Appendix B

Woodward-Clyde Consultants

GEOTECHNICAL INVESTIGATION
GREAT AMERICA PARK OVERFLOW PARKING LOT
Santa Clara, California

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CITY OF SANTA CLARA
ENGINEERING DEPARTMENT

Prepared for

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Woodward-Clyde Consultants

August 23, 1990

Project 90C-0397-R

Ruth and Going, Inc.
Post Office Box 611600
San Jose, CA 95161-1600

Attention: Mr. Salvatore Duckworth-Lanzo

Gentlemen:

We are pleased to submit the accompanying report presenting the results of our geotechnical engineering study for the Great America Park Overflow Parking Lot in Santa Clara, California. This study has been conducted in accordance with our proposal for geotechnical services dated May 24, 1990, as authorized by Ruth and Going, Inc. Agreement for Subconsultant Professional Services dated August 3, 1990. Our geotechnical recommendations presented in the attached report are based on our engineering experience and judgment, as well as the results of the field investigation and laboratory testing. All field and laboratory data are also presented in this report.

It has been a pleasure working on this project. Please contact our office if you have any questions or comments regarding this project, or if you need additional information to assist you.

Sincerely,

Scott M. Leck, G.E.
Project Engineer

Michael L. Larson, G.E.
Senior Project Engineer

SML/MLL/rm

Attachment

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GEOTECHNICAL INVESTIGATION
GREAT AMERICA PARK OVERFLOW PARKING LOT
Santa Clara, California

INTRODUCTION

This report presents geotechnical design recommendations at the site of the proposed Great America Park Overflow Parking Lot located in Santa Clara, California. The location and general layout of the property are shown on the attached Site and Boring Location Plan, Figure 1.

In addition to three borings taken in 1986 for the proposed bridge over San Tomas Aquino Creek, the soil conditions at the site were investigated in 1990 with 17 borings and 7 test pits. Detailed descriptions of the 1990 field investigation and laboratory testing are presented in Appendix A. This report includes the results of the 1986 and 1990 field and laboratory investigations, together with conclusions and recommendations for the design and construction of asphalt concrete pavements. In addition, this report includes conclusions about the stability of the existing levees of San Tomas Aquino Creek if additional fill is placed on them at the new bridge location.

PREVIOUS INVESTIGATION

Applied Soil Mechanics, Inc. previously performed a geotechnical study of the proposed bridge over San Tomas Aquino Creek, located about 1,000 feet south of Tasman Drive. The results of that study, including the logs of three borings, are contained in a report entitled, "Great America Bridge at San Tomas Aquino Creek, Santa Clara, California, Soil Investigation," dated November 7, 1986 (File No. A6-0125-J49).

PROJECT DESCRIPTION

As shown in Figure 1, the site is located on both the east and west sides of San Tomas Aquino Creek, south of Tasman Drive and west of Centennial Boulevard. It is understood that a new 12-acre parking lot will be constructed on the east side of the creek. The project also includes a new bridge to connect the existing parking lot located on the west side of the creek with the proposed lot. Construction of the bridge will require placement of additional fill on both sides of the creek and on the existing levees to create approaches to the bridge. Foundation recommendations for the bridge are beyond the scope of this study; they were addressed in the previously referenced report prepared by Applied Soil Mechanics, Inc.

It is understood the parking lot will be paved with an asphalt concrete pavement, subject only to automobile traffic. No moderate or heavy trucks or buses are anticipated. It is also understood that grading to prepare the area for paving will be relatively minor, with maximum cuts of about 1 foot and fills on the order of 2 to 3 feet with a maximum 5 foot thick fill section required at the northwest corner of the site. Approach fills on both the west and east sides of the creek will be placed as a part of the site grading operations. This will result in raising the creek levees by about 4 feet. Maximum approach fill height will be about 11 feet. On the west side of the creek, the approach fill side slopes will either be flat enough to allow parking up to the edge of the roadway or be at a slope of 2 to 1 (horizontal to vertical). On the east side of the creek, the approach fill side slopes will be flat enough to allow parking up to the edge of the roadway. The stockpiles of soil at the site will be used in grading for the project.

SITE AND SOIL CONDITIONS

Site Conditions

West of the creek, the site is used as a parking lot for Great America Park, and is paved with asphalt concrete. A visual observation of the pavement surface indicated

that it is in relatively good condition, with no observed rutting, heaving or other undulations present. It is understood this pavement was constructed prior to 1975. There are no available structural pavement section drawings of this existing pavement. Borings 1 and 2, recently drilled through this pavement, exposed sections consisting of 2 inches of asphaltic concrete over 5 to 6 inches of aggregate base over 4 to 10 inches of aggregate subbase.

San Tomas Aquino Creek passes through the site, and is flanked by flood control levees. According to the topographic information provided by Ruth and Going, Inc. the top of the levees are approximately 15 feet above the bottom of the channel and 7 feet above the surrounding ground surface. The side slopes appear to be between 2 to 1 and 3 to 1 (horizontal to vertical). Water was present in the creek at the time the present field exploration was conducted.

The portion of the site located east of the creek is a trapezoidal shaped parcel bounded by San Tomas Aquino Creek on the west, Tasman Drive on the north, Centennial Boulevard on the east and an electrical substation on the south. This area is a grass covered field, parts of which have been disced. According to the topographic plan provided by Ruth and Going, Inc., the existing grade in this area ranges from about Elevation 12 at the toe of the levee to Elevation 9 along Centennial Boulevard. An approximately 400 foot by 400 foot parcel located at the northeast corner of the site is not included in this project.

Several stockpiles of soil are present at the site. The largest is located at the northwest corner of the site, with approximate plan dimensions of 280 feet by 400 feet. Based on the topographic information and visual reconnaissance in the field, it appears that the stockpile is approximately 6 to 8 feet high. The second stockpile is located on the adjacent 400 foot by 400 foot parcel at the northeast corner of the site. This pile appears to be approximately 70 feet by 180 feet in plan dimensions,

and about 12 feet high. The third stockpile, located at the southeast corner of the site, has plan dimensions of approximately 60 feet by 400 feet and is about 6 feet high. All of these stockpiles were investigated either by exploratory borings and/or pits. A description of the soils contained in these stockpiles is presented in the following section.

Elevations given in this report for ground surface were interpolated from topographic contours shown on the previously described topographic survey of the site.

Soil Conditions

Subsurface information regarding the soil conditions at this site was obtained from exploratory borings and pits excavated at the site in 1990. Detailed descriptions of the soil and groundwater conditions are given on the Logs of Borings, Figures 3 through 19, and the Logs of Exploratory Pits, Figures 20 through 26, respectively. Subsurface information obtained from the 1986 study are given on the Logs of Borings, Figures 32 through 34. These borings are noted as Borings 86-1, 86-2 and 86-3.

At the west side of the creek, Borings 1 and 2 were drilled at the proposed location of the approach ramp to the bridge. These borings encountered a pavement section consisting of 2 inches of asphalt concrete over 5 to 6 inches of aggregate base over 4 to 10 inches of aggregate subbase. Below the pavement section, the borings encountered 3 to 3-1/2 feet of highly plastic stiff silty clay overlying 2 to 4-1/2 feet of low to moderately plastic silty clay. The lower clay layer is stiff to hard in consistency. The borings were terminated in the lower plasticity silty clay at depths of 6-1/2 to 9-1/2 feet.

At the east side of the site, the borings and exploratory pits encountered 1-1/2 to 5 feet of highly plastic silty clay at the ground surface. R-values for surface clays and fills ranged from less than 5 to 11. This stratum generally graded into a low plasticity silty and sandy clay throughout the site in which the borings were terminated at depths of between 6-1/2 feet and 10 feet. At the location of Boring 14, the low plasticity silty clay was underlain by 2-1/2 feet of gravelly, clayey sand and a stratum of sand with a little gravel, which extended to the terminal depth of 10 feet.

At Boring 3, drilled at the location of the east side bridge approach fill, the boring encountered 5 feet of hard silty clay of high plasticity overlying 2 feet of stiff silty clay of lower plasticity. Below the silty clay, the boring encountered 5 feet of medium dense clayey sand. Below a depth of 12 feet, the boring was advanced through a stratum of silty clay with a little fine sand. This stratum appears to have low to moderate plasticity and moderate compressibility. The boring was terminated at a depth of 30 feet in this stratum.

Borings 16 and 17 and Exploratory Pits 1 and 2 were advanced into the fill stockpile located at the northwest corner of the site. The fill comprising the stockpile consists of a combination of highly plastic silty clay, similar to that occurring naturally on the site, with some silty clays of lower plasticity. However, a 1-1/2 foot thick layer of silty sand fill was found at the location of Boring 16 at a depth of 1 foot.

The stockpile located at the northeast corner of the site was investigated by Exploratory Pit 3. This excavation revealed approximately 10 feet of sandy to silty clay fill of moderate to high plasticity. This soil contained roots and grass throughout the excavated section, indicating that it is likely topsoil stripped from another site.

The stockpile located at the southeast corner of the site was investigated by Exploratory Pit 5. A visual examination of the soils present at this location revealed approximately 5-1/2 feet of moderately to highly plastic silty clay which contains roots and grasses in the lower 3 feet. This material also appears to be topsoil removed from another site.

Groundwater was encountered in only two borings, Borings 3 and 14 at depths of 10 feet and 8-1/2 feet, respectively. These levels approximate the water level in the creek at the time the subsurface investigation was conducted. Cohesive soils, which dominate the project site, require a long time for the groundwater to seep into the borehole and attain an equilibrium position with the present hydrostatic groundwater table. Thus, the immediate readings obtained during drilling may or may not be representative of the actual groundwater level. It is anticipated that water may travel through the thin sandy layers in the soil and that the water level at the site may fluctuate seasonally.

DISCUSSION

Existing Fill Material

The existing fill material present on the site in the three stockpiles is moderately to highly plastic silty clay, similar to the upper stratum present naturally at the site. This material is generally difficult to compact and provides a very low strength subgrade on which to construct pavements. It is understood that it is desired to use this material for site grading. The stockpile located at the northwest corner of the site can be used as fill under the proposed pavement section. Because of the very limited amount of low plasticity material in this stockpile, it would be impractical to separate it for use as higher quality pavement subgrade. The fill material located in stockpiles at the northeast and southeast corners of the site contains organic materials and is inadvisable for use as fill under pavements. Specific recommendations for site grading are presented in following sections of this report.

Pavements

In areas to be paved with asphalt concrete, there is a moderate to high potential for volume change of the subgrade soils if they are subject to moisture content fluctuations. As discussed previously, the existing pavement section (west of the creek) appears to be in relatively good structural condition; this observation infers the pavement has not undergone cyclic differential vertical movements (heaving) associated with moisture content changes. It appears the moisture content of the subgrade soils below the pavement section has remained relatively constant (with time) for the following reason(s):

- Periodic sealing of asphalt concrete pavement surface cracks with a slurry seal, minimizing rainfall infiltration
- Surface drainage features such as adequate pavement slopes and catch basins to minimize ponding of surface water
- Four year drought period in the Santa Clara Valley
- Moderately thick pavement structural sections

With regard to the proposed pavement section, several alternatives including lime treatment of the subgrade soils have been considered. Lime treatment is usually effective in reducing the plasticity of the soil, which in turn normally reduces the expansion potential of the soil.

Over the past 20 years or so, municipalities in the Santa Clara Valley and adjacent areas have constructed some pavement sections consisting of asphalt concrete placed directly on lime treated subgrade soils, with varying degrees of success. (No aggregate base was included.) This practice is being phased out in this area. The experience gained from using this pavement section indicates that some pavements constructed in this way have experienced long term distress. The reasons for this distress and damage are as follows:

- This pavement section has no provision for drainage of water that seeps through the asphalt and saturates and softens the subgrade soils
- It appears that there may be some long term reversibility of the chemical reaction which occurs between the soil and the lime; some soil stabilized by lime undergoes short term increase of strength followed by long term degradation of the strength

For these reasons, a pavement section consisting of asphalt concrete placed directly on lime treated soil is not recommended.

The two pavement sections recommended in the following section of this report consist of the following:

- Asphalt concrete over aggregate base
- Asphalt concrete over a reduced aggregate base section placed on a section of lime-treated soil

The selection of the final pavement section should be based on economic comparisons. If the costs are comparable, the lime treated section is recommended in order to take advantage of the incidental reduced swelling potential of the native soils.

Levee Stability

Stability of the existing levee with an addition of 4 feet of fill placed on it with a maximum side slope of 2 to 1 (horizontal to vertical) was estimated using a computer aided slope stability analysis program. Subsurface information was used from both this investigation and the 1986 report. Based on results of this analysis, as well as engineering judgment and local experience, it is concluded that the enlarged levee should remain stable under both static and dynamic (earthquake) loading conditions.

RECOMMENDATIONS

Pavement Design - Asphalt Concrete

Near-surface soils in the area of the proposed asphalt concrete parking lot have low support capacity when used as pavement subgrade. An R-value of 5 was selected for pavement design based on the results of the laboratory R-value tests. R-value test results were less than 5, 8, 10 and 11 on bulk samples of near surface soil taken from selected locations in Borings 6, 2, 9 and 17, respectively. The test results are shown in Figures 28 through 31.

Based upon these subgrade soil conditions and our engineering judgment and experience, the following pavement structural sections are recommended:

Alternative 1

Asphalt Concrete (Type B):	3 inches
Class 2 Aggregate Base:	8 inches
Compacted Subgrade: (92 percent minimum relative compaction)	6 inches

Alternative 2

Asphalt Concrete (Type B):	3 inches
Class 2 Aggregate Base:	4 inches
Lime treated Compacted Subgrade: (95 percent minimum relative compaction)	7 inches

It is recommended that the pavement materials and construction conform to the applicable sections of the Caltrans Standard Specifications (latest edition), as follows:

Pavement Material	Type of Material	Specification Section
Asphalt Concrete	Class B, 1/2 inch maximum, medium gradation	39
Aggregate Base	Class 2, 1-1/2 or 3/4 inch maximum size	26

If the lime treatment alternative is selected, the soils should be thoroughly pulverized to a depth of 7 inches. The pulverized soil should be thoroughly mixed with at least 5 percent (by dry weight) lime and compacted. It is recommended that the lime treatment construction for this alternative also conform to the Caltrans Standard Specifications (latest edition) Section 24, Lime Treatment, including compaction to at least 95 percent relative compaction. Scarification of the subgrade beneath the lime treated section for this alternative is not required. If the lime treatment alternative is selected, it is recommended that a laboratory testing program be performed in accordance with California Test 373 to verify lime content prior to construction.

Settlement - Bridge Approach Fills

Construction of the bridge approach fills will cause relatively small settlement due to the compression of the foundation soils. For an 11 foot high embankment, the maximum long term consolidation settlement is estimated to be of the order of 1 to 2 inches. Most of this settlement is estimated to occur over a period of about 5 years.

Site Preparation and Grading - Pavement Areas

It is recommended that all site preparation and earthwork construction be done under the observation of the Geotechnical Engineer and in accordance with the applicable sections of Caltrans Standard Specifications.

Stripping - In general, all areas to be excavated or to receive fill should be stripped of all surface vegetation, organic materials, existing fills and any deleterious substances that may be present. It is estimated that stripping will be on the order of 3 to 6 inches below the existing site grade. Final stripping depths should be determined by the Geotechnical Engineer in the field during site preparation.

Subgrade Preparation - After the site surface in the proposed pavement areas has been properly cleared, the exposed native clay soils (if lime treatment is not used) should be scarified to a depth of at least 6 inches, moisture conditioned to about 3 percent above optimum, and compacted to at least 92 percent relative compaction.

The exploratory pits excavated for this investigation were not backfilled with engineered fill. For that reason, all exploratory pits should be completely re-excavated during construction and recompacted to the requirements of engineered fill.

Fill Material - It is expected that the majority of the existing on-site material can be reused as engineered fill, provided it is free from debris or organic materials, and meets the fill requirements. Any import material should be a low plasticity material with a plasticity index of 15 or less. All fill materials should be approved by the Geotechnical Engineer prior to the placement of materials as fill.

The stockpile located at the northeast corner of the site and the stockpile located at the southeast corner contain fill that should only be used for landscaping purposes.

Fill Placement and Compaction - All engineered fill should be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to about 1 to 3 percent above optimum and compacted to a minimum relative compaction of 90 percent. Pavement subgrade and aggregate base placed as part of the pavement

structural section should be compacted to a minimum relative compaction of 92 and 95 percent, respectively. Final subgrade compaction should be accomplished immediately prior to placement of aggregate base. The maximum dry density, as determined in the laboratory, should be performed in accordance with Caltrans Method 216-F. The fill should be brought to a uniform moisture content by aerating the material if it is too wet or spraying and mixing the material with water if it is too dry. Fill slopes should be no steeper than 2 to 1 (horizontal to vertical).

Drainage - The aggregate base to be placed under the pavement will be somewhat permeable, allowing rainfall infiltration through the asphalt concrete or surface pavement cracks and collection of water within the aggregate base in low areas. For this reason, it is recommended that the sides of drop inlets placed in low areas be perforated with holes to relieve water pressure. The holes should be placed at a level near the bottom of the aggregate base layer, with a diameter of about 3/4 inch and spaced at about 6 inches. A geotextile filter fabric should be placed over the perforations to prevent migration of material from the aggregate base. At the perimeter areas where the pavement slopes downward toward the nonpaved areas, the aggregate base should extend at least 2 feet beyond the pavement edge and daylight at the gravel surface. The purpose is to allow water to drain from the aggregate base. At the perimeter areas where the pavement slopes downward into the paved areas, the aggregate base should not extend beyond the asphalt surface. This will minimize the potential for inflow of surface water into the aggregate base section. Alternatively, an underdrain system could be provided near the pavement edge in low areas. It is also recommended that good surface drainage be provided, including a surface slope in excess of 1 percent. It is recommended that the Geotechnical Engineer review grading and drainage drawings, when available, to determine the need, if any, for underdrains.

LIMITATIONS

The recommendations made in this report are based on the assumption that subsurface conditions at the site do not deviate appreciably from those encountered in the borings and test pits. They are also made for the specific site development described in the report. If changes are anticipated or if any variations or undesirable conditions are encountered during construction, the Geotechnical Engineer should be consulted for further recommendations.

It is recommended that the Geotechnical Engineer review the foundation and grading plans and specifications to ensure that the intent of the recommendations presented herein has been properly interpreted and incorporated into the contract documents. A representative of the Geotechnical Engineer should also observe the foundation excavations and the subgrade preparation and fill placement to verify that the subsurface conditions used as a basis for design are encountered throughout the site.

The recommendations presented in this report were developed with the standard of care commonly used as state of the practice in the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this report.

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING
GREAT AMERICA PARK OVERFLOW PARKING AREA
Santa Clara, California

FIELD EXPLORATION

Seventeen (17) exploratory borings were drilled and seven (7) exploratory test pits were excavated in the project area, to explore the subsurface conditions. The exploratory borings were advanced to depths ranging from 6-1/2 to 30 feet. The drilling equipment used was a 8-inch diameter hollow stem auger. The exploratory pits were excavated with a tractor-mounted backhoe. The drilling was done on July 20, 1990; while the exploratory pits were excavated on August 7, 1990 under the supervision of a representative of Woodward-Clyde Consultants. Visual classifications of the soils encountered were made from the cuttings at the time of drilling. Samples of the underlying soils taken from the borings were obtained using a modified California drive sampler (2 inch inside diameter and 2-1/2 inch outside diameter).

The modified California sampler was driven into the soil with a 140 pound hammer falling freely through 30 inches.

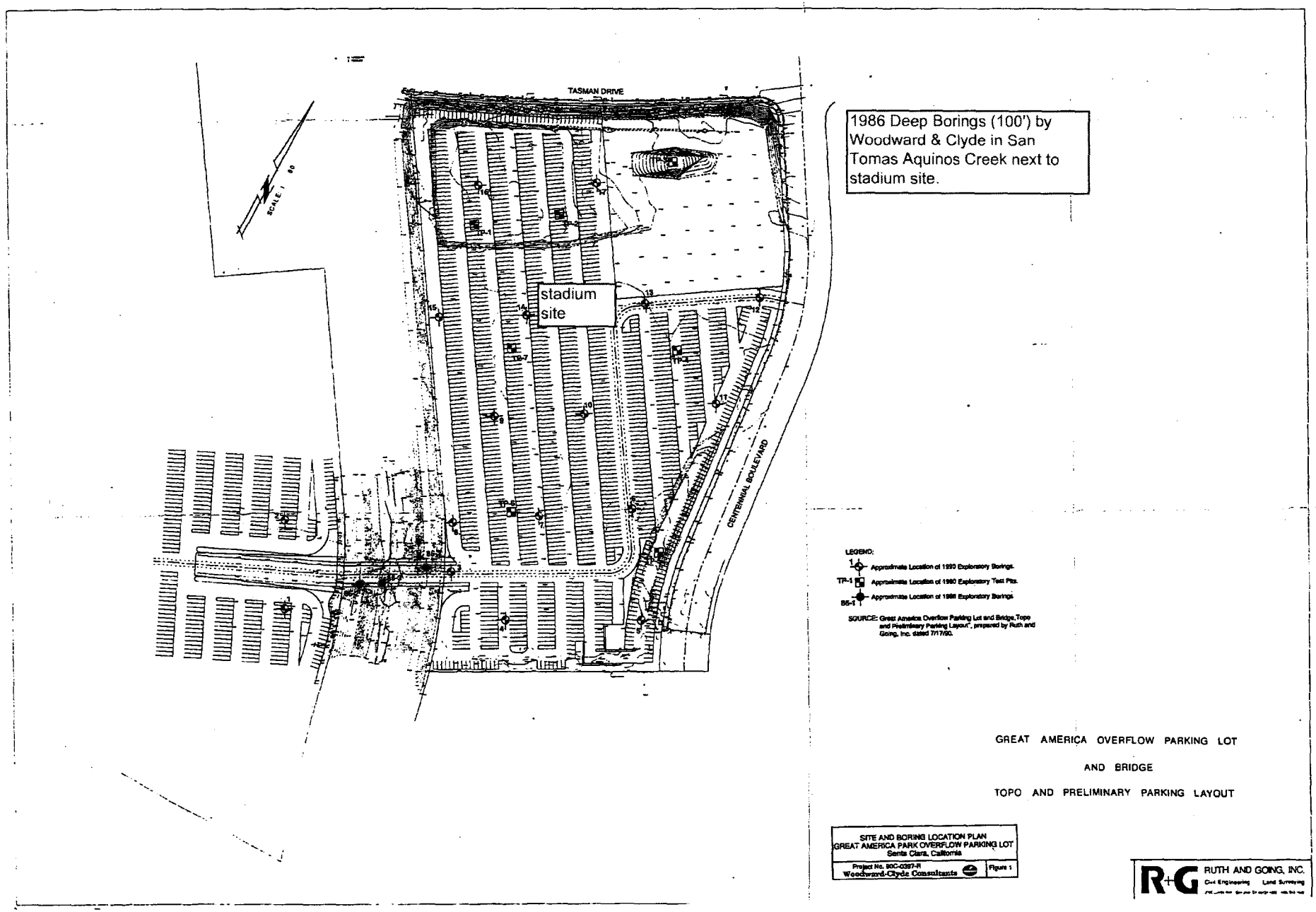
When the sampler was withdrawn from the hole, the samples were carefully removed, sealed to prevent moisture loss and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further examination and testing. The attached Boring Log Legend Sheet, Figure 2, illustrates the notation used for the types of samplers and methods of advancing them are presented on the Logs of Borings, Figures 3 through 19.

LABORATORY TESTING

The water content, dry density and unconfined compressive strength were determined for selected samples to estimate the strength and compressibility of the underlying soils. The results of these tests, together with the resistance to penetration of the sampler, are shown at the corresponding sample locations on the Logs of Borings.

Atterberg Limits (liquid and plastic limits) were determined for fine grained soil samples. The results of these tests are presented on Figure 27.

Four (4) R-value tests were made on selected surface samples to assist in pavement structural design. The results of these tests are presented graphically on Figures 28 through 31.


TASMAN DRIVE
CENTENNIAL BOULEVARD
SCALE: 1" = 80'
stadium site
1986 Deep Borings (100') by Woodward & Clyde in San Tomas Aquinos Creek next to stadium site.
LEGEND:
Approximate Location of 1990 Exploratory Borings.
TP-1 Approximate Location of 1990 Exploratory Test Pits.
B5-1 Approximate Location of 1986 Exploratory Borings.
SOURCE: Great America Overflow Parking Lot and Bridge,Topo and Preliminary Parking Layout", prepared by Ruth and Going, Inc. dated 7/17/90.
GREAT AMERICA OVERFLOW PARKING LOT
AND BRIDGE
TOPO AND PRELIMINARY PARKING LAYOUT
SITE AND BORING LOCATION PLAN
GREAT AMERICA PARK OVERFLOW PARKING LOT
Santa Clara, California
Project No. 90C-0397-R
Woodward-Clyde Consultants
Figure 1
R+G
RUTH AND GOING, INC.
Civil Engineering
Land Surveying

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Boring Log Legend Sheet

Date Drilled:
Type of Boring:
Hammer:

Remarks:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
5			2-inch I. D. Modified California Sampler			
10		29	Blow count with 140-pound hammer falling 30 inches per blow			
15						
20			Liquid Limit → LL = 86 Plasticity Index → PI = 56			
25			Water Level Measured: ▽ At Time of Drilling → ATD			
30						

Project:90C-0397-R | **Woodward-Clyde Consultants** | Figure 2

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 1

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			2" Asphalt Concrete, over 5" Aggregate Base over 4" Subbase — FILL			
	1	21	SILTY CLAY (CH) Stiff, moist, gray-brown and light brown mottled — LL = 82, PI = 63	31	91	2130
5	2	35	SILTY CLAY (CL) Hard, damp, light brown gray mottling with sand and gravel	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: 90C-0397-R | **Woodward-Clyde Consultants** | Figure 3

Project: **GREAT AMERICA PARKING LOT**
Santa Clara, California

Log of Boring No. 2

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			Pavement Section — FILL 2" Asphalt Concrete, over 6" Aggregate Base over 10" Select Subbase			
	1	24	SILTY CLAY (CH) Stiff, moist, dark gray-brown	26	97	3540
5	2	36	SILTY CLAY (CL) Very stiff, moist, brown and gray mottled trace fine sand and gravel	–	–	–
	3	16	Stiff	–	–	–
10			Bottom of boring @ 9-1/2 feet Boring dry upon completion			
15						
20						
25						
30						

Project: 90C-0397-R | **Woodward-Clyde Consultants** | Figure 4

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 3

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	26	SILTY CLAY (CH) Hard, damp, dark gray-brown to black	–	–	–
	2	31	Dark gray brown mottled LL = 56 PI = 38	17	105	11890
5			SILTY CLAY (CL) Very stiff, moist, light brown with white mottling			
	3	17	CLAYEY SAND (SC) Medium dense, very moist, light brown, little fine gravel ATD	18	111	1420
10						
	4	20	SILTY CLAY (CL) Soft, very moist, gray and brown mottled	24	101	970
15						
	5	11		23	103	1240
20						
	6	43	Gray and brown mottled Silty Clay (CL) moist, stiff, sandy	–	–	–
25						
	7	56	Bottom of boring @ 30 feet	–	–	–
30						

Project: 90C-0397-R | **Woodward-Clyde Consultants** | Figure 5

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 4

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
	1	37	SILTY CLAY (CH) Hard, damp, dark brown-black, porous, fine root hairs	–	–	–
	2	45	SILTY CLAY (CL) Very stiff, moist, light and dark brown mottled, sandy	–	–	–
5	3	65	Gray mottled	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 5

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	32	SILTY CLAY (CH) Hard, damp, gray-black, porous, many fine roots and root hairs	13	91	–
	2	26	Gray mottled LL = 57 PI = 39	–	–	–
5	3	45	SILTY CLAY (CL) Hard, moist, brown and gray mottled, some fine sand	–	–	–
	4	18	Medium, very moist	–	–	–
10			Bottom of boring @ 9-1/2 feet Boring dry upon completion			
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 6

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	34	SILTY CLAY (CH) Hard, damp, dark brown to black, porous, many fine roots and root hairs	15	82	–
	2	32	SILTY CLAY (CL) Stiff, damp, moist, brown and tan mottled, little sand	18	106	4100
5	3	43		–	–	–
	4	12	SANDY CLAY (CL) Stiff, very moist, brown	–	–	–
10			Bottom of boring @ 9-1/2 feet Boring dry upon completion			
15						
20						
25						
30						

Project: 90C-0397-R | Woodward-Clyde Consultants | Figure 8

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 7

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
	1	41	SILTY CLAY (CH) Very stiff, damp, dark brown-black porous, many root hairs	14	91	5840
	2	23	SILTY CLAY (CL) Stiff, moist, brown and gray mottled	26	97	3590
5	3	26	Little sand	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 8

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	32	SILTY CLAY (CH) Very stiff, damp, dark brown-black, porous, many fine roots, and root hairs	17	93	5940
	2	24	SILTY CLAY (CL) Very stiff, moist, brown-tan and gray mottled	25	94	4060
5	3	40	Hard	–	–	–
	4	10	Soft, very moist, gray and rust mottled	–	–	–
10			Bottom of boring @ 9-1/2 feet Boring dry upon completion			
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 9

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	38	SILTY CLAY (CH) Very stiff, damp, dark gray-brown to black LL = 65 PI = 45	14	88	4340
	2	34	SILTY CLAY (CL) Very stiff,moist, light brown and little fine sand and tan mottled	–	–	–
5	3	56	Hard, sandy	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 10

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	33	SILTY CLAY (CH) Very stiff, damp, dark brown-black many fine roots and root hairs	15	88	5380
	2	28	SILTY CLAY (CL) Very stiff, moist, brown and tan mottled	–	–	–
5	3	36		–	–	–
	4	12	SANDY CLAY (CL) Stiff, very moist, light brown dark brown mottled	–	–	–
10			Bottom of boring @ 9-1/2 feet Boring dry upon completion			
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 11

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
	1	34	SILTY CLAY (CH) Hard, damp, dark brown, porous, many fine roots and grass, roots hairs	10	85	–
	2	48	SILTY CLAY (CL) Very stiff, moist, brown and tan mottled	17	108	6610
5	3	23		–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 12

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	47	SILTY CLAY (CH) Hard, damp, dark gray-brown to black, trace fine roots	16	90	10370
	2	33	SILTY CLAY (CL) LL = 34 PI = 15 Very stiff, moist, brown and gray mottled, trace fine sand	–	–	–
5	3	37	Light brown and gray mottled, some fine sand	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 13

Date Drilled: 7-20-90

Remarks: -

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
	1	28	SILTY CLAY (CH) Hard, damp, black, trace fine roots LL = 66 PI = 44	–	–	–
	2	23	SILTY CLAY (CL-CH) Stiff, moist, light brown and dark brown mottled	27	94	2960
5	3	53	SILTY CLAY (CL) Very stiff, moist, light brown, trace fine sand	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: **GREAT AMERICA PARKING LOT**
Santa Clara, California

Log of Boring No. 14

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Depth Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
	1	31	SILTY CLAY (CH) Very stiff, damp, black	15	88	4640
	2	38	SILTY CLAY (CL-CH) Moist, very stiff, light brown and dark brown mottled	21	101	5810
5	3	66	GRAVELLY CLAYEY SAND (SC) Very dense, moist, brown	–	–	–
	4	46	SAND (SP) Dense, wet, brown, some fine gravel	–	–	–
10			Bottom of boring @ 10 feet			
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 15

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	30	SILTY CLAY (CH) Medium, damp, dark gray to black, trace fine roots	17	80	1600
	2	29	SILTY CLAY (CL-CH) Hard, gray-brown mottled, trace of fine gravel	21	97	8640
5	3	59	Light brown, white mottling	–	–	–
			Bottom of boring @ 6-1/2 feet Boring dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 16

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	24	SANDY CLAY FILL (CL-SC) Moderate compacted, moist, trace gravel Silty Sand from 1' to 2-1/2'	–	–	–
5	2	30	SILTY CLAY FILL (CH) Poorly compacted, moist, dark brown LL = 50 PI = 33 FILL	17	102	7440
	3	38	SILTY CLAY (CH) Very stiff, moist, dark brown with white mottling	–	–	–
10	4	60	SILTY CLAY (CL) Very stiff, moist, brown, and light brown mottled	–	–	–
	5	20	Stiff	–	–	–
15			Bottom of boring @ 15 feet Boring dry upon completion			
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Boring No. 17

Date Drilled: 7-20-90

Remarks:

Type of Boring: 8" Hollow Stem Auger

Hammer: 140 lb

Surface Elevation:

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
	1	20	SILTY CLAY FILL (CL) Poorly compacted, moist, brown and gray mottled	–	–	–
5	2	38	Brown FILL	15	99	5030
	3	49	SILTY CLAY (CH) Very stiff, moist, dark gray-brown	23	99	7840
10	4	12	SILTY CLAY TO CLAYEY SILT (CL-ML) Soft, very moist, light brown, some fine sand	–	–	–
15			Bottom of boring @ 13 feet Boring dry upon completion			
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Exploratory Pit No. 1

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			CLAYEY SAND FILL (SC) Moist, light brown, little fine gravel			
			SILTY CLAY FILL (CH) Moist, dark brown — FILL			
5			SILTY CLAY (CH) Moist, dark brown, some fine root hairs			
			Bottom of exploratory excavation at 7 feet Excavation dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Exploratory Pit No. 2

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			CLAYEY SAND FILL (SC) Moist, light brown, some gravel, some debris			
			SILTY CLAY FILL (CH) Moist, dark brown			
5			SILTY CLAY (CH) Moist, dark brown-black, root hairs and grass (FILL)			
			Bottom of exploratory excavation at 8 feet Excavation dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Exploratory Pit No. 3

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			SANDY CLAY FILL (CL-CH) Moist, dark brown,			
5			SILTY CLAY FILL (CL-CH) Moist, dark brown, some grass and roots, organics			
			(FILL)			
10			SILTY CLAY (CH) Moist, dark brown, root hairs and grass			
			Bottom of exploratory excavation at 12 feet Excavation dry upon completion			
15						
20						
25						
30						

Project: 90C-0397-R | Woodward-Clyde Consultants | Figure 22

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Exploratory Pit No. 4

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			SILTY CLAY (CH) Damp, dark gray-brown Moist @ 1-1/2 feet			
			SILTY CLAY (CL) Light brown and, gray mottled			
5			Bottom of exploratory excavation at 5 feet Excavation dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Exploratory Pit No. 5

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			SILTY CLAY FILL (CH) Damp, black-dark brown Roots and grass below 1-1/2 feet (FILL)			
5			SILTY CLAY (CH) Damp, black-dark brown, becoming moist with depth			
			Bottom of exploratory excavation at 7 feet Excavation dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

Log of Exploratory Pit No. 6

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			SILTY CLAY (CH) Damp, dark brown-black			
			SILTY CLAY (CL) Moist, light brown and gray mottled			
5			Bottom of exploratory excavation at 5 feet Excavation dry upon completion			
10						
15						
20						
25						
30						

Project: GREAT AMERICA PARKING LOT
Santa Clara, California

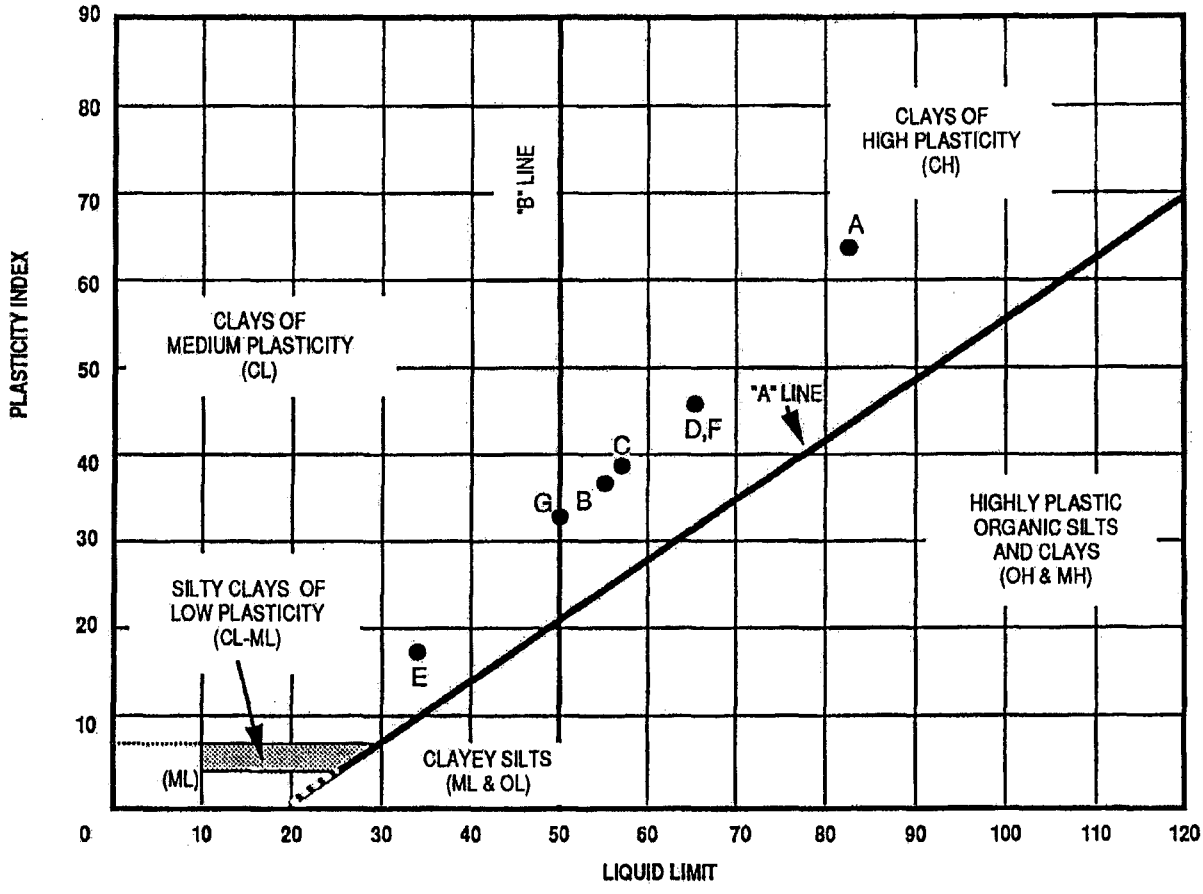
Log of Exploratory Pit No. 7

Date Excavated : 8-7-90

Remarks :

Type of Excavation : Backhoe

Depth Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compress. Strength, psf
			Surface Elevation:			
			SILTY CLAY (CH) Damp, dark brown-black, loose and blocky Becomes moister with depth			
5			SILTY CLAY (CL) Moist, light brown and gray mottled			
			Bottom of exploratory excavation at 6 feet Excavation dry upon completion			
10						
15						
20						
25						
30						



SAMPLE IDENTIFICATION				ATTERBERG LIMITS		
LETTER DESIG'N	BORING NO.	SAMPLE NO.	DEPTH, FT.	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
A	1	1	3-1/2	82	19	63
B	3	2	4	56	18	38
C	5	2	3	57	18	39
D	9	1	1	65	20	45
E	12	2	3	34	19	15
F	13	1	1	66	22	44
G	16	2	4-1/2	50	17	33

Project No. 90C-0397-R **Woodward-Clyde Consultants**	PLASTICITY CLASSIFICATION GREAT AMERICA PARKING LOT Santa Clara, California	Figure 27

TESTING ENGINEERS, INCORPORATED

401 ALDO AVENUE • SANTA CLARA, CALIFORNIA 95050 • 408-988-8888

SANTA CLARA
MONTEREY/SALINAS
OAKLAND
STOCKTON
SACRAMENTO
DIABLO VALLEY

LABORATORY NUMBER L0457-1 8-9-90

TEI PROJECT NO. 02636

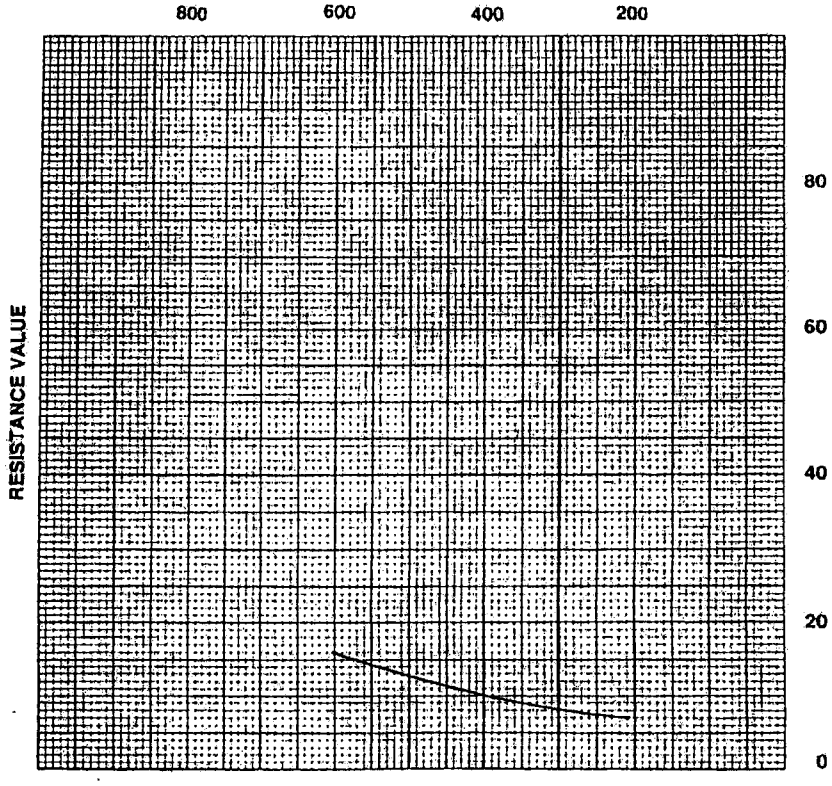
Job Data: Woodward Clyde Consultants
Great America Parking Lot
Job 90C-0397-R

Sample Data:
Very dark grayish brown clay
Boring 2, Depth 18"

Sample Received: 7-26-90

Resistance Value Test

Exudation Pressure, PSI



Compactor Pressure psi	75	50	0
Moisture @ Compaction %	21.2	23.7	26.3
Dry Density @ Compaction pcf	104.9	100.3	97.2
Exudation Pressure, psi	600	390	210
Resistance Value "R"	16	10	7
Expansion, Dial Reading .00	89	27	12
Expansion, psf	385	117	52

GRADING ANALYSIS

Percent Passing

Sieve Size	As Rcvd.	As Tested	As Rcvd. Specified Limits
3"			
2½			
2			
1½			
1			
3/4			
1/2			
3/8			
#4			
8			
16			
30			
50			
100			
200(wash)			

Sand Equivalent Test (Aver. of 3)

S.E. SPECIFIED (MINIMUM) _____

Design R-Value

R-Value (By Exudation Pressure) 8

Specified Minimum _____

Durability Index

Coarse Durability _____

Fine Durability _____

Durability Index _____

Specified Minimum _____

3cc: Woodward Clyde Consulting
Attention: Mr. Scott M. Leck

TESTING ENGINEERS, INCORPORATED

Reviewed by Walter C. Leonard
Walter C. Leonard
Laboratory Supervisor

Figure 28

Form T-30

TESTING ENGINEERS, INCORPORATED

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SANTA CLARA
MONTEREY/SALINAS
OAKLAND
STOCKTON
SACRAMENTO
DIABLO VALLEY

LABORATORY NUMBER L0457-4 8-9-90

TEI PROJECT NO. 02636

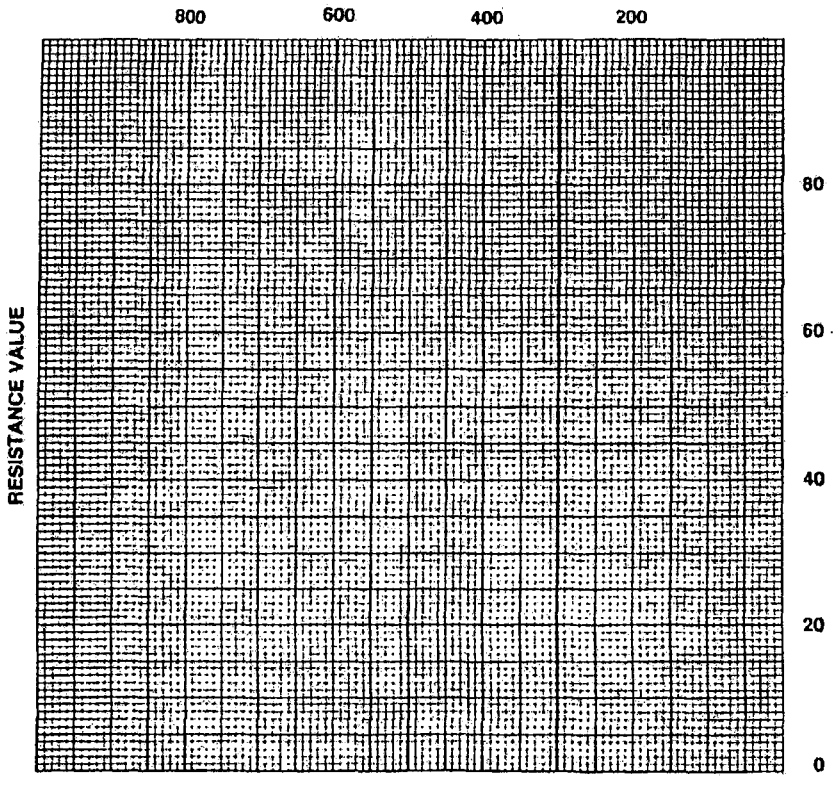
Job Data:
Woodward Clyde Consultants
Great America Parking Lot
Job 90C-0397-R

Sample Data:
Dark brown clay with
organic material
Boring 6
Depth 12"

Sample Received: 7-26-90

Resistance Value Test

Exudation Pressure, PSI



Compactor Pressure psi	0
Moisture @ Compaction %	24.5
Dry Density @ Compaction pcf	98.0
Exudation Pressure, psi	----
Resistance Value "R"	<5
Expansion, Dial Reading .00	16
Expansion, psf	69

GRADING ANALYSIS

Percent Passing

Sieve Size	As Rcvd.	As Tested	As Rcvd. Specified Limits
3"			
2½			
2			
1½			
1			
3/4			
1/2			
3/8			
#4			
8			
16			
30			
50			
100			
200(wash)			

Sand Equivalent Test (Aver. of 3)

S.E. SPECIFIED (MINIMUM) _____

Design R-Value

R-Value (By Exudation Pressure) <5

Specified Minimum _____

Durability Index

Coarse Durability _____

Fine Durability _____

Durability Index _____

Specified Minimum _____

3cc: Woodward Clyde Consultants
Attention: Mr. Scott M. Leck

TESTING ENGINEERS, INCORPORATED

Reviewed by Walter C. Leonard
Walter C. Leonard
Laboratory Supervisor

Form T-30

Figure 29

TESTING ENGINEERS, INCORPORATED

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SANTA CLARA
MONTEREY/SALINAS
OAKLAND
STOCKTON
SACRAMENTO
DIABLO VALLEY

LABORATORY NUMBER L0457-3 8-9-90

TEI PROJECT NO. 02636

Job Data:

Woodward Clyde Consultants
Great America Parking Lot
Job 90C-0397-R

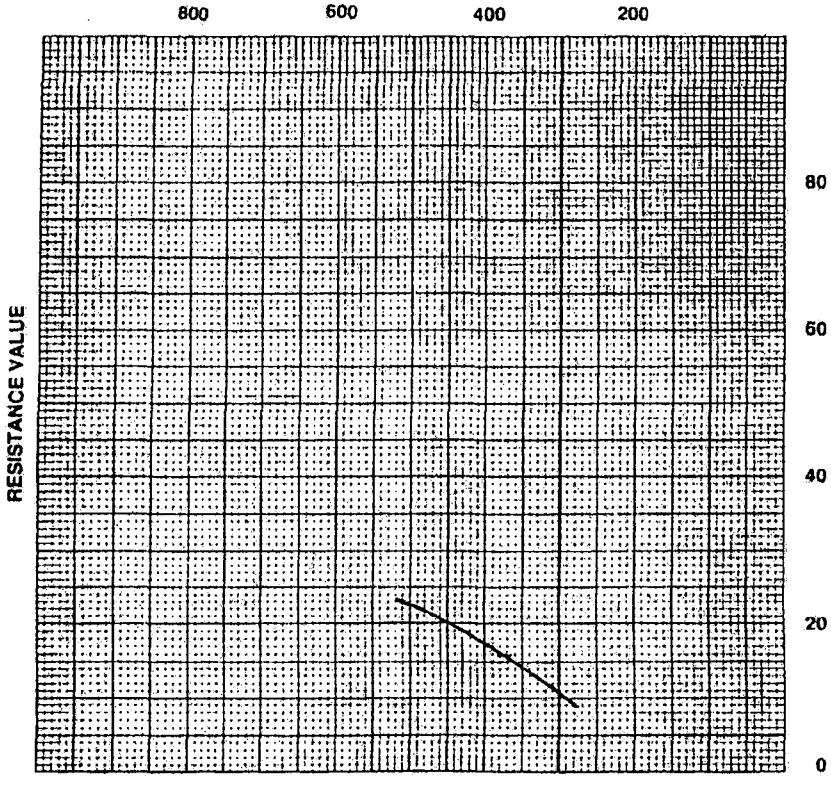
Sample Data:

Very dark brown clay with orgainc material
Boring 9, Depth 12"

Sample Received: 7-26-90

Resistance Value Test

Exudation Pressure, PSI



Compactor Pressure psi	150	100	50
Moisture @ Compaction %	21.9	23.7	26.5
Dry Density @ Compaction pcf	103.3	99.4	94.7
Exudation Pressure, psi	510	380	280
Resistance Value "R"	23	16	9
Expansion, Dial Reading .00	76	40	21
Expansion, psf	329	173	91

GRADING ANALYSIS

Percent Passing

Sieve Size	As Rcvd.	As Tested	As Rcvd. Specified Limits
3"			
2½			
2			
1½			
1			
3/4			
1/2			
3/8			
#4			
8			
16			
30			
50			
100			
200(wash)			

Sand Equivalent Test (Aver. of 3)

S.E. SPECIFIED (MINIMUM) _____

Design R-Value

R-Value (By Exudation Pressure) 10

Exudation Specified Minimum _____

Durability Index

Coarse Durability _____

Fine Durability _____

Durability Index _____

Specified Minimum _____

3cc: Woodward Clyde Consultants
Attention: Mr. Scott M. Leck

TESTING ENGINEERS, INCORPORATED

Reviewed by Walter C. Leonard
Walter C. Leonard
Laboratory Supervisor

Figure 30

Form T-30

TESTING ENGINEERS, INCORPORATED

401 ALDO AVENUE • SANTA CLARA, CALIFORNIA 95050 • 408-988-8888

SANTA CLARA
MONTEREY/SALINAS
OAKLAND
STOCKTON
SACRAMENTO
DIABLO VALLEY

LABORATORY NUMBER L0457-2 8-9-90

TEI PROJECT NO. 02636

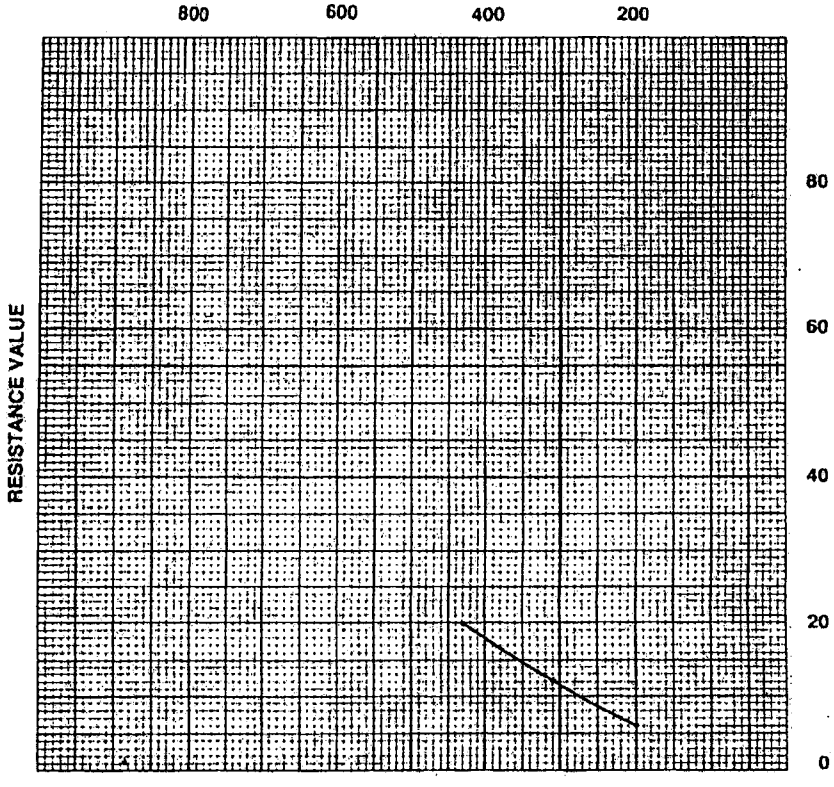
Job Data: Woodward Clyde Consulting
Great America Parking Lot
Job 90C-0397-R

Sample Data:
Olive gray clay
Boring 17, Depth 2'-3'

Sample Received: 7-26-90

Resistance Value Test

Exudation Pressure, PSI



Compactor Pressure psi	125	75	50
Moisture @ Compaction %	16.5	18.6	21.2
Dry Density @ Compaction pcf	112.4	109.1	104.5
Exudation Pressure, psi	430	310	200
Resistance Value "R"	20	12	6
Expansion, Dial Reading .00	75	21	8
Expansion, psf	325	91	35

GRADING ANALYSIS

Percent Passing

Sieve Size	As Rcvd.	As Tested	As Rcvd. Specified Limits
3"			
2½			
2			
1½			
1			
3/4			
1/2			
3/8			
#4			
8			
16			
30			
50			
100			
200(wash)			

Sand Equivalent Test (Aver. of 3)

S.E. SPECIFIED (MINIMUM)

Design R-Value

R-Value (By Exudation Pressure) 11

Exudation Specified Minimum

Durability Index

Coarse Durability

Fine Durability

Durability Index

Specified Minimum

3cc: Woodward Clyde Consulting
Attention: Mr. Scott M. Leck

TESTING ENGINEERS, INCORPORATED

Reviewed by Walter C. Leonard
Walter C. Leonard
Laboratory Supervisor

Figure 31

Form T-30

Depth in Feet	Sample No.	Sample Location	Blows/ft	Date Drilled: [illegible] Logged By: [illegible]		
0				Existing Levee Surface		
2				LEVEE FILL: Brown grey SILTY SAND (SM) with gravel and trace clay, medium dense, moist, numerous rocks 3" to 4" in diameter.		
4						
6	1-1		26	LEVEE FILL: Brown SANDY CLAY (CL) with gravel, moist, very stiff to hard, PP = 4.5+ ksf.	133	9
8						
10	1-2		27	NATIVE SOIL: Dark grey SILTY CLAY (CH) with roots, moist, very stiff, TV = 3.2 ksf, PP = 2.8 ksf.	103	23
12						
14				Brown SANDY CLAY (CL), moist, stiff to very stiff, TV = 2.6 ksf, PP = 1.8 ksf.		
16	1-3		19		114	18
18						
20				Brown SILTY SAND (SM) with gravel, wet, dense.		
22	1-4		68		127	12
24						
26	1-5		13	Becoming loose to medium dense.	105	21
28				Brown and grey mottled SILTY CLAY (CL) (Continued on Figure A2)		

Figure A1 - Test Boring No. 1 (0 to 28 feet)

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-1	Figure 32a

Date Drilled: 6-30-86
Logged By: [illegible]

Depth	Sample	Blows	Description	Moisture	Density
28			Continued from Figure A1		
30, 32	1-6	25	Brown and grey mottled SILTY CLAY (CL), moist, very stiff, TV = 3.0 ksf, PP = 2.3 ksf.	119	17
34, 36, 38	1-7	13	Brown and grey mottled SILTY CLAY (CH), very moist, firm, LL = 60%, PI = 34%.	85	35
40, 42	1-8	12	PP = 0.8 ksf.	96	27
44, 46, 48	1-9	24	Brown with grey mottling SILTY CLAY (CL), moist, stiff to very stiff, TV = 2.6 ksf, PP = 1.5 ksf.	106	23
50, 52, 54	1-10	50/6"	Brown SILTY SAND (SM) with gravel, dense. No sample recovered 50 - 51.5 feet.		
56	1-11	70	No sample recovered 55 - 56.5 feet. (Continued on Figure A3)		

Figure A2 - Test Boring No. 1 (28 to 56 feet)

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-1	Figure 32b

Depth	Sample	Blows	Description	Dry Density	Moisture
			Date Drilled: 6-30-86 Logged By: MG		
56			Continued from Figure A2		
58			Brown SILTY SAND (SM) with gravel, moist, medium dense to dense.		
60	1-12	54		112	17
62					
64			Grey SILTY CLAY (CH), moist, stiff to very stiff. LL = 54%, PI = 33%		
66	1-13	22		93	28
68					
70	1-14	45	TV = 2.8 ksf, PP = 3.3 ksf.	97	27
72					
74					
76	1-15	26			
78			Greenish grey CLAYEY SAND (SC), fine grained, moist, medium dense, PP = 1.6 ksf.	110	21
80	1-16	29	Grey with brown mottling SILTY CLAY (CL), moist, stiff, TV = 1.1 ksf, PP = 1.5 ksf, LL = 30%, PI = 14%.	105	22
82					
84			(Continued on Figure A4)		

Figure A3 - Test Boring No. 1 (56 to 84 feet)

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-1	Figure 32c

Depth in Feet	Sample No.	Blows	Description	Moisture Content (%)	Dry Density (pcf)
			Date Drilled: 6-30-86 Logged By: [illegible]		
84			Continued from Figure A3		
86	1-17	27	Brown and grey mottled SILTY CLAY (CL), moist, stiff to very stiff, TV = 2.1 ksf, PP = 1.8 ksf.	106	21
88					
90	1-18	22	TV = 1.2 ksf, PP = 1.8 ksf.	101	25
92					
94					
96	1-19	23	Contains fine sand, PP = 1.8 ksf	109	20
98					
100	1-20	27	TV = 1.1 ksf, PP = 1.5 ksf.	108	22
102			Bottom of Boring = 101.5 feet.		

NOTES:

1) The depth to free groundwater could not be measured due to the use of drilling fluid.

2) TV = Torvane shear strength
PP = Pocket penetrometer shear strength
LL = Liquid limit
PI = Plasticity Index

Figure A4 - Test Boring No. 1 (84 to 101.5 feet)

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-1	Figure 32d

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION Date Drilled: 7-1-86 Logged By: MG	IN-PLACE DRY DENSITY pcf	IN-PLACE MOISTURE CONTENT % dry wt
0				Existing Levee Surface		
2, 4, 6	2-1		24	LEVEE FILL: Brown SANDY CLAY (CL) with gravel, damp, very stiff.		
8				NATIVE SOIL: Dark grey SILTY SAND (SM), damp, medium dense.	102	17
10, 12	2-2		9	Brown SILTY SAND (SM), mostly fine grained, very moist, loose. 2-inch thick silty clay seam at 11 feet.	98	21
14, 16	2-3		13	Brown SILTY CLAY (CL), moist, very stiff, TV = 2.3 ksf, PP = 2.0 ksf, LL = 43%, PI = 24%.	105	22
18, 20, 22, 24	2		42	Brown SILTY SAND (SM) with gravel, moist, medium dense.	129	11
26	2-5		17		110	19
28				Brown SILTY CLAY (CL) (Continued on Figure A6)		

Figure A 5 - Test boring No. 2 (0 to 28 feet)

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-2	Figure 33a

Depth in Feet	Sample No.	Sample Type	Blows per 6 in.	Date Drilled: 7-1-86 Logged By: MG	Dry Density (pcf)	Moisture Content (%)
28				Continued from Figure A5		
30, 32	2-6		31	Brown SILTY CLAY (CL), moist, very stiff, TV = 2.3 ksf, PP = 2.5 ksf.	113	19
34, 36, 38	2-7		34	Brown and grey mottled SILTY CLAY (CH), moist, stiff, TV = 1.7 ksf, PP = 1.3 ksf.	94	31
40, 42	2-8		43	Brown and grey mottled SILTY CLAY (CL), moist, very stiff, TV = 3.1 ksf, PP = 3.5 ksf.	112	20
44, 46, 48	2-9		41	Becoming mottled brown, TV = 2.7 ksf, PP = 2.3 ksf.	112	19
50, 52	2-10		22	Becoming brown, stiff, LL = 26%, PI = 12%.	109	20
54, 56	2-11		36	Brown-grey CLAY SILT (CL-ML), very moist, very stiff, PP = 2.3 ksf. (Continued on Figure A7)	102	25

Figure A6 - Test Boring No. 2 (28 to 56 feet)

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-2	Figure 33b

DEPTH IN FEET	SAMPLE NO.	LOCATION OF SAMPLE	Blows/ft	Date Drilled [illegible] Logged By [illegible]		
				Continued from Figure A6		
56				Brown-grey CLAY SILT (CL-ML), very moist, very stiff.		
58						
60	2-12		40	Brown SILTY CLAY (CL), moist, hard, PP = 4.5+ ksf.	116	17
62				Grey SILTY SAND (SM) with gravel, moist, medium dense.		
64				Greenish grey SILTY CLAY (CH), moist, stiff to very stiff, TV = 2.5 ksf, PP = 2.5 ksf.		
66	2-13		33		102	25
68						
70	2-14		29	TV = 1.8 ksf, PP = 1.3 ksf.	105	23
72						
74						
76	2-15		45	TV = 2.8 ksf, PP = 1.8 ksf	105	24
78						
80	2-16		29	TV = 1.2 ksf, PP = 1.0 ksf.	108	22
82						
84				Mottled brown-grey SILTY CLAY (CL) (Continued on Figure A8)		

Figure A7 - Test Boring No. 2 (56 to 84 feet)

Project No. 90C-0397-R
Woodward-Clyde Consultants

LOG OF BORING No. 86-2

Figure 33c

Depth (feet)	Samples No.	Blows per 6 inches	Moisture Content (%)	Description	Dry Density (pcf)	Liquid Limit (%)
				Date Drilled: 7-1-86 Logged By: MG		
84				Continued from Figure A7		
86	2-17		40	Mottled brown-grey SILTY CLAY (CL), moist, very stiff, TV = 2.8 ksf, PP = 2.5 ksf.	107	22
88						
90	2-18		24	Grey CLAY SILT (CL-ML), moist, stiff, PP = 1.1 ksf.	108	21
92						
94				Grey with brown mottling SILTY CLAY (CL), moist, stiff, PP = 1.8 ksf.		
96	2-19		33		110	21
98				Grading to grey		
100	2-20		30	PP = 1.0 ksf.	106	22
102				Bottom of Boring = 101.5 feet. NOTES: 1) The depth to free groundwater could not be measured due to the use of drilling fluid. 2) TV = Torvane shear strength PP = Pocket penetrometer shear strength LL = Liquid Limit PI = Plasticity Index.		

Figure A8 - Test boring No. 2 (84 to 101.5 feet)

Applied Soil Mechanics

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-2	Figure 33d

November 7, 1986

Feet	Sample	Description			
0		Existing creek bottom.			
	3-1-1	Grey Silty CLAY (CH), wet, very soft.	64	62	
	3-1-2				
2		Brown SILTY SAND (SM), wet, loose	95	29	
4					
6	3-2	Greenish brown SANDY CLAY (CH), wet, soft.	75	49	
8	3-3	Brown SILTY SAND (SM), very moist, medium dense.	115	18	
10	3-4	Brown SILTY CLAY (CL), moist, stiff.	110	23	
12		Bottom of boring = 10.5 feet.			

Figure A9 - Test Boring No. 3

Applied Soil Mechanics

Project No. 90C-0397-R Woodward-Clyde Consultants	LOG OF BORING No. 86-3	Figure 34

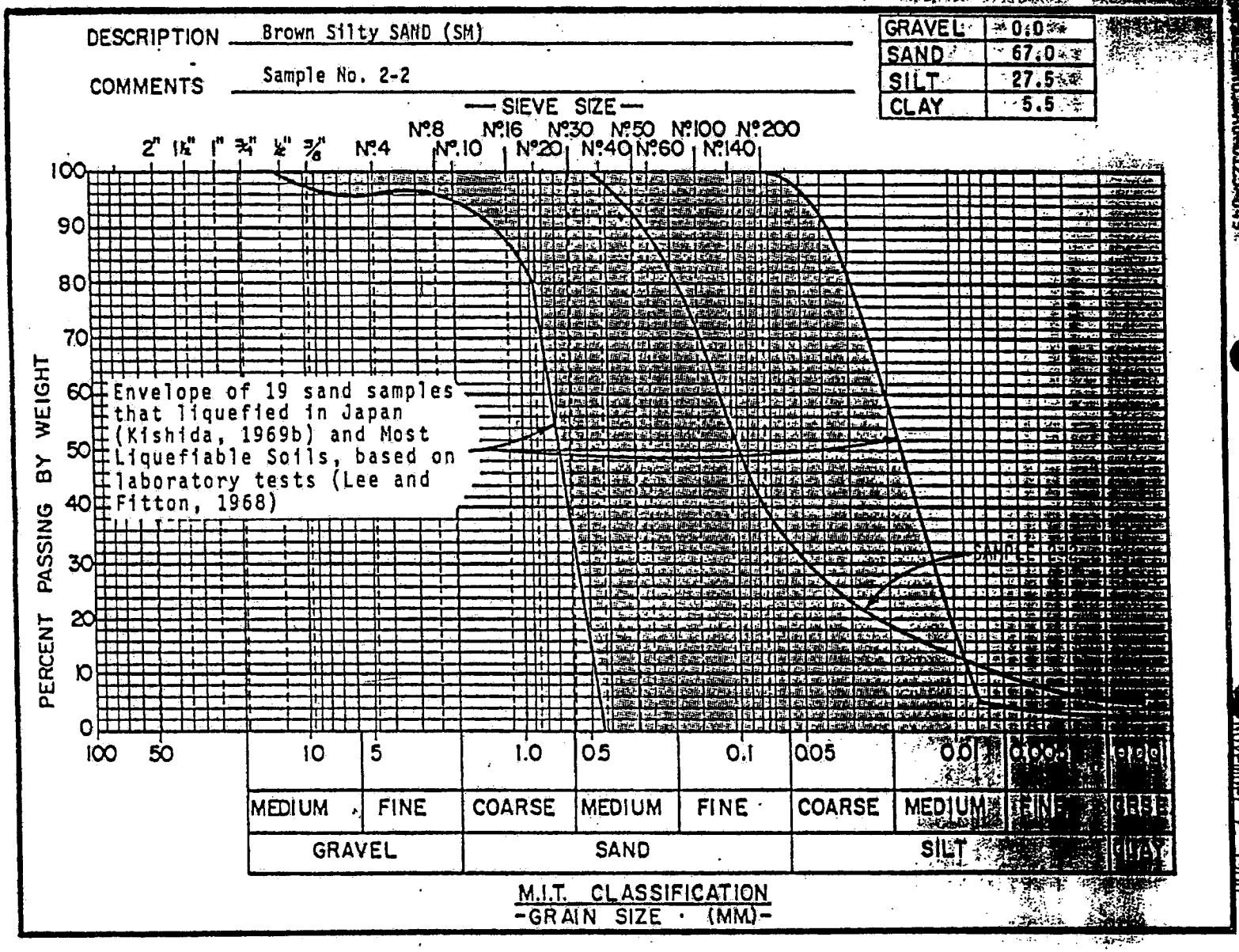


Figure B - Results of Laboratory Gradation Analysis

Project No. 90C-0397-R

Woodward-Clyde Consultants

Limited Soil Mechanics

Figure 35

November 7, 1986

FINAL REPORT

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ENGINEERING DEPT.
CITY OF SANTA CLARA

GEOTECHNICAL STUDY
SANTA CLARA
CONVENTION CENTER
Santa Clara, California

Prepared for:
City of Santa Clara
1500 Walburton Avenue
Santa Clara, CA 95050
July 14, 1993

Woodward-Clyde

Woodward-Clyde Consultants
55 South Market Street
Suite 1650
San Jose, California 95113

Project 93C-0250-R

(58)

Woodward-Clyde Consultants

Engineering & sciences applied to the earth & its environment

July 14, 1993

Project 93C-0250-R

Mr. Bruce Augason
City Engineer
City of Santa Clara
1500 Walburton Avenue
Santa Clara, CA 95050

Re: Geotechnical Study
Santa Clara Convention Center Expansion
Santa Clara, California

Dear Mr. Augason:

As authorized, we have performed a geotechnical study for the subject Convention Center Expansion. The purpose of this study was to review available data and develop recommendations for foundation design and associated geotechnical aspects of the planned construction.

We have reviewed available data in our files and visited the project site to familiarize ourselves with the conditions in the vicinity of the project site; we also reviewed grading records for the existing building provided by the City of Santa Clara. During our study, we consulted with Mr. Michael Shekhner of Ellerbe Becket regarding the geotechnical aspects of the various possible types of foundation systems. Our findings, engineering opinions and recommendations are presented in the accompanying report; they are based on the results of our review and engineering analysis, as well as our experience at the site and engineering judgement.

We are pleased to have been of service on this project. If you have any questions regarding this report, or we may be of further service, please contact our office.

Sincerely,

S. Stephen Huang
Project Engineer

Paul J. Boddie
Consultant

SSH/PJB/sk

NETDOC\SANDY\30250GEO.RPT\7/14/93
55 South Market Street, Suite 1650 • San Jose, CA 95113 • (408) 297-9585 • Fax: (408) 297-6962

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1.0
INTRODUCTION

We have performed a geotechnical study for the proposed expansion at the existing Convention Center in Santa Clara, California. The expansion site is located in the southern portion of the Convention Center complex as shown in the Site Layout Plan, Figure 1. Based on a review of data available in our files, as well as our experience at the Convention Center site and engineering judgement, we have developed geotechnical recommendations for the foundation design and slab-on-grade floor preparation. These recommendations were developed in consultation with the Project Structural Engineer, Mr. Michael Shekhner. Our findings and recommendations are presented in this report.

1.1. PROJECT DESCRIPTION

The proposed expansion of the Santa Clara Convention Center will be a triangular shaped, two-story building located adjacent to the southern wing of the existing main building. Based on an undated project plan prepared by Ellerbe Becket, entitled "First Level Plan," the expansion is about 12,000 square feet in plan.

The new building will be structurally separated from the main building. It will be a steel frame structure with a lightweight concrete second level. Ellerbe Becket estimate the maximum column loads will be 320 kips due to combined dead plus live loads; the maximum additional braced column load due to seismic forces will be 145 kips.

It is planned that the finished floor grade of the expansion building will match that of the main building at Elevation 12.5 and will have a slab-on-grade floor. An elevator pit is planned near building line T, between lines 4 and 5. The pit bottom will be about 4 feet below the finished floor, which corresponds to about Elevation 8.5. An escalator pit is planned near building D, between lines 5 and 7; its bottom also will be about 4 feet below finished floor at about Elevation 8.5.

