Treadwell&Rollo

7 August 2001
Project No. 3195.01

Mr. Mark Darley
SyWest Development
150 Pelican Way
San Rafael, California 94901

Subject: Preliminary Geotechnical Investigation
557 East Bayshore Boulevard
Redwood City, California

Dear Mr. Darley:

This letter presents the results of our preliminary geotechnical investigation for the property at 557 East Bayshore Boulevard in Redwood City, California. We understand our investigation will be used for preliminary project planning. Our work has been performed in accordance with our *Purchase Order and Standard Conditions Agreement* dated 11 July 2001, which incorporates our proposal dated 11 July 2001.

The subject property is on the north side of East Bayshore Boulevard between Whipple Avenue and Bair Island Road, as shown on the attached Site Location Map, Figure 1. The site is bordered by East Bayshore Boulevard on the south, the Boardwalk used car dealership and Alan Steel to the west, a levee and drainage ditch to the north, and Bair Island Mini Storage to the east. An asphalt-paved parking lot and the Century Park 12 Movie Theater currently occupy the site. The theater lies in the northwest corner of the property, at the approximate location shown on the Site Plan, Figure 2. The existing grading plans for the property entitled *12 Plex Motion Picture Theater Site, Proposed Grading Plan,* prepared by Brian Kangas Faulk, dated 6 March 1989, indicates the site is relatively flat. The finish floor for the theater is at Elevation 108 feet[1], and surface elevations in the parking lot vary between 104 and 107.5 feet. The top of the existing levee is at Elevation 110 feet, with the flow line of the ditch at Elevation 100 feet.

The proposed development will consist of construction of several new commercial and/or residential structures. At the time we prepared this report, the size, type, number, and location of the buildings had not been determined. In addition, it had not been determined if site grades will be raised to accommodate the proposed development.

SCOPE OF SERVICES

The purpose of our investigation was to evaluate subsurface conditions at the site and evaluate foundation alternatives for use in project planning. We evaluated the subsurface conditions by

[1] All elevations are based on the existing grading plans, and are relative to National Geodetic Vertical Datum of 1929 (NGVD) plus 100 feet.

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advancing six cone penetration tests at the site. Based on the results of our field investigation and engineering analyses, we developed preliminary conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- appropriate foundations for the new structures
- preliminary design criteria and estimated settlement for the recommended foundation types
- areal settlement due to placement of new fill
- site grading and excavation, including criteria for fill quality and compaction
- 1997 Uniform Building Code site soil profile type and near-source factors
- construction considerations

Field investigation

We investigated subsurface conditions by performing six cone penetration tests (CPTs), designated CPT-1 through CPT-6, at the approximate locations shown on Figure 2. The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone tip measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measure soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by a computer to provide engineering information such as the types of soil encountered and approximate strength characteristics of the soil. The CPTs were generally advanced to depths of 99 to 100 feet below the existing ground surface, except CPT-1 and CPT-2, which met refusal at depths of 76.5 and 73.0 feet below the existing ground surface (bgs), respectively. The logs of CPT-1 through CPT-3, CPT-5, and CPT-6, showing tip resistance and friction ratio by depth, as well as interpreted SPT N-values, soil shear strength parameters, and an interpreted soil classification, are presented on Figures A-1 through A-5. Due to technical problems by our subcontractor, the log of CPT-4 could not be retrieved from the computer, although we understand the soil conditions encountered are similar to the remaining CPTs. The classification chart for the CPT logs is presented on Figure A-6. Upon completion, the CPT holes were backfilled with neat cement grout in accordance with County of San Mateo, Environmental Health Division.

SUBSURFACE CONDITIONS

Based on the results of the CPTs, we conclude 7 to 11 feet of clayey fill with thin sand and gravel lenses blanket the site. The fill generally appears thinnest at the front of the site (near East Bayshore Boulevard) and thickest at the rear of the site. The fill is underlain by marine clay known locally as Bay Mud. The Bay Mud encountered in our CPTs is quite variable and ranges from medium stiff to stiff. The strength and compressibility characteristics are also highly variable. Because of the variability of the Bay Mud, we believe the property is located in an area that was a historic marsh prior to development. Therefore, the Bay Mud is expected to be the softest and most compressible where historic sloughs and tidal pools previously existed, and stiffer and less compressible where the material was exposed above water and allowed to desiccate. A summary of interpreted Bay Mud conditions for each CPT are summarized in Table 1.

TABLE 1
Summary of Bay Mud Properties at CPT locations

CPT	Depth to Top of Bay Mud (feet bgs)	Depth to Bottom of Bay Mud (feet bgs)	Total Thickness of Bay Mud (feet)	Thickness of Medium Stiff (Compressible) Bay Mud (feet)
1	7	11.5	4.5	2.5
2	10.5	15	4.5	0
3	7	16.5	9.5	2.5
5	11	18.5	7.5	6
6	9.5	18.5	9.0	3.5

The Bay Mud is underlain by stiff to very stiff silt and clay with interbedded medium dense to dense sand and silty sand layers. The sand layers vary in depth and thickness, and therefore do not appear to be continuous across the site. The sand lenses may be the remnants of buried, braided stream channel deposits. CPT-1 and CPT-2 met refusal in dense sand at depths of 76.5 and 73 feet bgs, respectively. CPT-3 met refusal in dense sand at a depth of 99 feet bgs. CPT-5 and CPT-6 were both advanced to the design depth of 100 feet bgs, the maximum depth explored.

Groundwater was encountered between depths of 6.6 and 11.2 feet bgs at the time we performed the CPTs. These depths correspond to groundwater elevations ranging from about 96 to 98 feet. Because the CPTs were grouted upon completion, these depths may not represent the stabilized groundwater level. In addition, we anticipate the groundwater level will vary a few feet annually, depending on seasonal conditions.

SEISMICITY AND SEISMIC HAZARDS

Regional Seismicity

The closest active faults to the site are the Monte Vista, San Andreas, San Gregorio, and Hayward Faults, which are located 6.7, 7.2, 21, and 23 kilometers from the site, respectively. Strong to very strong ground shaking from a moderate to large earthquake on one of these faults or one of the other major faults in the Bay Area is likely to be felt at the subject site over the next few decades. Figure 3 shows the earthquake epicenters for events occurring in the Bay Area with magnitude greater than 5.0 from January 1800 through January 1996. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, with a Moment magnitude[2] (M_w) of 6.9. The epicenter of the earthquake was located in the Santa Cruz Mountains approximately 60 km from the site.

Seismic Hazards

The seismicity of the site is governed by the activity of the San Andreas Fault. However, ground shaking from future earthquakes on any of the nearby faults will be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, magnitude and duration of the earthquake, and specific subsurface conditions. We judge ground shaking at the site during a large earthquake on one of the faults discussed above will be strong to very strong.

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2 Moment magnitude is directly related to average slip and rupture fault area, while the Richter magnitude scale reflects the amplitude of a particular type of seismic wave. Moment magnitude provides a physically meaningful measure of the size of a faulting event.

