GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

Geotechnical Report
Potrero Power Plant
San Francisco, California

June 2004

Prepared for:

CH2M HILL

SF03015

3004 16th Street, Suite 204 • San Francisco, CA 94103 • (415) 565-7366

TABLE OF CONTENTS

INTRODUCTION

GENERAL STATEMENT

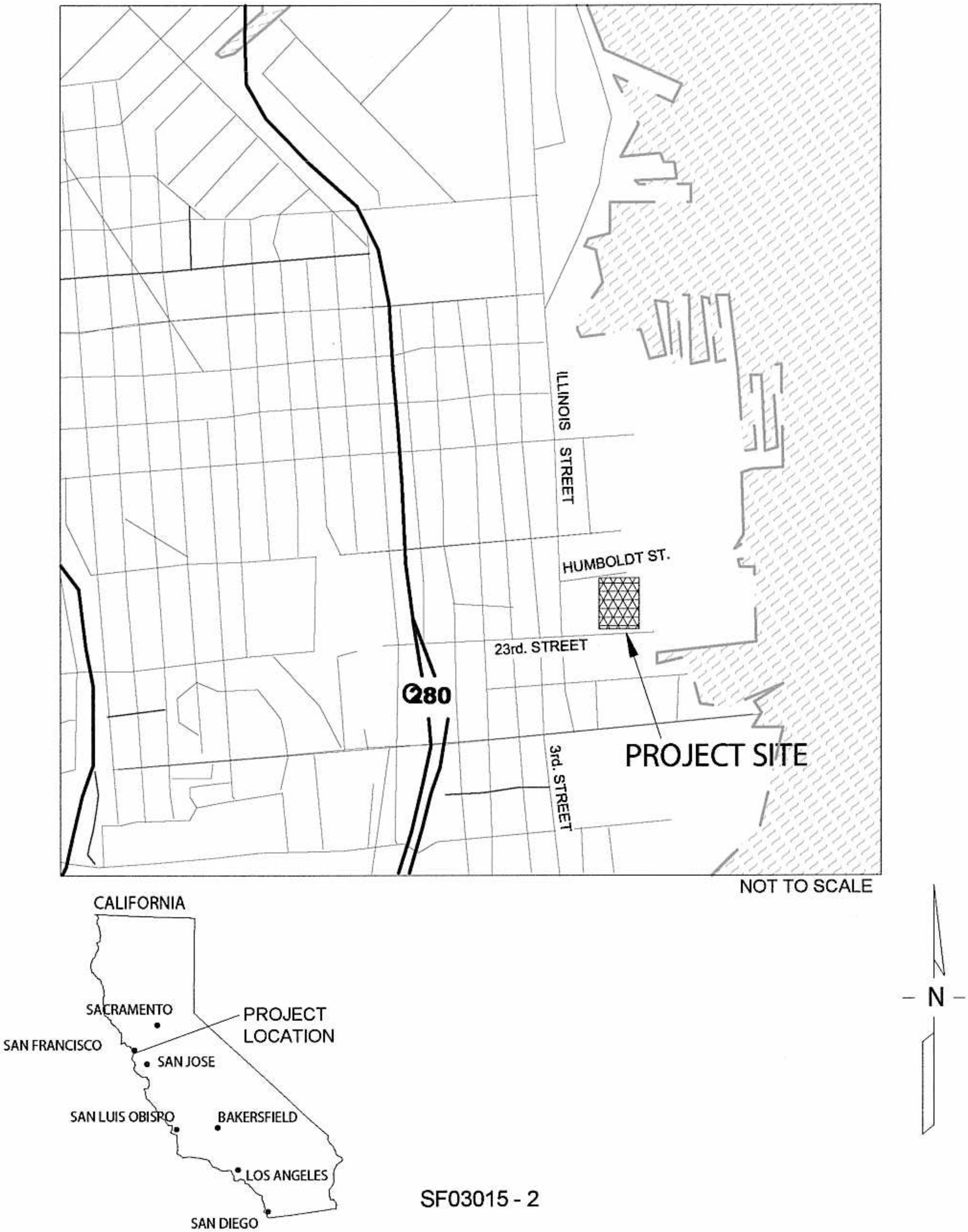
This geotechnical report presents the findings, conclusions, and recommendations of our geotechnical evaluation performed at the request of CH2M HILL for a portion of the Potrero Power Plant. The project site is located at 1201A Illinois Street, near the intersection of 23rd Street, in San Francisco. It is comprised of the 4.5-acre portion within the Potrero Power Plant site referred to as SF ERP (San Francisco Electrical Reliability Project), and is bound by Humboldt Street to the north and 23rd Street to the south. The site consists of a number of abandoned buildings and structures that were part of the former PG&E power plant. The Station "A" building borders the eastern edge of the site and the gas holder tank foundation is near the western edge of the site.