1.2 BACKGROUND

Woodward-Clyde Consultants performed the original geotechnical studies for the project. The results of the investigations and our construction related services are presented in the following reports:

- "Preliminary Geotechnical Investigation, Santa Clara Convention Center, Santa Clara, California," Project Number 15582V, dated June 6, 1983;

- "Geotechnical Investigation, Santa Clara Convention Center, Santa Clara, California," Project Number 16029A, dated March 30, 1984;

- "Indicator and Load Test Program, Santa Clara Convention Center, Santa Clara, California," Project Number 16029X, dated January 15, 1985;

At a meeting on March 25, 1993, Mr. Paul Boddie of our office met with Messrs. Bruce Auguson, Sho Yoshida, and Eric Chin of the City of Santa Clara, and Mr. Michael Shekhner of Ellerbe Becket. The advantages and disadvantages of various types of foundation systems were discussed.

In our original geotechnical investigation for the Convention Center, settlement was a primary concern because of the heavy foundation loads and long spans. A driven pile foundation ultimately was selected to minimize post-construction settlement and reportedly has performed well. Since the Expansion foundation loads are moderate, by comparison, it was agreed at the meeting that other foundation types, such as spread footings and piers, should be considered. In this way, the disruption associated with pile driving could be minimized. However, it also was agreed that a driven pile foundation should not be ruled out if settlement of the alternative systems would exceed tolerable limits established by Ellerbe Becket.

1.3 SCOPE

We performed our geotechnical assessment based on existing available data. Specifically, our study has included the following:

- Reviewed existing subsurface boring and cone penetrometer data in the geotechnical reports listed above;

- Reviewed production pile driving records of adjacent driven piles for the Convention Center;

- Compared original topographic base maps with as-built drawings supplied by the City of Santa Clara to assess the areal extent and thickness of fill placed as part of the original mass grading;

- Visited the project site to review the performance of the existing building, roadways and concrete flatwork in the expansion area;

- Performed engineering analysis based on available data to develop geotechnical design parameters and provide recommendations for other aspects of development, including:

  - foundation type, depth and parameters for alternative systems, including spread footings, drilled piers and driven piles
  - settlement
  - lateral and uplift resistance, as required
  - slab-on-grade floor preparation

- Consulted with Ellerbe Becket and the City of Santa Clara regarding feasible foundation systems.

2.1 SITE CONDITIONS

On June 1, 1993, we visited the project site. The site is currently covered with concrete flatwork which slopes down away from the building. An as-built drawing entitled "Surface Improvements Area 18, Santa Clara Conference and Convention Center, Assessment District No. 183," Sheet Number 26, dated January 26, 1986, indicates the finished floor is at Elevation 12.5; the perimeter roadway curb slopes slightly upward from Elevation 10.95, near Building Corner T11 (southwest corner of the project site), to about Elevation 11.64, near Building Corner V19 (northwest corner of the project site). An area between the perimeter roadway curb and the edge of the concrete flatwork of about 10 feet wide is landscaped with grass and mature trees. At the time of our visit, we did not observe any apparent signs of cracking or distress in the exposed portions of the interior floor slabs and columns, as well as the exterior concrete flatwork, walkways and walls.

2.2 EXISTING FILL

2.2.1 Existing Building Area

Based on a review of construction data provided by the City of Santa Clara, including construction grading plans, results of laboratory compaction and field density testing performed by the City of Santa Clara, we understand that imported select fill was used in the main building areas as specified. In accordance with an untitled grading plan, the fill section was over-built slightly near the corner at Building Lines T.4 and 16.2. Reportedly, 2 feet of imported select fill was placed to form the floor support. As recorded, the import material was a clayey gravelly sand from the Curtner Pit in Milpitas, California. The City of Santa Clara laboratory data indicates the plasticity index (PI) is between 7 and 10. The native subgrade and the imported fill were compacted to a minimum degree of compaction of 90 percent based on Caltrans Test Method 216F.

2.2.2 Existing Concrete Flatwork Area

Landscaping is present between the flatwork and the roadway. No records are available regarding the nature or compaction of the fill in the flatwork or landscaped area. We expect it to be about 2 feet thick near the building and decreases in thickness towards the roadway.

2.3 SUBSURFACE SOIL CONDITIONS

For our 1983 and 1984 studies, one exploratory boring and three cone penetrometer tests (CPT) were made within 100 feet of the perimeter of the expansion site as shown on Figure 1. Boring 10 extended to a depth of about 80 feet, whereas the CPTs, C9, C10 and C12, extended to depths of 120 to 130 feet. Logs of these boring and CPTs are presented in Appendix A as reference.

Below the floor slab and fill, the native soils were found to consist of thick deposits of clays and silts with interbedded sand layers. The upper 3 to 4 feet of clay was hard and desiccated. The underlying 8 to 10 feet of low plasticity silty clays were found to be very stiff to medium in consistency; occasional clayey silt zones were noted to occur in this clay stratum. A medium plasticity silty clay was found to extend to a depth of about 110 feet and was described as variable in strength and found to grade occasionally to clayey silt. This clay stratum was found to be interrupted by two apparently continuous sandy zones. The shallower stratum of medium dense sand occurred at about Elevation -12 to -16. The deeper sand stratum was also medium dense and occurred at about Elevation -35 to -50. Between Elevations -100 and -110, a zone of sand, silty sand and silty clay was revealed. Below that depth to the terminal depths of the boring and CPTs, a medium to high plasticity clay of very stiff consistency was encountered.

Laboratory consolidation tests and field pressuremeter tests indicated the subsurface clays were overly consolidated; the over-consolidation ratio was found to be about 2 to 3. These soils were considered somewhat compressible with respect to the existing building loads.

2.4 GROUNDWATER

The groundwater level measured in Boring 10 of the preliminary study was at a depth of about 6-1/2 feet, which corresponded to approximately Elevation 3-1/2. Subsequently in the 1984 study, groundwater levels measured in other exploratory borings were about 9 feet below the ground surface (Elevation 1 $\pm$) at that time. This suggests some seasonal changes in the groundwater level.

3.0
DISCUSSION

Since the native soils beneath the fill are relatively strong and only moderately compressible under the estimated column loads, we believe a shallow type foundation such as spread footings will be capable to support the building. Foundation settlement remains to be the main geotechnical issue; in particular, the differential settlement between columns in the expansion building and the differential movement between the new and existing buildings. Considering that the soils are highly overconsolidated, it appears all compression caused by new building loads or fill will be recompression. We have estimated the settlement for the anticipated foundation loads and we discussed them with Mr. Shekhner, Structural Engineer. It is his opinion the estimated settlement would be structurally tolerable.

The existing fill within the expansion area was placed in two stages. The fill near the existing building was placed as a part of the original pad construction. Records indicate the fill is probably of select quality and is well compacted. The fill beneath the existing concrete flatwork and landscaping was placed in the latter part of the Convention Center construction. There are no records regarding its thickness, quality, or compaction. The transition between the two fill areas is not well defined.

Because of the uncertainties regarding the nature and density of the existing fill, we recommend that the existing fill be recompacted as a part of the grading for the new addition. The quality of the existing fill and its suitability for re-used should be reviewed by a representative of our firm as it is being excavated.

4.1 SPREAD FOOTINGS

We recommend the expansion building be supported on spread footings bearing on native soils or engineered fill. The bottom of the footings should extend to a minimum embedment depth of 2 feet below the minimum adjacent finished grade. Where applicable, the engineered fill section beneath footings should be at least equipment width and should extend at least 2 feet horizontally beyond the perimeter of the footings.

Spread footings should be designed for bearing pressure not to exceed 2,000 psf due to dead load, 3,000 psf for dead plus live loads, and 4,000 psf for all loads including wind and seismic. These values are net values and represent the supporting capabilities above and beyond the weight of the footing concrete. If the weight of the footing is included, the values can be increased by 20 percent. These recommended values are lower than those provided in our 1985 study for a footing width less than or equal to 10 feet. The main reason is to minimize the induced foundation pressures that may cause settlement. A more detailed discussions of settlement is presented in the following section.

Lateral load resistance for the foundations can be mobilized by a combination of passive pressure against the adjacent soil and frictional resistance between the base of the footings and the soils. It is recommended that the passive resistance of the on-site soils be determined using the lateral pressure of an equivalent fluid weight of 400 pcf. Frictional resistance between the soil and foundations placed against undisturbed soil should be determined using a coefficient of friction of 0.3, with the total resistance not to exceed 1,000 psf.

4.2 SETTLEMENT

Total settlement occurring in the area of the proposed expansion building will be due to loads imposed by the fill and that due to the building. Based on the proposed finished floor elevations, the net increase in fill thickness will be about 1-1/2 feet near the roadway curb and taper gradually toward the existing building; no additional fill is anticipated near the

existing building. Using the design bearing pressures presented above, we first estimated the size of the individual footings using estimated column loads provided by Ellerbe Becket. Then, the foundations pressures corresponding the estimated footing size at the column locations were used for estimating settlement.

Based on subsurface information, we estimate the native clays are overly consolidated with an over-consolidation ratio (OCR) of about 3. The foundation pressures that may be induced by the column loads are expected to be within the recompression range as the clays consolidate. Since the upper 3 to 4 feet of clays are strong and unsaturated, we expect consolidation settlement of this upper layer is unlikely. For the estimated footing size (9 to 12 foot square), we expect the footing pressures will influence the subsoils to about 20 to 30 feet below ground surface. The maximum long term settlement under the anticipated loading conditions is estimated to be 3/4 to 1 inch. Maximum differential settlement between adjacent columns is expected to be about 3/8 to 1/2 inch. Mr. Shekhner has confirmed these estimated settlements will be within tolerable limits of the structure.

4.3 SLAB-ON-GRADE FLOORS

All concrete slab-on-grade floors and sidewalks should be supported on a minimum of 24 inches of select quality import fill. The select fill should be non-expansive material having a maximum plasticity index of 15. The fill should be placed and compacted as described below in Section 4.5, Site Preparation and Grading. External concrete walkways also will be subject to movement due to swelling and shrinking of the native soils. To minimize this disturbance, external walkways also should be supported on a minimum of 24 inches of select import fill.

Moisture will come into contact with the floor slabs due to moisture vapor migration and/or capillary water rise through the soil. If floor coverings that will be placed on the slabs which will be susceptible to damage by moisture contact, moisture barriers should be used under the slabs. Any sand or gravel placed as a part of the capillary break/moisture barrier system can be used as part of the recommended select fill section.

4.4 ELEVATOR AND ESCALATOR PITS

Below grade elevator and escalator pits walls will experience lateral pressures due to the unbalanced horizontal thrust of the soil in contact with the walls. We expect that walls will be designed as restrained type walls and the walls should be designed to resist lateral earth pressure corresponding to the "at-rest" condition. We recommend that an equivalent fluid pressure of 55 pounds per cubic foot (pcf) be used. Surcharge loads such as loading equipment and foot traffic could apply additional loads to the walls. For design purposes, it should be assumed the force on the wall resulting from a uniform surcharge would be equal to one-third of the surcharge load.

The recommended design earth pressure has been based on a compacted backfill consisting of Structure Backfill material as defined in the Caltrans Standard Specifications. This zone of permeable material should be 18 inches wide and extend from the base of the wall up to within 24 inches of finished grade.

Since the groundwater levels measured were below Elevation 3-1/2 whereas the bottom of the pits is about Elevation 10-1/2, we expect buildup of hydrostatic pressures behind the wall be unlikely. Uplift at the bottom of the elevator and escalator is also unlikely.

Retaining wall footings should be designed in accordance with the criteria presented above for spread footing foundations.

4.5 SITE PREPARATION AND GRADING

The fill section for concrete floor slab support should be constructed as a uniform unit to minimize potential for differential movements due to inconsistent compaction and irregular composition of fill material. Therefore, we recommend over excavating the existing fill and recompacting the new fill section as a continuous unit.

Prior to any site grading, the existing concrete flatwork, pavement and landscape area should be removed in their entirety. When demolition and removal is completed, the existing fill materials should be excavated; we expect the bottom of the fill section to be at about Elevation 10. To avoid undermining the existing grade beam along the main

building perimeter, we recommend the excavation horizontal limits be maintained no closer than 2 feet from the perimeter. When the fill materials are excavated, the exposed subgrade conditions should be examined by the Geotechnical Engineer. If soft subgrade is encountered, it should be overexcavated and backfilled with bridging rock. Bridging rock is defined in the attached Guideline Specifications (Appendix B). The need to over-excavate the subgrade will be determined at that time. We expect the over-built section of imported select fill near the existing building and the aggregate base section beneath the asphalt concrete will be reusable as the recommended select fill section. The existing fill beneath the conrete flatwork and landscaping also may be of select quality and suitable for re-use. The suitability of the existing fills should be determined during grading by the Geotechnical Engineer.

After demolition and excavation have been completed, the exposed subgrade surfaces to receive fill should be scarified to a minimum depth of 6 inches, moisture conditioned and recompacted to the requirements of engineered fill. The native soils should be compacted at a moisture content of 1 to 3 percent above the optimum. Fill then can be placed to provide the desired finished grades. When the subgrade compaction has been approved by the Geotechnical Engineer, fill can be placed to provide the desired finished grade.

4.5.1 Fill Material

All fill placed for the building pad should meet the requirements for select material. Select material is a soil having a plasticity index of 15 or less meeting the requirements outlined in the attached specifications. All fill material should be approved by the Geotechnical Engineer prior to placement.

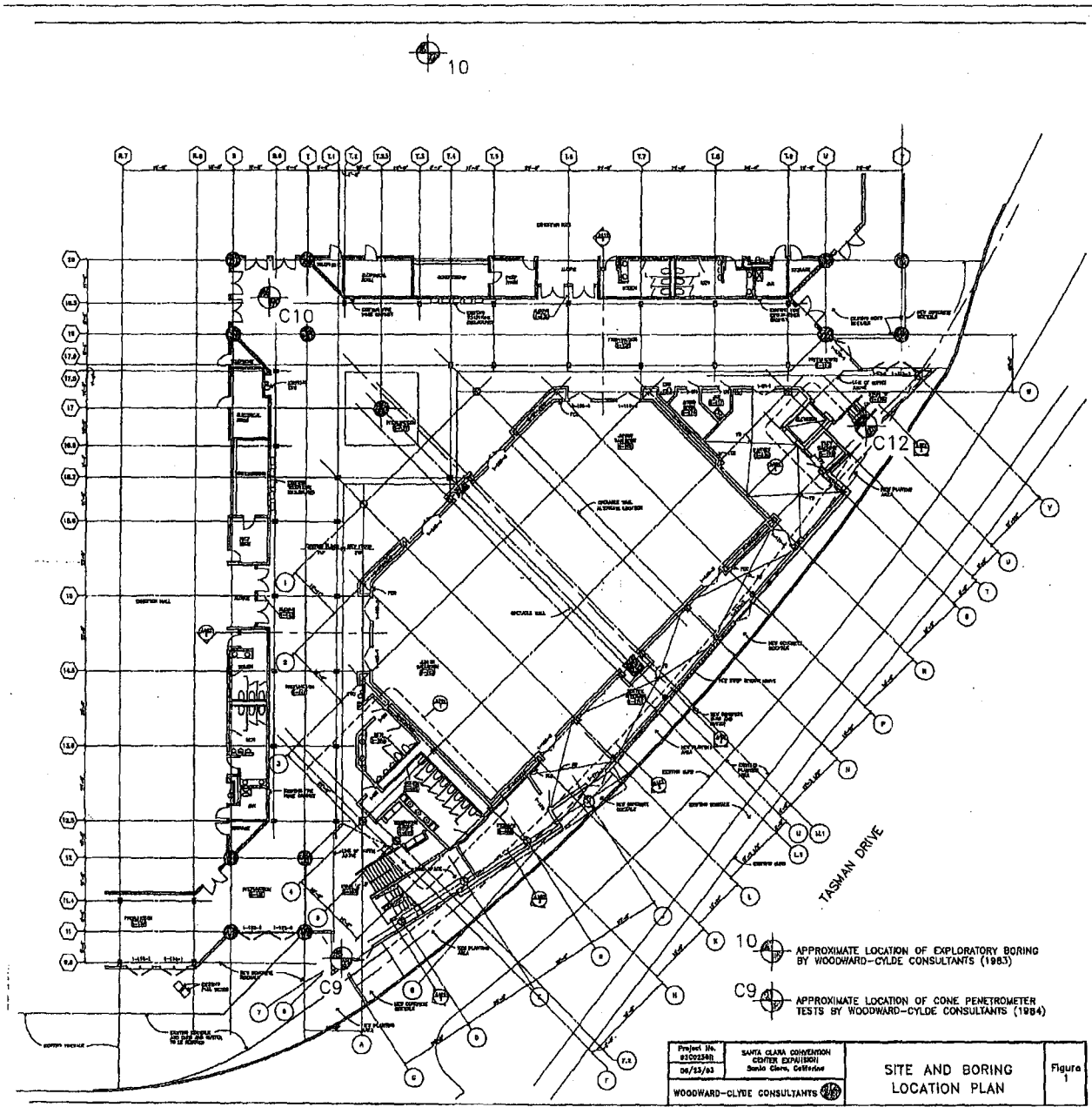
4.5.2 Fill Placement and Compaction

Fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to a minimum relative compaction of 90 percent. Relative compaction is defined in the attached specifications. The finished grade surface of the compacted fill under concrete flatwork or pavements should be kept in a moist condition prior to the placement of concrete.

5.0
LIMITATIONS

Our opinions and conclusions have been based on a review of available data from the original geotechnical investigation and subsequence geotechnical studies for the construction of the Santa Clara Convention Center. They are also based on previous experience at the site and engineering judgement. If the proposed construction will be different from that planned at the present time, the Geotechnical Engineer should be notified so that recommendations can be modified as needed.

The opinions expressed in this report were developed with the standard of care commonly used as state of the practice in the profession. No warranties are included, either expressed or implied, as to the professional advice included in this report.



KEY PLAN

MECHANICAL and ELECTRICAL ENGINEERS

Santa Clara California

Ellerbe Becket

FIRST LEVEL PLAN

A111

APPENDIX A
LOGS OF EXPLORATORY BORING AND CONE PENETROMETER TESTS FROM PREVIOUS STUDIES

Project: SANTA CLARA CONFERENCE CENTER Santa Clara, California	Log of Boring No. 10

Date Drilled: April 5, 1983 — Remarks:

Type of Boring: 6" Auger

Hammer Weight: 140 lbs. — (See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 10.0±

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	13	SILTY CLAY (CH) Stiff, moist, dark gray-brown	22	99	3310
5	2	20	SILTY CLAY (CL) Stiff, moist, brown mottled with gray and dark brown, with clayey sand and sand zones in upper three feet 2 Hrs Very stiff	-	-	-
10	3	19		25	99	6090
15	4	5	SILTY CLAY (CL) Medium, moist to wet, brown and gray mottled with dark brown Stiff ATD	28	95	3210
20	5	15	SILTY CLAY (CL) Stiff, brown and gray mottled Very stiff	19	108	4680
25	6	40	SILTY SAND (SM) Medium dense, gray and brown Grading to Sand (SP), dense, with some gravel	-	-	-
30	7	20	SILTY CLAY (CL-CH) Stiff, brown and gray mottled Less plastic, CL, with trace of fine sand	24	101	2730

Proj. No. 15582V	Woodward-Clyde Consultants	Figure 12a

Project: SANTA CLARA CONFERENCE CENTER
Santa Clara, California

Log of Boring No. 10 (Continued)

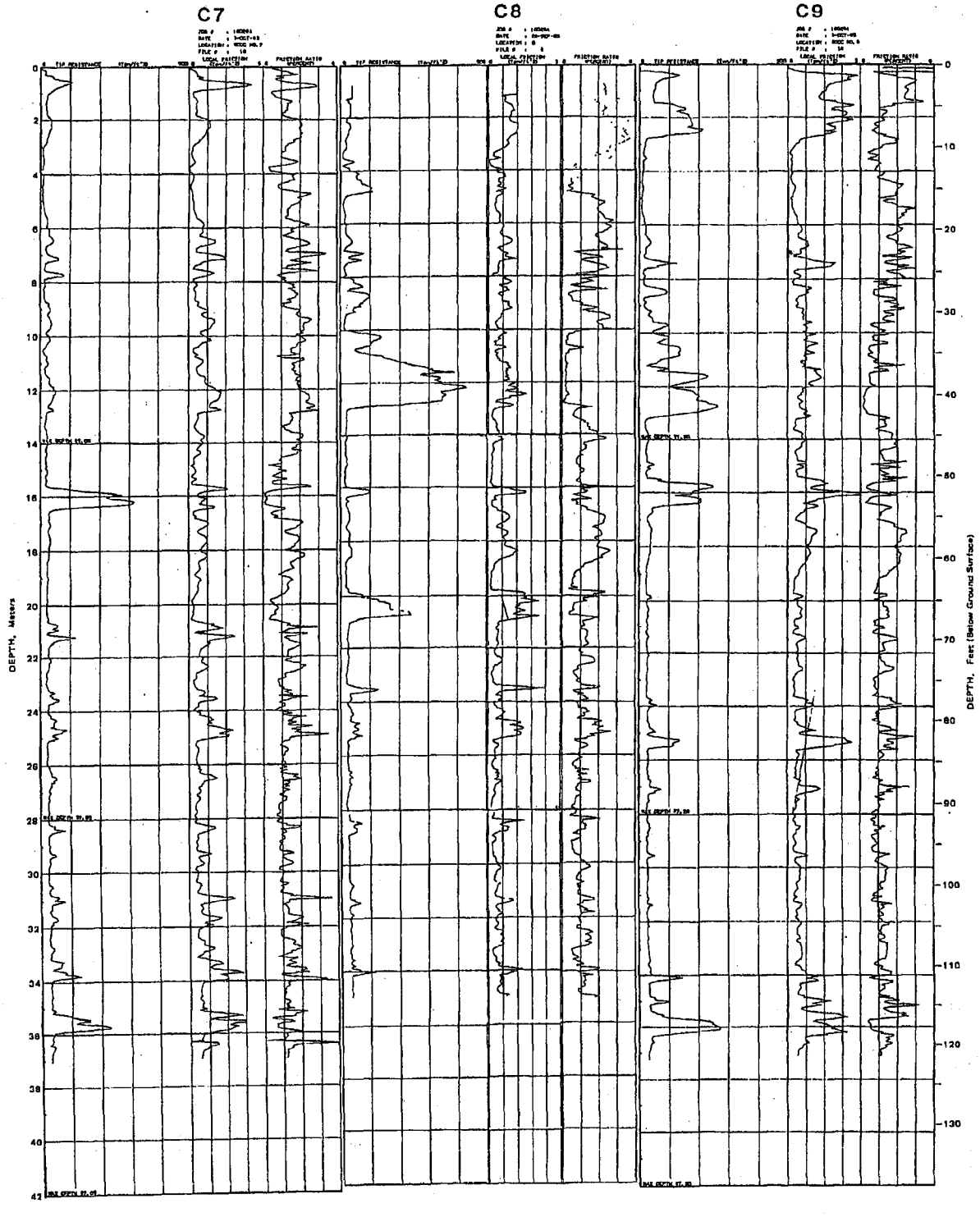
Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SILTY CLAY (CL) .....Cont'd			
	8	24	Very stiff	22	105	4970
35						
	9	21	Hard, with occasional calcareous nodules	18	111	8770
40						
	10	21	Grading to Sandy Clay (CL), stiff; -#200 = 51%	19	109	3070
45						
	11	30	SILTY SAND (SM)	-	-	-
50			Medium dense, gray and brown, with sandy silt and sand layers			
			With some clayey sand lenses			
	12	20		-	-	-
55						
			SILTY CLAY (CL)			
	13	50	Hard, gray-brown and gray mottled	23	102	11,450
60						
			Very stiff, with fine sand			
	14	20		18	109	5650
65						

Project: SANTA CLARA CONFERENCE CENTER
Santa Clara, California

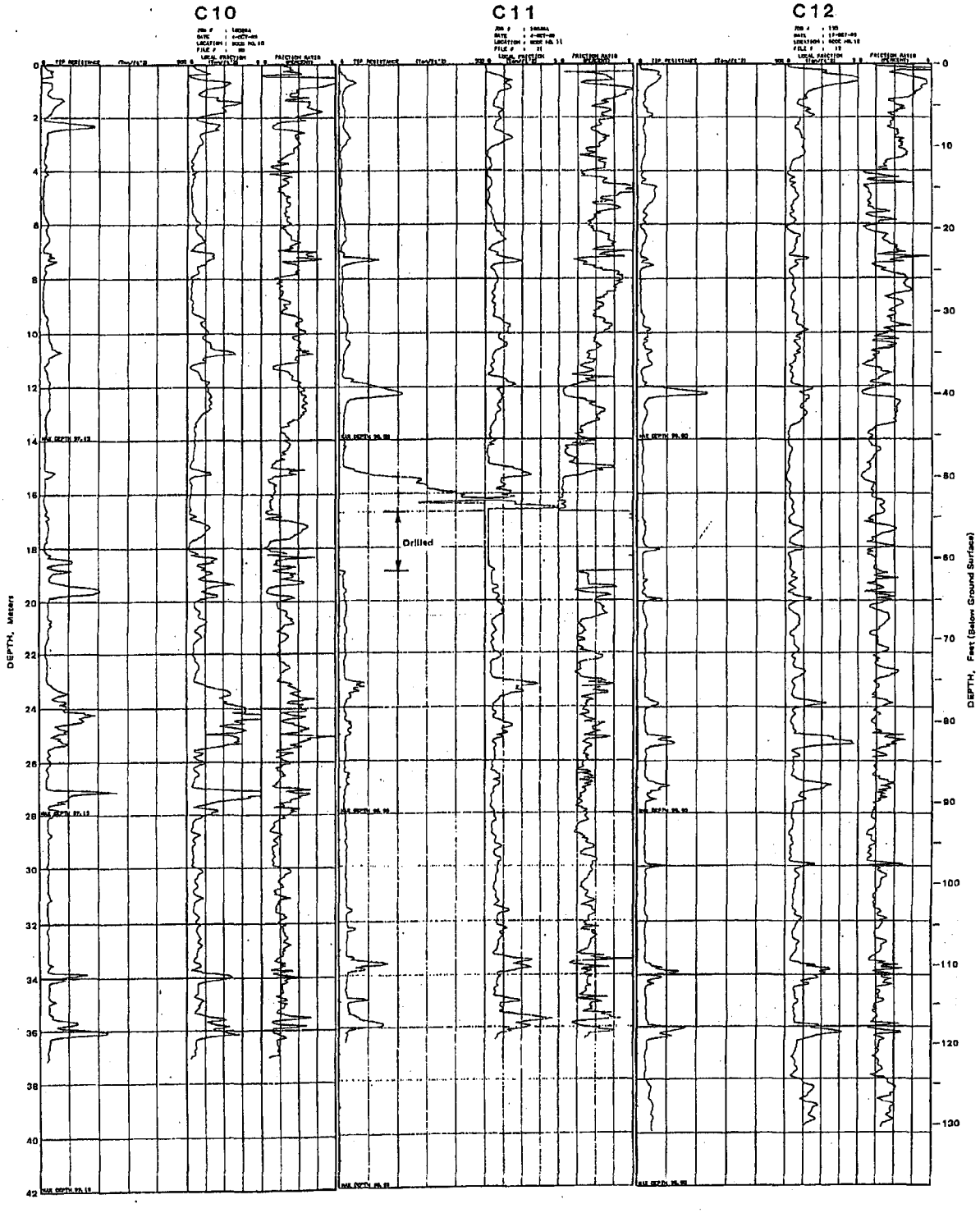
Log of Boring No. 10 (Continued)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Becoming very silty, CL-ML, stiff			
	15	22		24	98	3510
70			More plastic, CL, very stiff			
	16	33		30	94	6090
75						
	17	25		26	100	4870
80			BOTTOM OF BORING @ 80'			
85						
90						
95						

Proj. No. 15582V | Woodward-Clyde Consultants | Figure 12c



Project No. 18029A	Santa Clara Conference Center	CONE PENETEROMETER TESTS C7, C8, AND C9	Figure A-7
Woodward-Clyde Consultants			



Project No. 16029A	Santa Clara Conference Center	CONE PENETEROMETER TESTS C10, C11, C12	Figure A-8
Woodward-Clyde Consultants			

I - GENERAL CONDITIONS

A. Definition of Terms

Fill - is a soil or soil-rock material placed to raise the existing grade of the site or to backfill excavations.

Import Material - is hauled in from off-site borrow areas.

Engineered Fill - is a fill that has been constructed to specification requirements, in the opinion of the Geotechnical Engineer.

Standard Specifications - are the 1988, or later, edition of the Standard Specifications of the State of California, Department of Transportation, latest revision.

Materials Manual - is that of the State of California, Department of Public Works, Division of Highways, latest revision.

Relative Compaction - is the ratio of the in-place dry density of constructed fill to the maximum dry density determined by Test Method ASTM Designation D 1557.

In-Place Density - is the dry density of the constructed fill determined in accordance with the moisture-density gauge method, ASTM Designation D 2922-71.

B. Geotechnical Engineering Services

The Geotechnical Engineer will be the Owner's representative to observe the grading operations both during preparation of the site and the compaction of engineered fill. He will make visits to the site to familiarize himself generally with the progress and quality of the work. He will make field observations and tests to enable him to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials and the

extent to which the earthwork construction and the relative compaction comply with the specification requirements.

II - SITE PREPARATION AND EXCAVATION

A. Site Preparation

The fill section for concrete floor slab support shall be constructed as a uniform unit to minimize potential for differential movements due to inconsistent compaction and irregular composition of fill material. Therefore, the existing fill shall be over excavated, and the new fill section shall be compacted as a continuous unit.

Prior to any site grading, the existing concrete flatwork, pavement and landscape area shall be removed in their entirety. When demolition and removal is completed, the existing fill materials shall be excavated; we expect the bottom of the fill section to be at about Elevation 10. To avoid undermining the existing grade beam along the main building perimeter, the excavation horizontal limits shall be maintained no closer than 2 feet from the perimeter. When the fill materials are excavated, the exposed subgrade conditions shall be examined by the Geotechnical Engineer. If soft subgrade is encountered, it shall be overexcavated and backfilled with bridging rock. The need to over-excavate the subgrade will be determined at that time. We expect the over-built section of imported select fill near the existing building and the aggregate base section beneath the asphalt concrete be reusable as the select fill section. The existing fill beneath the concrete flatwork and landscaping also may be of select quality and suitable for re-use. The suitability of the existing fills shall be determined during grading by the Geotechnical Engineer.

After demolition and excavation have been completed, the exposed subgrade surfaces to receive fill shall be scarified to a minimum depth of 6 inches, moisture conditioned and recompacted to the requirements of engineered fill. The native soils shall be compacted at a moisture content of 1 to 3 percent above the optimum. When the subgrade compaction has been approved by the Geotechnical Engineer, fill then can be placed to provide the specified finished grade.

B. Excavation

Excavation is unclassified and shall be done to the lines, grades and dimensions indicated on the Drawings regardless of the character of the materials encountered.

When the required excavations have been made, the Geotechnical Engineer shall examine the exposed conditions. If pockets of debris or soft, weak soils are encountered at the required subgrade, carry the excavations deeper to the limits designated in the field by the Geotechnical Engineer.

C. Subgrade Preparation

After the site has been properly prepared, all necessary excavations shall be made to allow for construction of the recommended sections of engineered fill beneath the floor slab. The exposed subgrade soils shall be scarified, moisture conditioned to between 1 to 3 percent over optimum moisture content and compacted to the requirements of engineered fill to a minimum depth of 6 inches. When the subgrade has been compacted and approved by the Geotechnical Engineer, fill can be placed to provide the desired finished grades. The Contractor shall obtain the approval of the Geotechnical Engineer of subgrade compaction before further grading is done.

III - MATERIAL USED FOR FILL

A. General Fill Materials

All fill material shall be approved by the Geotechnical Engineer. The material shall be a soil or soil-rock mixture which is free from organic matter or other deleterious substances. The fill material shall not contain rocks or lumps over 6 inches in greatest dimension, and not more than 15 percent larger than 2-1/2 inches. It is expected that materials from the site, but below the strippings, may be reused as engineered fill.

B. Import Fill Requirements

In addition to the General Fill Requirements presented in III-A, import fill shall have a plasticity index of 15 or less.

C. Bridging Rock

Bridging rock shall consist of coarse granular mixture of rock fragments having a maximum particle size of 3 inches. It is anticipated that bank run or crusher run materials may be satisfactory. The material shall be well graded between the maximum and minimum size with no more than 15 percent passing the U.S. Standard Number 200 sieve.

IV - PLACING AND COMPACTING FILL MATERIAL

All fill material shall be compacted as specified below or by other methods, if approved by the Geotechnical Engineer. Fill material shall be spread in uniform lifts of not more than 8 inches in uncompacted thickness. Soil comprising the required fill section shall be compacted to a minimum relative compaction of 90 percent.

Before compaction begins, the fill shall be brought to a water content near the optimum 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. Native materials shall be compacted at a moisture content of 1 to 2 percent above the laboratory optimum. It shall be the responsibility of the Grading Contractor to attain the proper moisture content during compaction. No fill shall be placed during the rain or when saturation will hinder proper compaction. Jetting or flooding of the fill will not be permitted. The finished grade surface of the compacted fill under concrete flatwork should be kept in a moist condition prior to the placement of concrete.

V - TREATMENT AFTER COMPLETION OF GRADING

After grading is completed and the Geotechnical Engineer has finished his observation of the work, no further excavation or filling shall be done except with the approval of and under the observation of the Geotechnical Engineer.

It shall be the responsibility of the Grading Contractor to prevent erosion of freshly graded areas during construction and until such time as permanent drainage and erosion control measures have been installed.

Woodward-Clyde Consultants

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ENGINEERING DEPT.
CITY OF SANTA CLARA

INDICATOR AND LOAD TEST PROGRAM

SANTA CLARA CONVENTION CENTER

Santa Clara, California

Prepared for

City of Santa Clara
1500 Warburton Street
Santa Clara, CA 95050

81

1180 Coleman Avenue
San Jose, California 95110
408-297-9585

Woodward-Clyde Consultants

January 15, 1985

Project 16029-X

City of Santa Clara
1500 Warburton Avenue
Santa Clara, CA 95050

Attention: Mr. James Gleeson

Gentlemen:

As authorized, we have provided geotechnical engineering services during the installation of the foundation piles for the Santa Clara Convention Center. The accompanying report presents the results of the indicator pile and load test program along with a summary of the opinions and recommendations which were provided during the course of our geotechnical review of the production pile driving.

It has been a pleasure to have been a part of such a significant project. If any questions should arise regarding the information presented in this report, or if we can be of further assistance, please contact our office.

Sincerely,

Paul J. Boddie
Senior Project Engineer

William A. Bischoff
Vice President

PJB/WAB/rm

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APPENDIX A - Descriptions of Dynamic Monitoring Procedures

Figure 1 - Indicator Pile and Load Test Locations

Figures 2 through 5 - Pile Load Test Results

Figure 6 - Recommended Pile Capacity vs. Embedment Length

Table 1 - Summary of Indicator and Load Test Pile Data

Tables 2 and 3 - Summaries of Pile Driving Analyzer Results

Table 4 - Summary of Case Capacity Results

Table 5 - Summary of CAPWAP/C Results

Attachment - Pile Driving Records

INDICATOR PILE AND LOAD TEST PROGRAM
SANTA CLARA CONVENTION CENTER
Santa Clara, California

INTRODUCTION

This report presents the results of an indicator pile and load test program which was conducted for the Santa Clara Convention Center project in Santa Clara, California. The primary purposes of the program were to refine our engineering estimates regarding the capacity of driven pile foundations at the site and to recommend the lengths of piles to be cast and driven for the final production work.

Upon completion of the testing program, geotechnical consultation was provided to the City of Santa Clara's on-site representatives on an "as-needed" basis during the production pile driving. Opinions and recommendations provided during the course of the geotechnical review also are summarized herein.

GEOTECHNICAL INVESTIGATION AND FOUNDATION DESIGN

A geotechnical investigation was made by Woodward-Clyde Consultants to provide geotechnical parameters for the design of the subject project. The results of that investigation and recommended design parameters were contained in a report entitled "Geotechnical Investigation, Santa Clara Conference Center" (Project 16029-A) and dated March 30, 1984. In that report, it was recommended that the heavy building loads be supported using precast, prestressed concrete pile foundations driven into the stiff clay and dense sand strata which underlie the project site. Recommended axial compression and tension load capacities versus embedment depth were provided.

Subsequently, 12-inch square prestressed concrete piles were selected by Ellerbe Associates for the design foundation support system. Two lengths of piles, 40 feet and 70 feet,

were specified to be capable of individual downward load capacities of 40 tons and 80 tons, and uplift capacities 20 tons and 50 tons for the respective lengths.

INDICATOR PILE AND LOAD TEST PROGRAM

General

A total of 21 piles were driven at selected locations within the building footprint for the indicator pile and load test program. The driving of these piles was done between the dates of September 15, 1984 and October 12, 1984 by Riedel International, pile driving contractor for the project. The piles consisted of 12 inch square prestressed concrete piles which were driven at structural pile locations selected in the field by a representative of Woodward-Clyde Consultants and staked by Hensel Phelps, General Contractor. The approximate locations and field numbering of the indicator and load test piles are shown on Figure 1. The corresponding pile numbers, in accordance with the numbering and lettering system adopted by the City of Santa Clara, are summarized in Table 1 along with the pile lengths, approximate embedment depths, and tip elevations.

The piles were driven using two separate driving systems. The first driving system consisted of an ICE 520 double-acting diesel hammer with a maximum rated energy of 30,000 ft-lbs. The second system consisted of a larger ICE 640 double-acting diesel unit with a maximum rated energy of 40,000 ft-lbs. Each of the diesel hammers were positioned within a set of fixed leads. During driving of the indicator and load test piles, our field representative recorded the penetration resistances through the various soil strata for comparison with the nearby previous geotechnical explorations. The installation of piles C9-BF4-1, 2, 3 and 4 also was observed

by our field representative subsequent to the indicator and load test program. Copies of those driving records are attached.

Dynamic Monitoring

All 21 piles were dynamically monitored during initial driving and/or restrike with the assistance of Goble, Rausche, Likins and Associates. The monitoring consisted of the measurement of pile top force and acceleration in the field during hammer impact using the closed form Case Method solutions, and data processing in the laboratory using the CAPWAP/C Method of analysis. Descriptions of these analytical procedures are presented in Appendix A.

Measurement of force and acceleration were obtained using two strain transducers and two accelerometers mounted on opposite sides of the piles about 3 feet below the pile tops. Signals from these gages were conditioned and field processed with a Model GB Pile Driving Analyzer (PDA), and recorded in analog form on a 7-channel cassette recorder. Maxima of forces, transferred energies and estimated pile capacities were computed by the PDA for each recorded blow.

The data recorded in the field using the PDA then were returned to the laboratory for reanalysis. Eight selected data sets were digitized and processed by a minicomputer in a manner comparable to the PDA method. One blow from each of the digitized data sets then was selected for a CAPWAP/C (Case Pile Wave Analysis Program) analysis. In this method of analysis, the field measured acceleration values are used as input and the assumed soil resistance properties are changed in an iterative process until a best match is achieved between the computed and measured pile top forces.

Capacities estimated by the CAPWAP/C and Case Method then were correlated to obtain a Case damping factor, J, judged to be representative of the project site. In most cases, a J factor of 0.5 was judged to be reasonable for piles deriving their capacity principally through frictional resistance. However, at Test Pile Location EF-2 (revised), a J factor of 0.25 was selected since this pile derived its capacity through a combination of end bearing and friction. Next, each of the field data sets was preprocessed through the PDA using the selected damping factor to confirm or adjust the field results.

Pile Load Tests

Vertical Compression: Four vertical compression load tests were performed at the project site in general accordance with ASTM Designation D1143-81, "Standard Method of Testing Piles Under Static Axial Compression Load". The tests were performed between the dates of October 1, 1984 and October 16, 1984, under the observation of a representative of Woodward-Clyde Consultants. The standard loading and measuring procedures of the method were used.

At each of the test locations the pile driving contractor erected a reaction frame to apply the test loads to the piles. The reaction frame consisted of a system of steel I-beams anchored by reaction piles (a total of 4 reaction piles for each test pile). A hydraulic jacking system with a calibrated pressure gage was used to apply the axial compressive load to the tops of the test piles. Vertical movements of the pile tops were measured by either two or four dial gages fixed to a pair of reference beams positioned adjacent to two opposite sides of the test pile. The assembly and operation of the load test equipment were performed by Northwest Testing Labs, Inc.

In general, the test piles were loaded in increments of 10 and 20 tons for the 40 and 70 foot design lengths, respectively, (increments of 25 percent of design load) to 200 percent of the design load. These loads were sustained for a period of 12 hours. The piles then were unloaded in decrements of 20 and 40 tons for the 40 and 70 foot lengths (25 percent of the total applied load) and allowed to rebound under zero load for 12 hours. Subsequently, the piles were reloaded in increments consistent with the ASTM Method until rapid settlement (plunging) occurred or a maximum applied load of 200 tons was attained. Two hundred (200) tons was considered by the manufacturer to be the maximum load the pile could safely sustain without risk of compression failure. Each pile then was unloaded and a final rebound reading taken after 12 hours.

Uplift: Uplift tests also were performed on each of the vertical compression test piles. The uplift tests were conducted in general accordance with ASTM Designation 3689-78, "Standard Method for Testing Individual Piles Under Static Axial Tensile Load". The standard loading and measuring procedures of this method also were used.

For each of the uplift tests, the load was applied to the test pile by a hydraulic jacking system having a calibrated pressure gage and automatic load maintenance mechanism. A steel I-beam centered over the test pile and supported at its ends on beams bearing directly on the pad grade was used as the main element of the reaction system. Parallel steel threadbars coupled with two smaller I-beams then were used to provide a connection between the prestressing strands of the test pile and the hydraulic jacking system. Vertical movements of the pile top were measured by either two or four dial gages fixed to a pair of reference beams

positioned adjacent to two opposite sides of the test pile.

Test piles AF-1 and DF-3, each with embedment depths of about 41 feet, were loaded in eight increments of 7-1/2 tons to a total applied uplift load of 60 tons. The load then was sustained for a period of 24 hours. After the holding period, the load was removed in decrements of 15 tons and the pile was allowed to rebound under zero load for 12 hours. Test pile EF-2 revised (S27-EF2-7), which had an embedment of 49-1/2 feet, was loaded in the same general manner using increments of 9 tons to a total load of 72 tons, and load decrements of 18 tons. Test Pile BF3 (71 feet of embedment) was loaded in increments of 17-1/2 tons until one of the prestressing strands failed while the applied load was being increased from 70 to 87-1/2 tons.

DISCUSSION

Results of Pile Load Tests

Vertical Compression: The results of the vertical compression load tests are presented graphically on Figures 2 through 5. The short load test Piles AF-1 and DF-3 (N.5-9.5-AF1-5 and M26-DF3-5) each sustained twice the design load (80 tons total applied load) with a vertical movement of the pile top of about 0.1 inches. After the pile was unloaded and subsequently reloaded beyond the 80 ton load, rapid settlement occurred, indicating that the respective failure loads had been reached. Based upon the load deflection results presented on Figures 2 and 3, it is estimated that the ultimate capacities of Test Piles AF-1 and DF-3 were about 126 and 142 tons, respectively.

Test Piles BF-3 and EF-2 revised (B9-BF3-1 and S27-EF2-7), each of which was designed to carry axial loads of 80 tons, sustained twice that design loading (160 tons) with a vertical movement of the pile top of about 0.5 inches. Each of these piles subsequently was loaded to 200 tons without indication of failure (Figures 4 and 5).

Uplift: The results of uplift tests on the four load test piles also are presented graphically on Figures 2 and 5. Test Piles AF-1 and DF-3 each had embedment lengths of about 41 feet below grade at the time of the uplift tests. These piles sustained tension loads of 60 tons with vertical pile top movements between about 0.2 and 0.4 inches. Although rapid upward movement indicative of failure loading was not achieved, the straining which occurred during the sustained loading period and shape of the load-deformation plots (Figures 2 and 3) for these two piles suggest that the 60 ton applied load was near the ultimate uplift load capacity.

Test Pile EF-2 revised had an embedment length of about 49-1/2 feet at the time of uplift testing and sustained a total applied load of 72 tons without failure. However, the shape of its load-deflection plot (Figure 4) also suggests that the applied load was approaching the ultimate uplift load capacity. Test Pile BF-3, which had the deepest embedment of the piles tested at 71 feet, had a pile top movement of less than 0.2 inches when one of the prestressed strands failed while the applied load was being increased from 70 to 87-1/2 tons (Figure 5).

Results of Dynamic Monitoring

The results of the dynamic pile testing performed using the Pile Driving Analyzer are summarized in Tables 2 and 3. Table 2 contains the results of dynamic monitoring which was performed on piles driven with the ICE 520 diesel hammer; whereas, Table 3 presents the results of testing of piles driven with the ICE 640 diesel hammer. In order to easily compare the amounts of setup which were estimated to have developed between the initial driving condition and restrike, Table 4 has been prepared with a listing of only the estimated Case Method Capacities. The results of the CAPWAP/C analysis are summarized in Table 5.

Case Method Capacity Estimates: The pile capacities estimated during the end of initial driving (EOID) for the 40 and 45 foot pile lengths ranged from 0 to 85 tons; whereas, after waiting periods varying from 17 to 22 hours, the estimated capacities at the beginning of restrike (BOR) increased to between 80 and 115 tons. A second restrike (BO2R) was performed on Indicator Pile X-3 after an additional waiting period of 15 days and revealed a further setup gain of about 35 percent, from 115 tons (BOR) to 155 tons (BO2R).

The estimated capacities for the longer (70 to 75 foot) piles ranged from 65 to 135 tons at the end of initial drive and increased to between 130 and about 242 tons during restrike after waiting periods of typically about 14 hours to one day. The exception to the typical waiting period was Test Pile EF-2 revised, which was restruck after only a 4 hour waiting period. In several cases, relatively high blow counts (20 blows/inch or more) and low transferred energies (less than 7 kip-feet) were recorded during restrike. Considering these data, it is likely that the piles were not fully mobilized by the hammer during restrike. In addition,

only partial setup gain was likely to have occured within the restrike waiting period. For these reasons, it is expected that the Case Method capacity estimates summarized on Table 4 are somewhat conservative. This opinion is supported by the fact that the loads sustained by each of the static compression load test piles exceeded the corresponding Case Method capacity estimates.

Hammer Performance: It is estimated that the ICE 520 hammer transferred between 6.7 and 10.3 kip-feet of energy to the piles during initial driving and between 6.6 and 12.2 kip-feet during restrike testing using either 2 or 4 inch thick plywood cushion blocks. The average energies transferred during initial drive and restrike for this hammer were about 8.9 and 9.1 kip-feet, respectively. These average transferred energies correspond to a transfer efficiency of about 29 percent of the manufacturer's energy rating which is considered to be very good for a double-acting diesel hammer.

During initial driving with the ICE 640, it is estimated that an average of 7.9 kip-feet of energy was transferred to the piles when a 4 inch thick plywood cushion was used, and an average of about 12.4 kip-feet when a 2 inch thick plywood cushion was used. These averages correspond to transfer efficiencies of about 20 to 31 percent of the manufacturer's energy rating. During restrike, all dynamic measurements were taken while a 2-inch thick cushion was used in order to maximize the energy output. During restrike testing, the ICE 640 transferred an average of about 10.1 kip-feet to the piles. The general averages of transfer energy during installation and restrike correspond to transfer efficiencies of 24 and 26 percent of the rated energy which are considered to be average.

Pile Stress Levels and Damage: The maximum measured compressive stresses developed in the piles during driving with the ICE 520 hammer averaged about 2.9 ksi and did not exceed 3.8 ksi. During the installation of Test Pile EF-2 (V20-EF2-2), damage was detected and reported at about 50 feet below the gage location. The pile driving contractor elected to continue driving the pile and three additional breaks at 39, 33 and 16 feet below the gage location were detected and reported at the time of monitoring. At the completion of the installation, the top of the pile also was out of plumb and visibly damaged. The average stress levels recorded during the installation of this pile did not exceed 3.0 ksi in compression and 0.54 ksi in tension. Although these stresses represent dynamic levels and do not account for prestressing, it is unlikely that they would have caused pile damage providing that proper hammer-pile alignment was maintained. It is believed that Test Pile EF-2 probably had been damaged during handling prior to its installation.

When the ICE 640 hammer was used, the maximum measured compressive stresses varied between 1.7 and 4.0 ksi in all piles monitored and the maximum computed tensile stresses did not exceed 0.50 ksi.

CONCLUSIONS AND RECOMMENDATIONS

Revised Pile Capacity Estimates

Based upon the results of the compression load tests, dynamic monitoring and engineering judgment, it was concluded that the actual compression load capacities of the piles are somewhat higher than those which had been estimated based upon the original subsurface investigation program and predictive procedures published in the technical literature. Revised estimates of dead plus live load pile capacities versus embedment length are presented graphically on Figure 6.

Although the ultimate uplift capacities of the load test piles were not determined, the loads sustained were in excess of the values used for design. It is expected that the design uplift capacity estimates presented in the geotechnical report remain applicable.

Production Pile Order Lengths

The following lengths were recommended for the ordering and casting of piles to be driven for the balance of the production work:

Design Pile Capacity (tons)	Recommended Length (feet)
40	35
80	55

These recommended lengths are expected to be capable of resisting the design uplift loads.

Driving Criteria

In the event of hard driving in a dense sand stratum near the design embedment depth, it was recommended that the piles be stopped only when "refusal" driving of 200 blows/foot for 3 consecutive feet was encountered. In addition, it was recommended that the driving resistance necessary for termination should indicate a trend of increasing blow count of at least 20 blows/inch for the last 4 inches of penetration. These criteria were established for the ICE 640 hammer using a 4 inch thick plywood cushion block and were in addition to the project "Driving" specifications.

PRODUCTION PILE DRIVING

Geotechnical Consultation

Production pile driving was begun on October 23, 1984 and completed on December 19, 1984, under the observation of the resident engineering staff and pile driving inspectors of the City of Santa Clara. During the course of the production work, geotechnical review and consultation services were provided on an "on-call" basis when the City of Santa Clara's representatives perceived problem conditions existed or driving conditions were encountered which differed from those expected. As part of the consultation, opinions and recommendations were provided regarding damaged or broken piles, the use of other pile hammers, and the selection of pile lengths for pedestrian bridge connections to the convention center. Those topics are discussed below.

Damaged Piles: During production pile driving, several piles were spalled near their tops. Recommendations were developed in consultation with Mr. Shekhner and the City of Santa Clara's representatives to deepen the excavation for the pile caps at these locations such that the spalled portion would fully extend into the cap.

Replacement of Broken Pile: As discussed previously in this report, Test Pile EF-2 (V20-EF2-2) was broken during installation. This situation was reported to Mr. Mike Shekhner, Structural Engineer for the Project, and a replacement pile location was selected. Prior to the installation of the replacement pile and other piles of the group, the potential for driving into the broken pile was discussed with Mr. Guy Harrison of Riedel International and Mr. Ken Kratz of the City of Santa Clara. It was suggested that predrilling of

the pile be considered by the piling contractor to minimize the risk of damaging those piles. This approach was adopted and pile locations V20-EF2-1, 3, 4 and 8 were predrilled with a 12 inch diameter auger to depths of about 45 feet prior to pile installation. After the locations were found to be clear, piles in the cap were driven to the design embedment depths.

Other Pile Hammers: During the course of production pile driving, the contractor submitted requests to use Delmag D16-32 and D22-23 diesel hammers. Each of these single acting diesel hammers was approved for production pile driving following review of the manufacturer's specifications.

Piles for Pedestrian Bridge Connections: Near the end of the producion pile driving, three groups of piles for pedestrian bridge connections to the convention center were added for installation as a change order to the contract. Based upon our review of the pile group locations and design loads (60 tons for dead plus live load), we concluded that piles 55 feet long also would be applicable for these foundations. This recommendation was transmitted verbally to Mr. Bruce Augason of the City of Santa Clara. It is understood that these piles subsequently were driven.

Opinions and Conclusions

Information supplied by the City of Santa Clara's representatives indicates that the production pile installation generally progressed in the manner expected and consistent with our design recommendations. Our firm was not retained by the City of Santa Clara to review the installation conditions for each and every pile. Therefore, specific opinions cannot be provided regarding the capacity of every pile driven. However, if all piles were installed consistent with the

recommended lengths, and if the driving conditions were consistent with those observed by our representatives, then it is expected that the piles will be capable of supporting the design loads.

LIMITATIONS

The opinions, conclusions and recommendations presented in this report have been based upon the results of dynamic and static testing of piles at widely spaced locations across the project site, engineering analysis of the test results, and judgment. This report has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included.

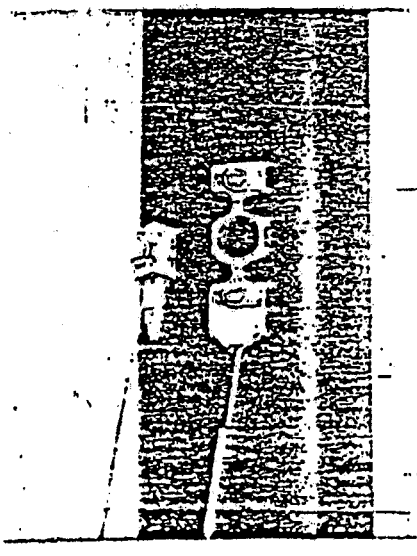
APPENDIX A

Description of Dynamic Monitoring Procedures

STRAIN TRANSDUCERS AND ACCELEROMETERS

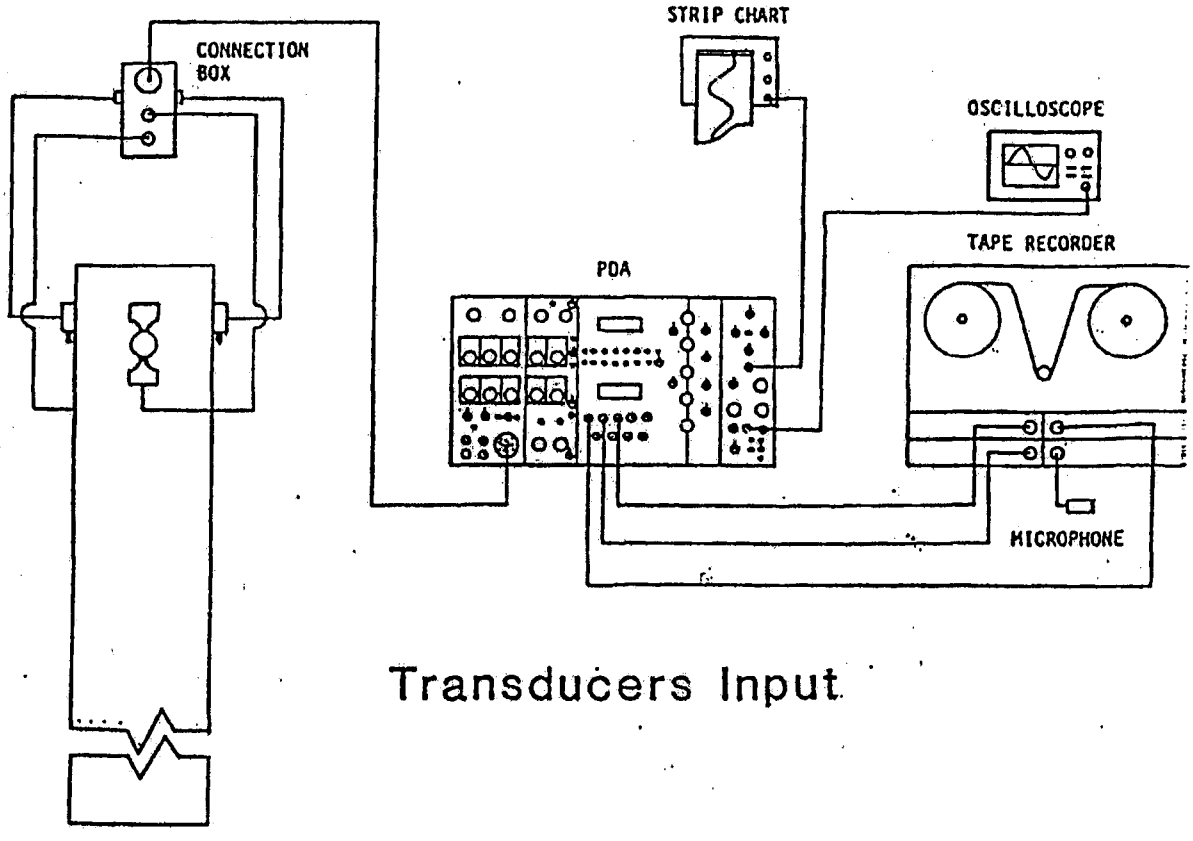
Strain Transducers: Strain Transducers are reusable frames to which foil gages are attached. They can be quickly attached to any type of pile under any kind of weather condition. Calibration of strain transducers is done by application of a known strain to the transducer and by measurement of its corresponding voltage output. Amplification is made such that a known shunt resistance produces a certain reference signal, a process that is repeated in the field. This method of calibration eliminates the need to determine the actual strain in the transducer and is independent of the strain gage type used. The known strain is usually applied through a pipe of known cross section under a calibrated load or through a calibration bar whose strain is monitored by resistance strain gages.

Accelerometers: Pile top motion is most easily measured using accelerometers. Most commonly used are the piezoelectric accelerometers because of their high natural frequence and ruggedness. These units consist of a quartz crystal which is subjected to a pressure of a mass when that mass is accelerated. The quartz crystal will produce a voltage linearly proportional to the pressure. In order to allow for an output under decelarations, the quartz crystal is prestressed by a spring which is the accelerometer housing itself. Piezoelectric transducers are dynamic instruments whose output tends towards zero under constant pressures (accelerations). A block is used for ease of bolting to a pile and for cable clamping.

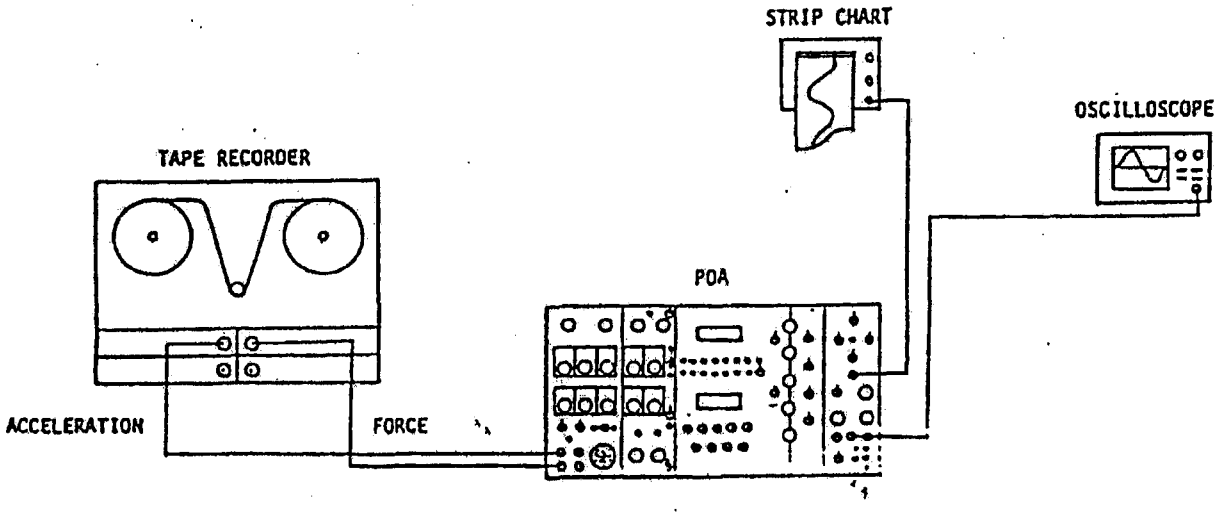


An accelerometer (left) and a strain transducer (right) bolted to a pipe pile

Schematic of Equipment Setup



Transducers Input



Tape Recorder Input

DETERMINATION OF PILE PROPERTIES

Dynamic measurements used to determine bearing resistance require that elastic properties, such as the modulus of elasticity, E, or stress wave speed, c, be known. For steel piles this is a simple matter since steel is a homogenious medium and its stress wave speed and elastic modulus are known to be 16,800 ft/sec (5120 m/sec) and 30,000 ksi (210 kn/mm^2). However, the elastic properties of concrete depend upon the amount of cement used, quality of aggregate, curing methods, and a number of other factors. This non-uniformity is also common among wooden piles, therefore, it becomes necessary to measure either the stress wave speed or elastic modulus.

The most convenient property to measure in the field is the speed of wave propagation. Since it is known that the elastic modulus is related to the stress wave speed through the specific mass, ρ, as:

$$E = \rho c^2$$

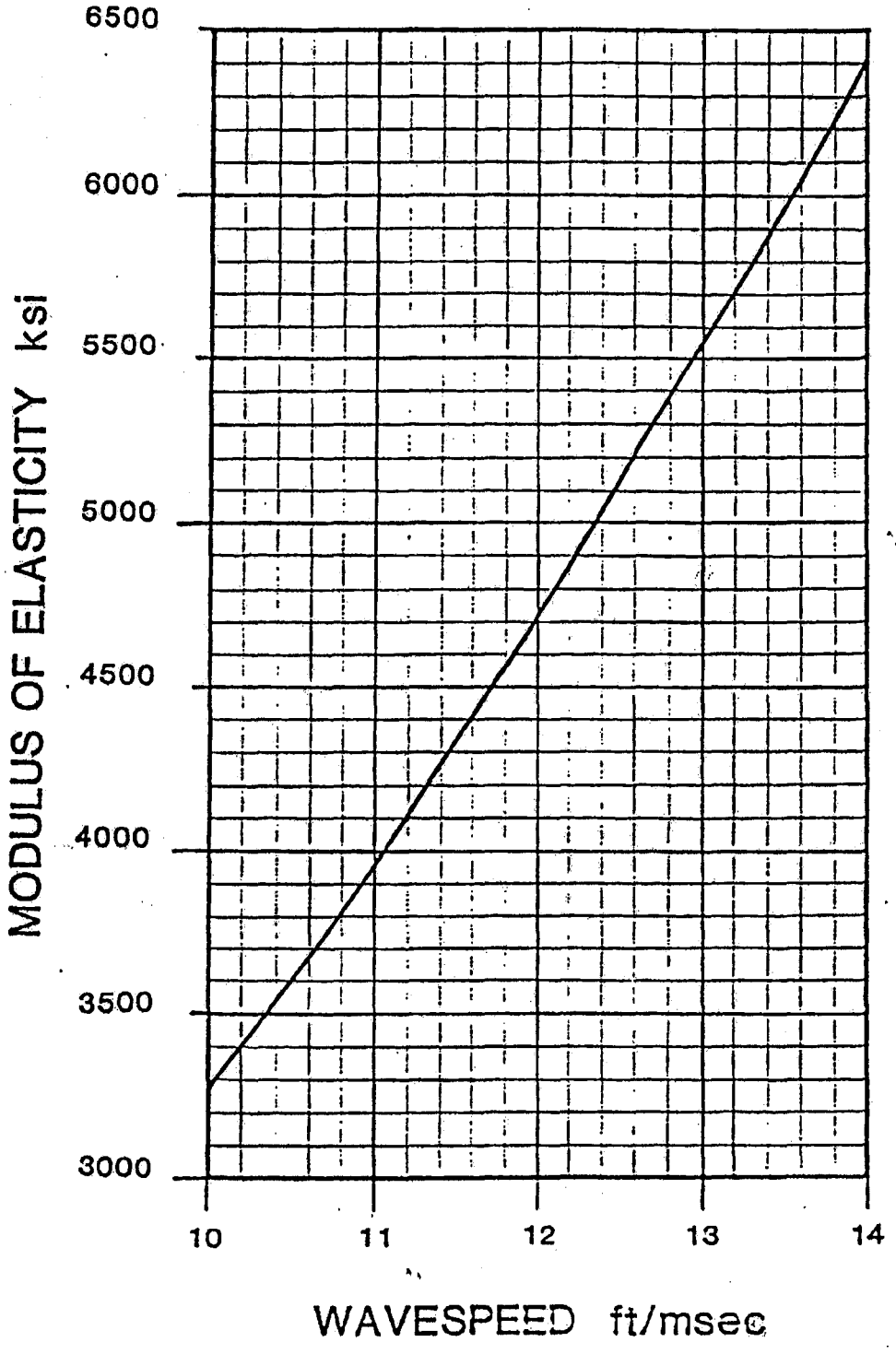
There are two procedures which are commonly used to determine wave speed. The first is measurement of longitudinal waves; the second is the measurement of transverse waves. In both cases a force is suddenly applied to the pile. Since the effects of this action are not transmitted instantly to all parts of the pile, the more remote sections remain undisturbed. A stressed "region" is generated which propagates through the length at a specific speed, c. When the stressed region reaches a free end a reflection occurs and the wave propagates back to the end where the force was applied. In most cases an accelerometer is placed near the end which is struck with a light hammer. An oscilloscope is used to monitor the accelerations and the first observed maximum peak is referred to an impact. Additional peaks occur each time the wave returns. Thus, knowing the pile length and time difference between peaks (as given by the oscilloscope), the stress wave speed becomes:

$$C = \frac{n2L}{\Delta t_n}$$

where "n" is the number of cycles corresponding to time, Δt_n. In general, the first peak is ignored since non-uniform impacts and slightly different cycle times are usually present. This is exactly the procedure used to estimate longitudinal waves.

Transverse measurements are usually performed when free piles are not available or, since concrete properties can vary along the length, to determine local properties. In this method, an accelerometer is attached perpendicular to one side of the pile and the opposite side is struck with the hammer. Since the traveled length in this case is actually the width of the pile, reflections occur at a high frequency. Tape recorders are therefore prohibited in this technique as they usually possess filters which dramatically alter these frequencies. Thus, it is best to use the latest noticeable reflection for wave speed calculations and count the number "n" of peaks after impact.

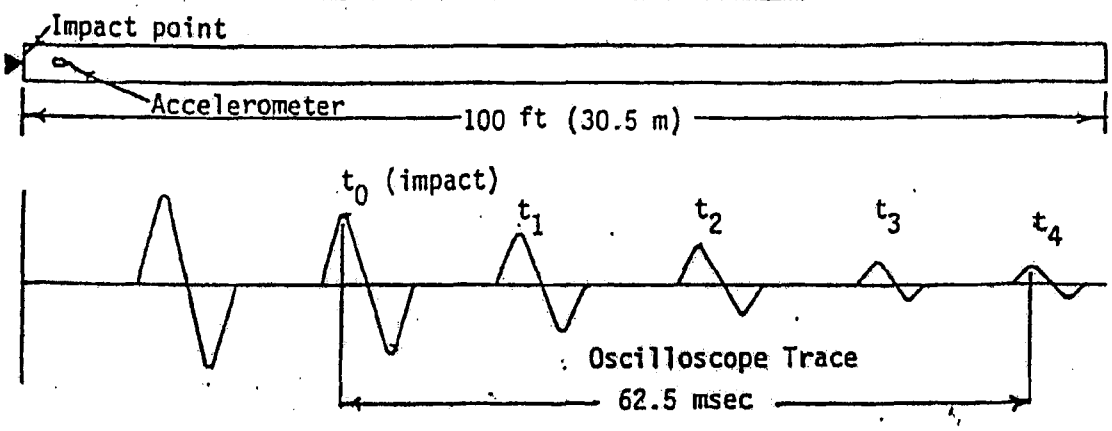
CONCRETE PILES



ASSUMED UNIT WEIGHT = 152 lb

DETERMINATION OF PILE PROPERTIES

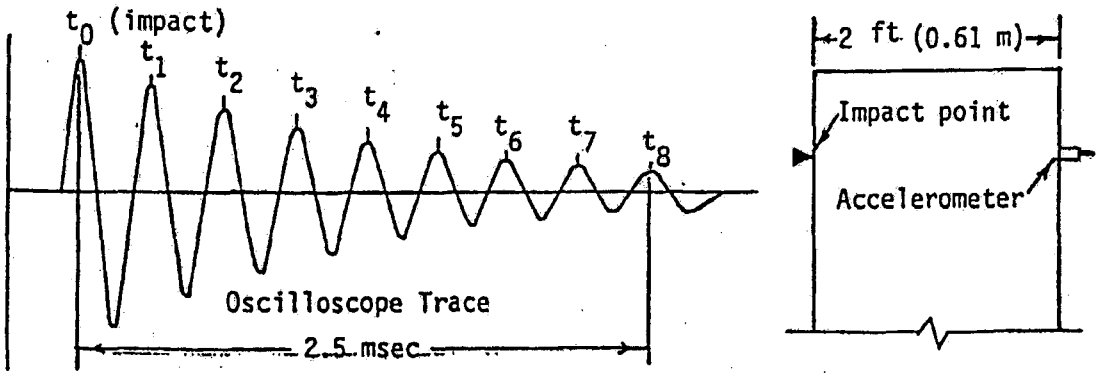
LONGITUDINAL MEASUREMENT METHOD



Note: If impact point is on same end as accelerometer, signal at t_0 should be ignored and timed cycles start with t_1.

n = 4
t = 62.5 msec
c = (4)(2)(100)/62.5 = 12.8 ft/msec (3.90 m/m

TRANSVERSE MEASUREMENT METHOD



n=8

Δt=2.5 msec

$$c = \frac{(8)(2)(2)}{(2.5)} = 12.8 \text{ ft/msec } (3.90 \text{ m/msec})$$

$\rho = 4.72 \text{ lb-s}^2/\text{ft}^4$

$$E = \rho c^2 = \frac{(4.72)(12800)^2}{(1000)\ (144)} = 5370 \text{ ksi} \quad (380 \text{ Mp/cm}^2)$$

WAVE MECHANICS

When a pile is suddenly loaded by a force, P, at one end, a stress wave is generated which begins to travel at a speed c down the pile. The wave speed, c, is a function of the material properties of the pile since it can be shown, using Newton's Second Law, that

$$c^2 = E/rho \qquad (1)$$

where E is Young's modulus and where rho is the mass density, both of the pile material. Using Hookes Law, the change of particle velocity, du', for a specific point on the pile can be shown to be related to the pile force, P, at that point.

$$du' = Pc/EA \qquad (2)$$

It is important to distinguish between the "Wave Speed" (speed of the compression or tension wave moving along a pile) from the "Particle Speed" (speed of a particle in the pile as a wave passes by).

Often velocity, u', instead of change of velocity, du', is used, but then proportionality holds if only one wave traveling in a given direction is present. The force-velocity proportionality constant is also called the "impedance". This term implies that the pile offers a resistance to (impedes) the change in velocity (the term Zu' has units of force). Note the following alternate forms of impedance.

$$Z = EA/c = rho(c)A = Mc/L \qquad (3)$$

The basic one dimensional wave equation is

$$rho(\partial^2 u/\partial t^2) = E(\partial^2 u/\partial x^2) \qquad (4)$$

and has a general solution for the pile particle displacement

$$u = g(x+ct) + f(x-ct) \qquad (5)$$

consisting of two components, g and f with time; the g and f waves merely shift positively and negatively without changing shape at a speed c. Within the downward input wave (f), there are compressive forces, causing proportional downward particle velocities (denoting the wave forces by F)

$$F_d = Zu'(down) \qquad (6)$$

With no soil resistance, as the wave arrives at the pile tip it has no mass to accelerate and a reflection occurs. In fact, because the pile tip is free, the tip force must be zero, and due to force equilibrium an equivalent upward wave in tension is generated which pulls the pile particles downward. We can state that compressive wave particle velocities have the same direction as the wave propagation while for tension waves, particle velocities and wave propagation have the opposite sign (velocity is positive downward and compression is a positive force). Thus, for upward traveling waves

$$F_u = -Zu'(up) \tag{7}$$

The total force, P, and velocity, u', measured at any location in a pile is the result of superposition of all downward and upward traveling waves

$$\begin{aligned} P &= F_d + F_u \\ u' &= u'_d + u'_u \end{aligned} \tag{8}$$

Multiplying the velocities by the impedance Z, the forces in the upward and downward waves can be obtained from these two simultaneous equations

$$\begin{aligned} F_d &= (P+Zv)/2 \\ F_u &= (P-Zv)/2 \end{aligned} \tag{9}$$

CAPACITY

If a resistance force starts to act at time $t = x/c$ at some intermediate point, x, along the pile (caused by an impact at time $t = 0$ at the pile top), then two waves are created, each having a magnitude of $Rx/2$. To satisfy equilibrium and continuity, the upward wave is in compression and the downward wave in tension. The upward compressive resistance wave reaches the top at time $t = 2x/c$. The tensile resistance wave reaches first the pile bottom at time $t = L/c$ where it is reflected in compression. It then travels upward to the top where it arrives at time $t = 2L/c$.

