2	5	18.5	112.3				
3-3	10	18.9	109.2				
3-4	15	16.6	107.4				
3-5	20	10.5	110.6				
3-6	25	16.3	103.2				
3-7	30	29.0	93.4				
3-8	35	28.2	99.0				
3-9	40	22.7	95.1				
4-1	3	17.2	100.7				
4-2	5	24.9	98.8				
4-3	10	25.9	96.4				

**TABLE I (CONTINUED)****SUMMARY OF MOISTURE/DENSITY, DIRECT SHEAR,  
PLASTICITY INDEX, & LIQUID LIMIT TESTS**

Sample No.	Depth Ft.	In-Place Conditions		Direct Shear Testing		Liquid Limit L.L.	Plasticity Index P.I.
		Moisture Content % Dry Wt.	Dry Density p.c.f.	Unit Cohesion k.s.f.	Angle of Internal Friction Degrees		
4-4	15	15.7	112.5				
4-5	20	23.5	99.9				
5-1	3	21.1	96.1				
5-2	5	16.4	108.6				
5-3	10	20.6	102.6				
5-4	15	16.0	110.7				
5-5	20	20.3	112.2				



**TABLE II****PROPOSED ALTERNATE PAVEMENT SECTIONS**

Location: Proposed Mixed-Use Development  
138 Stockton Avenue  
San Jose, California

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

**TABLE IIA****PROPOSED NON-EXPANSIVE PAVEMENT SECTIONS**

Location: Proposed Mixed-Use Development  
138 Stockton Avenue  
San Jose, California

	<u>PARKING STALLS</u>			<u>DRIVEWAY</u>		
Design R-Value	24.0			24.0		
Traffic Index	4.5			5.5		
Gravel Equivalent	14.0			16.0		
Recommended Alternate Pavement Sections:	<u>1A</u>	<u>1B</u>	<u>1C</u>	<u>2A</u>	<u>2B</u>	<u>2C</u>
Asphalt Concrete	3.0"	3.5"	4.0"	3.0"	3.5"	4.0"
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	6.0"	5.0"	4.0"	9.0"	8.0"	7.0"
Non-expansive soil fill material compacted to at least 90% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"
Native soil scarified & compacted to at least 87% relative maximum density	12.0"	12.0"	12.0"	12.0"	12.0"	12.0"

**TABLE IIB****PROPOSED LIME TREATMENT PAVEMENT SECTIONS**

Location: Proposed Mixed-Use Development  
138 Stockton Avenue  
San Jose, California

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

**TABLE III****PROPOSED RIGID PAVEMENT SECTIONS**

Location: Proposed Mixed-Use Development  
138 Stockton Avenue  
San Jose, California

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

\* Including trash enclosures, stress pads, and valley gutters. Reinforcement provided by Structural Engineer. Maximum control joints at 5' by 5' or as recommended by Structural Engineer. Vertical curbs should be keyed at least 3 inches into pavement subgrade.

**TABLE IV****PROPOSED PAVER PAVEMENT SECTIONS**

Location: Proposed Mixed-Use Development  
138 Stockton Avenue  
San Jose, California

	<b><u>DRIVEWAY/PARKING AREA*</u></b>			
Recommended Paver Pavement Sections:	<b>1A*</b>	<b>1B*</b>	<b>2A</b>	<b>2B</b>
Vehicular Rated Pavers	Min. 3.25" ± Permeable Paver Parking Stalls	Min. 3.25" ± Permeable Paver Driveway	Min. 3.25" ± Non- Permeable Paver Parking Stalls	Min. 3.25" ± Non- Permeable Paver Driveway
ASTM No. 8 Bedding Course & Paver Filler	2.0"	2.0"	2.0"	2.0"
3/4" Clean Crushed Rock or ASTM No. 57 Drain Stone	8.0"	12.0"	---	---
Class II Baserock (R=78 min.) compacted to at least 95% relative maximum density	---	---	10.0"	14.0"
Non-expansive soil fill material compacted to at least 90% relative max. density, if any	---	---	---	---
Native soil scarified & compacted to at least 87% relative max. density	12.0"	12.0"	12.0"	12.0"

\* (see next page)



- \* The subgrade should be lined with a geotextile membrane Mirafi 500X or equivalent. The liner should be placed and overlapped properly for drainage. The subgrade should be sloped at a minimum of 2% towards the subdrain system.
- \* The subdrain system should consist of a 4-inch diameter perforated pipe surrounded by  $\frac{3}{4}$  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

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FIGURE 1 – VICINITY MAP

FIGURE 2 – SITE PLAN

FIGURE 3 – FAULT LOCATION MAP

FIGURE 4 – PLASTICITY INDEX CHART

FIGURE 5 – COMPACTION TEST A

FIGURE 6 – R-VALUE TEST

FIGURE 7 – LIQUEFACTION-INDUCED GROUND DAMAGE

FIGURE 8 – SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE VS  
DEPTH

FIGURE 9 – VERTICAL LOAD VS TOTAL SETTLEMENTS

FIGURE 10 – ULTIMATE CAPACITY VS FOUNDATION DEPTH

FIGURE 11 – SIDE RESISTANCE VS RELATIVE MOVEMENT BETWEEN  
SOIL AND SHAFT

FIGURE 12 – TIP RESISTANCE VS TIP MOVEMENT

FIGURE 13 – PILE DEFLECTION & FORCE VS DEPTH (FREE END)

FIGURE 14 – PILE DEFLECTION VS LOADING (FREE END)

FIGURE 15 – PILE MOMENT VS LOADING (FREE END)

FIGURE 16 – SOIL RESISTANCE VS PILE DEFLECTION (FREE END)

FIGURE 17 – LATERAL LOAD VS DEFLECTION & MAXIMUM MOMENT  
(FREE END)

FIGURE 18 – PILE DEFLECTION & FORCE VS DEPTH (FIXED END)

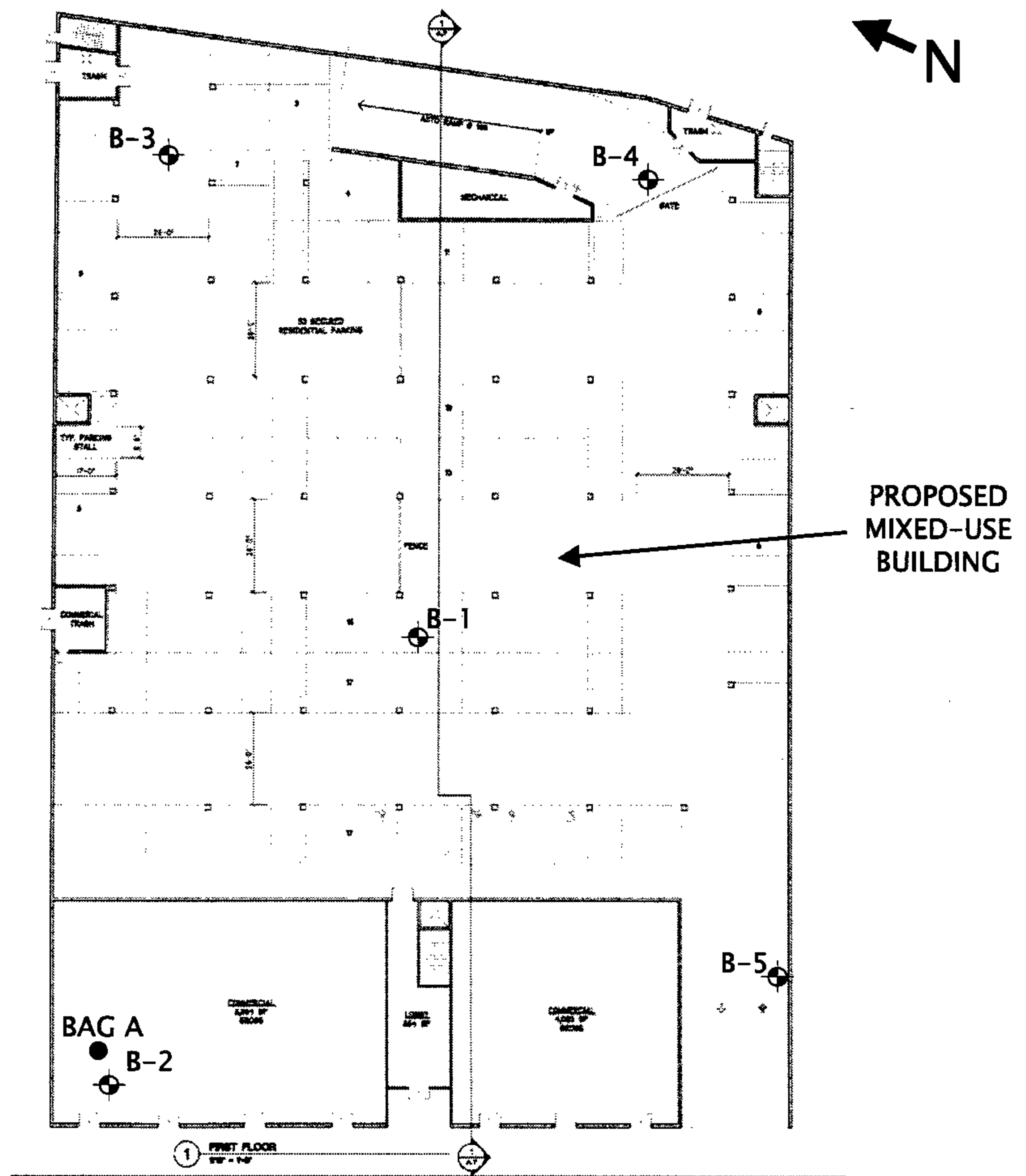
FIGURE 19 – PILE DEFLECTION VS LOADING (FIXED END)

FIGURE 20 – PILE MOMENT VS LOADING (FIXED END)



FIGURE 21 – SOIL RESISTANCE VS PILE DEFLECTION (FIXED END)

FIGURE 22 – LATERAL LOAD VS DEFLECTION & MAXIMUM MOMENT  
(FIXED END)

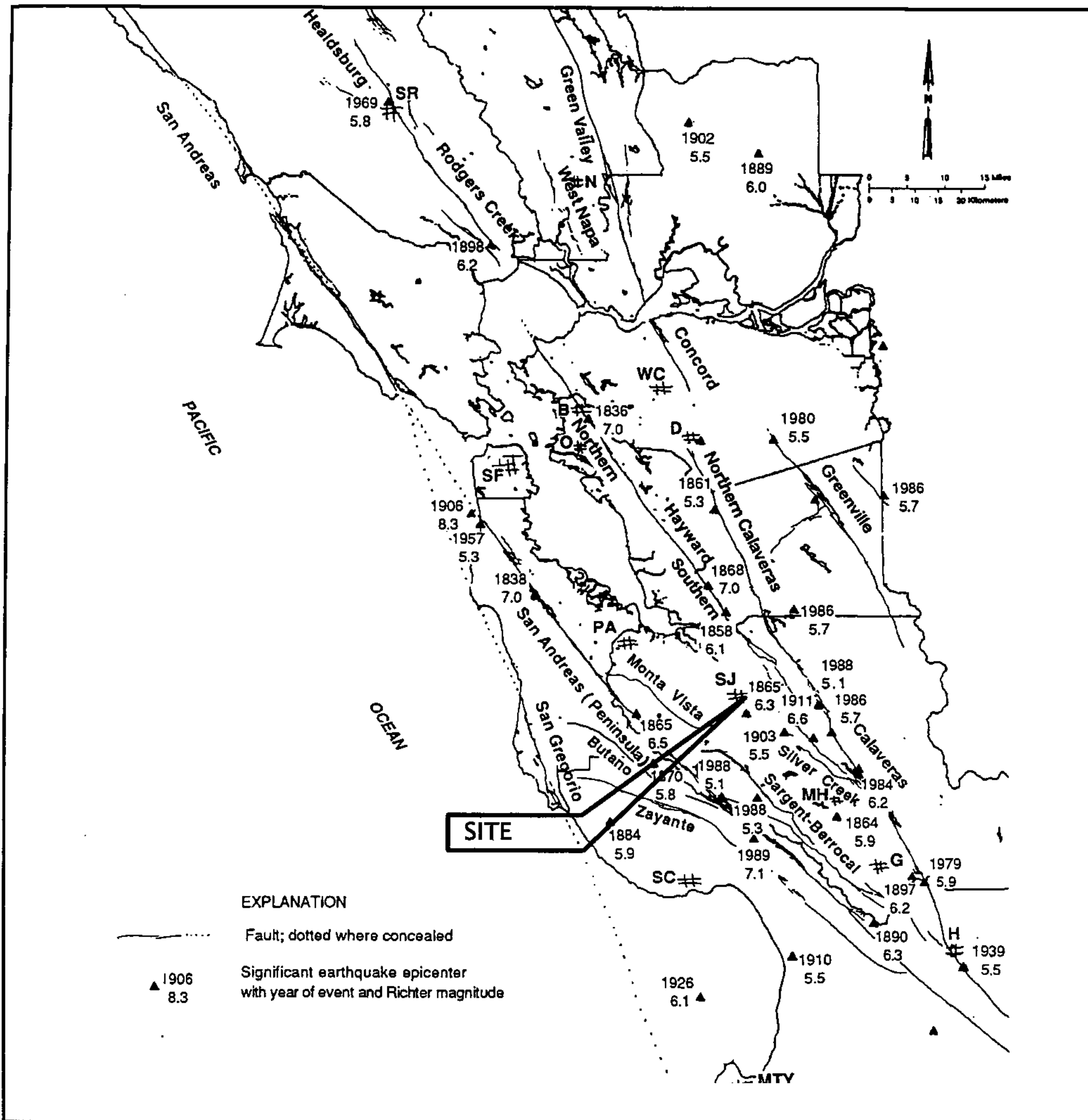
<p>Silicon Valley Soil Engineering</p> <p>2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400</p>	<p><b>VICINITY MAP</b></p> <p>Proposed Mixed-Use Development</p> <p>138 Stockton Avenue San Jose, California</p>	File No.: SV1404	FIGURE  1
		Drawn by: V.V.	
		Scale: NOT TO SCALE	October 2016



## 138 STOCKTON AVENUE

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

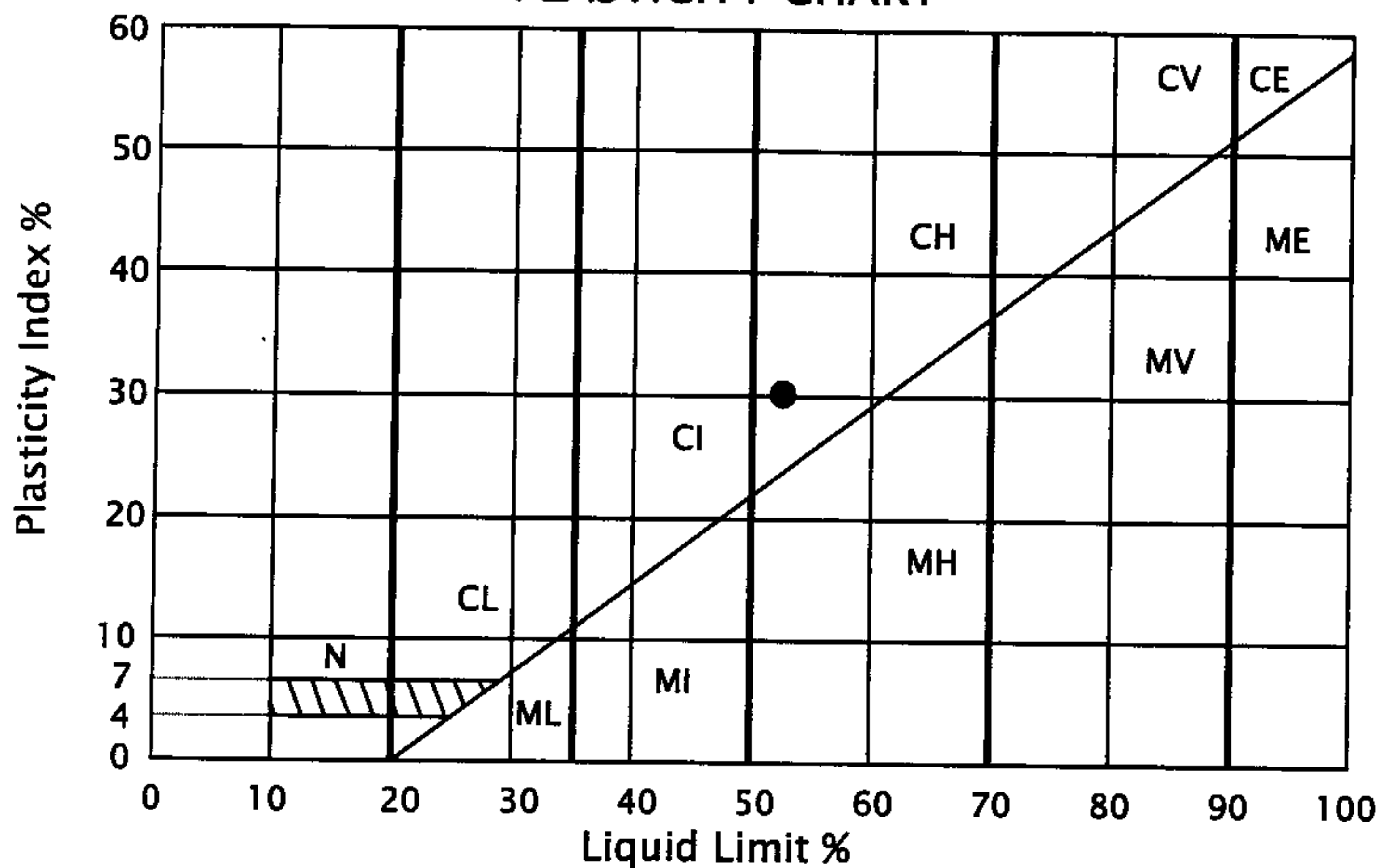
Silicon Valley Soil Engineering  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>SITE PLAN</b>  Proposed Mixed-Use Development  138 Stockton Avenue San Jose, California	File No.: SV1404	FIGURE  2
		Drawn by: V.V.	
		Scale: NOT TO SCALE	October 2016