The facility is owned by Mirant Corporation and was formerly owned by PG&E. The Hetch Hetchy Water and Power, City and County of San Francisco Public Utilities Commission (SFPUC), is currently involved in a due diligence effort to purchase the property from Mirant. This geotechnical report was prepared for CH2M HILL as part of this effort, and was based on a proposed schematic layout prepared by PB Power. Because much of the site is inaccessible due to the presence of existing structures and foundations and because layouts are not finalized, additional geotechnical investigation may be required for final design after structural schemes and foundation loadings are finalized, and after the proposed demolition of the structures on site. This report does not include environmental site characterization, hazardous materials testing, or other environmental services.

PROPOSED DEVELOPMENT

The proposed power plant project entails the construction of three combustible turbine generator units and numerous appurtenant structures for the SFPUC Electrical Reliability Project. A plan of the proposed layout of structures is shown on Plate 1 – Electrical Reliability Project, Plot Plan, 3 Units Simple Cycle. Major appurtenant structures include an electrical building, switchyard structure, chiller/cooler tower package, shop/warehouse, water treatment facility, fuel gas compressors, and storage tanks.

FIGURE 1
LOCATION MAP



SF03015 - 2

To accommodate the new power plant, numerous existing abandoned buildings, structures, concrete slabs/footings, and utilities within the project site boundary will require demolition and removal from the site. Final site ground surface elevations, and the quantity of soil removal and/or fill to achieve final grades, are not known to us at this time.

PREVIOUS REPORTS

To assist us in our investigation, we reviewed logs and figures from previous studies performed in the site vicinity by Camp Dresser and McKee (October 1997), Fluor Daniel GTI (December 1997), Fluor Daniel GTI (June 1998), and Geomatrix Consultants (April 2000).

WORK PERFORMED

The scope of work was developed through discussions with John Carrier, Tom Lae, and Sarah Madams of CH2M Hill, and Steve Brock of PB Power, and a site meeting on November 19, 2003.

We performed the following work for this geotechnical evaluation:

1. **Exploratory Drilling.** Explored subsurface conditions by means of drilling eight hollow stem auger borings, B-1 though B-8. We drilled exploratory borings B-1, B-2, B-5, and B-8 on 2/24/04; and B-3, B-4, B-6 and B-7 on 2/25/04. Boring B-5 was drilled to a depth of 5 feet before hitting refusal from what may have been a concrete slab. The following table shows the depths of the borings.

TABLE 1 – BORING DEPTHS

Boring	Depth (feet)
B-1	14.5
B-2	18.5
B-3	30.5
B-4	35.5
B-5	5.0
B-6	20.5
B-7	25.5
B-8	20.5

We visually classified the soil and performed Standard Penetration Tests (SPT) at select depths during drilling, and recovered split-spoon SPT samples and relatively undisturbed 2-½ inch diameter sleeve samples using a split-barrel sampler for testing in the laboratory. The boring locations are shown on Plate 2 – Site Plan. Boring logs and laboratory test data are presented in Appendix A - Supporting Geotechnical Data. All surface elevations referred to in this report are with respect to the San Francisco City Datum (SFCD).

2. **Laboratory Testing.** Performed laboratory tests, including moisture, density, grain size distributions, Atterberg limits, direct shear, and corrosion on selected soil samples to measure pertinent index and engineering properties. Details of the laboratory testing program test results are presented in Appendix A.

3. **Engineering Analysis.** Analyzed findings to develop geotechnical recommendations for seismic criteria, earthwork, foundations, lateral earth pressures, and corrosion.

4. **Report.** Prepared this report presenting our geotechnical findings, conclusions, and recommendations for the design and construction of the proposed project.

