GEOTECHNICAL INVESTIGATION
Emerystation No. 2
Emeryville, California

Wareham Development Group
San Rafael, California

24 September 1999
Project No. 2254.04

Treadwell&Rollo
Environmental and Geotechnical Consultants

24 September 1999
Project 2254.04

Treadwell&Rollo

Mr. Richard K. Robbins, President
Wareham Development Group
1120 Nye Street, Suite 400
San Rafael, California 94901

SEP 30 1999
WEBCOR BUILDERS

Subject: Geotechnical Investigation
Emerystation No. 2
Emeryville, California

Dear Mr. Robbins:

Our geotechnical investigation report dated 24 September for Emerystation No. 2 in Emeryville, California is attached. Additional copies have been distributed as shown on the distribution page at the end of this report. This investigation was performed in general accordance with our proposal dated 19 July 1999.

The project site is underlain by fill and alluvial clay deposits consisting of moderately strong to strong clay. Alternatives including an excavation with a mat and a deep foundation system gaining support within the strong material were considered. We understand an excavation combined with a mat is not feasible. Therefore, we mutually agreed that deep piles should be used to support the building.

The recommendations contained in the report are based on limited subsurface exploration and laboratory testing programs. Consequently, variations between expected and actual soil conditions may be encountered in localized areas during construction. Therefore, we should be retained to observe foundation installation and fill placement, during which time we may modify our recommendations, as deemed necessary.

We appreciate the opportunity to continue to provide our services to Wareham Development Group. If you have any questions, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

Christian J. Divis
Christian J. Divis
Civil Engineer

CHRISTIAN J. DIVIS
No. C-059451
Exp. 12/31/99
CIVIL

Frank L Rollo
Frank L. Rollo
Geotechnical Engineer

FRANK L ROLLO
GE733
Exp 6/30/01
GEOTECHNICAL

22540402.CJD

Treadwell & Rollo, Inc. *Environmental & Geotechnical Consultants*
555 Montgomery Street, Suite 1300, San Francisco, CA 94111
Telephone (415) 955-9040 Facsimile (415) 955-9041

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TABLE OF CONTENTS

LIST OF FIGURES

GEOTECHNICAL INVESTIGATION
Emerystation No. 2
Emeryville, California

1. INTRODUCTION

This report presents the results of our geotechnical investigation for the Emerystation No. 2 development in Emeryville. The project location is shown on the Site Location Map, Figure 1. The project site presently includes a paved parking and an undeveloped lot. The site is north of Emerystation No. 1 as shown on the Site Plan, Figure 2, and is on the northeast corner of 59th and Landregan Streets. Site grades currently range between Elevation 13.6 and 17 feet[1].

Current plans are to construct a six-story, steel-framed, office building. The building will contain parking, commercial, and office space. The proposed building will be rectangular in shape and approximately 283 by 178 feet in plan. Typical column spacing for the building will be approximately 20 feet each way, center to center. The project structural engineer, Mr. George Fu of Hratch Kouyoumdjian and Associates, estimates typical column loads range from 500 kips to 850 kips for dead plus live loads. Total design loads, including seismic loads, may approach 2000 kips. Site grading within the building footprint is expected to be minimal; cuts and fills are expected to be on the order of 1/2 to 2-1/2 feet. The finished floor elevation will vary between 14-1/2 and 16-1/2 feet. Final soil subgrade elevation will be approximately 12 inches below finished floor. Site development plans also include landscaping, and concrete flatwork.

Treadwell&Rollo, Inc. previously performed other geotechnical investigations in the project area, including Emerystation No. 1 and Emerystation No. 3. Information contained in those studies was used in conjunction with the results of our current investigation.

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[1] All elevations discussed in this report are based on City of Emeryville datum. Current site grades are based on Sheet C2, Grading, Drainage, and Utility Plan, *Emerystation No. 1, Emeryville, CA,* prepared by Kier and Wright, and dated 11 June 1999.

2. SCOPE OF SERVICES

The geotechnical investigation was performed in accordance with our proposal dated 19 July 1999. The scope of our services consisted of drilling test borings, performing cone penetrometer tests (CPTs), laboratory tests, and engineering analyses. From the results of our investigation, we developed conclusions and recommendations regarding:

- soil and groundwater conditions
- corrosion potential of the soil
- appropriate foundation type for the building
- design criteria for the recommended foundation type
- estimates of foundation settlement
- subgrade preparation for the floor slab and concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- site seismicity and seismic hazards
- 1997 Uniform Building Code site soil factors

3. FIELD INVESTIGATION AND LABORATORY TESTING

We obtained information about subsurface conditions at the site by reviewing data from previous investigations, both by us and by others, at or near the project site, drilling three test borings (B-1 through B-3) and performing two CPTs (CPT-1 and CPT-2). Approximate locations of the test borings and CPTs are shown on Figure 2. Details of the field exploration activities are described in the remainder of this section.

3.1 Test Borings

Test borings were drilled from 26 through 28 July 1999, to depths of approximately 100 feet below the ground surface. These borings were performed using truck-mounted, hollow-stem auger drilling equipment. Our field engineer logged the soil conditions encountered in the borings and obtained samples for visual classification and laboratory testing. The boring logs are presented as Figures A-1 through A-12 in Appendix A. The soil encountered was classified in accordance with the soil classification system shown on Figure A-13.

Soil samples were obtained using a split-barrel sampler and a Shelby-tube sampler. The split barrel sampler employed was a Sprague and Henwood (S&H) with 3.0-inch and 2.43-inch, inside and outside diameters, respectively. The S&H sampler was driven with a 140-pound, down-hole safety hammer with a drop height of 30-inches. The blow counts required to drive the S&H sampler the final 12-inches of an 18-inch drive were converted to SPT blow counts (N-values) and are shown on the boring logs. Shelby-tube samples were obtained using 30-inch-long Shelby tubes with 2.875-inch and 3.0-inch, inside and outside diameters, respectively. Shelby-tube samples were obtained by hydraulically pushing the Shelby tubes into the soil. The maximum hydraulic pressure, in pounds per square inch, needed to obtain each sample is shown on the boring logs.

3.2 Cone Penetration Tests

The CPTs were performed on 26 July and 30 July 1999. The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe into the ground. The cone measures tip resistance and electrical gauges within the cone continuously measure other soil parameters during the entire depth of each probing. A sleeve behind the cone tip measures frictional resistance. A small, porous stone between the cone and the friction sleeve monitors pore pressures in the soil during penetration. Soil data, including tip resistance, frictional resistance, porewater pressure, and probe inclination were recorded in the field and transferred to a computer. Accumulated data was processed using a computer to provide engineering information such as the soil type and approximate strength characteristics of the materials encountered.

The CPT logs, which show tip resistance and friction ratio with depth, as well as interpreted SPT blow counts, are presented on Figures A-14 and A-15 in Appendix A. The soil profile was generated using the Classification Chart for Cone Penetration Tests, which is presented on Figure A-16 in Appendix A. The CPTs were advanced until they met practical refusal at a depth of 86.5 and 92.5 feet below the existing ground surface.

3.3 Laboratory Testing

To measure the physical properties of the soil, moisture-density, consolidation, and strength test were performed on samples of soil recovered from the test borings. Results of the tests are shown on the boring logs at the appropriate depths and in Appendix B. Additionally, corrosivity analyses were performed, on a sample of the near surface soil. The results of the corrosivity tests are discussed in Section 5.4 and presented in Appendix C.

3.4 Soil Cutting Disposal

All soil cuttings and drilling spoils were drummed in 55-gallon drums. Each drum was labeled and stored on site. A total of 19 drums were generated. Four samples of the drilling spoils were collected from each test boring location. Composite samples of the drilling spoils were tested for contamination. The results of this analysis are presented in Appendix D. Drums were disposed of in a manner appropriate to the test results.

4.0 SITE AND SUBSURFACE CONDITIONS

The northern edge of the site is currently an undeveloped lot. The remainder of the site is a paved parking lot. The site is blanketed by 1.5 to 6 feet of fill. The fill consists of silty gravel and gravelly clay. The upper 18-inches of the fill within the southern half of the site was lime treated and compacted to a relative compaction[2] of at least 90 percent during the construction of Emerystation No. 1. The fill underlying the lime treated soil is heterogeneous and unpredictable in regard to strength and compressibility. Additionally, portions of this fill may be expansive[3]. During our investigation, we encountered 6-inches of concrete at test boring B-2. This concrete is most likely a slab that covers the southern half of the site. The soil beneath the fill consists of native alluvial deposits.

Subsurface information from our test borings and CPTs indicates the alluvial deposit soil consists of clay. This clay is moderately strong and moderately compressible to a depth of approximately 40 feet. Below 40 feet the clay is strong. The clay is overconsolidated; it has experienced a greater overburden pressure in the past, than currently exists. Inter-bedded layers of medium-dense to very-dense sand and clayey sand is present in the clay deposits to the maximum depths explored.

During our earlier investigations, we encountered groundwater levels at the surrounding sites between Elevation 1 and 4-1/2- feet. These groundwater levels measurements were approximate; they were measured at soil boring locations where the groundwater elevation did not have time to stabilize. Previous environmental studies by others, which included the installation of many shallow monitoring wells, found large variations in groundwater elevations. These studies found the groundwater to be between Elevation 2 and 10 feet.

During the construction of Emerystation No. 1, we observed groundwater tables close to or at the ground surface. However, it is likely that the installation of a storm sewer system at the Emerystation No. 1 development has lowered the natural groundwater elevation. Our current investigation found the groundwater at about Elevation 4 feet. Groundwater levels should be expected to fluctuate depending on rainfall amounts and time of year; therefore, after a review of

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[2] 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.

[3] Expansive clays tend to undergo volume changes, i.e. shrink and swell, with changes in moisture.

available subsurface information by others and our observations, we judge a groundwater elevation of 6 feet should be used in design.

5.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, the proposed office building can be constructed as planned. The primary geotechnical issues at this site are the presence of heterogeneous, expansive fill and moderately compressible clay. Because the building will be built at or near existing grade, the potential for excessive settlement of the fill and underlying clay must be considered in the foundation design. This and other issues are discussed in the remainder of this section.

5.1 Foundations and Settlement

On the basis of the results of this investigation, our past experience with similar projects, and discussions with the design team, we conclude that a shallow foundations system is not appropriate for this project. A shallow spread-type foundation would impose high pressures and cause the underlying soil to settle differentially, creating erratic building performance. We judge the most suitable foundation would be driven piles that gain support in the strong clay below 40 feet. From our experience with the Emerystation No. 1 and similar projects, we conclude that precast, prestressed concrete piles are the most appropriate pile type for the project. After discussions with the structural engineer, we mutually agreed that 14-inch-square prestressed precast piles would be best suited for this project.

Although the piles will transfer building loads to less compressible strata, some minor settlement (less than 1/2 inch) of the pile foundations will still occur as the building loads are applied.

5.2 Subgrade Beneath Floor Slabs

The subgrade soil beneath the proposed floor slabs is moderately compressible and potentially expansive. Floor slabs supported on these materials may be subject to vertical differential movements that could approach one inch. We understand this potential movement is acceptable.

Consequently, a slab-on-grade, at least five-inches thick, will be used throughout the building. To provide uniform support and reduce the expansion potential, the final soil subgrade will require moisture conditioning and recompaction.

5.3 Site Seismicity

The Bay Area is a seismically active region. Strong ground shaking from an earthquake should be expected. The major active faults in the area are the San Andreas, Hayward, and Calaveras Faults. These and other active[4] or potentially active[5] faults in the region are shown on Figure 4. For each of the active faults, the distance from the site and estimated maximum credible event are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault	Approximate Distance From Site (kilometers)	Direction From Site	Maximum Magnitude[6, 7]
Hayward (North segment)	5	Northeast	6.9
Calaveras (North of Calaveras Reservoir)	23	Southeast	6.8
San Andreas (Peninsula segment)	25	Southwest	7.1

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[4] Active faults are defined as those exhibiting either surface ruptures, topographic features created by faulting, surface displacements of geologically Recent (younger than about 11,000 years old) deposits, tectonic creep along fault lines, and/or close proximity to linear concentrations or trends of earthquake epicenters.

[5] Potentially active faults are those that have evidence of displacement of deposits of Quaternary age (the last 2 million years).

[6] Maximum Magnitude Earthquake (Moment magnitude), as referenced from *Probabilistic Seismic Hazard Assessment for the State of California* by the California Department of Conservation, Division of Mines and Geology, Open File Report 96-08.

[7] 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.

In 1990, the Working Group on California Earthquake Probabilities (WGCEP), organized by the U.S. Geological Survey (USGS), predicted a 67 percent probability of a magnitude 7.0 earthquake occurring in the San Francisco Bay area between 1990 and 2020 (WGCEP, 1990). More specific estimates of these probabilities for different fault segments in the Bay Area, pertinent to this project, are presented in Table 2.

TABLE 2
USGS (1990) Estimates of 30-year Probabilities of a Moment Magnitude 7.0 Earthquake

Fault Segment	30-year Probability M = 7.0 (percent)
San Andreas – Peninsula	23
Hayward – North	28
Hayward – South	23

5.4 Geologic Hazards

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. 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 is very low. Therefore, we conclude the hazard of fault offset at the site from a known active fault is low.

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[8], lateral spreading[9], and differential compaction[10]. Soil most susceptible to these phenomena is saturated sand or silt of low or medium relative density that is relatively free of clay. On the basis of our subsurface investigation, we conclude the risk of liquefaction, lateral spreading, and differential compaction at this site is low because of the high relative densities and/or cohesion of the soil underlying the site.

5.5 Corrosivity Potential

On the basis of the results of the corrosivity analyses (presented in Appendix D) performed on a sample obtained during the field investigation, the soil at the site is considered "corrosive." All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be protected against corrosion.