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Strong shaking during an earthquake can result in ground failure such as that associated with liquefaction[3], lateral spreading[4], and cyclic densification[5]. We used the results of the CPTs to evaluate the potential of these phenomena occurring at the project site. The results of our evaluation are presented below.

Soil Liquefaction and Associated Hazards

When saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength due to a transient rise in excess pore pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

Data obtained from our CPT indicate the silt and clay alluvium beneath the site is interbedded with layers of medium dense to dense sand and silty sand; most of the medium dense layers were encountered between depths 30 and 40 feet bgs. The results of our engineering analyses indicate excess pore pressures will develop in the medium dense layers during a large earthquake on a nearby fault. During a Magnitude 7-1/2 (or greater) earthquake generating a ground surface acceleration of 0.49g, our analyses also indicate these layers may liquefy. This acceleration is consistent with that recommended by the 1997 Uniform Building Code (UBC). Table 2 presents our estimates for liquefaction-induced settlement at the site.

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3 Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand and silt of low plasticity that is relatively free of clay.

4 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

5 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground surface settlement.

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TABLE 2
Estimated Liquefaction-Induced Settlement

CPT	Depth of Thickest Liquefiable Zone (feet bgs)	Estimated Settlement (inches)
1	31 to 34.5	1/2
2	30 to 40	1-1/2
3	20 to 28	1-3/4
5	32 to 36	3/4
6	32 to 34.5	3/4

We estimate differential settlement at the ground surface due to liquefaction will be on the order of 1/2 inch or less in 30 feet in the vicinity of all the CPTs except CPT-2 and CPT-3. In the vicinity of CPT-2 and CPT-3, we estimate differential settlement will be on the order of 3/4 to 1 inch in 30 feet. Because the potentially liquefiable soil is overlain by 20 to 30 feet of medium stiff to stiff clay, we conclude the potential for sand boils, lateral spreading, and lurch cracking is low.

5.2.3 Cyclic Densification

Seismically induced compaction or densification of non-saturated sand (sand above the groundwater table) due to earthquake vibrations may cause differential settlement. However, layers of clean sand fill were not generally encountered above the water table. Therefore, we judge that ground-surface settlement at the site due to cyclic densification during a major earthquake will be negligible.

5.2.4 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

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CONCLUSIONS AND RECOMMENDATIONS

On the basis of the results of our field investigation and engineering analyses, we conclude there are no adverse soil conditions on the site that would preclude constructing residential or commercial structures at the site. The primary geotechnical factors to be considered for design include:

- the potential for variable building settlement due to the variable thickness and properties of Bay Mud at the site
- the potential for areal settlement if site grades are raised
- the potential for liquefaction and liquefaction-induced settlement.

Our conclusions regarding these and other issues for the proposed building are presented below.

Settlement and Foundations

The principal factor influencing the choice of a foundation system for the proposed project is the presence of Bay Mud near the ground surface, which will settle under new fill or foundation loads. Additional ground-surface settlement is expected to occur due to liquefaction following a large earthquake on a nearby fault. We used the results of the CPTs to preliminarily estimate settlement under typical building and foundation loads for the types of structures to be considered. The results of our analyses, for three types of soil conditions encountered at the site (CPT-1, -2, and -5), are presented in Table 3. We estimate about half the total settlement presented in Table 3 will occur differentially over a distance of about 30 feet. The magnitude of additional liquefaction-induced settlement expected to occur at the site following a large earthquake on a nearby fault was previously presented in Table 2.

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TABLE 3
Estimated Settlement Due to Assumed Structural Loads

Type of Load	Size (feet)	Applied Pressure (psf)	Estimated Settlement (inches)		
			CPT-1	CPT-2	CPT-5
200-kip column	8 x 8	3,000	1	1/2	2
400-kip column	10 x 10	4,000	2	3/4	3-1/2
Mat foundation	150 x 200	1,200	1-1/2	1/2	4
New Fill	1-foot-thick	130 (areal)	<1/4	<1/4	3/4
New Fill	3-foot-thick	390 (areal)	1/4	1/4	1-1/2

psf = pounds per square foot

As Table 3 indicates, the soil conditions vary considerably across the site and there are a wide range of possible settlement estimates depending on the soil conditions and anticipated structural loads. Therefore, the choice of the most appropriate foundation system will depend on the building type and height, structural loads, location on the site, and allowable differential settlement. In general, we believe it will be feasible to support relatively lightweight, wood-framed residential structures on a stiffened grid foundation or rammed aggregate piers. However, where soft soil conditions are encountered, such as in CPT-5, shallow foundations are likely not feasible. Mid-rise commercial structures (steel- or concrete-framed) can be supported on rammed aggregate piers or driven piles. Preliminary design parameters for these foundations systems are presented in the following subsections.

Stiffened Grid Foundation

A stiffened grid foundation consists of a continuous, well-reinforced perimeter footing with interior footings tied together by well-reinforced grade beams. For this foundation type, differential settlement is reduced by the ability of the stiffened foundations to transfer loads over "soft spots" in the subsoil. Some settlement-related damage should be expected with this foundation system; however, most of the settlement should occur within one year of construction and we believe the damage will generally be cosmetic in nature and should be readily repairable.

To develop adequate stiffness in the foundation system and resist differential settlement, interior footings should be tied together with well-reinforced grade beams. The grade beams connecting interior footings and continuous perimeter footings should be sized and reinforced so they span up to 10 feet as a simple beam or three feet as a cantilever (for corners), limiting maximum deflection to 1/360th of the span. In the vicinity of CPT-5, the footings should be reinforced to span up to 15 feet as a simple beam or five feet as a cantilever (at corners). This is an empirical design criterion that has been successfully used on other projects to achieve stiff foundation systems for light buildings.

Footings should be at least 18 inches wide, and embedded at least 18 inches below the lowest adjacent soil subgrade. For interior footings, the soil subgrade is defined as the bottom of the capillary moisture break. For exterior footings, the soil subgrade is defined as the exterior soil grade or top of pavement, whichever is lower. However, the footings should not be embedded more than three feet below the lowest adjacent soil subgrade. If deeper embedment is required, we should be consulted to provide revised settlement estimates. We preliminarily recommend the footings be designed for an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus live load conditions. For total load conditions, including wind or seismic, we preliminarily recommend an allowable bearing pressure of 4,000 psf be used. These values are controlled by settlement rather than bearing capacity.

Lateral loads can be resisted by a combination of passive pressures on the vertical faces of the footings and friction along the bases of the footings. Passive resistance may be preliminarily computed using an uniform pressure (rectangular distribution) of 1,000 psf; the upper foot of soil should be ignored unless confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30. These values include a factor of safety of at least 1.5.