If a resistance force, R_b, starts to act at time $t = L/c$ at the pile bottom, then it will create a compressive upward traveling wave of magnitude R_b, which arrives at the pile top at time $t = 2L/c$.

If all resistance forces act constant throughout the time $x/c<t<2(L-x)/c$, then at time $2L/c$ the force and velocity records contain the effects of

(1) the upward traveling tension wave due to reflection at the pile bottom of the initial downward moving compression input at a time $2L/c$ earlier, $-F_d(t_1)$

(2) the summation of all upward traveling compression resistance waves $(R_x/2)$

(3) the initially downward traveling tension resistance waves now traveling upward in compression after reflection at the bottom $(R_x/2)$ and the upward wave from the tip resistance (R_b), both arriving at the pile top together with (1)

(4) all downward traveling waves, $F_d(t_2)$.

Wave (2) and wave (3) have a total magnitude, R $(R = R_x + R_b)$ since they contain both half waves of skin friction and the full end bearing. Thus, the combination of all upward traveling waves contains the resistance and the bottom reflected (negative) impact wave of time t_1.

$$(P_2 - Zu'_2)/2 = -(P_1+Zu'_1)/2 + R \qquad (10)$$

Rearranging, we can now solve for the total resistance

$$R = (P_1+Zu'_1+P_2-Zu'_2)/2 \qquad (11)$$

where the indices 1 and 2 refer to times t_1 and $t_2 = t_1 + 2L/c$.

R is the total resistance encountered during a complete passage of the wave (during a time period of 2L/c). There are differences between this resistance and the ultimate static capacity of the pile and various considerations are necessary to predict R_s.

(1) Elimination of soil damping

(2) Proper choice of time t_1 such that R is already at full magnitude when P and u' samples are taken

(3) Correction for an R_s that decreases during 2L/c because of early pile rebound (negative velocity before 2L/c)

(4) Time dependent soil strength changes (setup or relaxation). Since the dynamic methods give the resistance at the time of testing, it is always recommended to test piles at the end of driving for the strength of remolded soil, or by restrike after a wait period for the long-term service load, and at both times to determine strength changes. It should not be surprising that the capacity at the end of driving may not be equal to the service capacity after a wait due to reconsolidation, dissipation of excess pore pressures, etc. Static test correlations should always be made with restrike data.

(5) The pile must experience permanent set during the testing. If no° (or very little) movement is achieved then the indicated capacity relates to the mobilized value only, roughly analogous to a static proof test not run to failure but rather still in the elastic range.

Considerations 4 and 5 above are self explanatory. The first three considerations will now be investigated in more detail.

Damping is associated with velocity. We can obtain the tip velocity from our top measurements as

$$u'_b(t) = F_d(t-L/c)/Z - F_u(t+L/c)/Z = (P_1 + Zu'_1 - R)/Z \qquad (12)$$

By defining the damping force $R_d = J_c Z u'_b$ (J_c is a dimensionless damping constant), we can also solve for the damping. Since the total resistance is the sum of the static and damping forces, the static resistance can be obtained from

$$R_s(t) = R(t) - R_d(t) = R - J_c(P_1-Zu'_1-R) \qquad (13)$$

or expanding into terms of only P and u',

$$R_s = (1-J_c)[P_1+Zu'_1]/2 + (1+J_c)[P_2-Zu'_2]/2 \qquad (14)$$

The damping constant relates to the soil grain size near the pile tip or can be computed directly from this R_s equation if measurements and the failure load (from static test or total static resistance from CAPWAP) are known, since J_c is the only unknown in the equation.

Resistance mobilized is also a function of displacement. The usual assumptions of elasto-plastic resistance are valid and have a resistance which increases linearly to a maximum at some specified displacement (termed the "quake") and then remains constant (plastic) until the displacement decreases. Typical quakes are 0.1 inch (2.5 mm), although values up to 1.0 inch (25mm) have been observed.

For each time t, a resistance R may be determined. Usually, the time of the first major velocity peak is selected for time t_1. In most cases, the integral of the velocity (i.e. displacement) at the first arrival of the peak input at any point along the pile is larger than the soil quake, assuring that the full resistance is mobilized. However, it may be necessary to delay this time to get to a (a) second major peak or (b) wait until a maximum resistance, Rmax, is found. Item (a) is always applicable if a second major velocity peak is still at a time where force and velocity are proportional. Item (b) is applicable if a large quake soil condition exists, i.e., if it takes a considerable compression of the soil before its ultimate capacity is reached. Large quakes are most often observed for displacement piles with large diameters or in saturated soils. Item (b) may also be necessary if the velocity integral is small (low input or sharp rise time) at the initial peak.

If the expression for R_d (damping forces) is set to zero, this implies that the pile tip velocity is zero and any resistance which is present at this time is static and therefore independent of a damping constant. This solution occurs when graphically the resistance versus time curves for R(t) and $R_s(t)$ are for the first time equal. Since this equation assumes resistance to be at the pile tip, it is generally applicable when the resistance is primarily end bearing. This is the basis for the R_{auto} Method. For piles with little skin friction, the pile toe force, velocity and displacement can be computed directly from the pile top measurements and one dimensional wave theory.

$$P_b(t) = F_d(t-L/c) + F_u(t+L/c) \qquad (15a)$$

$$u'_b = [F_d(t-L/c) - F_u(t+l/c)]/Z \qquad (15b)$$

$$u_b = \int u'_b dt \qquad (15c)$$

A static toe resistance force-displacement graph may be obtained by reducing P_{toe} by the damping JZu'_b and plotting this force for each time increment against the displacement of the toe (integral of equation 15b). This is the PEBWAP procedure.

The Case Method of capacity prediction "measures" the resistance (capacity) acting simultaneously. For long piles having a significant portion of resistance coming from shaft friction, the Case Method may underpredict

during hard driving, i.e., when the pile top velocity becomes negative before time 2L/c; the pile top is moving upward and some skin friction begins to unload. The basic Case Method can be "corrected" for this situation by adding the resistance in this upper portion of the pile that has unloaded. The dynamic component is then subtracted.

STRESSES

Pile damage is usually the result of either poor hammer alignment (high local contact stresses) or high driving stresses. For concrete piles, tension stresses are important. From the upward wave, we can easily investigate whether tension is present. The input compression stress wave will be reflected as an upward tension wave from the pile bottom at time L/c and will arrive at time 2L/c. This upward tension force has been transmitted along the entire pile shaft, but is not necessarily the net tension at any location since downward waves exist. The maximum net tension (CTN) occurs when the downward compression stress is a minimum (time force) and can be found mathematically by

$$CTN = F_u(t_2=2L/c) + F_d(t_3<2L/c)_{min} \leq 0 \qquad (16)$$

High compression stresses can also be of concern. In general, the maximum compression force at the measuring location is very close to the maximum force in the pile (absolute upper limit is the total resistance R if it is greater than the maximum pile top force).

DAMAGE DETECTION

For a uniform pile, an upward traveling tension wave should be observed only after reflection from the pile tip and should therefore come at time 2L/c. If an upward tension wave is observed prior to 2L/c, it must be due to a change in impedance (reduced section area, modulus, or possible damage). Consider the equilibrium conditions for downward $F_{d,1}$ and $F_{d,2}$ and upward $F_{u,1}$ and $F_{u,2}$ waves at a cross section change with impedance Z_1 Z_2, respectively.

$$F_{d,1} + F_{u,1} = F_{d,2} + F_{u,2} \qquad (17)$$

Requiring velocity continuity, and solving for the input wave reflection ($F_{u,2}$ will be zero), we obtain for the relative cross sectional change BETA $= Z_2/Z_1$

$$BETA = [F_{d,1}+F_{u,1}]/[F_{d,1}-F_{u,1}] \qquad (18)$$

The force $F_{d,1}$ at any location x can be found from the superposition of the initial downward wave with the downward resistance tension waves.

$$F_{d,1} = F_d(t_1) - R_x/2 \qquad (19)$$

The upward wave at time $2x/c = t_4$ is the sum of the resistance effects above location x and the cross section change effect (negative if $Z_2<Z_1$)

$$F_u(t_4) = R_x/2 + F_{u,1} \qquad (20)$$

$$(P_2 - Zu'_2)/2 = -(P_1+Zu'_1)/2 + R \qquad (10)$$

Rearranging, we can now solve for the total resistance

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We can then solve

$$BETA = [(F(down,t_1) - R_x + F(up,t_4)]/[F(down,t_1) - F(up,t_4)] \quad (21)$$

where t_4 is the time of a local minimum in F_u after a peak $R_x/2$ compression increase. For a uniform pile, $F_u(t_4)$ will be a monotonic increasing function equal to $R_x/2$ and BETA will then be equal to 1.0. If a uniform pile should indicate a BETA less than 1.0 prior to 2L/c, the pile is damaged at location $X = ct_4/2$ and the cross section reduction can be calculated. The following classification scale has been proposed:

BETA =	1.0	uniform
	0.8-1.0	slight damage
	0.6-0.8	damage
	below 0.6	broken

HAMMER PERFORMANCE

The energy in the pile can be found from the work done on the pile which we can obtain if we integrate the product of force P and velocity u' over time.

$$W = \int P du = \int P\ u'\ dt \quad (22)$$

The maximum value is the maximum transferred energy EMX. It is important to realize that only this transferred value EMX is capable of actually doing work on pile and soil, rather than the hammer's rated energy.

For air steam or drop hammers with ram mass, m_r, the principles of impulse and momentum can be used to obtain the maximum ram velocity, u'_r, prior to impact from

$$MFO = \int_0^{u'=0} P(t)dt = M_r\ u'_r \quad (23)$$

From this the ram kinetic energy, KE, may be calculated and compared with the ram potential energy, PE, to obtain hammer efficiency. Comparing the kinetic energy with the maximum transferred energy EMX will demonstrate the effectiveness of the driving system (capblock, helmet, cushion).

Three energy ratios of importance are:

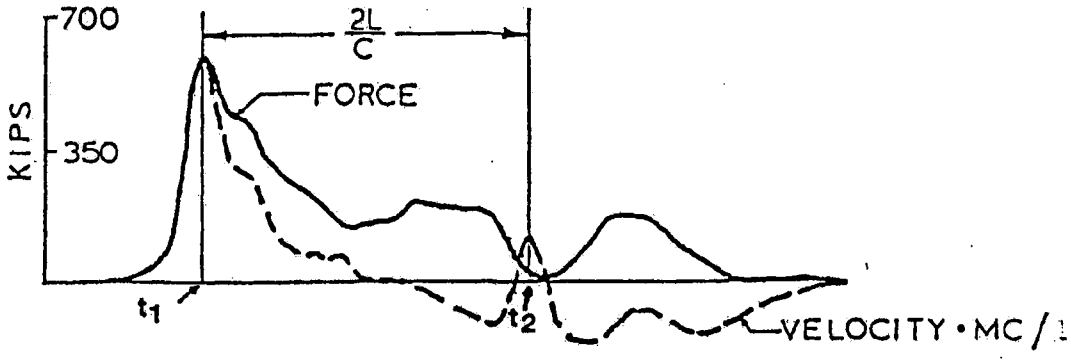
$$e_h = \text{hammer efficiency} = KE/PE$$

$$e_d = \text{drive system efficiency} = EMX/KE \quad (24)$$

$$e_t = \text{total transfer ratio or efficiency} = EMX/PE$$

For diesel hammers, the maximum kinetic energy is not as meaningful due to the compression of the gasses just prior to impact. This is especially true in cases with preignition.

CASE METHOD – EXAMPLE



$F1 = F(t_1) = 601$ kips $\qquad V1 = Mc/L\ [V(t_1)] = 589$ kips

$F2 = F(t_2) = 17$ kips $\qquad V2 = Mc/L\ [V(t_2)] = 111$ kips

A) Total Capacity, RTL = 1/2 (F1 + V1) + 1/2 (F2 - V2)

= 1/2 (601 + 589) + 1/2 (17 - 111)

= 548 kips

Total Capacity = Static Resistance + Dynamic Resistance

RTL = RSP + RDN

B) RDN = J (F1 + V1 - RTL) for sand J = 0.1

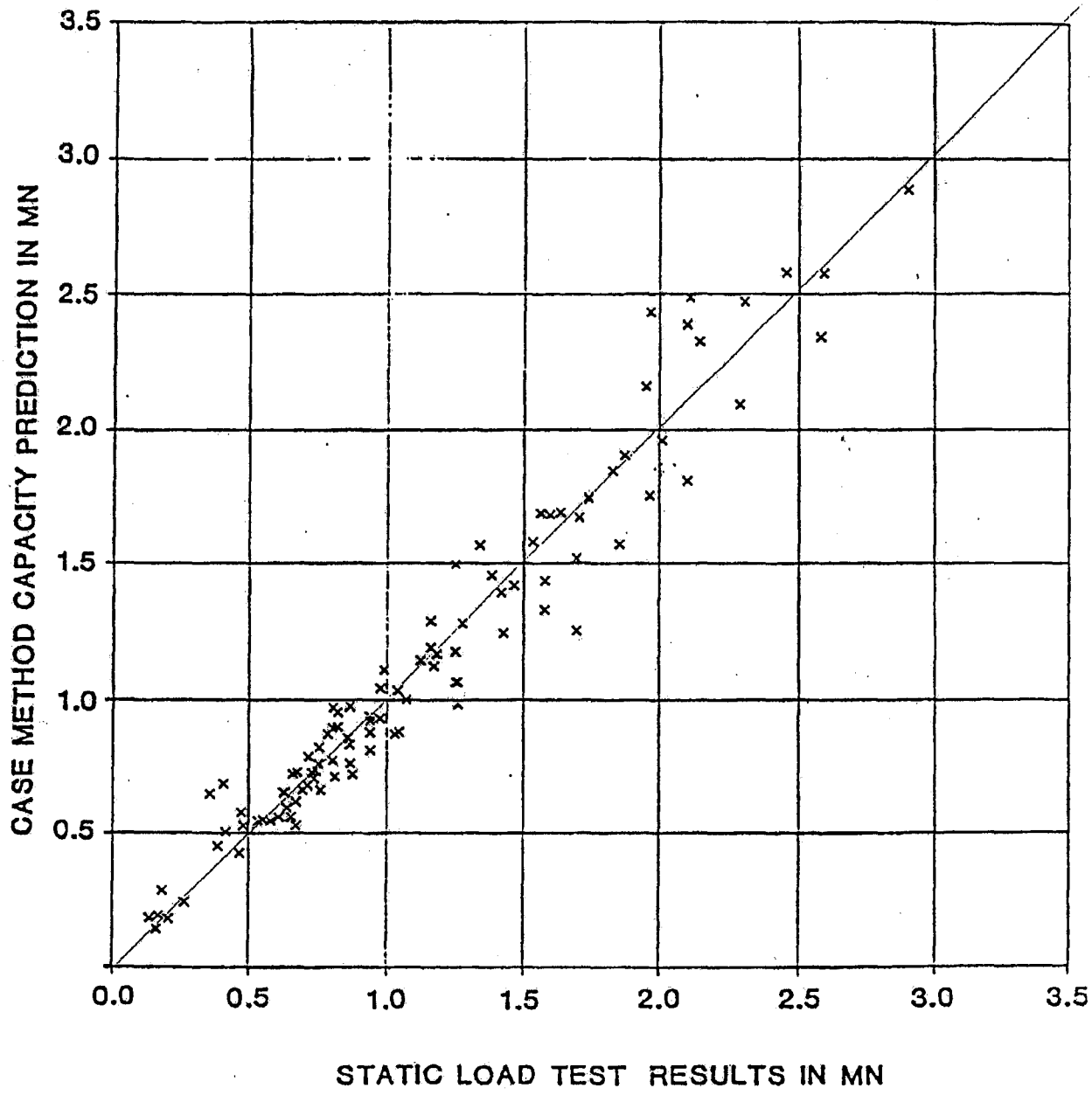
= 0.1 (601 + 589 - 548)

= 64.2 kips

C) RSP = RTL - RDN

= 548 - 64.2 = 483.8 kips

By comparison, a static test was run which failed at 470 kips.



1 MN = 112 TONS

CASE METHOD CORRELATION

DATA PROCESSING

Goble Rausche Likins and Associates, Inc. (GRL) has provisions for additional analyses. The equipment used contains both mini or micro computers with disk storage. Further hardware of the system include an optional analog to digital converter (A/D), including multiplexer, printer and plotter. A schematic of the data processing system is included. The standard pile data processing is as follows:

1) The tape recorder data is replayed into the Pile Driving Analyzer where the force and velocity are digitized at 10,000 Hz each. The data is then sent by an RS 232 port either directly into the computer or through telephone communications (modem) from remote job sites.

2) Optional provisions to replay tape recorded velocity and force through a multiplexer and A/D directly into a computer are provided. A third quantity like another force record, hammer pressure, or other dynamic quantities may also be digitized.

3) Store the digitized data on disk.

4) Integrate velocity to obtain pile displacement, differentiate velocity to obtain acceleration.

5) Check final displacement and compute an acceleratoin shift for agreement of final displacement with blow count; alternatively shift acceleration for a zero velocity at end of record or do no correction at all.

6) Reintegrate acceleration including correction from 5 and plot the result, the pile velocity, as a function of time together with the force record. Integrate the velocity to obtain displacement.

7) Integrate the product of velocity and force to obtain energy.

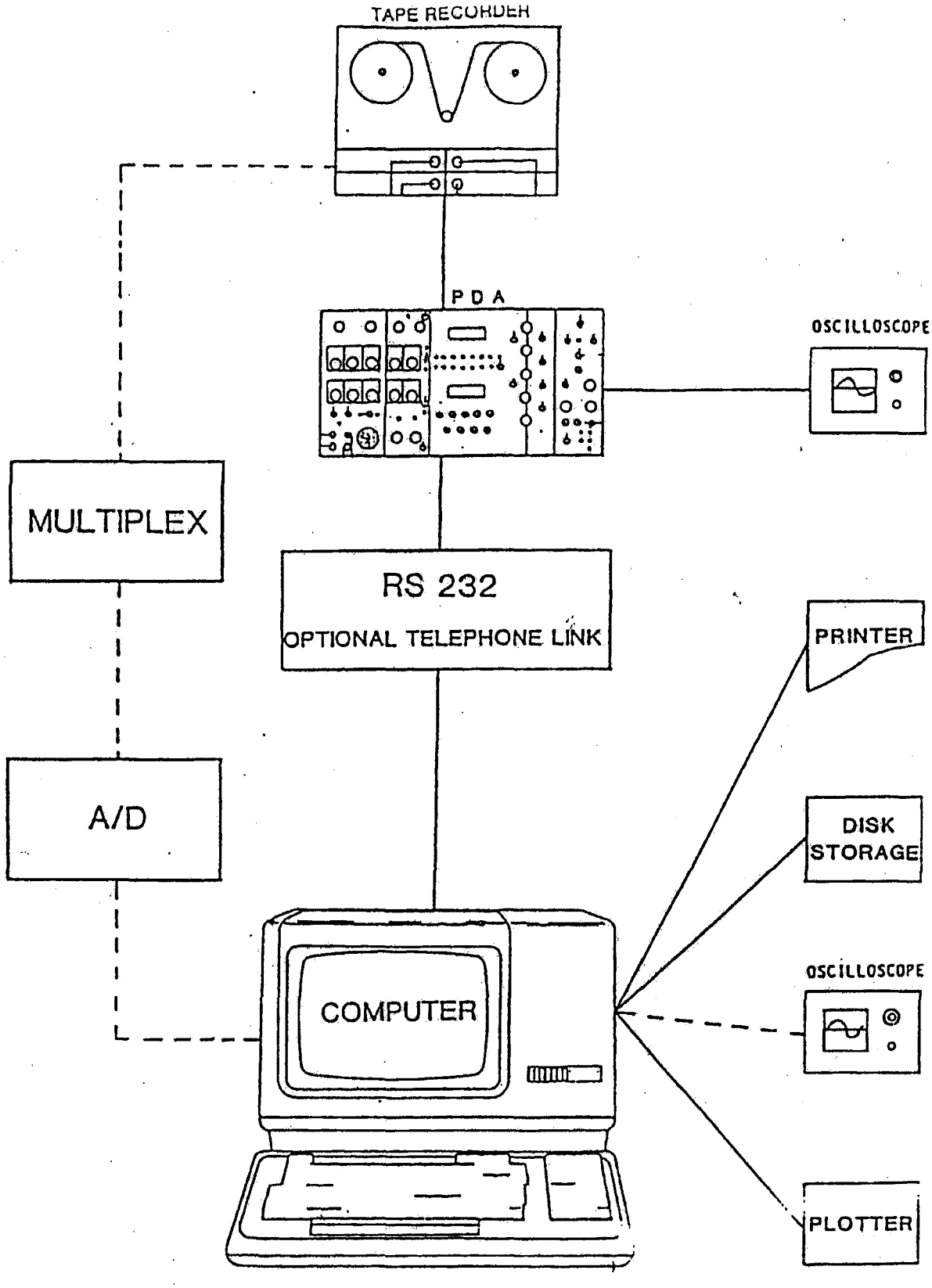
8) Determine the maxima of force, velocity, energy, acceleration and displacement.

9) Determine capacity from Case-Goble Method for different J-values as a function of time.

10) Print results.

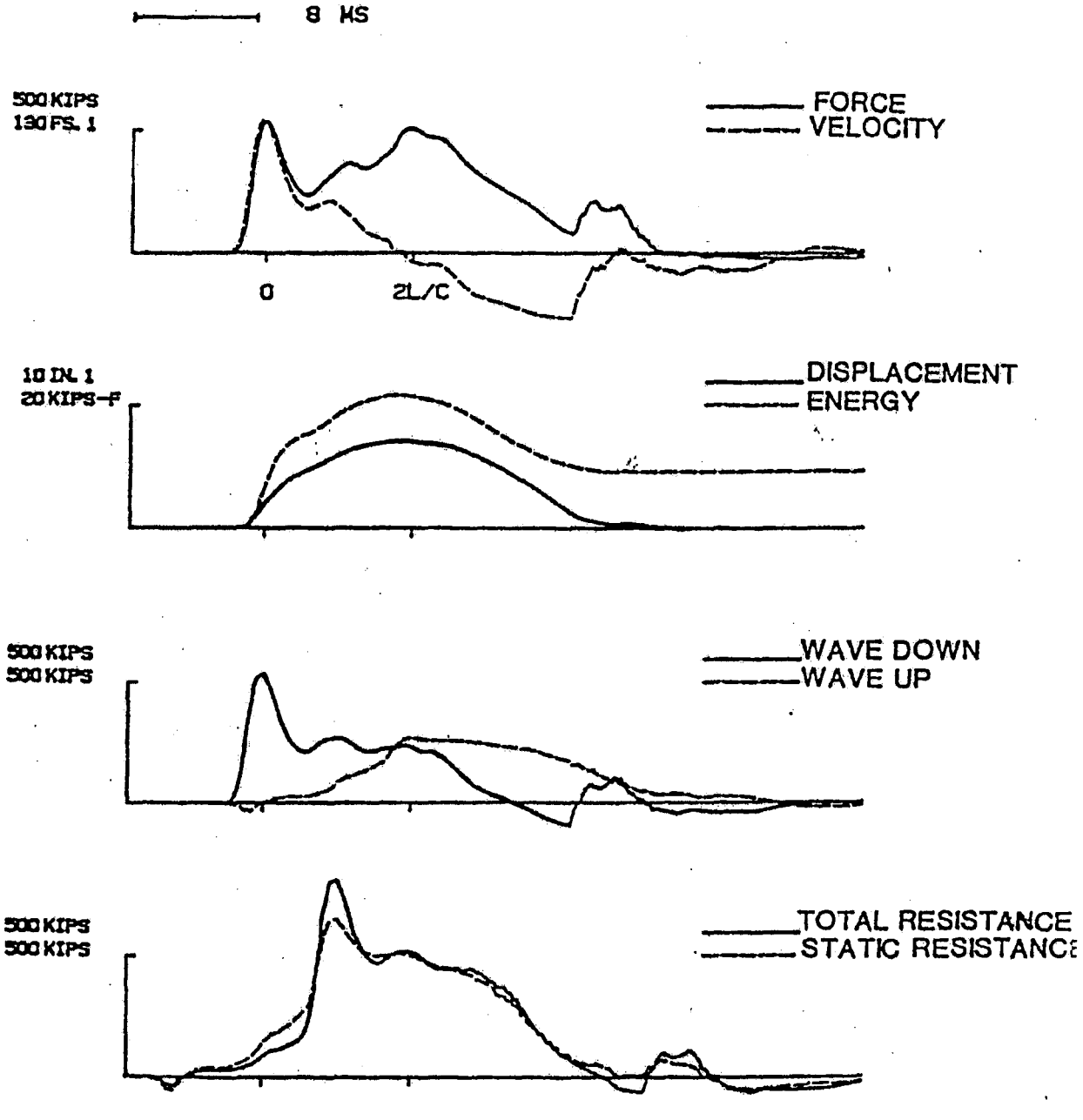
11) For piles with little skin friction, compute and plot pile tip force displacement relationship (formerly the Pile End Bearing Wave Analyses Program, PEBWAP).

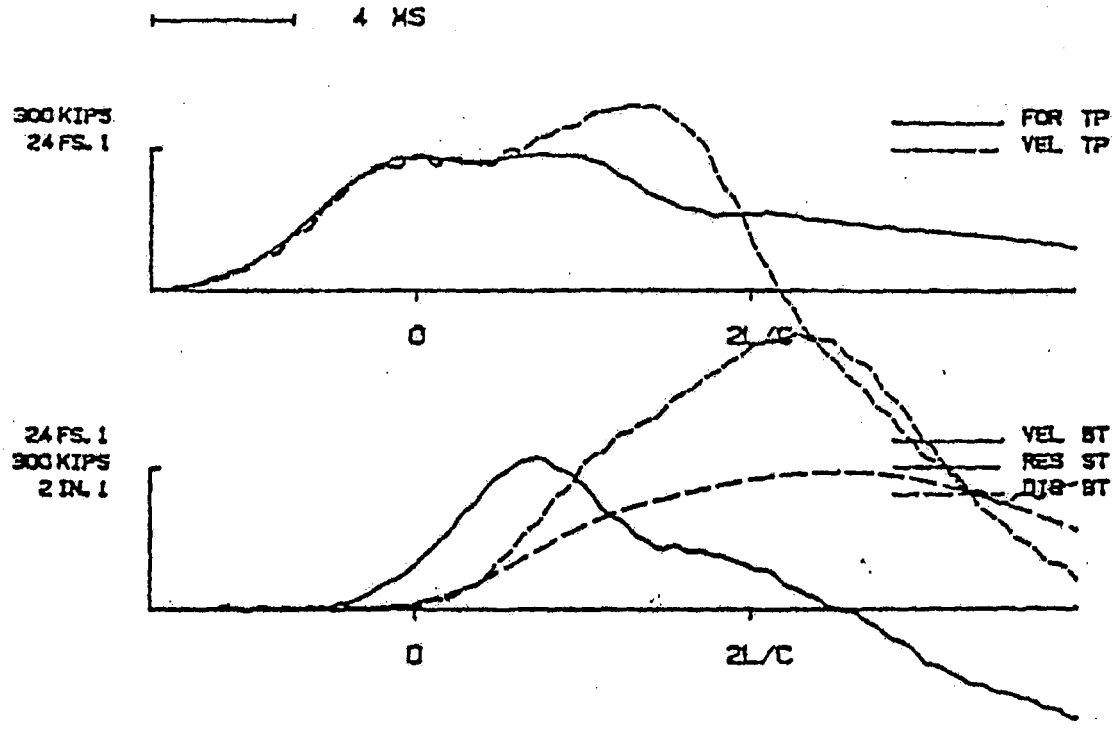
Of course, variations of these procedures are possible. Great flexibility of the special purpose software is incorporated. Various types of plots and output tables can be made.

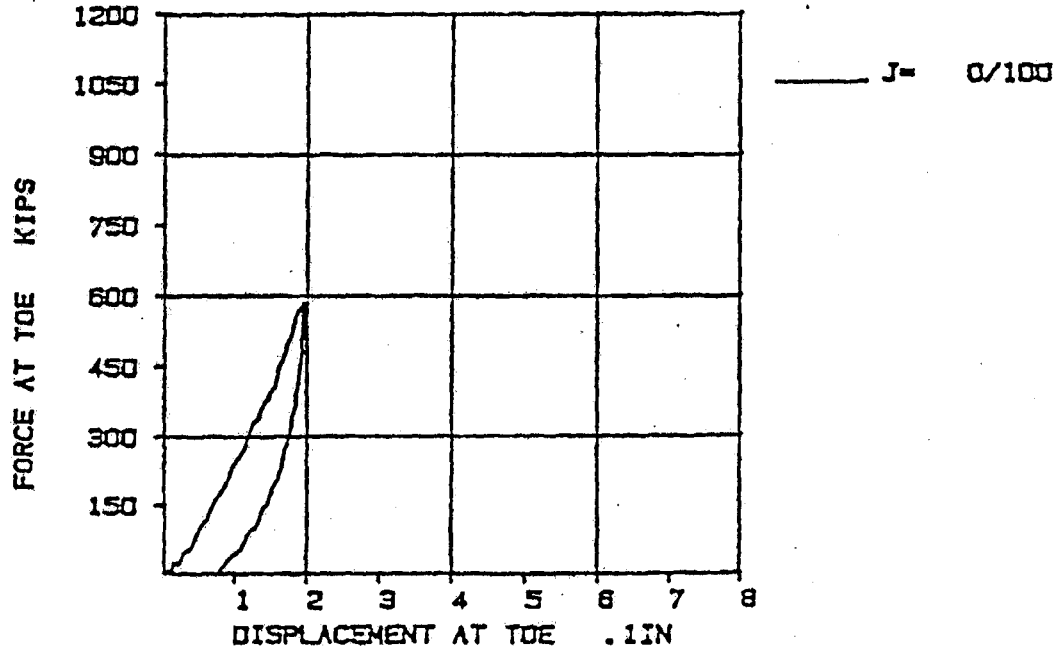


Schematic of Data Processing

Data Processing Results




4 MS
300 KIPS
24 FS. 1
FOR TP
VEL TP
0
2L/C
24 FS. 1
300 KIPS
2 IN. 1
VEL ST
RES ST
DIS ST
0
2L/C


1200
1050
900
750
600
450
300
150
FORCE AT TOE KIPS
J= 0/100
1
2
3
4
5
6
7
8
DISPLACEMENT AT TOE .1IN

The CAPWAP Method (CAse Pile Wave Analysis Program)

Either pile top force or pile top velocity can be used in a dynamic analysis as a boundary value (both together would not lead to satisfactory results). An analysis can then be performed either in closed form or in a so-called wave analysis procedure, i.e., in a discrete form. Of course it is then necessary to describe the soil resistance forces.

The soil reaction forces are passive and up to now it has been found sufficiently accurate to express them as a function of pile motion only. It is furthermore assumed that the soil reaction consists of a static (elasto-plastic) and a dynamic (linear damping) component. In this way the soil model has at each point three unknowns (elasticity, plasticity and viscosity).

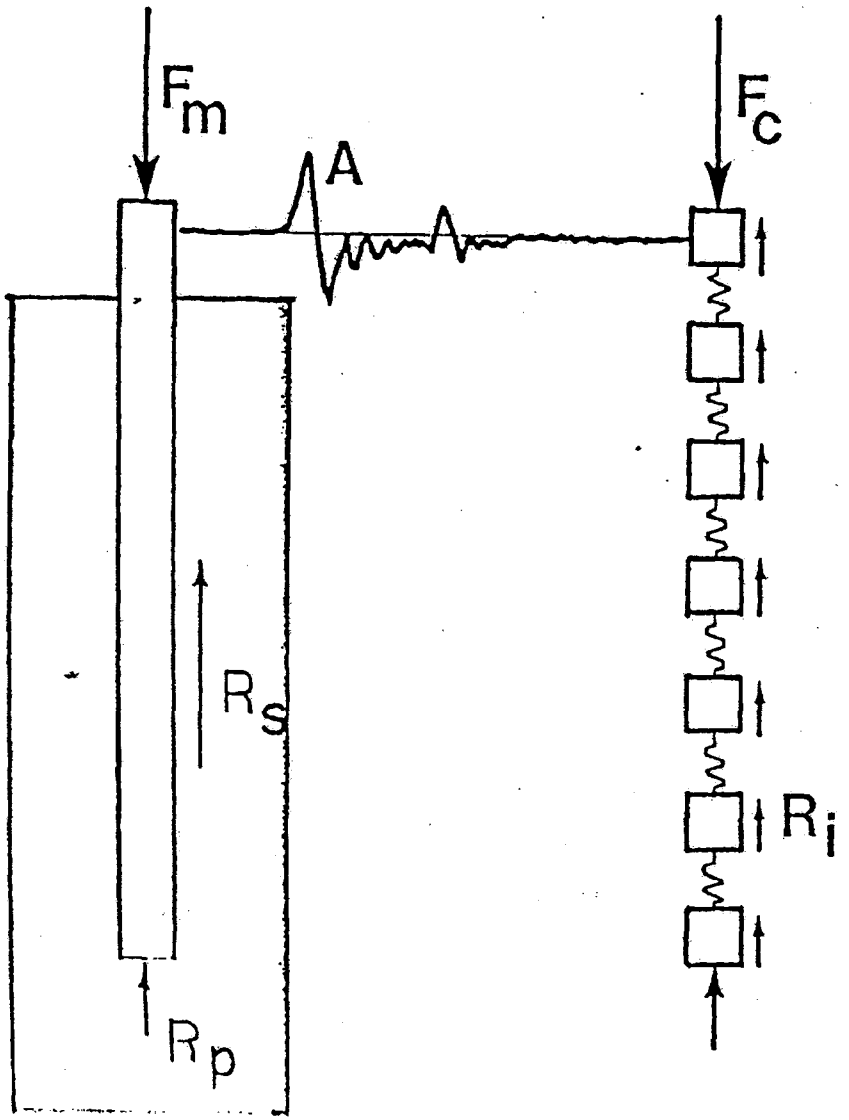
The dynamic analysis is performed in the CAPWAP Method after the procedure that was introduced by Smith. This procedure divides the pile in a number of mass points and springs. In this way there are three times as many unknown soil parameters as pile elements. First, a reasonable assumption is made regarding the soil parameters, and then the motion of the pile is assumed using the measured pile top acceleration as a boundary value. Output results are not only the pile element motions and soil resistance forces, but also the computed pile top force, all as a function of time.

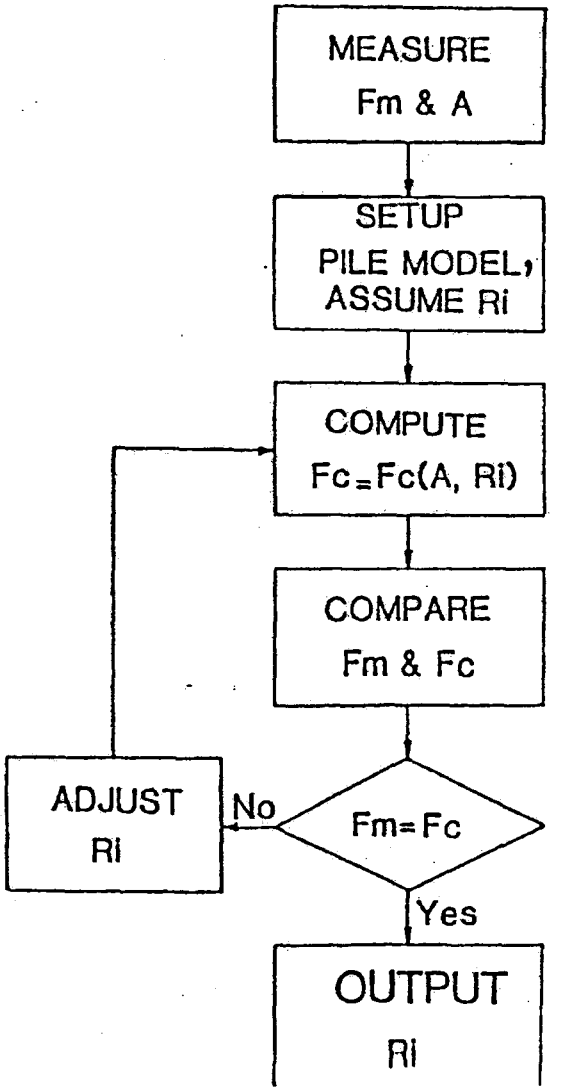
The computed and the measured pile top force will in general not agree with each other. It is necessary to improve this match iteratively by changing the assumed soil resistance parameters. Finally, a computed pile top force will be obtained which cannot be further improved. The corresponding parameters of the soil model are then considered the correct values. The results of the CAPWAP analysis then are the magnitude and location along the pile of both static and dynamic resistance forces. Static computations can be used to predict the static load test curve of the pile.

In 1970 a program was written that performed the necessary computations and decisions automatically. This program resulted in satisfactory solutions for piles which were not more than 75 feet in length. For longer piles computation times became excessive. A recent program performs the computations "interactively". In the interactive mode one analysis is obtained using a minicomputer, and then the engineer determines the necessary changes of soil parameters for the next analysis. This method uses a machine with approximately 16k core memory. Of course, one also needs a plotter to draw the measured and the predicted pile top force curves. Even for longer piles it is usually sufficient to analyze 10 to 20 times.

A reverse analysis, called WAPCAP, is often done as a check on the CAPWAP results. WAPCAP uses the measured pile top force as an analysis input and produces a match of computed with acceleration derived velocity. WAPCAP gives also very reliable force predictions in the pile at locations other than the top.

THE CAPWAP METHOD





CAPWAP/C
(Capwap Analysis using the Continuous Pile Segments)

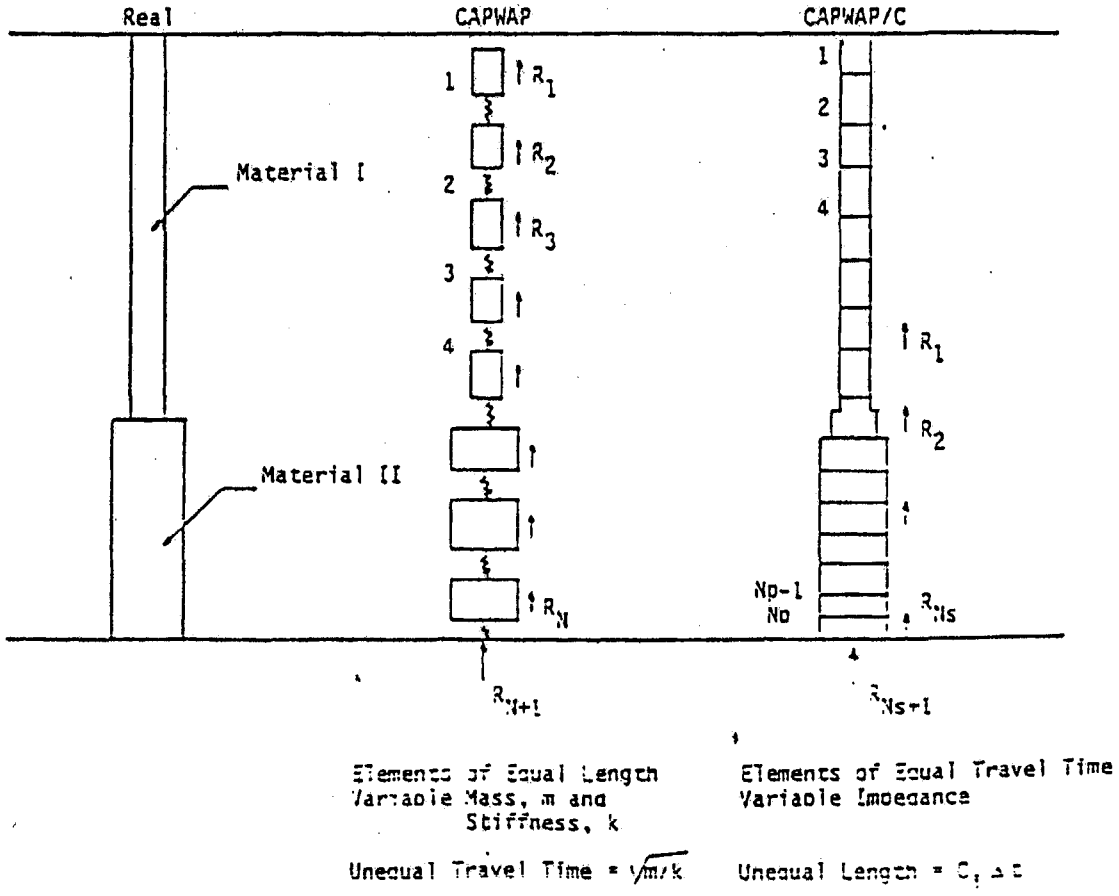
CAPWAP/C is a program that in general works like CAPWAP, except that it uses the characteristics method rather than the lumped mass approach for analysis. The characteristics method divides the pile into Np segments which are of uniform cross-section. Each element, i, has a length, dti, equals the analysis time increments, dt. Thus for variable pile properties Ei, Wi (elastic modulus, specific weight), the wave speed of a segment is

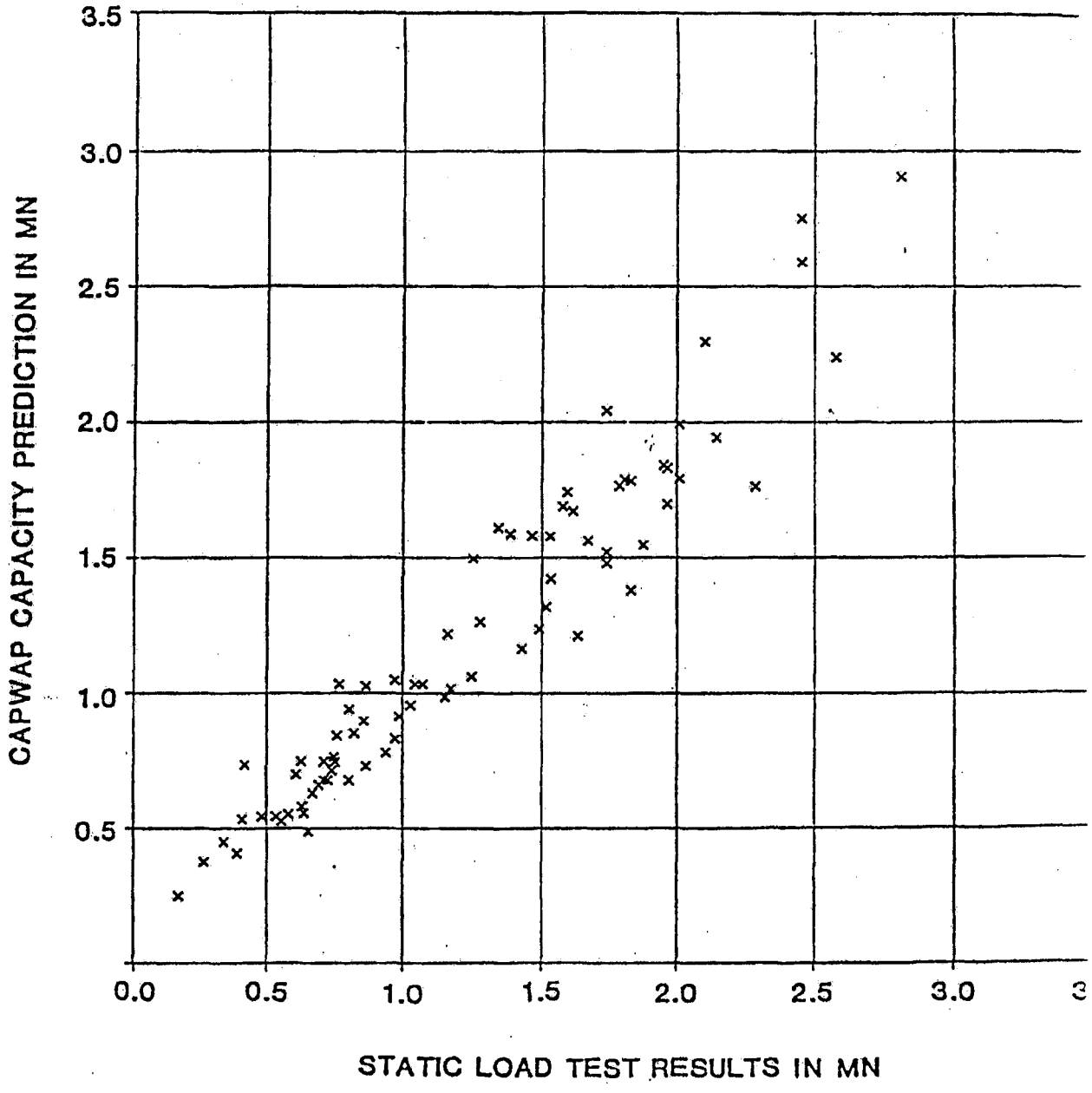
$$c_i = (E_i \quad g/W_i)^{1/2}$$

where Ci, Ei and Wi, may be average properties over a segments length if the properties change within the corresponding length increment, dLi, and g is the earth gravitational constant.

$$dL_i = (dt)\, c_i$$

Note that the segments are not of equal length. Resistance forces Rk may act at the bottom of any segment. They are the sum of the usual elasto-plastic and linearly viscous resistance values.





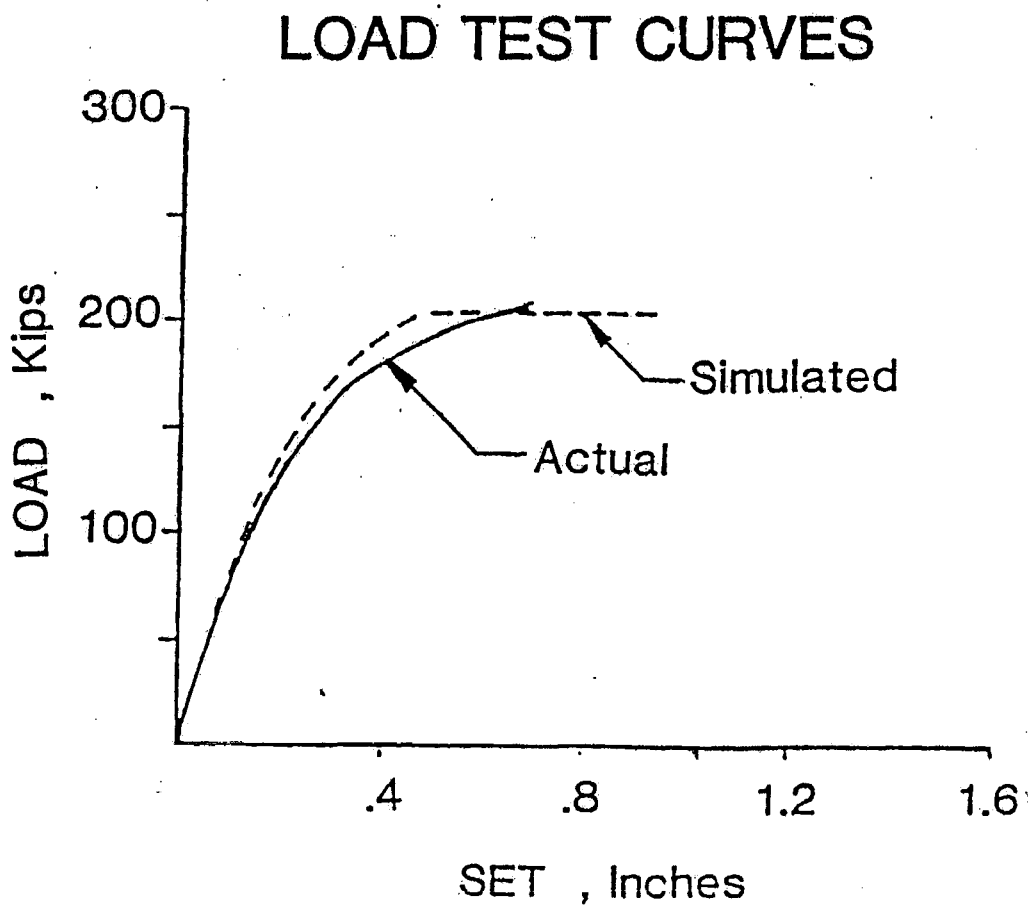
1 MN = 112 TONS

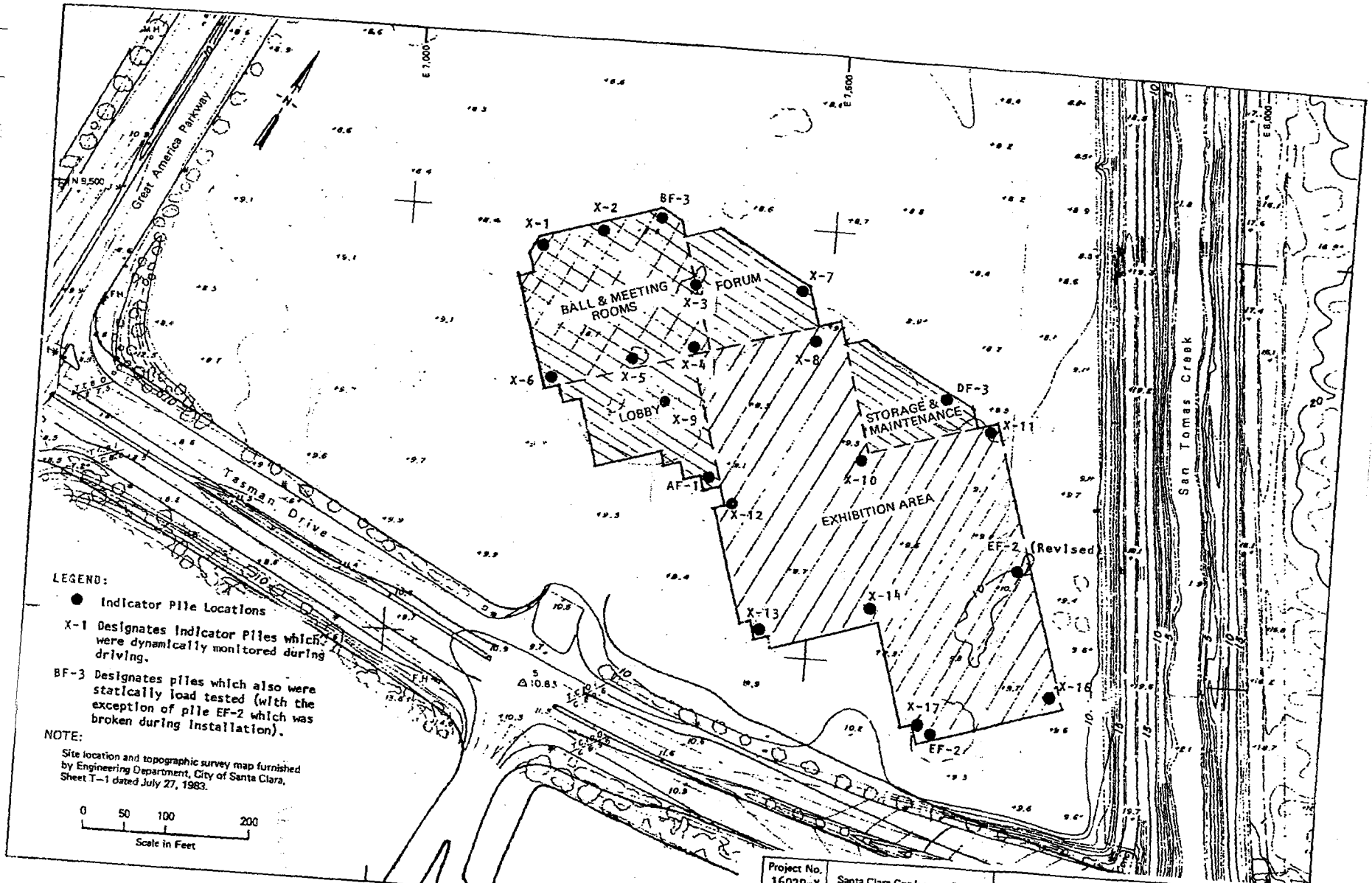
CAPWAP CORRELATION

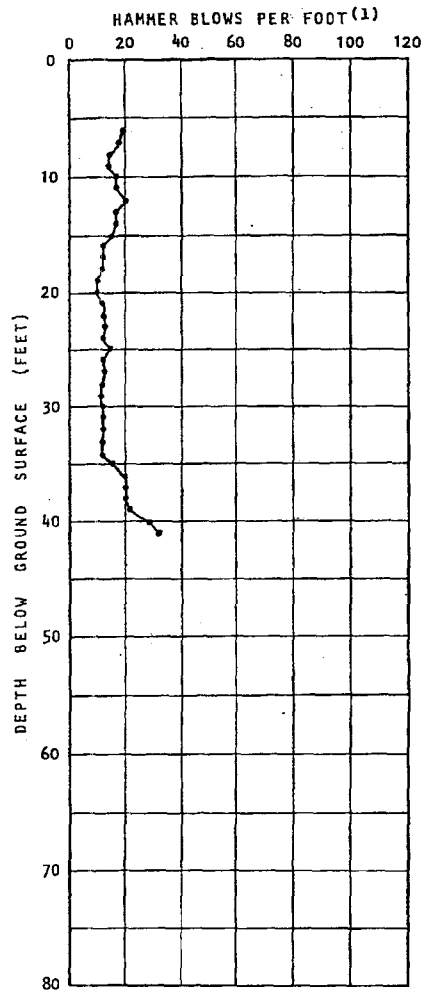
STATIC

After a CAPWAP analysis has been performed the complete pile and soil model is available for static analysis. This analysis is analogous to a static load test and is therefore often referred to as a "simulated load test". The static analysis is done by incrementally loading the pile at the top and computing the resulting element penetrations and associated static soil resistance values. The final result of this analysis is the applied pile top force and the pile top penetration.

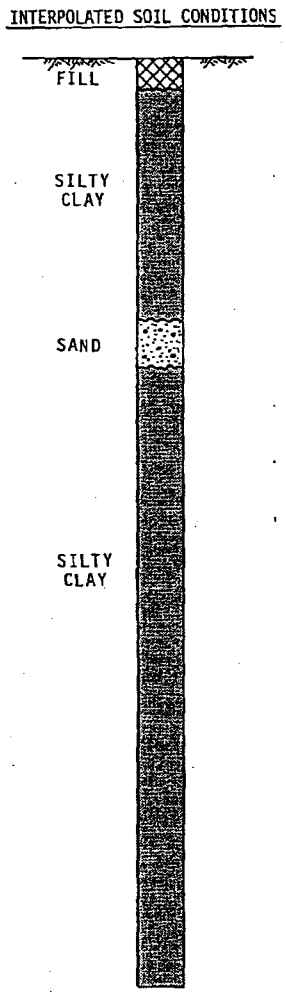
Usually the static analysis uses the same pile and soil model as in CAPWAP. Modifications are, however, possible. First, the pile model may be shortened, corresponding to a cutting off of the real pile. Second, for pile materials other than steel a different, usually lower, pile elastic modulus may be used. Finally, rather than using the dynamic quakes a static one may be introduced in order to include creep effects. Of course such static quakes are generally not known and the dynamic ones are therefore introduced. Then the resulting load-deformation curve excludes creep.

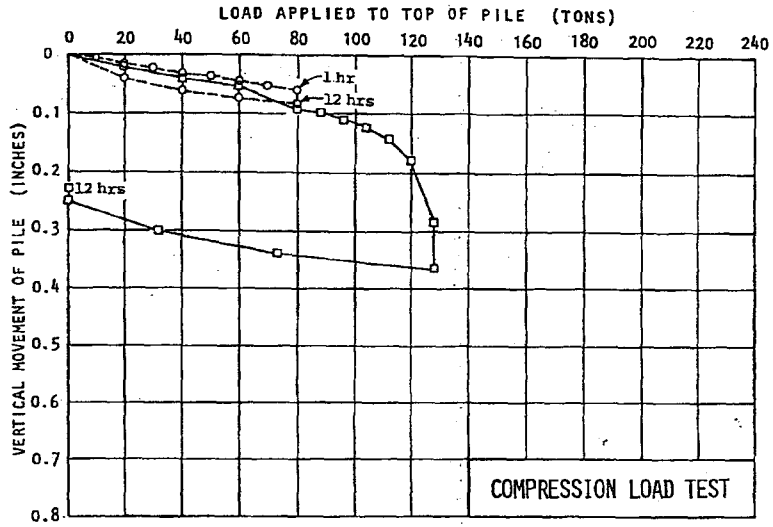


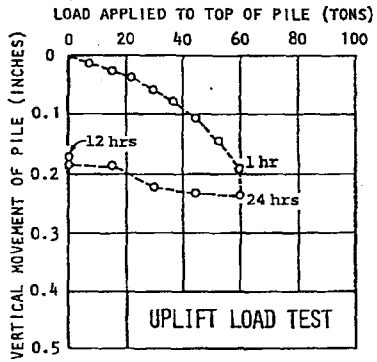

Great America Parkway
Tasman Drive
San Tomas Creek
N
E 7,000
E 7,500
E 8,000
N 9,500
BALL & MEETING ROOMS
FORUM
LOBBY
STORAGE & MAINTENANCE
EXHIBITION AREA
X-1
X-2
BF-3
X-3
X-4
X-5
X-6
X-7
X-8
X-9
X-10
X-11
X-12
X-13
X-14
X-16
X-17
DF-3
AF-1
EF-2 (Revised)
EF-2
LEGEND:
Indicator Pile Locations
X-1 Designates Indicator Piles which were dynamically monitored during driving.
BF-3 Designates piles which also were statically load tested (with the exception of pile EF-2 which was broken during installation).
NOTE:
Site location and topographic survey map furnished by Engineering Department, City of Santa Clara, Sheet T-1 dated July 27, 1983.
0 50 100 200
Scale in Feet
Project No. 16029-X
Santa Clara Conference Center
Woodward-Clyde Consultants
INDICATOR PILE AND LOAD TEST LOCATIONS
Figure 1



(1) Pile driven using ICE 520 diesel hammer having a maximum rated energy of 30,000 ft-lbs.







LOAD TEST RESULTS: TEST PILE AF-1

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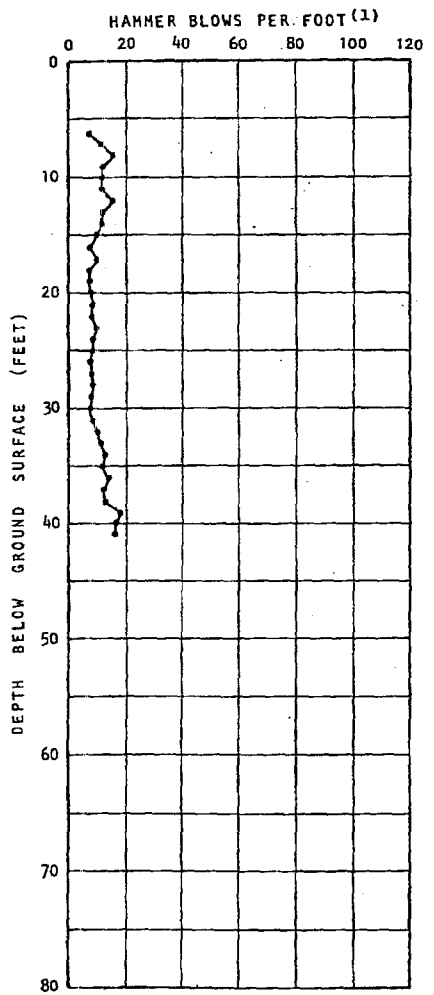
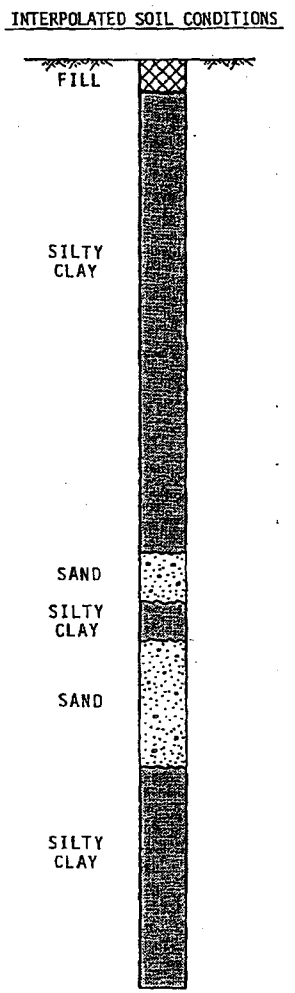
Santa Clara, California

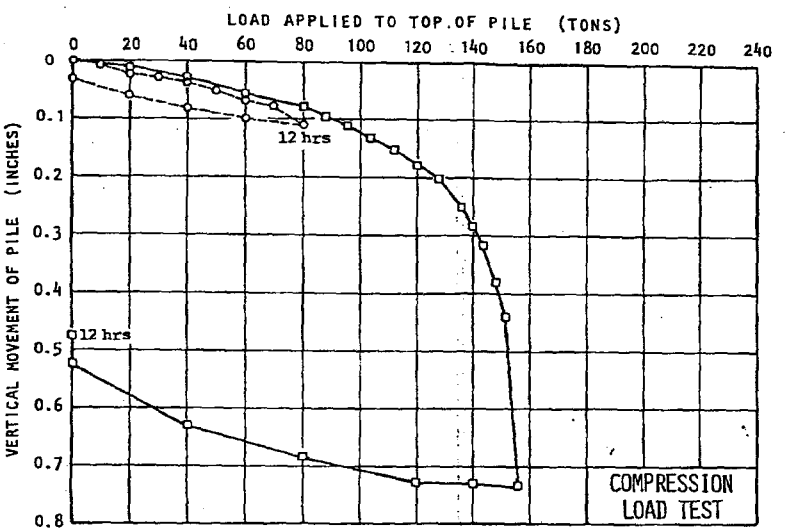
Project No. 16029X
Woodward-Clyde Consultants

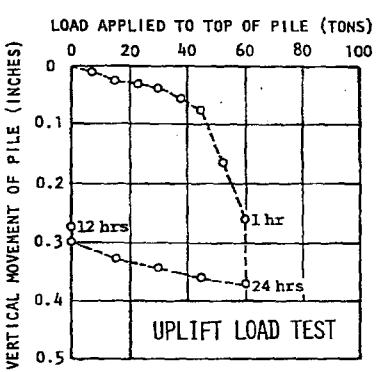
Figure 2



(1) Pile driven using ICE 520 diesel hammer having a maximum rated energy of 30,000 ft-lbs.







LOAD TEST RESULTS: TEST PILE DF-3

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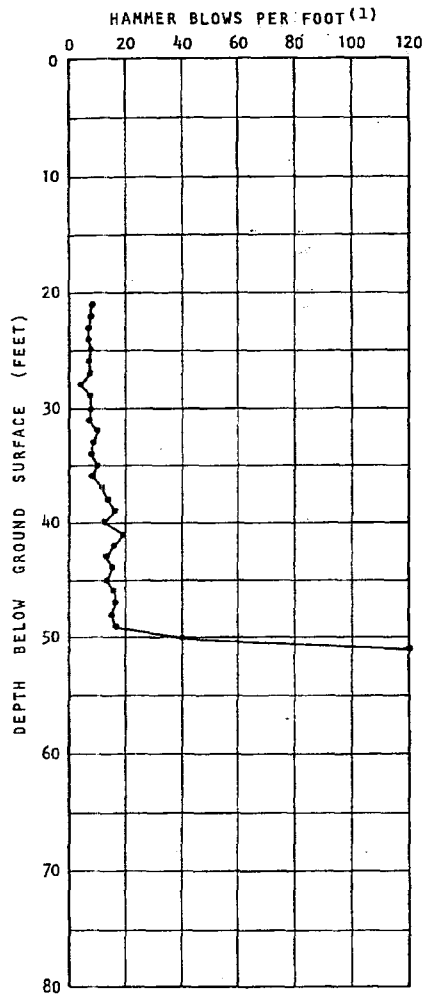
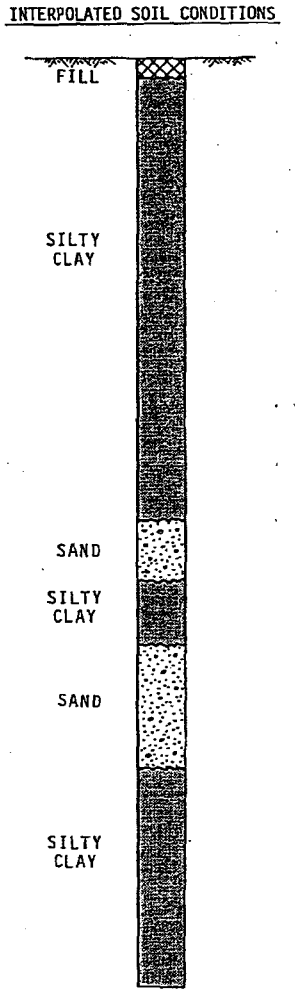
Santa Clara, California

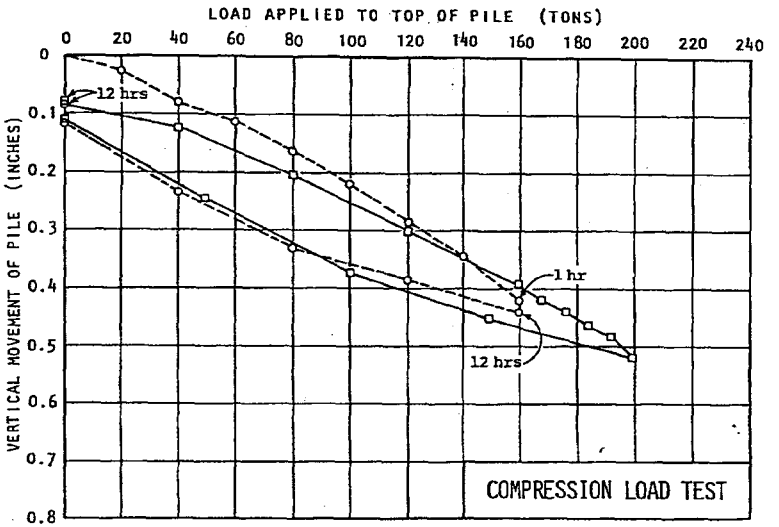
Project No. 16029X
Woodward-Clyde Consultants

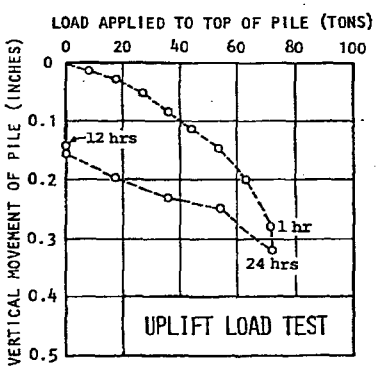
Figure 3



(1) Pile driven using ICE 640 diesel hammer having a maximum rated energy of 40,000 ft-lbs.







LOAD TEST RESULTS: TEST PILE EF-2 (REV.)

SANTA CLARA CONFERENCE & CONVENTION CENTER

Santa Clara, California

Project No. 16029X

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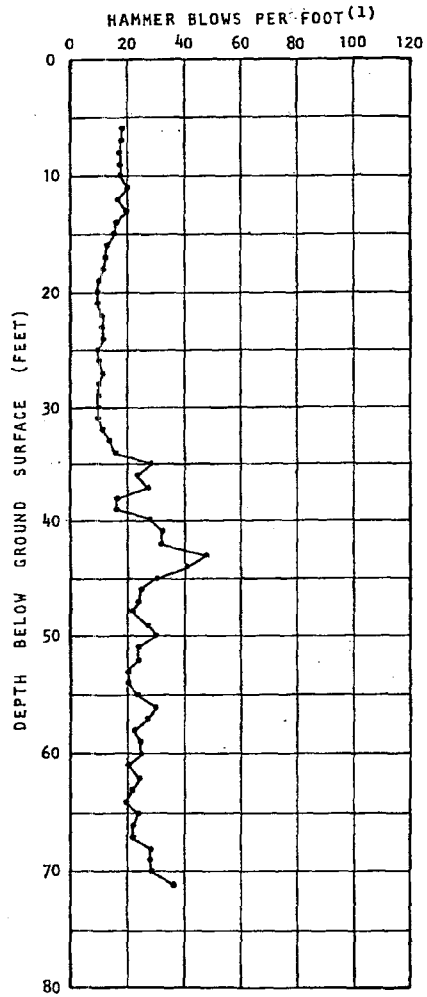
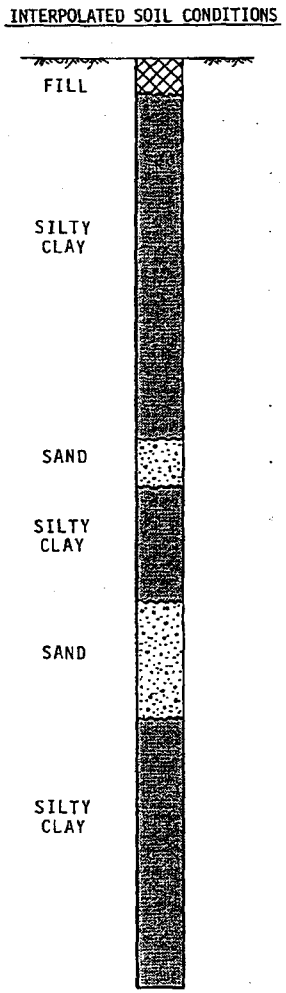
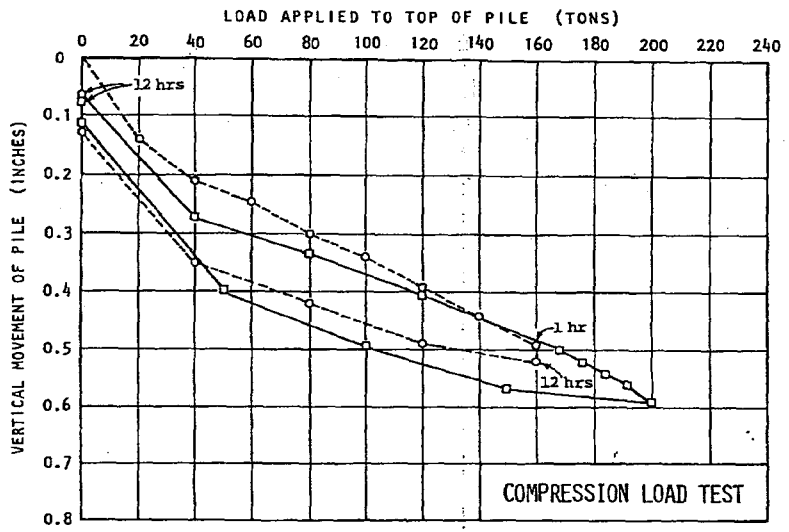
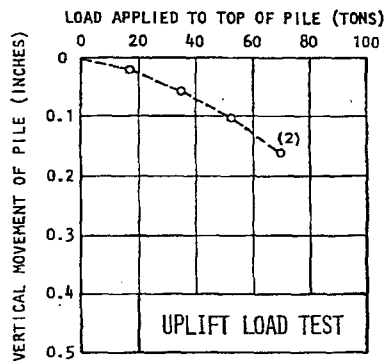
Figure 4



(1) Pile driven using ICE 520 diesel hammer having a maximum rated energy of 30,000 ft-lbs.