<p>Silicon Valley Soil Engineering</p> <p>2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400</p>	<p><b>FAULT LOCATION MAP</b></p> <p>Proposed Mixed-Use Development</p> <p>138 Stockton Avenue San Jose, California</p>	<p>File No.: SV1404</p>	<p>FIGURE</p> <p>3</p>
		<p>Drawn by: V.V.</p>	
		<p>Scale: NOT TO SCALE</p>	<p>October 2016</p>



# PLASTICITY CHART

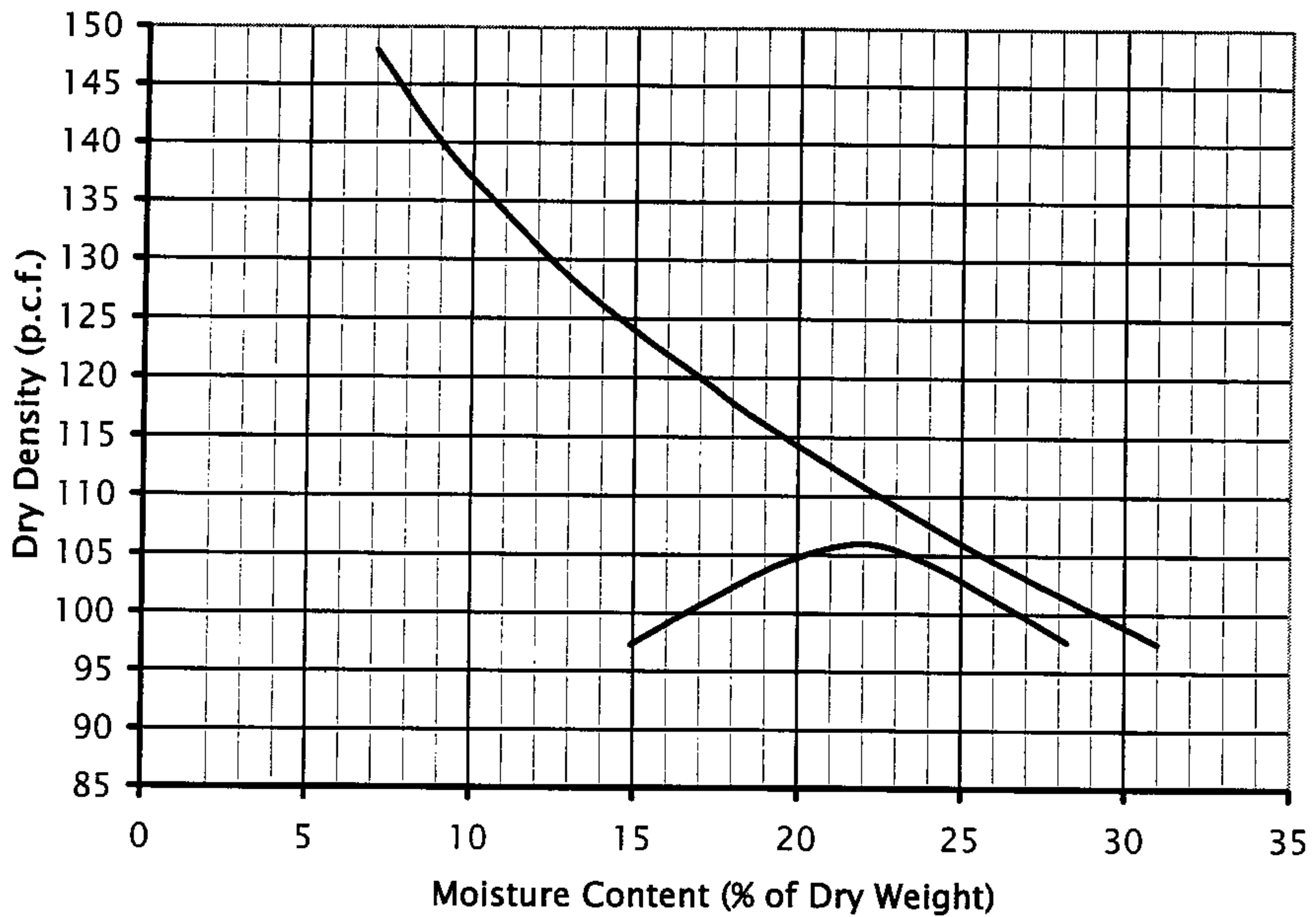


## PLASTICITY DATA

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

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

<b>Silicon Valley Soil Engineering</b>  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>PLASTICITY INDEX</b>  Proposed Mixed-Use Development  138 Stockton Avenue San Jose, California	File No.: SV1404	FIGURE  4
		Drawn by: V.V.	
		Scale: NOT TO SCALE	October 2016



SAMPLE: A

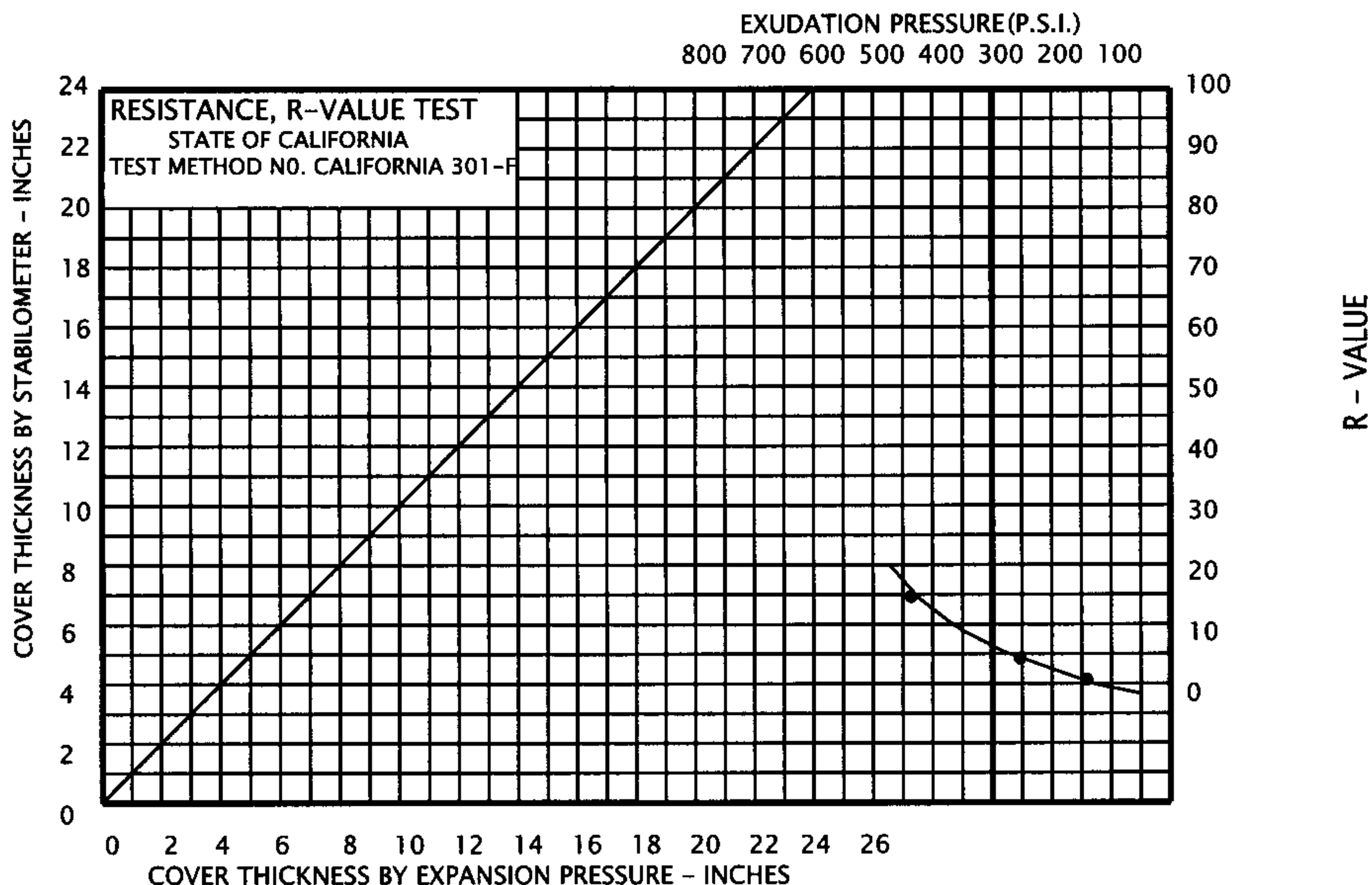
DESCRIPTION: Black Silty CLAY

LABORATORY TEST PROCEDURE: ASTM D1557-12

MAXIMUM DRY DENSITY: 106.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 22.0 %

Silicon Valley Soil Engineering  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>COMPACTION TEST A</b>  Proposed Mixed-Use Development  138 Stockton Avenue San Jose, California	File No. SV1404	FIGURE  5
		Drawn by: V.V.	
		Scale: NOT TO SCALE	October 2016



SAMPLE: A  
DESCRIPTION: Black Silty CLAY

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

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### R-VALUE TEST

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No. SV1404

Drawn by: V.V.

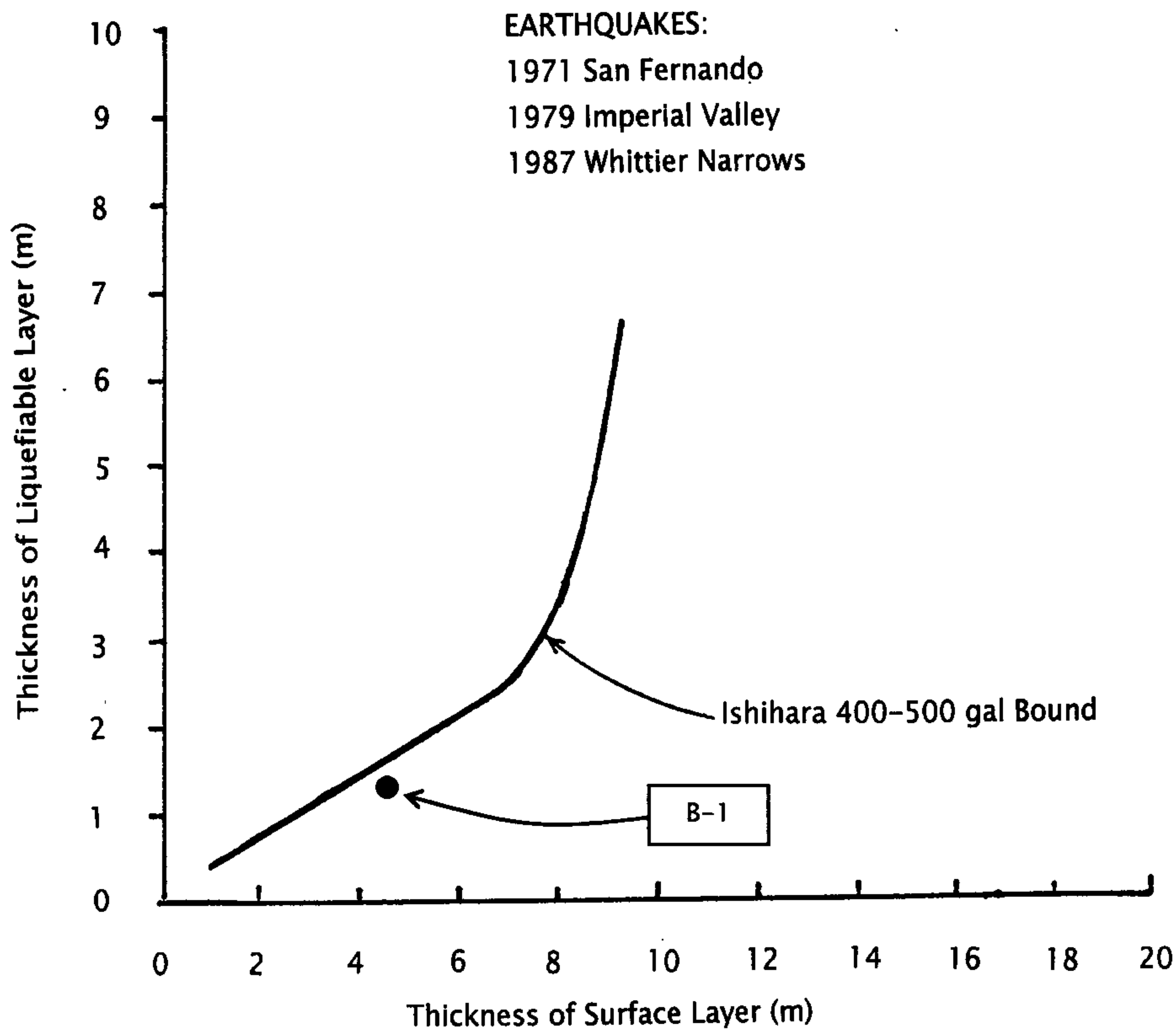
Scale: NOT TO SCALE

FIGURE

6

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Magnitude Range: 5.9 to 7.0  $M_w$   
 Acceleration Range: 0.37 to 0.55g



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**LIQUEFACTION-INDUCED  
GROUND DAMAGE**  
Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No. SV1404

Drawn by: V.V.

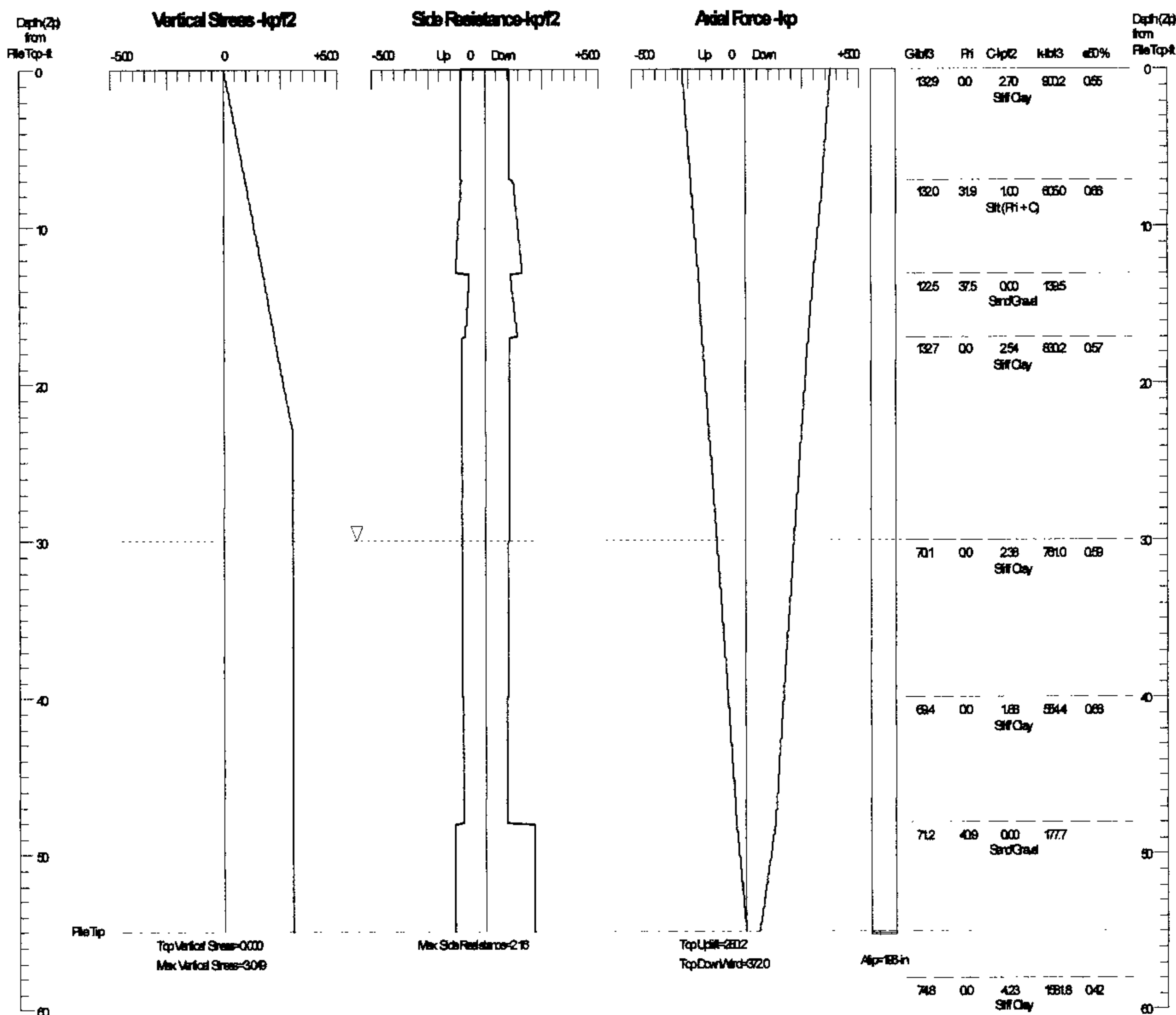
Scale: NOT TO  
SCALE

FIGURE

7

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



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SOIL STRESS, SIDE  
RESISTANCE, & AXIAL  
FORCE vs DEPTH