Rammed Aggregate Piers

Rammed aggregate piers are typically constructed by drilling an approximately 30-inch-diameter shaft and replacing the excavated soil with compacted aggregate fill. The aggregate generally consists of clean, open-graded crushed rock below the water table and Class 2 aggregate base above the water table. The aggregate is compacted in approximately 12-inch-thick lifts using a modified hydraulic hammer mounted on an excavator. Rammed aggregate piers develop vertical foundation support through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix, allowing use of significantly larger bearing capacities than feasible with the unimproved soil. Therefore, we believe the buildings could be supported on conventional spread footings if rammed aggregate piers are used. Rammed aggregate piers can also be designed to resist uplift loads by installing steel rods in the center of the pier; the rod is attached to a flat plate at the base of the pier.

Rammed aggregate piers are typically constructed through a design-build contract. Locally, rammed aggregate piers typically consist of a proprietary system called Geopiers, installed by the Geopier Foundation Company of Northern California (GFCNCA). Because the piers are designed and installed under a design-build contract, we have not analyzed this foundation alternative and cannot provide recommendations regarding spacing, costs, capacity, or settlement. However, based on our previous experience with this foundation system, we anticipate allowable bearing pressures of up to 6,000 psf may be feasible with less than one inch of settlement. The design capacity should be confirmed by a load test program. Lateral loads on the footings can be designed in accordance with our previous recommendations for stiffened grid foundations.

Where installed in the Bay Area, Geopiers are typically on the order of 12 to 18 feet long; the depth of the piers is generally limited the length of the hydraulic tamper. Because the Geopiers would need to extend a few feet below the soft soil to be improved (Bay Mud), we estimate the Geopiers will be installed to depths of up to 21 feet near the rear of the site (in the vicinity of CPT-5 and CPT-6). Therefore, it may be necessary to pre-excavate and/or overexcavate the footing locations if Geopiers or another rammed aggregate pier system is used.

Driven Concrete Piles

A third alternative consists of support the proposed buildings on deep foundations consisting of driven piles gaining support below the compressible Bay Mud. In the Bay Area, the most common type of pile for this type of application is 14-inch-square, precast, prestressed concrete piles. We preliminarily recommend the piles be designed to gain support through skin friction below the Bay Mud. It may be possible to gain support through end bearing in the dense sand encountered at a depth of about 75 feet in CPT-1 and CPT-2; however, further investigation is required to determine the thickness and lateral extent of this layer. Because we anticipate the Bay Mud will have a high sulfate content, Type II (sulfate resistant) concrete should be used as a minimum.

For preliminary planning purposes, we estimate the allowable capacity of 90-foot-long, 14-inch-square, precast, prestressed concrete piles will be 250 kips (125 tons) for dead plus live load conditions. For total load conditions, this value can be increased by a factor of 1.33. These values include factors of safety of 2.0 and 1.5 for dead plus live load and total load conditions, respectively. We estimate settlement of a properly constructed driven pile foundation will be less than 3/4 inch, with less than 1/2 inch of differential settlement in 30 feet.

Lateral loads can be resisted by a combination of passive pressures acting on the face of pile caps and grade beams and the bending resistance of the piles. Friction on the base of the pile caps should be neglected. Passive resistance can be computed using our previous recommendations for stiffened grid foundations. Bending resistance can be computed during the final investigation.

The piles will gain support below the potentially liquefiable soil previously discussed. Therefore, the piles will be subject to temporary downdrag loads due to liquefaction-induced settlement. We estimate this downdrag load will be on the order of 60 kips. The structural engineer should check the structural section of the pile under the combined effect of both the design loads and downdrag load. Because it may take up to a day for the excess pore pressures to dissipate, we recommend the downdrag load be applied to dead plus live load conditions. We estimate the application of the downdrag load will temporarily reduce the factor of safety against soil failure to about 1.75 for dead plus live load conditions.

If new fill is added to raise site grades, consolidation settlement of the Bay Mud will also result in application of downdrag loads to the piles. These downdrag loads will act as long-term loads on the piles, and must be considered for long-term design. We preliminarily estimate the downdrag load due to new fill will be on the order of 25 kips. For preliminary planning purposes, we recommend the pile length presented above be increased by 10 feet to account for this downdrag load.

Slab-on-Grade Floors

If a stiffened grid foundation or rammed aggregate piers are used, we conclude a conventional concrete slab-on-grade can be used for all ground floor slabs. Some settlement and cracking of the floor slabs should be expected because of ground surface settlement due to consolidation of the underlying Bay Mud (if site grades are raised) or liquefaction. This cracking can be controlled to some degree by using additional reinforcement in the slabs. Where pile foundations are used, we recommend the floor slab be designed to span between the pile caps and grade beams (i.e., structurally supported) because the ground surface is expected to settle relative to the piles, particularly if site grades are raised.

To prevent moisture transmission through the slabs, they should be underlain by a moisture barrier. A moisture barrier typically consists of four inches of 1/2- to 3/4-inch, open-graded drain rock or gravel overlain by a vapor-proof membrane at least 10 mils thick. The membrane should be covered with two inches of sand prior to placement of concrete.

Fill Placement and Compaction

All areas to be graded should be cleared of vegetation and stripped of the upper 1 to 2 inches of surface soil containing organic matter in landscape areas. Deeper stripping will be required to remove brush and tree roots. Stripped materials should not be used in engineered fills; however, the material may be stockpiled for future use in landscaping if approved by the project architect. The existing asphalt pavement should also be removed prior to placement of fill. The asphalt can be reused as fill, provided it is acceptable from an environmental standpoint. Asphalt to be re-used should be crushed such to less than four inches in greatest dimension (with less than 15 percent greater than three inches and less than 50 percent greater than one inch) and mixed with on-site or imported soil to prevent nesting of the particles.

The surface exposed by stripping and/or excavation (if required) should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[6]. We anticipate the majority of on-site soil below the stripped layer can be used as fill, provided it is not expansive or otherwise detrimental to the proposed construction. During the final investigation, we should collect samples of the near-surface fill to check the expansion potential of the soil and determine the resistance value (R-value) for pavement section design.

All fill placed at the site, including the on-site soil, should be free of debris and contain no rocks or lumps larger than four inches in greatest dimension, with no more than 50 percent of the fill being larger than one inch. If imported fill is required, it should also be free of rock and debris and have a low expansion potential (defined by a liquid limit of less than 40 and a plasticity index lower than 15). All fill placed at the site should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.

Seismic Considerations

The San Andreas Fault, which is a Type A fault located about 7.2 kilometers from the site, will govern seismic design. The proposed buildings should be designed in accordance Zone 4 seismic standards as a minimum. For design in accordance with the 1997 Uniform Building Code (UBC), the site is classified as soil profile type S_F because of the presence of the potentially liquefiable soil layers. Soil profile type S_F requires a site-specific investigation be performed to determine the appropriate seismic design parameters. However, because the potentially liquefiable layers are moderately dense, we believe they will behave like medium stiff to stiff clay layers during a large earthquake. Therefore, we conclude a soil profile type of S_D, with near-source factors of 1.11 and 1.42 for N_a and N_v, respectively, can be used for preliminary design. The appropriateness of these parameters should be confirmed once the size and type of the buildings has been determined.