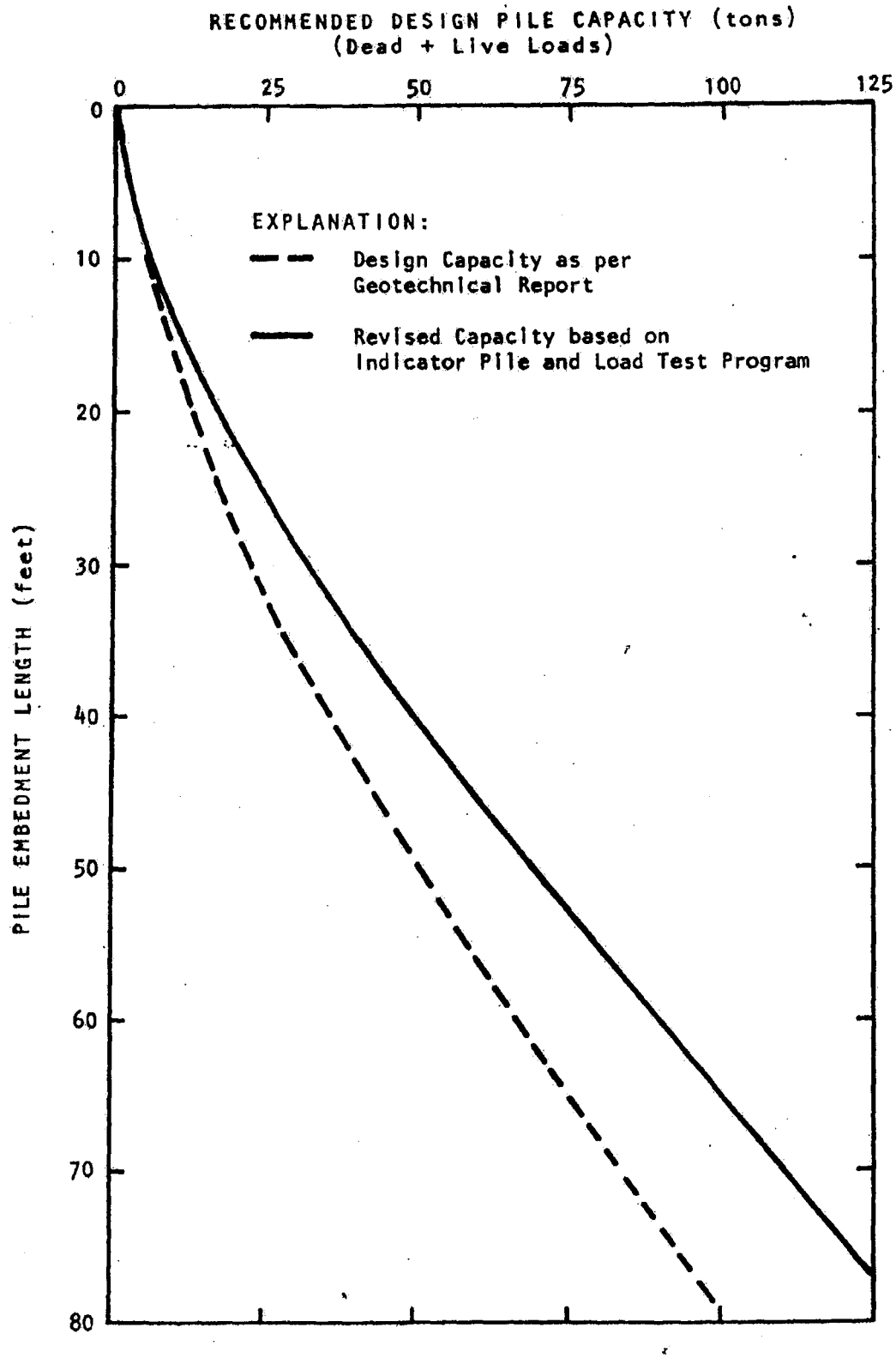
(2) Prestressing strand failed during the process of increasing the uplift load from 70 to 87.5 tons

LOAD TEST RESULTS: TEST PILE BF-3

SANTA CLARA CONFERENCE & CONVENTION CENTER
Santa Clara, California

Project No. 16029X
Woodward-Clyde Consultants

Figure 5



Project No. 16029-X **Woodward-Clyde Consultants**	SINGLE PILE COMPRESSION LOAD CAPACITY 12 Inch Square Pile	Figure 6

TABLE 1

Summary of Indicator and Load Test Pile Data

Pile Number		Length	Embedment Depth[2]	Approximate Tip Elevation
Field Designation	Plan[1] Location	(feet)	(feet)	(feet)[3]
X-1	A2-BF1-1	75	68-1/2	-57
X-2	A6-BF2-1	75	70	-62
X-3	F9-EF1-1	40	38-1/2	-26-1/2
X-4	CE-BF4-1	70	62	-54
X-5	F6-BF2-1	75	70-1/2	-59
X-6	E1-BF1-1	70	63-1/2	-55
X-7	E19-CF4-2	40	36	-25
X-8	H19-EF3-4	70	50	-44-1/2
X-9	J.5-7.5-AF2-1	70	66-1/2	-58
X-10	Q19-EF4-8	70	62	-53
X-11	Q28-EF3A-8	70	65-1/2	-57-1/2
X-12	Q11-EF2-8	75	71	-65-1/2
X-13	S11-EF2-8	75	69-1/2	-64
X-14	S19-EF2-8	75	66	-60-1/2
X-15[4]	---------	--	--	--
X-16	U28-EF2-8	75	65	-56-1/2
X-17	U19-EF2-4	70	66-1/2	-58-1/2
AF-1	N.5-9.5-AF1-5	45	41	-29-1/2
DF-3	M26-DF3-5	45	41	-30-1/2
EF-2	V20-EF2-2	75	(5)	(5)
EF-2(revised)	S27-EF2-7	75	49-1/2	-42
BF-3	B9-BF3-1	75	71	-59-1/2

Notes:

(1) Based upon numbering system adopted by the City of Santa Clara.

(2) Depth below existing pad grade or bottom of excavation at time of driving.

(3) Based upon existing pad grade of Elevation 11-1/2± (project datum).

(4) Indicator pile X-15 was deleted.

(5) Pile broken during installation. Embedment depth and tip Elevation uncertain.

TABLE 2

Summary of Pile Driving Analyzer Results - ICE 520 Diesel Hammer

Pile Number	Data Set	Length Below Gages (feet)	Pene-tration (feet)	Blow Count (bl/ft)	Max Measured Compressive Force (kips)	Max Measured Compressive Stress (ksi)	Max Computed Tensile Force (kips)	Max Computed Tensile Stress (ksi)	Maximum Transf. Energy (kip-ft)	Transfer[1] Efficiency (%)	Estimated Ultimate Pile Capacity[2] (kips)	(tons)
X-11	EOID[3]	67	65	69	410	2.9	30	0.21	8.9	29	270	(135)
BF-3	EOID	72	70	28	400	2.8	57	0.40	9.1	29	230	(115)
EF-2[4]	EOID	72	--	---	430	3.0	78	0.54	9.5	31	---	(---)
AF-1	EOID	42	40	32	350	2.4	10	0.07	9.3	30	100	(50)
DF-3	EOID	42	40	17	294	2.0	15	0.10	10.3	33	54	(27)
X-11	BOR[5]	67	65	156	366	2.5	--	----	10.2	33	360	(180)
X-10	BOR	67	59	53	470	3.3	--	----	11.1	36	300	(150)
X-4	BOR	68	67	75	501	3.5	38	0.26	11.8	38	320	(160)
X-5	BOR	73	63	132	467	3.3	56	0.39	9.4	30	340	(170)
X-6	BOR	66	63	200	441	3.1	--	----	11.1	36	300	(150)
X-3	EOID	42	40	40	375	2.6	13	0.09	8.4	27	170	(85)
X-7	EOID	37	37	---	226	1.6	25	0.17	6.7	28	0	(0)
DF-3	BOR	42	41	100	370	2.7	--	----	6.5	21	208	(104)
AF-1	BOR	42	41	60	470	3.3	--	----	7.2	23	230	(115)
X-7	BOR	37	37	228	327	2.3	--	----	6.6	21	160	(80)
X-3	BOR	37	36	88	415	2.9	--	----	8.1	26	230	(115)
BF-3	BOR	72	72	280	370	2.7	--	----	6.5	21	380	(190)
X-1	BOR	72	70	85	432	3.0	--	----	7.6	25	300	(150)

NOTES:

(1) Based on the maximum rated energy of the ICE 520 diesel hammer.

(2) Based on the Case Method of analysis.

(3) Dynamic Testing performed at the End Of Initial Driving (EOID).

(4) Pile broken during installation (see report text for discussion).

(5) Dynamic Testing Performed at the Beginning Of Restrike (BOR).

TABLE 3

Summary of Pile Driving Analyzer Results - ICE 640 Diesel Hammer

Pile Number	Data Set	Length Below Gages (feet)	Pene-tration (feet)	Blow Count (bl/ft)	Max Measured Compressive Force (kips)	Max Measured Compressive Stress (ksi)	Max Computed Tensile Force (kips)	Max Computed Tensile Stress (ksi)	Maximum Transf. Energy (kip-ft)	Transfer[1] Efficiency (%)	Estimated Ultimate Pile Capacity[2] (kips)	(tons)
X-14	EOID[3]	72	66	29	250	1.7	12	0.08	5.7	14	130	(65)
X-13	EOID	72	68	23	280	1.9	35	0.24	8.2	21	190	(95)
EF-2: (revised)	BOR[4]	53	48	640	350	2.4	--	----	10.0	25	380	(190)
X-12	EOID	72	69	50	400	2.8	38	0.26	10.9	27	155	(77)
X-9	EOID	67	67	23	290	2.0	17	0.12	7.7	19	140	(70)
X-8	EOID	67	46	91	470	3.3	50	0.35	12.2	31	330	(165)
X-2	EOID	72	70	20	510	3.5	72	0.50	12.8	32	160	(80)
X-2	BOR	72	72	84	550	3.8	--	----	12.2	31	360	(180)
X-9	BOR	67	66	60	570	4.0	--	----	12.2	31	260	(130)
X-8	BOR	67	45	240	430	3.0	--	----	11.5	29	400	(200)
X-3	BO2R[5]	37	35	144	424	2.9	--	----	12.1	30	310	(155)
X-12	BOR	72	71	101	560	3.9	--	----	12.1	30	415	(207)
X-13	BOR	72	67	280	460	3.2	--	----	9.8	25	485	(242)
X-14	BOR	72	67	204	340	2.4	--	----	6.6	17	300	(150)
X-17	BOR	72	66	120	330	2.3	--	----	6.8	17	260	(130)
X-16	BOR	72	65	340	400	2.8	--	----	9.5	24	420	(210)

NOTES:

(1) Based on the maximum rated energy of the ICE 640 diesel hammer.

(2) Based on the Case Method of analysis.

(3) Dynamic testing performed at the End Of Initial Driving (EOID).

(4) Dynamic testing performed at the Beginning Of Restrike (BOR).

(5) Dynamic testing performed at the Beginning Of a 2nd Restrike (BO2R).

TABLE 4

Summary of Case Capacity Results

Pile Number	Length Below Gages (feet)	Estimated Ultimate Capacity EOID (tons)	Estimated Ultimate Capacity BOR (tons)	Estimated Ultimate Capacity BO2R (tons)
X-11	67	135	180	---
BF-3	72	115	190	---
EF-2 (1)	72	---	---	---
AF-1	42	50	115	---
DF-3	42	27	105	---
X-10	67	---	150	---
X-4	68	---	160	---
X-5	73	---	170	---
X-6	66	---	150	---
X-3	37	85	115	155 (2)
X-7	37	0	80	---
X-1	72	---	150	---
X-14	72	65	150	---
X-13	72	95	242	---
EF-2 (revised)	53	---	190 (3)	---
X-12	72	77	207	---
X-9	67	70	130	---
X-8	67	165	200	---
X-2	72	80	180	---
X-17	72	---	130	---
X-16	72	---	210	---

NOTES:

(1) Pile broken during initial installation.

(2) 2nd restrike after an additional waiting period of 15 days.

(3) Restrike after only 4 hour waiting period.

TABLE 5

Summary of CAPWAP/C Results

Pile Number	Data Set	Quakes Skin (inch)	Quakes Toe (inch)	Smith Damping Skin (s/ft)	Smith Damping Toe (s/ft)	Case Damping Skin	Case Damping Toe	Estimated Ultimate Static Resistance Skin (kips)	Estimated Ultimate Static Resistance Toe (kips)	Estimated Ultimate Static Resistance Total (kips)	Estimated Ultimate Static Resistance Total (tons)
BF-3	BOR(1)	0.05	0.06	0.26	0.30	1.70	0.10	365	20	385	(192)
DF-3	BOR	0.05	0.06	0.24	0.09	0.60	0.10	143	67	210	(105)
X-7	BOR	0.05	0.02	0.11	0.26	0.25	0.15	135	35	170	(85)
X-12	BOR	0.06	0.05	0.19	0.18	1.38	0.03	403	10	413	(206)
X-9	BOR	0.05	0.10	0.17	0.24	0.60	0.25	200	61	261	(130)
EF-2 (revised)	BOR	0.13	0.15	0.10	0.28	0.45	0.64	245	135	380	(190)
X-3	BOR	0.05	0.15	0.15	0.19	0.45	0.20	164	66	230	(115)
X-3	BO2R(2)	0.05	0.15	0.08	0.19	0.35	0.20	247	63	310	(155)

NOTES:

(1) Data obtained at Beginning Of Restrike (BOR).

(2) Data obtained at Beginning Of 2nd Restrike (BO2R).

PILE DRIVING RECORDS

PILE NO. A2-BF1-

RECORD NO. X1

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No: 16029-X

Ground Elev. 11.5 ± (68' 8" below pad grade) No excavation

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	20'	
2			42	23	
3			43	23	
4			44	25	
5			45	21	
6			46	28	
7			47	23	
8			48	30	
9			49	53	
10			50	56	
11	26		51	37	
12	24		52	27	
13	22		53	27	
14	18		54	26	
15	18		55	28	
16	14		56	28	
17	11		57		
18	10		58		
19	11		59	~~100~~	7:20 AM 9/26/84
20	11		60	59	
21	14		61	46	
22	12		62	45	
23	12		63	44	
24	14		64	45	
25	12		65	34	
26	10		66	37	
27	11		67	39	
28	23		68	51	
29	19		69	28/4½"	41/6"
30	15		70		Restrike ~~9/27/84~~ ~8:20 AM
31	14		71		
32	11		72		
33	12		73		
34	12		74		
35	13		75		
36	16		76		
37	15		77		
38	15		78		
39	15		79		
39-40	28		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: X1 (A2-BF1-1 (9/14/84) 440P

PILE CAPACITY: 80T (9/14/84) (440P)

LENGTH AS CAST: 75'

DATE DRIVEN: 9/25/84

TIME DRIVEN: 10:40 / 10:58

HAMMER: ICE -520

ENERGY: (5) 30,000 FT-Lb

PLUMBNESS: O-K

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -57 ±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F.C.

NOTES:

(5): ~4 inch cushion initial driving & restrike

RECORD NO. X2

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (73' 8" below pad grade / 70' below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	47'	Hit minimum
2			42	59	
3			43	49	
4			44	34	
5			45	24	
6			46	21	
7			47	23	
8			48	32	
9			49	67	
10			50	74	
11	17		51	67	
12	15		52	79	
13	16		53	58	
14	15		54	37	
15	17		55	24	
16	17		56	19	
17	16		57	17	
18	14		58	14	
19	14		59	17	
20	16		60	17	
21	19		61	16	
22	22		62	17	
23	20		63	15	
24	20		64	15	
25	20		65	14	
26	18		66	14	
27	17		67	14	
28	15		68	16	
29	17		69	17	
30	18		70	16	
31	17		71	17	
32	17		72	19	
33	16		73	20	4:51 PM
34	16		74	7/1" Refusal	7:11 A.M.
35	17		75	9/1"	10/12/84
36	16		76		
37	17		77		
38	21		78		
39	38		79		
39-40	59		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: X2 (A6-BFZ-1)

PILE CAPACITY: 80T

9/25/84

LENGTH AS CAST: 75'

DATE DRIVEN: 10/11/84

TIME DRIVEN: ~4:15 P.M.

HAMMER: ICE 640

ENERGY: (5) 40,000 ft-lb.

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -62±

PAY LENGTH:

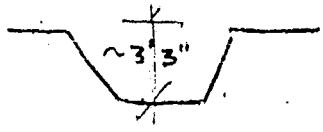
PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F.C.

NOTES:

(5): 2 inch cushion for initial driving & restrike



PILE NO.

RECORD NO. X3

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No: 16029-X

Ground Elev. 11.5 ± (38'4" below pad grade; 32' below bottom of excavation on 10/12/84)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	Redrive to grade on	
2			42	11-5-84 w/ICE 640	
3			33	129	
4			44	154	
5			45	78	
6	10		46	53	
7	9		47	39	
8	10		48	33	
9	10		49	19	
10	10		40	15	
11	10		51		
12	11		52		
13	7		53		
14	5/6" 4" 1"	Instrument hook up & installation	54		
15	8		55		
16	6		56		
17	7		57		
18	6		58		
19	7		59		
20	8		60		
21	7		61		
22	10		62		
23	9		63		
24	9		64		
25	8		65		
26	9		66		
27	9		67		
28	9		68		
29	9		69		
30	9		70		
31	8		71		
32	10		72		
33	15		73		
34	21		74		
35	31		75		
36	39	1 dozen instrumentation	76		
37	126	drive extra ft	77		
38	22/3" 4" 19		78		
39	10" 12" 14" 15" 1"	Restrike 9/27/84	79		
39-40		~8:00	79-80		

2nd Restrike 10/12/84 8:03 AM

TCE 640

PILE TYPE: 12" ϕ PC

PILE LOCATION: X3 (F9-EF1-1)

PILE CAPACITY: 40T

(9/10/84) 44 BP

LENGTH AS CAST: 40'

DATE DRIVEN: 9/26/84

TIME DRIVEN: 11:45 AM

HAMMER: ICE 520

ENERGY: 30,000 ft-lb

PLUMBNESS: OK

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -26½ ± (During indicator program)

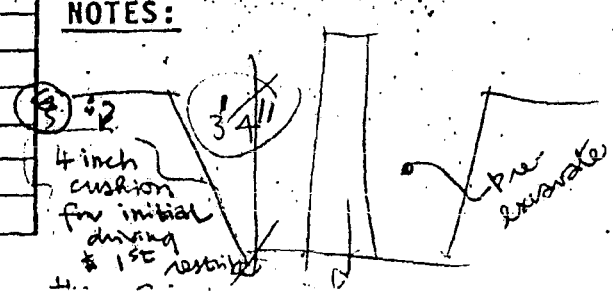
PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: FC

NOTES:



PILE NO. CE11-BF4-1

RECORD NO. X4

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER

Project No: 16029-X

(66' below pad grade
62' below bottom of excavation hole) (~3' 7" deep)

Ground Elev. 11.5±

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	22	
2			42	21	
3			43	20	
4			44	21	
5			45	19	
6			46	18	
7			47	22	
8			48	21	
9			49	25	wrap
10			50	23	
11	12		51	34	
12	10		52	31	
13	13		53	34	
14	11		54	30	
15	10		55	31	
16	16		56	30	
17	12	stop for lunch	57	33	
18	10		58	34	
19	9		59	31	
20	8		60	32	
21	8		61	32	
22	8		62	31	
23	7		63	30	
24	8		64	32	
25	8		65	48	restrike 11:10 AM 9/25/84
26	8		66	42/~6"	
27	9		67		
28	9		68		
29	9		69		
30	9		70		
31	11		71		
32	11		72		
33	13		73		
34	19		74		
35	33		75		
36	42		76		
37	39		77		
38	32		78		
39	25		79		
39-40	21		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: X4 (CE11-BF4-

PILE CAPACITY: 80T (9/10/84) 448P

LENGTH AS CAST: 70'

DATE DRIVEN: 9/25/84

TIME DRIVEN: 10:50 (12:37-12:48 2:35 → 2:40)

HAMMER: ICE 520

ENERGY: 30,000 FT-LB

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -54±

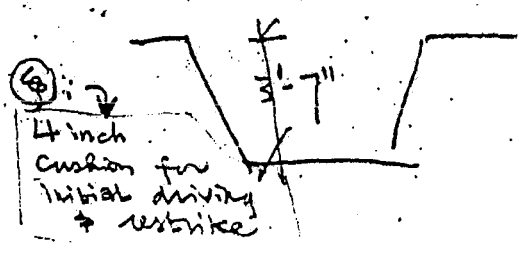
PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: FC

NOTES:



PILE NO. F6 BFZ

RECORD NO. X5

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER

Project No. 16029-X

Ground Elev. 11.5± (70' 4" below pad grade No EXCAVATION)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	36	
2			42	33	
3			43	31	
4			44	28	
5			45	28	
6	22		46	20	
7	18		47	24	
8	12		48	27	
9	12		49	28	
10	16		50	25	
11	16		51	24	
12	15		52	24	
13	14		53	24	
14	12		54	27	
15	11		55	25	Retap
16	9		56	45	
17	15		57	51	
18	10		58	78	
19	8		59	97	
20	10		60	75	
21	10		61	57	
22	10		62	51	
23	7		63	49	
24	9		64	41	
25	8		65	38	
26	7		66	35	
27	9		67	34	
28	9		68	39	
29	9		69	39	
30	11		70	39	
31	11		71	22/2"	Restrike 9/26/84 11:15AM
32	11		72	32/3"	
33	13		73		
34	20		74		
35	47		75		
36	60		76		
37	74		77		
38	93		78		
39	87		79		
39-40	59		79-80		

PILE TYPE: 12" ⊕ PC

PILE LOCATION: X5 (F6-BFZ-1)

PILE CAPACITY: 80T (9/14/84) 44KP

LENGTH AS CAST: 75'

DATE DRIVEN: 9/25/84

TIME DRIVEN: 10:~~0~~5 / 10:25 ~~2:51~~ 3:00

HAMMER: ICE 520

ENERGY: (4) 30,000 FT-Lb

PLUMBNESS: OK

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -59 ±

PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: FC

NOTES:

(4) 4 inch cushion for initial driving & restrike

PILE NO. E1-BF1-1

RECORD NO. X6

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER

Project No. 16029-X

(66'8" below pad grade
63'6" below bottom of excavation) ~3' hole

Ground Elev. 11.5±

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	33	
2			42	25	
3			43	21	
4			44	22	
5			45	26	
6			46	24	
7			47	24	
8			48	23	
9			49	23	
10			50	24	
11		shift	51	25	
12		up to	52	27	
13		match	53	23	
14	12	the pad grade	54	28	
15	10		55	27	retap
16	7		56	40	
17	8		57	40	
18	7		58	45	
19	7		59	37	
20	6		60	40	
21	6		61	41	
22	7		62	41	
23	9		63	47	
24	12		64	44	
25	21		65	42	
26	30		66	39	
27	25		67	42	
28	17		68	45	
29	13		69	87/4"	restrike
30	10		70	59/8"	11:35 AM 9/26/84
31	13		71		
32	20		72	27/1"	2nd retap
33	28		73	25/1"	10/19/84
34	30		74	24/1"	ICE 640
35	34		75	22/1"	
36	25		76		
37	40		77		
38	63		78		
39	56		79		
39-40	47		79-80		

PILE TYPE: 12" ⌀ PC

PILE LOCATION: X6 (E1-BF1-1

PILE CAPACITY: 80T

(9/10/84 440P)
LENGTH AS CAST: 70'

DATE DRIVEN: 9/25/84

TIME DRIVEN: 11:15 / 11:30
3:07 → 3:15 P.M.

HAMMER: ICE 520

ENERGY: (E) 30,000 Ft-lb.

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -55±

PAY LENGTH:

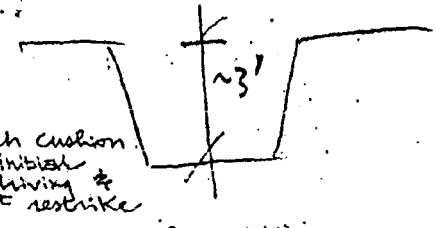
PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F.C.

NOTES:

(E):



4 inch cushion for initial driving & 1st restrike

2 inch cushion for 2nd retap

PILE NO. [illegible]

RECORD NO. X7

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (40' below pad grade, 36' 3" below excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	19 ~ 1"	Restrike ~~9/27/84~~ ~7:53 AM
2			42		
3			43		
4			44		
5			45		
6			46		
7			47		
8	7		48		
9	7		49		
10	7		50		
11	8		51		
12	7		52		
13	8	← Instrumentation work up	53		
14	4 #4		54		
15	8		~~55~~		
16	8		56		
17	7		57		
18	6		58		
19	7		59		
20	6		60		
21	6		61		
22	6		62		
23	6		63		
24	6		64		
25	6		65		
26	7		66		
27	7		67		
28	6		68		
29	6		69		
30	8		70		
31	9		71		
32	7		72		
33	7		73		
34	10		74		
35	10		75		
36	13		76		
37	14		77		
38	16	x 1"	78		
39	16	~~[illegible]~~	79		
39-40	12	detach instrument	79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: X7 (EM-CF4-2

PILE CAPACITY: 40T (9/10 ϕ 104)

LENGTH AS CAST: 40'

DATE DRIVEN: 9/26/84

TIME DRIVEN: 12:55 → 1:15 PM

HAMMER: ICE 524

ENERGY: (E) 30,000 ft-lb

PLUMBNESS: O.K

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -25±

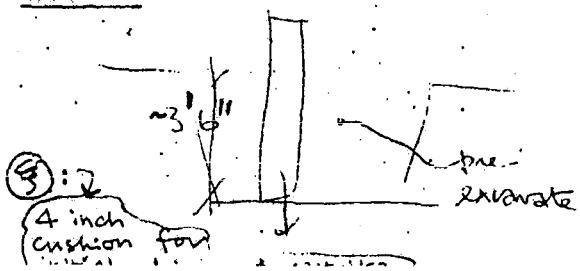
PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: PC

NOTES:



PILE NO. ___

RECORD NO. X8

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (56" below pad grade / 50" below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	22	hole up minimum
2			42	25	
3	2" cushion		43	22	
4			44	22	
5			45	27	
6			46	27	
7			47	35	
8			48	65	
9			49	77	
10			50	96	
11	10		51	102	
12	7		52	91	
13	4		53	101	4:10 PM
14	5		54 130#	[illegible]	10/12/84
15	3		55 87	22/1"	
16	3		56 30/6"	16/1"	New cushion 2"
17	4		57	17/1"	
18	5		58	13/1"	
19	6		59	14/1"	
20	9		60	13/1"	
21	9		61	16/1"	
22	9		62	13/1"	
23	12		63	17/1"	
24	11		64	12/1"	
25	12		65		
26	10				
27	11				
28	11				
29	11				
30	10				
31	12				
32	14				
33	16				
34	17				
35	17				
36	19				
37	23				
38	26				
39	28				
39-40	23	hole up minimum	79-80		

Redrive to grade ON 11-15-84

DEPTH	BLOWS	DEPTH	BLOWS
51	58	61	41
52	56	62	41
53	53	63	45
54	66	64	43
55	65	65	45
56	43	66	44
57	46	67	42
58	47	68	45
59	46	69	42
60	44	70	19/6" ±

PILE TYPE: 12" ϕ PC

PILE LOCATION: X8 (H19-~~F3~~)

PILE CAPACITY: 80T

LENGTH AS CAST: 70' (9/25/84)

DATE DRIVEN: 10/11/84

TIME DRIVEN: 3:50 PM

HAMMER: 1 CE 64ϕ

ENERGY: ~ 40,000 ft-lb

PLUMBNESS: OK

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -44½± (AT TIME OF INDICATOR PROGRAM)

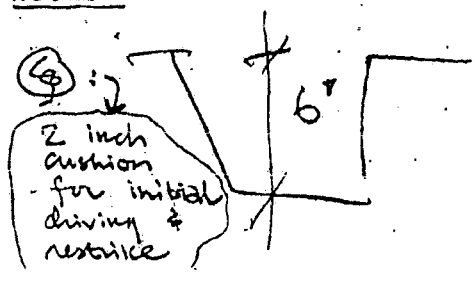
PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: F.C.

NOTES:



WOODWARD-CLYDE CONSULTANTS

RECORD NO. X9

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No: 16029-X

Ground Elev. 11.5± (69' 8" below pad grade; 66' 8" below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41		Miss blow count
2			42		
3			43	2Φ	
4			44	17 (18)	
5			45	18	
6			46	15	
7	"	cushion block	47	17	
8	"		48	19	
9			49	16	
10			50	18	
11	11		51	19	
12	9		52	17	
13	1Φ		53	18	
14	8		54	18	
15	9		55	18	
16	9		56	16	
17	8		57	16	
18	8		58	18	
19	1Φ		59	21	
20	11		60	2Φ	
21	1Φ		61	2Φ	
22	1Φ		62	19	
23	9		63	19	
24	9		64	2Φ	
25	9		65	23	
26	9		66	22	
27	9		67	22	
28	11		68	23	
29	12		69	23	Restrike 7:20 AM 10/12/84
30	12		70	5/1"	
31	13		71	5/1"	
32	11		72	20/3½"	2nd retap 10/19/84 10:55 AM
33	14		73	20/1"	
34	16		74	18/1"	
35	18		75	18/1"	
36	34		76		
37	48		77		
38	26		78		
39	16/10"	hook up monitoring Miss blow count	79		
39-40			79-80		

PILE TYPE: 12" Φ PC

PILE LOCATION: X9 (J.5-7.5-AP2)

PILE CAPACITY: 80T

LENGTH AS CAST: 70' (9/25/84 (440-P))

DATE DRIVEN: 10/11/84

TIME DRIVEN: 2:52 PM → 3:13 P.M

HAMMER: 1 CE 64Φ

ENERGY: ④ 4Φ,ΦΦΦ ft-lb

PLUMBNESS: O.K

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -58±

PAY LENGTH:

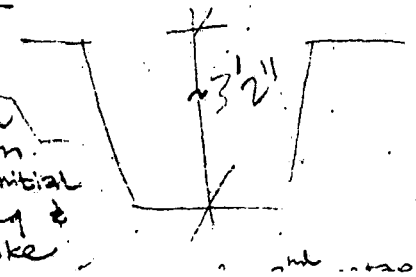
PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: E.C.

NOTES:

④: 2 inch cushion for initial driving & restrike



PILE NO. [illegible]

RECORD NO. X10

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

(65'6" below pad grade
62' below bottom of excavation
59' below " " on 10/12/84) 3' 3" fine

Ground Elev. 11.57

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	8	
2			42	8	
3			43	12	
4			44	18	
5			45	15	
6			46	12	
7			47	10	
8			48	9	
9			49	8	
10			50	10	
11			51	8	
12			52	9	
13			53	9	
14			54	9	
15			55	12	
16	5		56	13	
17	5		57	38	
18	5		58	28	
19	7		59	20	
20	8		60	19	
21	8		61	18	
22	11		62	53	restrike 11:00 AM 9/26/84
23	10		63	48	
24	8		64	52	
25	8		65	49	
26	8		66		
27	8		67		
28	7		68		
29	6		69		
30	8		70		
31	7		71		
32	7		72		
33	10		73		
34	8		74		
35	9		75		
36	8		76		
37	7		77		
38	9		78		
39	9		79		
39-40	9		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: X 10 (Q19-EF4-E

PILE CAPACITY: 80T

(9-10-84 448P)

LENGTH AS CAST: 70'

DATE DRIVEN: 9/25/84

TIME DRIVEN: 1:05 / 1:18

HAMMER: ICE 520

ENERGY: (4) 30,000 FT-LB

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -53 ±

PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: FC

NOTES: partial energy top 15'

(4): 4 inch cushion initial driving & restrike

RECORD NO. X11

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (65' 6" below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	20	
2			42	37	
3			43	43	
4			44	25	
5			45	20	
6			46	16	
7			47	13	
8			48	16	
9			49	27	
10			50	29	
11	6		51	52	
12	5		52	63	
13	5		53	63	
14	7		54	52	
15	10		55	100	
16	12		56	154	
17	10		57	133	
18	9		58	144	
19	9		59	153	
20	9		60	157	
21	9		61	162	
22	9		62	153	
23	8		63	168	
24	7		64	138	
25	6		65	69	detach monitoring
26	6		66	51	
27	5		67	47	
28	6		68	53	
29	6		69	37	
30	5		70	13/1"	Restrike 9/26/84 10:55 AM
31	6		71		
32	8		72		
33	9		73		
34	10		74		
35	9		75		
36	9		76		
37	8		77		
38	10	hook up monitoring	78		
39	9		79		
39-40	13		79-80		

PILE TYPE: 12" ⌀ PC

PILE LOCATION: X 11 (Q28-EF30)

PILE CAPACITY: 80T

9/10/84

LENGTH AS CAST: 70'

DATE DRIVEN: 9/25/84

TIME DRIVEN: 1:40 P.M.

HAMMER: ICE 520

ENERGY: (E) 30,000 FT-Lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -57½±

PAY LENGTH:

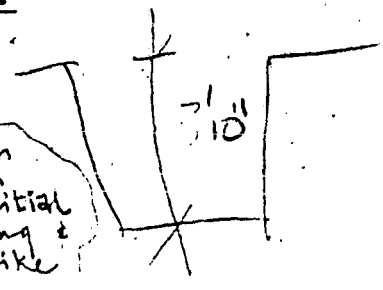
PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F. C.

NOTES:

(E): 4 inch cushion for initial driving & restrike



PILE NO. 4 EFC

RECORD NO. X12

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (77' below pad grade / 71' below bottom of excavation on 10-12-84)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	21	
2			42	21	2¼" cameron wedge & hook up mandrel
3			43	19	
4			44	17	
5			45	16	
6			46	15	
7			47	16	
8			48	15	
9			49	15	
10			50	16	
11	7		51	15	
12	7		52	16	
13	6		53	15	
14	7		54	18	
15	6		55	17	
16	7		56	18	
17	8		57	18	
18	8		58	19	
19	9		59	21	
20	9		60	22	
21	10		61	22	
22	14		62	19	
23	15		63	22	
24	19		64	22	
25	18		65	21	
26	20		66	23	
27	19		67	22	
28	16		68	24	
29	16		69	22	
30	14		70	27	
31	14		71	29	
32	12		72	28	
33	15		73	30	
34	15		74	33	
35	16		75	51	
36	16		76	50	
37	16		77	44/5"	@ 1:19 PM 10/12/84
38	18		78		
39	20		79		
39-40	20		79-80		

PILE TYPE: 12" ⌀ PC

PILE LOCATION: X12 (Q11 EF2)

PILE CAPACITY: 80T

LENGTH AS CAST: 75'

DATE DRIVEN: 10/11/84

TIME DRIVEN: 2:00 P.M.

HAMMER: ICE 640

ENERGY: (8) 40,000 ft-lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -65½±

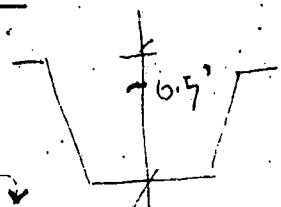
PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: FC

NOTES:



(8): 4 inch cushion for initial driving

WOODWARD-CLYDE CONSULTANTS

PILE NO. [illegible]

RECORD NO. X13

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (74' 6" below pad grade / 69' 6" below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	25	
2			42	26	
3			43	20	
4			44	16	
5			45	14	
6			46	12	
7			47	13	
8			48	11	
9			49	12	
10			50	12	
11	10		51	13	
12	10		52	12	
13	12		53	12	
14	12		54	14	
15	13		55	23	
16	14		56	22	
17	18		57	15	
18	17		58	12	
19	19		59	15	
20	25		60	13	
21	30		61	16	
22	35		62	13	
23	38		63	14	
24	42		64	14	
25	63	hook up monitoring	65	20	
26	25		66	20	
27	23		67	22	
28	21		68	24	
29	20		69	21	
30	21		70	22	
31	19		71	19	
32	23		72	20	
33	34		73	21	
34	50		74	23	
35	68		75	24	
36	71		76	35/12	Restrike 10/11/84
37	71		77		
38	45		78		8:38 AM
39	20		79		
39-40	25		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: (X13) (S11-EF2-8)

PILE CAPACITY: 80T

LENGTH AS CAST: 75' (91.25 | 84)

DATE DRIVEN: 10/11/84

TIME DRIVEN:

HAMMER: ICE 640

ENERGY: (Eg) 40,000 ft-lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -64±

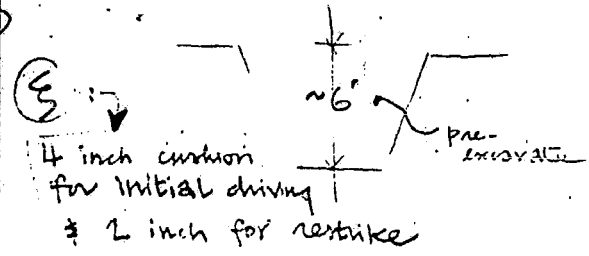
PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F.C.

NOTES:



PILE NO. [illegible]

RECORD NO. X14

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No: 16029-X

Ground Elev. 111.5± (~72' below pad grade ~66' below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	30	
2			42	41	
3			43	34	
4			44	31	
5			45	30	
6			46	22	
7			47	22	
8			48	23	
9			49	24	
10			50	24	
11	14		51	25	
12	13		52	29	
13	15		53	28	
14	15		54	26	
15	18		55	45	
16	31		56	41	
17	34		57	32	
18	25		58	29	
19	21		59	26	
20	20		60	26	
21	18		61	26	
22	18		62	27	
23	18		63	25	
24	16		64	30	
25	19		65	29	
26	19		66	28	
27	20		67	31	
28	16		68	31	
29	14		69	27	
30	13		70	31	
31	15		71	29	11:37 AM
32	16		72	18/", 15/", 13/", 13/"#	
33	17		73		
34	18		74	17/", 17/", 15/"	
35	20		75	15/", 13/", 10/"	Restrike 10/12/84 8:35 AM
36	18	Stop for hooking up mufflers	76		
37	35		77		
38	26	*	78	2nd restrike 10/19/84	
39	36		79		
39-40	30		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: X14 (519-EF2-8

PILE CAPACITY: 80T (9/25/84)

LENGTH AS CAST: 75'

DATE DRIVEN: 10/11/84

TIME DRIVEN: 10:50 AM

HAMMER: ICE 640

ENERGY: (5) 44,000 ft-lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -60 1/2±

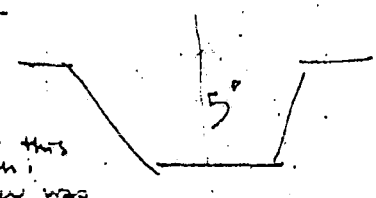
PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: FC

NOTES:



* Below this depth hammer was not fired properly

(5): 4 inch cushion for initial driving

WOODWARD-CLYDE CONSULTANTS

PILE NO. 4[illegible]

RECORD NO. X16

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5 ± (68' below pad grade / 65' below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	17	
2			42	18	
3			43	19	
4			44	20	
5			45	22	
6			46	22	
7			47	22	
8			48	21	
9			49	22	
10			50	33	
11	11		51	43	
12	11		52	60	
13	11		53	57	
14	10		54	52	
15	9		55	52	
16	11		56	46	
17	11		57	46	
18	11		58	31	
19	11		59	28	
20	12		60	28	
21	14		61	26	
22	14		62	23	
23	13		63	23	
24	14		64	23	
25	15		65	23	
26	13		66	23	
27	15		67	28	
28	16		68	49'-10"	
29	16		69	28/11"	Restrike 10/12/84 9:10 A.M
30	13		70		
31	10		71		
32	12		72		
33	17		73		
34	17		74		
35	17		75		
36	17		76		
37	16		77		
38	22		78		
39	21		79		
39-40	18		79-80		

PILE TYPE: 12"φ PC

PILE LOCATION: X16 (428-EF2-E)

PILE CAPACITY: 80T

LENGTH AS CAST: 75' (9/25/84)

DATE DRIVEN: 10/11/84

TIME DRIVEN: 9:45 → 9:50 AM

HAMMER: ICE 640

ENERGY: (4) 40,000 ft-lb

PLUMBNESS: OK

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -56½ ±

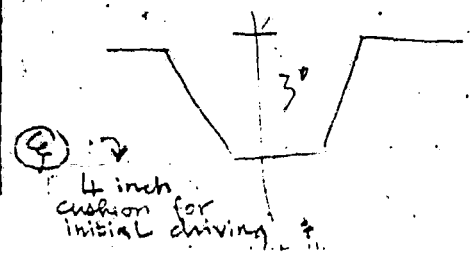
PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: FC

NOTES:



PILE NO. [illegible]

RECORD NO. X1

PILE INSPECTION RECORD

Project Name *SANTA CLARA CONFERENCE CENTER* Project No: *16029-X*

Ground Elev. 11.5± (70' below pad grade ~66' 8" below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	24	
2			42	23	
3			43	21	
4			44	23	
5			45	19	
6	10		46	20	
7	14		47	16	
8	16		48	16	
9	20		49	17	
10	22		50	14	
11	25		51	16	
12	24		52	15	
13	24		53	14	
14	23		54	15	
15	22		55	15	
16	23		56	15	
17	20		57	15	
18	19		58	14	
19	21		59	15	
20	18		60	16	
21	18		61	16	
22	18		62	16	
23	19		63	15	
24	19		64	17	
25	20		65	20	
26	21		66	22	
27	19		67	25	
28	17		68	21	
29	18		69	20	
30	14		70	19", 15", 8", 7", 7", 6", 6", 8"	
31	15		71		
32	15		72		
33	14		73	23/1"	
34	16		74	18/1"	Restrike 10/12/84 8:45AM
35	16		75	15/1"	
36	18		76		
37	18		77		Retap 10/19/84 11:11AM
38	28		78		
39	24		79		
39-40	28		79-80		

PILE TYPE: 12" ⊞ PC

PILE LOCATION: X17 (U19-EF2-

PILE CAPACITY: 80T

9/25/84

LENGTH AS CAST: 70'

DATE DRIVEN: 10/11/84

TIME DRIVEN: 10:21 → 10:37 AM

HAMMER: ICE 640

ENERGY: (4) 40,000 ft-lb.

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -58½±

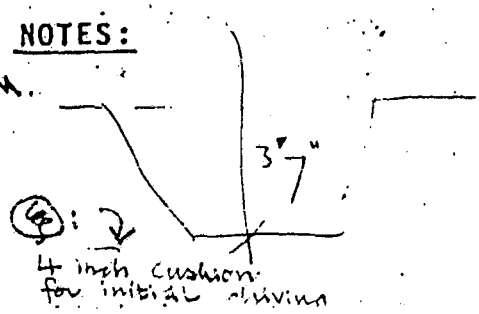
PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED:

INSPECTOR: FC

NOTES:



PILE NO. AW-35-AF1-

RECORD NO. AF-

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

(~41' below grade)

Ground Elev. 11.5 ±

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	16/6"	
2			42	20/4"	Restrike (9/27/84) - 7:38 AM
3			43		
4			44		
5			45		
6	19		46		
7	18		47		
8	14		48		
9	14		49		
10	8/5"	8/7" Rig up equipment	50		
11	17		51		
12	20		52		
13	17		53		
14	17		54		
15	15		55		
16	12		56		
17	12		57		
18	11		58		
19	10		59		
20	10		60		
21	11		61		
22	13		62		
23	13		63		
24	12		64		
25	14		65		
26	12		66		
27	13		67		
28	11		68		
29	11		69		
30	12		70		
31	12		71		
32	13		72		
33	12		73		
34	12		74		
35	15		75		
36	20		76		
37	20		77		
38	21		78		
39	22		79		
39-40	29		79-80		

PILE TYPE: 12" ⌀ PC

PILE LOCATION: AF1 (N.S-9.5-AF1-

PILE CAPACITY: 40T

LENGTH AS CAST: 45'

DATE DRIVEN: 9/26/84

TIME DRIVEN: ~9:45 A.M./10:03

HAMMER: ICE 520

ENERGY: 30,000 ft-lb.

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -29½ ±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: FC

NOTES:

(9): 4-inch cushion for initial driving & restrike

RECORD NO. DF-3

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± (~42' below grade)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	17	
2			42	7/2"	
3			43	67/8"	restrike 7:25 AM 9/27/84
4			44	118/-1"	
5			45		
6	8		46		
7	12		47		
8	16		48		
9	13		49		
10	12		50		
11	2/3" 10/8	hook up instruments	51		
12	15		52		
13	12		53		
14	11		54		
15	10		55		
16	8		56		
17	10		57		
18	7		58		
19	7		59		
20	7		60		
21	9		61		
22	8		62		
23	10		63		
24	9		64		
25	9		65		
26	8		66		
27	8		67		
28	9		68		
29	9		69		
30	7		70		
31	9		71		
32	10		72		
33	11		73		
34	13		74		
35	12		75		
36	14		76		
37	13		77		
38	13		78		
39	18		79		
39-40	17		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: DF-3 (M26-DF3-

PILE CAPACITY: 40T (9/14/84) 44BP

LENGTH AS CAST: 45'

DATE DRIVEN: 9/26/84

TIME DRIVEN: 10:20 AM / 10:35

HAMMER: ICE-520

ENERGY: (6) 30,000 ft-lb

PLUMBNESS: ok

HEAVE: –

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -30½±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: RC

NOTES:

(6) 4 inch cushion for initial driving + restrike

PILE NO. V20-EF2

Broken Pile

PILE INSPECTION RECORD

RECORD NO. EF-2
initially selected as test pile. Broken during driving.

Project Name SANTA CLARA CONFERENCE CENTER

Project No. 16029-X

Ground Elev. 11.5±

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	27	
2			42	45	
3			43		**23/6" → STOP 15/6"
4			44	20	
5			45	18	
6	13		46	17	
7	16		47	22	
8	16		48	20	
9	16		49	25	
10	16		50	34	
11	16		51	28	
12	19		52	30	
13	15		53	24	
14	16		54	17	
15	19		55	14	
16	18		56	18	
17	20		57	27	
18	16		58	22	
19	14		59	18	
20	13		60	18	
21	11		61	18	
22	14		62	25	
23	11		63	28	
24	11		64	33	
25	11		65	40	
26	14		66	36	
27	16		67	33	
28	12		68	38	
29	12		69	32	
30	12		70	23/4"	
31	16		71		
32	16		72		
33	17		73		
34	18		74		
35	20		75		
36	20		76		
37	28		77		
38	42		78		
39	43/	11" Take	79		
39-40	40/13"		79-80		

PILE TYPE: 12"φ PC

PILE LOCATION: EF2 (V20-EF2-2

PILE CAPACITY: 80T
(9/14/84) 448P

LENGTH AS CAST: 75'

DATE DRIVEN: 9/26/84

TIME DRIVEN: 8:45 A.M.

HAMMER: ICE 520

ENERGY: 30,000 ft-lb

PLUMBNESS: O.K

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: unknown

PAY LENGTH:

PILE APPROVED: ____ DATE: ____

PILE REJECTED: Broken During Drivn

INSPECTOR: FC

NOTES:

(**) 1st of four breaks in pile reported to contractor during driving. Breaks monitored during dynamic testing with pile Analyzer.

PILE NO S27-EF2-7

RECORD NO. EF-2 Revised

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5 ± (53" below grade, 49" 6" below bottom of excavation)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	19	
2			42	16	
3			43	14	
4			44	16	
5			45	14	
6			46	16	
7			47	17	
8			48	15	
9			49	17	
10			50	41	
11			51	10/6" +	(*) 9:00 AM
12			52	↓	(*)
13			53		(*) 1:40 P.M
14			54		
15			55		
16			56		
17			57		
18			58		
19			59		
20			60		
21	9		61		
22	8		62		
23	7		63		
24	7		64		
25	8		65		
26	7		66		
27	7		67		
28	5		68		
29	7		69		
30	7		70		
31	7		71		
32	10		72		
33	9		73		
34	9		74		
35	10		75		
36	9		76		
37	12		77		
38	14		78		
39	17		79		
39-40	13		79-80		

(*): 4 inch cushion

PILE TYPE: 12" ϕ PC

PILE LOCATION: EF2 (S27-EF2-7 Revised)

PILE CAPACITY: 80T

LENGTH AS CAST: 75' (9/25/84)

DATE DRIVEN: 10/11/84

TIME DRIVEN: ~8:45 AM

HAMMER: ICE 640

ENERGY: 40,000 ft-lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -42 ±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: EC

NOTES: Restrike Blow Count

(*) 10/1" 19/1" 16/1" 15/1" 21/1" 50½ ↓ 51'

25/1" 23/1" 25/1" 23/1" 30/1" 23/1" 52' ↓ 52⅓'

23/1" 23/1" 46/1" 50/1" 35/1" 32/1" 36/1" 52⅔ ↓ ~53½' (1:40 PM)

PILE NO. B9-BF3-1

RECORD NO. BF-3

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No: 16029-X

Ground Elev. 11.5 ± (~71' below grade)

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	36'	
2			42	35	
3			43	47	
4			44	41	
5			45	30	
6	18		46	25	
7	18		47	24	
8	18		48	22	
9	18		49	27	
10	18		50	30	
11	20		51	23	
12	17		52	24	
13	19		53	21	
14	17		54	21	
15	15		55	23	
16	13		56	30	
17	13		57	27	
18	11		58	22	
19	10		59	24	
20	10		60	25	
21	10		61	21	
22	11		62	24	
23	11		63	22	
24	11		64	20	
25	10		65	23	
26	10		66	22	
27	11		67	22	
28	10		68	28	
29	10		69	28	
30	10		70	28	
31	10		71	~35	* Dyn. instrumentation removed
32	11		72	12 / 4½"	Restrike 9/27/84
33	14		73	26 / 1-½"	** 8:07 AM
34	16		74		
35	29		75		
36	24		76		
37	27		77		
38	20		78		
39	~20	} Test*	79		
39-40	32		79-80		

PILE TYPE: 12" ⌀ PC

PILE LOCATION: BF3 (B9-BF3-1)

PILE CAPACITY: 80T (9/14/84) (448P)

LENGTH AS CAST: 75'

DATE DRIVEN: 9/26/84

TIME DRIVEN: 7:50 → 8:20 AM

HAMMER: IGE 520

ENERGY: (4) 30,000 ft-lb

PLUMBNESS: 0.1"

HEAVE: No

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -59½ ±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F. C

NOTES:

* Instrumentation for dynamic monitoring

** New cushion

(4) . 4 inch cushion for initial driving

PILE NO. C9-BF4-1

RECORD NO. _____

PILE INSPECTION RECORD

Project Name: SANTA CLARA CONFERENCE CENTER

Project No.: 16029-X

Ground Elev. 11.5 *

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1		cushion	40-41	37	
2	4 inch		42	32	
3			43	30	
4			44	33	
5			45	30	✓
6	20		46	32	
7	15	✓	47	24	
8	17	✓	48	27	
9	21	✓	49	34	✓
10	20	✓	50	57	✓
11	21	✓	51	65	
12	13	✓	52	84	
13	13	✓	53	120	
14	14	✓	54	139	
15	13	✓	55	104	
16	12	✓	56	101	
17	15	✓	57	42	
18	16	✓	58	36	
19	14	✓	59	34	
20	22	✓	60	34	
21	17	✓	61	32	
22	21	✓✓	62	32	
23	15	✓	63	30	
24	18	✓	64	30	
25	17	✓	65	30	
26	13	✓	66	32	
27	17	✓	67	32	
28	19	✓	68	34	
29	20	✓	69	32	
30	20		70	32	
31	20		71	39	
32	20		72	39	
33	~~20~~		73	12" 32	
34	20		74		
35	20		75		
36	20		76		
37	20		77		
38	20		78		
39	20		79		
39-40	20		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: C9-BF4-1

PILE CAPACITY: 80'

9/25/84
LENGTH AS CAST: 70'

DATE DRIVEN: 10/19/84

TIME DRIVEN: 9:17 AM → 9:48 AM

HAMMER: ICE 640

ENERGY: 40,000 ft-lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -61 ±

PAY LENGTH:

PILE APPROVED: _____ DATE: _____

PILE REJECTED:

INSPECTOR: F.C.

NOTES:

* 69' below [illegible]

✓ Hammer misfiring

PILE NO. C9-BF4-2

RECORD NO.

PILE INSPECTION RECORD

Project Name: SANTA CLARA CONFERENCE CENTER

Project No.: 16029-X

Ground Elev.: 11.5 ± ∞

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	17	
2	4 inch cushion		42	15	
3			43	17	
4			44	19	
5			45	17	
6	13		46	13	
7	13		47	21	
8	13		48	29	
9	15		49	55	
10	16		50	74	
11	17		51	103	
12	12		52	116	
13	13		53	104	
14	13		54	100	
15	14		55	83	
16	12		56	68	
17	13		57	37	
18	12		58	37	
19	12		59	40	
20	10		60	40	
21	12		61	32	
22	11		62	29	
23	11		63	28	
24	9		64	30	
25	9		65	27	
26	8		66	29	
27	10		67	26	
28	11		68	31	
29	11		69	31	
30	12		70	30	
31	11		71	32	
32	11		72	37	
33	14		73	34	
34	16		74	33	
35	20		75	39	
36	19		76	38	
37	16		77	30	
38	16		78		
39	15		79		
39-40	16		79-80		

PILE TYPE: 12" ϕ PC

PILE LOCATION: C9-BF4-2

PILE CAPACITY: 80T

9/25/84

LENGTH AS CAST: 75'

DATE DRIVEN: 10/19/84

TIME DRIVEN: 8:43 → 9:10 A.M.

HAMMER: ICE 640

ENERGY: 40,000 ft-lb

PLUMBNESS: 0.1c.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -65½±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: FC

NOTES:

∞ 74' below bottom of excavation

4" cushion

PILE NO. C9-BF4-

RECORD NO.

PILE INSPECTION RECORD

Project Name SANTA CLARA CONFERENCE CENTER Project No. 16029-X

Ground Elev. 11.5± *

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41		
2			42		
3			43		
4			44		
5			45		
6			46		
7			47		
8			48		
9			49		
10			50		
11			51		
12			52		
13			53		
14			54	100	
15			55	116	
16			56	67	
17			57	35	
18			58	36	
19			59	35	
20			60	32	
21			61	31	
22			62	27	
23			63	29	
24			64	30	
25			65	29	
26			66	31	
27			67	31	
28			68	32	
29			69	29	
30			70	31	
31			71	36	
32			72	34	
33			73	35	
34			74	39	
35			75	33	
36			76	37	
37			77	39	
38			78		
39			79		
39-40			79-80		

4" cushion

PILE TYPE: 12" ⌀ PC

PILE LOCATION: C9-BF4-3

PILE CAPACITY: 80T

(9/25/84)
LENGTH AS CAST: 75'

DATE DRIVEN: 10/19/84

TIME DRIVEN: ~ 8:10 → 8:37 AM

HAMMER: 1 CE 640

ENERGY: 40,000 ft-lb

PLUMBNESS: O.K.

HEAVE:

CUT-OFF ELEV.:

LENGTH CUT OFF:

FINISHED TIP ELEV.: -65½±

PAY LENGTH:

PILE APPROVED: DATE:

PILE REJECTED:

INSPECTOR: F.C.

NOTES:

* 74" below bottom of excavation

PILE NO. C9-BF4-4

RECORD NO. _____

PILE INSPECTION RECORD

Project Name SANTA CLARA CONVENTION CENTER Project No. 16029-X

Ground Elev. 11.5 ±

FEET	BLOWS	REMARKS	FEET	BLOWS	REMARKS
0-1			40-41	30	
2			42	29	
3			43	30	
4			44	34	
5			45	41	
6			46	48	
7			47	100	
8			48	166	
9			49	204	
10			50	176	
11	21		51	170	
12	18		52	160	
13	15		53	133	
14	13		54	121	
15	14		55	105	
16	12		56	58	
17	11		57	51	
18	13		58	44	
19	14		59	42	
20	18		60	43	
21	17		61	40	
22	16		62	42	
23	14		63	38	
24	14		64	35	
25	14		65	46	
26	16		66	39	
27	13		67	31	
28	13		68	60	new machine
29	13	✓	69	46	
30	17	✓	70	44	
31	12		71	41	
32	17		72	44	
33	17		73		
34	19		74		
35	23		75		
36	22		76		
37	21		77		
38	22		78		
39	22		79		
39-40	29		79-80		

Min cushion

PILE TYPE: 12" ⌀ PC

PILE LOCATION: C9-BF4-4

PILE CAPACITY: 30T

LENGTH AS CAST: 70' (9/25/84)

DATE DRIVEN: 10/19/84

TIME DRIVEN: 9:59 A.M. / 10:41 A.M.

HAMMER: ICE / 640

ENERGY: 40,000

PLUMBNESS: O.K.

HEAVE: _____

CUT-OFF ELEV.: _____

LENGTH CUT OFF: _____

FINISHED TIP ELEV.: -61 ±

PAY LENGTH: _____

PILE APPROVED: _____ DATE: _____

PILE REJECTED: _____

INSPECTOR: F.C

NOTES:

* 69' below bottom of excavation

WOODWARD-CLYDE CONSULTANTS

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

Geotechnical Investigation

Santa Clara Parking Structure
and Pedestrian Bridge
Tasman Drive
Santa Clara, California

Report No. 1883-1 has been prepared for:

International Parking Design, Inc.
Santa Clara, California

September 16, 2003

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GEOTECHNICAL INVESTIGATION

SANTA CLARA PARKING STRUCTURE AND PEDESTRIAN BRIDGE

TASMAN DRIVE

SANTA CLARA, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation for the Santa Clara Parking Structure and Pedestrian Bridge to be located in Santa Clara, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the proposed parking structure and pedestrian bridge.

For our use we received the following:

- Several topographic plans (electronic file), prepared by BKF Engineers.
- A preliminary Foundation Plan (electronic file), prepared by International Parking Design, Inc. (IPD)

We have also recently provided a Phase I Environmental Site Assessment for the project, dated January 27, 2003. Environmental services for the project are being provided under a separate contract with the City of Santa Clara.

1.1 Project Description

As presently planned, the project consists of constructing a six-story, concrete frame parking structure and a pedestrian bridge. We understand that the parking structure will be constructed at-grade. The pedestrian bridge will be constructed over the San Tomas Aquino Creek to allow foot traffic from the Santa Clara Convention Center to the new parking structure. There are two locations being considered at this time; one is near the center of the structure and the other is at the south side, near Tasman Drive. At the time our field exploration program was performed, only the southern location was planned. Associated underground utilities, pavements, and landscaping are also planned.

Preliminary structural loads, provided by Mr. Ed Workman of IPD, the project structural engineer, indicate interior column dead plus sustained live loads will be on the order of 800 to 1,200 kips and perimeter column dead plus live loads will be on the order of 630 kips. The 1,200 kip column loads are associated with the interior columns at the ends of the interior rows (i.e. at column line 2 at D, shown on the Site Plan, Figure 2). Based on the plan provided, we understand that exterior and interior columns are spaced at about 18 feet on center and the rows of interior columns spaced at about 60 feet on center. The span between the interior columns at the ends of the rows to the perimeter columns is about 30 feet.

1.2 Scope of Services

Our scope of services was presented in detail in our agreement with you dated December 13, 2002. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling five borings, advancing five Cone Penetration Tests (CPTs), and retrieving relatively undisturbed and bulk soil samples for visual observation and laboratory testing.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Interpretation of the subsurface soils by correlating our CPT data with the boring logs and laboratory data.
- Engineering analysis to evaluate site earthwork, building foundations, slabs-on-grade, retaining walls and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Exploration Program

Subsurface exploration was performed on December 11 to 13, 2002, using conventional, truck-mounted CPT and rotary-wash drilling equipment. We hydraulically pushed five CPTs to depths ranging from 50 to 80 feet. We also drilled five exploratory borings to depths ranging from 40 to 50 feet. The CPTs and borings were backfilled with cement grout in accordance with Santa Clara Valley Water District guidelines. Two representative bulk samples of the near-surface soil were obtained for pavement design purposes. The approximate locations of the CPTs, borings, and bulk samples are shown on the Site Plan, Figure 2. As discussed above and shown on Figure 2, the pedestrian bridge is depicted at two alternate locations. At the time we executed our exploration program, we understood that the bridge would be located near the south side of the structure. Our boring and CPT logs, Log of Test Boring Sheets, and details regarding our field investigation are included in Appendix A; our laboratory tests are discussed in Appendix B. A soil corrosion evaluation is presented in Appendix C.

2.2 Surface

We also performed a brief surface reconnaissance during our site exploration. The site is bordered by Tasman Drive to the south, San Tomas Aquino Creek and the Santa Clara Convention Center (across the creek) to the west, a vacant lot and Stars and Strips Drive to the east, and tennis courts and a City of Santa Clara landfill to the north. The site is currently being used as an overflow parking lot for the Santa Clara Convention Center. The parking lot is about 10 to 15 feet below the existing pavement grade of Tasman Drive. Topographic information provided by BKF Engineers, the project civil engineer, indicated the existing parking lot gently slopes

down to the east with a grade change from about Elevation 11 to 9 feet. At the time of our exploration, a 4-inch-diameter monitoring well was observed near the southwest corner of the site. We understand that the monitoring well is related to activities at the former landfill facility located north of the site.

2.3 Site History

As part of our environmental services under a separate contract, our representative contacted Mr. Rick Mauck, Director of Streets and Automotive Services, and Mr. Jim Parissenti, Principal Engineer of Field Services, for general information regarding past and current site usage.

Mr. Parissenti indicated that, prior to 1985, the site had been part of the adjacent landfill and used for waste disposal. Soil at the site had initially been excavated in a series of trenches, relatively parallel to Tasman Drive, which were then filled with waste materials. Based on a 1985 map showing the site topography, the landfill material appears to have been placed to elevations approximately 15 feet higher than the current site elevation. Subsequently, in 1985, the landfill debris was reportedly removed from the site. During the debris removal, the trenches were reportedly located and the debris within and overlying the trenches was moved to another portion of the landfill. The bottom portion of the trenches extended below ground water and, after the waste materials were removed, the trenches were reportedly backfilled with rock and other fill material that was not well compacted. Approximately three feet of additional fill material were reportedly placed above the trenches and compacted prior to construction of the existing overflow parking lot. The source of the fill material used to backfill the site is not known.

Based on our experience with a nearby site overlying similar slot fills, the performance of the fill material has been relatively poor. The building, which was supported on shallow foundations, has experienced significant differential settlements.

2.4 Subsurface

Our explorations EB-1 through EB-4, and CPT-1 through CPT-5, were advanced in the vicinity of the proposed parking structure (east side of the creek). Boring EB-5 was drilled on the driveway of the Santa Clara Convention Center (west side of the creek). As discussed above, we understood at the start of our investigation that the bridge would be located near the south side of the structure. The current site plan (also used for our boring location plan, Figure 2) depicts the bridge at two alternate locations. Based on recent conversations with IPD, the final location of the pedestrian bridge will be decided pending review with the City of Santa Clara. At this time we are assuming that the bridge subsurface conditions near the center of the structure will be similar to that encountered in Boring EB-5.

The pavement structural sections encountered varied, but generally consisted of about 2 inches of asphalt concrete over 6 inches of aggregate base in the overflow lot and about 7 inches of asphalt concrete over 6 inches of aggregate base in the convention center driveway. Below the pavement sections, our borings generally encountered about 2½ to 20 feet of undocumented fill. The deepest fill was encountered at the northwest portion of the site in Boring EB-4. The fills encountered generally consisted of stiff to very stiff, moderately to highly plastic clay with some layers of medium

dense clayey sand. The lower 5 feet of clay fill encountered in Boring EB-4 was soft to medium stiff. Below the fill, our explorations encountered predominantly medium stiff to very stiff, low to moderately plastic clays with occasional interbedded layers of medium dense to very dense sand to the maximum depth explored of 80 feet. The sand layers encountered contained variable quantities of silt and clay fines. The general soil profile is depicted in Cross-Sections A-A' and B-B', Figures 3A and 3B, respectively.

A Plasticity Index (PI) test was performed on a near-surface soil at a depth of 2 feet and resulted in a PI of 33, indicating high plasticity and expansion potential.

2.5 Ground Water

Free ground water was encountered during drilling in Boring EB-1 and our CPTs at a depth of about 10 feet. The upper range of historic ground water fluctuations in the vicinity is generally considered to be at the depth of approximately 7 feet according to mapping by the California Division of Mines and Geology (CDMG, 2002). Fluctuations in the level of the ground water may occur due to variations in rainfall, perched water conditions, and other factors not in evidence at the time our measurements were made. For our analyses, we assumed a design ground water level of 7 feet below existing site grades.

2.6 Site Infiltration

Our borings indicate the site is blanketed by at least 10 feet of moderately plastic to highly plastic clays and clayey fills. Generally, the higher the PI of the clay, the lower the permeability and hydraulic conductivity of the soil. Therefore, we judge the site infiltration rate will be low for any proposed site detention/retention facilities. As discussed above, ground water was encountered at a shallow depth. The Regional Water Quality Control Board (RWQCB) requires that a minimum of 10 feet be maintained between the seasonal high ground water level and the bottom of any infiltration facility, which would require pre-treatment of pavement runoff water and potentially roof runoff prior to entering any infiltration facilities.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 4. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). As shown on Figure 4, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to all developments in the San Francisco Bay Area. The "Seismicity" section that follows summarizes potential levels of ground shaking at the site.

3.3 Liquefaction

3.3.1 General Background

The site is located within a State of California Seismic Hazard Zone for liquefaction (CDMG, 2002 – Milpitas Quadrangle). Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are loose to moderately dense, saturated granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment.

When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause increased hydrostatic pressure that induces liquefaction. Liquefaction can cause softening, and large cyclic deformations can result. In loose granular soils, softening can also be accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground (NCEER/NSF, 1998).

Loose granular soil can also settle (compact) during liquefaction and as pore pressures dissipate following an earthquake. Very limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

3.3.2 Subsurface Conditions Encountered

The granular soils encountered in our explorations were generally medium dense to very dense. Several sand layers encountered below the design ground water depth of 7 feet were considered in our liquefaction analyses. Those layers, where the corrected blow counts were greater than 30 or where the corrected tip resistance was greater than 160 tons per square foot (tsf), have been screened in accordance with guidelines in Special Publication 117 and are not included in the following table. Boring EB-1 was performed adjacent to CPT-1 to provide correlating visual observations and laboratory data. Our liquefaction analyses are presented below.

Table 1. Granular Layers Considered in Liquefaction Analyses – Boring Logs

Boring Number	Soil Type	Depth to Top of Sand Layer (feet)	Thickness of Layer (feet)	*Total Fines Content (%)
EB-2	SP-SM	10	1.5	11
EB-2	SM	11.5	5.0	36
EB-5	SC	39	2.0	25

*Total fines content based on washed sieve testing

Table 2. Granular Layers Considered in Liquefaction Analyses – CPT Logs

CPT Number	Soil Type	Depth to Top of Sand Layer (feet)	Thickness of Layer (feet)	*Soil Behavior Type Index (I_c)	**Plastic or Non-plastic Fines
CPT-1	SP-SM	26.0	1.0	2.0	Non-plastic
CPT-2	SP-SM	26.5	1.5	2.1	Non-plastic
CPT-4	SP-SM	26.0	0.5	2.3	Non-plastic
CPT-4	SP-SM	34.0	1.0	2.1	Non-plastic
CPT-4	SP-SM	50.0	1.0	2.0	Non-plastic
CPT-5	SP-SM	26.0	0.5	2.0	Non-plastic
CPT-5	SP-SM	30.0	3.0	2.1	Non-plastic
CPT-5	SP-SM	34.0	1.0	2.2	Non-plastic
CPT-5	SP	41.0	3.0	1.8	Non-plastic
CPT-5	SP-SM	50.0	5.0	2.0	Non-plastic

* Calculated from normalized cone resistance and normalized friction ratio
** The presence of plastic or non-plastic fines determined from I_c values

3.3.3 Methods of Analysis and Results

Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd, et al., 2001) in accordance with guidelines set forth in CDMG Special Publication 117 (CDMG, 1997). The NCEER methods for SPT and CPT analyses update simplified procedures presented by Seed and Idriss (1971). The analysis method compares the cyclic resistance ratio (CRR) with the earthquake-induced cyclic stress ratio (CSR) at different depths due to the estimated earthquake ground motions. The relationship for CSR is presented as follows:

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

where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. CRR is a function of the soil density and grain characteristics.

The factor of safety (FS) against liquefaction is expressed as the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). If the FS is less than 1.0, the soil is considered to be potentially liquefiable during seismic shaking.

$$FS = CRR/CSR$$

We evaluated the liquefaction potential of the medium dense sand strata encountered using both an estimated peak horizontal ground acceleration (PGA) of 0.34g, resulting from a 7.9 M_w event on the San Andreas Fault. As discussed in the "Seismicity" section that follows, these are estimated peak horizontal ground accelerations based on acceleration-attenuation equations presented by Campbell and Bozorgnia (1994).

We corrected the field SPT blow counts from our borings for overburden, stress reduction versus depth, fines content, hammer energy ratio, boring diameter, rod length and sampling method (SPT sampler without liners). Our CPT tip pressures were corrected for overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor "n" applied to the Normalized Cone Resistance "Q" to evaluate how likely a layer is to contain significant plastic fines and have a low liquefaction potential.

Cyclic Resistance Ratios (CRR) were calculated for both SPT and CPT methods using normalized "N" values and CPT tip pressures corrected to clean sand values and the SPT and CPT clean sand base curves presented in the NCEER method. The CRRs were then corrected for the design ground water level and magnitude scaling factors. The factor of safety against liquefaction is the ratio of the CRR to the CSR (cyclic stress ratio) or seismic demand on a soil layer based on the Seed and Idriss (1971) equation. Estimates of volumetric change and settlement were determined by the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement. The results of our analyses are presented below.

Table 3. Results of Liquefaction Analyses – SPT Method

Boring Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	SPT (N_1)	*SPT $(N_1)_{60CS}$	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)	Estimated Differential Settlement (in.)
EB-2	10	1.5	13	23	0.8	Liquefaction Possible	0.20	0.10
EB-2	11.5	5.0	11	26	0.9	Liquefaction Possible	0.48	0.24
						Total =	**0.68**	**0.34**
EB-5	39	2.0	17	26	0.7	Liquefaction Likely	0.31	0.15
						Total =	**0.31**	**0.15**

* SPT blow counts corrected for overburden and fines content

Table 4. Results of Liquefaction Analyses – CPT Method

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	q_c (tsf)	*q_{c1N} (tsf)	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)	Estimated Differential Settlement (in.)
CPT-1	26	1.0	92.7	107.5	0.4	Liquefaction Likely	0.26	0.13
						Total =	**0.26**	**0.13**
CPT-2	26.5	1.5	88.6	122.3	0.7	Liquefaction Likely	0.36	0.18
						Total =	**0.36**	**0.18**

Table 4. Results of Liquefaction Analyses – CPT Method, con't

CPT-4	26.0	0.5	73.4	124.2	0.6	Liquefaction Likely	0.11	0.05
CPT-4	34.0	1.0	121.6	140.6	0.7	Liquefaction Likely	0.16	0.08
CPT-4	50.0	1.0	168.3	154.0	1.0	Low	-	-
						Total =	**0.27**	**0.13**
CPT-5	26.0	0.5	100.6	126.0	0.6	Liquefaction Likely	0.11	0.05
CPT-5	30.0	3.0	96.3	121.4	0.5	Liquefaction Likely	0.72	0.36
CPT-5	34.0	1.0	80.0	108.0	0.4	Liquefaction Likely	0.26	0.13
CPT-5	41.0	3.0	176.1	148.1	0.9	Possible	**	-
CPT-5	50.0	5.0	185.8	159.3	1.1	Low	-	-
* CPT tip pressure corrected for overburden and fines content						**Total =**	**1.09**	**0.54**

** The potential settlement from this layer was excluded due to its depth and confinement above and below by relatively thick clay layers

Our analyses indicate that some of the sand layers theoretically can liquefy, resulting in about ¼- to 1-inch of total settlement. Post-liquefaction volumetric strains and settlements were estimated using Ishihara and Yoshimine (1990) using "N" values corrected to clean sand values for the SPT analysis. For the CPT analysis, we determined equivalent "N" values from normalized tip pressures, corrected to clean sand values and soil behavior type (Robertson, 1985). As discussed in the SCEC (1999) report, anticipated differential settlements for level sites with deep sediments will be on the order of half of the total estimated settlements, resulting in differential settlement estimates of less than ¼- to about ½-inch over a span of about 100 feet.