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

Scale: NOT TO SCALE

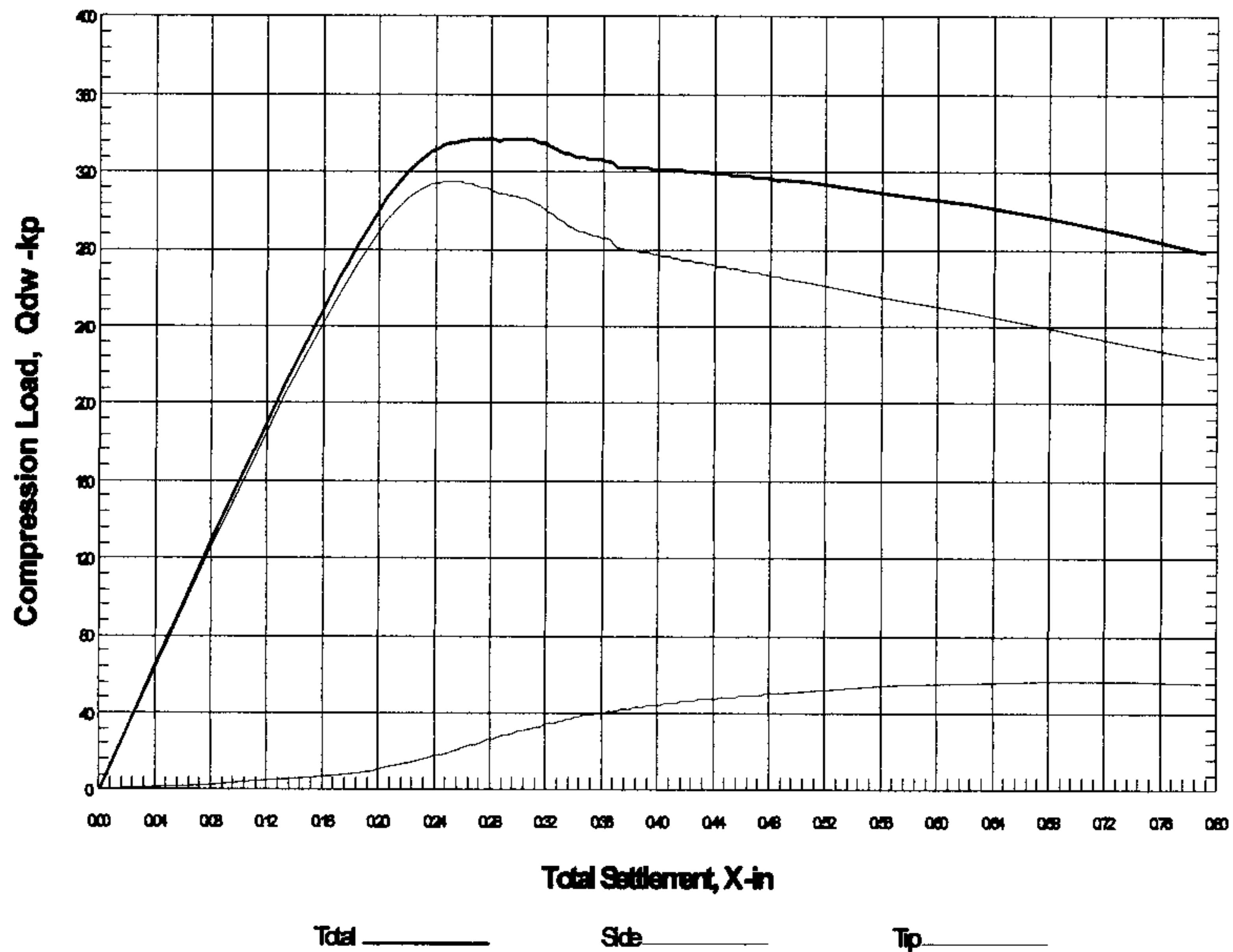
FIGURE

8

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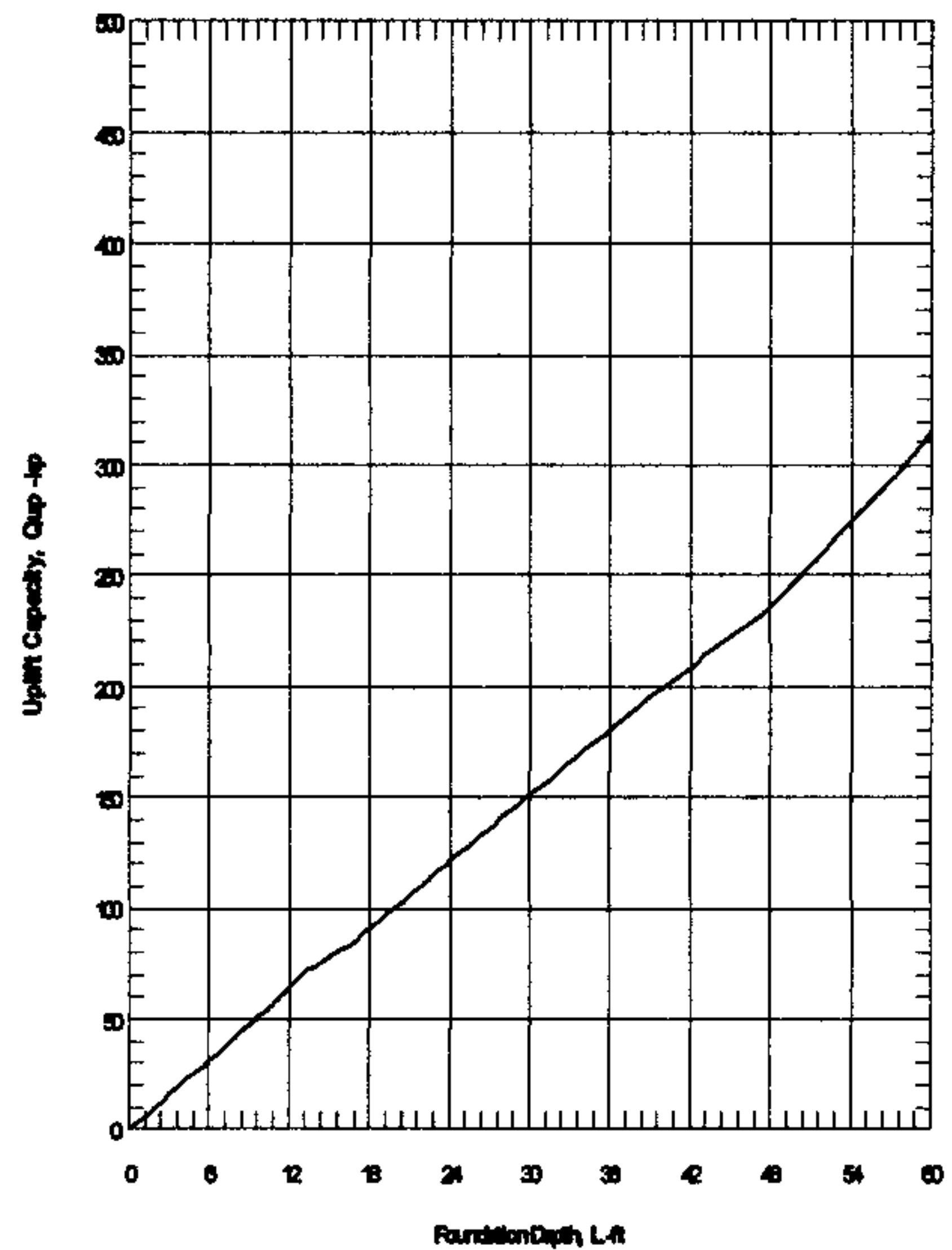
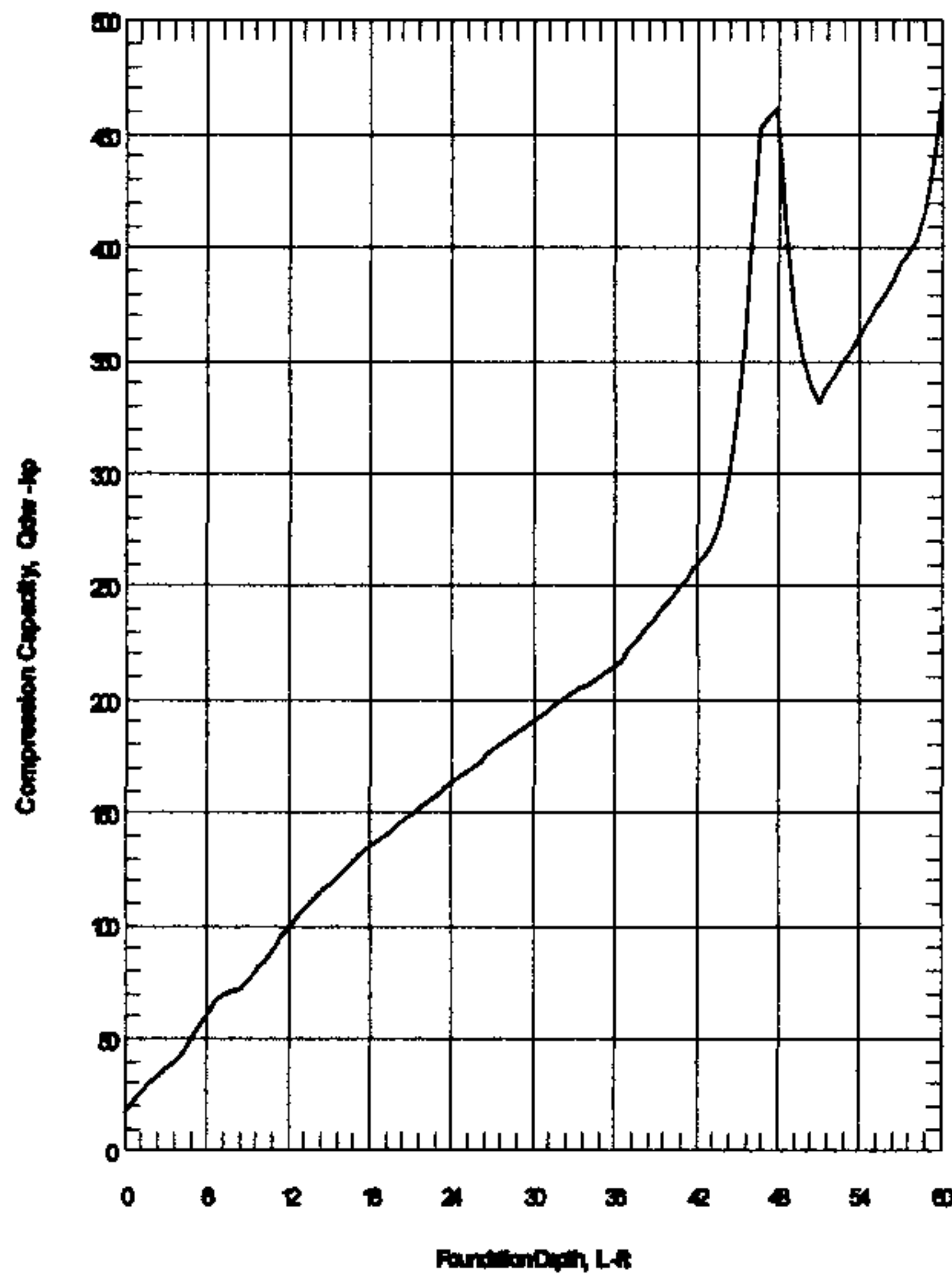
## Vertical Load vs. Total Settlement



Total \_\_\_\_\_ Side \_\_\_\_\_ Tip \_\_\_\_\_

<b>Silicon Valley Soil Engineering</b>  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>VERTICAL LOAD vs TOTAL SETTLEMENTS</b>  Proposed Mixed-Use Development  138 Stockton Avenue San Jose, California	File No.: SV1404	FIGURE  9
		Drawn by: V.V.	
		Scale: NOT TO SCALE	October 2016

## ULTIMATE CAPACITY vs FOUNDATION DEPTH



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Engineering**

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**ULTIMATE CAPACITY vs  
FOUNDATION DEPTH**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

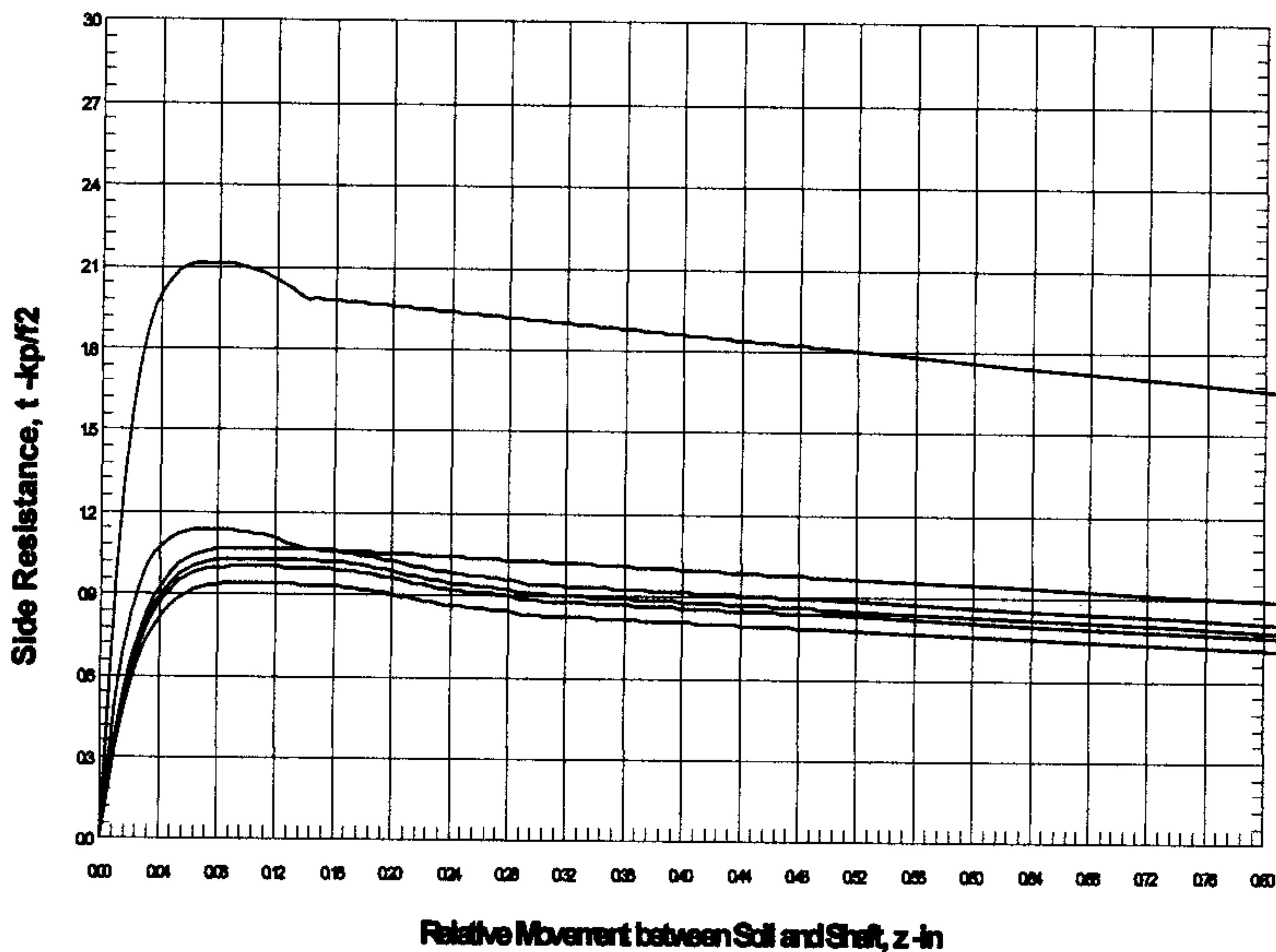
Scale: NOT TO SCALE

**FIGURE**

10

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



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**SIDE RESISTANCE VS  
RELATIVE MOVEMENT  
BETWEEN SOIL AND SHAFT**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