FINDINGS

SITE CONDITIONS

The project site is a 4.5-acre portion within the site of the Potrero Power Plant, and is located at 1201A Illinois Street, in San Francisco. Structures on site include several vacant and dilapidated masonry buildings; some of which were formerly utilized as a compressor building, a meter station building, a station "A" building, and a machine shop. Concrete pads exist throughout the site that appear to have been foundations of former buildings. Other subsurface foundations or footings may exist, as indicated by concrete obstructions encountered in two borings in the northern area of the site. Open faced walls, retaining walls, and walls of former buildings exist along the northern boundary of the site. A concrete tank foundation of unknown thickness, approximately 200 feet in diameter, is situated near the western edge of the site. The majority of the open areas and roadways on the site are paved with asphalt, with the exception of the previously noted concrete pads and the tank foundation.

The area of the site west of Station "A" building gently slopes from north to south with elevations decreasing from approximately 31 feet near Humboldt Street to approximately 23 feet near 23rd Street. Along this grade transition exists an approximate five-foot vertical drop formed by the existing 200-foot diameter tank ring wall. The area east of Station "A" building also slopes gently from north to south with elevations ranging from approximately 25 to 23 feet. The tank foundation is at approximately elevation 24 feet. All elevations herein were estimated based on a figure provided by CH2M HILL showing spot elevations.

Vegetation is sporadically present throughout the site, but is predominate in the areas just south and east of the tank foundation. The vegetation consists mostly of fennel stalks and grasses.

Numerous underground utilities exist on the site, most of which are assumed to be abandoned. Utilities noted during location of our borings include electrical conduits and storm drains. No major overhead lines were observed.

GEOLOGY AND SEISMIC SETTING

Local Geology. The project site is located on the eastern flank of the Potrero Hill rock mass, on the west margin of the San Francisco Bay. The project site and vicinity is underlain by serpentine rock (serpentinite) of the Franciscan Complex (KJsp) and artificial fill used to displace and build-out the pre existing shoreline of the San Francisco Bay. The Franciscan Complex is composed of a variety of rock types that occur as large to small discrete blocks or mixed in a tectonic mélange with sheared serpentinite and/or shale as the matrix between blocks. The Franciscan Complex makes up the bedrock beneath the site and extends to depths greater than 1,000 feet. Serpentinite that has been exposed at the surface for hundreds of years is typically soft and weathered within a few feet of the ground surface, but becomes very dense and hard with depth or when the weathered upper portion has been graded off. Franciscan serpentinite is typically closely sheared and fractured. The artificial fill along the southern boundary of the site is underlain by either Franciscan bedrock or interfingered layers of Younger

Bay Mud (Qybm) and Bayside Sand Deposits (Qa). The sand encountered is tentatively correlated with older beach or dune deposits found locally along the eastern margins of San Francisco peninsula. These undifferentiated Quaternary deposits are locally interbedded with or overlie the Younger Bay Mud. The Qa deposits mapped by the California Geological Survey in this region consist of late Pleistocene alluvial, estuarine and eolian deposits (CGS, San Francisco Quad Seismic Hazard Map, 2003).

Seismicity. The San Francisco Bay Area contains several active faults that could cause strong ground shaking at the project site. The San Andreas is the nearest active fault and is located approximately 8.3 miles west of the project site. The San Andreas is the primary component in a complex system of right-lateral, strike-slip faults; including the San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults; collectively known as the San Andreas fault system. The San Andreas, San Gregorio-Seal Cove, Hayward, and Calaveras faults have produced measurable historic ground motion and movement. The peninsular segment of the San Andreas fault is capable of producing an earthquake of an estimated maximum earthquake magnitude of 8.1. This segment is estimated to have recurrence intervals on the order of 200 years. A summary of nearby faults is presented in Table 2 - Active and Potentially Active Faults.