As the methods of analysis used to determine liquefaction potential do not take into account the capping action of the stiff clay stratum overlying liquefiable strata, we considered the effects of a capping layer. In order for liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer. Based on work by Youd and Garris (1995), a capping layer of non-liquefiable material on the order of 4½ to 5-feet-thick is adequate to prevent the occurrence of ground surface rupture for a liquefiable layer on the order of 2 to 3 feet in thickness. Since the localized potentially liquefiable sand strata is capped by at least 10 feet of medium stiff to very stiff clays and clayey fills, we judge the potential for sand boils or surface venting to be low during an earthquake.

3.3.4 Summary of Results

To summarize the results of our liquefaction analyses, some sand layers encountered are theoretically liquefiable. Theoretical total liquefaction-induced settlements are estimated to be on the order of ¼- to 1-inch. As discussed in the SCEC (1999) report, anticipated differential settlements for level sites with deep sediments will be on the order of half of the total estimated settlement. Therefore, surface differential settlement is estimated to be less than about ¼- to ½-inch over a span of about 100 feet.

3.4 Differential Compaction

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of the near-surface soils. In our opinion, provided the parking structure is supported on one of the foundation alternatives presented in the following sections of this report, we judge the probability of differential compaction impacting the structure to be low.

3.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation.

As shown on the Site Plan, Figure 2, the closest side of the parking structure is located about 40 feet east from the top of the San Tomas Aquino Creek bank. Since the sand layers encountered in our borings and CPTs near the creek were encountered at least 10 feet below the creek bed, were typically isolated and relatively discontinuous, we judged the probability of lateral spreading impacting the structure during a seismic event to be low; however, some cracking and/or minor displacements along the creekbank is possible.

4.0 SEISMICITY

4.1 Regional Active Faults

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The San Andreas Fault, which generated the great San Francisco earthquake of 1906, passes about 12 miles southwest of the site. Three other major active faults in the area are the Hayward Fault, located about 8 miles northeast, the Calaveras Fault, located about 9 miles northeast, and the Monte Vista-Shannon Fault, located about 8 miles southwest.

4.2 Maximum Estimated Ground Shaking

We performed a Deterministic Seismic Hazard Analysis (DSHA) for this site. This analysis utilizes the maximum Moment Magnitude (M_w) for the controlling faults, published attenuation curves, the shortest distance to the fault, and site-specific response characteristics. Based on the attenuation methods of Campbell and Bozorgnia (1994), the mean peak ground acceleration (PGA) expected for this site is 0.39g from a magnitude 7.9 M_w event on the San Andreas Fault, located 12 miles to the southwest. If a 7.1 M_w event were to occur on the closest portion of the Hayward Fault, located about 8 miles northeast, the expected PGA at the site would be approximately 0.34g.

4.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (1999), referred to as WG99, determined that there is a 70 percent chance (±10%) of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2000 and 2030. This result is the most important outcome of WG99's work, because any major earthquake can cause damage throughout the region.

This potential was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can inflict damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquake located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

4.4 UBC Site Coefficients

The CDMG has recently issued maps locating "Active Fault Near-Source Zones" to be used with the 1997 Uniform Building Code ("Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," CDMG/ICBO February 1998). Faults are classified as either "A," "B," or "C" as shown below. Only faults classified as "A" or "B" are mapped since faults classified as "C" do not increase the near-source factor.

Table 5. Seismic Source Definitions

Seismic Source Type	Seismic Source Description	Seismic Source Definition*	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/yr)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	M ≥ 7.0	SR ≥ 5
B	All faults other than Types A and C.	M ≥ 7.0 M < 7.0 M ≥ 6.5	SR < 5 SR > 2 SR < 2
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	M < 6.5	SR ≤ 2

*Note: Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining seismic source type.

The following table lists Type A and Type B faults within 25 kilometers of the site:

Table 6. Approximate Distance to Seismic Sources

Fault	Seismic Source Type	Distance (kilometers)
**Hayward (Southeast Extension)	B	9.4
*Hayward (Total Length)	A	13.1
Monte Vista – Shannon	B	14
Calaveras	B	15
San Andreas (1906)	A	19
San Andreas (Santa Cruz Mountains)	A	25

*Nearest Type A fault
**Nearest Type B fault

Based on our borings, CPTs, and alluvium thickness maps of Santa Clara County (Rogers and Williams, 1974), the site is underlain by alluvial deposits extending to depths in excess of 500 feet. Based on this information, the site may be characterized for design based on Chapter 16 of the 1997 UBC using the information in Table 7 below.

Table 7. 1997 UBC Site Categorization and Site Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_D
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	Hayward
Seismic Source Type (Table 16-U)	A
Distance to Seismic Source (kilometers)	13.1
*Near Source Factor N_a (Table 16-S)	1.00
Near Source Factor N_v (Table 16-T)	1.08
Seismic Coefficient C_a (Table 16-Q)	0.44
Seismic Coefficient C_v (Table 16-R)	0.69

*Note: For Seismic Zone 4, the near-source factor Na used to determine Ca need not exceed 1.1 for structures complying with all the conditions within UBC Section 1629.4.2.

5.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted four samples collected during our subsurface investigation to an analytical laboratory for pH, soluble sulfate and chloride content testing. We also subcontracted with JDH Corrosion Consultants, Inc. to evaluate the laboratory testing results and prepare a report summarizing the site corrosion potential. Their report is presented in Appendix C.

6.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

6.1 General

From a geotechnical engineering viewpoint the proposed development may be constructed as planned, provided design and construction are performed in accordance with the geotechnical recommendations presented in this report.

The primary geotechnical concerns at the site are as follows:

- Significant shallow foundation settlements
- Undocumented fills
- Highly expansive near-surface soils
- Potentially liquefiable sand layers
- Relatively shallow ground water

These concerns are briefly discussed below.

6.2 Foundation Settlements

We performed settlement analyses for a conventional spread footing foundation system bearing at about 4 feet below existing grade. We considered both footings over native materials and over engineered fill (i.e. assuming all the existing undocumented fill was removed and replaced as engineered fill). Our analyses were performed using exterior dead plus sustained column loads of 630 kips, interior dead plus sustained column loads of 805 to 1,200 kips, and an allowable bearing capacity of 3,000 pounds per square foot for dead plus live loads. As the column loads resulted in about 18-foot-square interior footings on an 18-foot-bay spacing, we considered the interior columns to be supported on uniformly loaded strip footings.

Our analyses indicate these building loads would result in total interior strip footing settlements of about 3 to 5 inches and exterior footing settlements of about 2 to 2¼ inches. Considering the column spacing and total settlement estimates, we judge differential settlements to be about ½- to ¾-inch between adjacent exterior columns; differential settlements between adjacent interior columns are anticipated to be about ½- to 1-inch; differential settlement between the exterior columns and the columns at the ends of the interior rows is anticipated to be about 1½ inches. As previously mentioned, an additional ¼- to about ½-inch of liquefaction-induced differential settlement across a span of about 100 feet may occur following strong seismic shaking, as discussed in the "Liquefaction" section.

If these total or differential settlements are not tolerable from an architectural or structural viewpoint, alternative foundation types should be considered. In our opinion, due to the significant estimated total and differential settlements, supporting the proposed parking structure and pedestrian bridge on footings over engineered fill is a high-risk option. Detailed recommendations for foundation alternatives, including footings over engineered fill, footings over improved soil such as stone columns or Geopiers, and deep foundations are presented in the "Foundations" section of this report.

6.3 Undocumented Fills

As previously discussed, undocumented fills on the order of 2½ to 20 feet were encountered at the site. If shallow foundations are desired and it is determined that total and differential settlements are tolerable from an architectural and/or structural viewpoint, all undocumented fills within the footing influence zone should be removed down to native soil and replaced with engineered fill. A minimum lateral distance of 10 feet beyond the building perimeter may be used for the deep fill areas. However, please note that construction dewatering will be required. In addition, aeration of soils removed from excavations below ground water will require considerable drying/aerating prior to reuse as engineered fill.

The fills should not be a significant geotechnical concern to a deep supported foundation system. However, measures should be taken to reduce the potential for distress to the floor slab if this fill were to settle over time. Possible measures would include designing lower level garage slabs as structural slabs, or providing individual deep support for floor slabs (i.e. piles). Partial mitigation measures could include removing and recompacting the upper portion of the fill. However, partial mitigation measures will present a significant risk of poor performance. In our opinion, removing and recompacting the upper 4 feet of the fill would reduce the potential for slab distress, but some future settlement and slab cracking may still occur.

If an increased potential for future maintenance is acceptable to exterior improvements, such as pavements, then the fill located outside of the recommended excavation limits need not be removed and recompacted. Otherwise, this fill should also be removed and recompacted, as discussed above.

6.4 Expansive Soils

The near-surface soils have a high plasticity and expansion potential. To reduce the potential for damage to the proposed parking structure, we recommend that any non-pavement slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill. The slab-on-grade pavement subgrade should be prepared as discussed in the "Earthwork" section below. Detailed recommendations addressing this concern are presented in the following sections of this report. No mitigation measures would be necessary for foundations due to the considerable gravity loads.

6.5 Liquefiable Sand Layers

As previously discussed, our borings and CPTs encountered predominantly medium stiff to very stiff clays with occasional interbedded layers of medium dense to very dense sand with variable quantities of silt and clay to the maximum depth explored of 80 feet. Based on our analyses, theoretically, some of these sand layers may liquefy, causing differential settlement across the structure. Liquefaction-induced differential settlements are estimated to be of less than ¼- to about ½-inch over a span of about 100 feet. If shallow foundations are used, they should be designed to resist or accommodate this movement in addition to the estimated static settlements. If the combined settlements are not structurally tolerable, soil improvement or deep foundations should be used. Detailed recommendations addressing this issue are presented in the "Foundations" section of this report.

6.6 Shallow Ground Water

Ground water may significantly impact grading and below-grade construction. These impacts typically consist of potentially wet and unstable subgrade soils, difficulty achieving compaction, and difficult underground utility installation. As previously discussed, the seasonal high ground water at the site is generally considered to be at a depth of approximately 7 feet according to mapping by the CDMG. We encountered ground water at a depth of about 10 feet, which will fluctuate seasonally. Therefore, the contractor should be aware that excavations extending near or below ground water may need to be stabilized and/or dewatered to facilitate placement and compaction of structures and fill, and that materials removed from excavations may need considerable aeration, conditioning, or chemical treatment prior to reuse.

6.7 Plans, Specifications, and Construction Review

Because subsurface conditions may vary from those encountered in our borings and CPTs, and to check that our recommendations have been properly implemented, we recommend we be retained to 1) review final construction plans and specifications and 2) observe the earthwork and foundation construction. Also, geotechnical conditions can be affected by the construction process. For the above reasons our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction. If the City of Santa Clara chooses to provide geotechnical observation and testing during construction, we have provided our requirements for review and oversight in Appendix D. If our requirements cannot be met, we will not be able to continue through construction as Geotechnical Engineer-of-Record.

6.8 Additional Subsurface Exploration

Additional subsurface exploration may be useful in helping delineate the locations and depths of the previous slot fills reportedly present at the site. Once these slot fills are identified, the fill depths of Zones A and B, as discussed in deep foundation sections below, may be revised to reflect the actual site conditions. In addition, if the pedestrian bridge location has permanently changed to near the center of the structure, we recommend that an additional boring be drilled at the new location on the west side of San Tomas Aquino Creek to confirm the subsurface conditions.

7.0 EARTHWORK

7.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing pavements, debris, buried utility lines, shrubs and associated roots. Removal of site fills and abandonment of existing buried utilities are discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of buried structures and utilities be carried out under the Geotechnical Engineer's observation and that the backfill be tested during placement. Alternatively, the loose backfill locations should be carefully documented during demolition for excavation and re-compaction during site grading.

7.2 Removal of Existing Fill

If shallow foundations are chosen without soil improvement such as stone columns or Geopiers, all fills should be removed down to native soil. The geotechnical engineer should be present during excavation to determine the required depth. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. Side slopes of fill excavations in building and pavement areas should be sloped at inclinations no greater than 3:1 (horizontal to vertical) in the upper 8 feet of the excavation to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

Ground water is anticipated at depths of 7 to 10 feet. As fill was encountered below ground water, the contractor should anticipate dewatering and subgrade stabilization. Even after the ground water level at the site is lowered by dewatering, the soils exposed by at the bottom of the excavation are expected to be wet and unstable under the weight of construction equipment. To provide a stable working platform for construction, a minimum of 18 inches of crushed rock will probably need to be placed across the excavations on a woven stabilization fabric, such as TC Mirafi 500X or equivalent prior to beginning backfill operations. The actual sections needed for subgrade stabilization should be determined at the time the excavations are completed.

If deep foundations are utilized for the project, and provided lower level pavements are designed as structural slabs or supported on deep foundations, the deeper site fills do not need to be removed and replaced as engineered fill. If soil improvement methods such as stone columns are used, the upper 3 feet below footing and slab depths will need to be sub-excavated and recompacted as the vibratory placement method for stone column placement needs overburden confinement for sufficient densification of fill to occur. Additional information regarding stone column construction is provided in the "Foundations" section.

7.3 Abandoned Utilities

Abandoned utilities within the proposed building area should be removed in their entirety. Utilities within the proposed building area would only be considered for in-place abandonment provided they do not conflict with new improvements, that the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.

Utilities outside the building area should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged. The potential risks are relatively low for small diameter pipes (4 inches or less) abandoned in-place and increasingly higher with increasing diameter.

7.4 Subgrade Preparation

After the site has been properly cleared and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, or pavements

should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

7.5 Material for Fill

All on-site native soils having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with no more than 15 percent larger than 2½ inches. Rocks or lumps larger than 4 inches should not be allowed to nest together. Rocks that nest together can cause bridging effects resulting in inadequate compaction. Imported and non-expansive fill should be a low plasticity material with a Plasticity Index of 15 or less.

Consideration should also be given to the environmental characteristics as well as the corrosion potential of import soils. Laboratory testing, including pH, soluble sulfates, chlorides, and resistivity will provide information regarding corrosion potential. Imported fill should not be more corrosive than the native materials.

The contractor should provide at least five working days notice prior to importing material to enable the Geotechnical Engineer to sample and test the geotechnical characteristics of the material. If environmental or corrosion test data is not available for review, sampling, testing, and evaluation of materials may require additional time.

7.6 Reuse of On-site Recycled Materials

If desired to reuse existing asphalt pavements as general fill or as engineered fill below sidewalks or pavements, we recommend that it be ground up to meet the gradation requirements of its intended use. If laboratory testing of the recycled material indicates that it meets Caltrans Class 2 specifications, it may be used as aggregate base beneath pavements and sidewalks. We should evaluate the proposed use of recycled materials prior to the work being performed. Recycled fill containing asphalt should not be used within 2 feet of finished grade in enclosed and habitable building areas such as any ground floor offices.

7.7 Compaction

All imported fill, as well as scarified surface soils with moderate plasticity in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction at a moisture content slightly above laboratory optimum as determined by ASTM Test Designation D1557, latest edition, except for the expansive clays. The expansive on-site clays should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent above laboratory optimum when placed within 3 feet of finished grade. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and non-yielding under the weight of the compaction equipment.

Since the on-site clayey soils have relatively high moisture contents, earthwork contractors should anticipate that these soils may require drying (aeration) prior to use as engineered fill or subgrade preparation even during summer months. Based on

our laboratory test results and engineering experience, we judge the on-site clayey soils are about 5 to 10 percent over the estimated laboratory optimum moisture content. Consideration should be given to the use of light weight grading equipment. The use of heavy vibratory equipment will tend to de-stabilize clays with high in-situ moisture contents.

In asphalt pavement and concrete slab areas subjected to vehicular traffic and wheel loads, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition) except for the native clays, which should be compacted as noted above. Aggregate base should be compacted at a moisture content near the laboratory optimum.

7.8 Wet Weather Conditions

Earthwork contractors should be made aware of the moisture sensitivity of clayey soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils and other stockpiled backfill materials may become saturated, soft and unworkable. Subgrade stabilization techniques might include the use of engineering fabrics and/or crushed rock or chemical treatment. Therefore, we recommend that consideration be given to construction during summer months, from late April to early October. As discussed in the "Compaction" section, the in-situ moistures are about 5 to 10 percent over anticipated laboratory optimum. Contractors should be aware that operation of heavy grading equipment, especially large vibratory equipment, can destabilize wet clays. Consideration should be given to the use of lighter weight equipment and sheepsfoot compactors to prepare the site subgrade.

7.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

The surficial soils encountered during this investigation may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. If the expansive clays are used as trench backfill, the expansive clay compaction requirements stated above should be followed. The upper 6 inches of low plasticity general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Where granular backfill is used in trenches, we recommend that a cut-off plug of low permeability material be placed where such trenches enter the building and pavement areas. This reduces the likelihood of water entering the trenches from the landscaped

areas and seeping through the trench backfill into the building and pavement areas and coming into contact with expansive subgrade materials.

If ground water is encountered in deeper utility trench excavations, crushed rock may be used as pipe bedding and initial backfill (if approved by the local jurisdiction and in conformance with the pipe manufacturer's recommendations) in order to provide a stable working platform for utility installation and backfill. The crushed rock should be consolidated in place by vibratory methods until no further volume reduction is observed.

7.10 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

7.11 Surface Drainage

Positive surface water drainage gradients should be provided adjacent to the building to direct surface water away from foundations and slabs towards suitable discharge facilities. We recommend a slope of at least 2 percent away from the building in landscaping and/or asphalt concrete pavement areas. Landscaping bio-swales and/or concrete swales in asphalt pavement areas should have a minimum slope of 1 percent. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundations and slabs-on-grade in a closed collection system that discharges to the storm drain system or paved surface that drains to storm drain system.

7.12 Storm Water Management

As discussed in the "Site Conditions" section of this report, the surficial, high plasticity clayey soils are anticipated to have a very low infiltration rate. In addition, ground water at the site was encountered at a relatively shallow depth of about 10 feet below the ground surface and can be expected to fluctuate seasonally. The Regional Water Quality Control Board (RWQCB) requires that a minimum of 10 feet be maintained between the seasonal high ground water level and the bottom of any infiltration facility. Since this requirement cannot be met, infiltration facilities, if required, would require pre-treatment of pavement runoff water, and potentially roof runoff, prior to entering any infiltration facilities. Due to the low infiltration rate and regulatory restrictions, significant infiltration of storm water may not be feasible as part of a storm water retention/detention program. In addition, as discussed below, due to the high plasticity surficial soils, it is recommended to restrict surface water infiltration adjacent to foundations and pavements.

7.13 Landscaping Considerations

As the near-surface soils are highly expansive, we recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 5 feet of structures, slabs-on-grade, or pavements,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements, and
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.

We recommend that the landscape architect incorporate these items into the landscaping plans, and that we review the plans before construction.

7.14 Construction Observation

All grading and earthwork should be performed under the observation of our representative to check that the site is properly prepared, that selected fill materials are satisfactory, and that placement and compaction of fills is performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. The project plans and specifications should incorporate all recommendations contained in this report.

We understand that the City of Santa Clara may choose to provide geotechnical observation and testing during construction while requiring that we remain as Geotechnical Engineer-of-Record. We have provided our requirements for review and oversight in Appendix D that must be met; otherwise we will not be able to continue through construction as Geotechnical Engineer-of-Record.

8.0 FOUNDATIONS

Provided the site is prepared as described in Section 7.0, "Earthwork", the proposed parking structure may be supported on the following foundation systems. While we are providing the recommendations for footings over engineered fill, please note that the estimated settlements are significant. This foundation option, in our opinion, should not be chosen due to the high risk associated with future building distress.

- Shallow spread foundations supported on engineered fill
- Shallow spread foundations supported on Geopiers or other ground improvement methods
- Friction piers
- Friction pre-cast concrete piles
- Friction Auger-cast piles

We have assumed that due to the depth of fill noted in Boring EB-5 (on the Convention Center side of the creek) and the proximity to the Convention Center, that the pedestrian bridge will be supported on either shallow footings on ground improvement or deep foundations. Detailed recommendations are presented below.

8.1 Footings over Engineered Fill

Provided the estimated static and seismic settlements provided below are determined to be tolerable, the proposed parking structure may be supported on conventional continuous strip footings bearing on engineered fill. We do not recommend that isolated column footings be used due to the magnitude of the settlements and the overlapping settlement influence zones.

As discussed previously, the site fills should be removed in their entirety and be replaced as engineered fill. Based on our conversations with the City of Santa Clara, the fill was placed in slot trenches roughly parallel with Tasman Drive. We anticipate that abrupt transitions in fill depth will be encountered. To reduce the potential for additional differential settlement due to abrupt changes in fill thickness, we recommend that fill thickness transitions be limited to no more than 3 feet. In addition, native materials should be sub-excavated to provide at least 2 feet of engineered fill beneath all footings.

8.1.1 Vertical Loads

All footings should have a minimum width of 24 inches and should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing. Utilities that pass through or shallowly beneath footings should cross perpendicular to the footings. These utilities should also be protected from anticipated foundation pressures and movement by sleeving through footings or extending the concrete depth to encapsulate the utility and place the foundation bearing surface below the utility.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. If footing excavations are allowed to dry out prior to pouring concrete, the soil will shrink and potentially cause the footings to heave once the soil is re-moisturized during the winter rains. It is essential that we observe the footing excavations before the reinforcing steel is placed.

8.1.2 Estimated Settlements

Based on the preliminary loads provided by Mr. Ed Workman of IPD, the project structural engineer, we estimated the following total and differential static footing settlements. Our analyses indicate these building loads would result in total interior column footing settlements of about 3 to 5 inches and exterior column footing settlements of about 2 to 2¼ inches. Considering the column spacing and total settlement estimates, we judge differential settlements to be about ½- to ¾-inch between adjacent exterior columns; differential settlements calculated between adjacent interior columns based on structural loading are anticipated to be about ½- to 1-inch; differential settlement between the exterior columns and the columns at the ends of the interior rows is anticipated to be about 1½ inches. Due to the variability of the subsurface conditions and engineered fill thicknesses, differential settlement estimates are typically considered to be up to about one-half of the total settlements. The static settlements are summarized below for clarity.

Table 8. Summary of Estimated Footing Settlements

Footing Location (Example Columns)	Dead + Sustained Live Load (kips)	Total Static Settlement (inches)	*Differential Static Settlement (inches)
Exterior 2 at B	630	2 to 2½	½ (B - 2 to 3) ¾ (1 - B to C)
Interior 2 at D	1,200	3¼ to 4	1½ (2 – B to D) 1 (2 - D to E) ½ (D - 2 to 3)
Interior 2 at E	805	4 to 5	½ (2 - E to F) ½ (E - 2 to 3)

* Calculated based on idealized soil profiles. Due to variable soil conditions, differential settlement could be up to about one-half of the estimated total settlement.

An additional of about ¼- to about ½-inch of liquefaction-induced differential settlement across a span of about 100 feet may occur following strong seismic shaking, as discussed in the "Liquefaction" section. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

We also estimated modulus of subgrade reaction values (referred to as the subgrade modulus) for the interior strip footings. In developing our estimates of modulus of subgrade reaction values, we considered all loads to be uniformly distributed over the interior strip footings. Our analyses indicate a modulus value of 10 pci at the edge of the footing (i.e. column line D at 2), and a modulus of 5 pci 10 feet from the footing edge (i.e. column line 2 from D.6 to L.4). Our estimates are considered reasonable initial values for finite element structural analysis, such as RISA or SAFE analyses. We would be pleased to provide supplemental consultation in refining the soil subgrade modulus values, if desired. In order to proceed with further analysis, we would need the output from the next iteration the SAFE analysis or other finite element analysis of the mats and large footings indicating contact pressures and verified settlements.

8.1.3 Lateral Loads

Lateral loads may be resisted by friction between the footings and the supporting subgrade. A maximum allowable coefficient of friction of 0.3 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We recommend that an allowable passive soil resistance based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

8.2 Footings over Soil Improvement

If the estimated total and seismic settlements are not tolerable, or if excvation and replacement of existing fills is not desirable, spread foundations may be supported on improved soil. Soil improvement methods can consist of Geopiers, stone columns, jet grouted soil-cement columns, and/or other similar methods. Any of the soil improvement methods discussed will be design-build contracts. The performance criteria should be based on reducing the estimated foundation settlements to tolerable levels. We should review all design-build submittals prior to construction. Conventional slab-on-grade pavements may be used with the methods described below; however, measures should be taken to reduce the potential for distress to the floor slab if this fill were to settle over time. Possible measures would include designing lower level garage slabs as structural slabs, or providing individual deep support for floor slabs (i.e. Geopiers). Partial mitigation measures could include removing and recompacting the upper portion of the fill. However, partial mitigation measures will present a significant risk of poor performance. In our opinion, removing and recompacting the upper 4 feet of the fill would reduce the potential for slab distress, but some future settlement and slab cracking may still occur.

8.2.1 Footings over Geopiers

We contacted a representative from Geopier™ Foundation Company of Northern California (GFCNC) who provided the preliminary recommendations below. Geopier™ foundation elements are constructed similarly to drilled piers in that a volume of material is excavated by augering, however, they are backfilled by compacting well-graded aggregate in lifts using a modified hydraulic hammer. Below the ground water table, clean crushed rock is used. This method pre-stresses the soil and provides increased stiffness in the composite soil zone. If caving conditions are encountered during excavation through sand layers, casing may be required.

8.2.2 Geopier Vertical Loads

We contacted Geopier™ Foundation Company of Northern California (GFCNC) to provide preliminary recommendations. Based on the preliminary design, a 30-inch Geopier element, extended to the bottom of the fill or to the minimum depth to reduce total settlements, could potentially provide an allowable composite bearing pressure of 6,500 psf for dead plus live loads, which may be increased by one third for short term transient or seismic loads. The preliminary analyses are based on a 90-kip Geopier with the varying shaft lengths. Geopier elements equipped with a steel uplift anchor can provide 60 to 75 kips of allowable uplift capacity depending on design length. The following table presents some preliminary criteria for geopier foundation system.

Table 9. Geopiers Supported Footing Sizes

Maximum Total Loads (kips)	Number Geopier Elements	Length Geopier Elements (feet)	Footing Width Dim. (feet)	Footing Length Dim. (feet)	Footing Bearing Pressure (psf)	Total Settlement (inches)
630	7	16	10	10	6,300	1
810	9	18	11.5	11.5	6,100	1
1260	14	18	11	18	6,400	¾

Shallow foundations placed on the Geopier™ elements will have similar embedment criteria as those stated in the "Footing" section above. Based on the preliminary analysis as shown on Table 9, the total and differential settlements are estimated to be less than 1-inch and ½-inch, respectively.

8.2.3 Geopier Load Testing

We recommend that modulus and uplift load tests be performed to verify the bearing and uplift capacities of the Geopier. Due to the variability of fill depths, we suggest one load test in the deep fill area and one in the shallow fill area. In addition, we recommend that a 24-hour load test be performed to confirm settlement estimates. We should observe and monitor installation of all Geopiers on a full-time basis.

8.2.4 Stone Columns

The installation of stone columns would increase the density of the fills and weaker native soils by laterally displacing the existing in-place soil. The degree to which the density is increased will depend on the diameter and spacing of the stone columns. In addition to increasing the density, stone columns would provide an additional increase in bearing capacity at the individual stone column locations and reduce the potential for detrimental total and differential settlements across the improved area. The stone columns should be designed by the design/builder, such as Hayward Baker, to meet maximum settlement criteria of 1-inch post construction total settlement and ½-inch post construction differential settlement between adjacent columns under static and seismic loads. We recommend that the stone column design include, but not be limited to: 1) drawings showing the layout, spacing and diameter of the stone columns, 2) column length, 3) top and bottom elevation of the column, and 4) minimum post CPT tip criteria to be achieved in the sand after installation and refusal criteria. We should be retained to review the design/builder's stone column installation plan and settlement estimates prior to construction of the stone columns.

Due to the lack of confining pressure, the effectiveness of stone columns is anticipated to be minimal near the ground surface. Additionally, we anticipate that the upper few feet of ground surface will be disturbed due to installation of the stone columns. Therefore, it will be necessary to sub-excavate and recompact the surficial soils in those areas improved with stone columns in order to provide a uniform bearing surface.

Stone columns would generally be constructed as follows: 1) clear the site of existing demolition debris and excavate the building pad to the design pad grade, 2) install stone columns on an approximately 6- to 8-foot-square and/or triangular grids beneath the footings. Installation of stone columns beneath the garage slab should be considered to reduce the potential of settlement of the floor slabs during seismic shaking. Alternatively, maintenance, repair and replacement of the concrete slab should be anticipated following a seismic event. The stone columns would be installed to a depth of 4 feet below the bottom of the footings, 3) excavate the existing fill to a depth of 3 feet below the bottom of the proposed footings, 4) recompact the subgrade soils, 5) replace the excavated soils as compacted engineered fill to build a compacted soil mat over the stone columns, and 6) construct the footings.

8.2.5 Stone Column Performance Testing

Performance testing typically consists of a test section with post-installation CPT testing to confirm that the necessary composite soil strength increases were achieved to meet the settlement criteria. Post-installation CPT testing is also required during production installation. We should observe and monitor installation of all stone columns on a full-time basis and review the post-installation settlement analyses provided by the contractor.

8.2.6 Footings over Jet Grouted Soil-Cement Columns

The design use of jet grouted soil-cement columns is similar to the other mitigation methods discussed above. Jet grouting should be designed by the design/builder working in conjunction with SCC Technology, Inc. to meet settlement criteria of 1-inch post construction total settlement and ½-inch post construction differential settlement between adjacent columns under static and seismic loads.

8.3 Friction Piers

As an alternative to spread footings or spread footings supported on improved soil, the proposed parking structure may be supported on drilled, cast-in-place, straight-shaft friction piers. Due to shallow ground water and the requirements for pier bottom observations, end bearing piers are not recommended. Conventional slab-on-grade pavements may be used with the methods described below; however, measures should be taken to reduce the potential for distress to the floor slab if this fill were to settle over time. Possible measures would include designing lower level garage slabs as structural slabs, or providing individual deep support for floor slabs (i.e. piers). Partial mitigation measures could include removing and recompacting the upper portion of the fill. However, partial mitigation measures will present a significant risk of poor performance. In our opinion, removing and recompacting the upper 4 feet of the fill would reduce the potential for slab distress, but some future settlement and slab cracking may still occur.

8.3.1 Vertical Loads

The piers should have a minimum diameter of at least 18 inches and extend to a depth of at least 40 feet below the existing ground surface. Piers may be designed for the

allowable skin friction values presented below for combined dead plus live loads with a one-third increase allowed for either transient wind or seismic loading. These allowable skin friction values are based upon factor of safety of 1.5 for dead plus live loads.

At this time, the slot fill depths are not well known. We have broken the site into two zones of fill thickness. Zones A and B are shown on the Site Plan, Figure 2, and represent 15 and 20 feet of fill, respectively. The vertical capacity of the existing site fills should be neglected. Downdrag forces also could be experienced due to future consolidation of the existing fills. As a result of the settlement or consolidation of the fill, negative skin friction or downdrag will act on the piers. The downdrag force or negative skin friction and allowable skin friction for drilled piers are presented below. If additional site investigation is performed to better determine the slot fill locations and depths, these recommendations may be revised.

Table 10. Allowable Drilled Pier Skin Friction Values

Depth Below Existing Grade (feet)	Downdrag or Negative Skin Friction (psf)	Allowable Skin Friction (psf)
5 to 12	850	--
12 to 18	425	--
15 to 20	--	330
20 to 30	--	650
30 to 50	--	900
50 to 80	--	1,000

Piers should have a minimum center-to-center spacing of at least three pier diameters. Grade beams should be designed to span between piers in accordance with structural requirements. Resistance to uplift loads will be developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 80 percent of the allowable skin friction be used.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil before reinforcing steel is installed and concrete is placed. We recommend that the excavation of all piers be performed under direct observation of the Geotechnical Engineer to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

Due to the high ground water and the nature of some of the sand layers, casing of each shaft may be necessary. If water cannot be removed from excavations prior to concrete placement, then concrete will need to be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole. If permanent steel casing is required, the frictional capacity should be reduced by 20 percent over the length of casing used.

Total settlement for the recommended pier foundations should not exceed ½-inch and post construction differential settlement across the building founded on piers should be less than ½-inch due to static loads. Since the length of the pier extended beyond

the potentially liquefiable sand layers, no liquefaction-induced differential settlement should be added.

8.3.2 Lateral Loads

To estimate the lateral capacities of the piers we used the computer program LPILE that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piers to resist the expected lateral loads. The magnitude of the lateral load resistance is dependent upon many factors, including pier stiffness and embedment length, conditions of fixity at the pier cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pier. The proposed piers would gain their lateral support primarily from the existing fill and should be designed for the deflections and maximum bending moments shown in Table 13 below. We performed our analysis for 18-inch, 24-inch, and 36-inch piers with a free-head condition.

Table 11. Lateral Capacities of Drilled Piers

Lateral Pier Capacities - Free Head Condition						
	¼-inch Deflection			½-inch Deflection		
Pier Diameter	Maximum Shear (kips)	Maximum Moment (kip-in)	Depth to Maximum Moment (feet)	Maximum Shear (kips)	Maximum Moment (kip-in)	Depth to Maximum Moment (feet)
18-inch	34	875	4.5	46	1,380	5
24-inch	49	1,600	5.5	65	2,380	6
36-inch	80	3,480	7	105	5,000	9

The above results represent the probable response of the piers under short-term loading conditions and include no factor-of-safety. Pier stiffness values (EI) of 1.6 x 10^{10} lb-in^2, 5.1 x 10^{10} lb-in^2, and 25.7 x 10^{10} lb-in^2 have been assumed in our calculations of load deflection for the 18-, 24-, and 36-inch piers, respectively. A minimum compressive strength of 3,000 pounds per square inch was assumed for concrete modulus calculations. If desired, we can modify our analysis, if necessary.

For calculation of allowable passive soil resistance against pier caps poured neat against native soils, for the free-head condition, we recommend a uniform pressure (rectangular distribution) of 350 psf for a pile head deflection of ¼-inch, and 700 psf for a pile head deflection of ½-inch. These values were developed to maintain strain compatibility between the piers and pier caps. Because of the potential for disturbance, the top foot of soil against the pier caps should be neglected when calculating passive resistance.

8.4 Friction Pre-Cast Concrete Piles

Precast, prestressed, driven concrete piles could be used to support the anticipated building loads with only minor settlement. Random and difficult driving conditions should be anticipated due to the presence of dense to very dense sands at variable depths and locations on the site. Where pre-drilling is needed, the maximum diameter

of pre-drilling should not exceed the width of the piles. Pre-drilling should not be allowed within 10 feet of the specified tip elevation of the piles.

Conventional slab-on-grade pavements may be used with the methods described below; however, measures should be taken to reduce the potential for distress to the floor slab if this fill were to settle over time. Possible measures would include designing lower level garage slabs as structural slabs, or providing individual deep support for floor slabs (i.e. piles). Partial mitigation measures could include removing and recompacting the upper portion of the fill. However, partial mitigation measures will present a significant risk of poor performance. In our opinion, removing and recompacting the upper 4 feet of the fill would reduce the potential for slab distress, but some future settlement and slab cracking may still occur.

8.4.1 Vertical Loads

Our exploration indicates there is no significantly thick or continuous dense sand layer that would provide suitable end-bearing support. Therefore, pile support is expected to come predominantly from frictional along the sides of the pile. Our borings and CPTs did indicate, however, that there is a dense sand layer approximately 5- to 10-feet-thick that occurs at a depth of about 45 to 50 feet below grade. Due to the thickness, driven piles may experience high driving stresses and may encounter shallow refusal on the sand layer without achieving the design tip elevation, should it be lower.

Column loads on the order of 630 to 1,200 kips for dead plus live loads are expected. We computed allowable downward vertical capacities for 12-, 14-, and 16-inch-square concrete piles, the results of which are presented on Figures 5A and 5B. As discussed in the "Drilled Pier" section and as shown on Figures 5A and 5B, the upper 15 feet in Zone A and 20 feet in Zone B from existing site grades should be neglected when determining vertical pile capacity. In addition, downdrag forces should be included in pile design when evaluating the vertical pile capacity. If additional site investigation is performed to better determine the slot fill locations and depths, these recommendations may be revised. The indicated capacities are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed 80 percent of the computed downward capacities. The pile capacities shown on Figures 5A and 5B and uplift loads may be increased by one-third under transient loading, including wind and seismic.

Gross capacity of the piles should not exceed the structural capacity, which is estimated at 125, 170, and 220 tons for typical 12-, 14-, and 16-inch-square pretensioned, prestressed concrete piles, respectively (Santa Fe-Pomeroy, Inc.). We have assumed a base of pile cap at 5 feet below existing site grades for our analyses. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile width.

Based on the maximum allowable loads for a single pile on Figures 5A and 5B, we estimate total settlements of about ½-inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of about ¼-inch should be considered.

8.4.2 Lateral Loads On Piles

Lateral load resistance for pile-supported structures may be developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile.

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 12-, 14-, and 16-inch-square, driven concrete piles with free-head conditions are presented in Tables 12 and 13. The reported values represent the probable response of the piles under short-term loading conditions and do not include a factor of safety. A minimum concrete compressive strength of 6,000 pounds per square inch (psi) and pile stiffness values (EI) of 7.4×10^9 lb-in^2, 13.9×10^9 lb-in^2, and 23.6×10^9 lb-in^2 were used in our calculations of load deflection for 12-, 14-, and 16-inch piles, respectively.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 and smaller under lateral loading. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

For calculation of allowable passive soil resistance against pile caps poured neat against native soils, for the free-head condition, we recommend a uniform pressure (rectangular distribution) of 350 psf for a pile head deflection of ¼-inch, and 700 psf for a pile head deflection of ½-inch. These values were developed to maintain strain compatibility between the piles and pile caps, and include a factor-of-safety of at least 1.5. Because of the potential for disturbance, the top foot of soil against the pile caps should be neglected when calculating passive resistance.

Table 12. Pile Lateral Load Characteristics (¼-inch Deflection)

Lateral Pile Capacities - Free Head Condition					
	¼-inch Deflection				
	Maximum Shear (kips)	Flexural Length* (feet)	Depth to Zero Moment (feet)	Depth to Maximum Moment (feet)	Maximum Moment (kip-in)
12-inch Piles	25	6	11.5	4	550
14-inch Piles	30	6.5	13	4.5	780
16-inch Piles	36	7.5	14.5	5	1,050

*Note: Defined as length of pile from the first point of zero lateral deflection to the underside of the pile cap.

Table 13. Pile Lateral Load Characteristics (½-inch Deflection)

Lateral Pile Capacities – Free Head Condition					
	½-inch Deflection				
	Maximum Shear (kips)	Flexural Length* (feet)	Depth to Zero Moment (feet)	Depth to Maximum Moment (feet)	Maximum Moment (kip-in)
12-inch Piles	34	6.5	13	4.5	970
14-inch Piles	42	7.5	14.5	5	1,240
16-inch Piles	49	9	16	5.5	1,600

*Note: Defined as length of pile from the first point of zero lateral deflection to the underside of the pile cap.

8.4.3 WEAP Analysis

At a minimum, we recommend that the pile contractor have a wave equation analysis of piles (WEAP) performed to confirm compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. The WEAP analysis results should include at least:

1. A Bearing Graph
2. A Drivability Graph
3. A completed copy of the pile driving information form
4. An ECHO print of the WEAP analysis input data, and
5. A copy of the printed output data for both the Bearing Graph and Drivability Graph.

The WEAP analysis results should be transmitted to the Geotechnical Engineer for review at least 72 hours prior to mobilization of pile driving equipment to the site.

8.4.4 Indicator Piles

It has been our experience that uncertainties associated with production pile driving can be reduced considerably by implementing an indicator pile program. An indicator pile program will also provide a better means of confirming the limits of layers where high driving resistance may be encountered, and to more accurately estimate final pile lengths.

We recommend that at least five indicator piles be installed before the final pile casting lengths have been selected. The indicator piles should be driven with the same equipment that will be used to drive the production piles. We should review or select the indicator pile locations when structural drawings are made available. The indicator pile cast lengths should be based on the design lengths required to meet the desired capacity, plus 5 feet. It is expected that some indicator piles may not be driven to their entire length and will require cutting to provide the desired butt elevation. Indicator piles can be used for support of the structure and, therefore, should be located appropriately. We also suggest that one or more spare piles be delivered to the site during the indicator program.

8.4.5 PDA Monitoring

We recommend PDA monitoring during the indicator pile program and CAPWAP analyses to determine approximate pile capacities through dynamic testing. PDA monitoring may allow a reduction in production pile lengths and thus cost savings to the project. PDA monitoring should be performed during indicator driving and on selected piles for restrikes. It is preferred to have restrikes perform no sooner than seven days after initial driving. *Please note that restrike testing more than one day after installation may significantly alter the contractor's sequencing. Therefore, if restrike testing is selected for this project, is should be clearly identified on the plans and specifications to avoid unexpected costly change-orders for out of sequence moves.* PDA monitoring would also be especially beneficial for checking tensile stresses in the piles and for evaluating pile integrity on any piles suspected of being damaged during indicator or production driving. Piles designated for PDA monitoring during indicator pile installation should be at least 10 feet longer than design length so that the gauges are not driven into the ground.

8.4.6 Production Pile Installation

We recommend that a pile hammer capable of delivering a minimum rated driving energy of 60,000 foot-pounds be used. If indicator piles are installed, the same hammer should be used for both the indicator piles and the production piles. The pile contractor should perform wave equation analysis to confirm the compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the wave equation results prior to mobilization of pile driving equipment to the site.

Since the piles are designed for skin friction support, they should be driven to the desired tip elevation. If difficult driving conditions are encountered, we should review the driving record and evaluate potential tip capacity to allow reduction in pile length. We may also recommend that a Pile Driving Analyzer (PDA) be used during production driving to determine approximate pile capacities through dynamic analyses. PDA monitoring would be especially beneficial for checking restrike capacities of any piles short of required tip elevation or for evaluating pile integrity on any piles suspected of being damaged during driving. We should observe all indicator and production pile installation on a full-time basis.

8.5 Augercast Piles

Alternatively, the proposed parking structure may be supported on the Augercast pile foundation system. Augercast piles are cast-in-place concrete piles that are drilled using a hollow-stem auger and pumping sand-cement grout through the bottom of the auger as the auger is retracted. Two types of augercast piles are available: APG piles, which like piers, remove the soil column and replace it with grout; and the APGD piles, which displace the soil prior to grout placement. Augercast piles are a low noise and vibration installation compared to driven piles, and would not require pre-drilling through the dense sand layers. At this time, Berkel & Company is not sure that displacement (APGD) piles are suitable with the subsurface profile; however, they were successfully installed in downtown San Jose in similar conditions.

Various types of steel reinforcing including rebar cages or H-piles may be installed into the still wet grout after drilling. If you desire design parameters, please contact our office and we can put you and the project structural engineer in contact with Berkel & Company, a licensed Augered Cast-in-Place Piles design-builder.

Conventional slab-on-grade pavements may be used with the methods described below; however, measures should be taken to reduce the potential for distress to the floor slab if this fill were to settle over time. Possible measures would include designing lower level garage slabs as structural slabs, or providing individual deep support for floor slabs (i.e. piles). Partial mitigation measures could include removing and recompacting the upper portion of the fill. However, partial mitigation measures will present a significant risk of poor performance. In our opinion, removing and recompacting the upper 4 feet of the fill would reduce the potential for slab distress, but some future settlement and slab cracking may still occur.

8.5.1 Vertical Loads

Based on the preliminary information provided by Berkel & Company, conventional APG piles will have about the same vertical capacity as driven concrete piles, and APGD piles will have about 20 percent higher vertical capacity compared to APG piles. The typical diameters for APG piles are 14-, 16-, 18-, and 24-inch piles; the typical diameters for APGD piles are 14-, 16-, and 18-inch piles. We understand from Berkel & Company that 24-inch APGD piles are not available.

As discussed in the "Drilled Pier" section and as shown on Figures 5A and 5B, the upper 15 feet in Zone A and 20 feet in Zone B from existing site grades should be neglected when determining vertical pile capacity. In addition, downdrag forces should be included in augercast pile design when evaluating the vertical pile capacity. The design-build contractor should incorporate this into the design.

Augercast piles should have a center-to-center spacing of at least three pile diameters. Grade beams and floors should be designed to span between the piles in accordance with structural requirements.

Total settlement of properly constructed auger-cast piles constructed with the minimum dimensions recommended above should not exceed 1-inch with estimated post construction differential settlement between pile caps of less than ½-inch due to static loads. Estimated pile capacities and settlements should be confirmed in the field by load tests as described in Section 8.5.3.

8.5.2 Lateral Load Capacity

Lateral load resistance for pile-supported structures will be developed through pile bending and soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile. Assuming there is similarity in these variables to pre-cast, prestressed concrete piles, lateral load capacity should be similar. As our lateral load analyses for pre-cast piles were based on a concrete compressive strength of 6,000 psi and augercast piles are typically constructed with a grout compressive strength of 4,000 psi, the lateral capacities will

be less than provided for pre-cast. We should review these parameters to estimate lateral load capacities if augercast piles are the preferred foundation type.

8.5.3 Pile Load Testing

We recommend that pile load tests be performed on at least three piles at the beginning of pile installation to confirm the capacity and settlement behavior of the augercast piles. The locations and specific piles to be tested should be selected by Lowney Associates with input from the pile contractor so as to address pile behavior in the various subsurface conditions that are present at the site. Installation of all augercast piles should be continuously observed and monitored by a member of our staff.

9.0 CONCRETE SLABS

As previously discussed, undocumented fills on the order of 2½ to 20 feet were encountered at the site. New concrete slab-on-grade pavements and flatwork constructed over undocumented fill may experience distress over time, such as localized settlement and/or irregular surface grades. If desired to reduce the potential for slab cracking and future maintenance/repair, we recommend that consideration be given to removing and recompacting the upper portion of the undocumented fill as discussed in "Removal of Existing Fill" section. If complete mitigation of the potential for distress is desired, then complete removal and replacement would be required or in-situ treatment. Alternatively, a structural slab may be designed.

9.1 Portland Cement Concrete Pavements

Portland Cement Concrete (PCC) pavements supporting automobile wheel loads should be a minimum of 5 inches thick. If heavier vehicular loading is anticipated (i.e. garbage trucks), the concrete thickness should be increased to at least 6 inches. Our design is based on an R-value of 10 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction, control, and expansion joints be used in design of the Portland Cement Concrete pavements to control the cracking inherent in this construction.

9.2 Interior Slabs-on-Grade (Non-Pavement)

At this time we are not aware of any interior slabs-on-grade other than stairwells. Due to the high expansion potential of on-site surface soils, we recommend that concrete interior slabs-on-grade be underlain by at least 12 inches of non-expansive fill (NEF) to reduce the likelihood of slab damage from heave. If a moisture barrier system as described below is used, the thickness of the gravel layer can be included as part of the NEF requirement. In addition, we recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been cast. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth and firm surface for slab support. The slab thickness and amount of reinforcing steel should be

determined by the project structural engineer in accordance with the anticipated use and loading of the slab.

Post-construction cracking of concrete slabs-on-grade is inherent in any project, especially where soil is expansive. In our opinion, consideration should be given toward a maximum control joint spacing of 10 feet in both directions for conventional, interior slab-on-grade construction.

9.3 Moisture Protection Considerations

Since the long-term performance of concrete slabs-on-grade depends on good design, workmanship, and materials, the following general guidelines are presented for consideration by the owner, design team, and contractor. We note that some of these guidelines are different from local practice, and emphasize that they should be considered as an owner's option.

The purpose of these guidelines is to aid in producing concrete slabs of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with slab construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- We recommend a minimum 10-mil-thick vapor barrier or better be placed directly below the slab-on-grade floors. A higher quality vapor barrier may be used at the owner's option. The vapor barrier should extend to the edge of the slab-on-grade floors and mat foundation. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The crushed rock should be consolidated in place with vibratory equipment. The crushed rock may be included in the required NEF thickness. The vapor barrier should be sealed at all seams and penetrations.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- If possible, hard troweling to a polished finish should be avoided since it can seal the slab surface and trap excessive moisture inside the concrete to be later released. Wood-floating followed by a light broom finish is recommended.

- When using Type I cement, all concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 7 days. When using Type II cement, all concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 14 days. The initial curing time for Type II cement concrete can be reduced to 5 days by reducing the concrete water/cement ratio to less than 0.40.

- Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period. The use of chemical curing compounds is not advised in areas where moisture-sensitive floor coverings will be placed, unless the slab surface is shot blasted prior to placing the floor covering.

- Water vapor emission levels and pH should be determined as required by the manufacturer's of the floor covering materials before floor installation. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI) and Portland Cement Association (PCA), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration.

9.4 Exterior Flatwork

Due to the high expansion potential of the surface soils, we recommend that exterior concrete flatwork and sidewalks are at least 4 inches thick and supported on at least 6 inches of Class 2 Aggregate Base. Recycled granular materials may be reused as discussed in the "Reuse of On-site Recycled Materials" section. The subgrade and aggregate base should be prepared and compacted in accordance with the recommendations presented in the "Earthwork" section. If concrete flatwork is subject to wheel loads, the recommendations presented in Section 9.1 should be used.

The contractor should take special care that the subgrade beneath structural soil, and especially beneath adjacent standard flatwork sections, does not dry out and dessicate. Differential expansion can occur at the transitions.

9.5 Concrete Pavers

If PCC paving blocks are planned for entranceways, we recommend that concrete pavers and bedding sand be supported on a 6-inch-thick PCC section designed in accordance with Section 9.1 above. The PCC sub-slab should be constructed to also provide edge restraint for the overlying pavers and bedding sand. In addition, the 1-inch leveling course placed beneath pavers may shift after construction. It would therefore be beneficial to check that no more than approximately 1-inch of bedding sand is placed during construction.

9.6 Pavement Cutoff

Because the native soils at the site are moderately to highly expansive, surface water infiltration beneath exterior pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect exterior at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

9.7 Aggregate Base and Subgrade

Aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

10.0 RETAINING WALLS

10.1 Lateral Earth Pressures

Any proposed conventional retaining walls, such as block masonry, wood walls, or cast-in-place concrete should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill as well as from any surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls be designed to resist the lateral earth pressures presented in Table 14 below.

Table 14. Conventional Retaining Wall Lateral Earth Pressures

Backfill Inclination (horizontal:vertical)	Equivalent Fluid Pressure*	
	Unrestrained	Restrained
Level	45 pcf	45 pcf + 8H psf
3:1	55 pcf	55 pcf + 8H psf
2.5:1	60 pcf	60 pcf + 8H psf
2:1	65 pcf	65 pcf + 8H psf

*Assumes drained conditions; add 40 pcf to the above values for undrained conditions.
H is the distance in feet between the bottom of the footing and the top of the retained soil.

Unrestrained walls should also be designed to resist an additional uniform pressure equivalent to one-third of any uniform surcharge loads applied at the surface; restrained retaining walls should also be designed to resist an additional uniform pressure equivalent to one-half of any uniform surcharge loads.

The above lateral earth pressures assume sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a

rise in the ground water level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended in Table 8 for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture would be undesirable.

10.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. Walls supporting less than 4 feet of material, including soil and concrete slabs, do not need to be drained. The drainage system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively impervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

10.3 Backfill

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

10.4 Foundation

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Footings" section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the "Lateral Loads" section.

12.0 LIMITATIONS

This report has been prepared for the sole use of International Parking Design, Inc., specifically for design of the Santa Clara Parking Structure and Pedestrian Bridge in Santa Clara, California. The opinions presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discreet locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Lowney Associates cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Lowney Associates' report by others. Furthermore, Lowney Associates will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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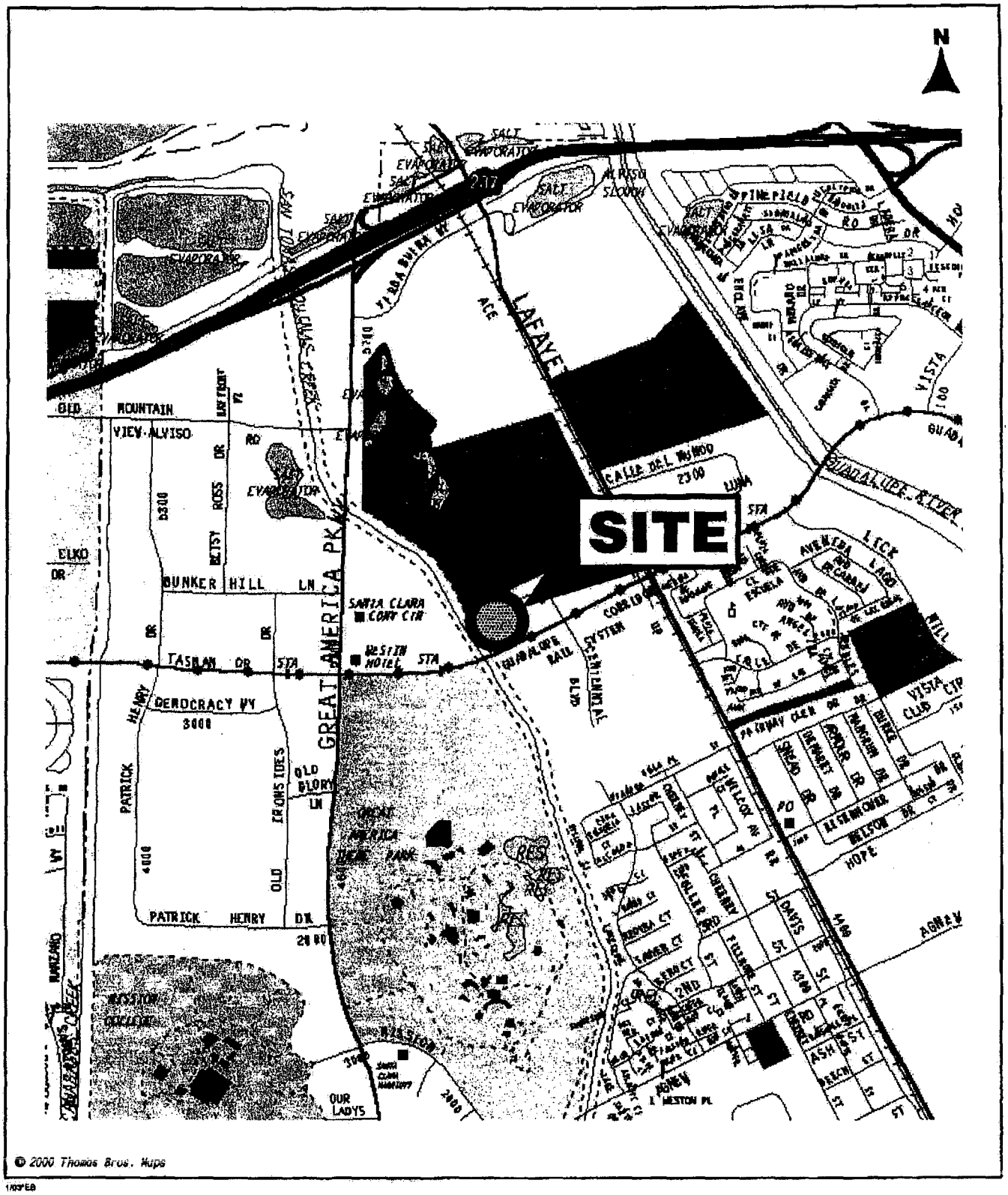


VICINITY MAP

SANTA CLARA PARKING STRUCTURE
Santa Clara, California

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 1
1883-1

LEGEND

- Approximate location of exploratory boring
- Approximate location of cone penetration test
- Approximate location of bulk sample
- Approximate location of cross section

Zone A: Area with fill depth assumed to be 15 feet thick
Zone B: Area with fill depth assumed to be 20 feet thick

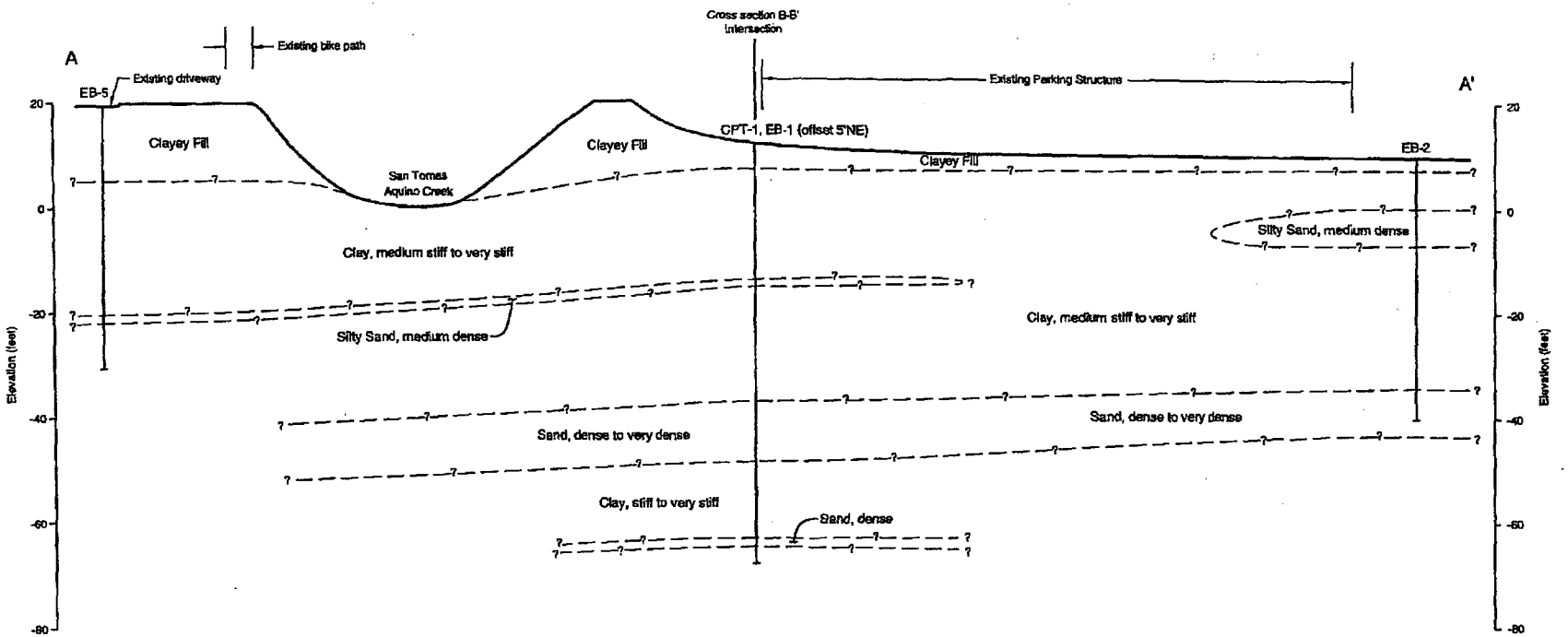
0 — 60
Scale — feet

Base topo by BKF Engineers.

SITE PLAN
SANTA CLARA PARKING STRUCTURE
Santa Clara, California

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 2
1883-1



Horizontal Scale: 1"=40'
Vertical Scale: 1"=20'

CROSS SECTION A-A'
SANTA CLARA PARKING STRUCTURE
Santa Clara, California

LOVNEY ASSOCIATES
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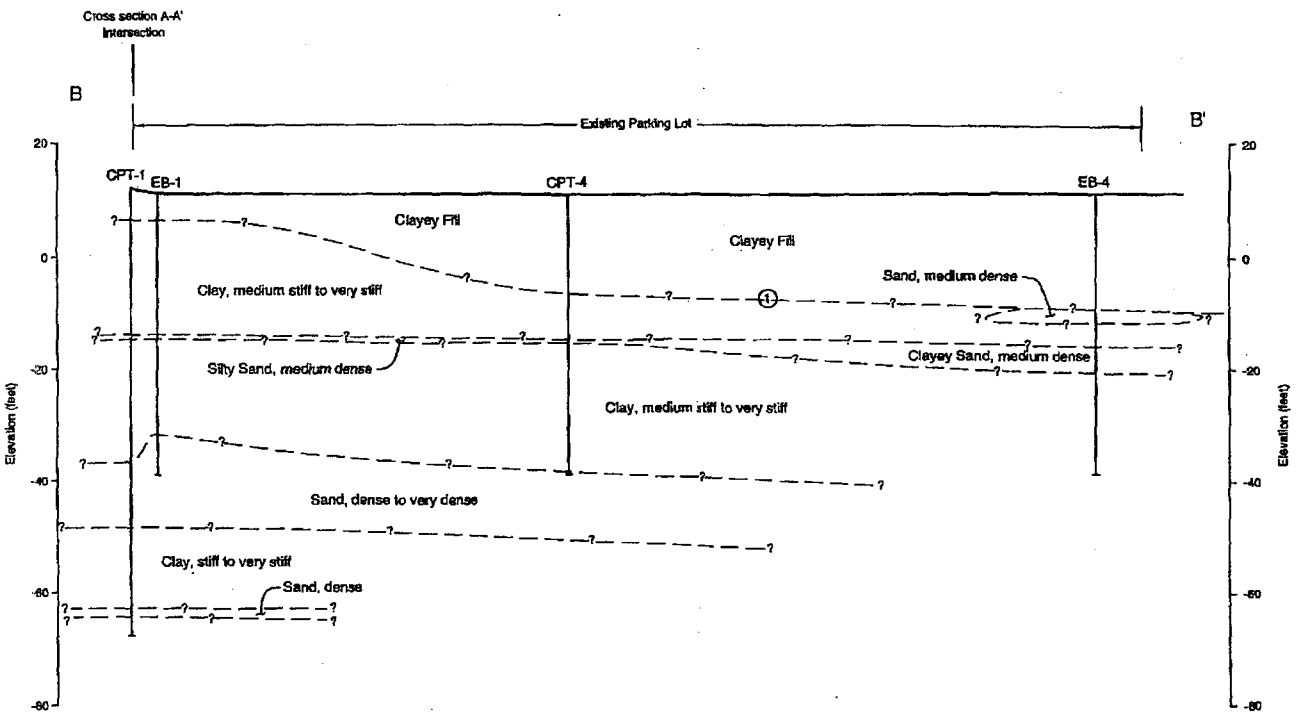
FIGURE 3A
1883-1



① Bottom elevation of fill inferred. We understand that slot trench fills perpendicular to the cross section were constructed at variable depths.

Horizontal Scale: 1"=40'
Vertical Scale: 1"=20'

CROSS SECTION B-B'
SANTA CLARA PARKING STRUCTURE
Santa Clara, California

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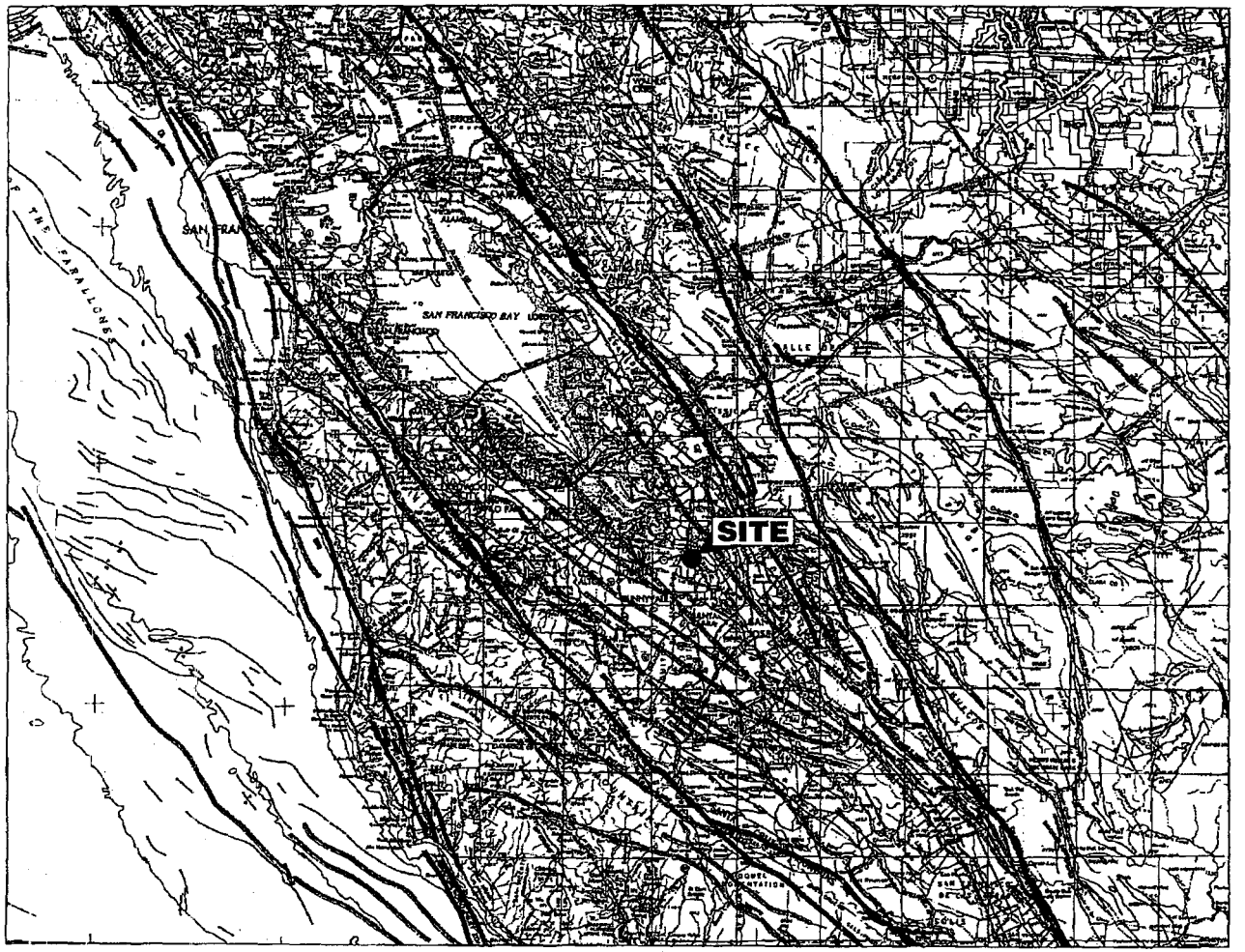
FIGURE 3B
1883-1



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906; includes areas of known fault creep).
		Holocene	200			Displacement during Holocene time[2]
			10,000			Faults showing evidence of displacement during late Quaternary time.[3,4]
	Early Quaternary	Pleistocene	700,000			Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of middle Miocene and Pliocene age.
Pre-Quaternary	Pliocene		2,000,000			
	Miocene		5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122–A1–TF–250–00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120–A1–TF–250–00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000–meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

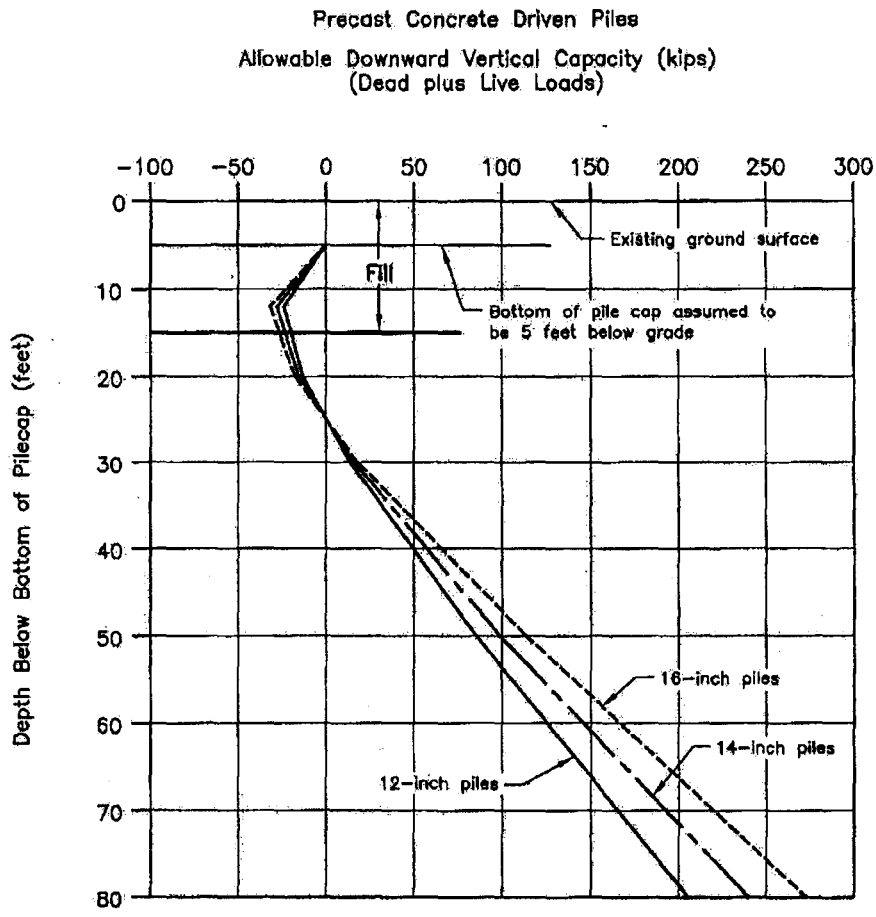
Scale 1:500,000

Division of Mines and Geology, James F. Davis, State Geologist

REGIONAL FAULT MAP
SANTA CLARA PARKING STRUCTURE
Santa Clara, California

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 4
1883-1

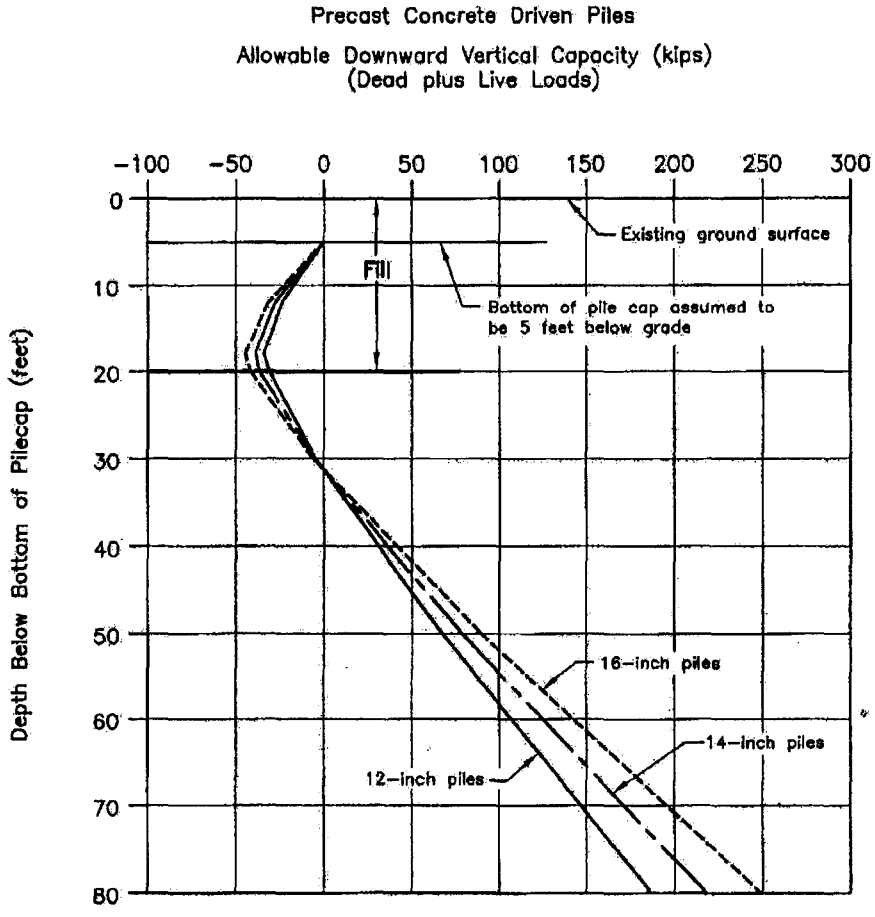


VERTICAL PILE CAPACITIES - ZONE A

SANTA CLARA PARKING STRUCTURE
Santa Clara, California

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 5A
1883-1



VERTICAL PILE CAPACITIES - ZONE B

SANTA CLARA PARKING STRUCTURE
Santa Clara, California

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 5B
1883-1

APPENDIX A

FIELD INVESTIGATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, CPT and drilling equipment. Five 8-inch-diameter exploratory borings were drilled on December 11 to 13, 2002 to a maximum depth of 50 feet. In addition, five CPTs were hydraulically advanced to a maximum depth of 80 feet. CPT data was obtained at 0.16 feet intervals, and consisted of cone tip resistance, local friction, pore pressure and other parameters. The data obtained was correlated using the references cited, to determine the indicated soil type, shear strength, equivalent Standard Penetration Test (SPT), N-value (blows per foot), and other parameters. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Our boring and CPT logs, as well as a key to the classification of the soil, are included as part of this appendix.