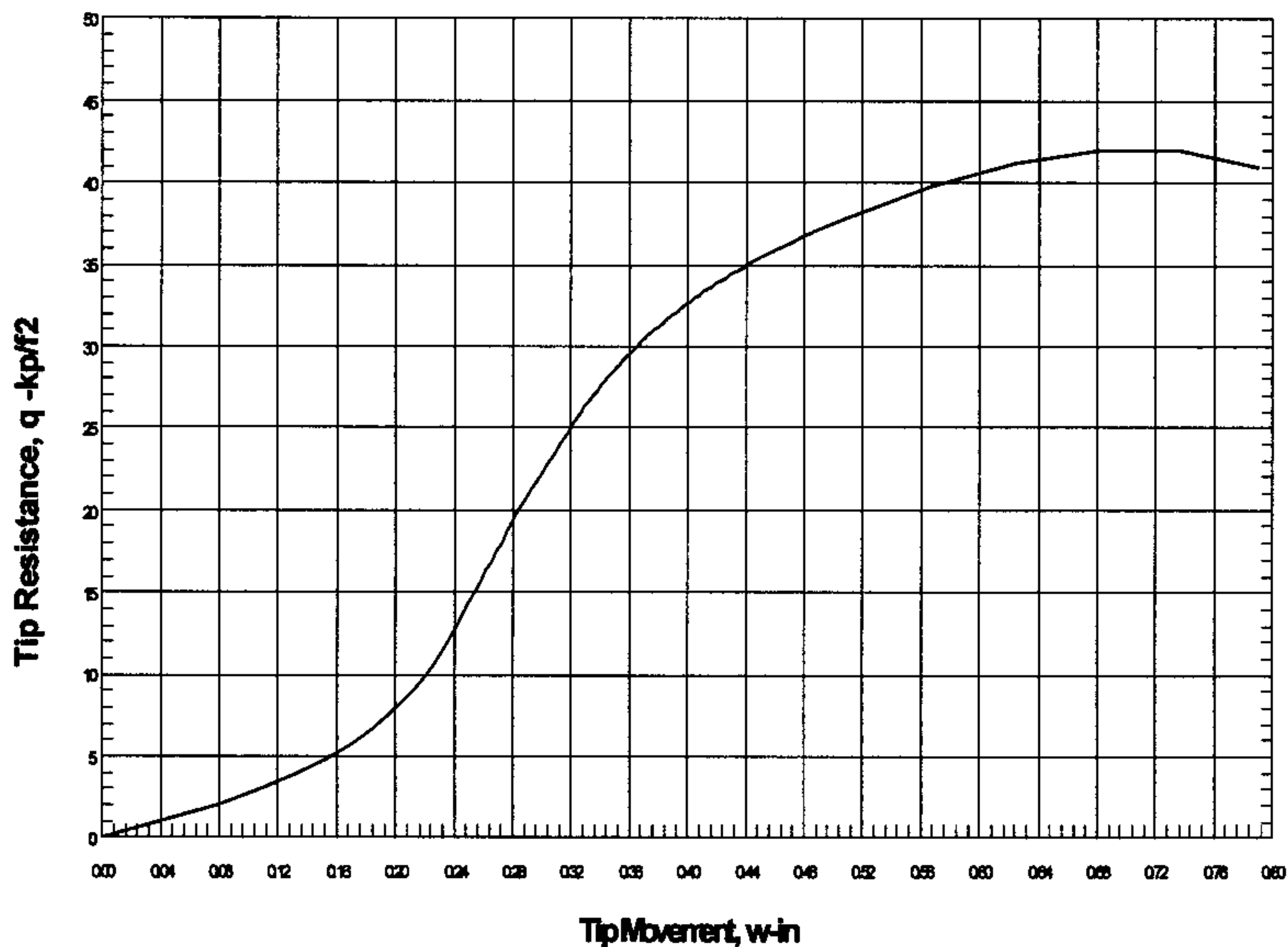
Scale: NOT TO SCALE

FIGURE

11

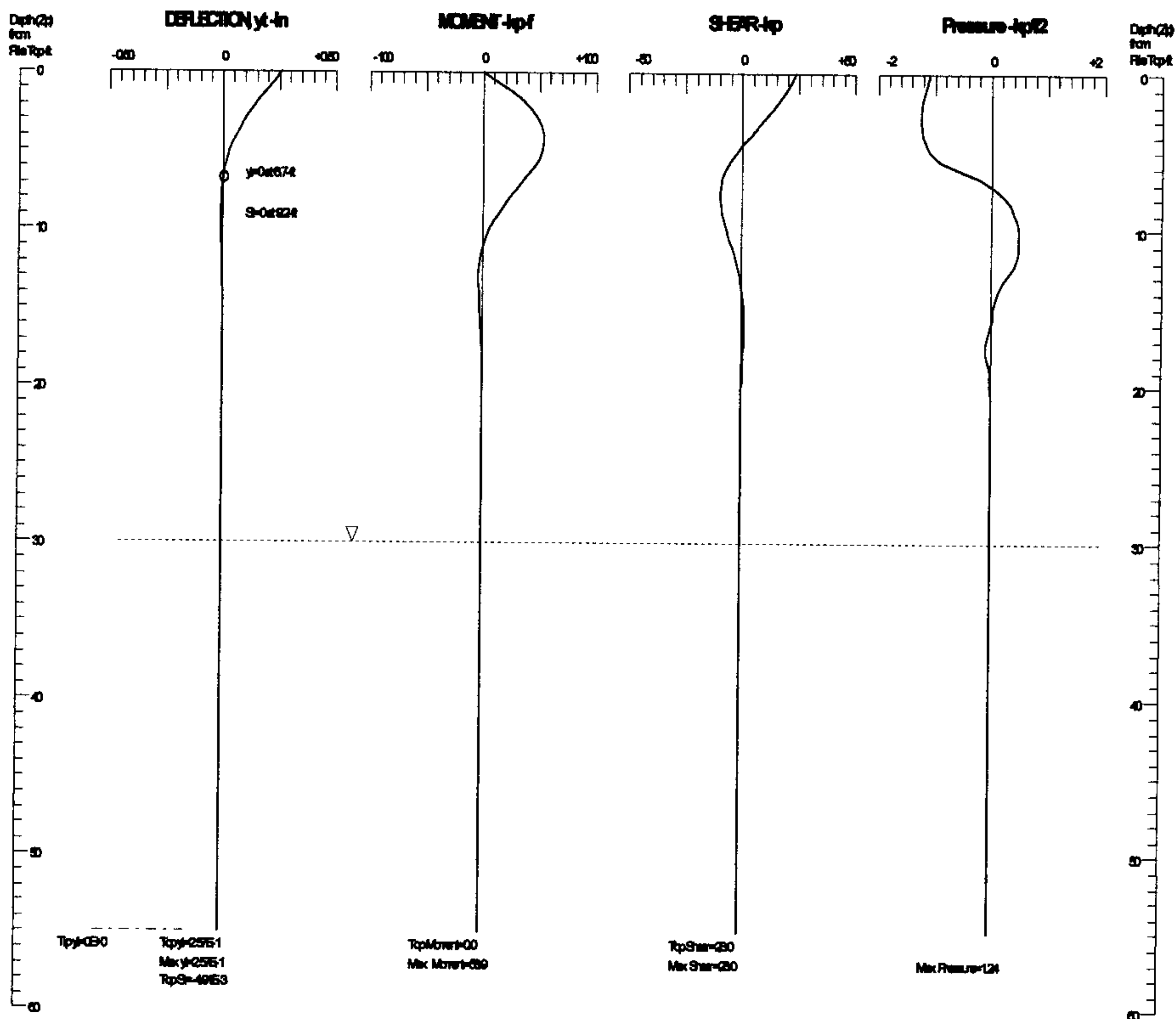
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## Tip Resistance vs. Tip Movement (q-w)



<p><b>Silicon Valley Soil Engineering</b></p> <p>2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400</p>	<p><b>TIP RESISTANCE VS TIP MOVEMENT</b></p> <p>Proposed Mixed-Use Development 138 Stockton Avenue San Jose, California</p>	<p>File No. SV1404</p> <p>Drawn by: V.V.</p> <p>Scale: NOT TO SCALE</p>	<p><b>FIGURE</b></p> <p>12</p> <p>October 2016</p>
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**PILE DEFLECTION & FORCE vs DEPTH**  
Single Pile, Head=1, Max=1



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Engineering**

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**PILE DEFLECTION &  
FORCE vs DEPTH  
(FREE END)**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

Scale: NOT TO SCALE

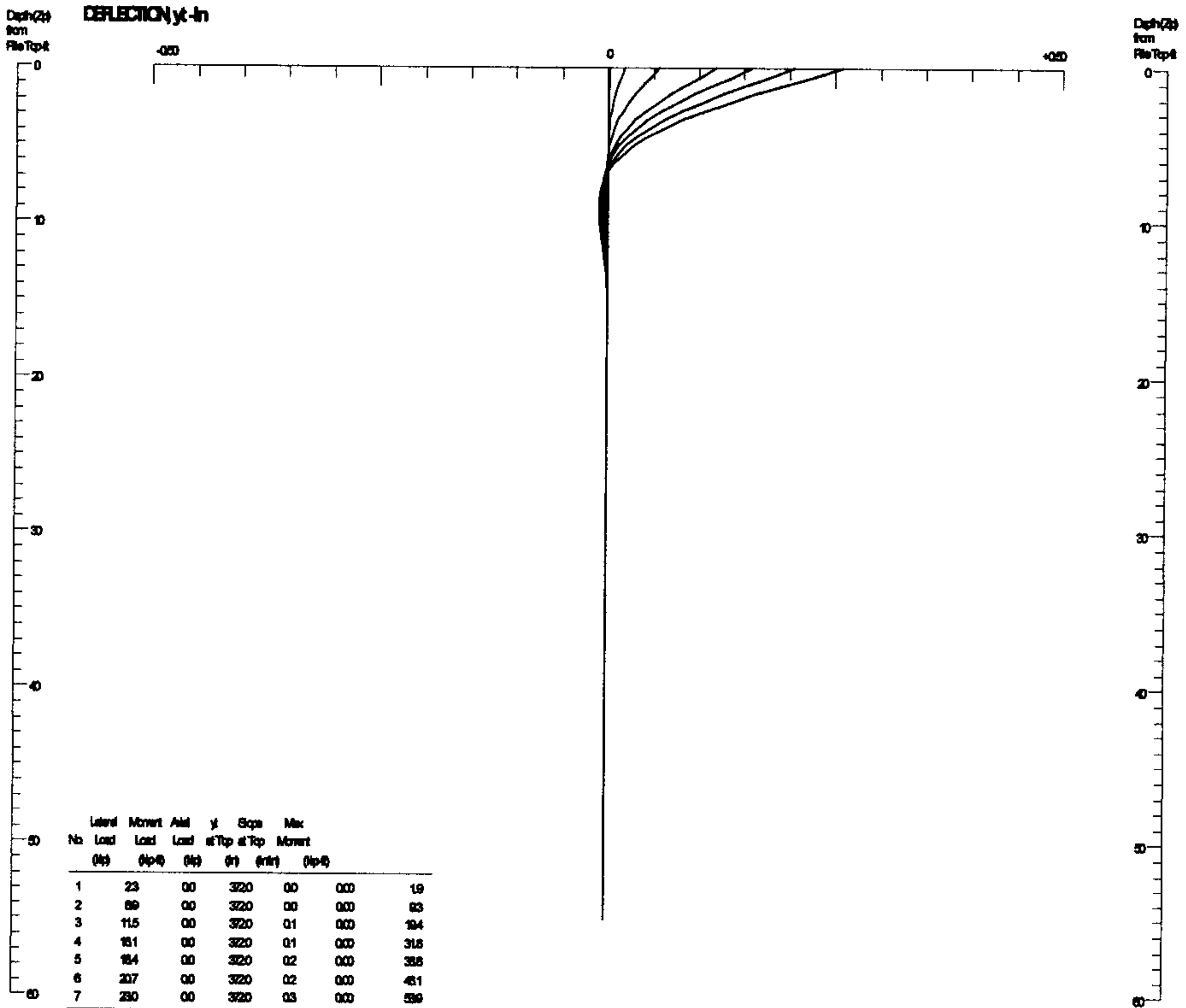
**FIGURE**

13

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**PILE DEFLECTION vs LOADING**  
Single Pile, Head=1, H=1



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Engineering**

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**PILE DEFLECTION vs  
LOADING  
(FREE END)**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

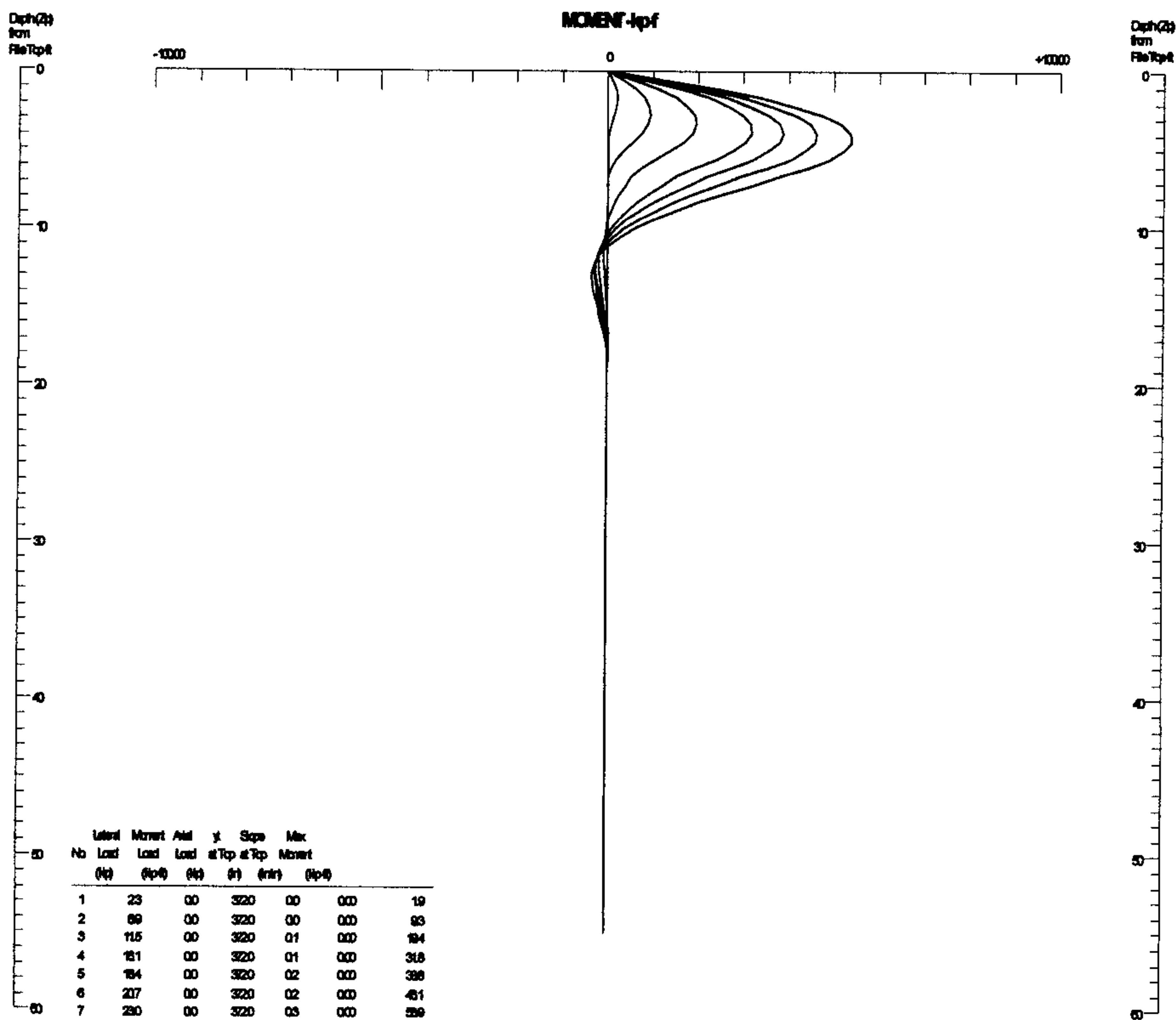
Scale: NOT TO SCALE

**FIGURE**

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# PILE MOMENT vs LOADING Single Pile, Head=1, K<sub>h</sub>=1



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PILE MOMENT vs  
LOADING  
(FREE END)  
Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