The sulfate ion concentration in the sample is sufficient to damage reinforced concrete structures and the redox potential is indicative of potentially "slightly corrosive" soil resulting from anaerobic conditions. Therefore, a sulfate resistant concrete mix using Type II cement should be used for concrete that comes into contact with this soil. Reinforcement steel should be protected in accordance with the prevailing code.

6.0 RECOMMENDATIONS

Recommendations for site preparation and fill placement, pile foundations, concrete slabs, and seismic factors are presented in the remainder of this report.

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[8] 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.

[9] 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.

[10] Differential compaction is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing differential settlement.

6.1 Site Preparation and Fill Placement

Where removed, asphalt can be used as fill provided it is pulverized. Existing concrete and buried utilities should be removed from areas to be developed. We understand portions of old foundations from previous buildings still exist beneath the site; where they interfere with the new construction, they should be removed. These materials should be disposed of offsite. The existing fill and asphalt may be used to grade the site, provided any organic material is removed and no rocks or lumps larger than four inches in greatest dimension are included.

Where the existing asphalt surface is below the final soil subgrade, it may remain in place provided it is broken up and/or ripped to allow water to migrate on through the existing pavement.

All imported materials to be used as fill or trench backfill should be free of organic material, contain no rocks or lumps larger than four inches in greatest dimension, and have a low expansion potential defined by a liquid limit of less than 40 and a plasticity index lower than 12. During construction, we should check that any proposed import material is suitable for use as fill.

After the exposed subgrade is cleared, grubbed and stripped, it should be scarified to a depth of six inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. All fill and backfills should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness and compacted to at least 90 percent relative compaction. The final soil subgrade should be moisture conditioned to two percent above the optimum moisture and compacted to at least 95 percent relative compaction.

6.2 Pile Axial Loads

On the basis of the length of pile anticipated for the project, preliminary load requirements, and construction considerations, we recommend that driven 14-inch-square, precast, prestressed, concrete piles be used to support the proposed building. The piles will gain support through skin friction in the underlying clay layers. Pile lengths should be determined using Figure 5. We recommend that all piles be at least 60 feet long. Pile capacities presented in Figure 5 are ultimate capacities; depending upon load conditions, factors of safety should be applied to the capacities presented. Typically, a factor of safety of 2.0 is used for calculating dead plus live load capacity. The structural capacity of the piles should be checked and may govern.

Because no appreciable fill will be placed at the site, the pile capacities need not be reduced for downdrag loads. Piles should be spaced no closer than three pile widths center to center to avoid reductions to the vertical capacities due to group effects.

6.3 Lateral Pile Capacity

Lateral load resistance can be mobilized by the individual piles in combination with other foundation elements embedded below the ground surface. We recommend using a passive resistance acting against the vertical faces of pile caps and grade beams equivalent to a fluid pressure of 300 pounds per cubic foot (pcf). This value includes a factor of safety of about 1.5.

The lateral capacity of piles will depend on the amount of deflection and bending moment that can be tolerated. The deflections and bending moments resulting from various lateral loads applied to the top of single 14-inch-square, precast, prestressed, concrete piles are presented on Figures 6 and 7, for the free- and fixed-head condition, respectively. The piles were analyzed under compressive loads of 260 kips, and a minimum pile tip depth of 60 feet. The geotechnical parameters used in the lateral pile capacity analyses do not include a factor of safety. The reinforcing steel needed to provide lateral capacity should extend at least 8 feet below the top of pile. At a deflection of one-quarter inch, the 8-foot depth is below the zone of passive resistance.

For pile groups where the center-to-center spacing is less than eight pile diameters in the direction of loading, the single pile lateral capacities should be reduced. Reduction factors, corresponding to the number of piles in a group, for three pile diameter center to center spacing, are given in Table 3. We can provide lateral load analyses for other pile groups and arrangements when they have been established.

TABLE 3
Pile Group Reduction Factors for Three Pile Diameter Center to Center Spacing

Number of Piles in Pile Group	Reduction Factor
2	0.84
3 and 4	0.83
5	0.82
6, 8, 9	0.73
10	.69

Before production piles are cast, we recommend that at least ten indicator piles be driven to observe the driving characteristics of the piles and the performance of the driving equipment. Indicator piles should be installed at production pile locations selected by us and approved by the structural engineer. The indicator piles will provide driving resistance data to correlate with information obtained from the test borings, to aid in evaluating predrilling requirements, and to be used as the basis for establishing final production pile lengths.

6.4 Pile Installation

Adjacent structures should be monitored for movement during pile installation. Survey points should be established at various locations on structures within 50 feet of the site. To check for movements, these points should be monitored weekly during production pile installation.

Determination of driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length. Special consideration should be given to selecting a hammer, which can deliver enough energy to the tip of the piles to drive them efficiently without damaging them. We recommend that the maximum energy delivered by the hammer be limited to 90,000 foot-pounds of energy per blow to reduce the potential of damaging the piles.

Predrilling allows piles to be driven with minimal damage and helps the contractor to maintain close alignment of the tops of the piles in the upper soil layer, where obstructions may exist. We recommend pile locations should be predrilled to a depth of at least 10 feet. The predrill auger should have a diameter no greater than the minimum pile width.

We understand the contractor plans to pre-drill pile locations and remove any obstructions encountered well in advance of production pile driving. Piles will be driven from existing grade with a follower. In so doing, portions of the existing parking lot can remain in use for an extended period of time.

Considering that: a) the piles should gain support in skin friction, b) the piles driven on the adjacent site (Emerystation No. 1) achieved friction length with little or no cut-off, c) each pile location will be predrilled well in advance of production pile driving, and d) the existing parking lot cannot be closed for an extended period of time, we mutually agreed that indicator piles will not be driven at this site.

The existence of dense sand lenses may cause the piles to stop short of design length. Therefore, some pile cut-off may be necessary.

6.5 Floor Slabs

The five-inch thick slab-on-grade floor should be underlain by at least six inches of open graded crushed rock.

Before placing the rock, the final subgrade should be rolled to expose a smooth, non-yielding surface. Where moisture migration would be detrimental to floor covering, such as in the lobby and retail spaces, the rock should be overlain by a moisture-proof membrane of at least 10 mil thickness. The membrane should be overlapped at least 12 inches at all joints. A two-inch protective layer of sand should be placed above the visqueen to prevent puncture and aid in curing concrete. The sand should not be allowed to get wet before concrete placement.

6.6 Seismic Design

If the provisions of the 1997 Uniform Building Code are used, we recommend the following:

- Seismic Zone Factor 4
- Soil profile Type S_D
- Near Source Factors N_a and N_v of 1.2 and 1.6, respectively.

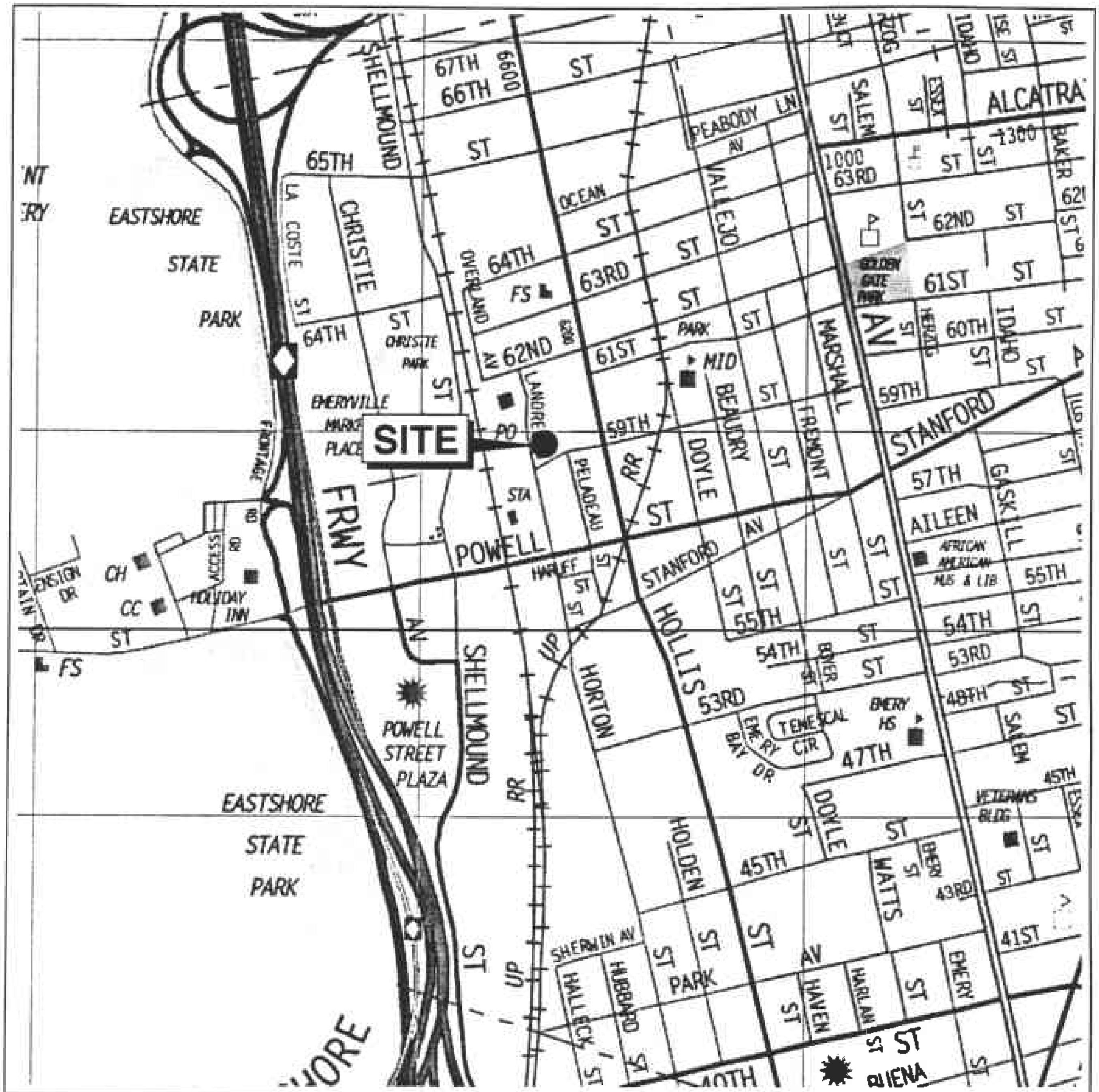
7.0 ADDITIONAL GEOTECHNICAL SERVICES

We should review the final plans and specifications to check that they are in general conformance with the intent of our recommendations. During construction, an engineer from our office should observe subgrade preparation for slabs-on-grade, indicator and production pile installation, and placement and compaction of any backfill. These observations will allow us to compare actual with anticipated soil conditions, check that the contractor's work conforms with the geotechnical aspects of the plans and specifications, and ensure that the work is performed as planned. When the preliminary design is complete, we would be pleased to provide you with a cost estimate for these services.

8.0 LIMITATIONS

We performed our services in a manner consistent with the level of care and skill ordinarily exercised by professional consultants performing comparable services under similar circumstances as those encountered at this project site. We make no representation, warranty or guarantee, expressed or implied.

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of engineering studies and our interpretations of the existing geotechnical conditions. Actual subsurface conditions may vary. Should conditions differ substantially form those we anticipate some modifications to our conclusions and recommendations may be necessary.



Base map: Thomas Brothers Maps
Alameda County
1999

EMERYSTATION NO. 2
Emeryville, California

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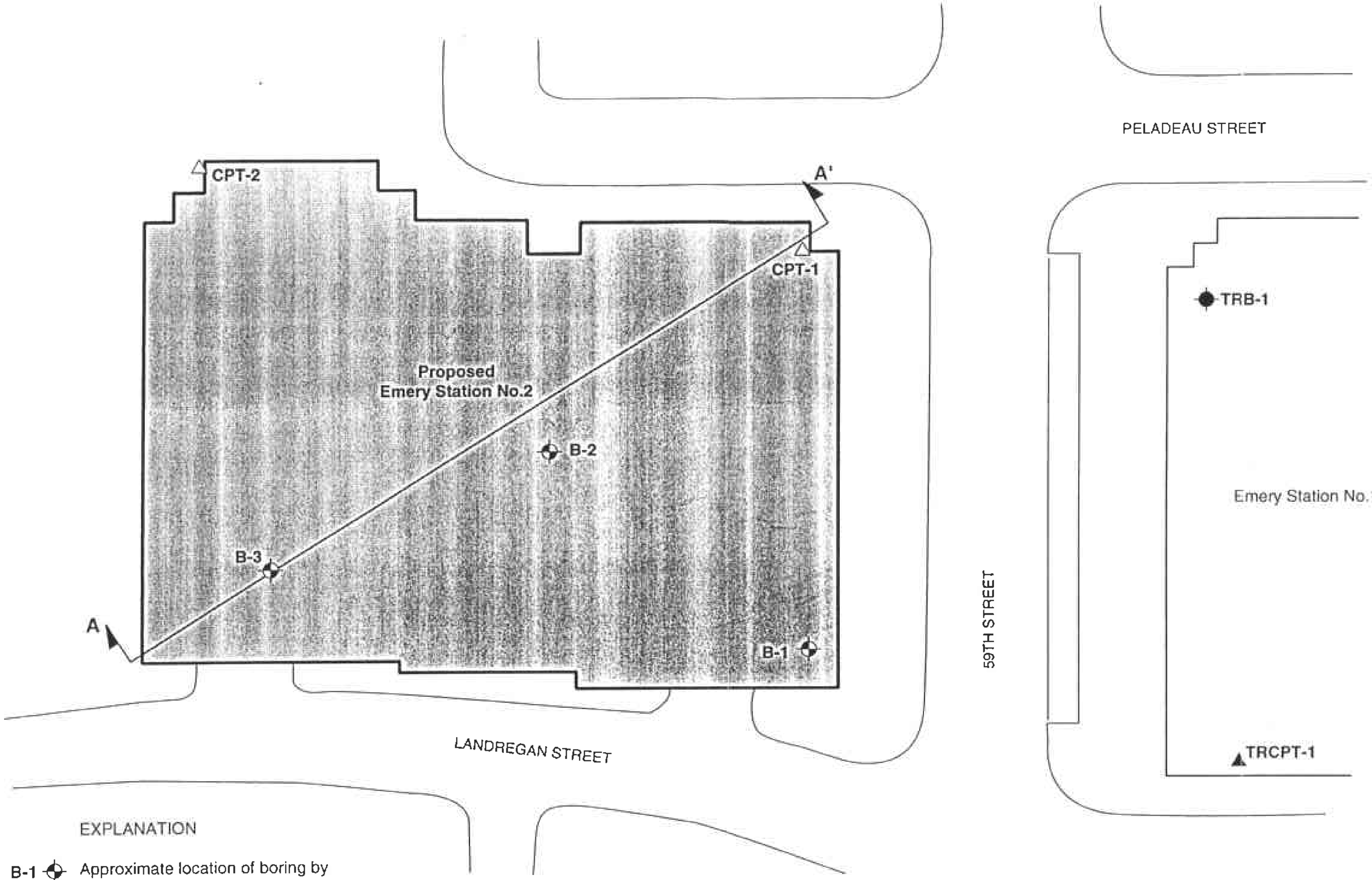
SITE LOCATION MAP