The boring and CPT locations were determined by using portable Global Positioning System (GPS) hand-held equipment and existing site boundaries for reference. Elevations of the borings were interpolated from survey plan contours. The locations and/or elevations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. [Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed.] Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE)	CLEAN GRAVELS (Less than 5% Fines)	GW	Well graded gravels, gravel–sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel–sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel–sand–silt mixtures, plastic fines
			GC	Clayey gravels, gravel–sand–clay mixtures, plastic fines
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE)	CLEAN SANDS (Less than 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand–silt–mixtures, non–plastic fines
			SC	Clayey sands, sand–clay mixtures, plastic fines
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT IS LESS THAN 50 %)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (LIQUID LIMIT IS GREATER THAN 50 %)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08, 0.4, 2, 5, 19, 76mm

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION

MODIFIED CALIFORNIA

D&M UNDERWATER SAMPLER

SHELBY TUBE

NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0–4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0–1/4	0–2
SOFT	1/4–1/2	2–4
MEDIUM STIFF	1/2–1	4–8
STIFF	1–2	8–16
VERY STIFF	2–4	16–32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE A-1

EXPLORATORY BORING: EB-1

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-11-02 FINISH DATE: 12-12-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION: 11 FT. (+/-)						
11.0 / 10.9	0	1 inch asphalt concrete						
		SANDY CLAY WITH GRAVEL (CL) [FILL] very stiff, moist, brown and dark brown mottled, fine to coarse sand, fine to coarse gravel, low plasticity	CL, FILL	18		17	106	
		stiff		23		24	96	
6.8	5	**FAT CLAY (CH)** very stiff, moist, dark brown, high plasticity	CH	19		33	97	
2.5	10	**LEAN CLAY (CL)** very stiff, moist, light brown, some fine to coarse sand, low to moderate plasticity		25		29	97	
		stiff	CL			28	97	
	15			13		28	97	
-6.0		**SANDY LEAN CLAY (CL)** stiff, moist, gray and brown mottled, fine sand, low plasticity		31		20	111	
	20		CL					
	25			23		21	108	
-16.0		**LEAN CLAY (CL)** stiff, moist, brown, some fine sand, low plasticity	CL	28		30	94	
-19.0	30	*Continued Next Page*						

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 10.0 FEET

Northing: 1,973,065
Easting: 6,134,063

LA CORP.GDT 9/16/03 MV FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-11-02 FINISH DATE: 12-12-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-19.0	30	**LEAN CLAY (CL)** stiff, moist, brown, some fine sand, low plasticity	CL	65		21	111	
	35	very stiff						
-26.0		**SANDY LEAN CLAY (CL)** very stiff, moist, brown, fine to medium sand, low plasticity	CL	38		18	117	
	40							
-32.0		**POORLY GRADED SAND WITH CLAY (SP-SC)** dense to very dense, wet, gray and brown, medium sand, some fine and coarse sand	SP-SC	50/6"		14		5
	45							
		some fine gravel		54		11		8
-39.0	50	Bottom of Boring at 50 feet						
	55							
	60							

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

GROUND WATER OBSERVATIONS:
∇: FREE GROUND WATER MEASURED DURING DRILLING AT 10.0 FEET

Northing: 1,973,065
Easting: 6,134,063

LA CORP.GDT 9/16/03 M/F FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-11-02 FINISH DATE: 12-11-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION: 10 FT. (+/-)						
9.5								
9.3	0	**GRAVELLY SAND (SP) [FILL]** dry to moist, gray and brown, fine to medium sand, some coarse sand, fine to coarse gravel	SP, FILL					
			CH, FILL	25		20	104	
7.3		**FAT CLAY (CH) [FILL]** very stiff, moist, dark brown with light brown mottles, trace fine gravel, high plasticity Plasticity Index = 33, Liquid Limit = 52	CH	37		26	102	
	5	**FAT CLAY (CH)** very stiff, moist, dark brown, high plasticity		25		21	112	
3.5		**LEAN CLAY (CL)** very stiff, moist, brwn with light brown mottles, trace fine sand, low to moderate plasticity	CL					
		medium stiff		19		20	110	
-0.3	10	**POORLY GRADED SAND WITH SILT (SP-SM)** medium dense, brown, fine to medium sand	SP-SM	22		14		11
-1.8		**SILTY SAND (SM)** medium dense, moist, brown with gray mottles, fine sand	SM	11		19 / 25		39 / 34
	15							
-7.0		**SANDY LEAN CLAY (CL)** medium stiff, wet, brown with gray mottles, fine sand, low plasticity	CL	16		20	112	
	20							
		stiff		23		22	107	
	25					30	92	
-18.5		**LEAN CLAY WITH SAND (CL)** stiff, moist, gray and brown mottled, fine sand, low plasticity	CL	27		24	105	
-20.5	30							

Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,973,167
Easting: 6,134,304

LA CORP.GDT 9/16/03 MV FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-2 Cont'd

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-11-02 FINISH DATE: 12-11-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-20.5	30	**LEAN CLAY WITH SAND (CL)** stiff, moist, gray and brown mottled, fine sand, low plasticity	CL					
-23.0		**SANDY LEAN CLAY (CL)** stiff to very stiff, moist, light brown, fine to medium sand, trace fine gravel, low plasticity		35		19	117	
	35		CL	67		19	117	
	40							
-34.5		**POORLY GRADED SAND WITH SILT (SP-SM)** dense, wet, light brown, fine to medium sand		79				12
	45		SP-SM					
-37.5		**POORLY GRADED SAND WITH SILT (SP-SM)** very dense, wet, gray and brown, medium to coarse sand, some fine sand, some fine gravel	SP-SM	50/6"				6
-40.5	50	Bottom of Boring at 50 feet						
	55							
	60							

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,973,167
Easting: 6,134,304

LA CORP.GDT 9/16/03 MV" FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-3

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-11-02 FINISH DATE: 12-11-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

SURFACE ELEVATION: 11 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
10.5								
10.3	0	2 inches asphalt concrete over 6 inches aggregate base						
9.8		**SANDY LEAN CLAY (CL) [FILL]** very stiff, moist, dark brown, fine sand, some fine gravel, low to moderate plasticity	CL, FILL	27		28	91	
6.8		**CLAYEY SAND (SC) [FILL]** medium dense, moist, dark gray and brown mottled, fine to medium sand	SC, FILL	36		15		
4.8	5	**SANDY LEAN CLAY (CL) [FILL]** very stiff, moist, dark brown, fine sand, some fine to coarse gravel, low plasticity	CL, FILL	17		16	117	
2.5		**LEAN CLAY WITH SAND (CL)** medium stiff to stiff, moist, brown, fine sand, low plasticity	CL			30	92	
	10			23		23	107	7
-4.3	15	**POORLY GRADED SAND WITH SILT (SP-SM)** medium dense to dense, wet, gray and brown, fine to medium sand, some fine gravel	SP-SM	25		30		
-5.5		**LEAN CLAY WITH SAND (CL)** stiff to medium stiff, moist, brown and gray mottled, fine sand, low to moderate plasticity	CL	15		24	102	
	20			23		18	117	
	25	increased sand						
-17.5		**SANDY LEAN CLAY (CL)** medium stiff, wet, brown, fine sand, low plasticity	CL	20		24	104	
-19.5	30							

Continued Next Page

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,973,340
Easting: 6,134,044

LA CORP.GDT 9/16/03 MY* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-3 Cont'd

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-11-02 FINISH DATE: 12-11-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-19.5	30		**SANDY LEAN CLAY (CL)** medium stiff, wet, brown, fine sand, low plasticity	CL					
-21.5			**LEAN CLAY WITH SAND (CL)** very stiff, moist, brown, fine sand, low plasticity		47	X	17	117	
	35			CL					
			stiff		37	X	22	109	
-29.5	40		Bottom of Boring at 40 feet						
	45								
	50								
	55								
	60								

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,973,340
Easting: 6,134,044

LA CORP.GDT 9/19/03 MV" FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-12-02 FINISH DATE: 12-12-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION: 12 FT. (+/-)						
11.7	0	2 inches asphalt concrete over 6 inches aggregate base						
11.5								
11.0		**LEAN CLAY (CL) [FILL]** very stiff, moist, dark gray, some fine sand, some fine to coarse gravel, some asphalt fragments, low plasticity	CL, FILL	29		14	116	
8.2		**SANDY LEAN CLAY WITH GRAVEL (CL) [FILL]** stiff, moist, brown with gray mottles, fine to coarse sand, fine to coarse gravel, low plasticity	CL, FILL	32		20	117	
	5							
4.7		**FAT CLAY (CH) [FILL]** stiff, moist, dark brown with bluish gray mottles, some fine sand, trace fine gravel, high plasticity		6		26	101	
	10		CH, FILL			27	97	
				9		27	98	
-3.3	15	**FAT CLAY WITH SAND (CH) [FILL]** soft to medium stiff, wet, dark gray, fine sand, moderate to high plasticity	CH, FILL			24	102	
		encountered wood and cloth debris		7		31	91	
	20	encountered plastic debris						
-8.8		**POORLY GRADED SAND (SP)** medium dense, wet, gray and brown, fine to medium sand, some fine to coarse gravel	SP	22		15		4
-11.3		**LEAN CLAY (CL)** very stiff, moist, light brown, low plasticity	CL	20		19		
	25							
-15.3		**CLAYEY SAND (SC)** medium dense, wet, light brown, fine sand, trace fine gravel	SC	22		24		45
-18.3	30							

Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,973,362
Easting: 6,133,898

LA CORP.GDT 9/16/03 MV FILL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-4 Cont'd

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-12-02 FINISH DATE: 12-12-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

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ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-18.3	30			SC					
-19.8			**LEAN CLAY WITH SAND (CL)** very stiff, moist, light brown, fine sand, low plasticity						
	35				57		18	121	
	40		becomes stiff	CL	30		21	112	
	45				43		23	105	
-38.3	50				30		23		
			Bottom of Boring at 50 feet						
	55								
	60								

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,973,362
Easting: 6,133,898

LA CORP.GDT 9/16/03 MV FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-5

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-13-02 FINISH DATE: 12-13-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		SURFACE ELEVATION: 20 FT. (+/-)						
19.5	0	7 inches asphalt concrete over 12 inches aggregate base		50/5"		3		
18.9								
17.9		**FAT CLAY (CH) [FILL]** very stiff, moist, dark gray, some fine to coarse sand, high plasticity	CH, FILL	52		19	113	
	5							
13.0		**SANDY LEAN CLAY (CL) [FILL]** very stiff, moist, dark gray, fine sand, some fine to coarse gravel, low plasticity	CL, FILL	37		16	121	
	10							
6.0		**LEAN CLAY WITH SAND (CL)** very stiff, moist, brown, fine sand, low plasticity	CL	37		19	113	
	15							
2.5		**SANDY LEAN CLAY (CL)** medium stiff, wet, light brown, fine sand, low plasticity	CL	11		26	99	
	20					27	96	
-3.5		**LEAN CLAY (CL)** stiff, moist, light brown, low plasticity	CL	18		34	91	
	25							
-7.5		**SANDY LEAN CLAY (CL)** very stiff, wet, brown with gray mottles, fine sand, low plasticity	CL	46		18	114	
-10.5	30							

Continued Next Page

GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,972,942
Easting: 6,133,833

LA CORP.GDT 9/16/03 MV FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-5 Cont'd

DRILL RIG: FAILING 1500
BORING TYPE: ROTARY WASH
LOGGED BY: MQL
START DATE: 12-13-02 FINISH DATE: 12-13-02

PROJECT NO: 1883-1
PROJECT: SANTA CLARA PARKING STRUCTURE
LOCATION: SANTA CLARA, CA
COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-10.5	30	**SANDY LEAN CLAY (CL)** very stiff, wet, brown with gray mottles, fine sand, low plasticity	CL					
-12.5		**LEAN CLAY WITH SAND (CL)** stiff, moist, gray with brown mottles, fine sand, low plasticity	CL	27		33	94	
	35							
-19.8		increasing fine sand		29		22	106	
	40	**CLAYEY SAND (SC)** medium dense, wet, brown, fine sand	SC					25
-21.5		**LEAN CLAY (CL)** stiff, moist, gray and brown mottled, some fine sand, low plasticity	CL	33		18		
	45							
		very stiff to hard		50/6"		22	111	
				47		21		
-30.5	50	Bottom of Boring at 50 feet						
	55							
	60							

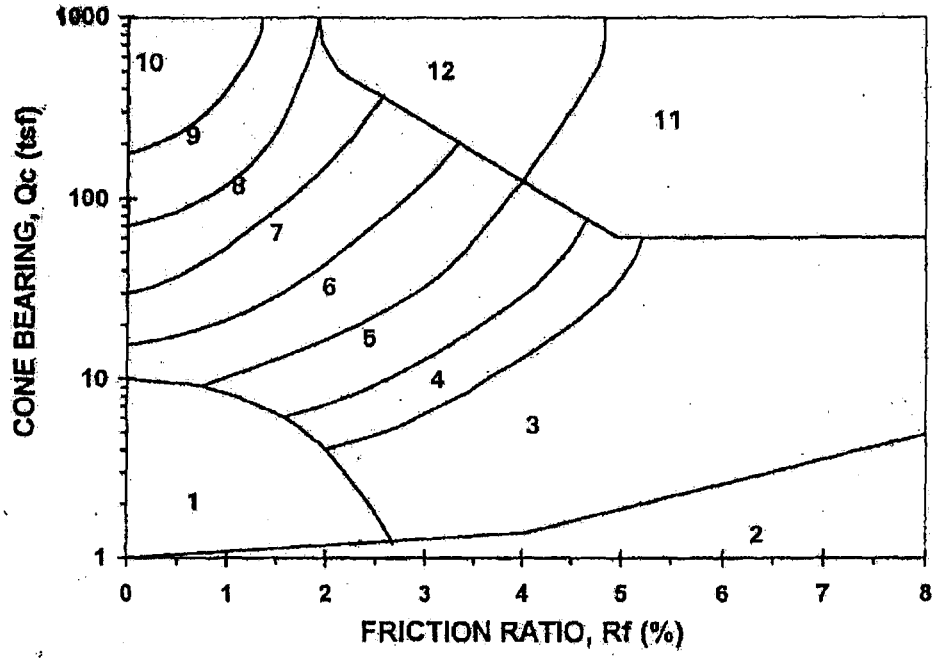
GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,972,942
Easting: 6,133,833

LA CORP.GDT 9/16/03 MV* FLL

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for Qc <= 9 tsf)	Sensitive Fine Grained
2	1	15 (10 for Qc <= 9 tsf)	Organic Material
3	1	15 (10 for Qc <= 9 tsf)	CLAY
4	1.5	15	Silty CLAY to CLAY
5	2	15	Clayey SILT to Silty CLAY
6	2.5	15	Sandy SILT to Clayey SILT
7	3	—	Silty SAND to Sandy SILT
8	4	—	SAND to Silty SAND
9	5	—	SAND
10	6	—	Gravelly SAND to SAND
11	1	15	Very Stiff Fine Grained (*)
12	2	—	SAND to Clayey SAND (*)

(*) Overconsolidated or Cemented

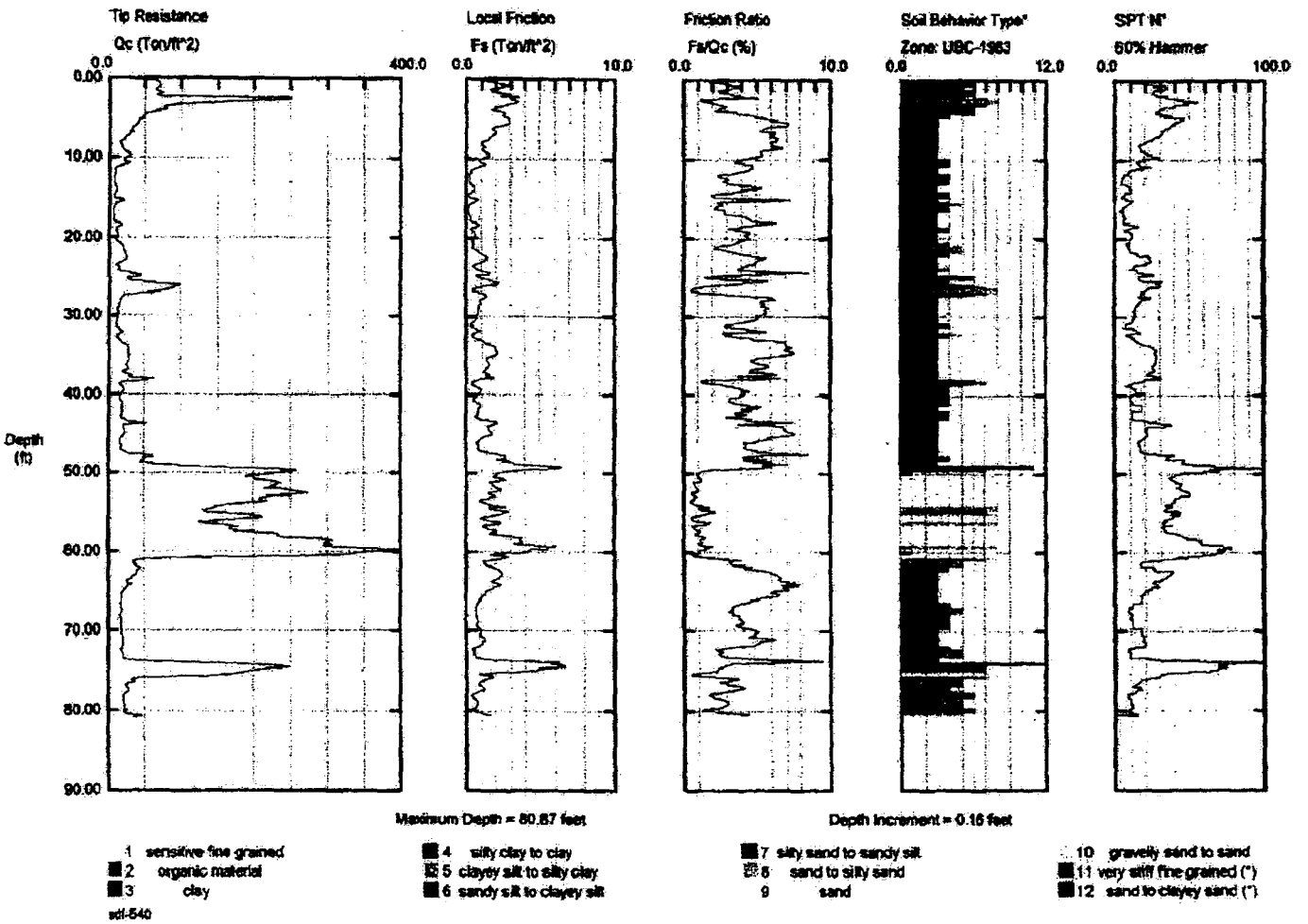
Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc*100 = Friction Ratio

References: [1]Robertson, 1986, Olsen, 1988
[2]Bonaparte & Mitchell, 1979 (young bay mud Qc <= 9)
[2]Estimated from local experience (fine grained soils Qc > 9)

Note: Testing performed in accordance with ASTM D3441

KEY TO CONE PENETROMETER TESTS

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

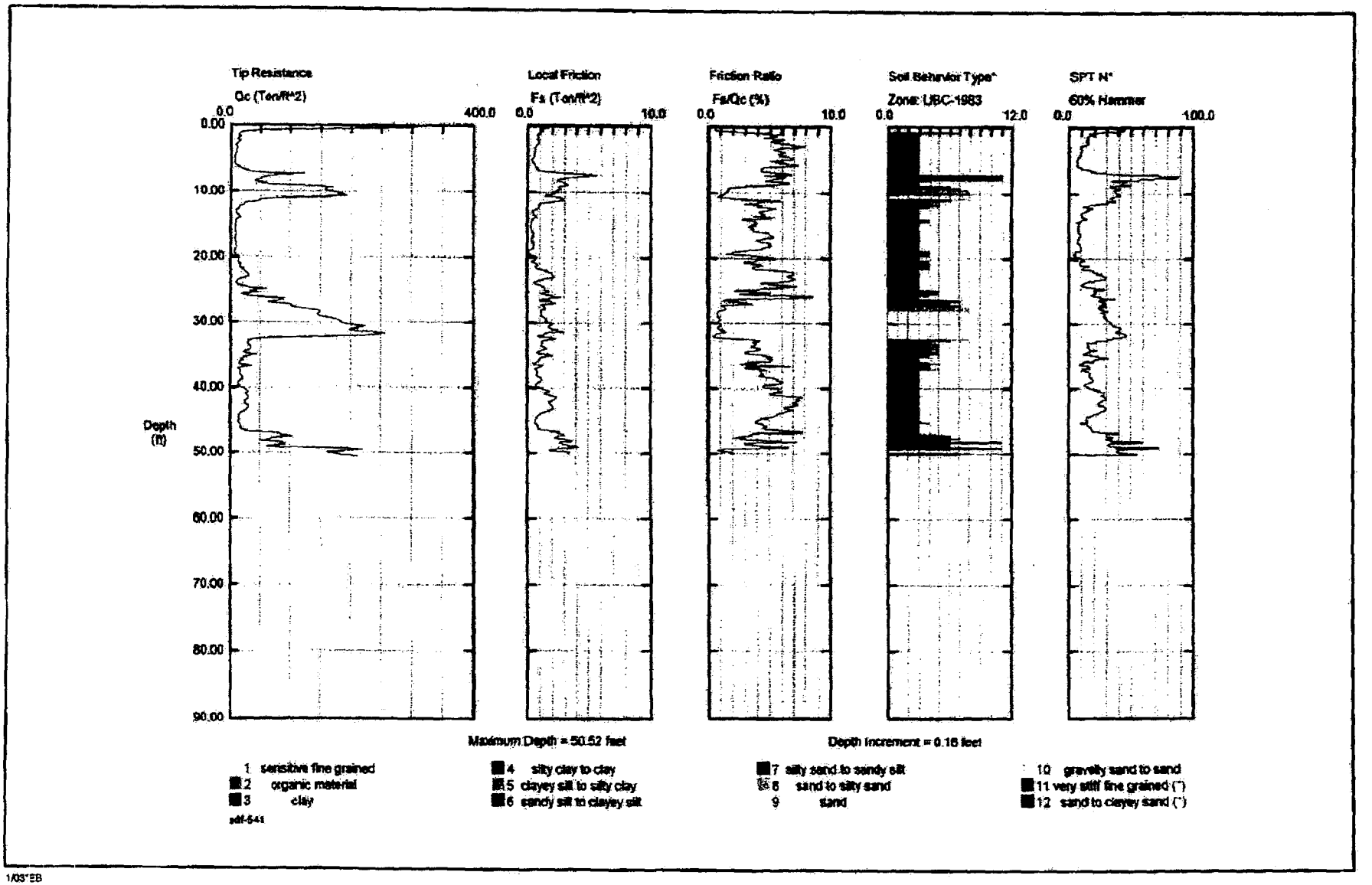


1/03 'E6

CONE PENETRATION TEST - CPT-1

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

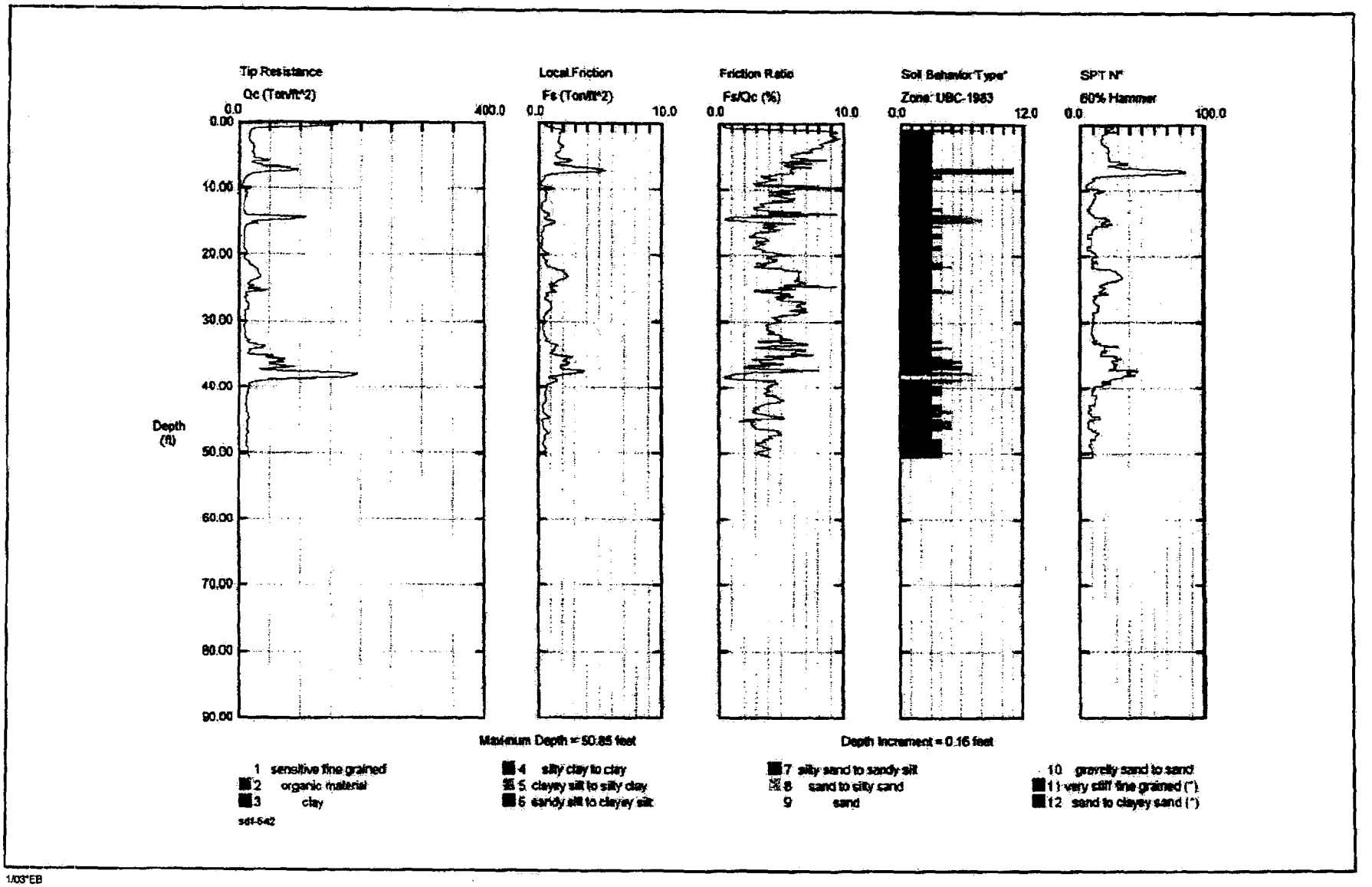
CPT-1
1883-1



1/03*EB

CONE PENETRATION TEST - CPT-2

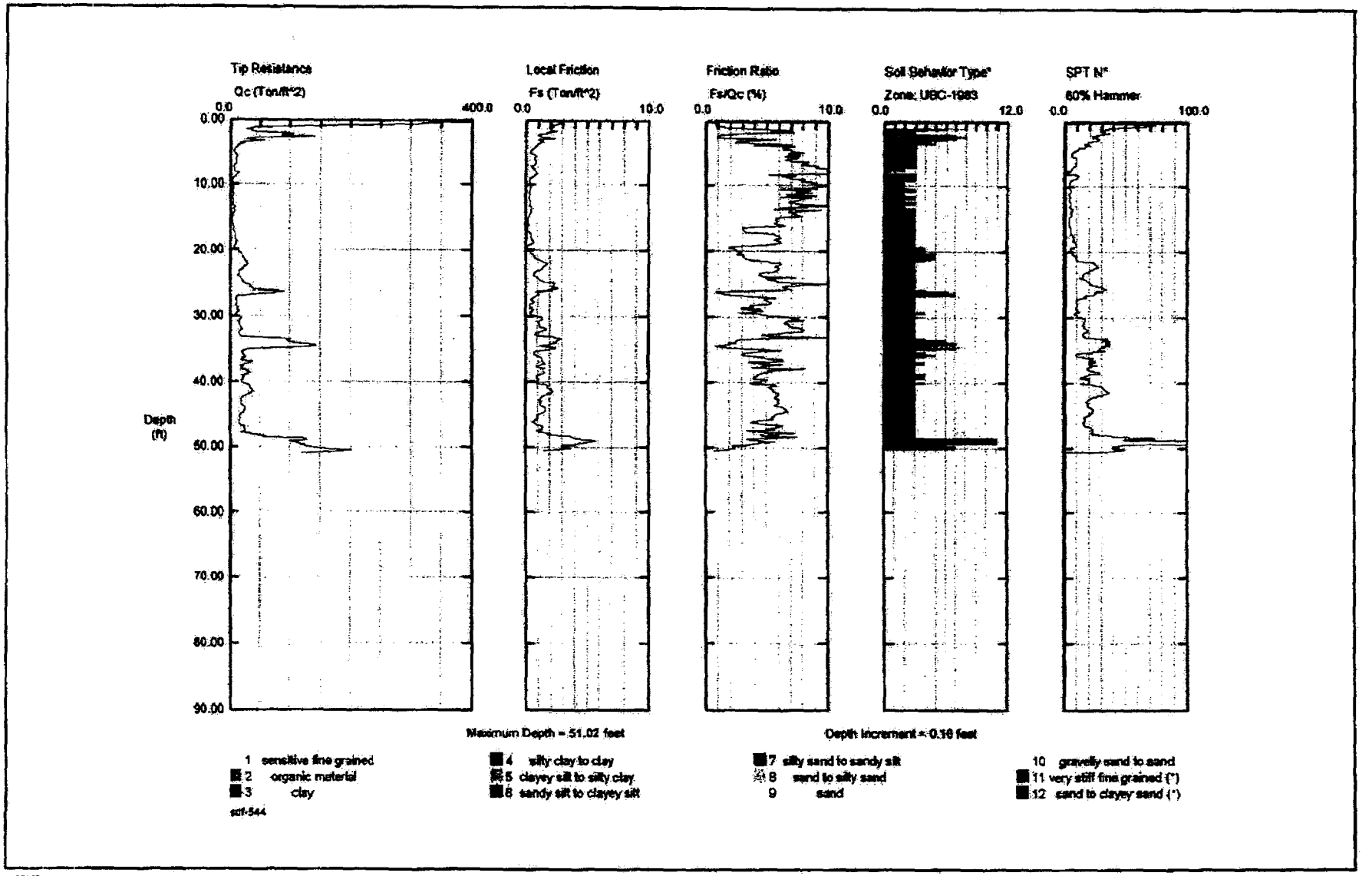
LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services



1/03*EB

CONE PENETRATION TEST - CPT-3

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

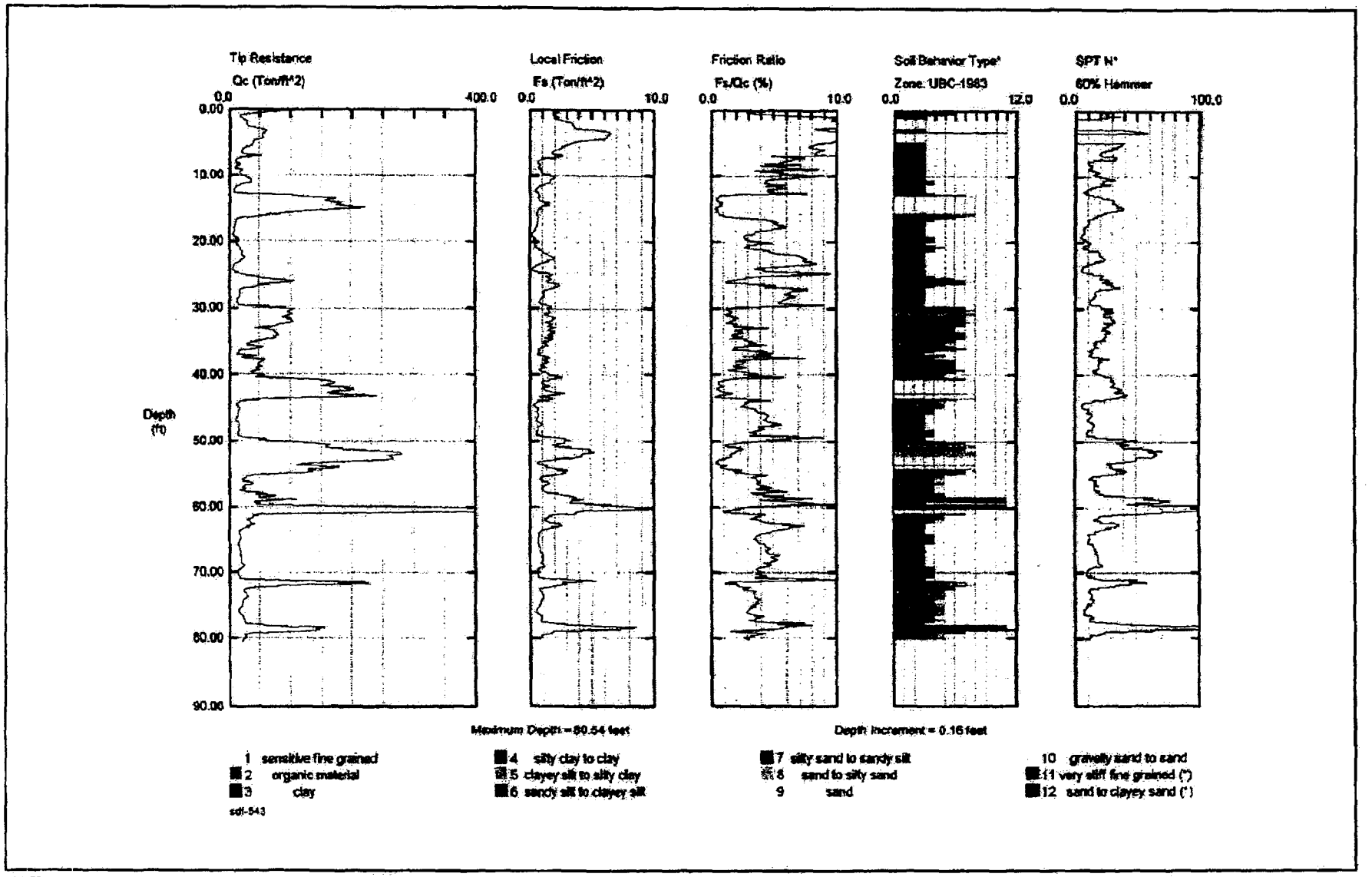


1/03*EB

CONE PENETRATION TEST - CPT-4

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

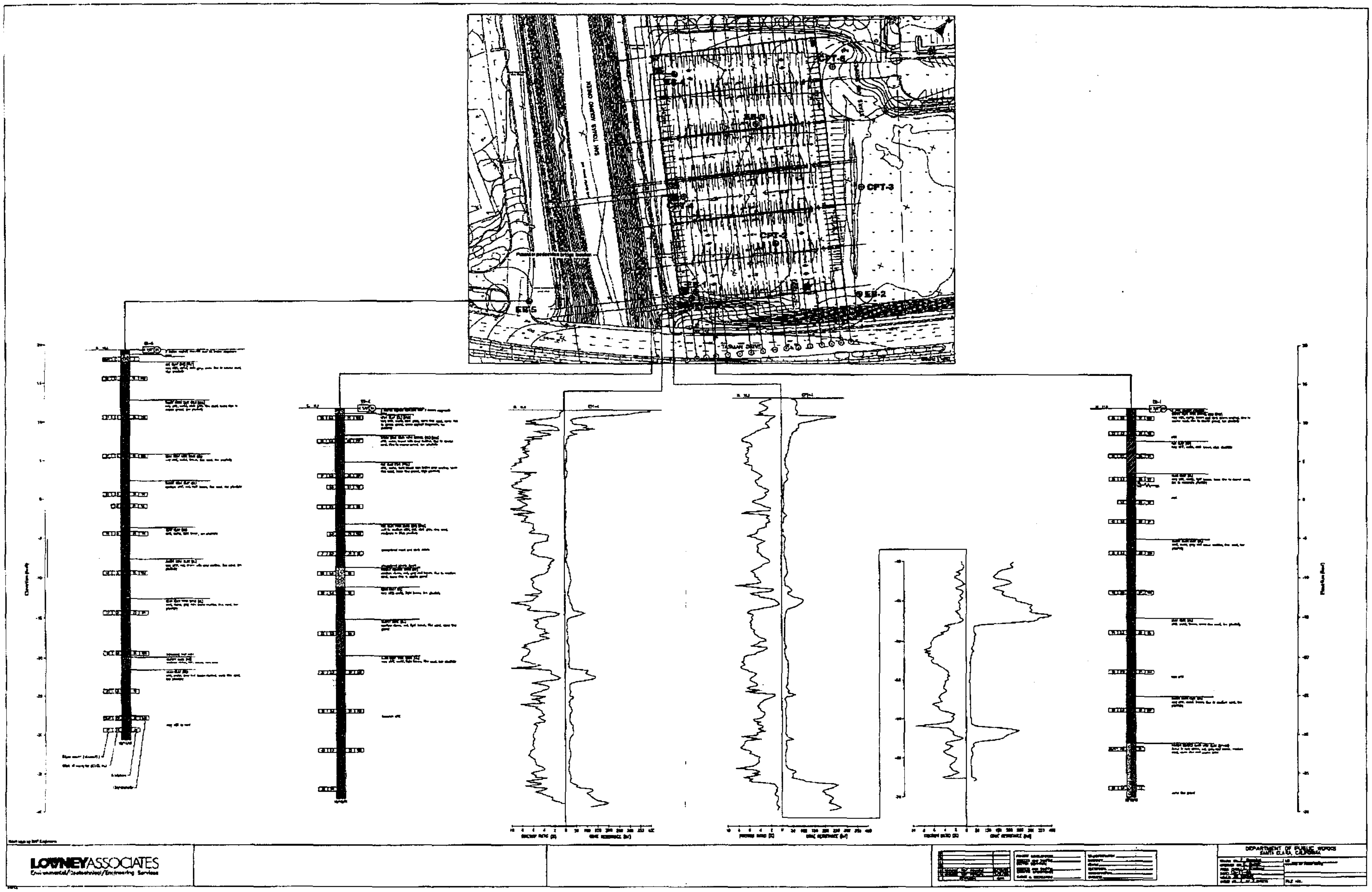
CPT-4
1883-1

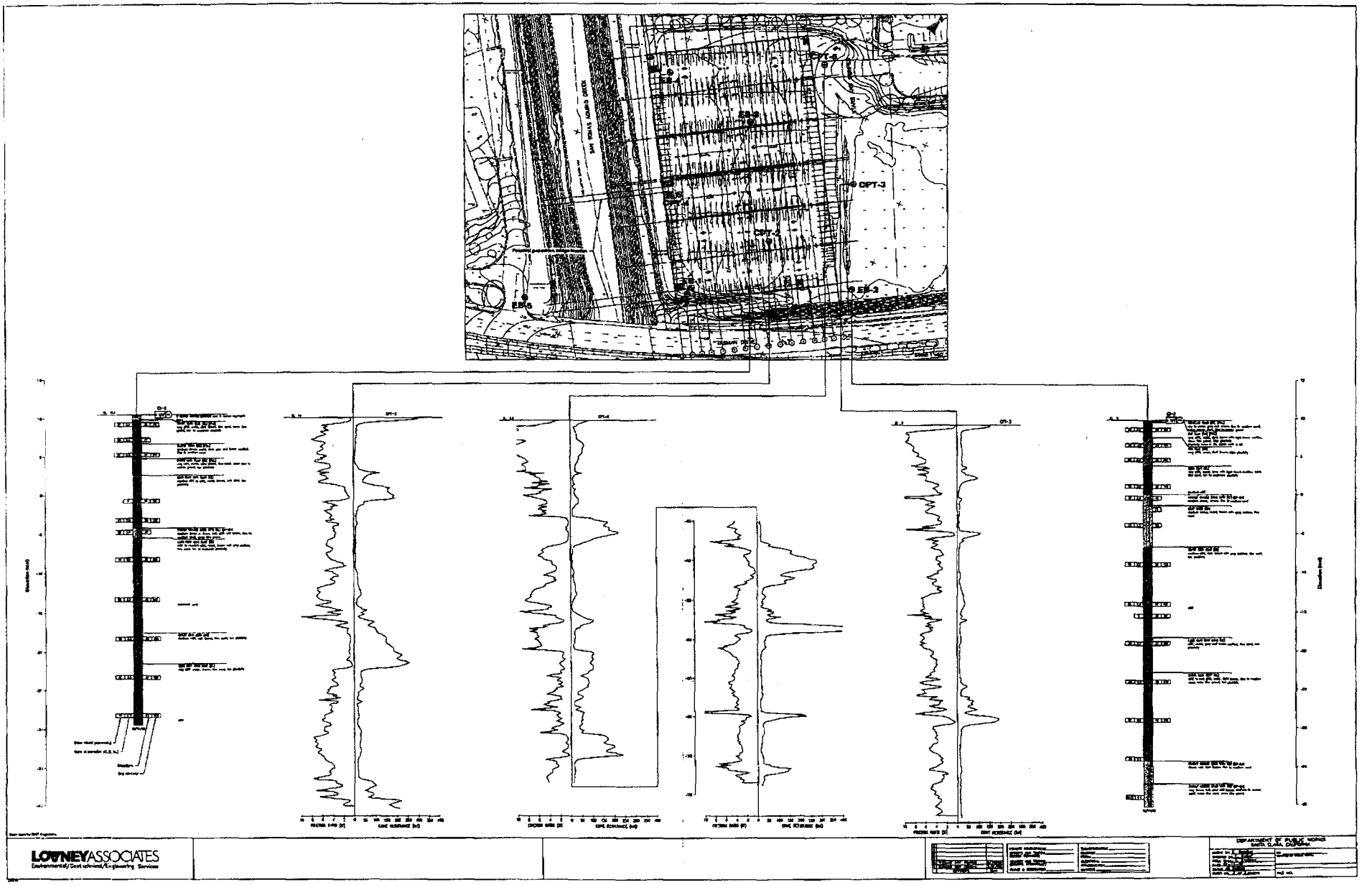


1/03 EB

CONE PENETRATION TEST - CPT-5

LOVNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services


SAN TOMAS AQUINO CREEK
TASMAN DRIVE
CPT-1
CPT-2
CPT-3
EB-2
EB-5
LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services
DEPARTMENT OF PUBLIC WORKS
SANTA CLARA, CALIFORNIA


CPT-3
CPT-2
SB-2
SB-1
SB-3
SB-4
SB-5
SB-6
SAN PEDRO ADAMS CREEK
Elevation (feet)
CPT-2
CPT-4
CPT-3
LOONEY ASSOCIATES
Environmental/Geotechnical/Engineering Services
DEPARTMENT OF PUBLIC WORKS
SANTA CLARA, CALIFORNIA

APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 63 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 49 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are presented on Figure B-1 and on the logs of the borings at the appropriate sample depth.

Washed Sieve Analysis: The percent soil fraction passing the No. 200 sieve (ASTM D1140) were determined on 11 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the logs of the borings at the appropriate sample depths.

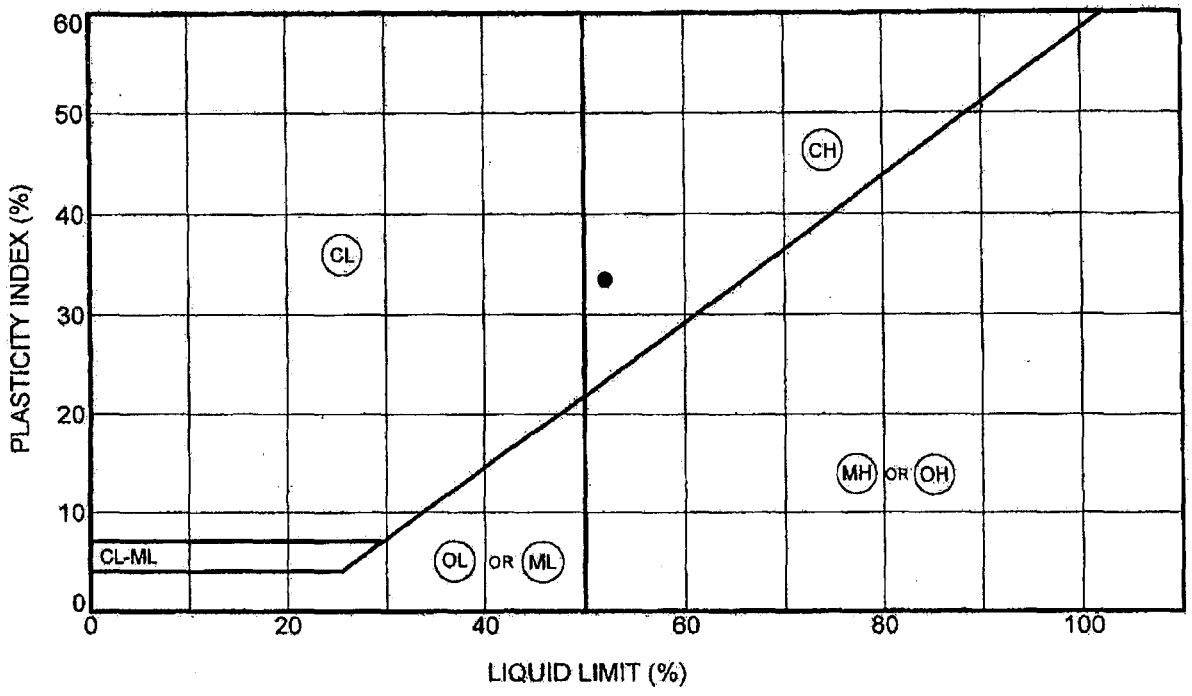
Unconfined Compression: Unconfined compression tests (ASTM D2166) were performed on six relatively undisturbed samples of the clayey subsurface soils to evaluate the undrained shear strengths of these materials. Samples tested had a diameter of 2.5 inches and a height-to-diameter ratio of at least 2. Failure was taken as the peak normal stress. Results of these tests are presented on the boring logs at the appropriate sample depths.

Unconsolidated Undrained Compression: Unconsolidated undrained triaxial compression tests (ASTM D2850) were performed on five undisturbed samples of soil to evaluate the undrained shear strengths of these materials. Samples tested had a diameter of 2.8 inches and a height-to-diameter ratio of at least 2. Failure was taken as the peak normal stress. Results of these tests are presented on the boring logs at the appropriate sample depths.

Consolidation: Consolidation tests (ASTM D2435) were performed on three undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility properties of these soils. Results of the consolidation tests are presented graphically on Figures B-4 through B-6.

R-Values: R-value tests (California Test Method No. 301) were performed on two representative samples of the surface soils at the site to provide data for pavement design. The tests indicated R-values of 8 and 16 for bulks 1 and 2, respectively, at an exudation pressure of 300 pounds per square inch. The results of the tests are presented on Figures B-2 and B-3.

* * * * * * * * * * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	EB-2	2.0	20	52	19	33		FAT CLAY (CH) [FILL]

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

PLASTICITY CHART AND DATA

Project: SANTA CLARA PARKING STRUCTURE

Location: SANTA CLARA, CA

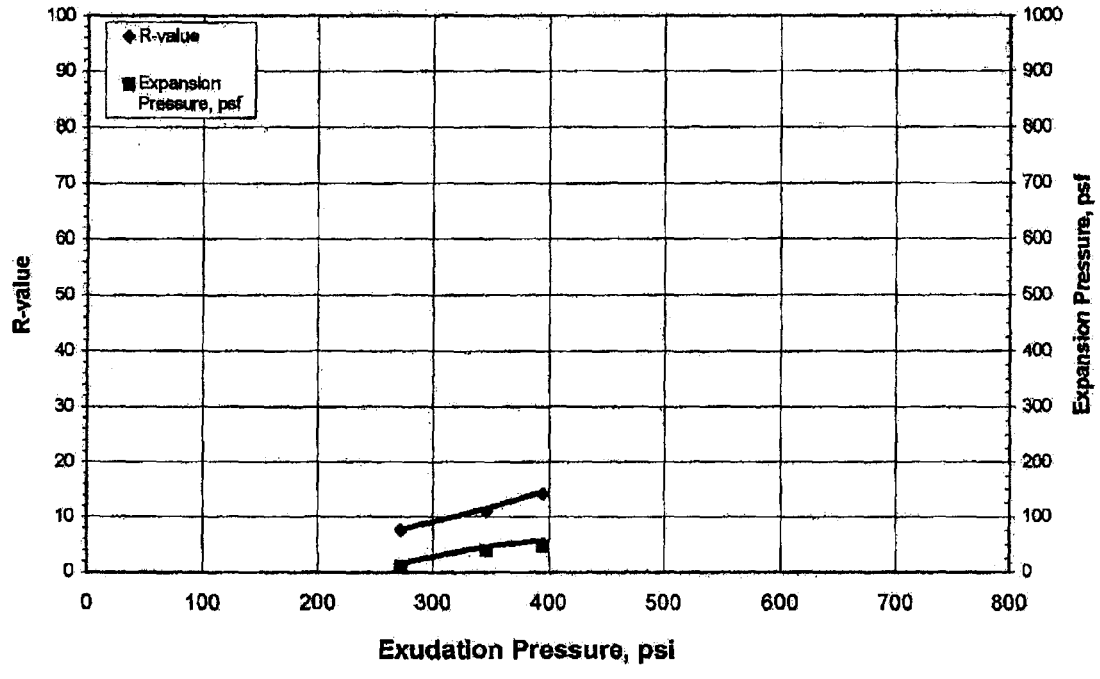
Project No.: 1883-1

FIGURE B-1

LA_CORP.GDT 1/8/03 MV* FLL

			Date:	09/16/03	Initial Moisture,	0.183032
			Tested	MD	R-value	8
			Reduced	DC		
Sample	Bulk-1 @ 1'		Checked	DC	Expansion Pressure	25 psf
Soil Type:	brownish gray CLAY w/sand					

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	346	272	395		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	70	92	49		
Weight of Soil & Mold, grams	3105	3068	3126		
Weight of Mold, grams	2096	2097	2101		
Height After Compaction, in.	2.51	2.48	2.51		
Moisture Content, %	25.2	27.4	23.1		
Dry Density, pcf	97.2	93.1	100.4		
Expansion Pressure, psf	38.7	8.6	47.3		
Stabilometer @ 1000					
Stabilometer @ 2000	140	146	135		
Turns Displacement	2.9	2.95	2.8		
R-value	11	8	14		



R-VALUE TEST

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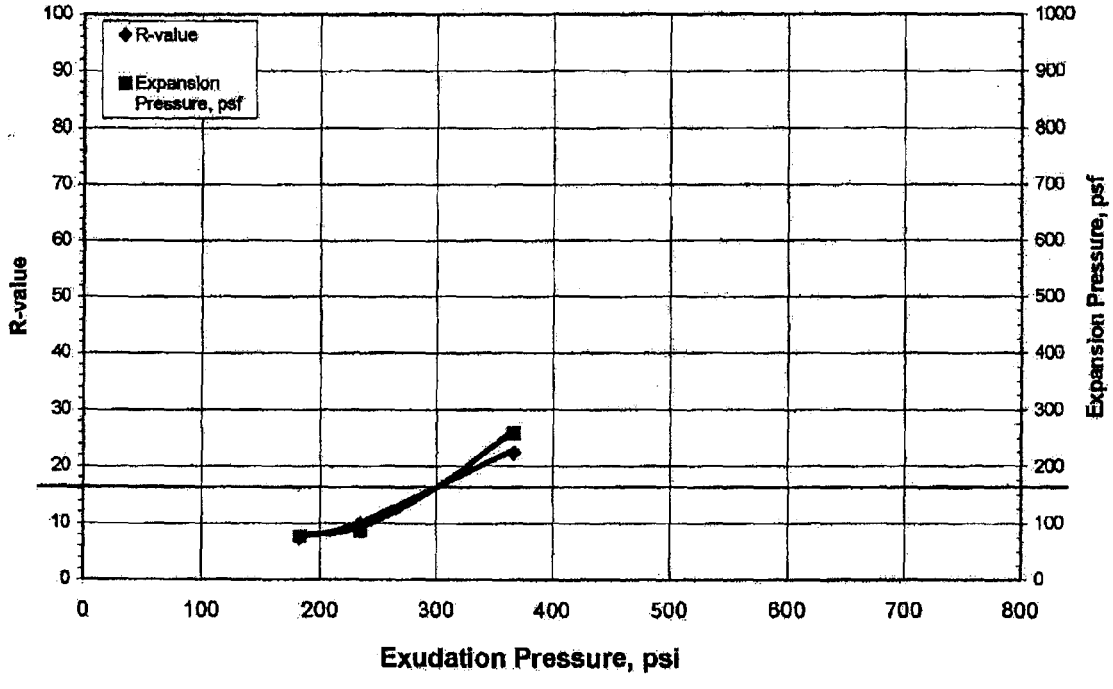
Environmental/Geotechnical/Engineering Services

FIGURE B-2
1883-1

		Date:	09/16/03	Initial Moisture,	0.141588
		Tested	MD	R-value	16
		Reduced	DC		
Sample	Bulk-2 @ 1'	Checked	DC	Expansion Pressure	160 psf
Soil Type:	gray brown sandy CLAY w/gravel				

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	234	367	183		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	52	28	74		
Weight of Soil & Mold, grams	3153	3159	3100		
Weight of Mold, grams	2115	2089	2081		
Height After Compaction, in.	2.53	2.5	2.52		
Moisture Content, %	19.1	16.8	21.2		
Dry Density, pcf	104.3	110.9	101.0		
Expansion Pressure, psf	86.0	258.0	77.4		
Stabilometer @ 1000					
Stabilometer @ 2000	140	120	144		
Turns Displacement	3.25	2.9	3.45		
R-value	10	22	7		

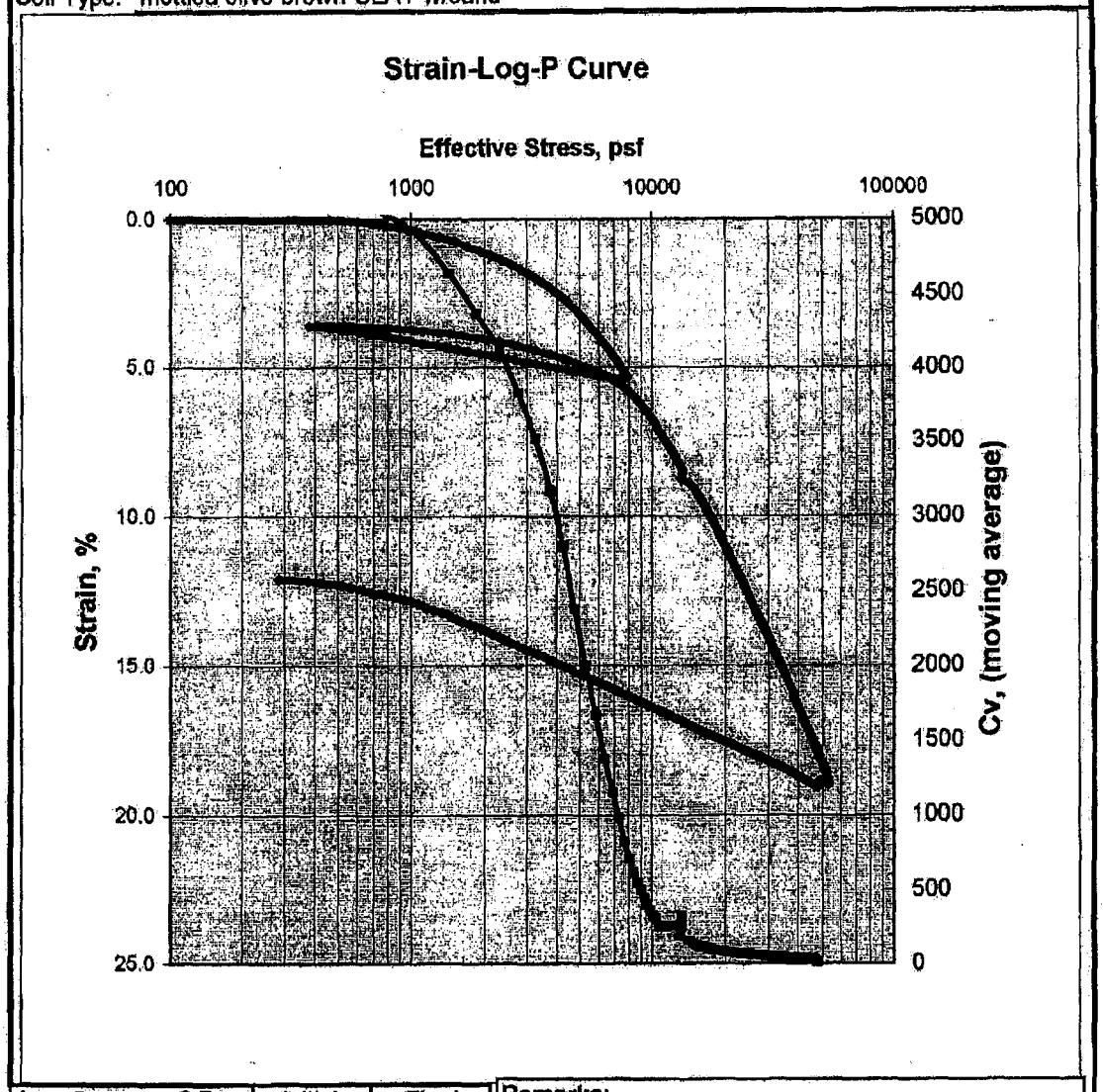


R-VALUE TEST

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-3
1883-1

		Boring:	EB-2	By:	DC
		Sample:	9A	Date:	1/16/2003
Project:	P11974	Depth:	26.0'		
Soil Type:	mottled olive brown CLAY w/sand				



Ass. Gs = 2.7	Initial	Final
Moisture %:	29.8	23.9
Density, pcf:	91.9	102.5
Void Ratio:	0.835	0.645
% Saturation:	96.3	99.9
Initial Back Press., psi	50.5	Max Pore Pressure Ratio 7

Remarks:

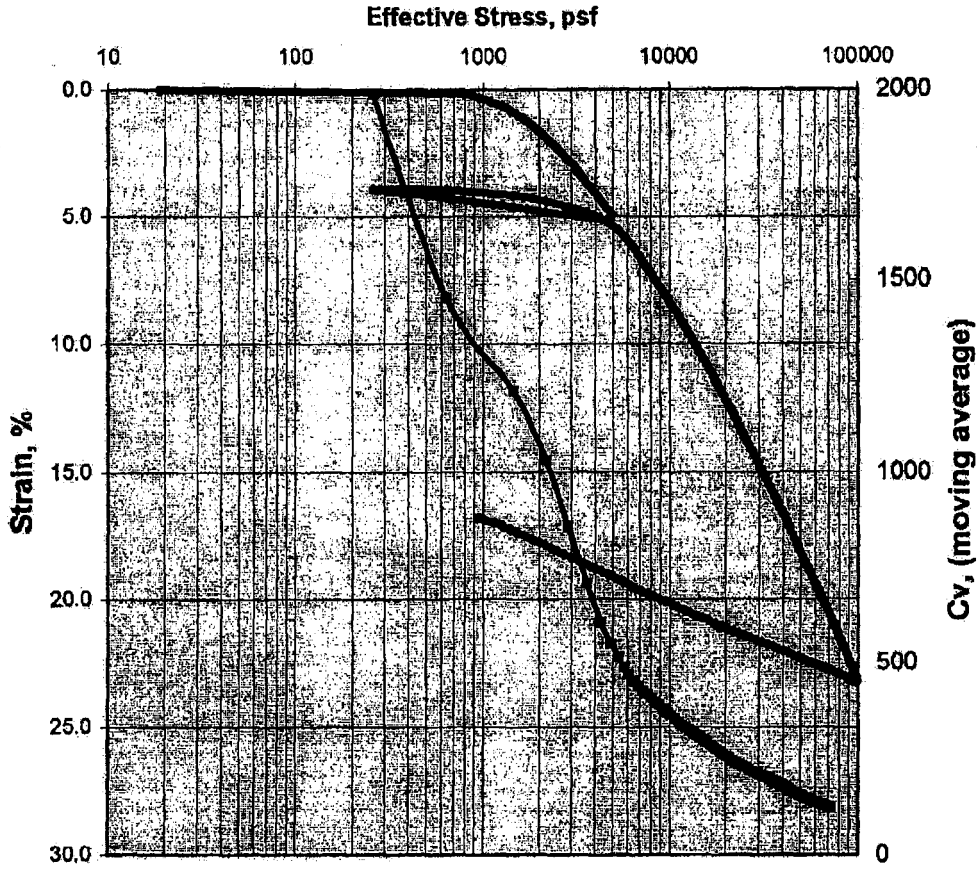
PI =

CONSOLIDATION TEST

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-4
1883-1

Project:	P11974	Boring: EB-3	By: DC
Soil Type:	olive brown CLAY w/fine gravel	Sample: 4A	Date: 9/16/2003
		Depth, ft: 12	



Ass. Gs = 2.75	Initial	Final
Moisture %:	29.8	24.2
Density, pcf:	91.9	103.4
Void Ratio:	0.869	0.661
% Saturation:	94.3	100.0
Initial Back Press., psi	50.0	Max Pore Pressure Ratio 6

Remarks:

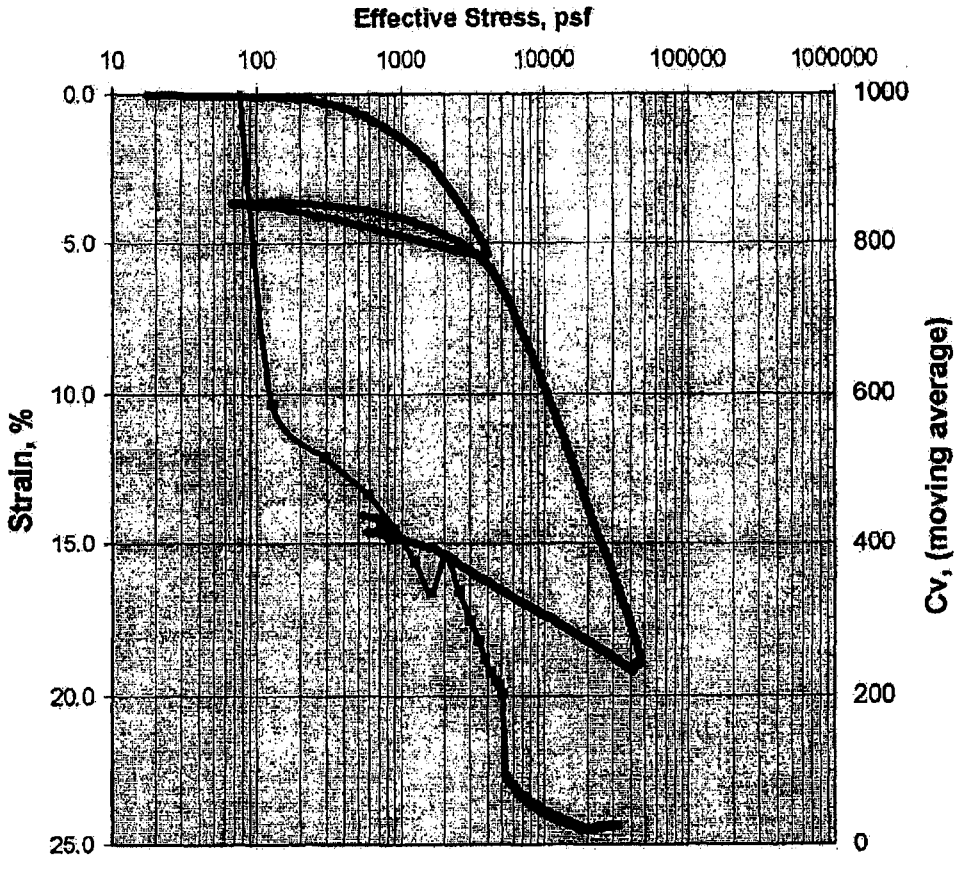
PI =

CONSOLIDATION TEST

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-5
1883-1

		Boring:	EB-4	By:	DC
		Sample:	4A	Date:	9/16/2003
Project:	P11974	Depth:	11.0'		
Soil Type:	dark brown CLAY w/coarse sand				



Ass. Gs = 2.7	Initial	Final	Remarks:
Moisture %:	27.3	20.3	
Density, pcf:	96.7	108.8	
Void Ratio:	0.744	0.549	
% Saturation:	99.0	99.9	
Initial Back Press., psi	49.8	Max Pore Pressure Ratio	37	PI =

CONSOLIDATION TEST

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE B-6
1883-1

APPENDIX C
SOIL CORROSION EVALUATION

* * * * * * * * * * * * *

JDH Corrosion Consultants
Incorporated

January 21, 2003

Lowney Associates
405 Clyde Avenue
Mountain View, CA 94043-2209

Attention: **Mr. Minh Q. Le**

Subject: **Soil Corrosivity Evaluation**
Santa Clara Parking Structure
Santa Clara, CA

Dear Mr. Lee,

Pursuant to your request, ***JDH Corrosion Consultants, Inc.*** has reviewed the soil chemical analysis provided by ***CERCO Analytical, Inc.*** dated 1/20/03 and we have provided herein, a site corrosivity evaluation for materials of construction that will be in contact with these soils based on this information.

SOIL TESTING AND ANALYSIS

Six (6) soil samples from the site were transported to a state certified testing laboratory, **CERCO Analytical, Inc.** (certificate no. 2153) located in Pleasanton, CA for chemical analysis. Each sample was analyzed for pH, chlorides, conductivity, and sulfates using Caltrans test methods as detailed in the table below. The preparation of the soil samples for chemical analysis was in accordance with the applicable specifications.

Soil Analysis Test Methods

Chemical Analysis	ASTM Method
Chlorides	CT422
pH	CT 532/643
Minimum Resistivity	CT532/643
Sulfate	CT417

The results of the chemical analysis are provided in CERCO Analytical, Inc. report dated 1/20/03. The results are summarized as follows:

CERCO Analytical, Inc.
Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	N.D. – 110 mg/kg	Non-corrosive
pH	7.9 – 8.6	Non-corrosive
Minimum Resistivity	500 – 900 ohms-cm	Severely Corrosive - Corrosive
Sulfate	170 - 520 mg/kg	Mildly Corrosive**

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Brief Explanation of Chemical Parameters

Chlorides: Chloride ions are cathode depolarizers which enhance the rate of corrosion. The higher the concentration, the greater the rate of corrosion.

pH: Acidic soils are more conducive to galvanic corrosion of ferrous materials than alkaline soils. The more acidic the soil the greater the rate of anticipated corrosion.

Resistivity: Measures the overall resistance of the soil to electric current flow. Since corrosion is an electrochemical process requiring the flow of electric current through the soil, this parameter relates directly to the degree to which specific soils allow corrosion currents to flow.

Sulfates: Sulfates in the soil can be extremely detrimental to concrete structures due to combined chemical and physical attack. They can react with the binding compounds such as calcium aluminate hydrates to effectively soften the concrete and they can also react physically through crystallization and resultant expansion and contraction processes to crack and weaken concrete structures. Under anaerobic soil conditions sulfates can be reduced to sulfides which can cause corrosion to buried steel structures.

Chemical Testing Analysis

The chemical analysis provided by **CERCO Analytical, Inc.** indicates that the soils are, in general, considered to be "corrosive" to steel and ductile iron based upon the minimum (i.e. saturated) resistivity measurements. Water soluble chloride ions are relatively low as are the sulfate ions, therefore these soils are considered to be "mildly-corrosive" with respect to concrete structures placed into these soils with regard to sulfate attack. The pH of the soils are slightly alkaline which classifies them as "non-corrosive" to buried steel and concrete structures.

EVALUATION

Reinforced Concrete Foundations

There are low levels of water-soluble sulfates in these soils, therefore, the type of concrete used should be resistant to these measured levels of water soluble sulfates. The type of cement used should be Type I or II with a maximum water-to-cement ratio of 0.55 and the minimum depth of cover for the reinforcing steel should be as specified in UBC.

Underground Metallic Pipelines

The soils at the project site are considered to be "corrosive" to ductile/cast iron, steel and dielectric coated steel. Therefore, we recommend the use of coatings, and/or polyethylene encasement, supplemented with cathodic protection for direct buried metallic pressure piping such as domestic and fire water pipelines. All underground pipelines should also be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to minimize potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees expressed or implied are provided.

We appreciate the opportunity to be of service to **Lowney Associates** on this project and trust that you find the analysis and recommendations contained herein satisfactory.

If you have any questions concerning the contents of this report or if we can be of any additional assistance, please do not hesitate to contact us at (925) 927-6630.