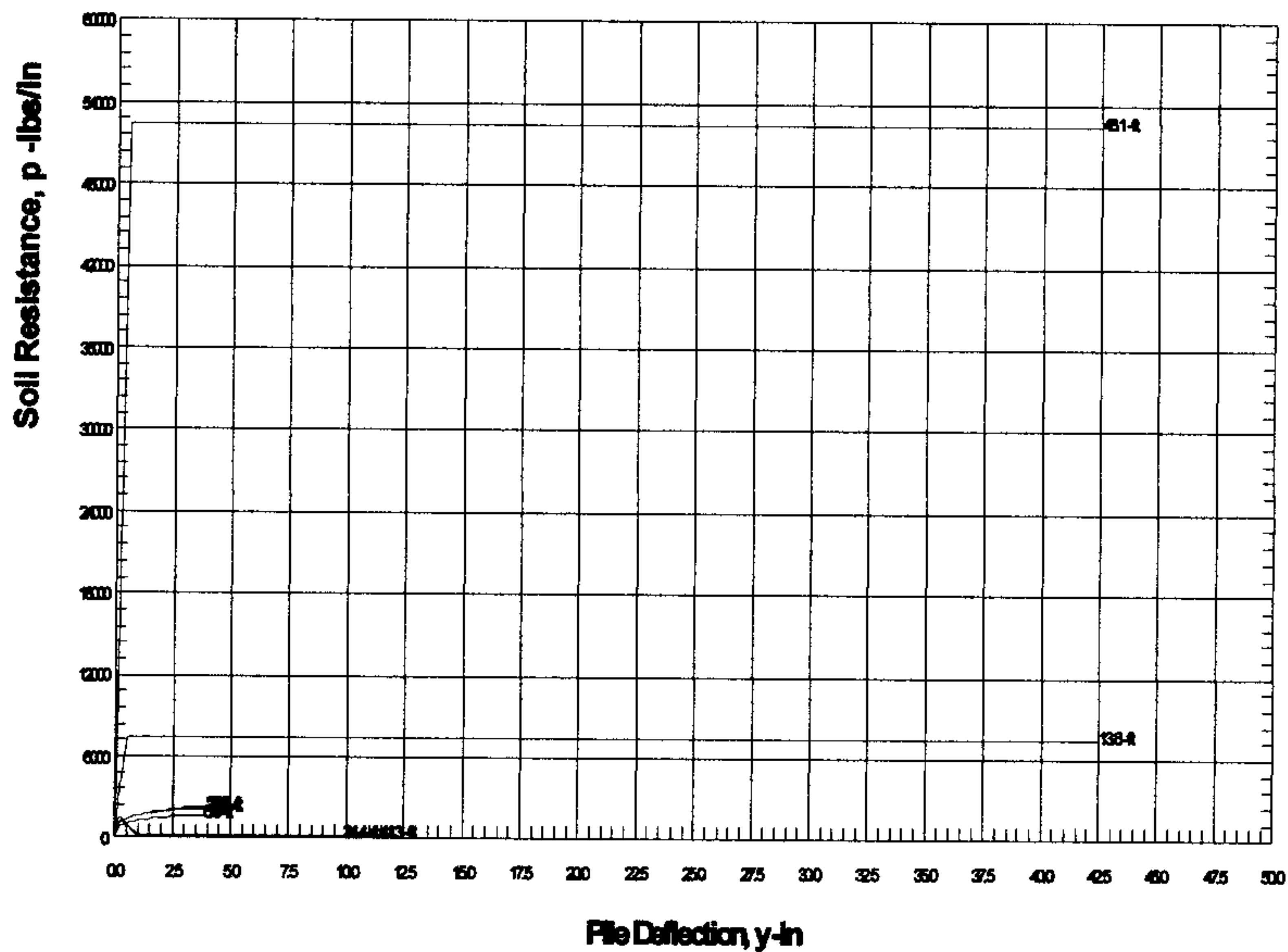
Scale: NOT TO SCALE

FIGURE

15

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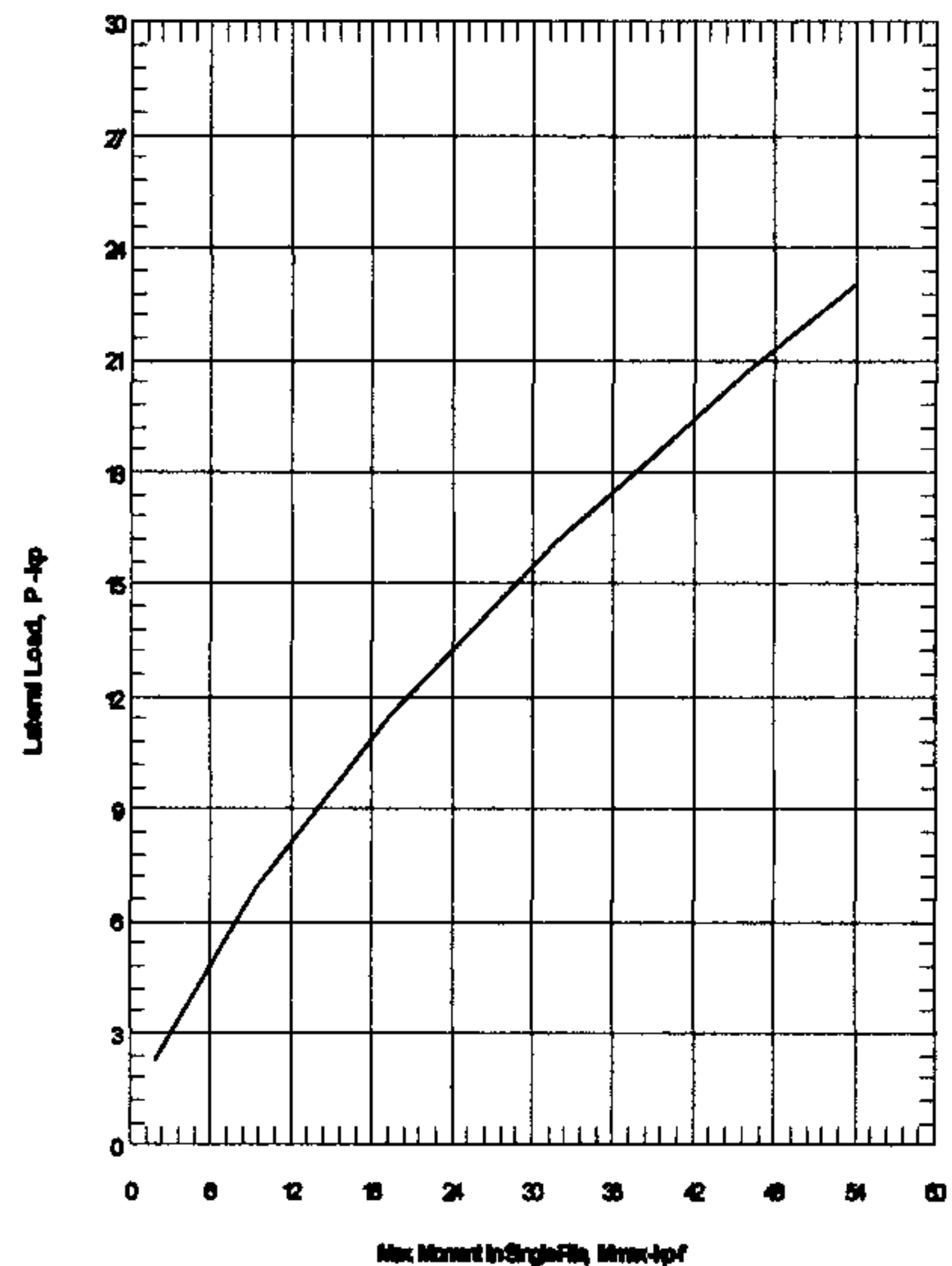
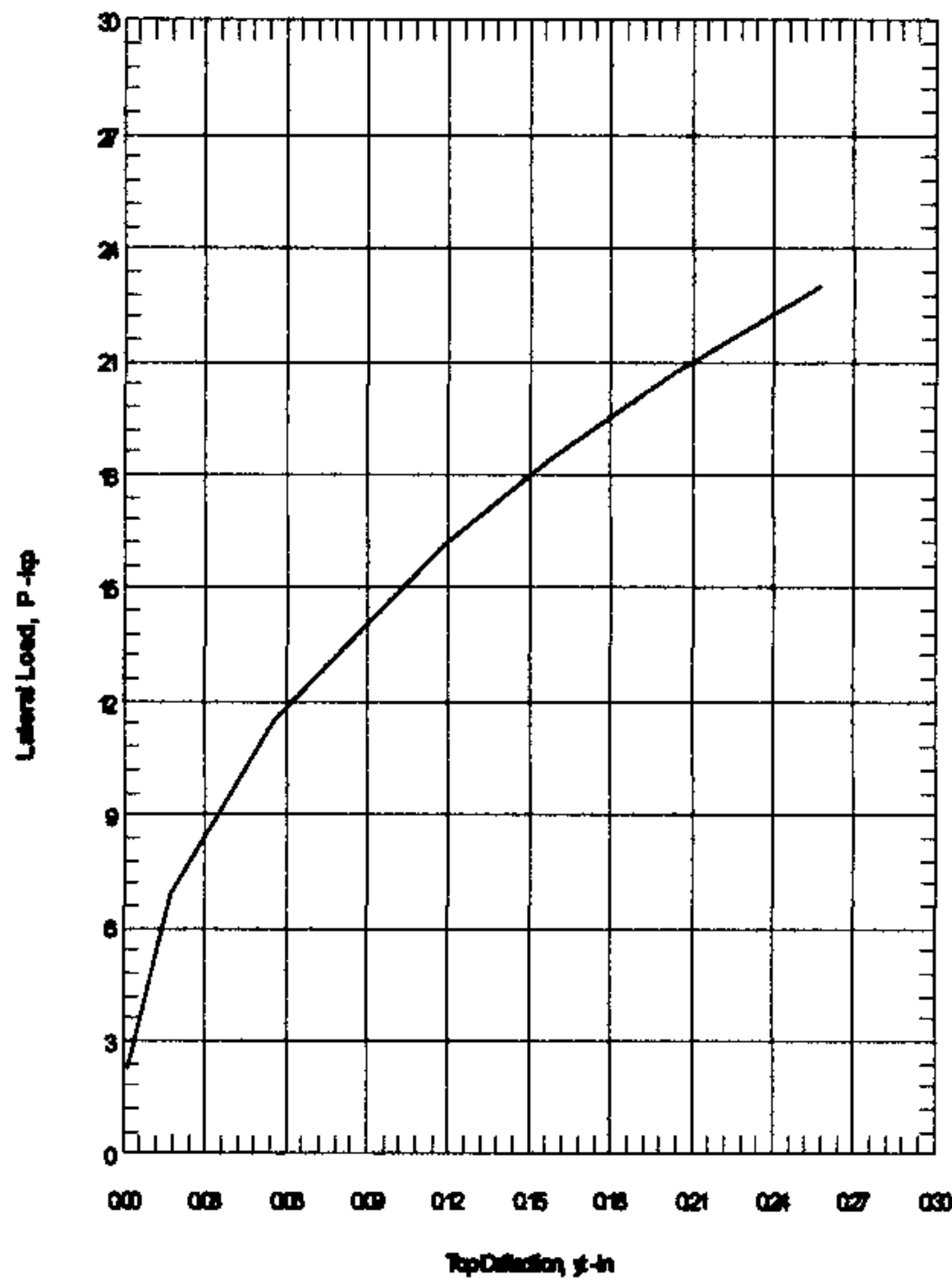
## Soil Resistance vs. Pile Deflection (p-y)



Soil Depth (ft): 0.69, 1.38, 2.06, 2.75, 3.4, 4.1, 4.3 ft

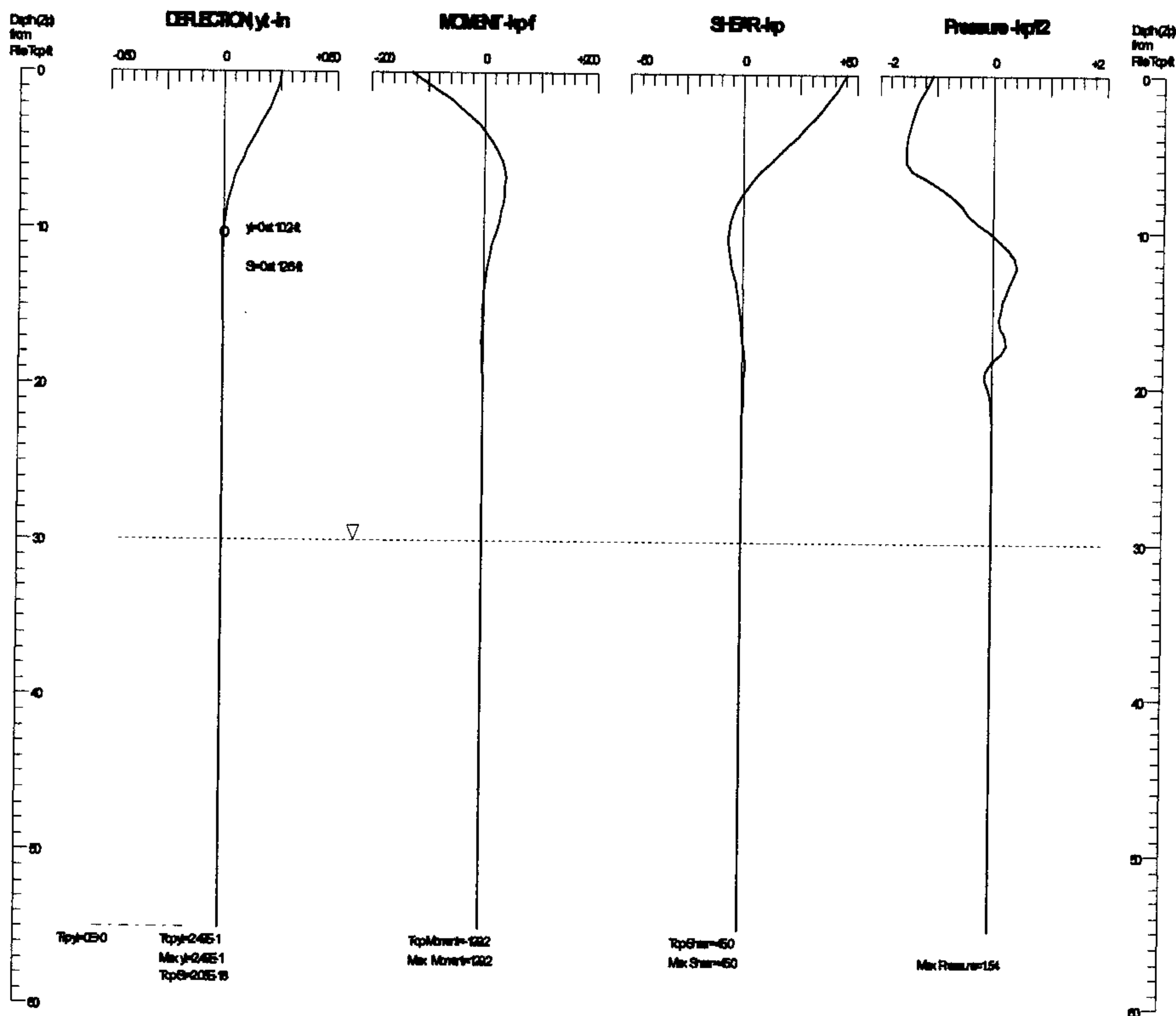
<b>Silicon Valley Soil Engineering</b>  2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400	<b>SOIL RESISTANCE vs PILE DEFLECTION (FREE END)</b>  Proposed Mixed-Use Development  138 Stockton Avenue San Jose, California	File No.: SV1404	FIGURE  16
		Drawn by: V.V.	
		Scale: NOT TO SCALE	October 2016

## LATERAL LOAD vs DEFLECTION & MAX. MOMENT



<p><b>Silicon Valley Soil Engineering</b></p> <p>2391 Zanker Road, #350 San Jose, CA 95131 (408) 324-1400</p>	<p><b>LATERAL LOAD vs DEFLECTION &amp; MAX. MOMENT (FREE END)</b></p> <p>Proposed Mixed-Use Development 138 Stockton Avenue San Jose, California</p>	<p>File No. SV1404</p>	<p><b>FIGURE</b></p> <p>17</p> <p>October 2016</p>
		<p>Drawn by: V.V.</p>	
		<p>Scale: NOT TO SCALE</p>	

**PILE DEFLECTION & FORCE vs DEPTH**  
Single Pile, Head=6, K=2



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**PILE DEFLECTION &  
FORCE vs DEPTH  
(FIXED END)**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