TABLE 2 - ACTIVE AND POTENTIALLY ACTIVE FAULTS

Fault	Distance (miles)	Estimated Maximum Earthquake Magnitude	Historic Earthquakes	
			Year	Magnitude
San Andreas (1906)	8.3	8.1	1838 1898 1906 1989	6.8 6.2 8.1 7.1
Hayward	10.7	7.1	1868	6.8
San Gregorio-Seal Cove	11.7	7.3	NA	NA
Calaveras	20.6	6.8	1861 1979 1984	5.3 5.9 6.1

EARTH MATERIALS

The project site can be divided into two areas that fall under two general characteristic subsurface profiles, one comprising shallow bedrock and the other deep fill. These two areas shall be referred to throughout this report as the (1) Shallow Bedrock Area, and (2) Deep Fill Area. The Shallow Bedrock Area and Deep Fill Area of the project site are shown on Plate 3 – Map of Characteristic Subsurface Profiles. The delineation of the Shallow Bedrock Area and Deep Fill Area was based on the findings in our borings as well as review of boring logs and data prepared by others (GTI, 1998 and Geomatrix, 2000). The Shallow Bedrock Area generally consists of one to seven feet of medium dense granular fill underlain by Franciscan Complex serpentine bedrock. The Deep Fill Area generally consists of seven to twenty-five feet of loose to medium dense granular fill underlain by either Franciscan Complex bedrock or inter-fingered marine and bayside deposits. Corresponding with the pre-existing shoreline of the San Francisco Bay, the depth of fill increases significantly as it trends southward from the approximate center of the site.

Geotechnical cross-sections, which further illustrate the Shallow Bedrock Area and Deep Fill Area subsurface stratigraphies, are shown on Plate 4 – Geotechnical Cross Sections. Locations of these cross sections are shown on Plate 3.

Artificial Fill (Qaf). Artificial fill was encountered in all of our borings. In the Shallow Bedrock Area the fill ranged from two to seven feet deep. The shallowest fill exists along the northern end of the site, and gradually increases trending southward. The fill in the Shallow Bedrock Area, as indicated by borings B-1, B-2, and B-5 through B-8, generally consists of granular soils presumably placed as aggregate base for the existing asphalt pavement. The granular soils range from poorly graded sands, silty sands, and sandy-gravelly clay. Various debris including concrete, and crushed brick fragments were encountered in the fill. Some of the fill may also be derived from local serpentinite bedrock placed during original site grading. The fill in the Shallow Bedrock Area was underlain by weathered rock of the Franciscan Complex.

Artificial fill in the Deep Fill Area, as indicated by borings B-3 and B-4 and review of pre-existing data (GTI, 1998), ranges from approximately 10 to 25 feet deep, and is underlain by serpentine bedrock, Younger Bay Mud, or Bayside

Sand deposits. The fill consisted of primarily 10 to 15 feet of medium dense poorly graded silty sand underlain by 10 feet of very loose clayey sand. Various debris including ash, slag, charcoal, and crushed brick fragments were encountered in the deep fill. Also, a one-foot layer of very loose disintegrated concrete with oily staining and hydrocarbon odor was encountered at a depth of 21 feet in B-4. Franciscan Complex bedrock underlies the fill in boring B-4 at a depth of 22 feet. In boring B-3, 23 feet of fill was underlain by Younger Bay Mud and Bayside Sand deposits. Franciscan bedrock was not encountered in boring B-3, which was drilled to a total depth of 36.5 feet.

Younger Bay Mud. (Qybm). A five-foot thick layer of Younger Bay Mud comprising very soft, wet, dark greenish-gray, low plasticity clay with sand was encountered in boring B-3 at a depth of 24 feet. The bay mud contained a strong sulfur odor.

Dune or Bayside Sand. (Qa). A deposit comprising dense to very dense poorly graded sand was encountered in boring B-3 at a depth of 29 feet, and extended to the bottom of B-3 at depth of 36.5 feet. The sand encountered is tentatively correlated with older beach or dune deposits found locally along the eastern margins of San Francisco peninsula. These undifferentiated Quaternary deposits are locally interbedded with or overlie the Younger Bay Mud.

Franciscan Complex Serpentinite (KJsp). Artificial fill in all borings except B-3 were underlain by serpentine rock of the Franciscan Complex that extended to the maximum depth of our exploration and constitutes the bedrock for the site. The formation encountered in our borings consisted of greenish gray to black serpentinite. The serpentinite was typically moderately to highly weathered in the upper five feet, becoming less weathered and very dense with depth.

Summary of Soil Parameters. Soil parameters obtained from laboratory testing, such as cohesion, friction angle, density, and water content, are summarized in Table 3 – Summary of Soil Parameters. Parameters used to develop our recommendations are also shown.