Date 9/15/99 | Project No. 2254.04 | Figure 1



EXPLANATION

B-1 ⌖	Approximate location of boring by Treadwell & Rollo, performed August 1999
CPT-1 △	Approximate location of cone penetration test by Treadwell & Rollo, performed August 1999
TRB-1 ●	Approximate location of boring by Treadwell & Rollo, performed November 1997
TRCPT-1 ▲	Approximate location of cone penetration test by Treadwell & Rollo, performed November 1997

0 40 Feet
Approximate scale

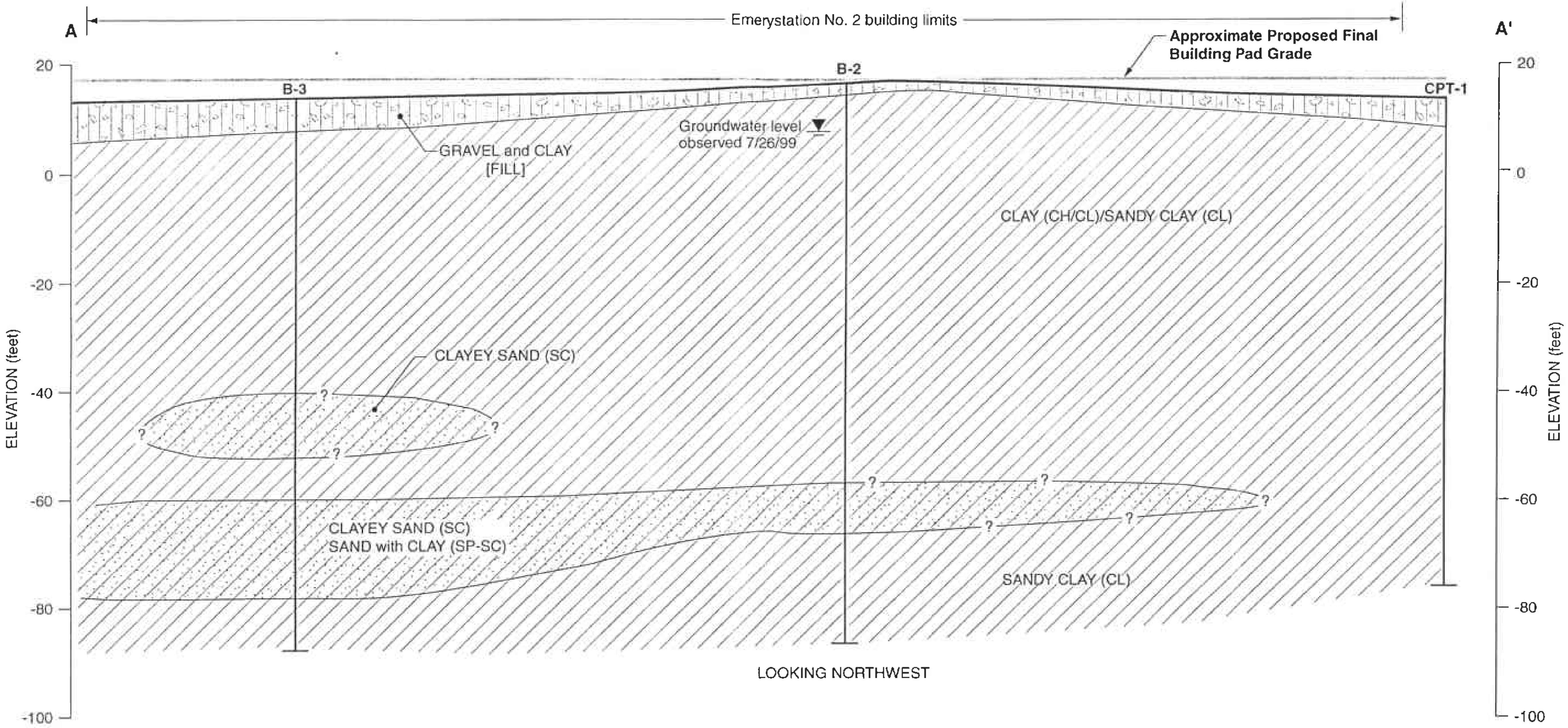
Reference: "1st Level Plan", by Heller Manus Architects, July 12, 1999.

EMERYSTATION NO. 2
Emeryville, California

SITE PLAN

Date 9/20/99	Project No. 2254.04	Figure 2

Treadwell&Rollo



EXPLANATION

B-2
Soil boring

Notes:
1. Elevation in feet, Emeryville datum.
2. Based on limited subsurface information soil type may vary in thickness, and physical properties.
3. Design groundwater elevation 6 feet.

20 Feet
0
0
20 Feet
Approximate scale

EMERYSTATION NO. 2
Emeryville, California

IDEALIZED SUBSURFACE PROFILE A - A'

Date 9/23/99	Project No. 2254.04	Figure 3

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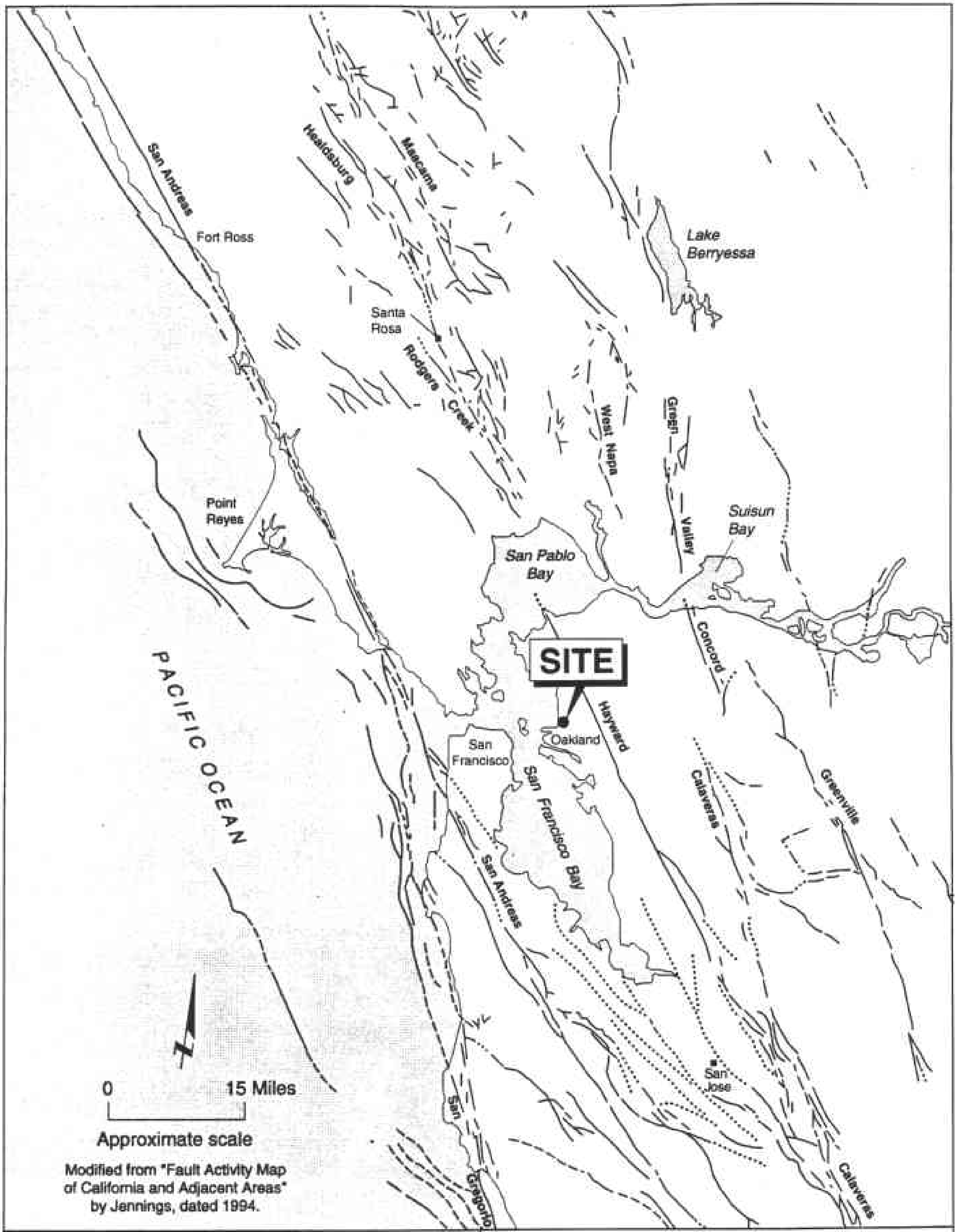

Modified from "Fault Activity Map of California and Adjacent Areas" by Jennings, dated 1994.

EMERYSTATION NO.2
Emeryville, California

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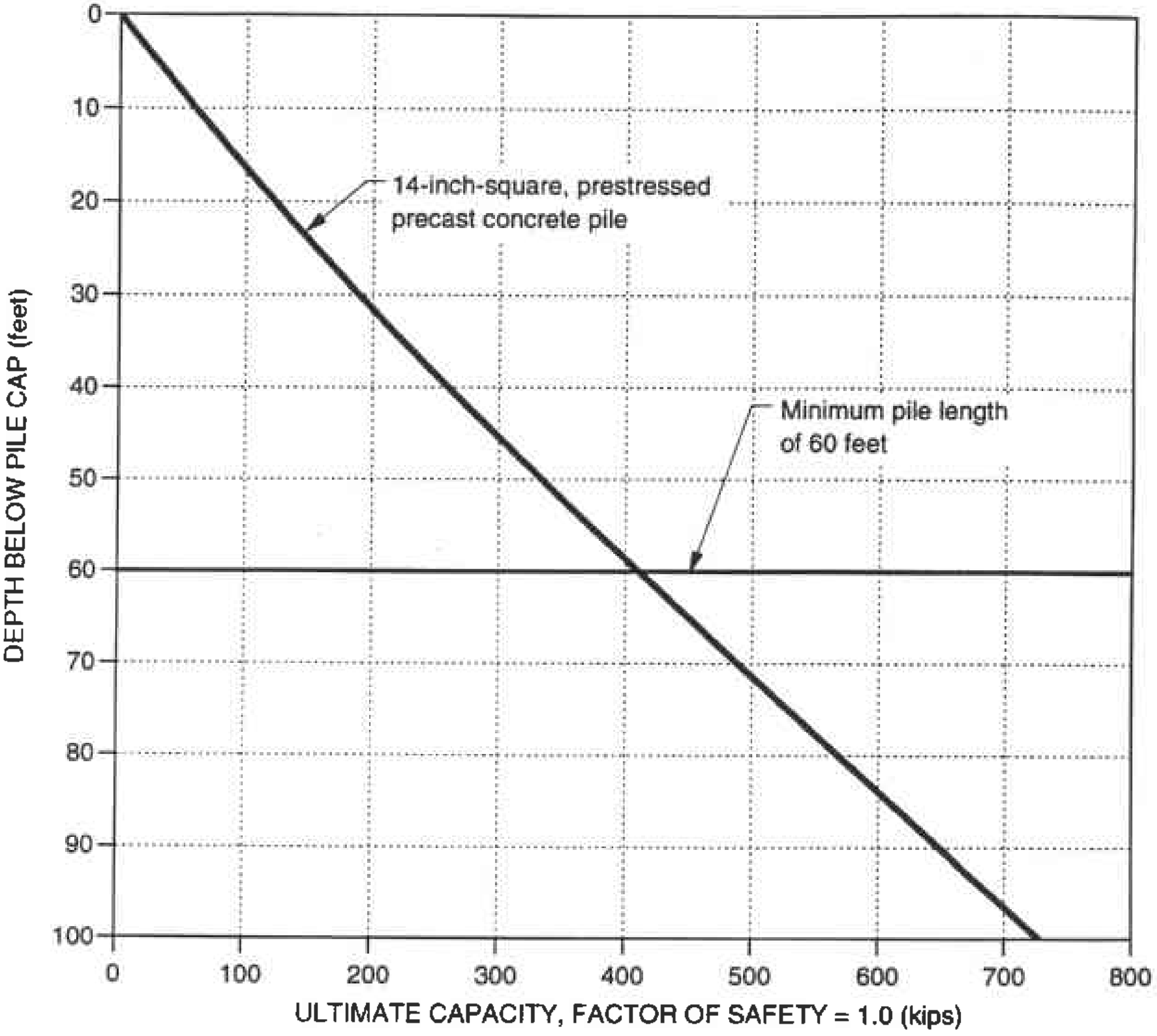
ACTIVE AND POTENTIALLY ACTIVE FAULT MAP

Date 9/15/99 | Project No. 2254.04 | Figure 4



Notes:
1. The indicated capacities are ultimate capacities (factor of safety = 1.0); factor of safety of 2.0 recommended for dead plus live load conditions.
2. For sustained uplift, use 60 percent of the indicated capacities.
3. Capacities are based on the strength of the supporting soil; the structural capacity of the pile may govern.
4. Piles should be spaced no closer than three diameters center to center.
5. Assumed garage finish floor at Elevation16.5 feet (City of Emeryville datum).
6. To reduce settlement, all piles should be driven at least 60 feet below pile cap.
7. To reduce vibration and potential heave, all pile locations should be predrilled at least 10 feet.