ADDITIONAL INVESTIGATION

Prior to final design, we should perform a final geotechnical investigation to evaluate the soil conditions at the proposed building locations. The final investigation should include several rotary-wash soil borings so that a more detailed evaluation of the consolidation properties of the Bay Mud and liquefaction potential of the medium dense sand layers can be performed. In

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6 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-91 laboratory compaction procedure.

addition, the seismic design parameters should be re-evaluated once the building types and heights have been determined.

We trust this letter presents the information you require at this time. If you have any questions, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

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Attachments: *Figures*
Figure 1 – Site Location Map
Figure 2 – Site Plan
Figures 3 – Map of Major Faults and Earthquake Epicenters in the San Francisco Bay Area
Appendix A – Logs of Cone Penetration Tests
Figure A-1 through A-5 – Logs of CPT-1 through CPT-3, CPT-5, and CPT-6
Figure A-6 – Classification Chart for Cone Penetration Tests

FIGURES

Base map: The Thomas Guide
San Mateo County
1999

No scale

557 EAST BAYSHORE BOULEVARD
Redwood City, California

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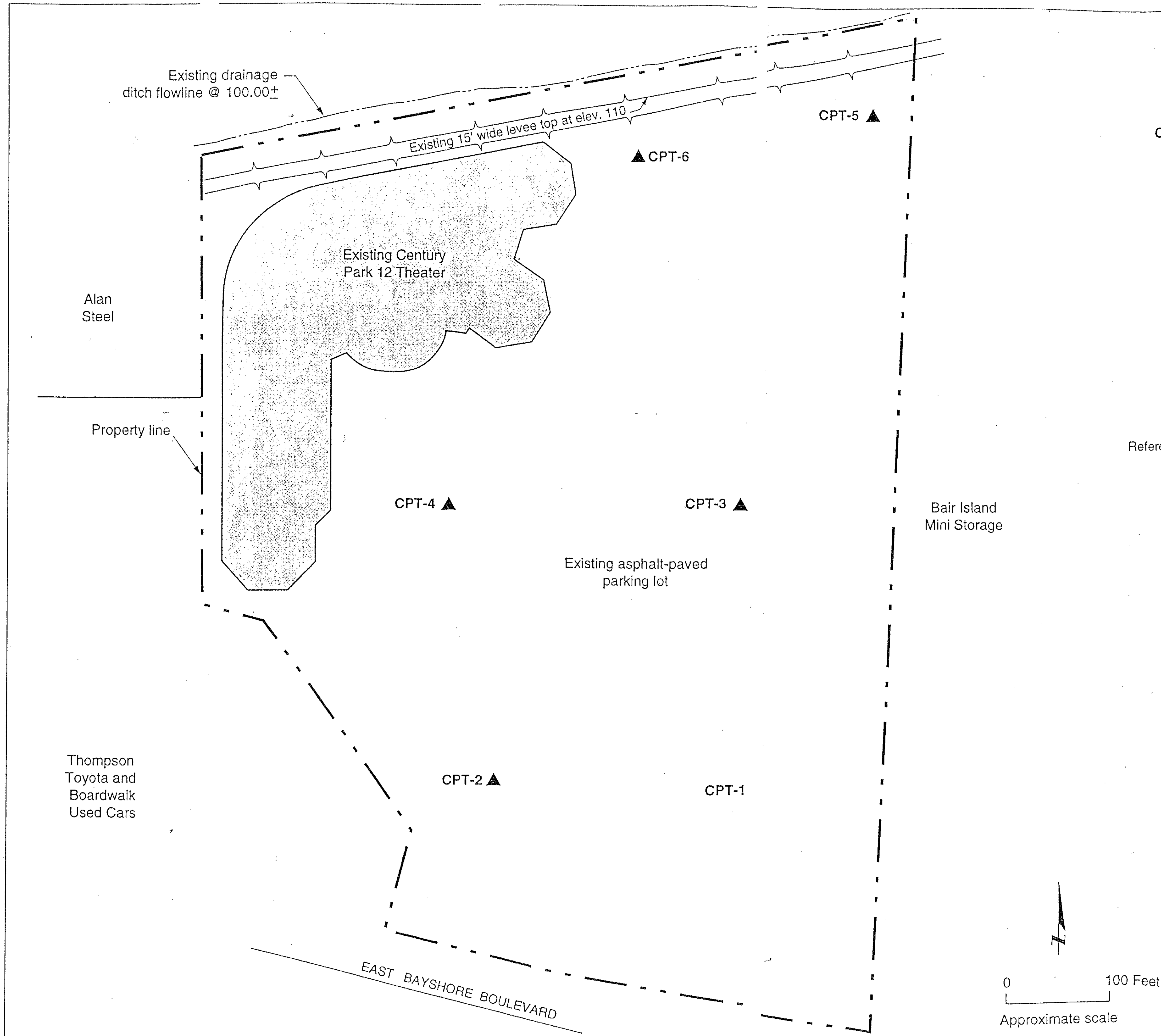
SITE LOCATION MAP

Date 07/31/01	Project No. 3190.01	Figure 1



EXPLANATION

CPT-1 ▲ Approximate location of cone penetration test performed by Treadwell & Rollo, Inc., July 2001

Reference: "12 Plex Motion Picture Theater Site, Proposed Grading Plan", by Brian Kangas Foulk Consulting Engineers, dated 6 March 1989.