Scale: NOT TO SCALE

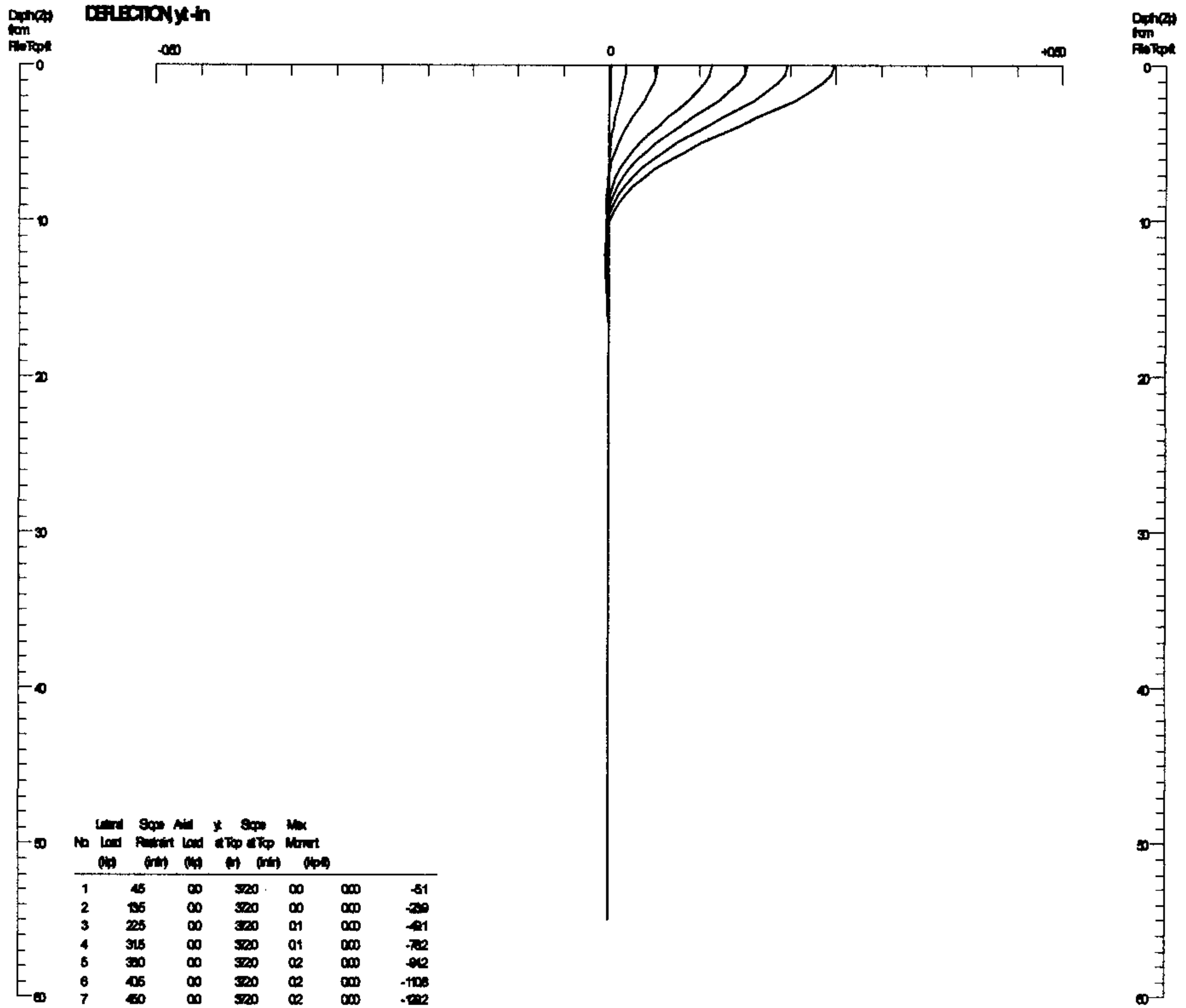
**FIGURE**

18

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**PILE DEFLECTION vs LOADING**  
Single Pile, Head=5, K=2



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**PILE DEFLECTION vs  
LOADING  
(FIXED END)**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

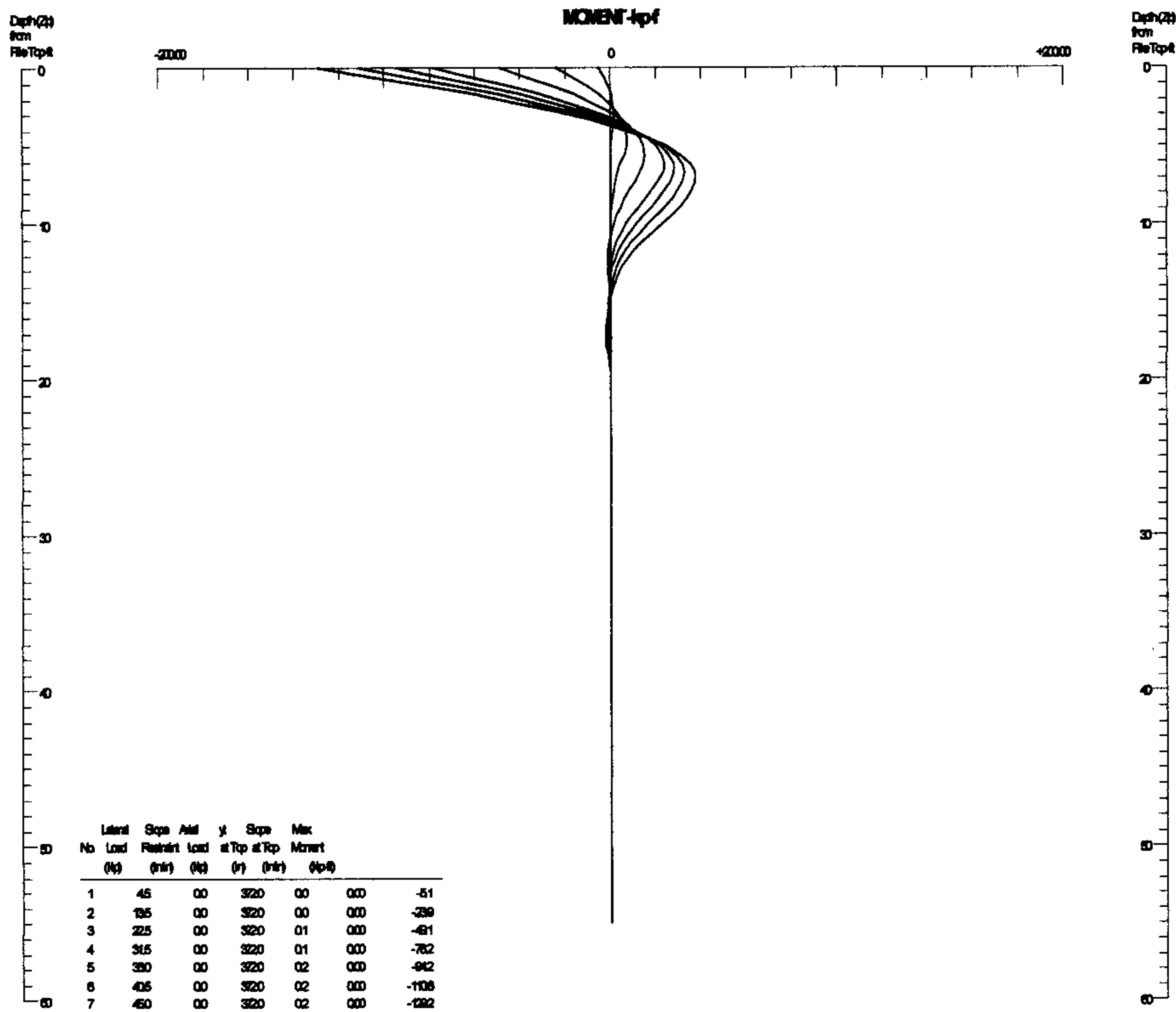
Scale: NOT TO SCALE

**FIGURE**

19

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# PILE MOMENT vs LOADING Single Pile, Head-5, No-2



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PILE MOMENT vs  
LOADING  
(FIXED END)  
Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

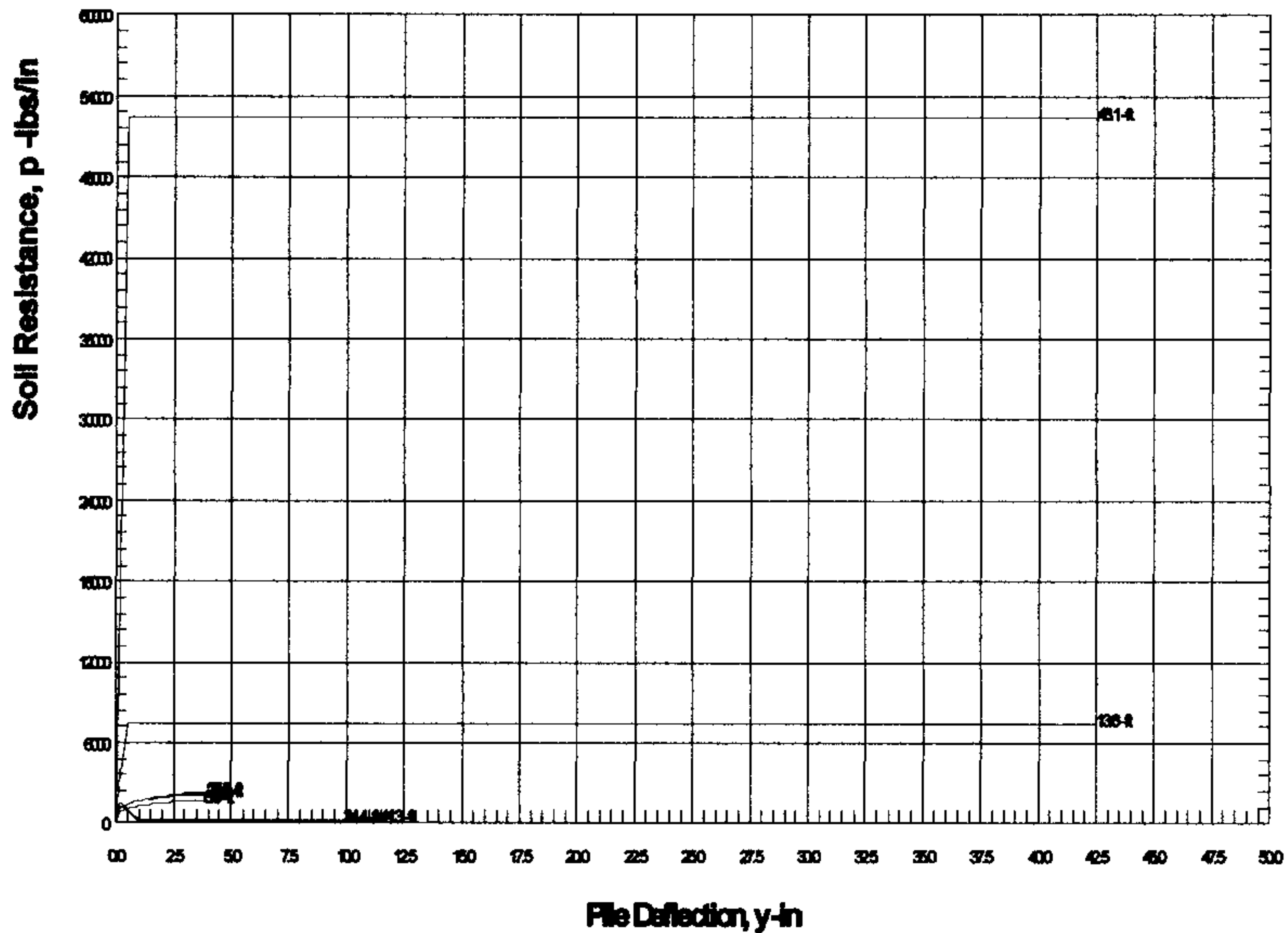
Scale: NOT TO SCALE

FIGURE

20

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## Soil Resistance vs. Pile Deflection (p-y)



Soil Depth (ft): 68, 138, 206, 275, 344, 413, 481-ft

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**SOIL RESISTANCE vs  
PILE DEFLECTION  
(FIXED END)**

Proposed Mixed-Use  
Development

138 Stockton Avenue  
San Jose, California

File No.: SV1404

Drawn by: V.V.

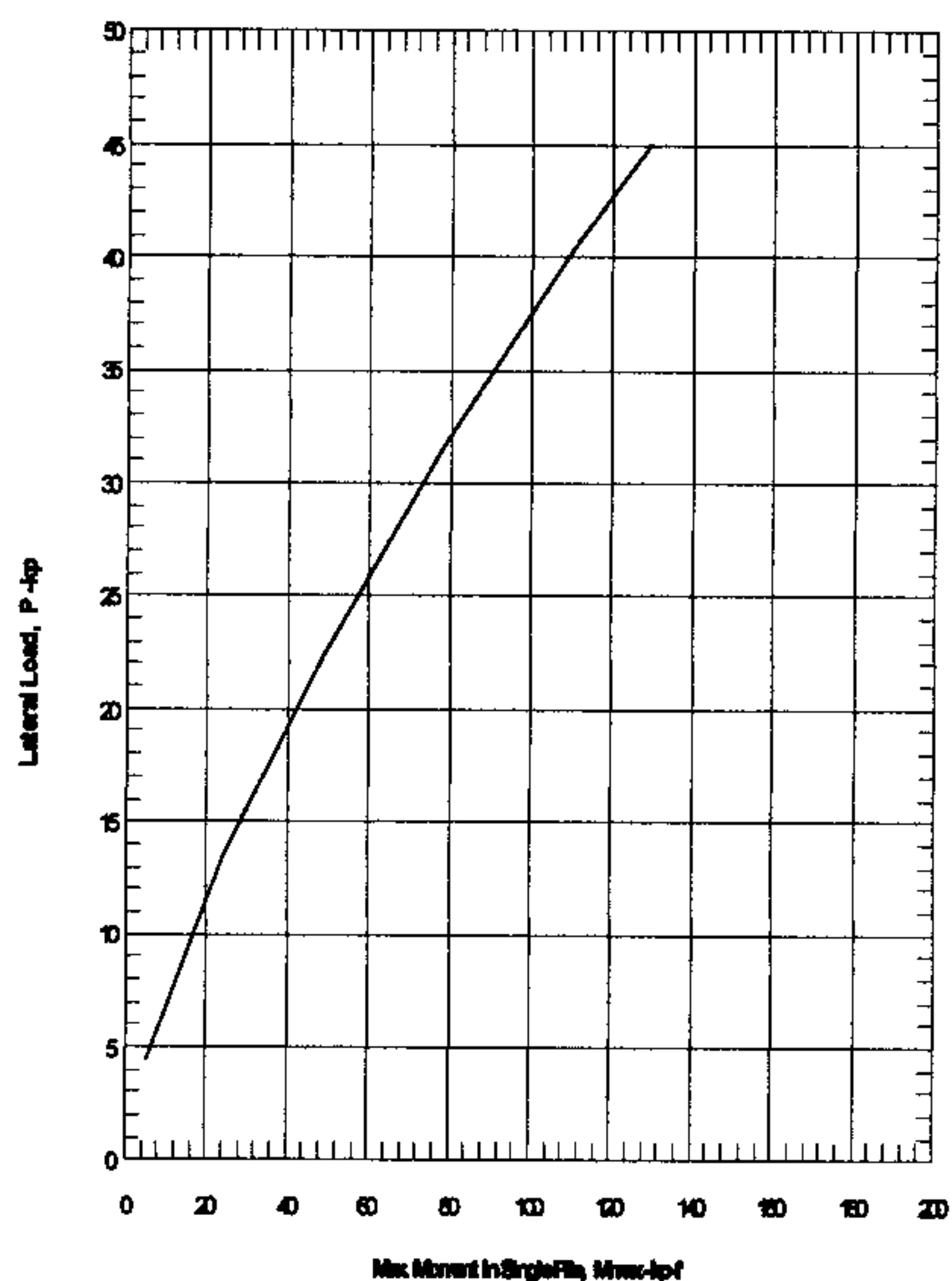
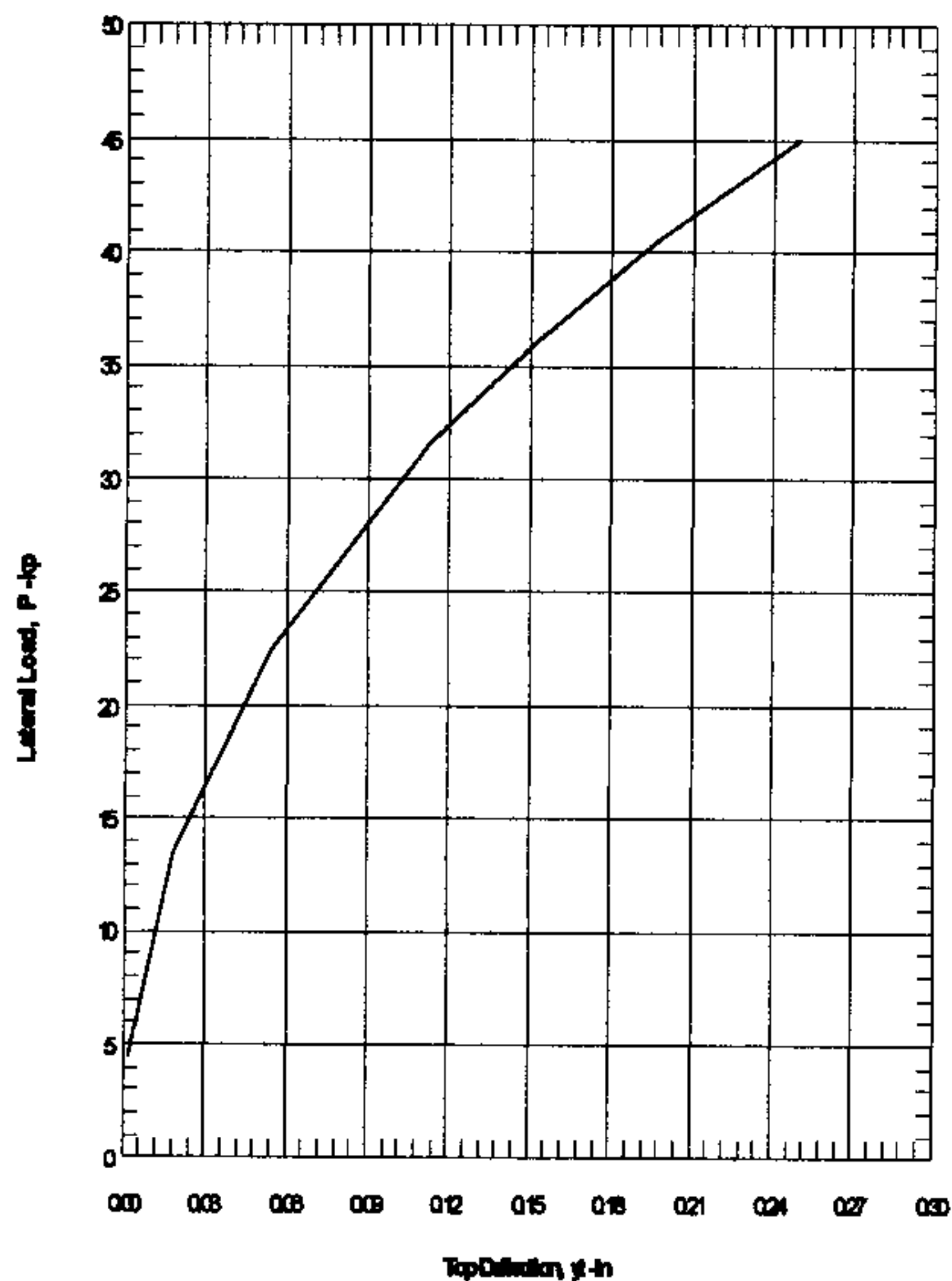
Scale: NOT TO SCALE

**FIGURE**

21

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## LATERAL LOAD vs DEFLECTION & MAX MOMENT



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LATERAL LOAD vs  
DEFLECTION & MAX. MOMENT  
(FIXED END)

Proposed Mixed-Use  
Development  
138 Stockton Avenue  
San Jose, California

File No. SV1404

Drawn by: V.V.