EMERYSTATION NO. 2
Emeryville, California

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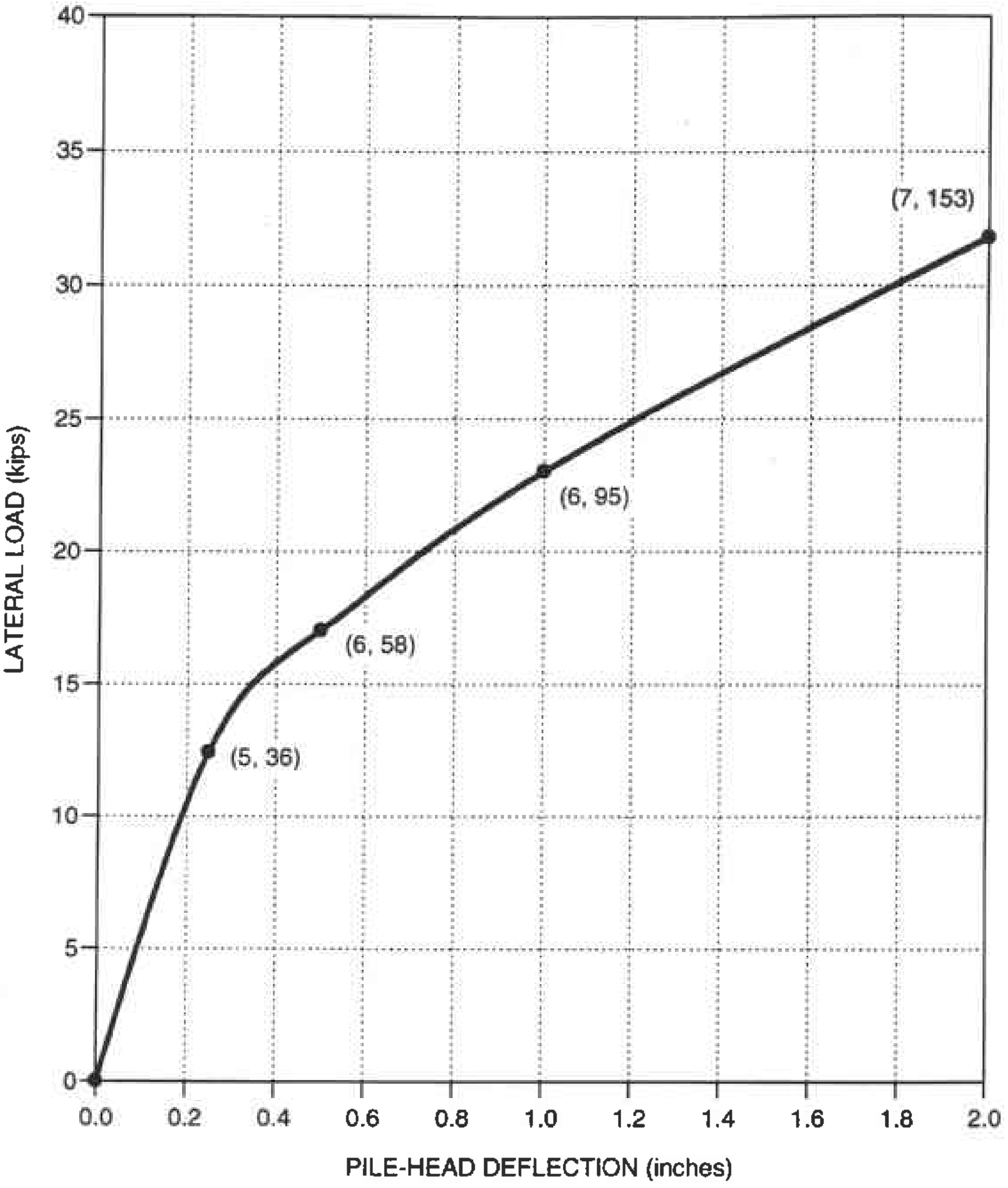
ULTIMATE PILE CAPACITY
14-INCH-SQUARE PILE

Date 8/25/99 | Project No. 2254.04 | Figure 5



EXPLANATION

(6,58)	Denotes depth below top of pile to maximum moment in feet; maximum moment in kip-feet.

Note: Assumes a minimum pile length of 60 feet below the pile cap. Capacities are based on the strength of the soil; the structural capacity of the pile may govern.

EMERYSTATION NO. 2
Emeryville, California

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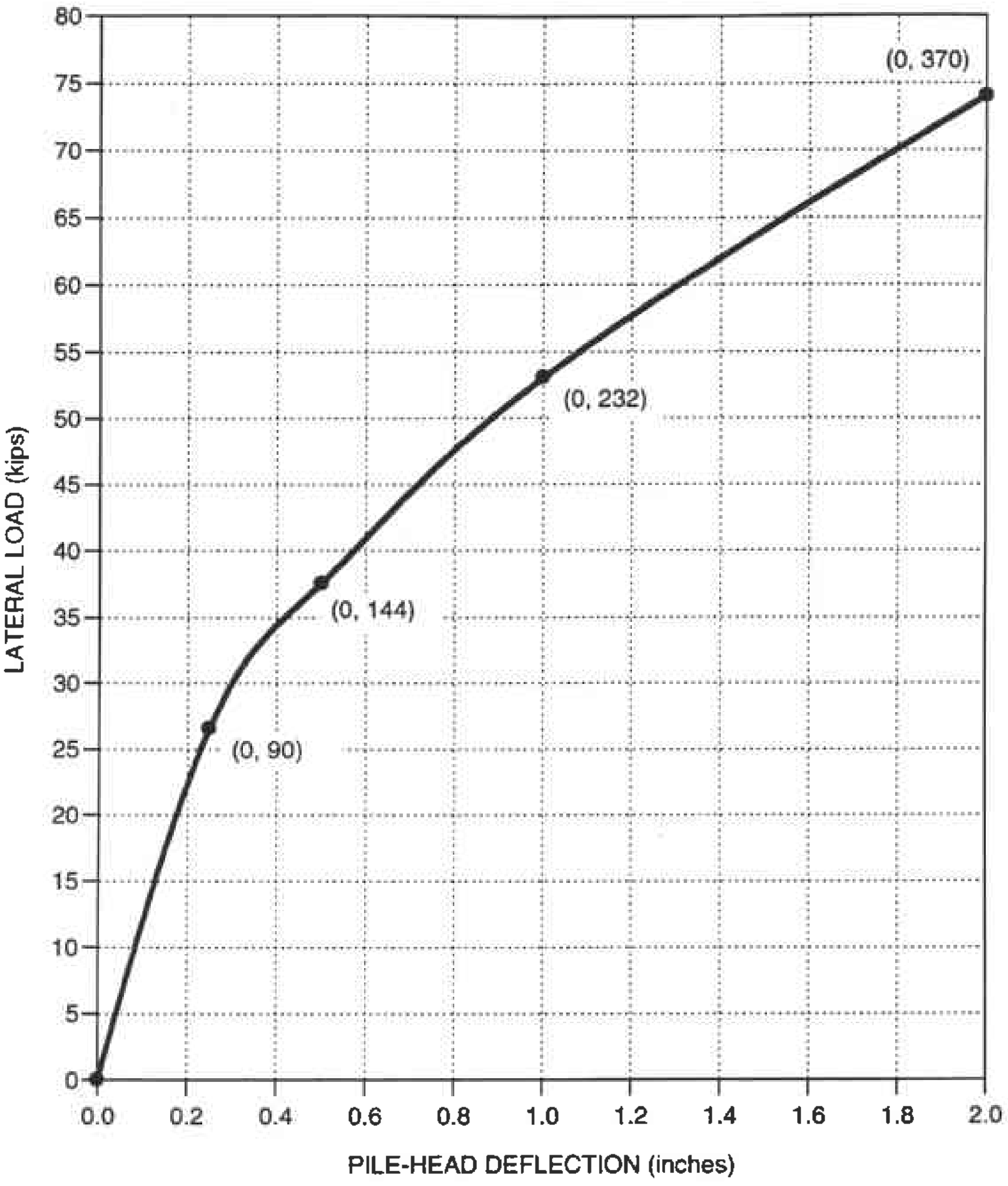
SINGLE PILE
LATERAL LOAD DEFORMATION
FREE-HEAD CONDITION

Date 9/24/99 | Project No. 2254.04 | Figure 6



EXPLANATION

(0,144)	Denotes depth below top of pile to maximum moment in feet; maximum moment in kip-feet

Note: Assumes a minimum pile length of 60 feet below the pile cap. Capacities are based on the strength of the soil; the structural capacity of the pile may govern.

EMERYSTATION NO. 2
Emeryville, California

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SINGLE PILE
LATERAL LOAD DEFORMATION
FIXED-HEAD CONDITION

Date 9/24/99 | Project No. 2254.04 | Figure 7

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-1

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Logged by: M. James

Date started: 7/27/99 | Date finished: 7/27/99

Drilling method: Hollow stem auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole safety hammer

Sampler: Sprague & Henwood

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					Ground Surface Elevation: 15 feet [2]						
0				GW ML	Asphalt concrete 3" thick FILL						
1					SILTY GRAVEL (GW - ML) gray-brown, dense, dry						
3				CH	SILTY CLAY (CH) black, medium stiff, wet						
5–6.5	S&H	☐	6								
6.5				CL	CLAY with SAND (CL) gray-brown and yellow-brown, very stiff, moist						
10–11.5	S&H	☐	16			TxUU	1,000	1,991		19.9	108
15–16.5	S&H	☐	17	CL	CLAY (CL) yellow-brown, olive, and black, very stiff, moist Consolidation Test, see Figure B-5					28.7	95
20–21.5	S&H	☐	19		gray and gray-brown					26.5	99
21.6				SC	CLAYEY SAND (SC) gray-brown, dense, wet, fine- to coarse-grained sand, some fine gravel						
25–26.5	S&H	☐	17								
25.5				CL ML	SILTY CLAY (CL - ML) gray-brown, very stiff, moist						
28.5				SP	SAND (SP) yellow-brown, medium dense, wet, fine-grained, trace silt						
30											

Treadwell&Rollo | Project No. 2254.04 | Figure A-1

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-1

PAGE 2 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/foot^1			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
31	S&H		23	SC	CLAYEY SAND (SC) yellow-brown and red-brown, dense, wet, some fine to coarse friable gravel						
32											
33				ML	SILT (ML) gray-brown, hard, moist						
34											
35				SP	SAND (SP) brown and red-brown, dense, moist to wet, fine-grained						
36	S&H		25							24.1	102
37					CLAY (CH) gray, very stiff, moist						
38				CH							
39											
40											
41	S&H	○	19		CLAY (CL) gray and gray-brown, stiff, moist						
42											
43											
44											
45				CL							
46	S&H		13							23.0	104
47											
48											
49											
50											
51	S&H		28		CLAY with SAND (CL) yellow-brown, red-yellow, and brown, very stiff, moist to wet, fine to coarse sand, some fine to coarse gravel						
52											
53											
54											
55				CL							
56											
57											
58											
59											
60											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-1

PAGE 3 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
61	S&H		23		CLAY (CL) orange-brown and gray-brown, very stiff, moist						
62				CL							
63											
64											
65											
66					CLAYEY SAND (SC) yellow-brown, dense, wet, fine- to coarse-grained, some fine gravel						
67											
68											
69											
70				SC							
71	S&H		38							22.5	103
72											
73											
74											
75											
76					SILT (ML) yellow-brown, hard, moist						
77											
78											
79											
80											
81	S&H		43	ML	occasional siltstone cobbles						
82											
83											
84											
85											
86					SILTY CLAY (CL - ML) gray/yellow-brown, hard, moist						
87											
88				CL ML							
89											
90											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-1

PAGE 4 OF 4

	SAMPLES					LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
	S&H		53 10"		SILTY CLAY (CL - ML) (continued)						
91											
92				CL ML							
93											
94					CLAY with SAND (CL) light gray and yellow-brown, hard, moist, fine sand						
95											
96				CL							
97											
98					SILT (ML) light brown, hard, moist						
99	S&H		44	ML							
100					Boring terminated at a depth of 100 feet. Boring backfilled with with cement grout. Groundwater obscured by drilling. [1] S&H blow counts converted to SPT N-values using a factor of 0.6. [2] Elevation based on Emeryville City datum.						
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-2

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 7/26/99
Date finished: 7/26/99
Drilling method: Hollow stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole safety hammer
Sampler: Sprague & Henwood
Logged by: M. James