Respectfully submitted,

J. Darby Howard, Jr., P.E.
JDH CORROSION CONSULTANTS, INC.
Principal

REGISTERED PROFESSIONAL ENGINEER
J. DARBY HOWARD, JR.
No. CR001055
Exp. 6/30/05
CORROSION
STATE OF CALIFORNIA

cc: File 23007

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client:	Lowney Associates	Date Sampled:	20-Dec-2002
Client's Project No.:	P11974	Date Received:	20-Dec-2002
Client's Project Name:	Santa Clara Parking Structure	Date of Report:	20-Jan-2003
Authorization:	Signed Chain of Custody	Matrix:	Soil

Job/Sample No.	Sample I.D.	Moisture (%)	pH	Min.Resistivity (ohms-cm)**	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0212211-001	EB-1 @ 3.5'/2A	-	7.9	670	-	31	430
0212211-002	EB-2 @ 3.5'/2A	-	8.6	710	-	N.D.	170
0212211-003	EB-2 @ 9.5'/4B	-	8.1	900	-	76	320
0212211-004	EB-3 @ 5.5'/3A	-	8.0	500	-	110	520
0212211-005	EB-4 @ 1.5'/1A	-	8.0	800	-	N.D.	340
0212211-006	EB-5 @ 14'/4A	-	8.5	620	-	21	190
Method:		226	CT 532/643	CT 532/643	-	CT 422/ASTM D4327	CT 417/ASTM D4327
Detection Limit:		-	-	-	50	15	15
Date Analyzed:		-	17-Jan-2003	9-Jan-2003	-	15-Jan-2003	15-Jan-2003

Cheryl McMillen
Laboratory Director

* Results Reported on an "As Received" Basis

** Small Box used in lieu of Caltrans Soil Box due to limited amount of soil

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D

LOWNEY ASSOCIATES REVIEW AND OVERSITE REQUIREMENTS

We understand that the City of Santa Clara may choose to retain another testing firm to provide geotechnical observation and testing during construction while requiring that we remain as Geotechnical Engineer-of-Record. The following presents our requirements for our review and oversight of the other firm's work that must be met; otherwise, we will not be able to sign off as Geotechnical Engineer-of-Record at the end of construction. For the tasks listed below where we will agree to provide review and oversight, the testing firm must provide resumes for qualified personnel for our review. Only those persons approved by our office will be allowed to perform the observation and testing. If it is determined that other staff has performed portions of the work, the work in question will not be approved. Our office must approve all staff changes before they begin work on the project. Please note that we are not responsible for data reported by others.

General Utility Construction: Review and approval of import backfill materials will be the responsibility of the Geotechnical Engineer-of-Record. Density testing should be performed on the backfill materials such that the full backfill depth over the pipes is tested at lateral frequencies no greater than 100 feet. Daily field reports (DFRs) transmitted to our office at the completion of the days work shall be required. The field reports should include the following information: field-determined moist and dry density, field-determined moisture content, any moisture corrections with backup laboratory test data, percent compaction, applicable compaction curve reference, probe depth, lift thickness, test elevation, and gauge calibration date. A plan indication all test locations is required. We will require all backup laboratory data for compaction curves and any Plasticity Index tests. An engineering technician from our staff will independently observe and test the trench backfill at least once every third day of trench construction.

Deep Fill Sub-Excavation and Backfill: If sub-excavation and recompaction of the undocumented fills is chosen, a member of our staff will be required to determine the excavation depth to confirm that the excavation extends into native materials and that the requirements for creating gradual changes in engineered fill depth are met. We will review the testing firms DFRs for the backfill operation. DFRs should include the same information as discussed in the "General Utility Construction" section above. Density testing should be performed on each lift of backfill. The contractor should not be allowed to proceed without documented passing tests. DFRs transmitted to our office at the completion of the days work will be required. The field testing and document review requirements will be as discussed above and will be similar for the shallower sub-excavation and backfill operations. An engineering technician from our staff will independently observe and test the trench backfill at least once every other day of excavation backfill.

Subgrade Preparation and Pavement Aggregate Base Compaction: Review and approval of import materials will be the responsibility of the Geotechnical Engineer-of-Record. The testing firm should plan on performing one or more Plasticity Index (PI) tests to determine where pavements will overlie expansive clays to determine the subgrade preparation requirements from our report. All backup laboratory test data should be forwarded to our office for review. Density testing should be performed for the subgrade and aggregate base compaction. The minimum testing frequency is one test per 1,000 square feet for each material. DFRs transmitted to our office at the completion of the days work will

be required. DFRs should include the same information as discussed in the "General Utility Construction" section above. DFRs should also comment on the stability of the subgrade and aggregate base sections prior to constructing the next section. The field testing and document review requirements will be as discussed above.

Shallow Footing Construction: DFRs transmitted to our office at the completion of the days work will be required. DFRs should include the foundation location (i.e. column line), the bearing material, if the material is expansive, whether dessication cracks are noted, whether the bottom of the excavation is uniformly firm when probed and free of loose deleterious material, whether the excavation sides are neat and vertical, and whether the excavation dimensions generally conform to project specifications. A master map shall be kept on site indicating the footing locations and date observed.

Geopier Soil Improvement Construction: As not many testing firms are familiar with this foundation system, observation will be the responsibility of the Geotechnical Engineer-of-Record.

Stone Column Soil Improvement Construction: As not many testing firms are familiar with this foundation system, observation will be the responsibility of the Geotechnical Engineer-of-Record.

Drilled Pier Construction: DFRs transmitted to our office at the completion of the days work will be required. DFRs should include the pier location (i.e. column line), the depth of fill material, overall drill depth, the tip elevation, the final shaft length, the pier diameter, depth to ground water, method of concrete placement (i.e. tremie when ground water cannot be removed), any casing required, casing depth and length, if any casing will be permanent, whether drilling fluids are used to stabilize the excavation, and whether the excavation generally conform to project specifications. A master map shall be kept on site indicating the pier locations and date observed. A staff engineer from our staff will independently observe at least one pile installation every other day.

Driven Pre-Cast Concrete Pile Construction: Observation of the indicator pile program and review of all PDA data will be the responsibility of the Geotechnical Engineer-of-Record. Once the pile lengths, refusal criteria, and allowable driving stresses are finalized, the testing firm may provide full-time observation. DFRs transmitted to our office at the completion of the days work will be required. DFRs should include the pile location (i.e. column line), the blows per foot, pile dimensions, cast date, reinforcing type, refusal depth if applicable, any spalling, and whether the pile installation generally conform to project specifications. A master map shall be kept on site indicating the pile locations and date observed. A staff engineer from our staff will independently observe at least one pile installation every other day.

Augercast Pile Construction: As not many testing firms are familiar with this foundation system, observation will be the responsibility of the Geotechnical Engineer-of-Record.

URS

June 28, 2006
Project No. 28649762

City of Santa Clara
Engineering Department – Design Division
1500 Warburton Avenue
Santa Clara, California 95050

Attention: Mr. Noel Lozano, CE I

Subject: Technical Memorandum
Preliminary Foundation Recommendations
Convention Center Ballroom Expansion
Santa Clara, California

Dear Mr. Lozano:

As authorized, we have prepared a summary of the anticipated subsurface conditions and existing foundation types at the Convention Center, and developed preliminary foundation recommendations for the proposed ballroom expansion. URS' predecessor firm, Woodward-Clyde Consultants, provided geotechnical design and construction services at the Convention Center, beginning with preliminary studies in 1983 and up to as recently as 1994 for the addition to the lobby area. Recommendations for driven piles and spread footing foundations were included in those reports. We understand the City, Project Manager (Gilbane Building Company), and the Structural Engineer (Biggs Cardosa Associates) will utilize the preliminary information contained herein, to evaluate feasible foundation support options for the proposed ballroom expansion before we conduct any further site-specific exploration for final design.

PROJECT DESCRIPTION

We understand that the City intends to expand the Ballroom space by approximately 24,000 square feet. The expansion will be contiguous to the west wall of the existing Ballroom, as shown on Figure 1. At this time, a one-story structure is planned with long spans to transfer rigging and roof loads to the perimeter walls. We understand that consideration has been given to designing the structure to support a future second story, although the approach is unlikely to be adopted. Column and wall loads have yet to be determined. A uniform floor load on the order of 250 pounds per square foot (psf) is anticipated, based on criteria for the existing Convention Center. A loading dock may also be constructed. The area of the proposed expansion is covered with landscaping and paved automobile parking.

URS Corporation
55 South Market Street, Suite 1500
San Jose, CA 95113
Tel: 408.297.9585
Fax: 408.297.6962

SUBSURFACE CONDITIONS

Three borings and two cone penetration tests (CPT) were previously performed within or in close proximity to the footprint of the proposed Ballroom Expansion during our aforementioned 1983 and 1984 studies. These explorations, labeled 1 and 5 (1983), and B-1, C-1, and C-4 (1984), on Figure 1, extended to depths ranging from 40½ to 141½ feet. Logs of these previous borings and CPTs are presented in Appendix A.

We believe that the proposed ballroom expansion area may be covered by fill placed to raise the site grade during construction of the Convention Center and adjacent hotel. The thickness, nature and density of this fill are unknown. Below existing pavements and existing fills, the site is generally blanketed by 3 to 4 feet of medium to hard, lean to fat clay, underlain by thick deposits of medium to very stiff silty and sandy clay with interbedded strata of medium dense to very dense silty sand to the maximum depth explored of 141½ feet. The sand layers typically range from 5 to 10 feet thick and were encountered as shallow as approximately 20 feet below the ground surface at the time of exploration. These granular layers were encountered in each of the five previous explorations; however, the depths and thicknesses of the layers varied considerably between locations.

Laboratory consolidation tests and insitu pressuremeter tests conducted for the previous studies indicate that the subsurface clays are generally overconsolidated, with an overconsolidation ratio (OCR) of approximately 2 to 4. An OCR in this range suggests the clays are of moderate compressibility under relatively high building loads.

Insitu and laboratory testing were also performed to evaluate the corrosion potential of the clayey soils at the site. Results of these tests indicate that the native soils are moderately to severely corrosive to buried concrete and steel.

Water level measurements taken in borings drilled across the Convention Center site indicated groundwater at a depth of approximately 6 feet (Elevation 3½ feet) in April 1983, and 9 feet (Elevation 1 foot) in September 1983. This suggests some seasonal changes in the groundwater level.

DISCUSSION

The primary geotechnical consideration at the site is the moderate compressibility of the native clays under relatively high building loads. At this time, column and wall loads have not been determined. Spread footing foundation settlement estimates presented in our previous studies for a range of typical column loads and feasible bearing pressures are discussed in the following section; these parameters could be used for preliminary sizing of shallow foundations. If the estimated shallow foundation settlements are deemed intolerable, from a structural engineering standpoint, a deep foundation system will need to be considered as discussed below.

Because much of the proposed expansion site is currently landscaped, the quality and relative compaction of any existing fills will need to be evaluated with regard to support of building and floor loads. It is likely that most, if not all, of the existing fill may require removal and

replacement. Measures also will need to be incorporated to minimize the potential for shrinking or swelling of the underlying moderately to highly expansive native clays.

Finally, the sand stratum generally encountered in previous explorations at a depth of approximately 25 or 35 feet is potentially liquefiable. However, because a thick blanket of generally stiff to hard clay caps this layer, we believe the most likely consequence would be post-liquefaction induced ground surface settlement. We estimate such settlements will be less than 1-inch.

FOUNDATION ALTERNATIVES

Both shallow and deep foundation systems have been used successfully at the Convention Center site. Below, we present a discussion of preliminary foundation alternatives that could be considered for the proposed Ballroom expansion.

Spread Footings

In our 1984 report for the Convention Center, we provided recommendations for spread footings and estimated that settlements due to building loads would be on the order of ½ inch per 100 kip column load increment. Design bearing pressures of 3,000 psf for dead loads, 4,000 psf for dead plus live loads, and 5,000 psf for total loads were recommended for footings less than 10 feet wide. For footings wider than 10 feet, the design pressures were reduced by 1,000 psf for each loading case. Similarly, shallow foundations at the 1994 lobby addition were designed for bearing pressures of 2,000 psf for dead loads, 3,000 psf for dead plus live loads, and 4,000 psf for total loads, and estimated footing settlements on the order of less than 1 inch. These recommendations were for anticipated column loads on the order of 320 kips for dead plus live load and 465 kips for total loads.

For preliminary footing sizing and foundation comparison purposes, we recommend bearing pressures of 2,000 psf for dead loads, 3,000 psf for dead plus live loads, and 4,000 psf for total loads be used for footings bearing on engineered fill, with an estimated ½ inch of total settlement for each 100 kip column load increment. Footings should be a minimum of 2 feet wide and extend at least 2 feet below lowest adjacent finish grade.

Rammed Aggregate Piers

Rammed aggregate piers (RAPs) represent an intermediate foundation type; they are typically much shorter than driven piles and can be used to increase the allowable bearing capacity for shallow foundations, since they transfer building loads to stronger, less compressible strata.

RAPs typically are constructed by pre-augering a 2 to 3-foot diameter shaft to the required depth and then backfilling with densely compacted thin lifts of high quality crushed rock. The vertical ramming action increases lateral stresses in the surrounding soil, which results in reduced foundation settlement and enhanced design bearing pressures; typically design bearing pressures of two to three times the unreinforced design allowable bearing pressure of the insitu soils are feasible. The noise and vibration levels associated with the ramming action are relatively low. RAPs can be installed individually or in groups to reinforce the soil

in the zone below spread footings, where stresses due to building loads are highest. They are typically installed to depths of one to two times the foundation width. It should be noted that RAPs are delivered through a proprietary, design/build process.

Once foundation loading information becomes available, we will advise whether RAPs might be a viable alternative for the Ballroom Expansion project.

Driven Piles

Because of the relatively high column loads, both the Convention Center and Parking Garage were supported on deep foundations consisting of 12-inch square driven precast, prestressed concrete (PCPS) piles. Based on the results of static load tests performed on 40 and 70-foot long, 12-inch square piles, and dynamic monitoring during indicator pile installation, the following design axial compression capacities were assigned for production piles at the Convention Center:

Design Axial Compression Capacity (tons)	Length (feet)
40	35
80	55

A graphical plot of pile capacity versus depth was presented as Figure 6 in our 1985 report entitled, "Indicator and Load Test Program, Santa Clara Convention Center;" a copy of Figure 6 attached for convenience of reference. We believe Figure 6 provides a reasonable basis for developing preliminary estimates of pile lengths that might be required for the Ballroom Expansion. Since 14-inch square PCPS piles are more commonly used at this time, we recommend the axial compression load capacity values plotted on Figure 6 be adjusted upward by 15 percent to account for the increased surface area of the pile shaft. The settlement expected for piles supporting axial compression loads of this order of magnitude is expected to be less than ½ inch.

It should be noted that pile capacity at the Convention Center site is developed primarily through skin friction. Therefore, an efficiency factor will need to be included account for pile group action. We recommend that a group efficiency factor of 0.70 be assumed for preliminary estimating purposes. This group efficiency factor is based on the assumption that piles would be spaced at least three diameters (pile widths) on center. Further refinements of these preliminary pile parameters should be made during final design.

Drilled Piers

Cast-in-place concrete drilled piers having a minimum diameter on the order of 30 inches also could be used to support the proposed structure. Based on shear strength data from previous explorations at the site, we suggest that a preliminary design side adhesion of 500 psf be used for estimating the embedment depth of drilled piers. The settlement of drilled piers supported in the primarily stiff to hard alluvial soils should not exceed about ½ inch.

The soils at the site consist primarily of clay to a depth of 141½ feet, with 5 to 10-foot-thick sand interbeds as shallow as approximately 20 feet. Groundwater has been encountered at the site at depths as shallow as 6 feet. Therefore, drilling probably would require the use of temporary casing or slurry to prevent caving during construction. If casing is used and the holes are dewatered, the casing would have to be withdrawn from the hole slowly as the concrete is being placed; a minimum head of concrete of 5 feet typically must be maintained above the bottom of the casing at all times. Alternatively, excavating and depositing concrete under slurry could be considered. Concrete deposited under slurry would require careful placement in a compact, monolithic mass and by a method that would prevent washing of the concrete.

Auger Cast Piles

Auger cast piles are installed by rotating a continuous flight hollow-stem auger into the soil to a specified depth. High strength cement grout then is pumped under pressure through the hollow shaft as the auger is slowly withdrawn. The grout pressure keeps the hole open so no casing is required and the grout displaces the existing soil for the entire length of the pile. The resulting grout column hardens and forms an auger cast pile. Reinforcing is installed while the cement grout is still fluid or, in the case of full length single reinforcing bars, through the hollow shaft of the auger prior to the withdrawal and grouting process. Typical ultimate pile capacities for auger cast piles can range from 10 to 50 tons for 12-inch diameter piles, and 75 to 100 tons for 16-inch diameter piles. Axial compression and tension pile load tests generally are conducted to verify the selected design load capacities. Although noise and vibration are of less concern with auger cast piles than driven piles, workmanship is an important consideration for achieving sound, non-driven piles in interbedded alluvial soils below groundwater.

SLAB-ON-GRADE FLOOR SUPPORT

For preliminary planning purposes, we recommend that the slab-on-grade floor be supported on a minimum section of 2 feet of non-expansive engineered fill. The fill is intended to provide a cap over the native lean to fat clay, and to minimize the risk of seasonal shrinking and swelling. It is conceivable some of the existing fill placed at the time of the original grading for the Convention Center might be reusable for support of the Ballroom Expansion floor. The nature of the fill and its suitability for reuse should be evaluated prior to final design. In any case, fill material used beneath the new slab-on-grade floor should be of select quality, having a plasticity index of 12 or less.

FUTURE EXPLORATION

As discussed in the scope of services included in our agreement, we plan to advance two additional cone penetration tests (CPTs) at the locations shown on Figure 1 to better define the subsurface conditions within the footprint of the proposed ballroom expansion. In addition, we will collect samples of the surficial soils in the landscape area using hand auger equipment for plasticity and other index property tests.

LIMITATIONS

The information provided in this technical memorandum is preliminary in nature, and the recommendations presented are for general planning use only and to evaluate general technical feasibility. The attached graphical plot of pile capacity versus depth was developed for the existing Convention Center, based on a combination of static pile load tests and engineering judgment. The plot is intended for preliminary sizing of precast, prestressed concrete piles for the Ballroom Expansion. Selection of design axial and lateral load capacities for piles, as well as design criteria for other geotechnical aspects of the site development, should be determined during final design studies, after additional site-specific subsurface data have been collected.

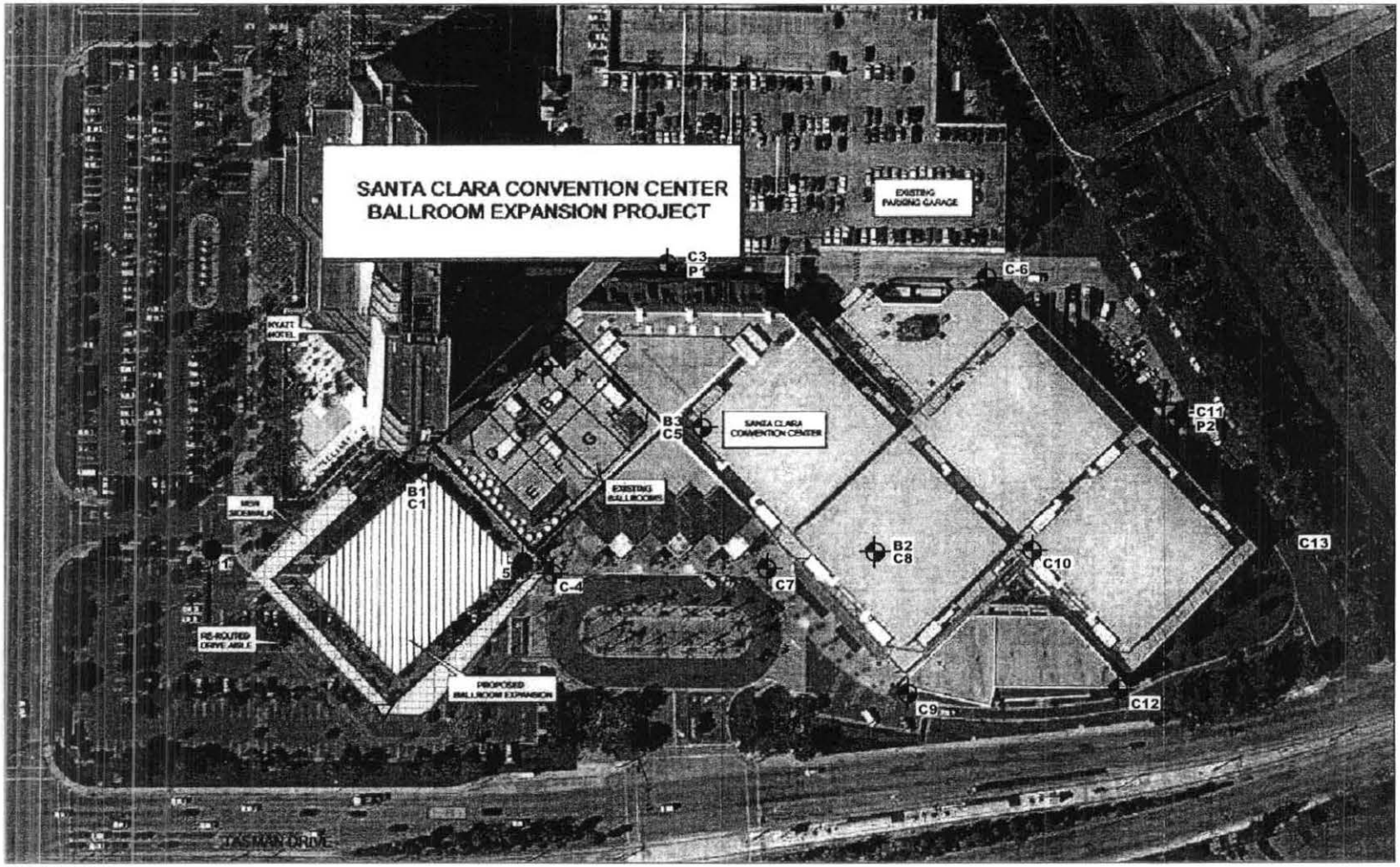
This preliminary assessment has been completed in accordance with the standard of care commonly used as state of the practice in the profession. No other warranties are included, either express or implied, as to the professional advice offered.

Sincerely,

Anne-Marie Moore, G.E.
Geotechnical Engineer

Paul J. Boddie, G.E.
Geotechnical Department Manager

Attachments: Figure 1 – Site and Exploration Plan
Appendix A – Logs of Previous Explorations at Site



N

LEGEND:

WOODWARD-CLYDE CONSULTANTS (URS) - 1983

5 - Exploratory Boring (AUGER)

WOODWARD-CLYDE CONSULTANTS (URS) - 1984

B1 - Exploratory Boring (ROTARY WASH)

C1 - Cone Penetrometer Test

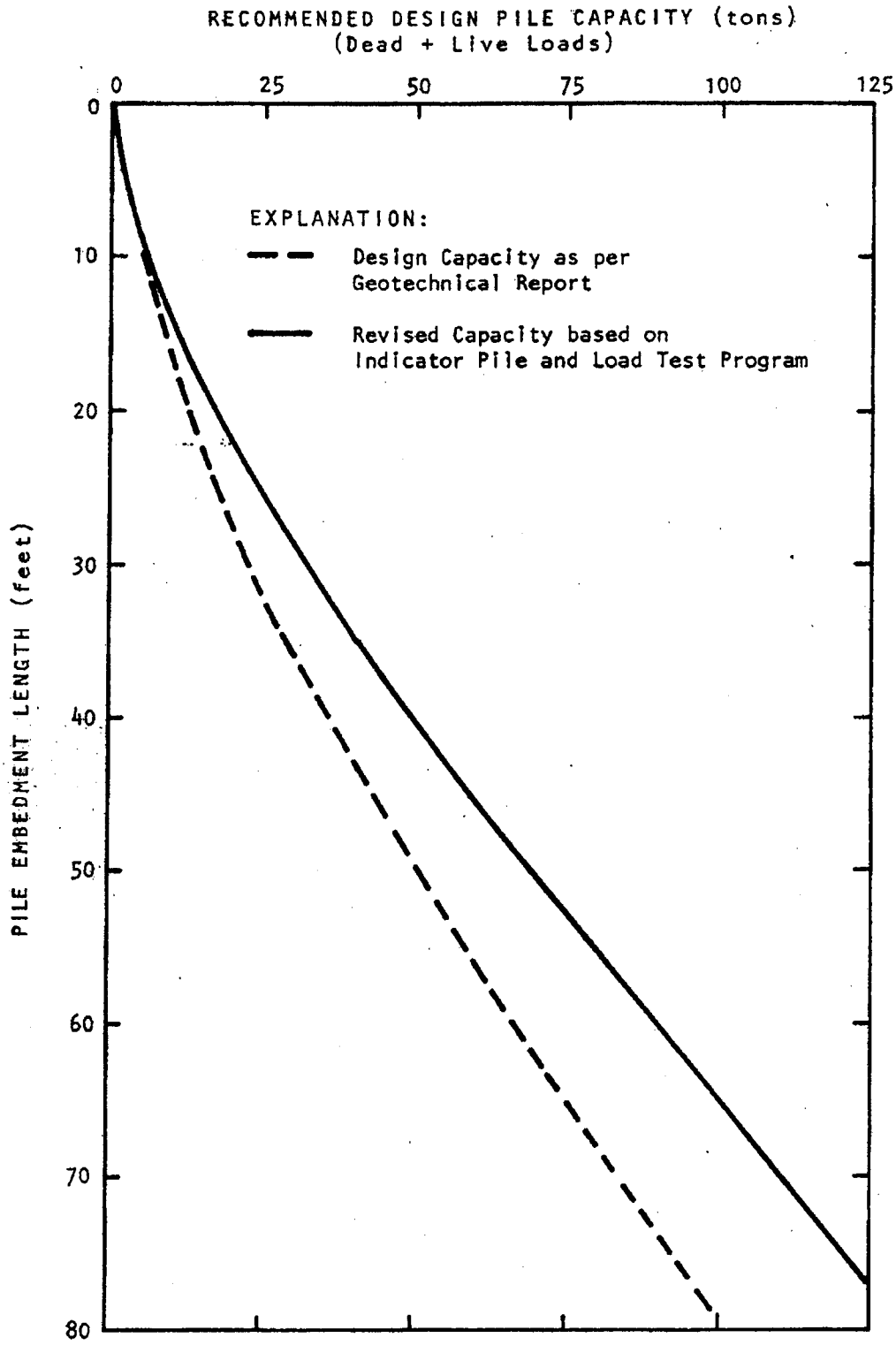
P1 - Menard Type Pressuremeter Test

PROPOSED CONE PENETROMETER TEST

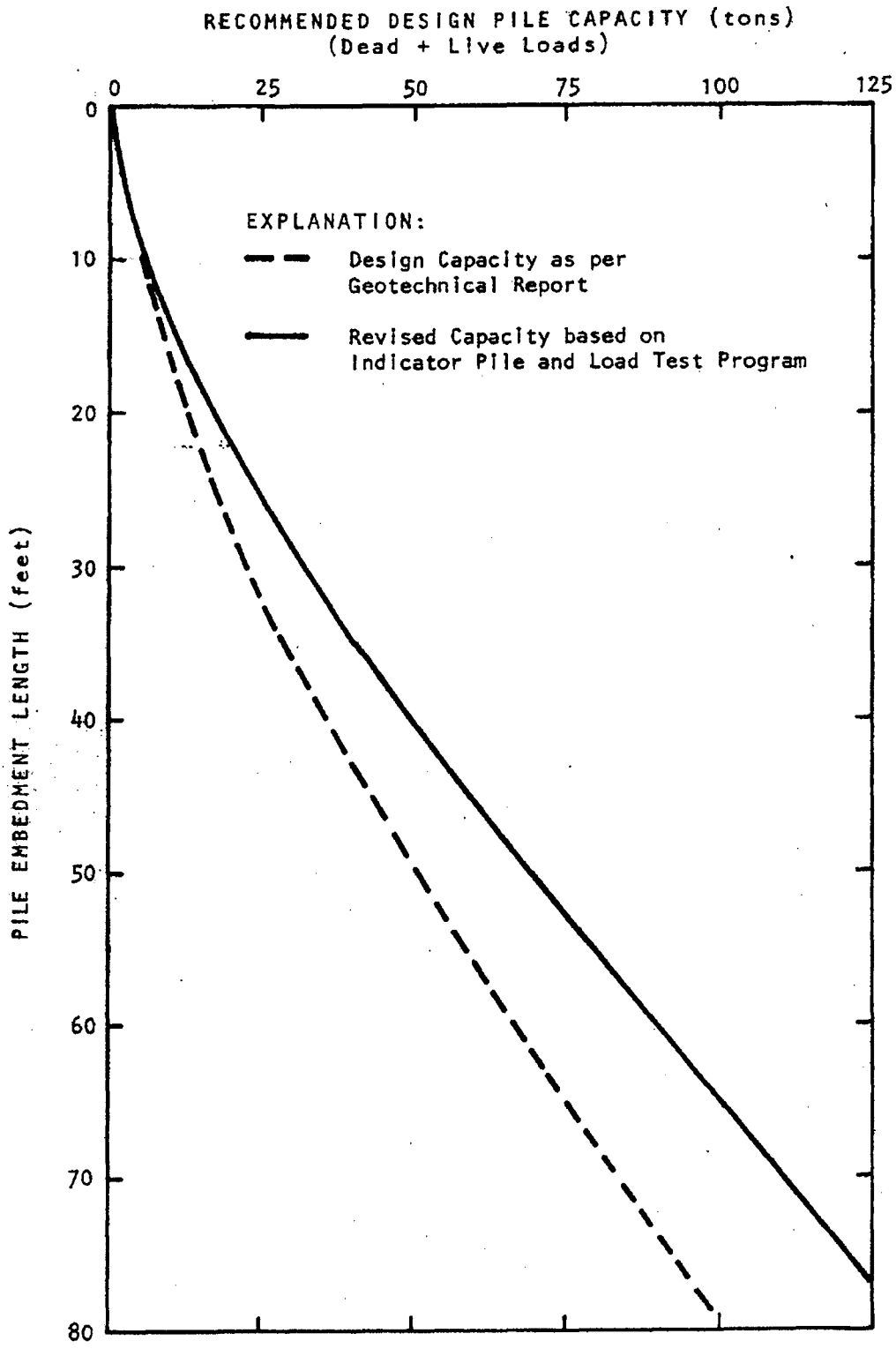
100 0 100

APPROXIMATE SCALE : 1 INCH=100 FEET

URS	Santa Clara Convention Center Ballroom Expansion Project	SITE AND EXPLORATION PLAN	JUNE 2006 Figure 1



Project No. 16029-X **Woodward-Clyde Consultants**	SINGLE PILE COMPRESSION LOAD CAPACITY 12 Inch Square Pile	Figure 6



Project No. 16029-X **Woodward-Clyde Consultants**	SINGLE PILE COMPRESSION LOAD CAPACITY 12 Inch Square Pile	Figure 6

Project: SANTA CLARA CONFERENCE CENTER Santa Clara, California	**Log of Boring No. 1**

Date Drilled: May 6, 1983
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9.0±

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	16	SILTY CLAY (CH) Stiff, wet, dark gray Moist, dark gray to brown-gray mottled	22	104	2920
5	2	22	SILTY CLAY (CL-CH) Medium, wet, light brown to brown mottled Very stiff, moist, CL ATD	26	98	5110
10	3	7	SILTY CLAY (CL) Medium, brown mottled with tan and gray, with some fine sand Becoming silty (CL-ML)	24	103	1560
15	4	22	SILTY CLAY (CL) Very stiff, brown and gray mottled	22	105	5750
20	5	13	CLAYEY SAND (SC) Medium dense, brown and gray Slightly coarser (SP-SM) -#200 = 32%	21	106	680
25	6	12	SILTY CLAY (CL) Stiff, brown and gray, with layers of sandy clay, clayey sand, and sand	23	103	2920
30						

Proj. No. 15582V	Woodward-Clyde Consultants	Figure 3a

Project: SANTA CLARA CONFERENCE CENTER
Santa Clara, California

Log of Boring No. 1 (Continued)

Depth, ft.	Samples	Blows/ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	7	18	Sand lens — SILTY CLAY (CL) with interbedded sandy layers.....Cont'd	-	-	-
35						
	8	14	Clayey sand lens — -#200 = 49%	21	106	1020
40			Medium, with some fine sand zones			
	9	12		25	99	1510
45						
	10	15		24	103	1900
50			Stiff			
	11	15		24	101	2440
55			Very stiff			
	12	25		25	102	7160
60			SILTY SAND (SM) Medium dense, gray and brown mottled			
	13	39	Grading to SAND (SP), dense	-	-	-
65			SANDY CLAY (CL): Very stiff, brown & gray mottled			

Proj. No. 15582V | Woodward-Clyde Consultants | Figure 3b

Project: SANTA CLARA CONFERENCE CENTER
Santa Clara, California

Log of Boring No. 1

(Continued)

Depth, Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	14	28	Grading to SILTY CLAY (CL), with some fine sand	21	107	5160
70						
			SILTY SAND / SANDY SILT (SM-ML) Medium dense, brown and gray mottled			
	15	30		25	99	5850
			SILTY CLAY (CL) Very stiff, gray and brown mottled, with some fine sand			
75						
			(CL-ML), Stiff			
	16	33		22	106	3460
80			BOTTOM OF BORING @ 80'			
85						
90						
95						

Proj. No. 15582V | Woodward-Clyde Consultants | Figure 3c

Project: SANTA CLARA CONFERENCE CENTER
Santa Clara, California

Log of Boring No. 5

Date Drilled: April 6, 1983
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9.0±

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	10	SILTY CLAY (CH) Medium, moist, black Stiff	23	98	2480
5	2	16	SILTY CLAY (CL) Very stiff, moist gray-brown mottled, with trace of lime nodules Light brown to brown mottled	22	104	5890
10	3	31		26	99	7110
			ATD			
15	4	8	SILTY CLAY (CL) Stiff, brown and gray mottled, with fine sand Sand lens Becoming Silty Clay (CL-CH), less fine sand	22	104	2140
20	5	12		25	98	2530
25	6	9	SILTY CLAY (CL-ML) Medium, gray-brown, with trace of fine sand With sandy clay layers	26	97	1460
30	7	15	CLAYEY SAND (SC) Medium dense, gray and brown, with sandy clay zones Grading to Sandy Clay (CL)	18	114	1510

Proj. No. 15582V | Woodward-Clyde Consultants | Figure 7a

Project. SANTA CLARA CONFERENCE CENTER Santa Clara, California	Log of Boring No. 5 (Continued)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
35	8	14	SAND (SW) Medium dense, gray	-	-	-
			SILTY CLAY (CL) Stiff, gray-brown			
40	9	24		-	-	-
			BOTTOM OF BORING @ 40½'			
45						
50						
55						
60						
65						

Proj. No. 15582V	Woodward-Clyde Consultants	Figure 7b

Project: SANTA CLARA CONFERENCE CENTER, Santa Clara, California

Log of Boring No. B1

Date Drilled: September 21,22, & 23, 1983
Type of Boring: Rotary
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9.0

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
	1	16	SILTY CLAY (CL-CH) Very stiff, damp, brown-gray	-	-	-
5	2		SILTY CLAY (CL) Stiff, moist, light brown-tan with dark brown specks, with layers of clayey silt; Becoming soft	-	-	-
10	3	4	9-24-83	34	86	1560
15	4	21	Becoming very stiff	-	-	-
20	5	41	SILTY CLAY (CL) Very stiff, brown with gray specks, traces of large sand nodules	-	-	-
25	6	40	SILTY SAND (SP-SM) Dense, brown and gray, medium to fine	-	-	-
30	7	29	SILTY CLAY (CL) Stiff, brown and gray, with layers of clayey silt and sandy clay, traces of sand nodules; Seams of dense, very fine silty sand	20	106	1500
35	8	39	Very stiff	21	107	4550
40	9	36		26	100	2820
45	10	32	Traces of coarse sand and fine gravel			
50	11	32		20	107	2430
55	12	62		28	94	3890
60	13	58	Changing to sandy clay	19	107	-
65	14	53	Seams of very fine silty sand	24	99	[illegible]
70	15	58	SILTY SAND (SP-SM) Very dense, gray, fine to very fine; Few fine gravel	-	-	-
75	16	51	SILTY CLAY (CL) Very stiff, mottled light brown and gray, with layers of clayey silt and sandy clay, traces of sand nodules	22	104	2130
80	17	58		17	112	2850
85	18	100 8"	SILTY SAND (SP-SM) Very dense, gray and brown, fine	-	-	-
90	19	41	SILTY CLAY (CL) Stiff, mottled brown and gray, with layers of clayey silt and sandy clay, traces of sand nodules	-	-	-
95	20	40	Becoming very stiff	-	-	-
100	21	58		26	98	1300
105	22	84		-	-	-
110	23	96	SILTY SAND (SM) Very dense, gray, fine to very fine, with layers of very stiff, gray-brown silty clay	-	-	-
115	24	78		-	-	-
120	25	53	SILTY CLAY (CL-CH) Very stiff, greenish gray, traces of sand nodules	-	-	-
130	26	46 9"	Occasional layers of sandy clay and silty sand	-	-	-
140	27	44		-	-	-

BOTTOM OF BORING @ 141.5 Feet

Project: SANTA CLARA CONFERENCE CENTER, Santa Clara, California

Log of Boring No. B2

Date Drilled: September 23 & 26, 1983
Type of Boring: Rotary
Hammer Weight: 140 and 280 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

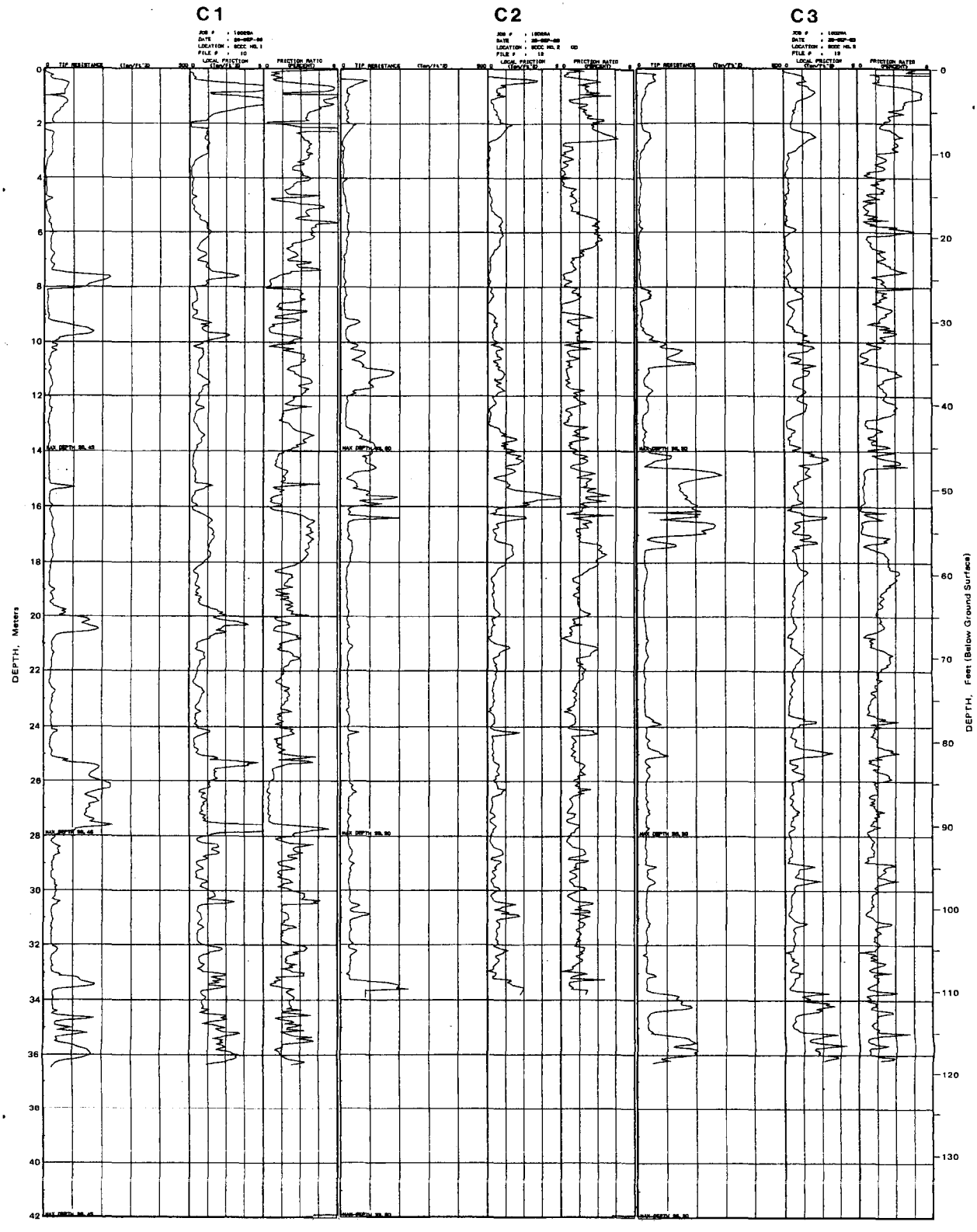
Surface Elevation: 9.7

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
	1	11	SILTY CLAY (CL-CH) Very stiff, damp, brown-gray	-	-	-
5	2	30	SILTY CLAY (CL) Stiff, moist, light brown-tan, with dark brown specks, with layers of clayey silt, traces of sand nodules; Becoming soft	-	-	-
10	3	17	9-27-83; Becoming medium stiff	-	-	-
15	4	12		-	-	-
20	5	36		24	103	3150
25	6	21	SILTY CLAY (CL) Stiff, brown with gray specks, with layers of clayey silt, traces of sand nodules	22	103	1810
30	7	26	Trace of large sand nodules	-	-	-
35	8	34		-	-	-
40	9	42	SILTY SAND (SP-SM) Very dense, brown-gray, fine to very fine, with occasional seams of stiff silty clay	-	-	-
45	10	27	SILTY CLAY (CL) Very stiff, brown and gray, with layers of clayey silt, traces of sand nodules	-	-	-
50	11	28		-	-	-
55	12	46		27	97	5020
60	13	52		27	97	3740
65	14	100*	SAND (SP) Very dense, brown and gray, fine to medium, with occasional seams of stiff silty clay	-	-	-
70	15	43		-	-	-
75	16	25*	SILTY CLAY (CL) Stiff, brown and gray, with layers of clayey silt, traces of sand nodules	22	106	1900
80	17	26*		-	-	-
85	18	31*		20	109	2950
90	19	32*		19	112	2600
95	20	41*		-	-	-
100	21	56* 8"	Seam of very fine silty sand	-	-	-
105	22	58*		-	-	-
110	23	56* 8"	SILTY SAND (SM) Very dense, brown and gray, fine to very fine, with occasional layers of stiff silty clay	-	-	-
	23A	56* 8"		-	-	-
115	24	65		-	-	-
120	25	44	SILTY CLAY (CL-CH) Very stiff, greenish gray, traces of sand nodules	-	-	-
130	26	82*	Occasional layers of sandy clay and silty sand	-	-	-
140	27	77*		-	-	-

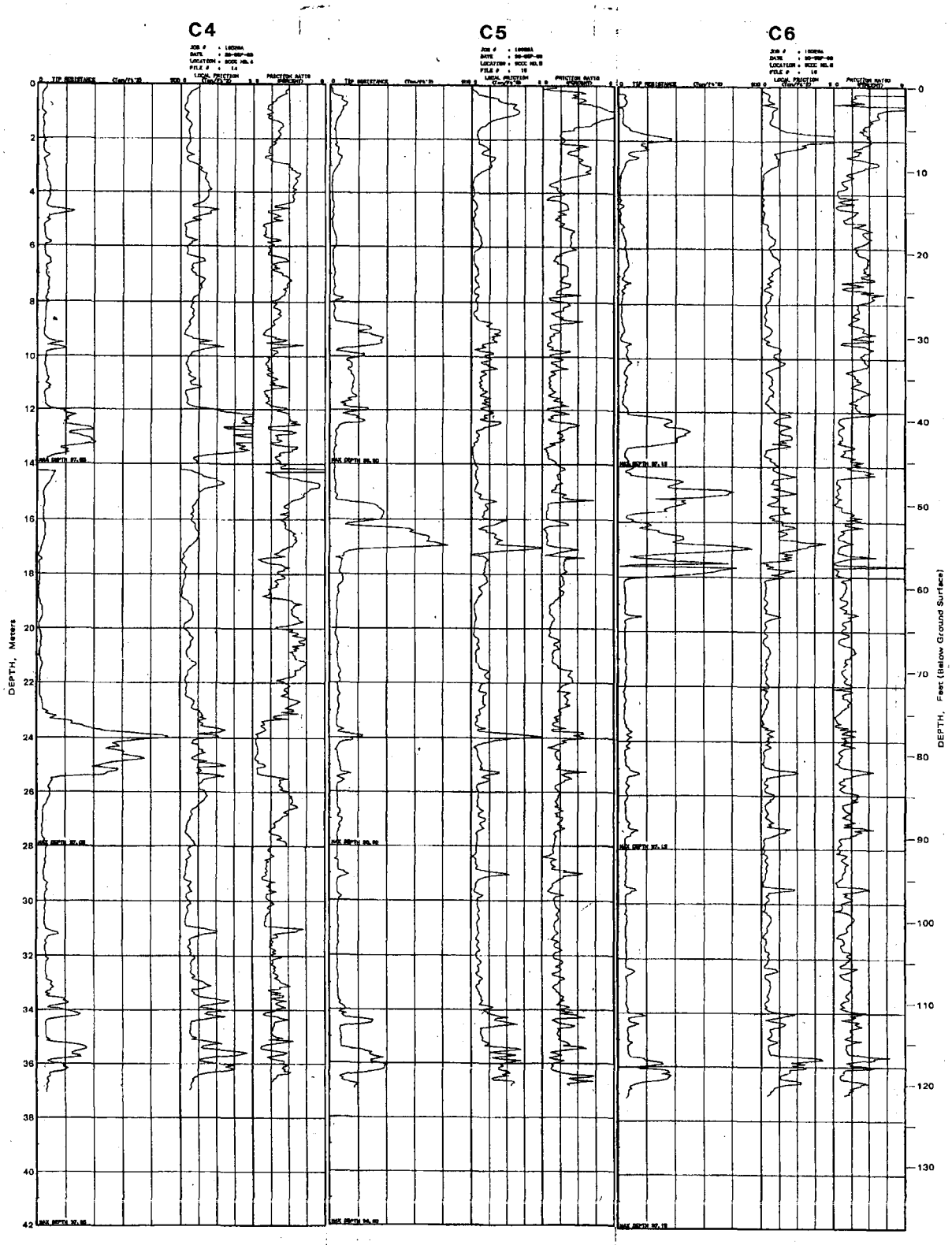
BOTTOM OF BORING @ 141.5 Feet

Project No. 16029A	Santa Clara Conference Center	BORING LOGS FOR B1 AND B2	Figure A-2

Woodward-Clyde Consultants



Project No. 16029A	Santa Clara Conference Center	CONE PENETEROMETER TESTS C1, C2, AND C3	Figure A-5
Woodward-Clyde Consultants			



Project No. 16029A	Santa Clara Conference Center	CONE PENETEROMETER TESTS C4, C5, AND C6	Figure A-6
Woodward-Clyde Consultants			

URS

May 18, 2007
Project No. 28649762

City of Santa Clara
Engineering Department – Design Division
1500 Warburton Avenue
Santa Clara, California 95050

Attention: Mr. Tom Supan, Principal Engineer

Subject: Geotechnical Engineering Report
Convention Center Ballroom Expansion
Santa Clara, California

Dear Mr. Supan:

This report presents the results of our geotechnical investigation for the Convention Center Ballroom Expansion in Santa Clara, California. It presents our opinions and recommendations regarding foundation types, depths and design parameters, site preparation and grading, and other geotechnical aspects of site development. The opinions and recommendations presented herein have been based upon existing historical subsurface information, the results of our supplemental field investigation, engineering judgment, and local experience.

BACKGROUND

URS' predecessor firm, Woodward-Clyde Consultants, provided geotechnical design and construction services at the Convention Center, beginning with preliminary studies in 1983 and up to 1994 for the addition to the lobby area. Recommendations for driven piles and spread footing foundations were included in those reports, respectively. More recently, we provided preliminary information in our June 28, 2006 Technical Memorandum, to assist the Design Team in evaluating feasible foundation support options for the proposed ballroom expansion.

PROJECT DESCRIPTION

We understand that the City of Santa Clara (City) intends to expand the Ballroom space by approximately 24,000 square feet. Ancillary space such as a Prefunction Room, Finishing Pantry, Services Corridor and Restrooms is proposed to be located around the perimeter of the Ballroom. The expansion will be contiguous to the west wall of the existing Ballroom, as shown on Figure 1. At this time, a one-story structure is planned with long spans to transfer rigging and roof loads to the perimeter walls. Based on discussions with Mr. Angelo Spiliotis, Project Structural Engineer with Biggs Cardosa Associates, we understand that a uniform floor load of about 200 pounds per square foot (psf) and column loads of about 200 kips dead load plus live load (DL+LL) along the perimeter walls are anticipated. No below

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grade structures are currently planned. We understand that the finished floor of the Ballroom Expansion is currently planned to be positioned as close to existing grade as possible to minimize earthwork.

SCOPE OF WORK

Our principal objective during this phase of the project has been to assist the City and Design Team with selection of a practical foundation support system for the proposed Ballroom Expansion that will be compatible with the subsurface conditions. We have advanced new explorations at strategic locations to supplement existing information in our files, evaluated the engineering properties of the soils, and developed site-specific geotechnical recommendations and design criteria for the following items:

- Geologic hazards update;
- Foundation design parameters;
- Estimated settlements for the selected foundation system;
- Resistance to lateral loads;
- Pavement structural sections;
- Support of slab-on-grade and concrete flatwork;
- Preparation of areas to receive fill;
- General site preparation and grading;
- Liquefaction assessment;
- Evaluation of expansive soils; and
- California Building Code (CBC) seismic coefficients.

SITE GEOLOGY AND GEOLOGIC HAZARDS

The following information sources were reviewed as part of our geologic hazards update:

- Geologic maps and reports published by the U.S. Geological Survey;
- Geologic maps and reports published by the California Geological Survey;
- Alquist-Priolo zone fault maps published by the California Geological Survey;
- On-line seismic hazard zone maps from the California Geological Survey;
- On-line geologic hazard zone maps from Santa Clara County; and.
- On-line flood hazard maps from ABAG.

Geologic Setting

The project site is located one to two miles south of the salt evaporation ponds in south San Francisco Bay, within the northern portion of the Santa Clara Valley. Santa Clara Valley is an alluvial basin located between the Santa Cruz Mountains to the southwest and the Diablo Range to the northeast. The Santa Clara Valley is located between the active San Andreas

fault to the west, and the Hayward and Calaveras faults to the east. Each of these faults has produced damaging earthquakes during historic time. The valley margins are marked by belts of active thrust faults - the Foothills fault system to the southwest and the East Valley thrusts (Southeast Extension of the Hayward fault) to the northeast (Fenton and Hitchcock, 2002).

The Foothills fault system is a series of southwest-dipping thrust faults located along the range front of the Santa Cruz Mountains (Bürgmann *et al.*, 1994). The Monte Vista-Shannon and Sargent faults are the main active faults in the Foothills thrust system. The Monte Vista-Shannon thrust is approximately 41 km long and dips at a moderate angle to the southwest, merging with the San Andreas fault at depth. The Sargent fault is approximately 56 km long and merges with the San Andreas fault near Loma Prieta.

The East Valley thrusts are a series of northeast-dipping thrust faults that mark the junction between the southern end of the Hayward fault and the southern and central segments of the Calaveras fault. These faults, which include the Quimby, Piercy, Evergreen and Silver Creek faults, are relatively short, less than 30 km long, and appear to merge with the Hayward and Calaveras faults at relatively shallow depths (Jones *et al.*, 1994). Recent geologic and geomorphic investigations along both the Foothills and East Valley thrust systems indicate that they are active and may be capable of generating damaging earthquakes (Hitchcock and Kelson, 1999; Fenton and Hitchcock, 2002).

The geology of the Santa Clara Convention Center site area has been mapped by Witter *et al.* (2006) as Holocene alluvial fan deposits. The geologic map of Santa Clara County, California (Brabb and Dibblee, 1974) maps the area as interfluvial fresh water basin deposits. These materials are described as organic clay and silty clay.

Geologic Hazards

Geologic Resources

Resources consulted for geologic hazard assessments included:

- Geologic maps of the U.S. Geological Survey;
- Alquist-Priolo Earthquake Fault Zone maps;
- On-line seismic hazard zone maps from the California Geological Survey;
- On-line geologic hazard zone maps from Santa Clara County;
- On-line flood hazard maps from the Association of Bay Area Governments (ABAG); and
- Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay region: U.S. Geological Survey, Open-File Report 2006-1037, in cooperation with the California Geological Survey.

Fault-Related Ground Rupture

Surface fault rupture tends to recur along existing fault traces. The highest potential for surface faulting is along existing fault traces that have had Holocene fault displacement. The California Geological Survey (formerly Division of Mines and Geology) has produced maps showing Alquist-Priolo Earthquake Fault Zones along faults with known Holocene activity

that pose a potential surface faulting hazard. There are no Alquist-Priolo (A-P) zones mapped in the vicinity of the site. In addition, the Santa Clara County Fault Rupture Hazard Zones map does not identify any fault hazard zones in the project area. The nearest mapped A-P zoned fault is the southern extension of the Hayward fault, located 9.3 km northeast of the site. A concealed trace of the northern extension of the Silver Creek fault (part of the East Valley thrusts) is mapped about 3.2 km northeast of the site by Bortugno et al (1991). The San Andreas fault is located about 19 km southwest of the site and the Calaveras fault is located about 15 km northeast of the site. Therefore, the potential for surface fault rupture at the site is considered low.

Landslide and Slope Failure

The project site is not within the mapped Santa Clara County Landslide Hazard Zone or the California Geological Survey Earthquake-Induced Landslide hazard zone. Due to the relatively flat topography at the site, landsliding is not a hazard.

Liquefaction

Soil liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary decrease of strength during earthquake ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the deposit becoming mobile and fluid-like.

The project site is within the mapped Santa Clara County Liquefaction Hazard Zone and the California Geological Survey Liquefaction hazard zone (CGS, 2001) for the San Jose East Quadrangle; it also is located in an area shown with a "high" liquefaction potential on the liquefaction susceptibility map (Witter, et al, 2006). Historic ground failures occurring as ground settlement from both the Loma Prieta and the 1906 San Francisco earthquake have been recorded by Knudsen, et al, (2000) along the margins of the Guadalupe River, located about 2 to 3 km east and northeast of the project site.

As described in more detail later in the report, the soil materials within approximately the upper 20 feet of the site are predominantly cohesive and generally not susceptible to liquefaction, although there is an interbedded zone that contains medium dense silty sand below depths of approximately 18 to 39 feet. These silty sand interbeds are potentially liquefiable under strong earthquake shaking. However, because the site is blanketed by a stiff to hard clay cap, we believe the risk of ground failure is low. Rather, the most likely consequence would be post-liquefaction induced ground surface settlement. We estimate such settlements could be up to about 1½ inches. A more detailed discussion of static and earthquake related settlement is presented in the section entitled, "Estimated Settlements".

Flooding

The project site is located immediately west of San Tomas Aquino Creek. However the creek is channelized at this location and flooding at the site is not a potential hazard. The site is located outside of the FEMA 100-year flood zone as shown on the ABAG flood hazard zone maps. The site is also outside of the Dike Failure Hazard Zone as mapped by Santa Clara County.

SITE AND SUBSURFACE CONDITIONS

Site Conditions

The project site is located within an area that is partly covered by parking lot and a landscaped area. The existing parking lot, located on the southern half of the site, is paved with asphalt concrete and has trees and shrubs along the borders. The landscaped area, located on the northern half of the site, is predominantly grass covered with trees at the northeastern end. Based on preliminary topographic information received from Steinberg Architects, the site area is relatively flat, ranging in elevation from about Elevation 9 to 11½. The landscaped area at the north end of the site is depressed to about Elevation 9, or about 2½ feet below adjacent grade (at about Elevation 11½ feet).

Subsurface Conditions

Field Exploration

Three borings and two cone penetration tests (CPT) were previously performed within or in close proximity to the footprint of the proposed Ballroom Expansion during 1983 and 1984 studies for the original Convention Center project. These explorations, labeled 1 and 5 (1983), and B-1, C-1, and C-4 (1984), on Figure 1, extended to depths ranging from 40½ to 141½ feet. Logs of these previous borings and CPTs are presented in Appendix A.

In order to supplement our existing subsurface information, we completed two (2) additional CPTs to a depth of 70 feet below the existing ground surface (bgs). In addition, three shallow (3 feet deep or less) exploratory borings were completed using a hand auger in order to better evaluate the characteristics of the near surface soils in the landscaped area. The approximate locations of the existing and new explorations are shown on the Site and Boring Location Plan, Figure 1. Samples of the soils retrieved from the hand auger holes were carefully sealed in the field and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further examination and laboratory testing of selected soil samples. These tests included moisture content and Atterberg Limits. The results of these tests are presented in Appendix B.

Logs of the exploratory borings were prepared based on review of field logs, visual examination of the soil samples, and results of laboratory tests. The Unified Soil Classification System and guidelines summarizing soil consistency and relative density terminology used in preparation of the boring logs are presented in Appendix B as Figure B-1. Figure B-2 illustrates the notation used for the types of samplers and methods of advancing them. Descriptions of the soils encountered at each location are presented on the Logs of Borings in Figures B-3 through B-5.

A more comprehensive discussion of the subsurface exploration techniques and the laboratory testing program is presented in Appendix B.

Soil and Groundwater Conditions

We believe that the proposed ballroom expansion area was covered by a thin layer of fill to raise the site grade during construction of the Convention Center and adjacent hotel. Based

on our 3 shallow hand auger borings, it appears that the fill consists of about 1 to 1½ feet of moderate to high plasticity clay. However, poorly graded sand also was noted at the terminal depth of Boring B3 (1¾ feet deep). Below existing pavements and existing fills, the site is generally blanketed by 3 to 9 feet of medium to hard, lean to fat clay of moderate to high plasticity, underlain by thick deposits of medium to very stiff silty and sandy lean clay. This lean clay layer extends to depths ranging from about 18 to 39 feet below the ground surface (bgs). Below the lean clay, about a 5 to 16-foot thick "upper" interbedded zone of medium dense to very dense silty sand and medium to stiff lean clay was encountered. Below the upper interbedded zone, the soils consist primarily of medium to very stiff lean clay to depths ranging from about 61 to 75 feet. This deeper clay layer is, in turn, underlain by a "lower" interbedded zone of dense to very dense silty sand and medium to very stiff lean clay to the maximum depth explored (141½ feet). The sand layers in this lower interbedded zone typically range in thickness from about 2 to 11 feet.

Idealized soil profiles A-A' and B-B' graphically present the subsurface conditions encountered in the field explorations completed in the vicinity of the Ballroom expansion and are presented in Figures 2 and 3, respectively.

Water level measurements taken in borings drilled across the Convention Center site indicated groundwater at a depth as high as approximately 6 feet (Elevation 3½ feet) in April 1983, and 9 feet (Elevation 1 foot) in September 1983. Although this indicates that some seasonal changes could occur with water levels rising during the winter months, the design high groundwater level should be 6 feet bgs.

DISCUSSION

The primary geotechnical considerations at the site are the moderate compressibility of the native clays under the anticipated building loads and the potential for liquefaction of interbedded sand deposits positioned below the groundwater table. The majority of the existing Convention Center is supported on driven piles. Driven piles were selected due to the heavy column loading. However, the Lobby addition completed in the mid-1990s had significantly lower column loads in the range of 200 to 320 kips DL+LL and was successfully supported on shallow spread footing foundations. Considering the similarity of subsurface conditions beneath the Lobby area and the current project site, as well as the loading of the structures, we believe that spread footing foundations also should be feasible for the Ballroom Expansion. Settlement estimates based on column loads provided by the Project Structural Engineer are discussed in the following section.

Because moderately to highly expansive clay is present across the site, either as fill in the landscaped area or native clay under the pavement, measures to minimize the potential for shrinking or swelling below the Ballroom Expansion floor also will be required.

RECOMMENDATIONS

Building Foundations

Shallow Foundations

Depending upon the final grading plans, engineered fill might be required to raise the site grade at some locations. Based on the anticipated column loads, we believe that the subsurface soils can support shallow foundations. Therefore, we recommend that the proposed Ballroom Expansion be supported on either spread footing or mat foundations bearing directly on the native soils or well compacted engineered fill. All existing fills located beneath the shallow foundations should be removed. Engineered fill sections beneath shallow foundations should extend a minimum of 5 feet outside of foundation plan areas.

Due to the moderately to highly expansive near surface clays, shallow foundations bearing on native soils or engineered fill should be embedded at least 30 inches below the lowest adjacent finished grade taken as bottom of interior slab-on-grade or lowest exterior grade, whichever is lower. Shallow foundations should be designed for bearing pressures not to exceed 2,000 pounds per square foot (psf) due to dead load, 3,000 psf due to dead plus live loads, and 4,000 psf for all loads including wind and seismic.

We understand that a shallow mat type foundation is currently being planned for storage racks at the project site. Typically, mat foundations are designed for initial deflection using a computer program that models the mat-soil interaction as a plate on an elastic foundation. We recommend a modulus of subgrade reaction (k) of 125 kips per cubic foot be used in these analyses.

Estimated Settlements

We estimate that the maximum consolidation settlement of spread footings designed as recommended herein should not exceed ¾ inch throughout the entire site. These settlement estimates assume a maximum column load of 200 kips (DL+LL). As discussed in the geology section of the report, we estimate that liquefaction settlement up to about 1½ inches could occur in the event of a strong earthquake. However, potentially liquefiable sand was absent in the exploration made in the north part of the building footprint. Based on the conditions revealed in the borings and CPTs, we estimate the liquefaction settlement could be distributed as follows:

Building Area	Estimated Liquefaction Settlement (inches)
North Corner	0
West Corner	½
East Corner	1
South Corner	1½

Based the variable thickness of potentially liquefiable sand across the project site, we estimate that a maximum differential settlement on the order of 1 inch per 150 feet could be realized at the site. Since the potentially liquefiable sands are overlain by a thick clay cap, we expect it will gradually distribute the effects over this length.

Foundation Resistance to Lateral Loads

Resistance to transient lateral loads from wind or earthquakes can be developed by friction between the bottom of the footings and the soil, and passive resistance on the front face of the footings. An ultimate coefficient of friction of 0.3 should be used between the bottom of the footings and underlying soil, with the total resistance not to exceed 1,000 psf; this assumes that the footings are cast neat against undisturbed native soil or engineered fill. Ultimate passive resistance of the soil should be estimated using an equivalent fluid weight of 400 pounds per cubic foot (pcf) acting against the footings. The upper 12 inches of embedment should be neglected for design. The recommended values presented above are ultimate values, and should be used with an appropriate factor of safety.

Slab-on-Grade Floors and Concrete Flatwork

All concrete slab-on-grade floors and flatwork should be supported on a minimum of 2 feet of select non-expansive engineered fill meeting the requirements presented in the "Fill Materials" section of this report. The fill is intended to provide a cap over the existing moderately to highly plastic clay, and to minimize the risk of seasonal shrinking and swelling. Due to the highly plastic nature of the existing fill in the landscaped area, we recommend that the material be removed and hauled offsite or to another landscaping area within the Convention Center area. In any case, fat clay should not be reused within the engineered fill section under the Ballroom Expansion floor.

Moisture will come into contact with the floor slab due to moisture vapor migration and/or capillary water rise through the soil. If floor coverings susceptible to damage by moisture contact will be placed on the slabs, moisture barriers should be used under the slabs. We recommend that Stego Wrap (minimum 15 millimeters thick) or equivalent vapor barrier be used in accordance with the manufacturer's specifications. Granular material placed 1) as damage protection above the vapor barrier, and/or 2) as part of a capillary break/moisture barrier system below the vapor barrier, can be considered as part of the recommended select fill section beneath the slab-on-grade floor.

For sidewalks and entryways, the top 6 inches of the engineered fill section should consist of Class 2 aggregate base. The fill should be placed and compacted as described later in this report.

Seismic Data and CBC Recommendations

The site is located in the seismically active San Francisco Bay region and will likely be subjected to strong shaking during the life of the project (Seismic Zone = 4). Several known faults in the region have the potential to generate strong shaking at the site. Those faults are listed in the table below along with the distance from the site, classification based on the California Building Code (CBC, 2001), and calculated peak ground acceleration (PGA).

Fault	Distance (km)	Fault Type per CBC	PGA (g)
East Valley Thrust	3.2	C	0.53
Hayward SE	9.3	B	0.28
Hayward fault	24	A	0.19
San Andreas fault	19	A	0.32
Calaveras fault	15	A	0.27

The subsurface conditions underlying the project site consist of relatively stiff cohesive soils with interbedded lenses and thin layers of medium dense to very dense sand. We recommend a CBC Soil Profile Type S_D be used to represent the subsurface conditions. The following near source factors should be applied due to the proximity of the site to SE extension of the Hayward fault:

- $N_a = 1.0$
- $N_v = 1.03$

Pavements

Based on field observations as well as discussions with City of Santa Clara staff, the existing pavement areas in the vicinity of this project site have been in place for about 20 years, or their typical design life. We understand that these existing pavement sections remain in reasonably good condition. Since similar traffic is anticipated for the areas adjoining the Ballroom Expansion project, we recommend the same pavement sections be used where new pavement is required. The following pavement sections are based on placing the pavement on the existing native soils or engineered fill.

Traffic Type	Recommended Pavement Section (inches)		
	Portland Cement Concrete (PCC)	Asphalt Concrete	Class 2 Aggregate Base
Automobile Traffic and Parking Lot Area	-	4	9
On-Site Access Road	-	6	12
Truck Access and Parking*	7	-	6

* We recommend that Portland Cement Concrete pavement be used in heavy truck traffic areas such as loading docks.

All pavement sections should be constructed in accordance with Caltrans Standard Specifications, latest edition, except that the relative compaction should be based on ASTM Test Designation D 1557. In particular, the asphalt concrete pavements should conform to Caltrans Section 39, the concrete pavements should conform to Sections 40 and 90 and the Class 2 Aggregate Base should conform to Section 26 of Caltrans Standard Specifications. The top six inches of the pavement section subgrade should be compacted to at least 95 percent relative compaction. Additionally, all aggregate base should be compacted to at least 95 percent relative compaction.

Additional recommendations for PCC pavement are as follows:

- Concrete should have a minimum modulus of rupture of at least 550 pounds per square inch (equivalent to a compressive strength of 3,700 psi) before the pavement is subjected to traffic.
- Provide expansion joints between buildings and pavements; the Contractor should provide a shop drawing indicating the proposed joint material.
- Provide weakened plane contraction joints at maximum 12-foot grid spacing by either saw cutting to a minimum depth of 3 inches or installing preformed material full depth; the purpose of these joints is to relieve tensile stresses, thereby minimizing the potential for volunteer cracking elsewhere in the pavement.
  - Saw cut width should be the minimum possible and less than ¼ inch.
  - Saw cut should occur within the time period specified in Caltrans Standard Specifications Section 40-1.08B (1). Timing of the saw cutting is of the utmost importance, since it is necessary to saw the joint before volunteer cracking occurs. Typically, this is within 12 to 24 hours after concrete placement.
  - All joints should be sealed with joint filler in accordance with Caltrans Standard Specifications Section 40-1.08B (1).
- Length of any given panel should not exceed its width by more than 25 percent.
- Provide 6X6-W1.5XW1.5 welded wire mesh.
  - Place in middle of slab.
  - Do not place across joints.

A representative of URS should be retained during construction to review the soil conditions encountered and the construction procedures used.

Site Preparation and Earthwork

All site preparation and earthwork should be done under the observation of a representative from our firm and in accordance with the recommendations presented herein. In addition, a URS representative should review all earthwork, foundation, shoring, and geotechnical related specifications prior to contract bidding.

Demolition

Existing improvements designated for demolition should be removed within the project area. Existing structures, asphalt, concrete, buried utilities and the root systems of existing trees

should be removed in their entirety. Depressions resulting from the demolition operation or tree removal should be left open for ease of identification during site preparation prior to construction of the building pad.

Site Preparation

Following the completion of demolition, areas to be graded should be stripped and cleared of surface vegetation, including all roots 1 inch diameter or larger, debris, and organic-laden topsoil. Materials resulting from clearing and stripping operations should be removed from the site. Organic topsoil that does not contain debris may be stockpiled, if desired, for re-use in landscape areas if approved by the Landscape Architect. The stripped materials should not be used as compacted fill or blended with other materials. Recycled asphalt and baserock from the existing pavement sections can be reused as engineered fill provided that it meets the requirements for select fill as described below in the "Fill Materials" section. However, no asphalt should be located under or within 5 lateral feet of any buildings proposed at the site. A URS field representative should review the general site preparation.

Excavations

After the site surface has been cleared, the building areas should be excavated as required to bring those areas to their finish subgrade elevations and to allow for construction of the recommended sections of engineered fill beneath the floor slab and footings, as applicable. All existing fill at the site located below or within 5 feet horizontally of structures should be removed in its entirety. Where new fill is required, foundation excavations should extend laterally a minimum of 5 feet outside of the edge of the footings. Because it is currently planned to keep the finished floor near existing grade, only minor cuts and fills should be anticipated.

Preparation for Filling

After demolition, stripping, and subgrade excavations are complete, the exposed surface to be filled should further be prepared by scarifying the top 6 inches, moisture conditioning the material to near the optimum moisture content, and recompacting it to a minimum relative compaction of 90 percent of the maximum dry density in accordance with ASTM Test Designation D 1557. It should be unnecessary to scarify the native soils exposed in footing excavations. However, all loose and/or disturbed materials should be removed prior to the placement of reinforcing steel and concrete.

Fill Materials

Soil materials, whether from sources on or off site, should be approved by a URS representative for the intended use and specifically for a required location or purpose.

Due to the relatively high plasticity of the existing fills and native soils at the site, their use should be avoided as these materials become difficult to compact when wet and have expansion potential that can be potentially damaging to structures, pavements and slabs. Less plastic onsite materials considered for reuse as general fill should not contain rocks or lumps over 6 inches in greatest dimension and not more than 15 percent greater than 2½ inches. The material should also be free of organic matter, rubble, or other deleterious substances.

In addition to the above requirements for general fill materials, select material should be a non-expansive soil or soil-rock material having a plasticity index not greater than 12. All imported fill material should meet the requirements of select material. In addition, fill material to be placed within a horizontal distance of 5 feet of any structure should be of select quality.

All imported fill material should be tested and approved by URS prior to being brought to the site. The Landscape Architect should also confirm the proposed import fill, when used near the finished grade of landscaped areas, is suitable for horticultural purposes with respect to both chemical makeup and drainage characteristics.

Fill Placement and Compaction

Fill material should be spread in uniform lifts not exceeding 8 inches in uncompacted thickness. If placed with hand operated equipment, the lift thickness should be reduced to 4 inches. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either aerating the material if it is too wet, or spraying the material with water if it is too dry. The native moderately to highly expansive clays should be compacted at 1 to 2 percent over the laboratory optimum moisture content. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of water content. To prevent drying of the subgrade soils, placement of fill should start immediately after surface preparation and should proceed in a continuous operation until the site is brought to grade. It should be the responsibility of the Grading Contractor to attain the proper moisture content during compaction. No fill should be placed during rain or when saturation will hinder proper compaction. The finished grade surface of the compacted fill under floor slabs or exterior concrete flatwork should be kept in a moist condition prior to the placement of concrete. Jetting or flooding of the fill should not be permitted.

All fill material below the bottom of footings and all other structural elements should be compacted to a minimum relative compaction of 95 percent. All other fill materials should be compacted to a minimum relative compaction of 90 percent. A URS representative should be present to observe all grading operations during both preparation of the site and compaction of engineered fill.

Utility Trench Backfill

All trench backfill should be placed in accordance with the compaction requirements and procedures for engineered fill described above and the additional recommendations presented in this section. For trenches deeper than 10 feet, compact the entire trench to 92 percent relative compaction.