557 BAYSHORE BOULEVARD Redwood City, California		
SITE PLAN		
Date 08/01/01	Project No. 3190.01	Figure 2
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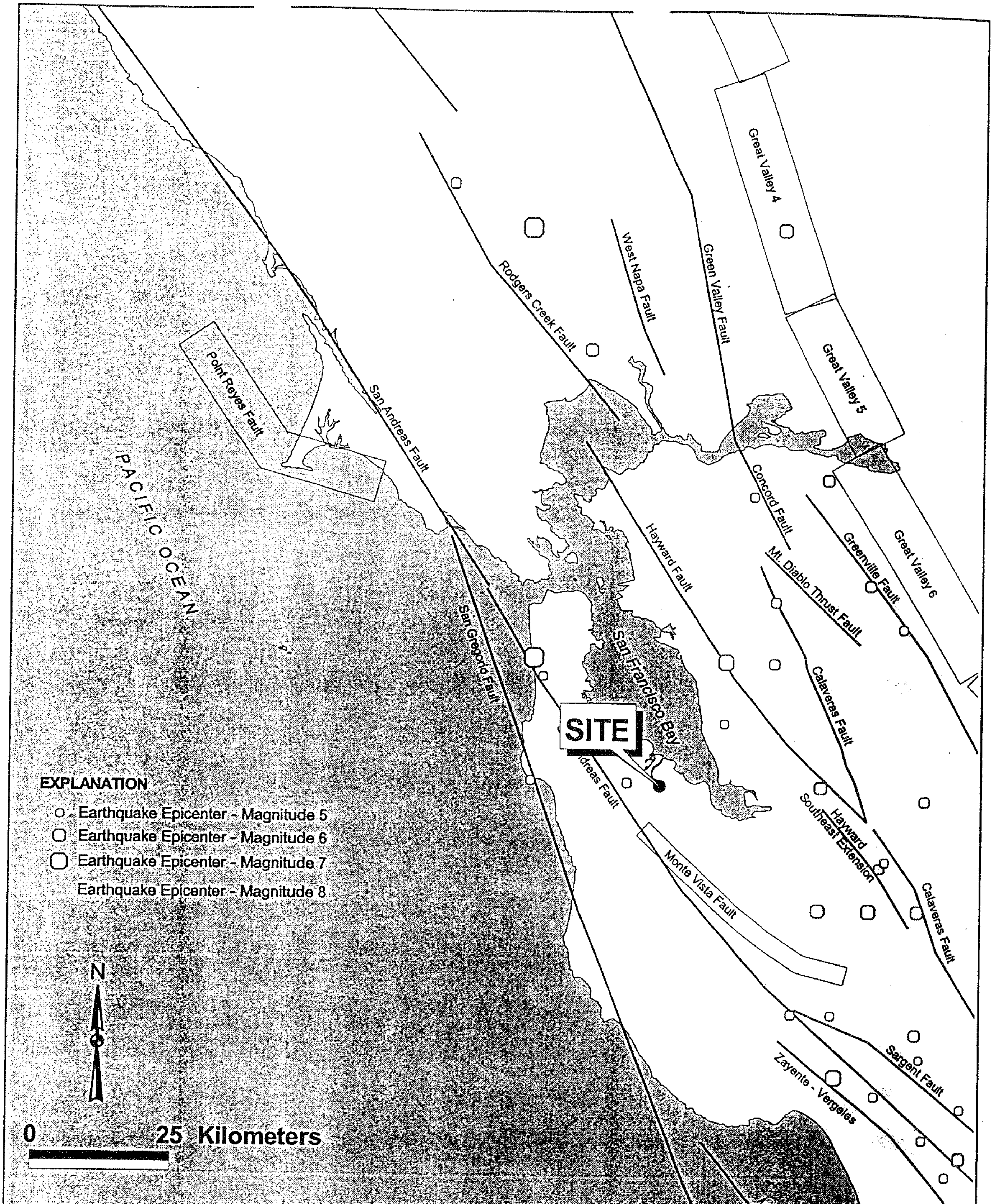
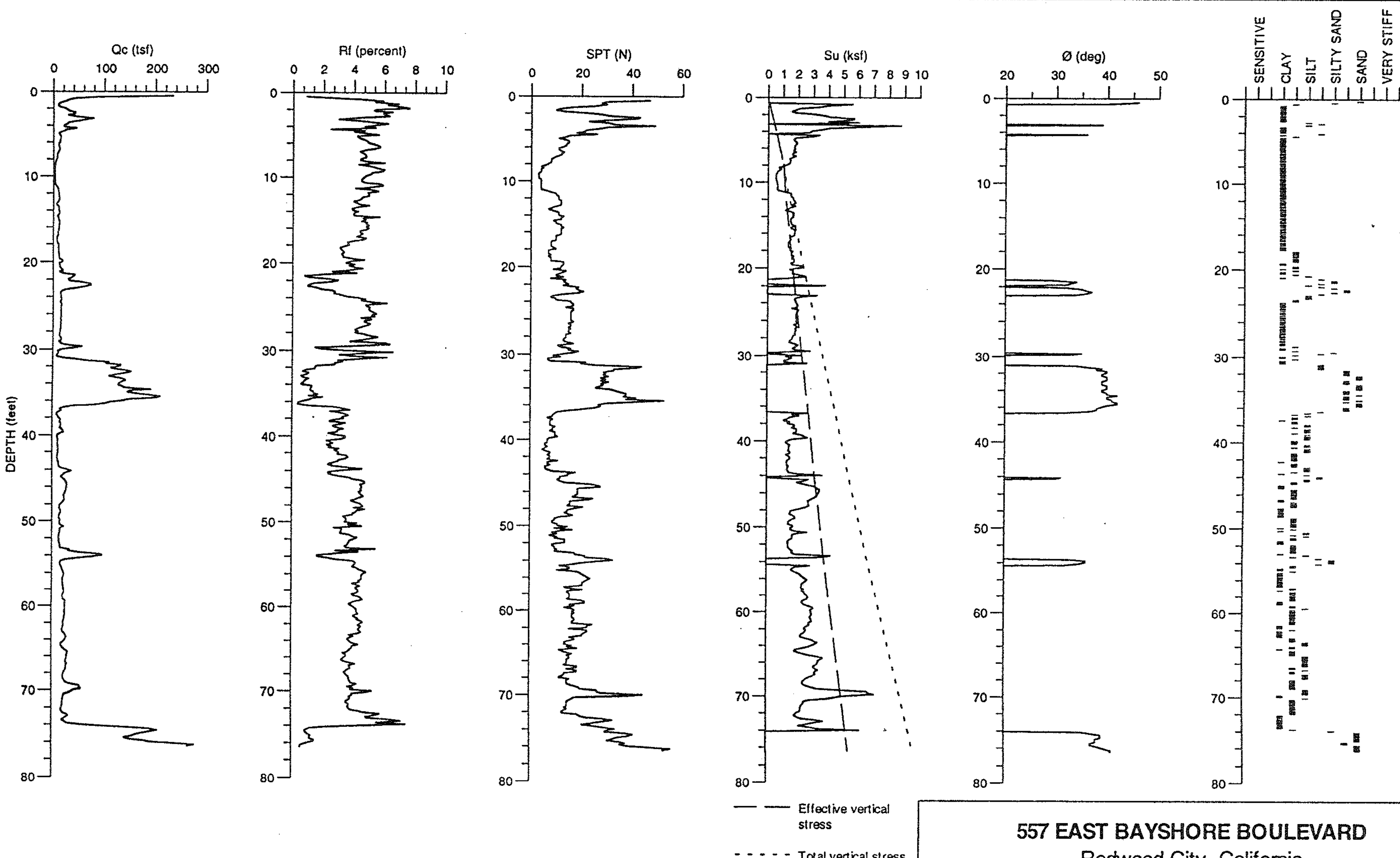


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to January 1996.

557 EAST BAYSHORE BOULEVARD Redwood City, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
Treadwell&Rollo	Date: 07/31/01 | Project No. 3190.01 | **Figure: 3**

APPENDIX A
Logs of Cone Penetration Tests



Terminated at 76.5 feet
Groundwater measured at 8.2 feet.
Date performed: 27 July 2001.
Elevation: 105.3 feet, datum: 1929 NGVD plus 100 feet.