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					Ground Surface Elevation: 16.6 feet [2]						
					Asphalt concrete 3" thick						
1				GW	SILTY GRAVEL (GW) gray-brown, dense, dry						
2					Concrete slab						
3					FILL						
4				CL	CLAY with SAND (CL) yellow-brown, stiff, moist, fine- to coarse-grained sand						
5											
6	S&H		11								
7				CL	CLAY with SAND (CL) dark gray to black, very stiff, moist, some coarse gravel and roots						
8											
9											
10				CL	CLAY with SAND (CL) gray, very stiff, moist, fine-grained sand						
11	S&H		20								
12					CLAY (CL) gray, very stiff, moist, some fine gravel						
13					▼						
14											
15											
16	S&H		17							25.3	101
17											
18											
19											
20											
21	S&H		17	CL	brown, no gravel						
22											
23											
24											
25											
26	S&H		20		gray-brown						
27											
28											
29											
30											

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Project No. 2254.04

Figure A-5

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
31	S&H		22		CLAY (CL) (continued) gray-brown and orange-brown	TxUU	3,000	1,986		26.7	98
32											
33				CL							
34											
35	S&H										
36			52		CLAY (CH) dark gray, hard, moist						
37											
38				CH							
39											
40	S&H		55 10"		CLAY (CL) dark gray, hard, moist						
41											
42											
43											
44											
45											
46											
47				CL							
48											
49											
50											
51	S&H	○	58 9"								
52											
53											
54											
55											
56											
57	ST		1,700 psi		SANDY CLAY (CL) yellow-brown, stiff, wet, coarse grained	TxUU	5,500	2,000		21.2	102
58				CL							
59											
60											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-2

PAGE 3 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	S&H	▭	30 2"		SANDY CLAY (CL) (continued)					24.9	100
61											
62											
63				CL							
64											
65					CLAY (CL) yellow-brown and black, hard, moist						
66											
67											
68											
69											
70				CL							
71	S&H	▭	55 10"								
72											
73											
74											
75					SAND with CLAY (SP) yellow-brown, light brown, and gray-brown, dense, moist, fine to coarse-grained						
76											
77											
78				SP							
79											
80											
81	S&H	▭	37		CLAY (CL) red-yellow and yellow-brown, with frequent black specks, hard, moist						
82											
83											
84											
85											
86											
87				CL							
88											
89											
90											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-2

PAGE 4 OF 4

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
91	S&H		52		CLAYEY SAND (SC) yellow-brown and red-brown, very dense, moist, fine- to coarse-grained sand, some fine gravel						
92											
93											
94											
95				SC							
96											
97											
98											
99											
100	S&H									21.4	106
101			57								
102					Boring terminated at a depth of 101.5 feet. Boring backfilled with with cement grout.						
103					Groundwater encountered at 13 feet.						
104					[1] S&H blow counts converted to SPT N-values using a factor of 0.6.						
105											
106					[2] Elevation based on Emeryville City datum.						
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-3

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 7/28/99
Date finished: 7/28/99
Drilling method: 8-inch hollow stem auger
Hammer weight/drop: 40 lbs./30 inches
Hammer type: Downhole safety hammer
Sampler: Sprague & Henwood
Logged by: M. James

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ foot[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 14.5 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
				GM ML	SILTY GRAVEL with SAND (GM - ML) gray-brown, dense, dry, trace rubble						
1											
2					GRAVELLY CLAY (CH) dark gray, very stiff, wet, fine gravel some organics, trace rubble						
3					FILL						
4				CH							
5											
6	S&H		24								
7					SILTY CLAY (CH) dark gray, stiff, wet, some organics						
8											
9				CH							
10											
11	ST		300 psi							25.1	98
12					SANDY CLAY (CL) light brown, yellow-brown, and gray-brown, very stiff, moist, coarse-grained sand						
13				CL							
14					Consolidation Test see Figure B-6						
15											
16	S&H		29								
17											
18					CLAY (CL) red-brown, very stiff, moist						
19											
20											
21	S&H		16							31.1	93
22				CL							
23											
24											
25											
26	S&H		15		gray-brown, stiff					23.4	104
27											
28					CLAY with GRAVEL (CL) yellow-brown, red-brown, and light brown, hard, moist						
29				CL							
30											

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Figure A-9

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-3

PAGE 2 OF 4

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
				CL	CLAY with GRAVEL (CL) (continued)						
31	S&H	☐	38	SM	SILTY SAND (SM) yellow-brown, very dense, moist, fine-grained sand						
32				CL	CLAY (CL) mottled gray-brown and yellow-brown, hard, moist						
33											
34				CL	CLAY (CL) gray, very stiff, moist						
35											
36	S&H	☐	21							21.1	107
37											
38				CL	CLAY (CL) olive-brown and gray, very stiff, moist						
39											
40											
41	S&H	☐	30							24.8	100
42											
43											
44											
45											
46											
47				CH	CLAY (CH) olive-gray and black, very stiff, moist						
48											
49											
50											
51	S&H	○	29								
52											
53											
54											
55											
56	S&H	☐	31							25.9	98
57				SC	CLAYEY SAND (SC) yellow-brown, gray-brown, and black, very dense, moist, coarse-grained sand						
58											
59											
60											

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Project No. 2254.04

Figure A-10

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-3

PAGE 3 OF 4

	SAMPLES					LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
61	S&H		38		CLAYEY SAND (SC) (continued) some fine gravel						
62											
63				SC							
64											
65											
66					CLAY (CL) yellow-brown and black, very stiff, moist						
67											
68											
69											
70				CL							
71	S&H		26			TxUU	7,000	1,748		22.2	105
72											
73											
74											
75											
76					CLAYEY SAND (SC) yellow-brown and light brown, dense, moist, fine- to coarse-grained sand						
77											
78											
79											
80											
81	S&H		56 10"							21.2	106
82											
83				SC							
84											
85											
86											
87											
88											
89											
90											

PROJECT: **EMERYSTATION NO. 2**
Emeryville, California

Log of Boring B-3

PAGE 4 OF 4

	SAMPLES					LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
91	S&H	☐	43	SC	CLAYEY SAND (SC) (continued)						
92											
93					CLAY (CL)						
94					yellow-brown and gray-brown, hard, moist						
95											
96											
97				CL							
98											
99	S&H	☐	49								
100											
101					Boring terminated at a depth of 100 feet. Boring backfilled with cement grout. Groundwater obscured by drilling.						
102											
103					[1] S&H blow counts converted to SPT N-values using a factor of 0.6.						
104											
105					[2] Elevation based on Emeryville City datum.						
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

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Project No. 2254.04

Figure A-12

UNIFIED SOIL CLASSIFICATION SYSTEM			
Major Divisions		**Symbols**	**Typical Names**
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half 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 Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART		
	Range of Grain Sizes	
Classification	**U.S. Standard Sieve Size**	**Grain Size in Millimeters**
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

SAMPLE DESIGNATIONS/SYMBOLS

⊟	Sample taken with split-barrel sampler other than Standard Penetration Test sampler. Darkened area indicates soil recovered
⧄	Classification sample taken with Standard Penetration Test sampler
◫	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample
⊡	Sampling attempted with no recovery
◫	Core sample
▼	Groundwater level at the time and date indicated

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

EMERYSTATION NO. 2
Emeryville, California

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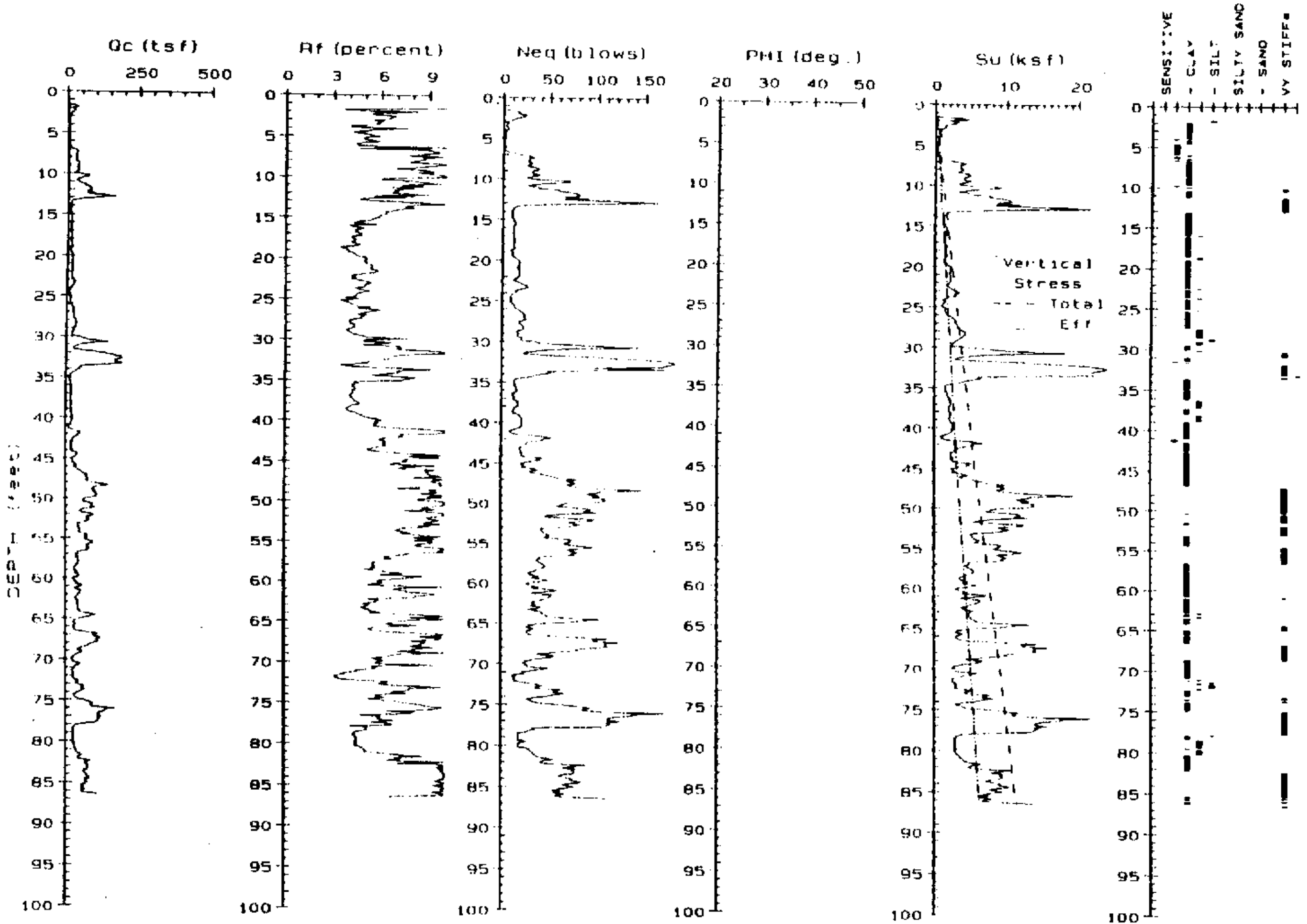
CLASSIFICATION CHART

Date 9/1/99 | Project No. 2254.04 | Figure A-13



Terminated at 86.5 feet

Groundwater estimated at 5.0 feet

Date performed: July 26, 1999

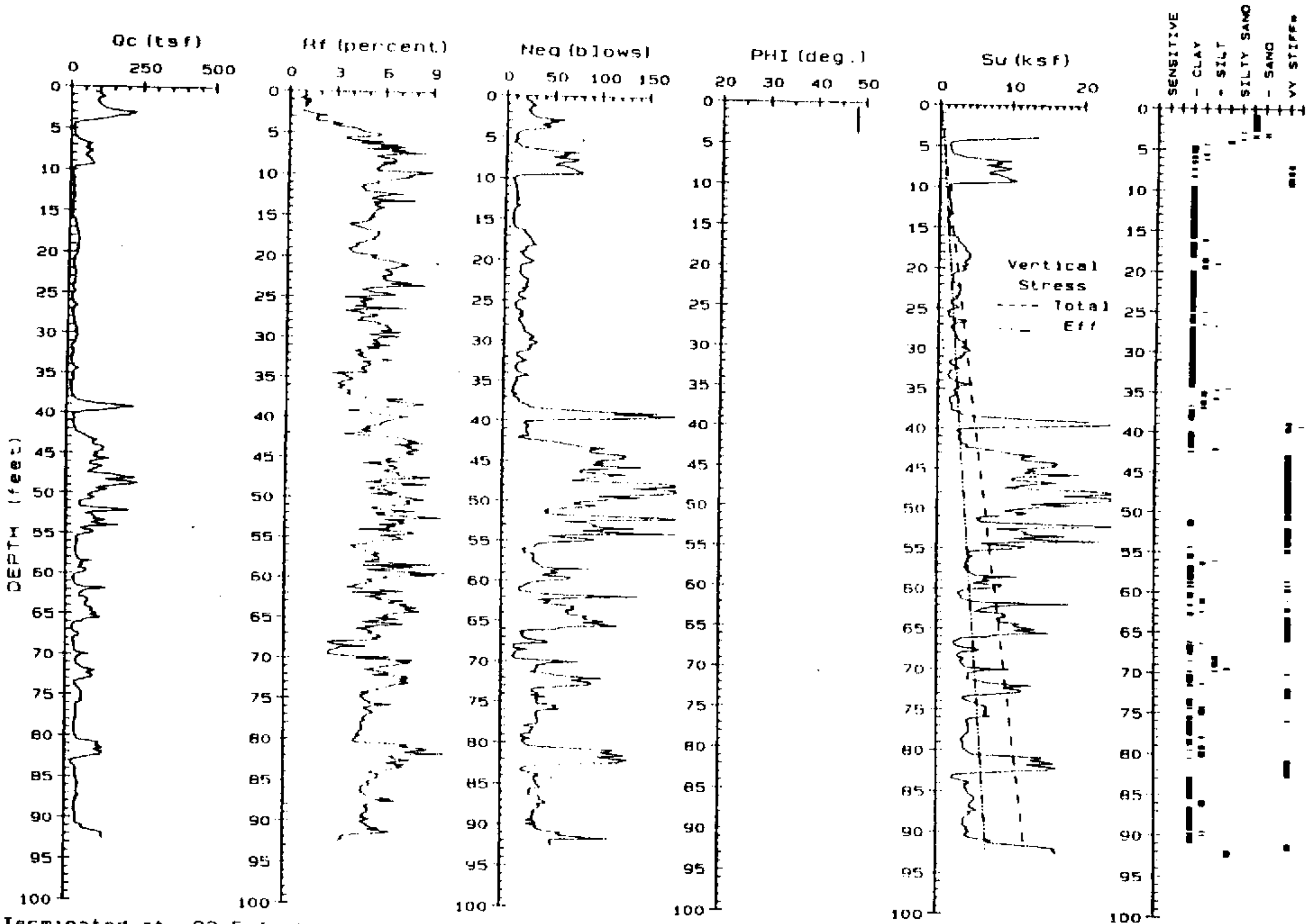
Elevation: 15.3 feet, Emeryville City datum.