Scale: NOT TO SCALE

FIGURE

22

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## APPENDICES

MODIFIED MERCALLI SCALE

METHOD OF SOIL CLASSIFICATION

KEY TO LOG OF BORING

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

LIQUEFACTION ANALYSIS

PILE SPECIFICATIONS

SANTA CLARA VALLEY WATER DISTRICT DRILLING PERMIT



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

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

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

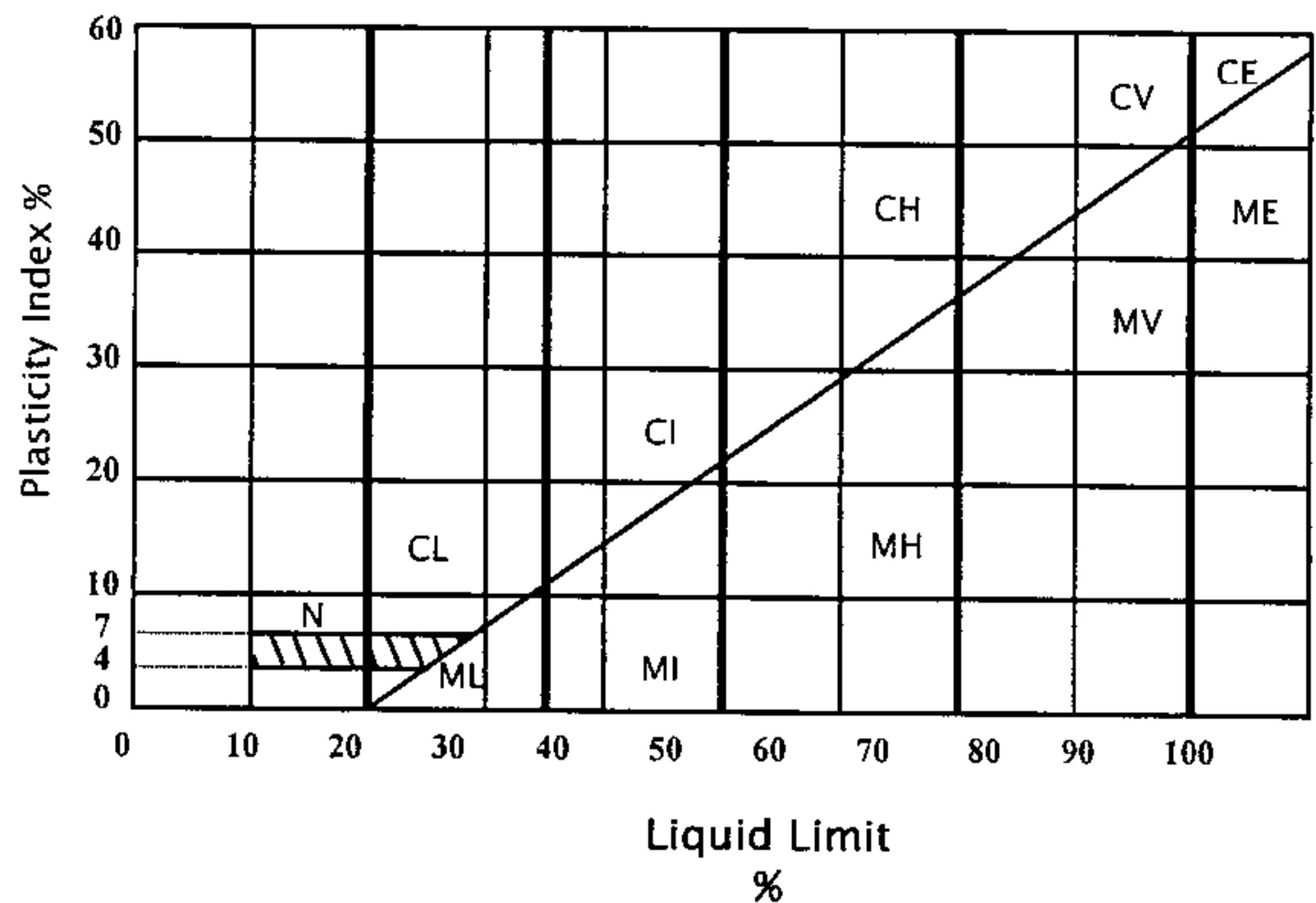
# METHOD OF SOIL CLASSIFICATION CHART

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

CLASSIFICATION CHART - UNIFIED SOIL CLASSIFICATION SYSTEM

PLASTICITY INDEX CHART

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



Project: Proposed Mixed-Use Development  
Project Location: 138 Stockton Avenue  
San Jose, California  
Project Number: SV1404

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

## Key to Log of Boring Sheet 1 of 1

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

### COLUMN DESCRIPTIONS





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


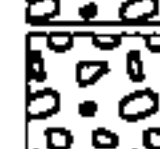
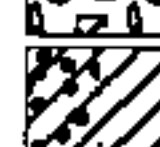

### FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity  
COMP: Compaction test  
CONS: One-dimensional consolidation test  
LL: Liquid Limit, percent







PI: Plasticity Index, percent  
SA: Sieve analysis (percent passing No. 200 Sieve)  
UC: Unconfined compressive strength test, Qu, in ksf  
WA: Wash sieve (percent passing No. 200 Sieve)




### MATERIAL GRAPHIC SYMBOLS

	Asphaltic Concrete (AC)
	Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)
	Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
	SILTY CLAY (CL-ML)






	Poorly graded GRAVEL (GP)
	Aggregate Base (AB)
	Clayey SAND to Sandy CLAY (SC-CL)
	Poorly graded SAND (SP)

### TYPICAL SAMPLER GRAPHIC SYMBOLS

	Auger sampler		CME Sampler
	Bulk Sample		Grab Sample
	3-inch-OD California w/ brass rings		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 Mixed-Use Development  
 Project Location: 138 Stockton Avenue  
 San Jose, California  
 Project Number: SV1404

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

# Log of Boring B-1 Sheet 1 of 3

Date(s) Drilled	6/30/15	Logged By	V.V.	Checked By	
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8-inch	Total Depth of Borehole	81.5 feet
				Approximate Surface Elevation	87 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, %
0.20				Asphalt		3.0 inches Asphalt Concrete (AC)						
0.58				CH		4.0 inches Aggregate Base (AB)						
						Black Silty CLAY						
						Moist, very stiff						
						Color changed to dark brown	25.9	89.5	1.0	10		
5		1-1	11									
		1-2	32				27.3	87.8				
7												
				CL-ML		Olive Brown Clayey SILT						
						Moist, stiff						
						Color changed to light olive brown	20.9	105.6	0.5	22		
10		1-3	16									
				SP		Light Olive Brown Silty SAND						
						Moist, medium dense						
						SAND: medium grained, poorly graded						
15		1-4	26				15.9	110.3				<12
				CL		Medium Bluish Gray Silty CLAY						
						Moist, very stiff						
20		1-5	23				32.0	90.6	0.8	10	41	19
25		1-6	14				23.0	101.6			40	20
30						Stabilized at the end of the drilling operation						



Log of Boring B-1  
Sheet 2 of 3



Log of Boring B-1  
Sheet 3 of 3



<b>Project:</b> Proposed Mixed-Use Development <b>Project Location:</b> 138 Stockton Avenue San Jose, California <b>Project Number:</b> SV1404	<b>Silicon Valley Soil Engineering</b> 2391 Zanker Road, Suite 350 San Jose, CA 95131 (408) 324-1400	<b>Log of Boring B-3</b> <b>Sheet 1 of 2</b>
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Date(s) Drilled <b>06/30/15</b>	Logged By <b>V.V.</b>	Checked By
Drilling Method <b>Hollow Stem Auger</b>	Drill Bit Size/Type <b>8-inch</b>	Total Depth of Borehole <b>41.5 feet</b>
		Approximate Surface Elevation <b>88 feet</b>
Groundwater Level and Date Measured	Sampling Method(s) <b>SPT</b>	Hammer Data <b>140 lbs</b>
Borehole Backfill <b>Grout</b>	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, %
0.25				Asphalt		3.0 inches Asphalt Concrete (AC)						
0.75				CH		6.0 inches Aggregate Base (AB)						
						Black Silty CLAY Moist, very stiff						
		3-1	18			Color changed to dark brown	18.6	89.8				
5		3-2	28				18.5	112.3				
7				CL-ML		Olive Brown Silty CLAY/Clayey SILT Moist, very stiff						
10		3-3	21				18.9	109.2				
15		3-4	15				16.6	107.4				
17				SP		Olive Brown Gravelly SAND Moist, medium dense SAND: medium grained, poorly graded						
20		3-5	24				10.5	110.6				
23				ML		Olive Brown Sandy SILT Moist, very stiff						
25		3-6	27				16.3	103.2				
28				CL		Dark Bluish Gray Silty CLAY Moist, very stiff						
30						Stabilized at the end of the drilling operation						

Log of Boring B-3  
Sheet 2 of 2







Liquefy.sum

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\*\*\*\*\*

LIQUEFACTION ANALYSIS SUMMARY

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Font: Courier New, Regular, Size 8 is recommended for this report.  
Licensed to , 7/6/2015 4:31:05 PM

Input File Name: \\FILE-SERVER\use\SVSE Files\SV Main File\SV MAIN FILE\SV  
(1400-1409)\SV1404 - Stockton\SV1404.Liquefaction Analysis.liq  
Title: SV1404 - Proposed Mixed-Use Development  
Subtitle: 138 Stockton Avenue, San Jose, CA

Surface Elev.=100  
Hole No.=B-1  
Depth of Hole= 51.50 ft  
Water Table during Earthquake= 13.00 ft  
Water Table during In-Situ Testing= 30.00 ft  
Max. Acceleration= 0.55 g  
Earthquake Magnitude= 7.90

Input Data:

Surface Elev.=100  
Hole No.=B-1  
Depth of Hole=51.50 ft  
Water Table during Earthquake= 13.00 ft  
Water Table during In-Situ Testing= 30.00 ft  
Max. Acceleration=0.55 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,
  7. Borehole Diameter,
  8. Sampling Method,
  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

Ce = 0.88  
Cb= 1.15  
Cs= 1.2

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	11.00	112.70	NoLiq
5.00	32.00	111.80	NoLiq
7.00	16.00	127.70	NoLiq
13.00	26.00	127.80	5.00
17.00	23.00	119.60	NoLiq
23.00	14.00	125.00	NoLiq
30.00	20.00	118.30	NoLiq
34.00	22.00	128.00	NoLiq
40.00	16.00	123.90	NoLiq
44.00	15.00	126.20	NoLiq

48.00 54.00 121.00 5.00 Liquefy.sum

Output Results:

Settlement of Saturated Sands=0.32 in.  
 Settlement of Unsaturated Sands=0.08 in.  
 Total Settlement of Saturated and Unsaturated Sands=0.39 in.  
 Differential Settlement=0.196 to 0.259 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.46	5.00	0.32	0.08	0.39
1.00	2.00	0.46	5.00	0.32	0.08	0.39
2.00	2.00	0.46	5.00	0.32	0.08	0.39
3.00	2.00	0.46	5.00	0.32	0.08	0.39
4.00	2.00	0.46	5.00	0.32	0.08	0.39
5.00	2.00	0.46	5.00	0.32	0.08	0.39
6.00	2.00	0.46	5.00	0.32	0.08	0.39
7.00	0.44	0.46	5.00	0.32	0.08	0.39
8.00	0.44	0.46	5.00	0.32	0.06	0.38
9.00	0.44	0.45	5.00	0.32	0.05	0.37
10.00	0.44	0.45	5.00	0.32	0.04	0.35
11.00	0.44	0.45	5.00	0.32	0.03	0.34
12.00	0.44	0.45	5.00	0.32	0.02	0.33
13.00	0.44	0.45	0.97*	0.32	0.00	0.32
14.00	0.44	0.47	0.94*	0.30	0.00	0.30
15.00	0.44	0.48	0.91*	0.30	0.00	0.30
16.00	0.44	0.50	0.88*	0.30	0.00	0.30
17.00	2.00	0.51	5.00	0.30	0.00	0.30
18.00	2.00	0.52	5.00	0.30	0.00	0.30
19.00	2.00	0.53	5.00	0.30	0.00	0.30
20.00	2.00	0.54	5.00	0.30	0.00	0.30
21.00	2.00	0.55	5.00	0.30	0.00	0.30
22.00	2.00	0.56	5.00	0.30	0.00	0.30
23.00	2.00	0.57	5.00	0.30	0.00	0.30
24.00	2.00	0.57	5.00	0.30	0.00	0.30
25.00	2.00	0.58	5.00	0.30	0.00	0.30
26.00	2.00	0.59	5.00	0.30	0.00	0.30
27.00	2.00	0.59	5.00	0.30	0.00	0.30
28.00	2.00	0.60	5.00	0.30	0.00	0.30
29.00	2.00	0.60	5.00	0.30	0.00	0.30
30.00	2.00	0.61	5.00	0.30	0.00	0.30
31.00	2.00	0.61	5.00	0.30	0.00	0.30
32.00	2.00	0.61	5.00	0.30	0.00	0.30
33.00	2.00	0.61	5.00	0.30	0.00	0.30
34.00	2.00	0.61	5.00	0.30	0.00	0.30
35.00	2.00	0.61	5.00	0.30	0.00	0.30
36.00	2.00	0.61	5.00	0.30	0.00	0.30
37.00	2.00	0.61	5.00	0.30	0.00	0.30
38.00	2.00	0.61	5.00	0.30	0.00	0.30
39.00	2.00	0.60	5.00	0.30	0.00	0.30
40.00	2.00	0.60	5.00	0.30	0.00	0.30
41.00	2.00	0.60	5.00	0.30	0.00	0.30
42.00	2.00	0.60	5.00	0.30	0.00	0.30
43.00	2.00	0.59	5.00	0.30	0.00	0.30
44.00	2.00	0.59	5.00	0.30	0.00	0.30
45.00	0.28	0.59	0.47*	0.08	0.00	0.08
46.00	0.41	0.59	0.70*	0.00	0.00	0.00
47.00	0.41	0.58	0.71*	0.00	0.00	0.00
48.00	0.41	0.58	0.71*	0.00	0.00	0.00
49.00	0.41	0.58	0.71*	0.00	0.00	0.00
50.00	0.41	0.57	0.71*	0.00	0.00	0.00