557 EAST BAYSHORE BOULEVARD
Redwood City, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 07/31/01	Project No. 3190.01	Figure A-1

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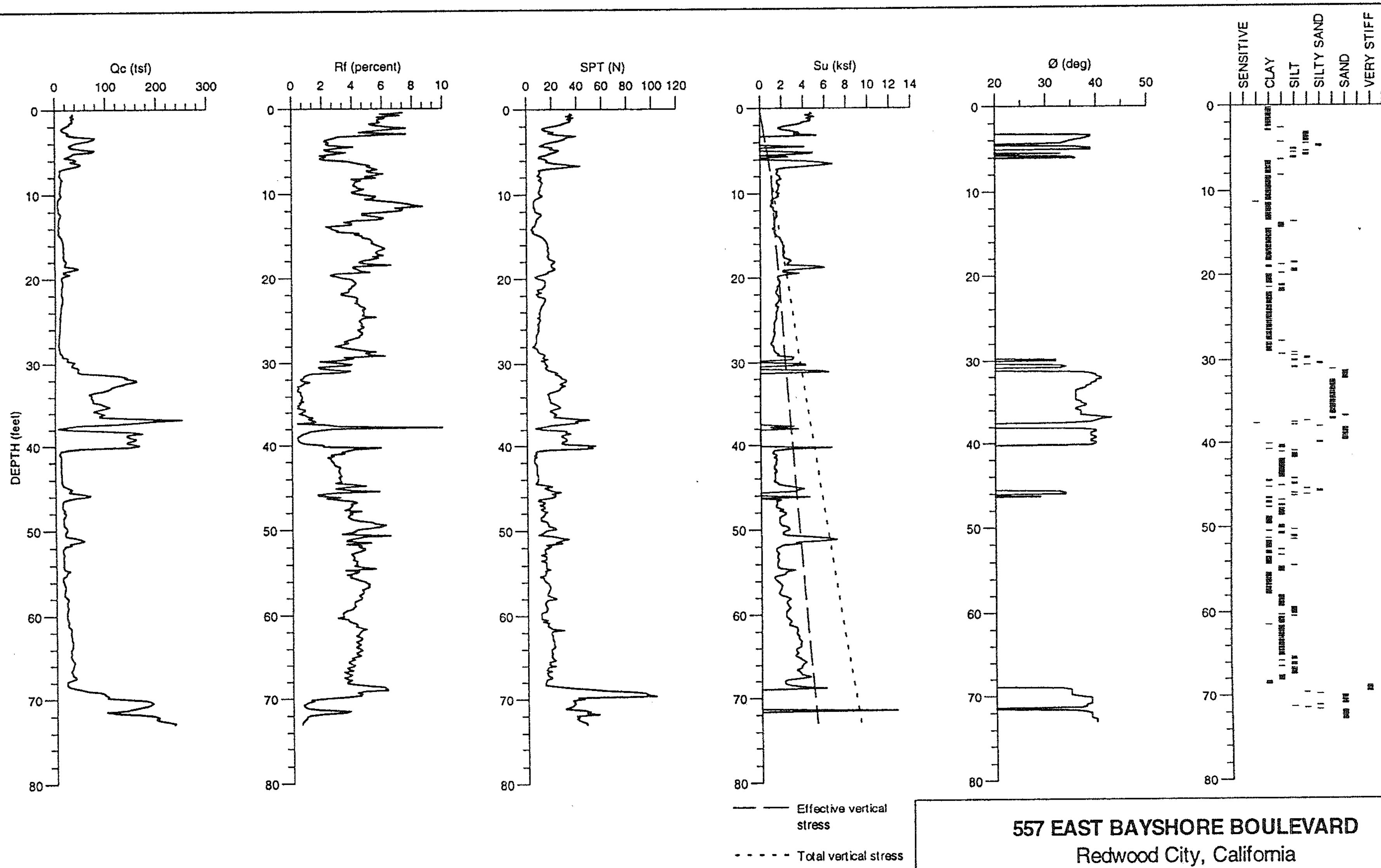

Terminated at 73.0 feet
Groundwater measured at 7.8 feet.
Date performed: 26 July 2001.
Elevation: 105.8 feet, datum: 1929 NGVD plus 100 feet.

557 EAST BAYSHORE BOULEVARD Redwood City, California		
CONE PENETRATION TEST RESULTS CPT-2		
Date 07/31/01	Project No. 3190.01	Figure A-2

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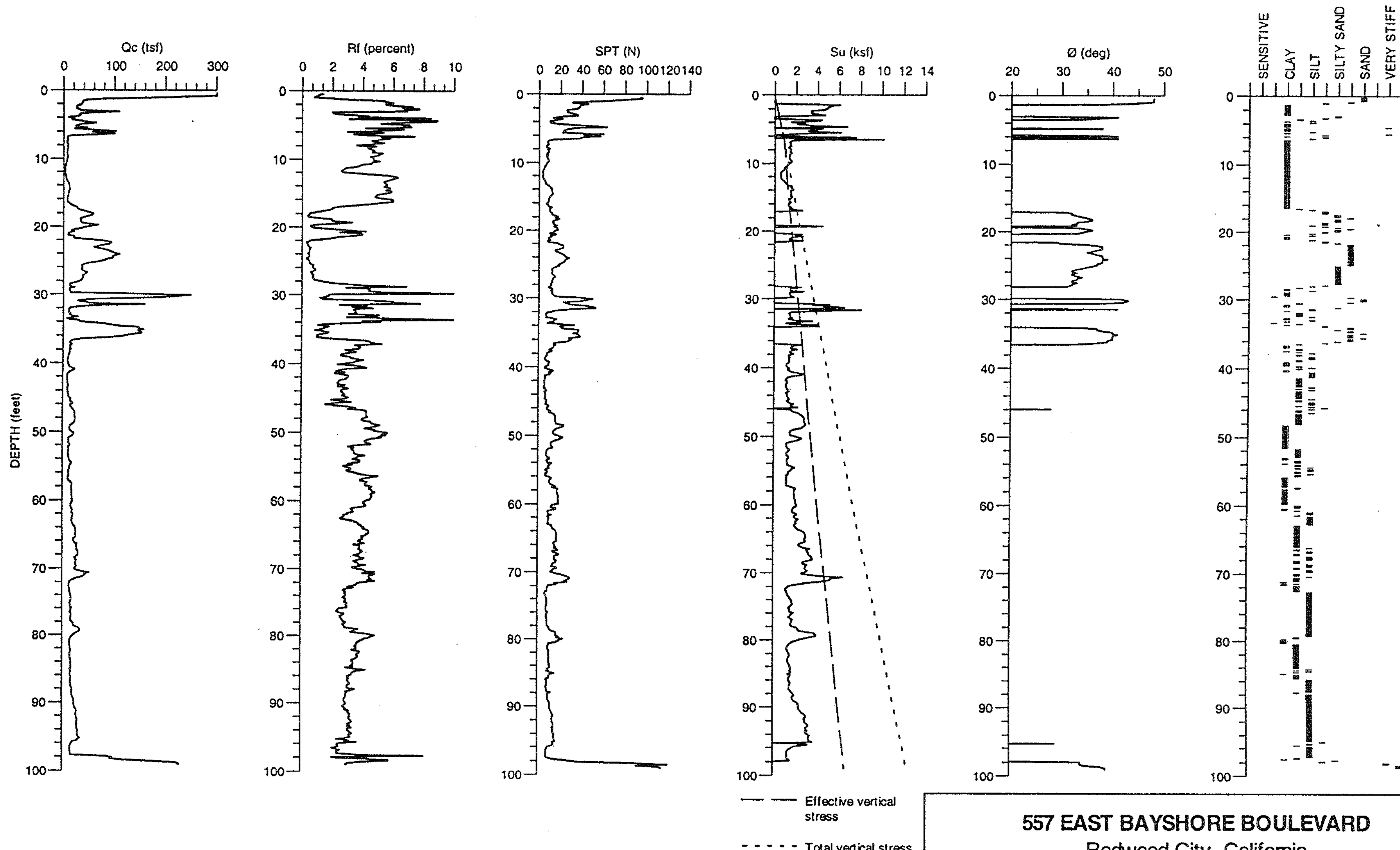