EMERYSTATION NO. 2
Emeryville, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 8/20/99	Project No. 2254.04	Figure A-14

Treadwell&Rollo



Terminated at 92.5 feet

Groundwater estimated at 5.0 feet

Date performed: July 30, 1999

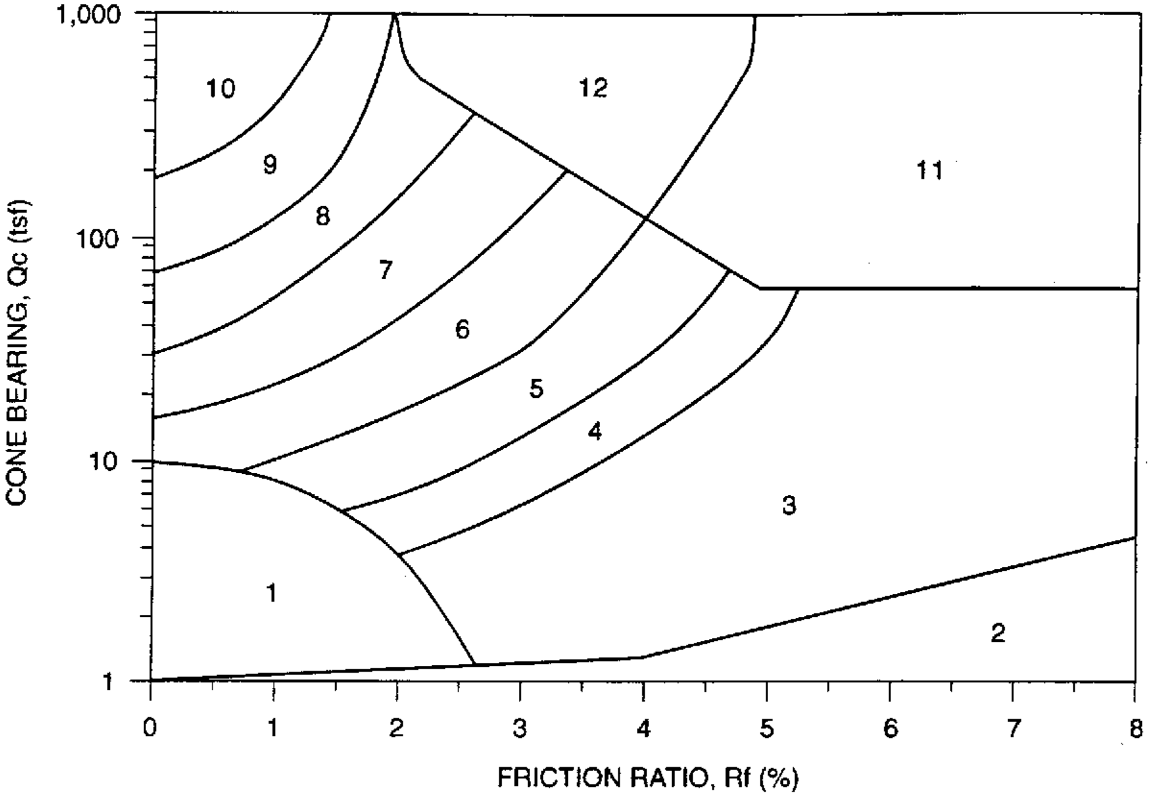
Elevation: 14.7 feet, Emeryville City datum.

EMERYSTATION NO. 2
Emeryville, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 8/20/99	Project No. 2254.04	Figure A-15

Treadwell&Rollo



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 x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud Qc ≤9).
Estimated from local experience (fine-grained soils Qc > 9).

EMERYSTATION NO. 2
Emeryville, California

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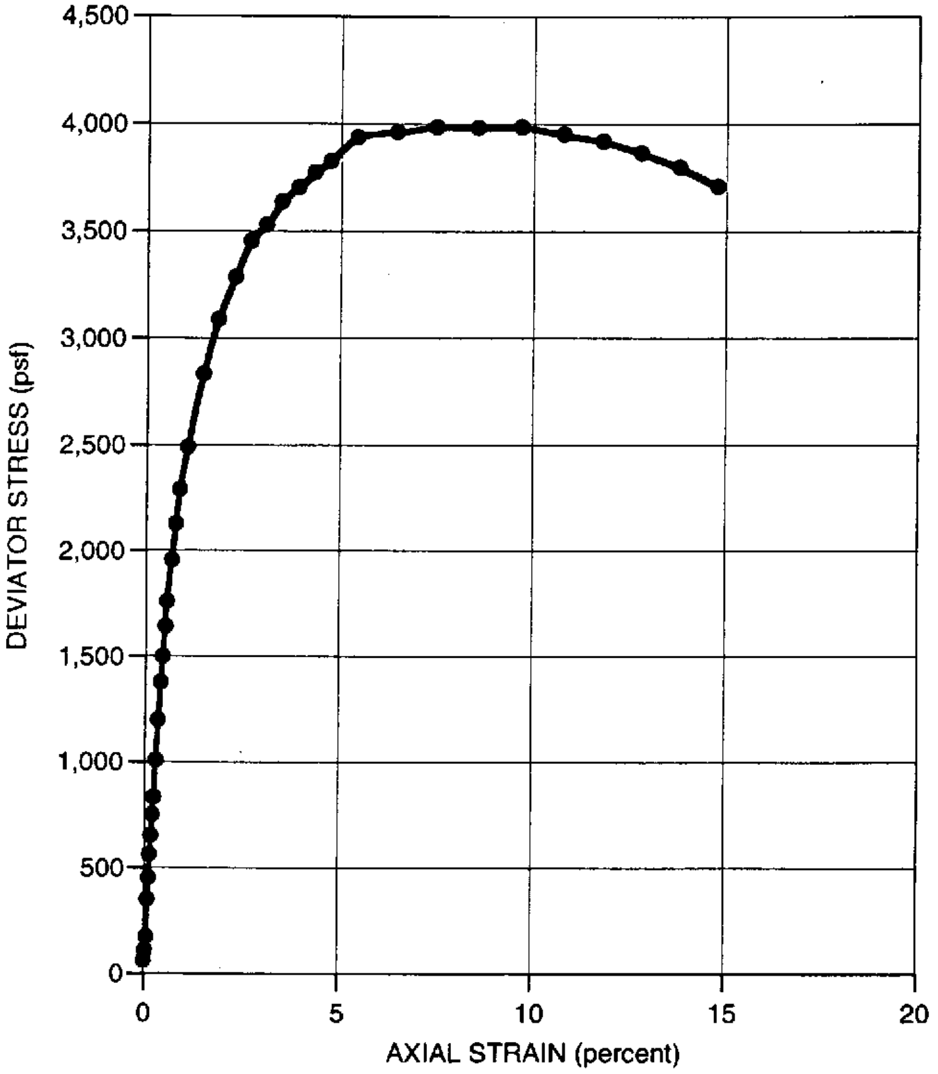
CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 9/1/99	Project No. 2254.04	Figure A-16



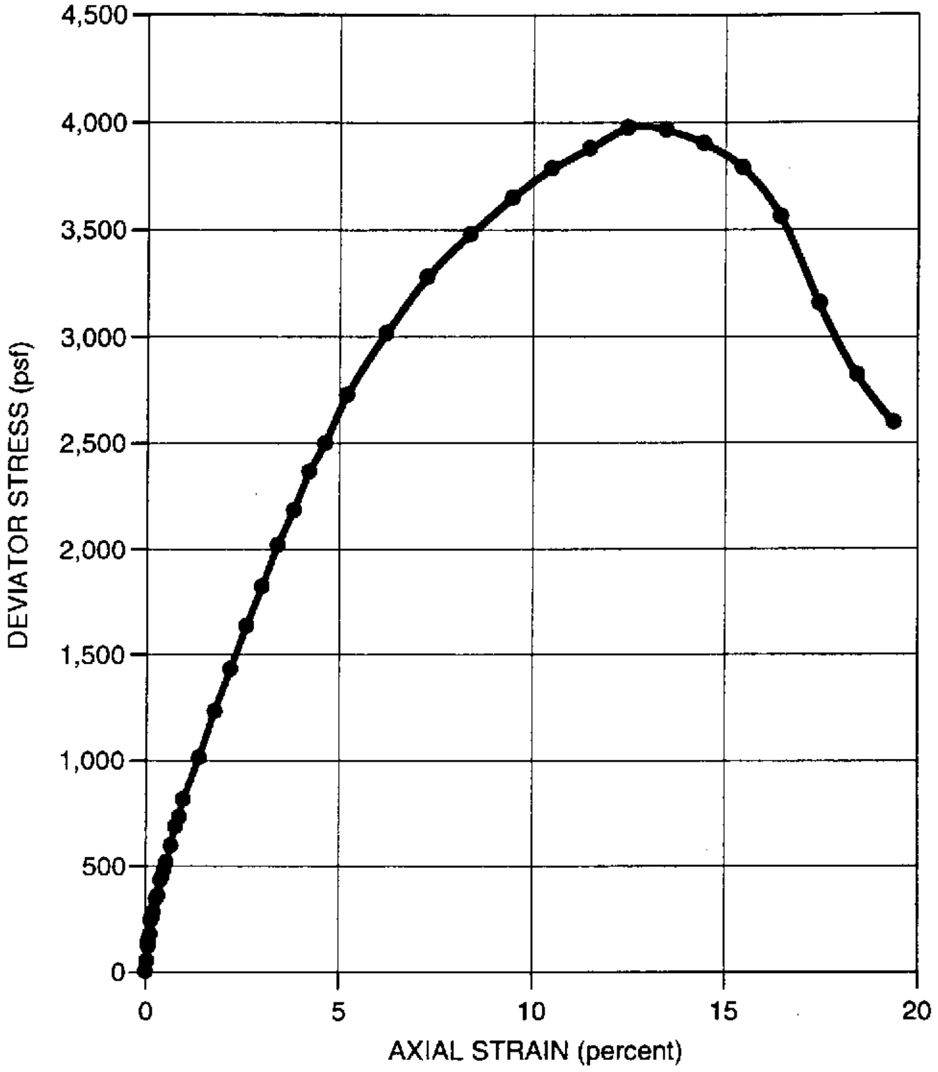
SPECIMEN TYPE Undisturbed		SHEAR STRENGTH	1991 psf
DIAMETER (in) 2.437	HEIGHT (in) 5.34	STRAIN AT FAILURE	7.5 %
MOISTURE CONTENT	19.9 %	CONFINING PRESSURE	1,000 psf
DRY DENSITY	108 pcf	STRAIN RATE	0.56 % /min
DESCRIPTION CLAY with SAND (CL)		SOURCE B-1 at 10 feet	

EMERYSTATION NO. 2
Emeryville, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 9/9/99 | Project No. 2254.04 | Figure B-1



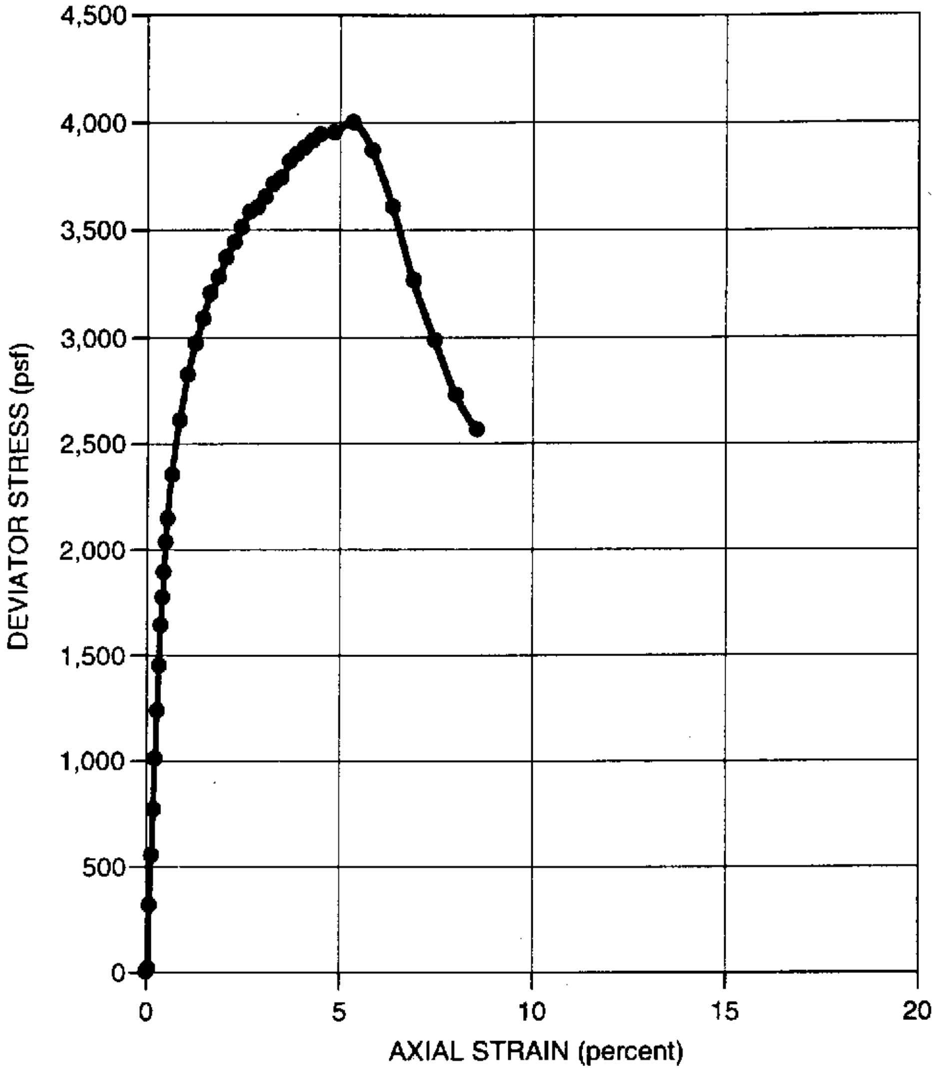
SPECIMEN TYPE Undisturbed		SHEAR STRENGTH	1,986 psf
DIAMETER (in) 2.438	HEIGHT (in) 5.38	STRAIN AT FAILURE	12.5 %
MOISTURE CONTENT	26.7 %	CONFINING PRESSURE	3,000 psf
DRY DENSITY	98 pcf	STRAIN RATE	0.56 % /min
DESCRIPTION CLAY (CL)		SOURCE B-2 at 30 feet	