51.00	0.41	0.57	0.72*	Liquefy.sum 0.00	0.00	0.00
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\* 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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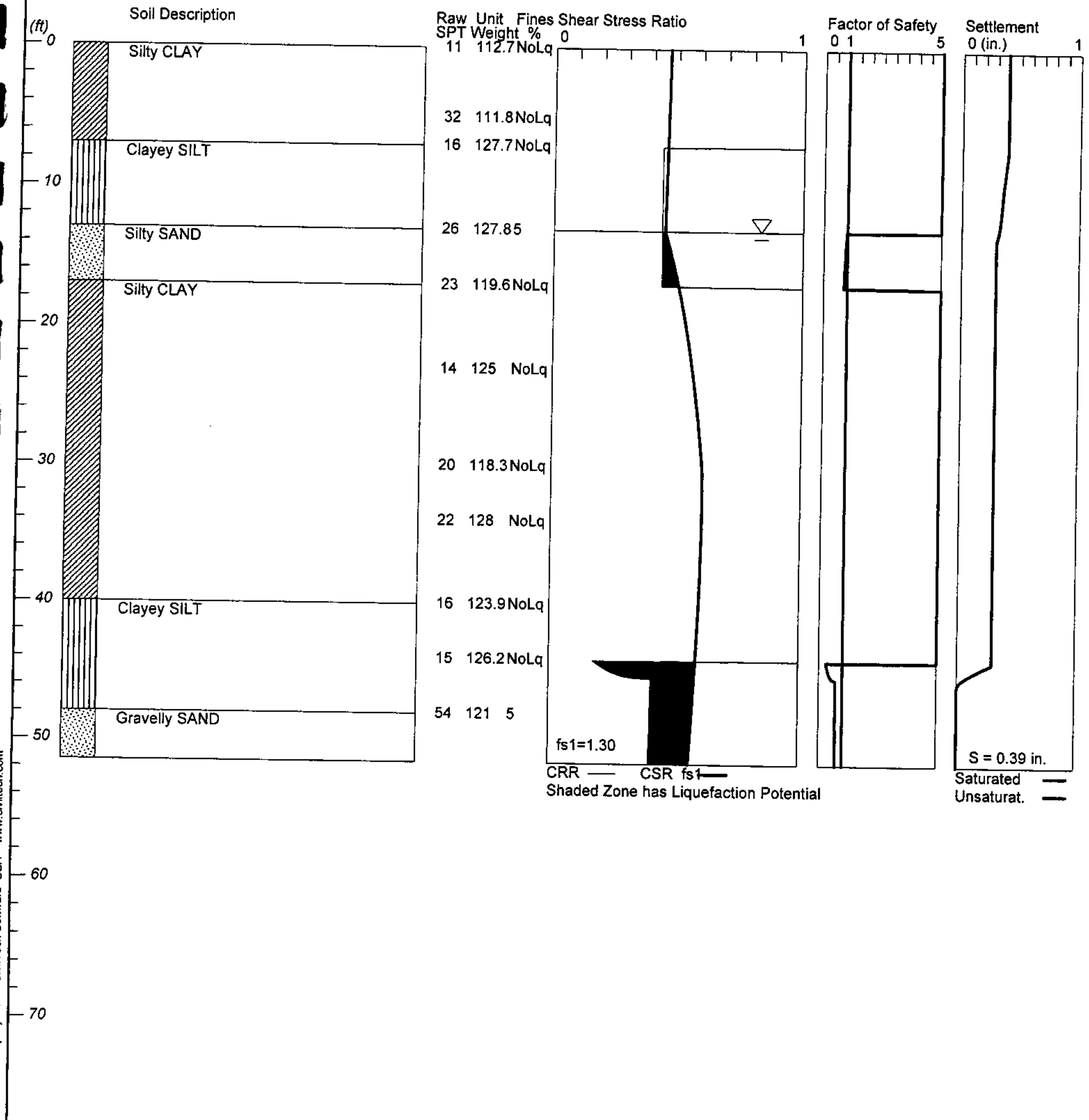
1 atm (atmosphere)	= 1 tsf (ton/ft2)
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)	
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NOLiq	No-Liquefy Soils

# LIQUEFACTION ANALYSIS

## SV1404 - Proposed Mixed-Use Development

Hole No.=B-1 Water Depth=13 ft Surface Elev.=100

Magnitude=7.9  
Acceleration=0.55g



PRE-CAST PRE-STRESS SKIN-FRICTION  
PILE SPECIFICATIONS

FOR

PROPOSED MIXED-USE DEVELOPMENT  
138 STOCKTON AVENUE  
SAN JOSE, 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 born 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 dimensions 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. Diving 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 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 CITERIA

a. Driving Energy

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

<u>Pile Type</u>	<u>Minimum Rated Hammer Energy</u>
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. Engineering Design

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. Pile Tests

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.



# APPLICATION TO DRILL EXPLORATORY BORINGS

FC 285 (10-18-10)  
Page 1 of 2

Date Issued: <b>6-18-15</b>	Expiration Date: <b>12-18-15</b>	District Permit No.: <b>E20150618003</b>
Client (if different from property owner):	Property Owner: <b>The Hudson Companies</b>	Name of Business/Residence at Site: <b>Proposed Mixed-Use Development</b>
Client's Address:  City, State, Zip	Property Owner's Address: <b>1510 South Bascom Ave., #7</b> City, State, Zip <b>Campbell, CA 95008</b>	Address of Site: <b>138 Stockton Avenue</b> City, State, Zip <b>San Jose, CA 95126</b>
Telephone No.:	Telephone No.: <b>408-628-0752</b>	Assessor's Parcel No. of Site: Book <b>259</b> Page <b>28</b> Parcel <b>004</b>
Consulting Company Name: <b>Silicon Valley Soil Engineering</b>		Drilling Company Name: <b>Exploration Geoservices Inc</b>
Address: <b>2391 Zanker Road, Suite 350</b> City, State, Zip <b>San Jose, CA 95131</b>		Address: <b>1535 Industrial Avenue</b> City, State, Zip <b>San Jose, CA 95112</b>
Telephone No.: <b>408-324-1400 / 408-910-5875</b>		Telephone No.: <b>408-280-6822</b>
<input type="checkbox"/> Check if address or phone number has changed		C-57/C-61 License No.: <b>484288</b>
<input type="checkbox"/> Check if address or phone number has changed		<input type="checkbox"/> Check if address or phone number has changed

In space at right, sketch location of proposed boring(s) in sufficient detail to identify location. In addition to distances to nearest street and intersection, show distances to any existing structures, landmarks, or topographic features.

How many borings will be installed on parcel?  
**1**

- ☐ Proposed borings on District property/easement  
(See General Condition F, page 2.)
- ☐ Within 50 feet of the top of a creek bank or District facility

Proposed depth of boring(s):

- ☒ 45 to 150 feet
- ☐ 151 to 300 feet
- ☐ Over 300 feet

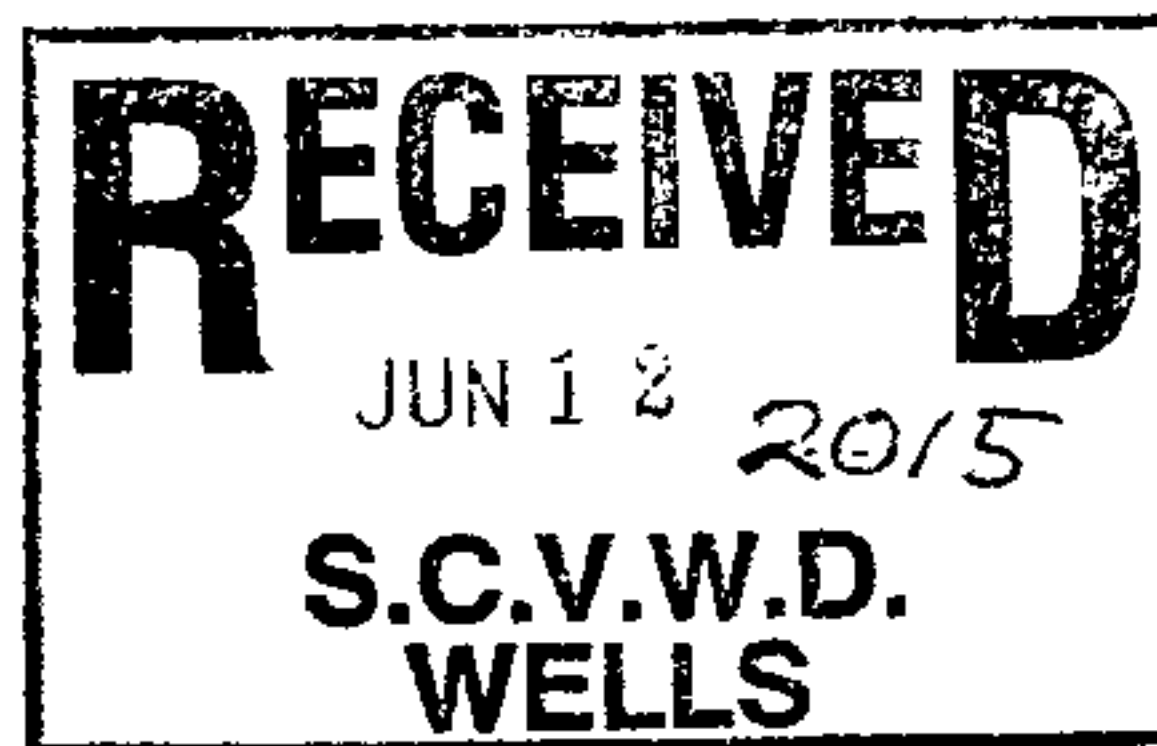
Type of boring(s):

- ☒ Hollow stem
- ☐ Rotary
- ☐ CPT/Hydropunch
- ☐ Other: \_\_\_\_\_

NOTE: No permit is required for borings under 45 feet deep.

SITE PLAN  
(Please draw accurately)

see attached



## SIGNATURES

I understand and agree that all work associated with this permit is required to be done in accordance with Santa Clara Valley Water District (District) Well Ordinance 90-1, the District Well Standards, and conditions of this permit (see page 2). I certify that the information given in this permit is correct to the best of my knowledge and that the signature below, whether original, electronic, or photocopied, is authorized and valid, and is affixed with the intent to be enforceable. I also certify that a right of entry/encroachment agreement has been formalized between the well owner and property owner, if parties differ.

Signature of Property Owner/Agent: 	Print/Type Name: <b>Sean Deivert</b>	Date: <b>6/11/15</b>
Signature of Client/Agent:	Print/Type Name:	Date:
Signature of Driller/Agent: 	Print/Type Name: <b>Sean Deivert</b>	Date: <b>6/11/15</b>
Signature of Consultant/Agent: 	Print/Type Name: <b>Sean Deivert</b>	Date: <b>6/11/15</b>

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



# APPLICATION TO DRILL EXPLORATORY BORINGS

FC 285 (10-18-10)

Page 2 of 2

## GENERAL CONDITIONS

- A. District (telephone 408-265-2607, ext. 2660) must be notified a minimum of one working day before the exploratory boring is backfilled. An authorized District representative must be on site to witness the sealing operation. This requirement may be waived by an authorized District representative. If the District waives the inspection requirement, the District may request the permittee(s) to furnish certification under penalty of perjury that the seal was constructed in accordance with the District Well Standards.
- B. This permit is valid only for the purpose specified herein. Boring destruction methods authorized under this permit may not be changed except by written approval of an authorized District representative, and only if the District believes that such a change will result in equal or superior compliance with the District and State Well Standards (e.g., if the District representative finds that site conditions warrant such a change).
- C. This permit is only valid for the Assessor's Parcel No. indicated on it.
- D. This permit may be voided if it contains incorrect information.
- E. Borings shall be sealed within 24 hours following completion of testing or sampling activities. Borings shall not be left in such a condition as to allow for the introduction of surface waters or foreign materials into them. Borings shall be secured such that they do not endanger public health.
- F. If any work associated with this permit will take place on District property/easement, an encroachment or construction permit must be granted by the District's Community Projects Review Unit (telephone 408-265-2607, ext. 2350, 2217, or 2253).
- G. The permittee(s) shall assume entire responsibility for all activities and uses under this permit and shall indemnify, defend, and hold the District, its officers, agents, and employees, free and harmless from any and all expense, cost, and liability in connection with or resulting from the granting or exercise of this permit including, but not limited to, property damage, personal injury, and wrongful death.
- H. Permittees are required to be in full compliance with Cal/OSHA California Labor Code Section 6300.
- I. A current C-57 or C-61 Contractor's License is required for work associated with this permit.
- J. Permittee, permittee's contractors, consultants, or agents shall be responsible to assure that all materials or waters generated during drilling, boring destruction, and/or other activities associated with this permit will be safely handled, properly managed, and disposed of according to all applicable federal, state, and local 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 the work is being completed.
- K. The driller and consultants (if applicable) shall have an active copy of their Worker's Compensation Insurance on file with District.
- L. This permit shall expire if not exercised within 180 calendar days of its approval, unless an extension of the permit expiration date is granted by an authorized District representative.
- M. This permit shall be kept on site during all activities associated with it and shall immediately be presented to an authorized District representative upon request.

Permit Approved by:

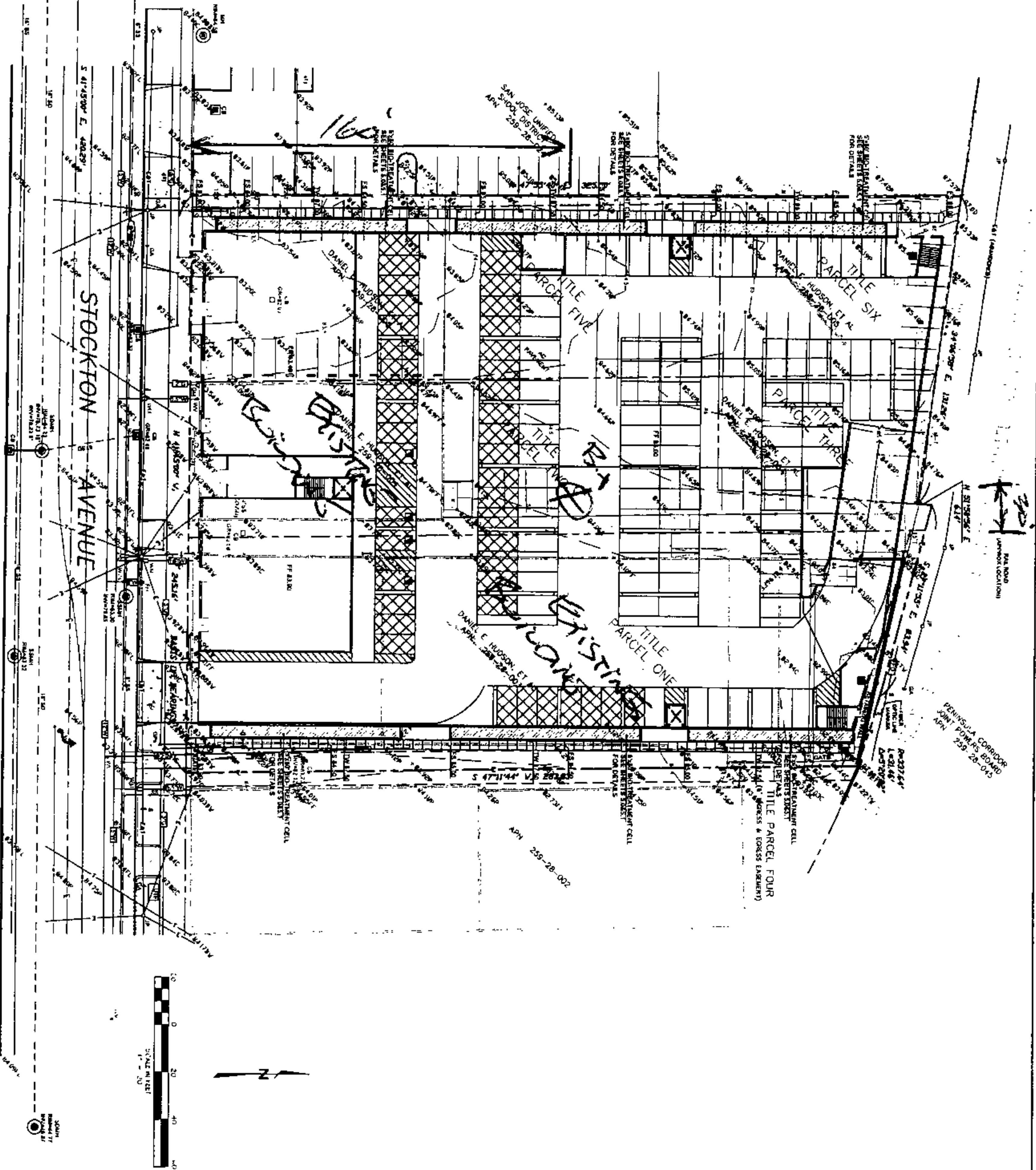
*Charles W. Rank*

Date:

*6-18-15*

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

NOTE: ~~PH~~ REVERTS SEE GRADING LOCATION



# PD ZONING 138 STOCKTON AVE GRADING AND DRAINAGE PLAN

SAN JOSE

CALIFORNIA



**VERO**  
consultants  
Professional Engineer License No. 50410  
Civil Engineering

NO.	DATE	DESCRIPTION
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PROJECT: 2012.356  
FILE: 2012.356.GPDWG  
DATE: FEB 3, 2015  
SCALE:  
DESIGNED BY: OGR  
DRAWN BY: VC  
REVIEWED BY: VER  
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