Terminated at 99.0 feet
Groundwater measured at 6.6 feet.
Date performed: 25 July 2001.
Elevation: 105.0 feet, datum: 1929 NGVD plus 100 feet.

557 EAST BAYSHORE BOULEVARD
Redwood City, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 07/31/01	Project No. 3190.01	Figure A-3

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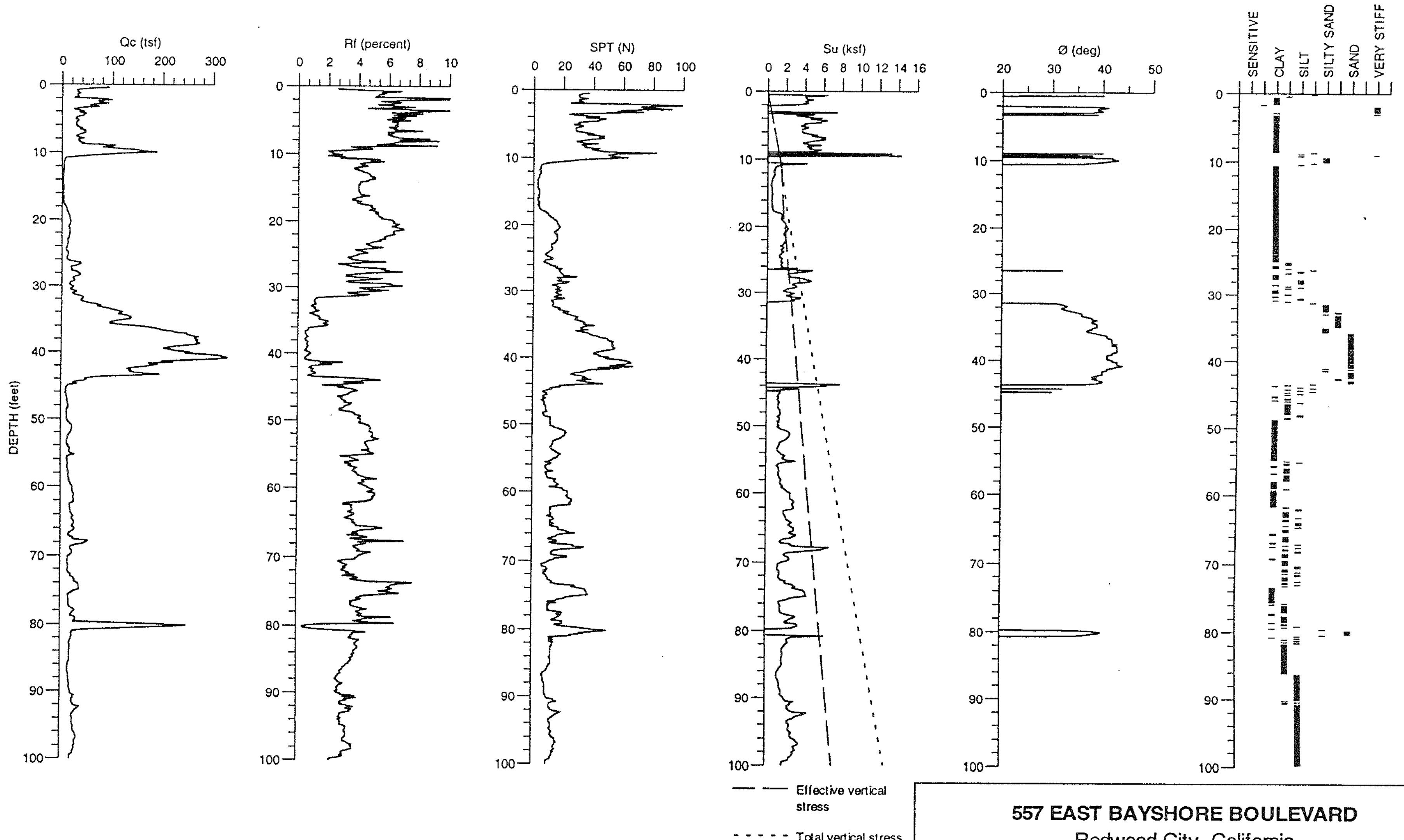
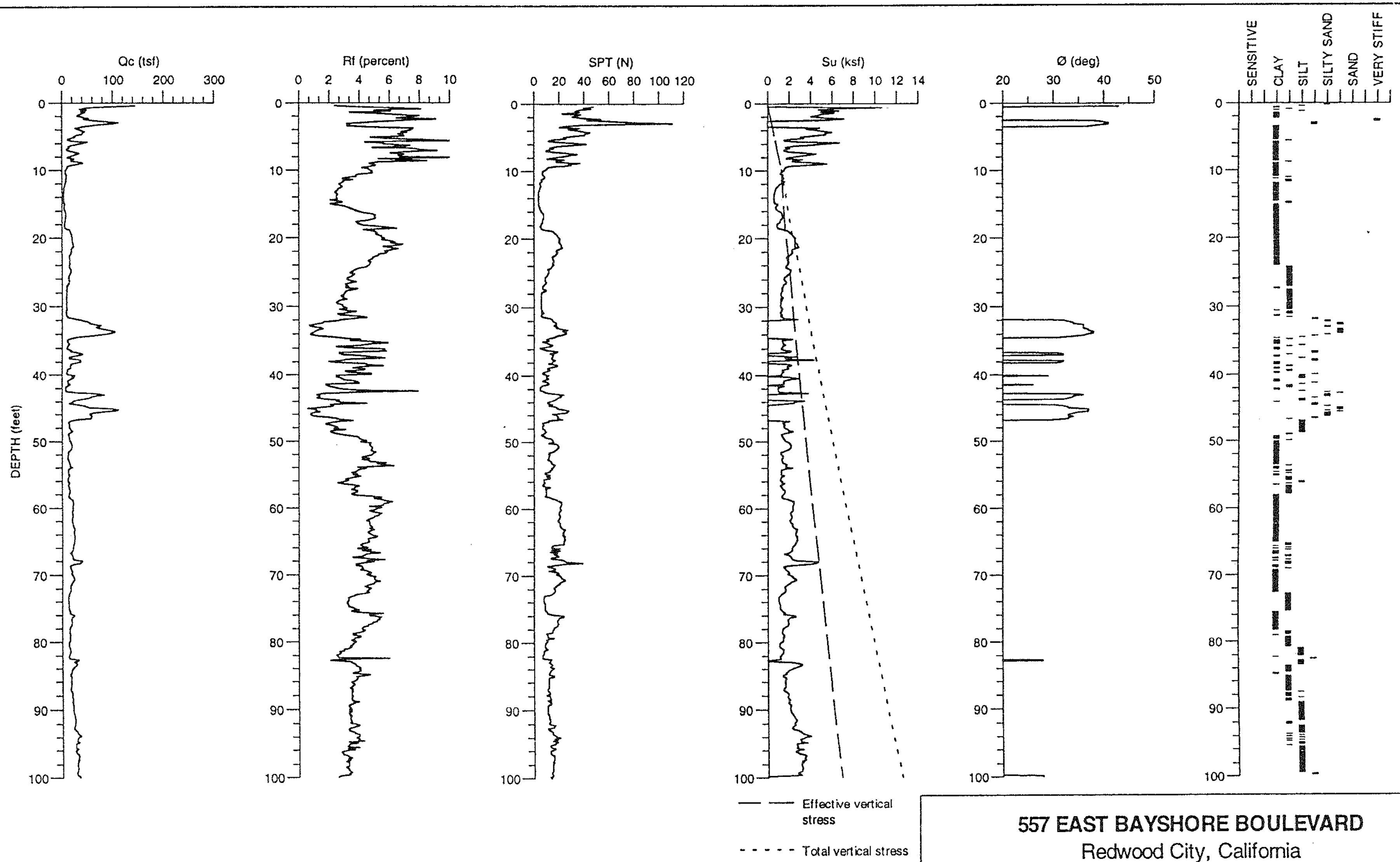