EMERYSTATION NO. 2
Emeryville, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 9/1/99 | Project No. 2254.04 | Figure B-2



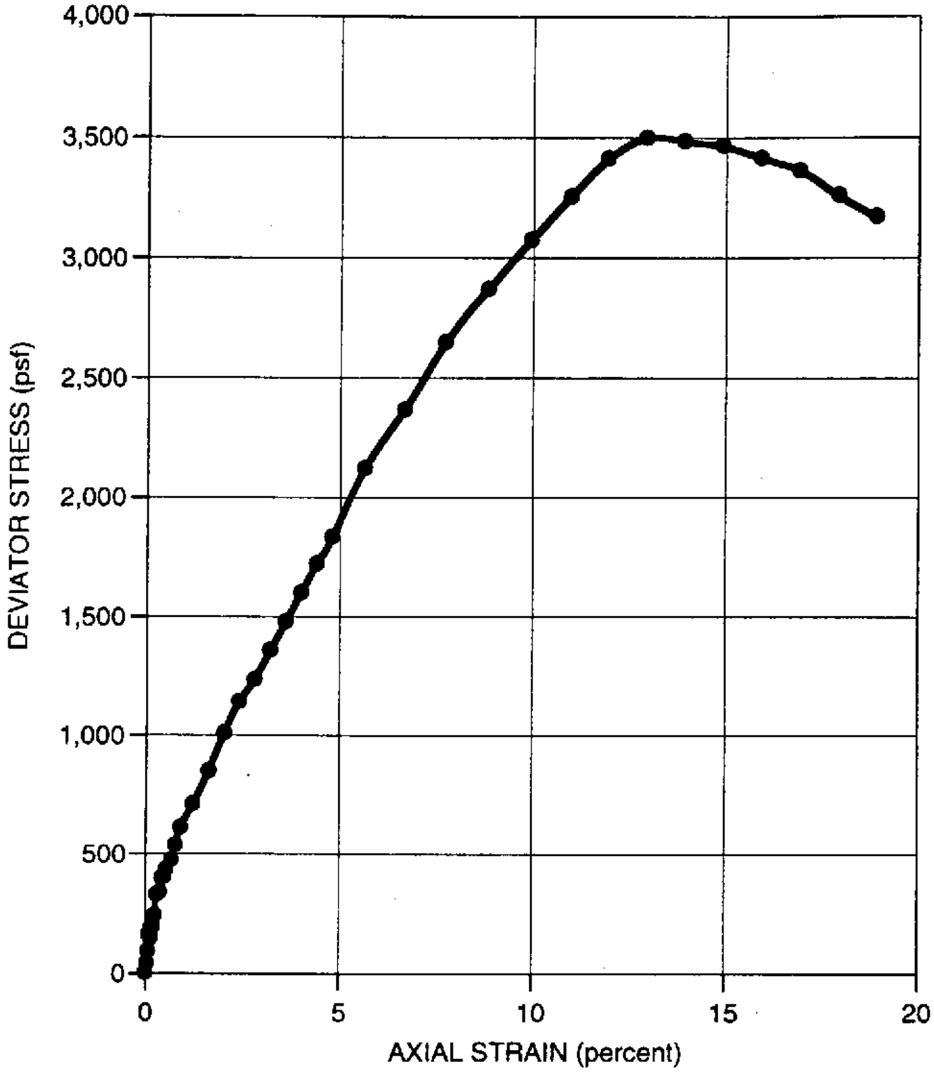
SPECIMEN TYPE Undisturbed		SHEAR STRENGTH	2,000 psf
DIAMETER (in) 2.871	HEIGHT (in) 5.98	STRAIN AT FAILURE	5.3 %
MOISTURE CONTENT	21.2 %	CONFINING PRESSURE	5,500 psf
DRY DENSITY	102 pcf	STRAIN RATE	0.67 % /min
DESCRIPTION SANDY CLAY (CL)		SOURCE B-2 at 55 feet	

EMERYSTATION NO. 2
Emeryville, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 9/23/99 | Project No. 2254.04 | Figure B-3



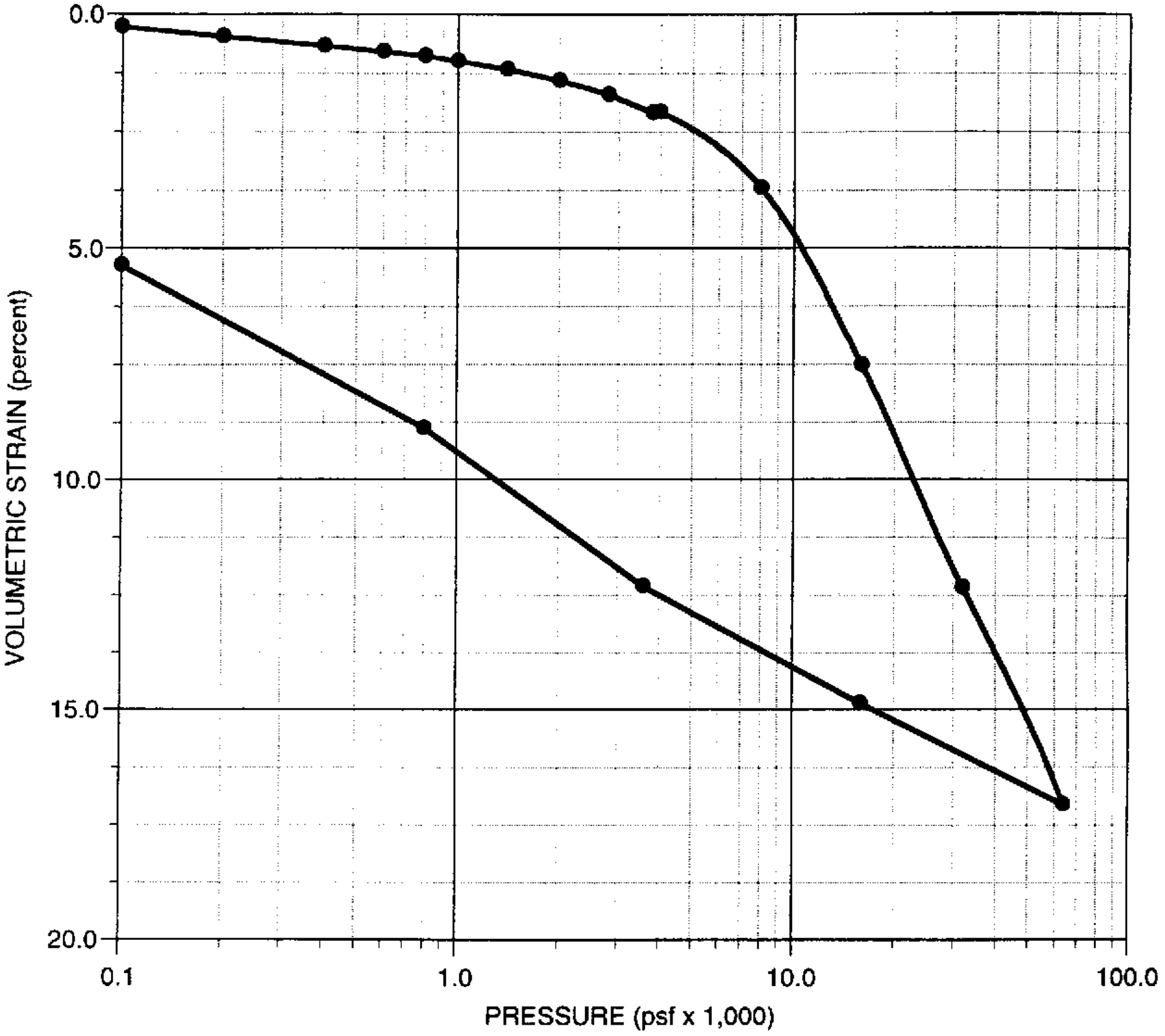
SPECIMEN TYPE Undisturbed		SHEAR STRENGTH	1,748 psf
DIAMETER (in) 2.438	HEIGHT (in) 5.00	STRAIN AT FAILURE	12.9 %
MOISTURE CONTENT	22.2 %	CONFINING PRESSURE	7,000 psf
DRY DENSITY	105 pcf	STRAIN RATE	0.60 % /min
DESCRIPTION CLAY (CL)		SOURCE B-3 at 70 feet	

EMERYSTATION NO. 2
Emeryville, California

Treadwell&Rollo

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 9/23/99 | Project No. 2254.04 | Figure B-4



Type of specimen Undisturbed	Condition		Before test	After test	
Diameter (in) 2.416 Height (in) 1.0	Water Content	w_o	28.7 %	w_f	26.3 %
Overburden Pressure, P_o 1,600 psf	Void Ratio	e_o	0.811	e_f	0.716
Preconsol. Pressure, P_c 8,900 psf	Saturation	S_o	97.2 %	S_f	100 %
Compression Ratio, C_{ec} 0.16	Dry Density	γ_d	95 pcf	γ_d	100 pcf

LL --	PL --	PI --	G_s 2.75

Classification CLAY (CL)

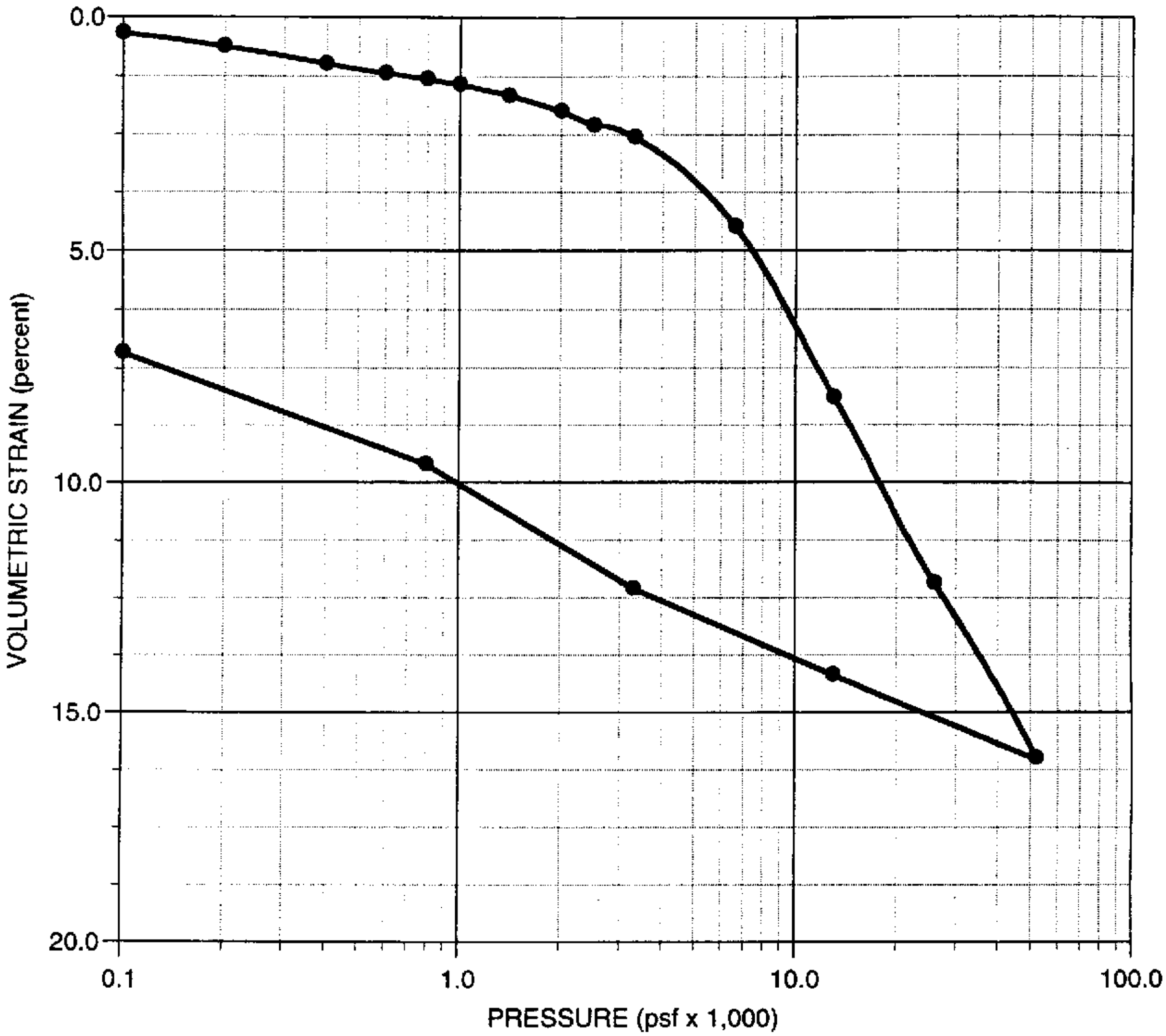
Source B-1 at 15 feet

EMERYSTATION NO. 2
Emeryville, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 9/9/99 | Project No. 2254.04 | Figure B-5