The material should be compacted thoroughly as it is placed, to provide uniform support for the pipe barrel, and to completely fill all voids under and around the pipe. The bedding materials surrounding the pipe should be brought up at substantially the same rate on both sides of the pipe, to balance side pressures. The difference in backfill elevation between the sides should not exceed 6 inches.

The material surrounding the pipe should be carefully compacted by hand operated mechanical compaction equipment only, to prevent disturbance of the trench subgrade and to

avoid overstressing the pipe. As an alternative, a sheepsfoot compactor mounted on the end of a backhoe boom might be used for compaction. Heavy equipment operating over the pipe could result in damage to the pipe where soil cover is not sufficient. General guide specifications developed by pipe manufacturers recommend that, in order to prevent excessive live loads on the pipe (rigid RCP), not less than 3 feet of compacted soil cover should be in-place before power operated equipment travels over the pipe. It is recommended that the minimum soil cover requirements specified by the manufacturers be used. The trench backfill materials below this level should be carefully compacted with hand operated compaction equipment to avoid overstressing the pipe. Jetting or flooding of backfill should not be permitted.

Treatment after Completion of Grading

After grading is completed and our field representative has finished observing the work, no further excavation or filling should be done except with the approval of and under the observation of a representative of our firm. If, after the grading is complete the subgrade is disturbed or an extended period of time has elapsed, we should be notified to re-evaluate the subgrade conditions prior to construction of overlying aggregate base or structures.

It should be the responsibility of the Grading Contractor to prevent erosion of freshly graded areas during construction and until such time as permanent drainage has been installed.

Temporary Construction Excavations

Safety standards set by OSHA limit the height of unshored vertical excavations to 5 feet if construction personnel will be working in the excavations. In the event excavations greater than 5 feet are required, we recommend using the guidelines published by OSHA. The guidelines classify soils in detail as Type A, B, or C. In general, Type A soils are stronger, Type B soils are intermediate, and Type C soils are weaker. Based on the soil type, depth, duration the excavation is open, and sequence of soils exposed in excavation, OSHA recommends maximum allowable slopes. For example, for excavations in homogeneous soils 20 feet or less in depth, they state that maximum allowable slopes (horizontal to vertical) should be ¾ to 1, 1 to 1, and 1½ to 1 for Type A, B and C soils, respectively. Based on the strength of the soil where temporary cuts will be made, the soils at this site are considered to be OSHA Type B.

Sloughing, raveling or erosion could require shoring or flattening of the temporary excavation slopes. Equipment and stockpiles should not be located within 10 feet from the edge of excavations.

The Contractor should be responsible for shoring if excavations are located within 5 feet of any existing utilities, structures or traveled roadway.

LIMITATIONS

The opinions, conclusions, and recommendations contained in this report are based upon the information obtained from exploratory borings and CPTs made at widely separated locations, review of available data, and upon local experience and engineering judgment. The recommendations presented in this report are based on the assumptions that the soil and

geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the exploratory borings and CPTs. In addition, our geotechnical opinions, conclusion and recommendations are based on currently available design information. Geotechnical issues may arise during design that are not apparent at this time. A URS representative should review any design changes that could impact our recommendations.

If any variations or undesirable conditions are encountered during construction, URS should be contacted so that supplementary recommendations can be made. Also, if the construction is changed from that presently conceived, URS should review the changes and make any necessary modifications to the original recommendations presented in the report in order to meet the project needs.

No investigation of subsurface environmental contamination was performed.

URS should review the final specifications and drawings when they are available, to verify these documents are consistent with the intent of the geotechnical recommendations.

The recommendations presented in this report were developed with the standard of care commonly used as state of the practice in the profession. No other warranties are included, either express or implied, as to the professional advice included in this report.

We are pleased to be of assistance to the City of Santa Clara. Please contact us if any questions arise.

Sincerely,

L. Allen Moore, G.E.
Project Manager

Paul J. Boddie, G.E.
Geotechnical Department Manager

Attachments

REGISTERED PROFESSIONAL ENGINEER
LAURENCE ALLEN MOORE
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GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
PAUL J. BODDIE
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Exp. 3/31/08
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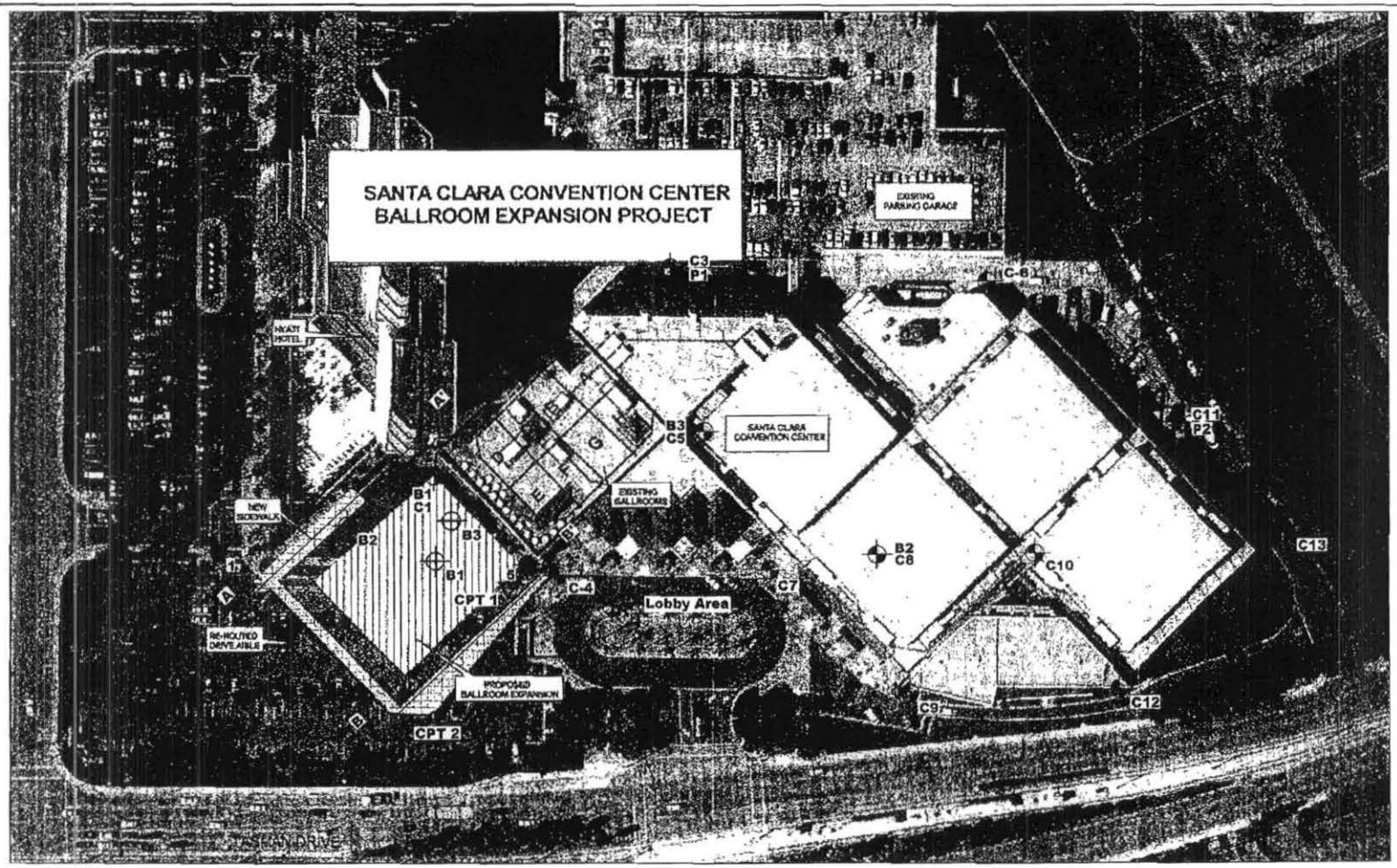
STATE OF CALIFORNIA



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LEGEND:

- WOODWARD-CLYDE CONSULTANTS (URS) - 1983
  - S - Exploratory Boring (AUGER)
- A A' - Cross section A-A'
- WOODWARD-CLYDE CONSULTANTS (URS) - 1964
  - B1 - Exploratory Boring (ROTARY WASH)
  - C1 - Cone Penetrometer Test
  - P1 - Menard Type Pressuremeter Test
- CONE PENETROMETER TEST COMPLETED FOR THIS STUDY
- B1 EXPLORATORY BORING COMPLETED AS PART OF THIS STUDY

100 0 100

APPROXIMATE SCALE : 1 INCH=100 FEET

URS No. 28649782

URS	Santa Clara Convention Center Ballroom Expansion Project	SITE AND BORING LOCATION PLAN	MARCH, 2007 Figure 1

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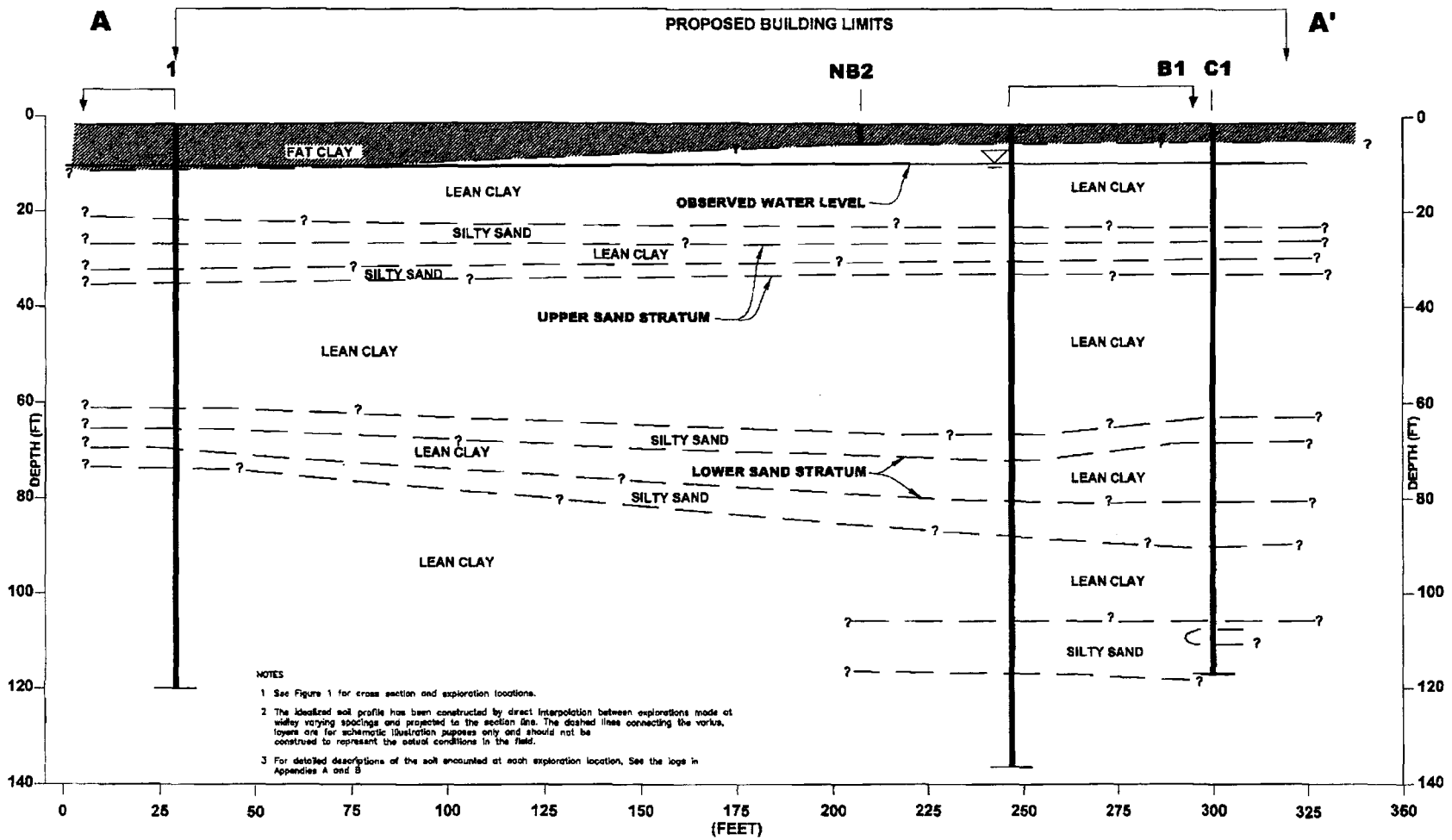

NOTES

1 See Figure 1 for cross section and exploration locations.

2 The idealized soil profile has been constructed by direct interpolation between explorations made at widley varying spacings and projected to the section line. The dashed lines connecting the varius, layers are for schematic illustration pupose only and should not be construed to represent the actual conditions in the field.

3 For detailed descriptions of the soil encountered at each exploration location, See the logs in Appendies A and B

URS	Santa Clara Convention Center Ballroom Expansion Project	CROSS SECTION A-A'	Project No. 28648762 Figure 2 3/08/07

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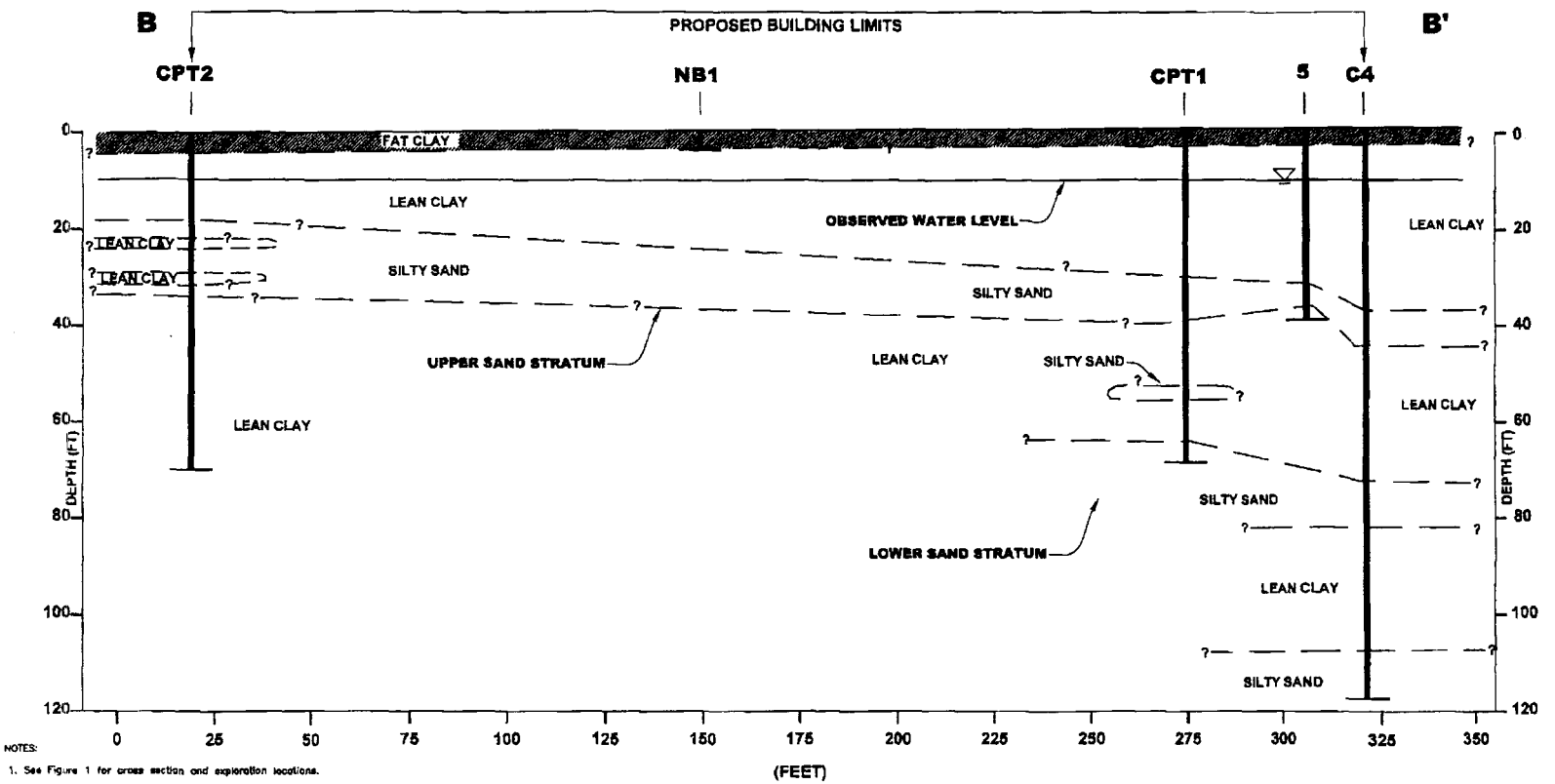

NOTES:

1. See Figure 1 for cross section and exploration locations.
2. The idealized soil profile has been constructed by direct interpolation between explorations made at widely varying spacings and projected to the section line. The dashed lines connecting the various layers are for schematic illustration puposes only and should not be construed to represent the actual conditions in the field.
3. For detailed descriptions of the soil encounted at each exploration location, See the logs in Appendix A and B.

URS	Santa Clara Convention Center Ballroom Expansion Project	CROSS SECTION B-B'	Project No. 29649762 Figure 3 3/09/07

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Project: SANTA CLARA CONFERENCE CENTER Santa Clara, California	**Log of Boring No. 1**

Date Drilled: May 6, 1983 — Remarks:
Type of Boring: 6" Auger
Hammer Weight: 140 lbs. — (See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9.0±

Depth, ft	Samples	Blows/ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	16	SILTY CLAY (CH) Stiff, wet, dark gray Moist, dark gray to brown-gray mottled	22	104	2920
5	2	22	SILTY CLAY (CL-CH) Medium, wet, light brown to brown mottled Very stiff, moist, CL ATD	26	98	5110
10	3	7	SILTY CLAY (CL) Medium, brown mottled with tan and gray, with some fine sand Becoming silty (CL-ML)	24	103	1560
15	4	22	SILTY CLAY (CL) Very stiff, brown and gray mottled	22	105	5750
20	5	13	CLAYEY SAND (SC) Medium dense, brown and gray Slightly coarser (SP-SM) -#200 = 32%	21	106	680
25	6	12	SILTY CLAY (CL) Stiff, brown and gray, with layers of sandy clay, clayey sand, and sand	23	103	2920
30						

LABORATORY TESTS

Proj. No. 15682V	Woodward-Clyde Consultants	Figure 3a

Project: SANTA CLARA CONFERENCE CENTER
Santa Clara, California

Log of Boring No. 1 (Continued)

Depth, ft.	Samples	Blows/ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	7	18	Sand lens — SILTY CLAY (CL) with interbedded sandy layers.....Cont'd	-	-	-
35						
	8	14	Clayey sand lens — -#200 = 49%	21	106	1020
40			Medium, with some fine sand zones			
	9	12		25	99	1510
45						
	10	15		24	103	1900
50			Stiff			
	11	15		24	101	2440
55			Very stiff			
	12	25		25	102	7160
60						
	13	39	SILTY SAND (SM) Medium dense, gray and brown mottled — Grading to SAND (SP), dense	-	-	-
65			SANDY CLAY (CL): Very stiff, brown & gray mottled			

Proj. No. 16582V | Woodward-Clyde Consultants | Figure 3b

Project: SANTA CLARA CONFERENCE CENTER Santa Clara, California	Log of Boring No. 1 (Continued)

Depth, ft.	Samples	Blows/ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	14	28	Grading to SILTY CLAY (CL), with some fine sand	21	107	5160
70						
	15	30	SILTY SAND / SANDY SILT (SM-ML) Medium dense, brown and gray mottled	25	99	5850
			SILTY CLAY (CL) Very stiff, gray and brown mottled, with some fine sand			
75			(CL-ML), Stiff			
	16	39		22	106	3460
80			BOTTOM OF BORING @ 80'			
85						
90						
95						

Proj. No. 15582V	Woodward-Clyde Consultants	Figure 3c

Project: SANTA CLARA CONFERENCE CENTER Santa Clara, California	**Log of Boring No. 5**

Date Drilled: April 6, 1983
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9.0±

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	10	SILTY CLAY (CH) Medium, moist, black Stiff	23	98	2480
5	2	16	SILTY CLAY (CL) Very stiff, moist gray-brown mottled, with trace of lime nodules Light brown to brown mottled	22	104	5890
10	3	31		26	99	7110
			ATD			
15	4	8	SILTY CLAY (CL) Stiff, brown and gray mottled, with fine sand Sand lens Becoming Silty Clay (CL-CH), less fine sand	22	104	2140
20	5	12		25	98	2530
25	6	9	SILTY CLAY (CL-ML) Medium, gray-brown, with trace of fine sand With sandy clay layers	26	97	1460
30	7	15	CLAYEY SAND (SC) Medium dense, gray and brown, with sandy clay zones Grading to Sandy Clay (CL)	18	114	1510

Proj. No. 15582V	Woodward-Clyde Consultants	Figure 7a

Project. SANTA CLARA CONFERENCE CENTER
Santa Clara, California

Log of Boring No. 5 (Continued)

Depth, ft.	Samples	Blows/ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	8	14	SAND (SW) Medium dense, gray	-	-	-
35						
			SILTY CLAY (CL) Stiff, gray-brown			
40	9	24		-	-	-
			BOTTOM OF BORING @ 40½'			
45						
50						
55						
60						
65						

Proj. No. 15582V | Woodward-Clyde Consultants | Figure 7b

Project: SANTA CLARA CONFERENCE CENTER, Santa Clara, California

Log of Boring No. B1

Date Drilled: September 21,22, & 23, 1983
Type of Boring: Rotary
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sample type and hammer weight)

Surface Elevation: 9.0

Depth, Ft.	Sample	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
	1	16	SILTY CLAY (CL-CH) Very stiff, damp, brown-gray	-	-	-
5	2		SILTY CLAY (CL) Stiff, moist, light brown-tan with dark brown specks, with layers of clayey silt. Becoming soft (9-24-83)	-	-	-
10	3	4		34	86	1660
15	4	21	Becoming very stiff	-	-	-
20	5	41	SILTY CLAY (CL) Very stiff, brown with gray specks, traces of large sand nodules	-	-	-
25	6	40	SILTY SAND (SP-SM) Dense, brown and gray, medium to fine	-	-	-
	7	29	SILTY CLAY (CL) Stiff, brown and gray, with layers of clayey silt and sandy clay, traces of sand nodules. Seams of dense, very fine silty sand	20	106	1900
30	8	39	Very stiff	21	107	4530
35	9	36		26	106	2020
40	10	37	Traces of coarse sand and fine gravel			
45	11	32		30	103	2430
50	12	57		28	94	3690
55	13	58	Changing to sandy clay	19	107	-
60	14	63	Seams of very fine silty sand	24	99	640
65	15	80	SILTY SAND (SP-SM) Very dense, gray, fine to very fine. Few fine gravel	-	-	-
70	16	51	SILTY CLAY (CL) Very stiff, mottled light brown and gray, with layers of clayey silt and sandy clay, traces of sand nodules	22	104	2130
75	17	60		17	112	2060
80	18	101	SILTY SAND (SP-SM) Very dense, gray and brown, fine	-	-	-
85	19	41	SILTY CLAY (CL) Stiff, mottled brown and gray, with layers of clayey silt and sandy clay, traces of sand nodules	-	-	-
90	20	45	Becoming very stiff	-	-	-
95	21	56		24	98	1700
100	22	24		-	-	-
105	23	84	SILTY SAND (SM) Very dense, gray, fine to very fine, with layers of very stiff, gray-brown silty clay	-	-	-
110	24	70		-	-	-
115	25	53	SILTY CLAY (CL-CH) Very stiff, greenish gray, traces of sand nodules	-	-	-
120	26	34		-	-	-
125	27	44	Occasional layers of sandy clay and silty sand	-	-	-
130			BOTTOM OF BORING @ 141.5 Feet			

Project: SANTA CLARA CONFERENCE CENTER, Santa Clara, California

~~Log of Boring No. B2~~

Date Drilled: September 22 & 26, 1983
Type of Boring: Rotary
Hammer Weight: 140 and 288 lbs.
Remarks:
(See Legend Sheet for sample type and hammer weight)

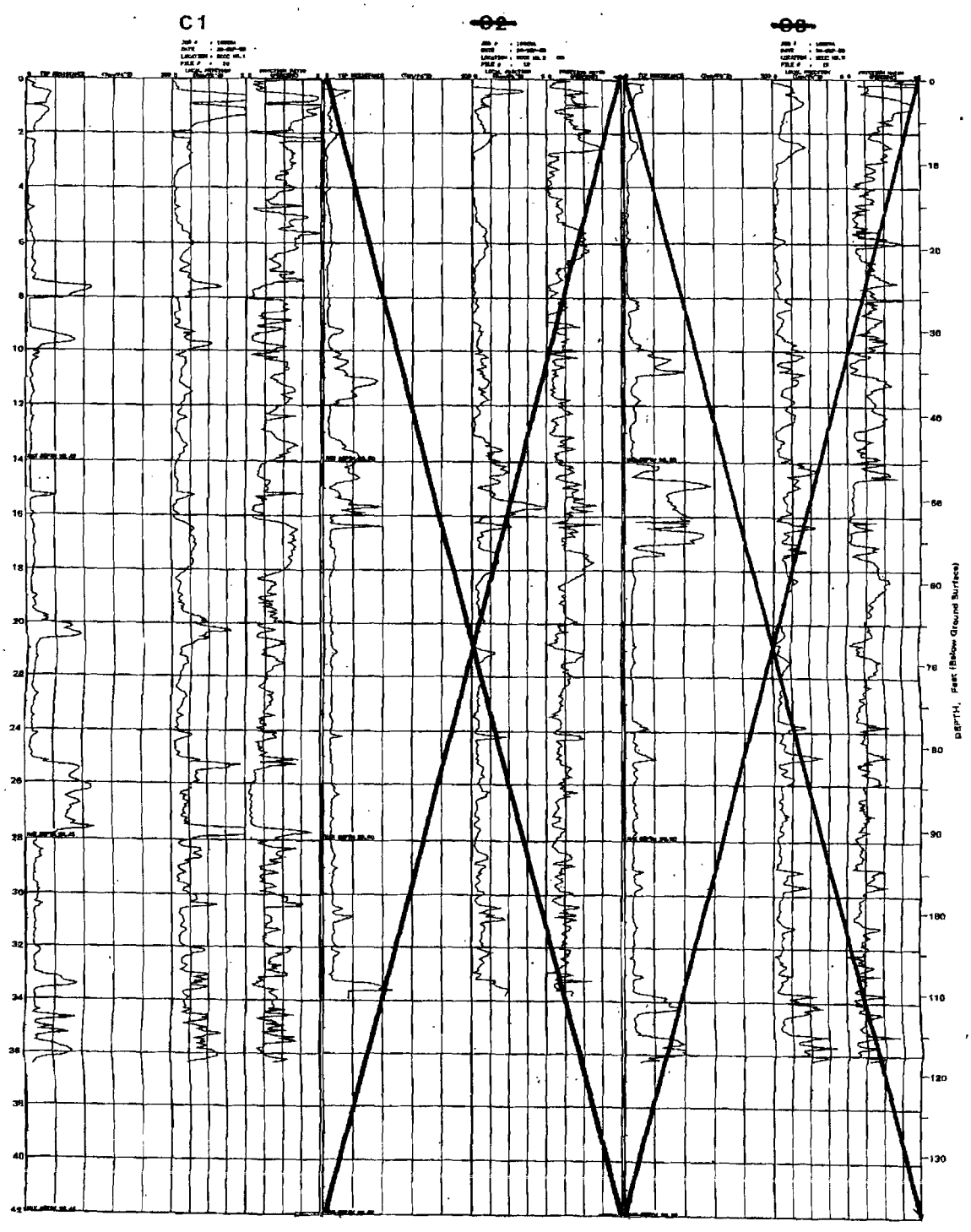
Surface Elevation: 9.7

Depth, Ft.	Sample	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
	1	11	SILTY CLAY (CL-CH) Very stiff, damp, brown-gray	-	-	-
5	2	20	SILTY CLAY (CL) Stiff, moist, light brown-tan, with dark brown specks, with layers of clayey silt, traces of sand nodules. Becoming soft (9-27-83)	-	-	-
10	3	17	Becoming medium stiff	-	-	-
15	4	12		-	-	-
20	5	30		24	103	3160
25	6	21	SILTY CLAY (CL) Stiff, brown with gray specks, with layers of clayey silt, traces of sand nodules	22	102	1810
30	7	25	Traces of large sand nodules	-	-	-
35	8	34		-	-	-
40	9	42	SILTY SAND (SP-SM) Very dense, brown-gray, fine to very fine, with occasional seams of stiff silty clay	-	-	-
45	10	37	SILTY CLAY (CL) Very stiff, brown and gray, with layers of clayey silt, traces of sand nodules	-	-	-
50	11	28		-	-	-
55	12	66		27	97	5020
60	13	52		27	97	3760
65	14	100*	SAND (SP) Very dense, brown and gray, fine to medium, with occasional seams of stiff silty clay	-	-	-
70	15	43		-	-	-
75	16	25*	SILTY CLAY (CL) Stiff, brown and gray, with layers of clayey silt, traces of sand nodules	22	106	1980
80	17	25*		-	-	-
85	18	31*		26	109	2950
90	19	32*		19	112	2600
95	20	41*		-	-	-
100	21	50*	Seam of very fine silty sand	-	-	-
105	22	85*		-	-	-
110	23	100*		-	-	-
	23A	50*				
115	24	45	SILTY SAND (SM) Very dense, brown and gray, fine to very fine, with occasional layers of stiff silty clay	-	-	-
120	25	44	SILTY CLAY (CL-CH) Very stiff, greenish gray, traces of sand nodules	-	-	-
125	26	82*	Occasional layers of sandy clay and silty sand	-	-	-
130	27	72*		-	-	-
135			BOTTOM OF BORING @ 141.5 Feet			

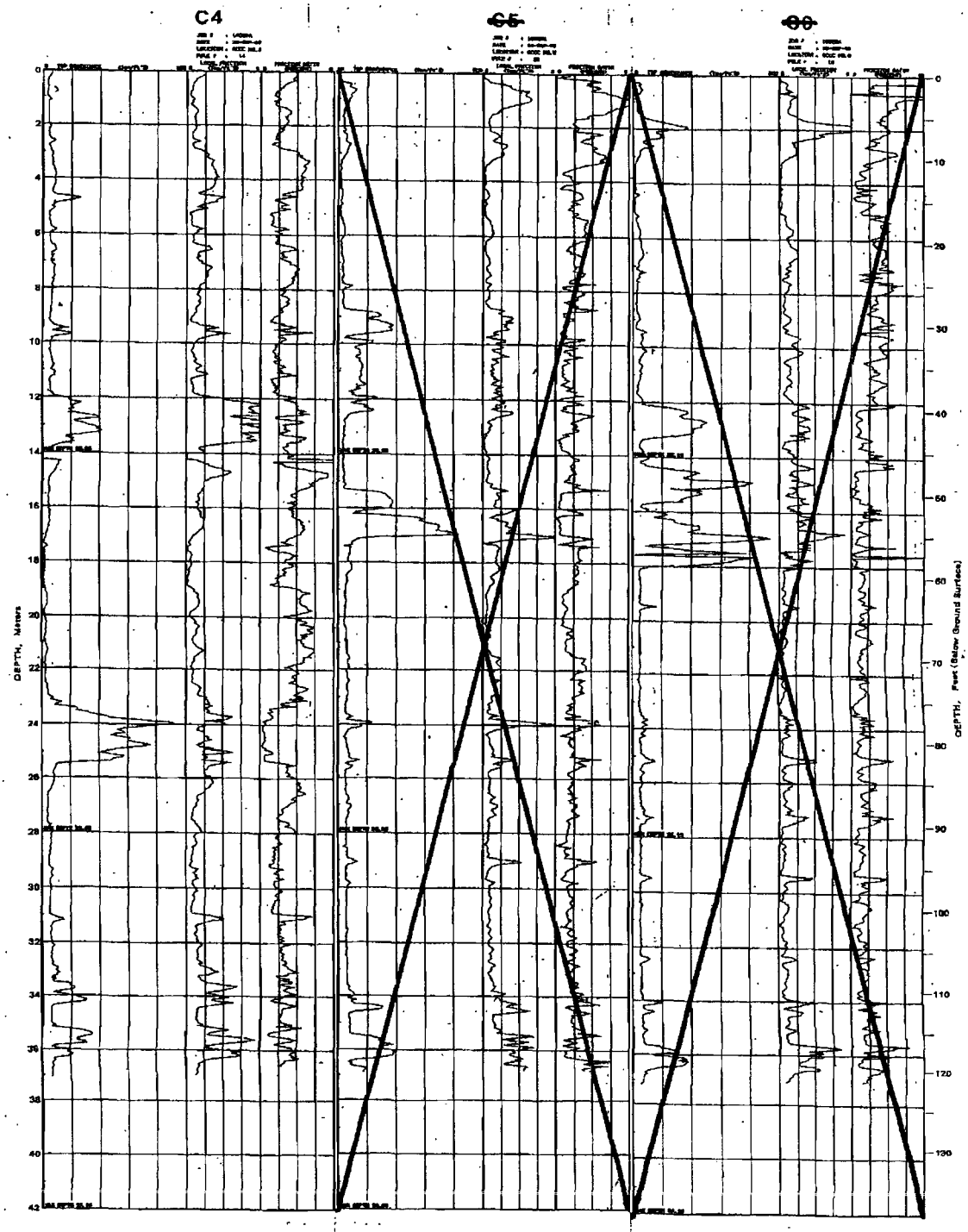
Project No. 15029A	Santa Clara Conference Center
Woodward-Clyde Consultants	

BORING LOGS FOR B1 AND B2

Figure A-2



Project No. 16029A	Santa Clara Conference Center	CONE PENETEROMETER TESTS C1, C2, AND C3	Figure A-5
Woodward-Clyde Consultants			



Project No. 16029A	Santa Clara Conference Center	CONE PENETEROMETER TESTS C4, C5, AND C6	Figure A-6
Woodward-Clyde Consultants			

FIELD EXPLORATION PROGRAM

Three (3) exploratory borings and 2 Cone Penetration Tests were performed for this investigation at the locations shown on the Site and Boring Location Plan, Figure 1. The borings were advanced to a maximum depth of 3 feet on August 14, 2006 using hand auger equipment. The CPTs were advanced on August 17, 2006 to a depth of 70 feet. The explorations were made under the supervision of a URS representative. At the completion of the work, the holes were backfilled with cement grout in accordance with Santa Clara Valley Water District requirements.

Exploratory Borings

Samples of the soils encountered in the exploratory borings were hand augered and collected with either a modified California sampler (2-inch inside diameter, 2-1/2-inch outside diameter) or as cuttings from the hand auger and placed in a sealed plastic bag. After advancing the sampler to the desired depth, the sampler was withdrawn from the borehole. The exposed soil was examined, classified, and the samples were sealed to preserve the natural moisture. The samples were taken to our geotechnical laboratory for further examination and testing.

Preliminary soil classifications were made in the field in accordance with the Unified Soil Classification System as shown on Figure B-1, and were verified by further examination of the samples in the laboratory and by testing. A legend of drilling and sampling operations is included as Figure B-2. Logs of the borings were prepared based on the field and laboratory test data and are presented in the Log of Test Borings, Figures B-3 through B-5.

Cone Penetration Tests

The CPT consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration. The CPTs described in this report were conducted in general accordance with ASTM specifications (ASTM D3441-79) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed all-wheel drive truck is used to transport and house the test equipment and to provide a 25-ton reaction to the thrust of the hydraulic rams.

The cone penetrometer assembly consists of a conical tip and a cylindrical friction sleeve. The conical tip has a 60-degree apex angle and a projected cross-sectional area of 15 square centimeters. The cylindrical friction sleeve has a surface area of 200 square centimeters. Both the conical tip and the cylindrical friction sleeve have outer diameters of 4.37 centimeters (about 1¾ inches). The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurement of cone tip and friction sleeve resistance during penetration. Continuous electric signals from the strain gauges are transmitted by a cable in the sounding rods to analog and digital data recorders in the CPT truck.

Data obtained during a CPT consist of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction

ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations that utilize the CPT data.

Computer plots for the reduced CPT data acquired for this investigation are presented in Figures B-6 and B-7.

LABORATORY TESTING

Samples recovered from the hand auger holes were sealed to prevent moisture loss. The samples were then transported to our San Jose laboratory for examination and testing. Laboratory tests were performed on selected samples as an aid in classifying the soils and to evaluate the physical properties of the soils. Descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

Moisture Content

Moisture content determinations were made on selected samples. The samples were first weighed and then were dried in accordance with D 2216 – 98, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass and D 2937 – 94, Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method. After drying, the weight of each sample was measured, and moisture content was calculated. The results of the individual tests are presented in the boring logs at the respective locations of the samples.

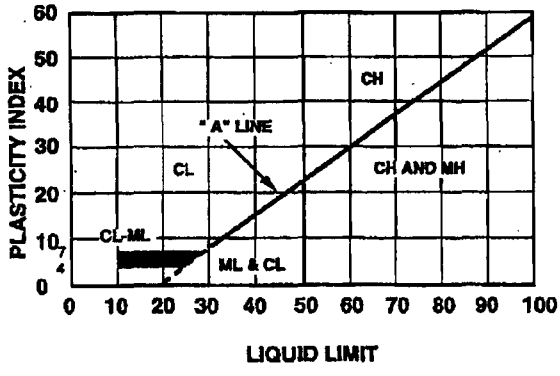
Plasticity Index

Plasticity characteristics were determined for selected samples by performing Liquid Limit and Plastic Limit tests generally in accordance with ASTM D 4318 – 98, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

SAMPLE CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATION SCHEME			
MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOIL (More than 1/2 of soil >no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
		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
	SAND (More than 1/2 of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOIL (More than 1/2 of soil <no. 200 sieve size)	SILTS & CLAYS Liquid Limit < 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey, fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

PLASTICITY CHART



GRAIN SIZE CLASSIFICATION		
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
GRAVEL	3" to No. 4	76.2 to 4.76
coarse (c)	3" to 3/4"	76.2 to 19.1
fine (f)	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse (c)	No. 4 to No. 10	4.76 to 2.00
medium (m)	No. 10 to No. 40	2.00 to 0.420
fine (f)	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

MOISTURE CONTENT	
DRY	-No sign of water and soil dry to touch
MOIST	-Signs of water and soil is relatively dry to touch
WET	-Signs of water and soil definitely wet to touch; granular soil exhibits some free water when densified

SOIL CONSISTENCY/RELATIVE DENSITY				
SILT, SAND AND GRAVEL	BLOWS/FT	SILT OR CLAY	UNCONFINED COMPRESSIVE STRENGTH (psf)	THUMB PENETRATION
Very loose	<4	Very Soft	< 500	Very easily - inches
Loose	5-10	Soft	500 - 1000	Easily - inches
Medium Dense	11-30	Medium (firm)	1000 - 2000	Moderate effort - inches
Dense	31-50	Stiff	2000 - 4000	Indented easily
Very Dense	>50	Very Stiff	4000 - 8000	Indented by nail
		Hard	> 8000	Difficult by nail

CLASSIFICATION MODIFIERS	
TRACE	0 - 12%
SOME	12 - 30%
± MODIFIERS	

URS

Figure B 1

SCCC BALLROOM; Santa Clara, California

BORING LOCATION:		GROUND SURFACE ELEVATION (ft): TOP OF WELL CASING ELEVATION (ft):
DRILLING AGENCY	DRILLER	DATE STARTED: DATE FINISHED:
DRILLING EQUIPMENT		COMPLETION DEPTHS BORING: 45.0 (ft) WELL: (ft)
DRILLING METHOD (as noted)	DRILL BIT	HAMMER/ DROP (as noted)
SIZE AND TYPE OF CASING		NUMBER OF SAMPLES DIST: UNDIST:
TYPE OF PERFORATION	FROM TO	WATER DEPTH (ft) FIRST: 28 COMPL.: 32 24 hr.:
SIZE AND TYPE OF PACK	FROM TO	LOGGED BY / CHECKED BY

TYPE OF SEAL	TYPE	FR	TO	TYPE	FR	TO
	No. 1:			No. 3:		
	No. 2:			No. 4:		

LOG OF LEGEND

(Sheet 1 of 1)

DEPTH (feet)	SOIL GRAPHIC	MATERIAL DESCRIPTION	ELEVATION (feet)	FIELD TESTS: POCKET PEN (tsf)	POCKET TV (psf)	STRAIN AT FAILURE,%	WATER LEVEL	DEPTH (feet)	SAMPLES: NUMBER	TYPE	RECOVERY (%)	BLOWS /foot	INDEX PROPERTIES: MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	NOTES
		Arrow denotes bottom of fill layer FILL														
5		2 inch inside diameter Modified California sample						5								
10		2 inch outside diameter Standard Split Spoon sample (Standard Penetration Test)						10								
		Bulk Sample														
15		Hydraulic Pressure required to push Shelby tube sampler						15				350 psi				
20		Blow count with 140-lb hammer falling 30 inches for 12 inches of penetration						20				29				
25		Blow count with 140-lb hammer falling 30 inches for 5 inches of penetration						25				50/ 5"				
		Groundwater level at time of drilling														
30		Groundwater at a time after drilling (as specified)						30								
35		**KEY TO LABORATORY TESTS**						35								
		PP= Pocket Penetrometer reading in tons per square foot (tsf)		3.0												PP=3.0tsf
		LL= Liquid Limit (%) PI= Plasticity Index (%) NOTE: PI= LL - (Plastic Limit (%))														LL=42 PI=21
40		+#4= Percentage of material retained on #4 sieve -#200= Percentage of material passing #200 sieve						40								+#4=13% -#200=10%

URS

PROJECT NO. 28649762

Figure: B-2

SCCC BALLROOM; Santa Clara, California

BORING LOCATION: Santa Clara Convention Center Ballroom			GROUND SURFACE ELEVATION (ft): TOP OF WELL CASING ELEVATION (ft): N/A
DRILLING AGENCY	DRILLER S.Ball		DATE STARTED: 8/14/06 DATE FINISHED: 8/14/06
DRILLING EQUIPMENT Hand Auger			COMPLETION DEPTHS BORING: 3.0 (ft) WELL: N/A (ft)
DRILLING METHOD Hand Auger	DRILL BIT ?		HAMMER/ DROP N/A
SIZE AND TYPE OF CASING N/A			NUMBER OF SAMPLES DIST: UNDIST:
TYPE OF PERFORATION N/A	FROM N/A TO N/A		WATER DEPTH (ft) FIRST: N/A COMPL.: N/A 24 hr.: N/A
SIZE AND TYPE OF PACK N/A	FROM N/A TO N/A		LOGGED BY S.Ball CHECKED BY A.Moore

TYPE OF SEAL	TYPE	FR	TO	TYPE	FR	TO
	No. 1: N/A	N/A	N/A	No. 3: N/A	N/A	N/A
	No. 2: N/A	N/A	N/A	No. 4: N/A	N/A	N/A

LOG OF BORING B1

(Sheet 1 of 1)

DEPTH (feet)	SOIL GRAPHIC	MATERIAL DESCRIPTION	ELEVATION (feet)	FIELD TESTS: POCKET PEN (tsf)	POCKET TV (psf)	STRAIN AT FAILURE,%	WATER LEVEL	DEPTH (feet)	SAMPLES: NUMBER	TYPE	RECOVERY (%)	BLOWS /foot	INDEX PROPERTIES: MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	NOTES
		Sandy lean to fat CLAY (CL/CH) with gravel Soft, wet, dark brown, some organics (grass)							1				37			
1		**Lean to fat CLAY (CL/CH)** Soft to medium, wet, dark gray brown with brown mottling, irrigation water							2							
									3				30			LL = 44 PI = 25
2		Gray with light red brown, sand mottling							4				28			
3		BOTTOM OF BORING AT 3 FEET														

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PROJECT NO. 28649762

Figure: B-3

SCCC BALLROOM; Santa Clara, California

BORING LOCATION: Santa Clara Convention Center Ballroom			GROUND SURFACE ELEVATION (ft): TOP OF WELL CASING ELEVATION (ft): N/A
DRILLING AGENCY	DRILLER S.Ball		DATE STARTED: 8/14/06 DATE FINISHED: 8/14/06
DRILLING EQUIPMENT Hand Auger			COMPLETION DEPTHS BORING: 2.5 (ft) WELL: N/A (ft)
DRILLING METHOD Hand Auger	DRILL BIT ?		HAMMER/ DROP N/A
SIZE AND TYPE OF CASING N/A			NUMBER OF SAMPLES DIST: UNDIST:
TYPE OF PERFORATION N/A	FROM N/A TO N/A		WATER DEPTH (ft) FIRST: N/A COMPL.: N/A 24 hr.: N/A
SIZE AND TYPE OF PACK N/A	FROM N/A TO N/A		LOGGED BY S.Ball CHECKED BY A.Moore

TYPE OF SEAL	TYPE	FR	TO	TYPE	FR	TO
	No. 1: N/A	N/A	N/A	No. 3: N/A	N/A	N/A
	No. 2: N/A	N/A	N/A	No. 4: N/A	N/A	N/A

LOG OF BORING B2

(Sheet 1 of 1)

DEPTH (feet)	SOIL GRAPHIC	MATERIAL DESCRIPTION	ELEVATION (feet)	FIELD TESTS: POCKET PEN (tsf)	POCKET TV (psf)	STRAIN AT FAILURE,%	WATER LEVEL	DEPTH (feet)	SAMPLES: NUMBER	TYPE	RECOVERY (%)	BLOWS /foot	INDEX PROPERTIES: MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	NOTES
		Sandy lean to fat CLAY (CL/CH) with gravel Soft to medium, moist to wet, gray brown, some fine gravel							1				29			LL=46 PI=26
1									2							
		Lean to fat CLAY (CL/CH) Medium, wet, dark gray with light red-brown sand mottling														
									3				29			
2		Hit gravel at 2-1/2 feet							4				29			
		BOTTOM OF BORING AT 2-1/2 FEET														
3																

URS

PROJECT NO. 28649762

Figure: B-4

SCCC BALLROOM; Santa Clara, California

BORING LOCATION: Santa Clara Convention Center Ballroom			GROUND SURFACE ELEVATION (ft): TOP OF WELL CASING ELEVATION (ft): N/A
DRILLING AGENCY		DRILLER S.Ball	DATE STARTED: 8/14/06 DATE FINISHED: 8/14/06
DRILLING EQUIPMENT	Hand Auger		COMPLETION DEPTHS BORING: 1.8 (ft) WELL: N/A (ft)
DRILLING METHOD	Hand Auger	DRILL BIT ?	HAMMER/ DROP N/A
SIZE AND TYPE OF CASING	N/A		NUMBER OF SAMPLES DIST: UNDIST:
TYPE OF PERFORATION	N/A	FROM N/A TO N/A	WATER DEPTH (ft) FIRST: N/A COMPL.: N/A 24 hr.: N/A
SIZE AND TYPE OF PACK	N/A	FROM N/A TO N/A	LOGGED BY S.Ball CHECKED BY A.Moore

TYPE OF SEAL	TYPE	FR	TO	TYPE	FR	TO
	No. 1: N/A	N/A	N/A	No. 3: N/A	N/A	N/A
	No. 2: N/A	N/A	N/A	No. 4: N/A	N/A	N/A

LOG OF BORING B3

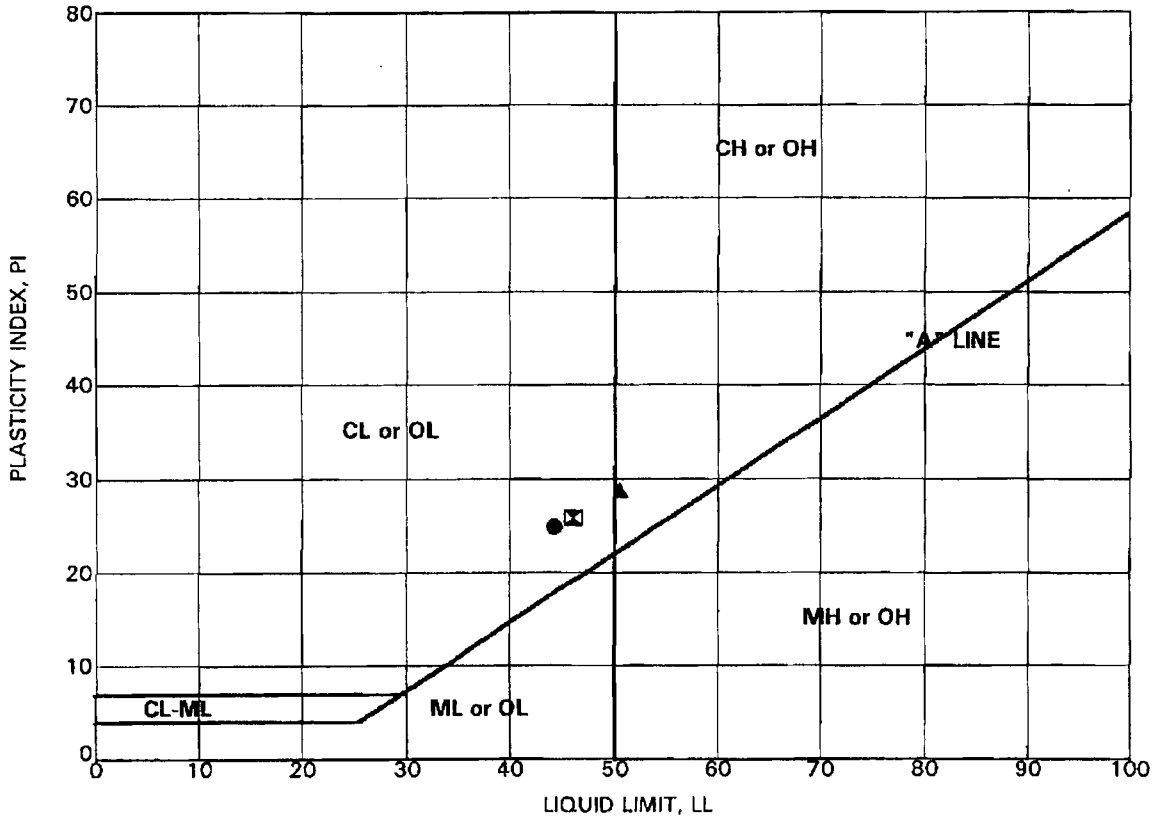
(Sheet 1 of 1)

DEPTH (feet)	SOIL GRAPHIC	MATERIAL DESCRIPTION	ELEVATION (feet)	FIELD TESTS: POCKET PEN (tsf)	POCKET TV (psf)	STRAIN AT FAILURE, %	WATER LEVEL	DEPTH (feet)	SAMPLES: NUMBER	TYPE	RECOVERY (%)	BLOWS /foot	INDEX PROPERTIES: MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	NOTES
		Sandy lean to fat CLAY (CL/CH) Medium, moist to wet, dark gray brown, trace fine gravel							1				26			
1									2				24			LL=51 PI=29
									3				18	111		
		Poorly graded SAND (SP) Medium dense, moist, light brown														
2		BOTTOM OF BORING AT 1-3/4 FEET														
3																

URS

PROJECT NO. 28649762

Figure: B-5



Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
B1	3	2	●	30	44	20	24	Sandy lean CLAY (CL) with gravel
B2	1	1	⊠	29	46	20	26	Sandy lean CLAY (CL)
B3	2	1	▲	24	51	22	29	Sandy lean to fat CLAY (CL/CH)

Project: SCCC BALLROOM
Project Number: 28649762

PLASTICITY CHART

Figure B-6

3/14/07 JATT94 97625

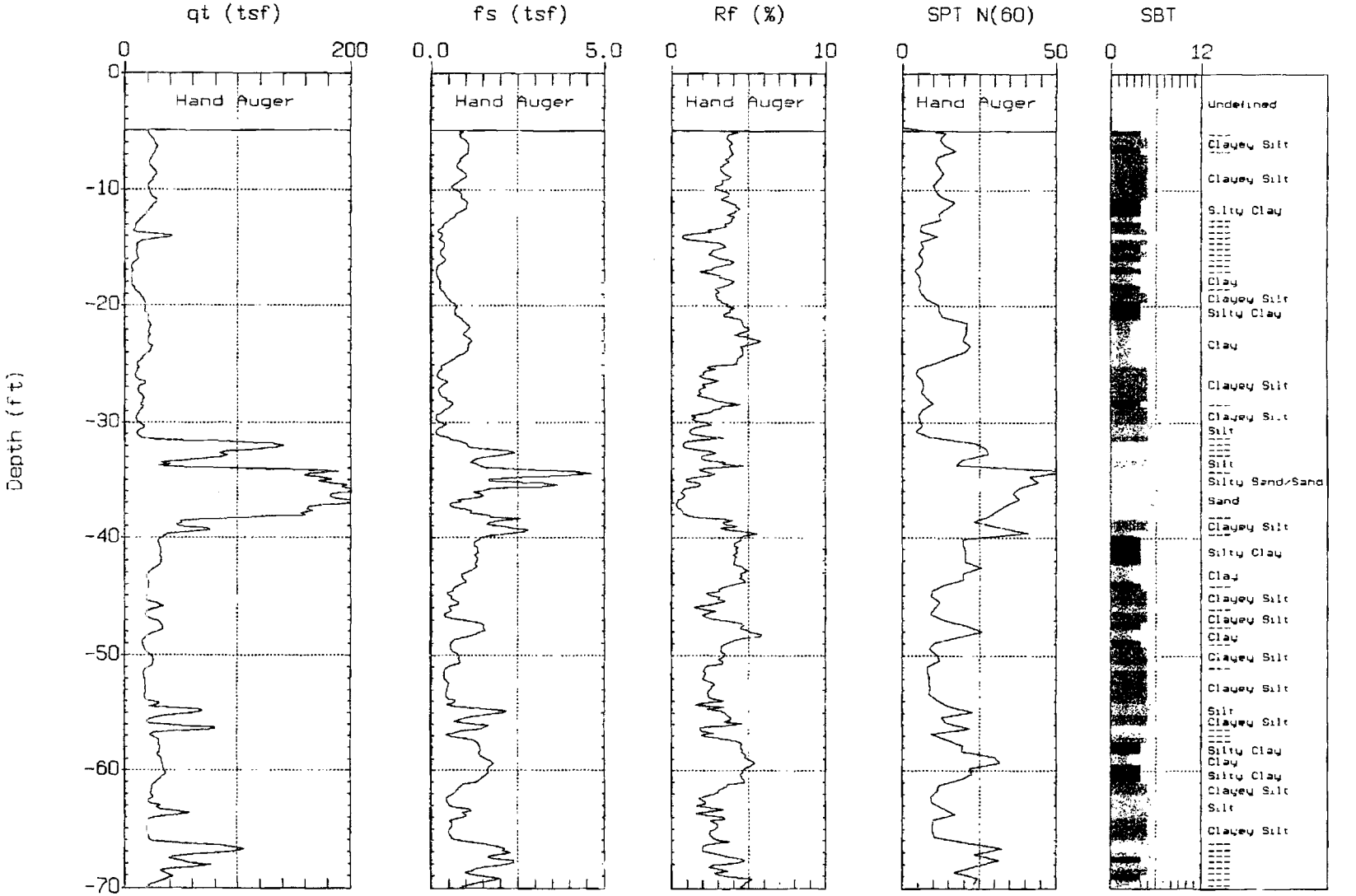
URS

GREGG

URS

Site: SANTA CLARA BALLROOM
Location: CPT-01

Engineer: R.KRESS
Date: 08:17:06 08:43



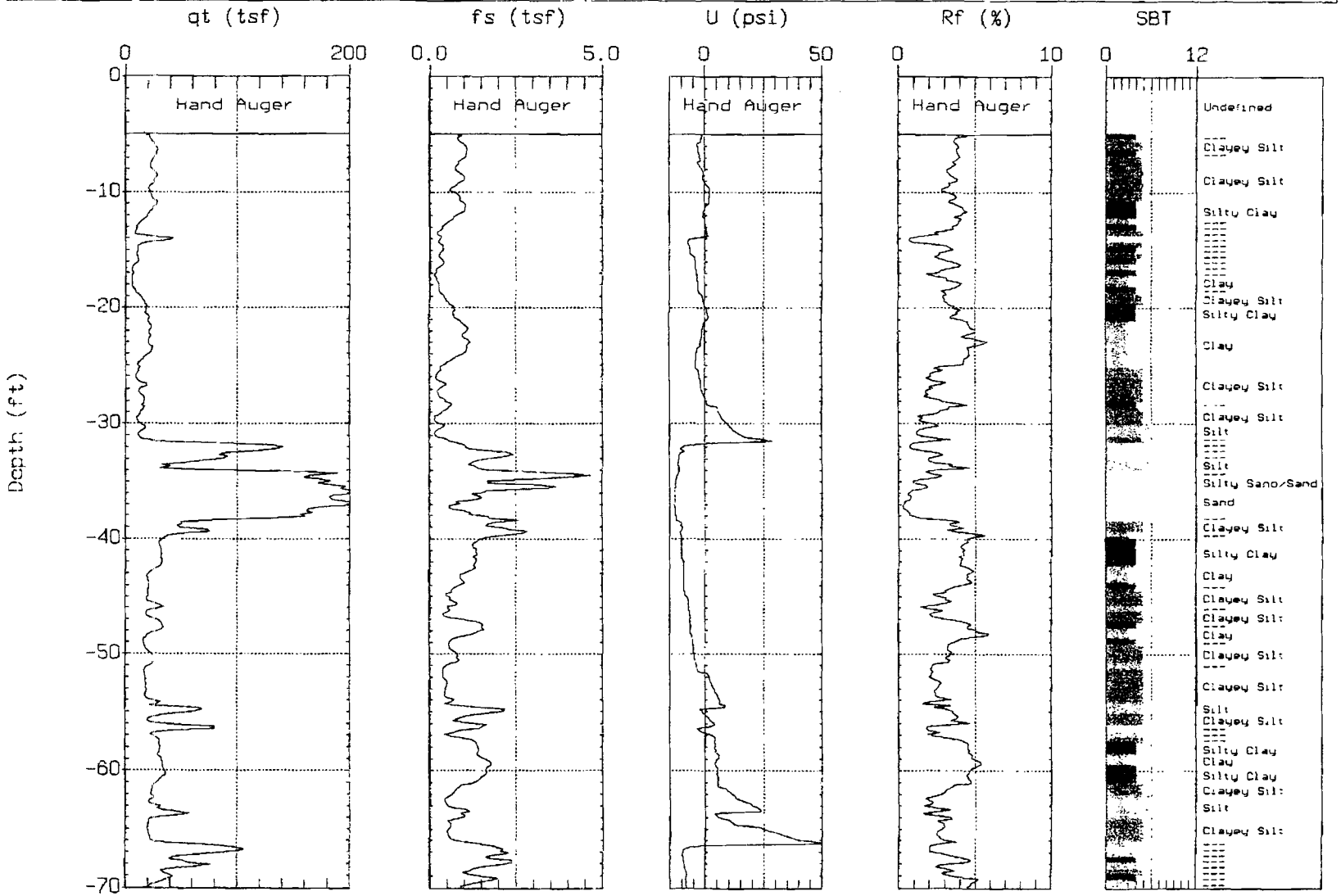
Max. Depth: 70.21 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

URS

Site: SANTA CLARA BALLROOM
Location: CPT-01

Engineer: R.KRESS
Date: 08:17:06 08:43



Max. Depth: 70.21 (ft)
Depth Inc.: 0.164 (ft)

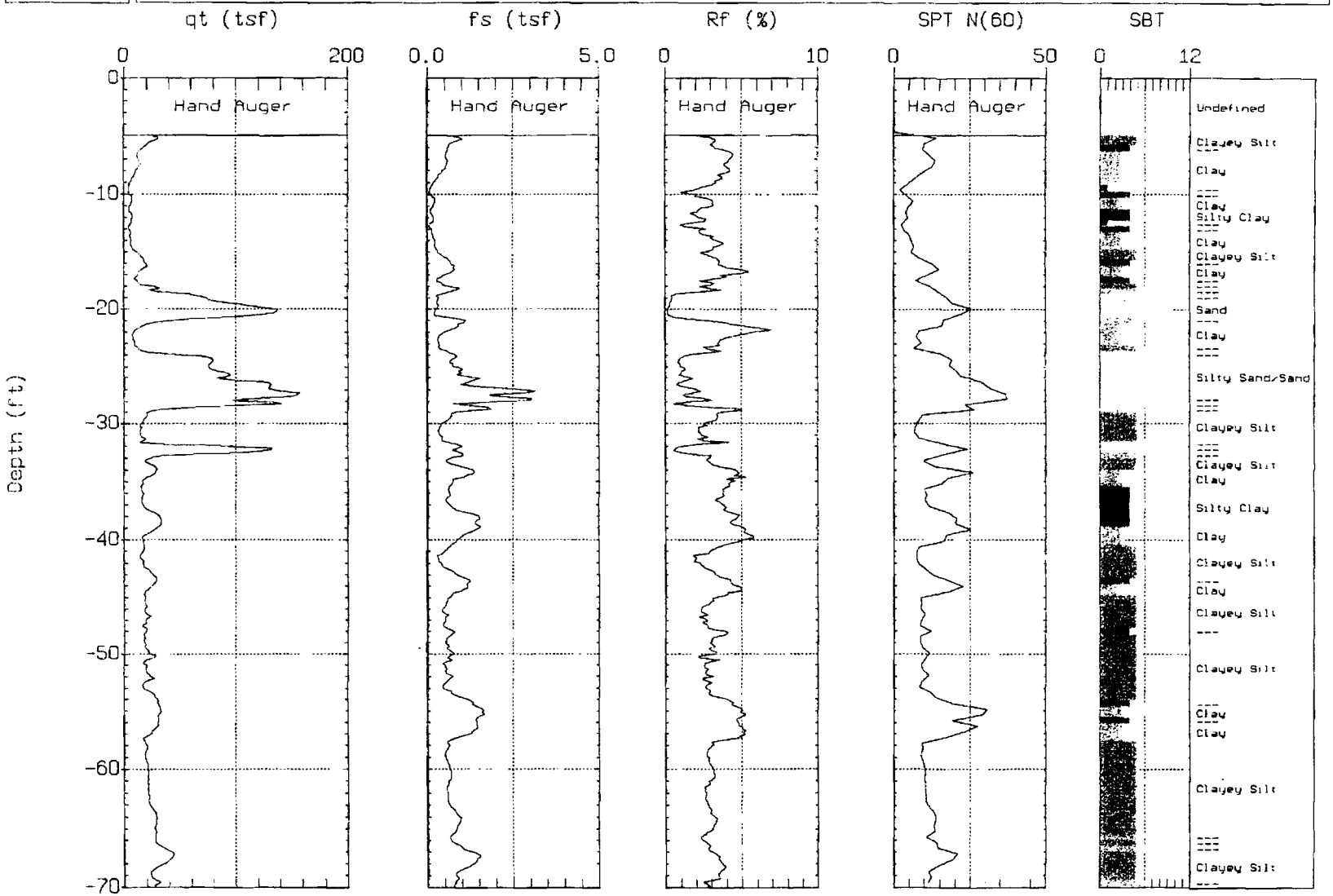
SBT: Soil Behavior Type (Robertson 1990)

GREGG

URS

Site: SANTA CLARA BALLROOM
Location: CPT-02

Engineer: R.KRESS
Date: 08:17:06 11:03



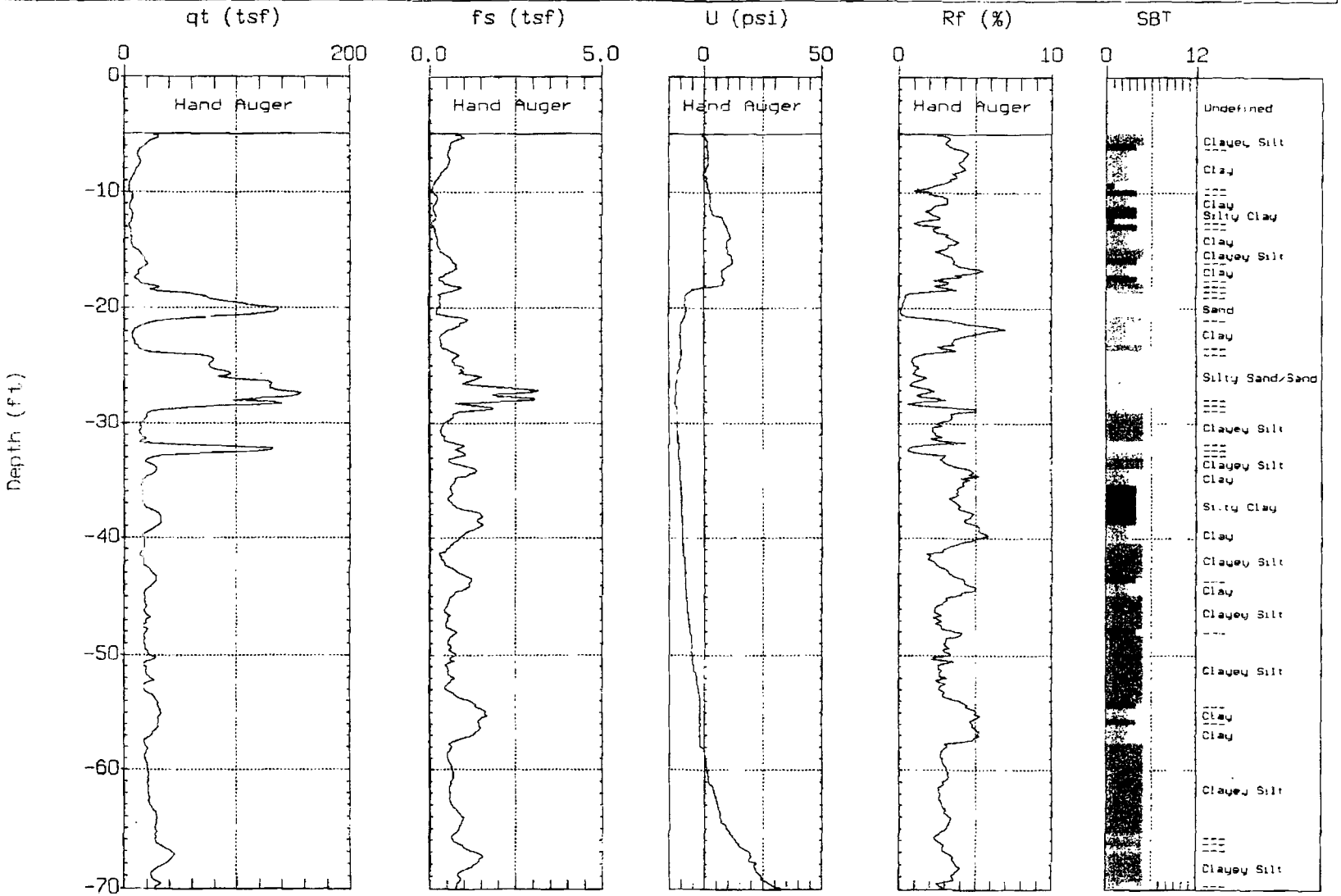
Max. Depth: 70.21 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

URS

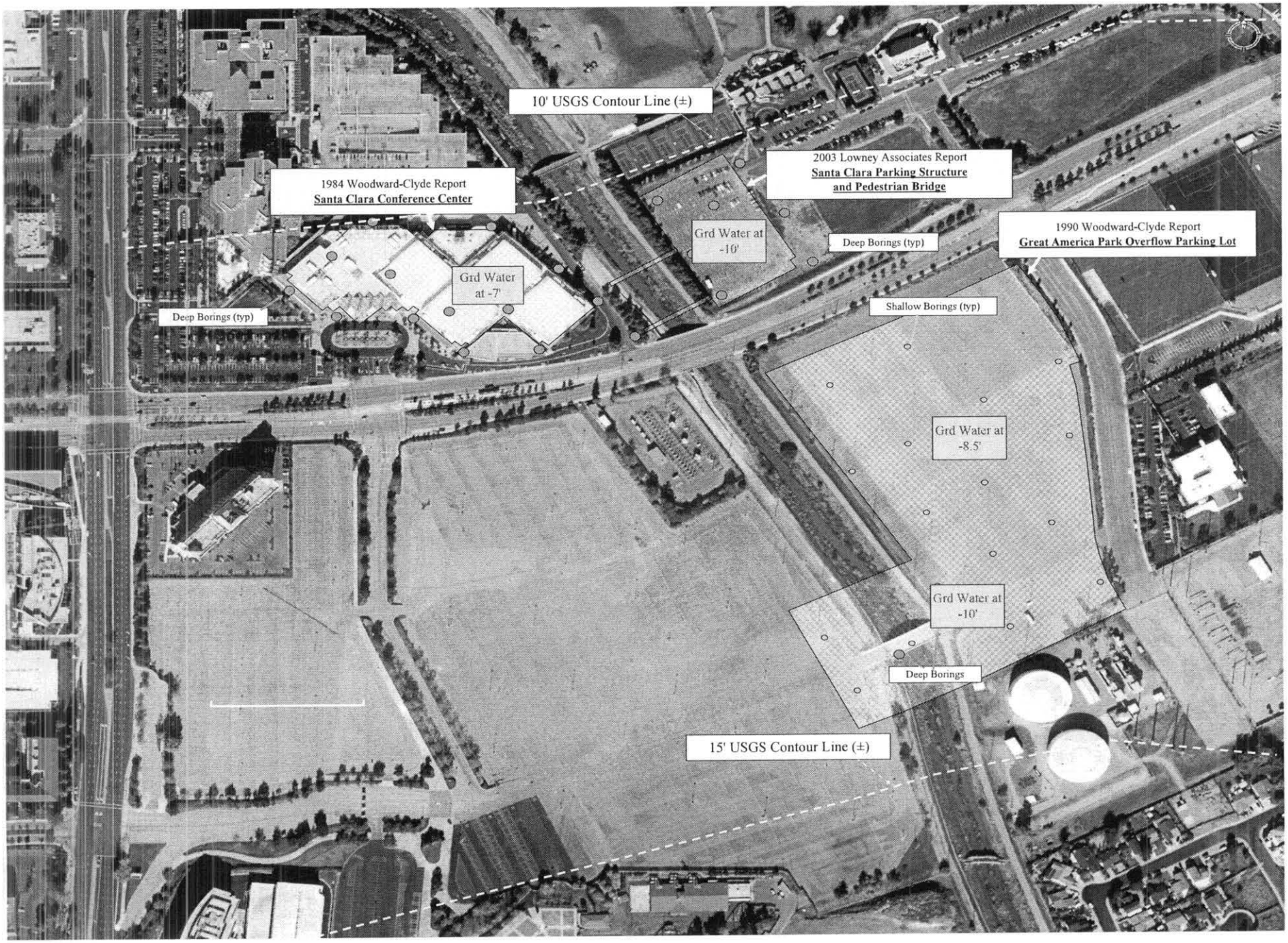
Site: SANTA CLARA BALLROOM
Location: CPT-02

Engineer: R.KRESS
Date: 08:17:06 11:03



Max. Depth: 70.21 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)


10' USGS Contour Line (±)
2003 Lowney Associates Report
Santa Clara Parking Structure
and Pedestrian Bridge
1984 Woodward-Clyde Report
Santa Clara Conference Center
Grd Water at -10'
Deep Borings (typ)
1990 Woodward-Clyde Report
Great America Park Overflow Parking Lot
Grd Water at -7'
Shallow Borings (typ)
Deep Borings (typ)
Grd Water at -8.5'
Grd Water at -10'
Deep Borings
15' USGS Contour Line (±)