Terminated at 100.0 feet
Groundwater measured at 11.2 feet.
Date performed: 26 July 2001.
Elevation: 107.0 feet, datum: 1929 NGVD plus 100 feet.

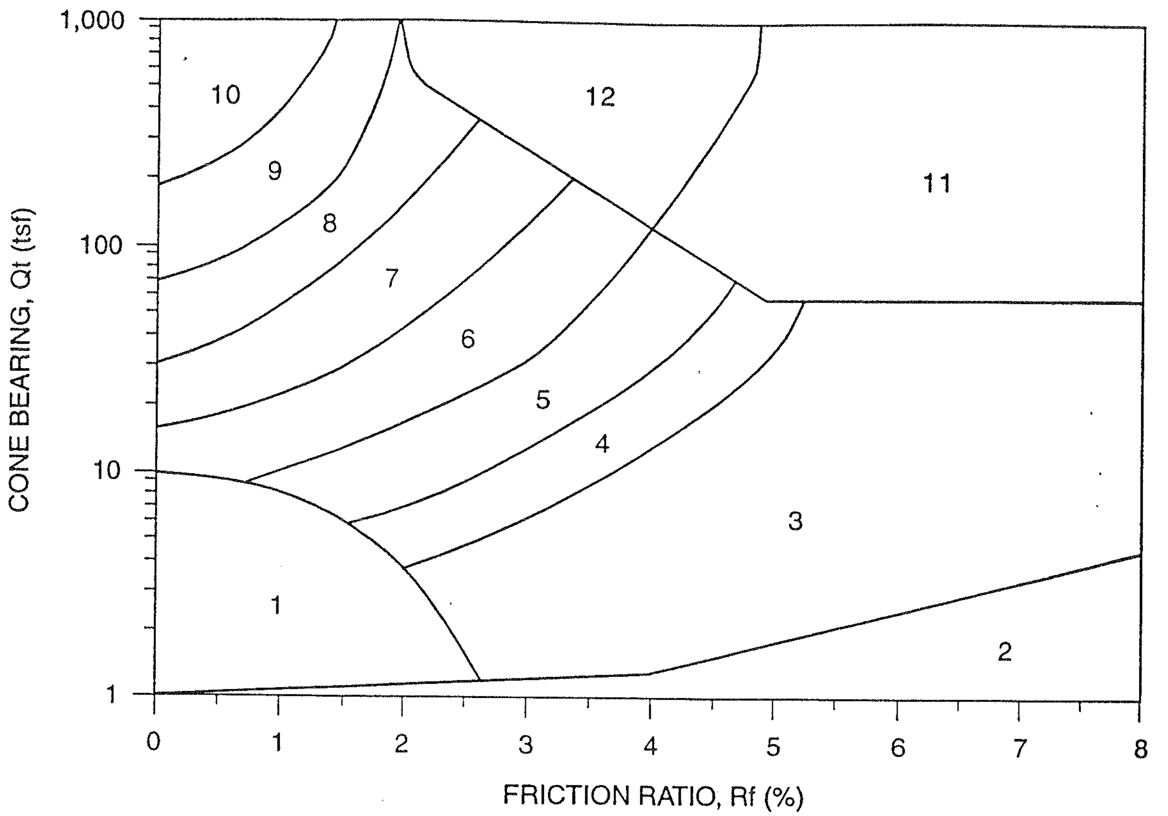
557 EAST BAYSHORE BOULEVARD Redwood City, California		
CONE PENETRATION TEST RESULTS **CPT-5**		
Date 07/31/01	Project No. 3190.01	Figure A-4
Treadwell&Rollo		



Terminated at 100.0 feet
Groundwater measured at 9.6 feet.
Date performed: 26 July 2001.
Elevation: 106.5 feet, datum: 1929 NGVD plus 100 feet.

557 EAST BAYSHORE BOULEVARD Redwood City, California		
CONE PENETRATION TEST RESULTS **CPT-6**		
Date 07/31/01	Project No. 3190.01	Figure A-5

Treadwell&Rollo



ZONE	Qt/N	SOIL BEHAVIOR TYPE
1	2	Sensitive Fine-Grained
2	1	Organic Material
3	1	CLAY
4	1.5	SILTY CLAY to CLAY
5	2	CLAYEY SILT to SILTY CLAY
6	2.5	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	Very Stiff Fine-Grained (*)
12	2	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
Qt = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qt x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

Reference: Robertson, 1990.

557 EAST BAYSHORE BOULEVARD
Redwood City, California

Treadwell&Rollo

GREGG

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 07/31/01 | Project No. 3190.01 | Figure A-6