Type of specimen Undisturbed		Condition		Before test		After test
Diameter (in) 2.416	Height (in) 1.0	Water Content	w_o	25.1 %	w_f	22.7 %
Overburden Pressure, P_o	1,000 psf	Void Ratio	e_o	0.752	e_f	0.634
Preconsol. Pressure, P_c	5,900 psf	Saturation	S_o	91.6 %	S_f	99.4 %
Compression Ratio, C_{ec}	0.13	Dry Density	γ_d	98 pcf	γ_d	105 pcf

LL	PL	PI	G_s 2.75

Classification SANDY CLAY (CL)

Source B-3 at 11 feet

EMERYSTATION NO. 2
Emeryville, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 9/9/99 | Project No. 2254.04 | Figure B-6

California State Certified Laboratory No.2153

CERCO
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

September 9, 1999

Job No.9909013
Cust.No.10727

Mr. Christian Divis
Treadwell & Rollo
555 Montgomery St., Ste 1300
San Francisco, CA 94111

SUBJECT: Project No.: 2254.042
Project Name: Emerystation No.2
Corrosivity Analysis - ASTM Test Methods

Dear Mr. Divis:

In accordance with your request, we have analyzed the soil sample furnished by your office and have evaluated it for corrosivity using ASTM Test Methods. The results are enclosed.

Based upon the conductivity measurement, this sample is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron fire water pipelines should be protected against corrosion.

The chloride ion concentration is 9 mg/kg. Because the chloride ion concentration is less than 300 ppm, it is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 350 mg/kg and is determined to be sufficient to damage reinforced concrete structures and a cement mortar coating at this location. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, in accordance with the Uniform Building Code requirements.

The pH of the soil is 7.4 which does not present corrosion problems for buried iron, steel, mortar coated steel and reinforced concrete structures.

The redox potential is 240-mV which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

The information provided in this report is general in nature. For specific design recommendations, please call for consultation.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require additional information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr

J. Darby Howard, Jr., P.E.
President

JDH/jdl

CERCO Analytical, Inc.

3942-A Valley Avenue, Pleasanton, CA 94566-4715 (925) 462-2771 Fax (925) 462-2775

FINAL RESULTS

Client: Treadwell & Rollo

Client's Project No.: 2254.04

Client's Project Name: Emerystation No.2

Authorization: Transmittal Dated 09/02/1999

Date Sampled: Not Indicated

Date Received: 3-Sep-1999

Date of Report: 9-Sep-1999

Job/Sample No.	Sample I.D.	Matrix	Redox (mV)	pH	Sulfate (mg/kg)*	Conductivity (umhos/cm)*	Sulfide (mg/kg)*	Chloride (mg/kg)*
9909013-001	B-3 @ 5'	Soil	240	7.4	350	1,300	-	9

Method:	ASTM D1498	ASTM D4972	ASTM D4327	ASTM D1125Mod	ASTM D4658Mod	ASTM D4327
Detection Limit:	-	-	25	10	5	5
Date Analyzed:	7-Sep-1999	9-Sep-1999	8-Sep-1999	9-Sep-1999	-	9-Sep-1999

* Results Reported on Wet Weight Basis

Cheryl McMille

Cheryl McMillen

Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Entech Analytical Labs, Inc. CA ELAP# I-2346

525 Del Rey Avenue, Suite E • Sunnyvale, CA 94086 • (408) 735-1550 • Fax (408) 735-1554

Treadwell & Rollo
555 Montgomery Street, Suite 1300
San Francisco, CA 94111
Attn: Christian Divis

Date: 8/5/99
Date Received: 7/29/99
Project: 2254.04
PO #:
Sampled By: Client

Certified Analytical Report

Soil Sample Analysis: (All results in mg/kg)

Sample ID	B-1 (#1,2,3,4)			B-2 (#1,2,3,4)			B-3 (#1,2,3,4)				
Sample Date	7/26/99			7/27/99			7/28/99				
Sample Time											
Lab #	15526-005			15526-010			15526-015				
	Result	DF	DLR	Result	DF	DLR	Result	DF	DLR	PQL	Method
Extraction	TTLC			TTLC			TTLC				3050
Analysis Date	8/3/99			8/3/99			8/3/99				
Lead	**6.2**	1.0	5.0	**ND**	1.0	5.0	**7.3**	1.0	5.0	5.0	6010
Analysis Date	7/30/99			7/30/99			7/30/99				
TPH-Diesel	**ND**	1.0	1.0	**2.6** ^{x}	1.0	1.0	**1.9** ^{x}	1.0	1.0	1.0	8015M
Analysis Date	7/30/99			7/30/99			7/30/99				
TPH-Gas	**ND**	1.0	1.0	**1.3** ^{x}	1.0	1.0	**ND**	1.0	1.0	1.0	8015M
MTBE	**ND**	1.0	0.05	**ND**	1.0	0.05	**ND**	1.0	0.05	0.05	8020
Benzene	**ND**	1.0	0.005	**ND**	1.0	0.005	**ND**	1.0	0.005	0.005	8020
Toluene	**ND**	1.0	0.005	**ND**	1.0	0.005	**ND**	1.0	0.005	0.005	8020
Ethyl Benzene	**ND**	1.0	0.005	**ND**	1.0	0.005	**ND**	1.0	0.005	0.005	8020
Xylenes (total)	**ND**	1.0	0.005	**ND**	1.0	0.005	**ND**	1.0	0.005	0.005	8020

DF=Dilution Factor ND= None Detected above DLR PQL=Practical Quantitation Limit DLR=Detection Reporting Limit

Analysis performed by Entech Analytical Labs, Inc. (CA ELAP #I-2346)

Michelle L. Anderson, Lab Director

Entech Analytical Labs, Inc. **CA ELAP# I-2346**

525 Del Rey Avenue, Suite E • Sunnyvale, CA 94086 • (408) 735-1550 • Fax (408) 735-1554

STANDARD LAB QUALIFIERS
July, 1998

All Entech lab reports now reference standard lab qualifiers. These qualifiers are noted in the adjacent column to the analytical result and are adapted from the U.S. EPA CLP program. The current qualifier list is as follows:

Qualifier	**Description**
U	Compound was analyzed for but not detected
J	Estimated valued for tentatively identified compounds or if result is below PQL but above MDL
N	Presumptive evidence of a compound (for Tentatively Identified Compounds)
B	Analyte is found in the associated Method Blank
E	Compounds whose concentrations exceed the upper level of the calibration range
D	Multiple dilutions reported for analysis; discrepancies between analytes may be due to dilution
X	Results within quantitation range; chromatographic pattern not typical of fuel

Entech Analytical Labs, Inc.

525 Del Rey Avenue, Suite E
Sunnyvale, CA 94086

QUALITY CONTROL RESULTS SUMMARY

Laboratory Control Spikes
METHOD: EPA 6010

QC Batch #: SM990729
Matrix: Solid
Units: mg/kg

Date Analyzed: 07/28/99
Date Digested: 07/28/99
Digestion Method: EPA 3050
Spiked Sample: Blank Spike

PARAMETER	Method #	MB	SA	SR	SP	**SP**	SPD	**SPD**	RPD	QC LIMITS	
		mg/kg	mg/kg	mg/kg	mg/kg	**%R**	mg/kg	**%R**		RPD	%R
Antimony	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Arsenic	6010	<1.0	50.	0.0	44.	**89**	46.	**92**	3.8	25.0	75-125
Barium	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Beryllium	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Cadmium	6010	<1.0	50.	0.0	42.	**85**	43.	**85**	0.4	25.0	75-125
Chromium	6010	<1.0	50.	0.0	46.	**92**	45.	**91**	0.8	25.0	75-125
Cobalt	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Copper	6010	<1.0	50.	0.0	45.	**90**	44.	**89**	1.0	25.0	75-125
Lead	6010	<1.0	50.	0.0	46.	**91**	44.	**89**	3.0	25.0	75-125
Molybdenum	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Nickel	6010	<1.0	50.	0.0	46.	**91**	46.	**91**	0.4	25.0	75-125
Selenium	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Silver	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Thallium	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Vanadium	6010	<1.0	50.	na	na	na	na	na	na	25.0	75-125
Zinc	6010	<1.0	50.	0.0	43.	**86**	43.	**86**	0.3	25.0	75-125

Definition of Terms:

na: Not Analyzed in QC batch
MB: Method Blank
SA: Spike Added
SR: Sample Result
SP: Spike Result
SP (%R): Spike % Recovery
SPD: Spike Duplicate Result
SPD (%R): Spike Duplicate % Recovery

Entech Analytical Labs, Inc.

525 Del Rey Avenue, Suite E
Sunnyvale, CA 94086

QUALITY CONTROL RESULTS SUMMARY

METHOD: Gas Chromatography
Laboratory Control Sample

QC Batch #: GBG4990730
Matrix: Soil
Units: µg/kg

Date Analyzed: 07/30/99
Quality Control Sample: Blank Spike

PARAMETER	Method #	MB µg/kg	SA µg/kg	SR µg/kg	SP µg/kg	**SP % R**	SPD µg/kg	**SPD %R**	% RPD	QC LIMITS RPD	%R
Benzene	8020	<5.0	80	ND	74	**92**	76	**95**	2.6	25	75-125
Toluene	8020	<5.0	80	ND	75	**92**	77	**97**	5.1	25	75-125
Ethyl Benzene	8020	<5.0	80	ND	75	**94**	77	**96**	1.7	25	75-125
Xylenes	8020	<5.0	240	ND	224	**93**	227	**94**	1.1	25	75-125
Gasoline	8015	<1000	1000	ND	1160	**116**	1070	**107**	8.1	25	75-125
aaa-TFT(S.S.)-PID	8020			86%	90%		91%				65-135
aaa-TFT(S.S.)-FID	8015			100%	99%		94%				65-135

Definition of Terms:

na: Not Analyzed in QC batch
MB: Method Blank
SA: Spike Added
SR: Sample Result
RPD(%): Duplicate Analysis - Relative Percent Difference
SP: Spike Result
SP (%R): Spike % Recovery
SPD: Spike Duplicate Result
SPD (%R): Spike % Recovery
NC: Not Calculated

QUALITY CONTROL RESULTS SUMMARY

Laboratory Control Spikes

QC Batch #: DS990715
Matrix: Soil
Units: mg/Kg

Date analyzed: 07/30/99
Date extracted: 07/30/99
Quality Control Sample: Blank Spike

PARAMETER	Method #	MB mg/Kg	SA mg/Kg	SR mg/Kg	SP mg/Kg	**SP %R**	SPD mg/Kg	**SPD %R**	RPD	QC LIMITS RPD	QC LIMITS %R
Diesel	8015M	<1.0	25	ND	18	74	18	73	0.6	25	44-119
Hexocosane				100%	100%		98%				65-135

Definition of Terms:

MB: Method Blank
na: Not Analyzed in QC batch
SA: Spike Added
SR: Sample Result
RPD(%): Duplicate Analysis - Relative Percent Difference
SP: Spike Result
SP (%R): Spike % Recovery
SPD: Spike Duplicate Result
SPD (%R): Spike Duplicate % Recovery
NC: Not Calculated

Treadwell&Rollo
555 Montgomery Street, Suite 1300
San Francisco, California
(415) 955-9040
(415) 955-9041 Fax

CHAIN OF CUSTODY RECORD

Project No. 2254-04 Project Name Emergstation No. 2 Date 28 July 1999 Page 1 of 1

15526

Date	Sample Number	Analysis					Number of Containers	Sample Information
		TPH gas/Diesel	BTEX/MTBE	Total Lead				
7/26/99	B-1 #1					-001	1	Please Composite
7/26/99	B-1 #2					-002	1	into 1 Soil Sample
7/26/99	B-1 #3					-003	1	-005
7/26/99	B-1 #4					-004	1	
7/27/99	B-2 #1					-006	1	
7/27/99	B-2 #2					-007	1	Please Composite
7/27/99	B-2 #3					-008	1	into 1 Soil Sample
7/27/99	B-2 #4					-009	1	-010
7/28/99	B-3 #1					-011	1	
7/28/99	B-3 #2					-012	1	Please Composite
7/28/99	B-3 #3					-013	1	into 1 Soil Sample
7/28/99	B-3 #4					-014	1	-015
						Total Number of Containers	12	

Relinquished by (Sampler):

Signature C. Divis

Printed Name C. DIVIS

Company

Date 7/29 Time 12:05

Received by: Babak Minasazi 811

Signature

Printed Name

Company World courier

Date 07-29-99 Time 12:10

Relinquished by:

Signature

Printed Name Babak Minasazi

Company World courier

Date 7-29-99 Time 1:30

Method of Shipment

Received by (Lab):

Signature ngtrazo

Printed Name V. TRAZO

Lab ENTECH

Date 7/29/99 Time 1:30pm

Lab Comments

report to:
Christian Divis

Remarks: Normal Turnaround Time

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QUALITY CONTROL REVIEWER

Richard D. Rodgers
Geotechnical Engineer