TABLE 3 – SUMMARY OF SOIL PARAMETERS

Boring Number	Depth (feet)	Material Type	Cohesion (psf)	Friction Angle (degrees)	Dry Density (pcf)	Water Content (%)	Total Unit Weight (pcf)	Plastic Limit (%)	Liquid Limit (%)
B-1	2.5	Serpentinite	-	-	119	10.5	131.5	-	-
B-2	8.0	Decomposed Serpentinite	-	-	-	-	-	20	47
	18.0	Serpentinite	-	-	83	32.5	110	-	-
B-3	31.0	Bayside Sand	-	-	106	22.4	-	-	-
B-4	3.0	Fill (Silty Sand)	430	34	-	-	-	-	-
B-6	10.0	Weathered Serpentinite	-	-	86	41.4	-	-	-
	20.0	Serpentinite	-	-	107	22		-	-
B-7	3.5	Fill (Gravelly Sand)	375	35	-	-	-	-	-

GROUNDWATER

Static groundwater was encountered only within the deep fill borings B-3 and B-4, at depths of 22 feet and 13 feet, respectively. These measured depths imply a south-trending hydraulic gradient from the bedrock towards bayside deposits. Groundwater was not encountered in borings drilled in the Shallow Bedrock Area. However, as some serpentinite samples were classified as moist to wet, groundwater is likely confined within fractured zones in the serpentinite bedrock, and would not have accumulated rapidly enough in our borings to be observed. Groundwater measurements by others (Geomatrix, 2000) confirm this finding and indicate a piezometric groundwater elevation of approximately +21 feet (msl) within the shallow bedrock.

CONCLUSIONS AND RECOMMENDATIONS

1.0 FEASIBILITY

Based on our exploration, laboratory testing, and geotechnical analyses, it is considered geotechnically feasible to develop the power plant at the proposed site, provided that our conclusions and recommendations presented in this report are considered in the project design. Geotechnical considerations that will have bearing on the development include: the presence of loose uncertified fill of varying thickness with liquefaction potential along the southern half of the site ("Deep Fill Area"); the site's close proximity to both the active San Andreas and Hayward faults, potentially causing strong motions; evaluation and selection of shallow or deep foundation alternatives; and, possible mitigation of unsuitable materials within the existing fill.

2.0 SEISMIC DESIGN CONSIDERATIONS

2.1 **Ground Shaking.** Due to the proximity of several faults, ground shaking during moderate to strong earthquakes is expected to be significant. Because of the varying subsurface profiles within the project boundary (i.e., Shallow Bedrock Area and Deep Fill Area), variable magnitudes of ground shaking and displacement may occur. Based on determinalistic analysis using mean, and mean plus one standard deviation peak ground motions, ground accelerations may be expected at the project site as shown in Table 4 – Design Ground Accelerations. The derivation of these accelerations is based on our review of several attenuation relationships, including Campbell (1994), Sadigh et al. (1987, 1989), and Idriss (1987, 1993). We recommend using mean plus one standard deviation due to relatively high peak ground accelerations associated with the nearby San Andreas fault.

TABLE 4 – DESIGN GROUND ACCELERATIONS

Ground Motion Parameter	Shallow Bedrock Area	Deep Fill Area
Mean Peak Acceleration	0.40g	0.45g
Mean Peak + One Standard Deviation Peak Acceleration	0.60g	0.65g

2.2 **Fault Rupture.** No known traces of active or potentially active faults underlie the project site. Consequently, the potential for fault rupture at the site is considered low.

2.3 **Liquefaction.** Liquefaction is a phenomena wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading.

The foundation soils underlying the Shallow Bedrock Area of the project site consist of dense to very dense formational materials that are not considered liquefiable. Consequently, the liquefaction potential of the foundation soils in the Shallow Bedrock Area is considered very low.

Liquefaction potential of deep fill encountered in Borings B-3 and B-4 was analyzed using the methods prescribed by Andrews and Martin (2000) and Seed and Idriss (1982). These methods evaluate liquefaction potential with respect to material type and density as indicated by a Standard Penetration Test (SPT) blow count.

Within the Deep Fill Area, as represented by borings B-3 and B-4, a zone of artificial fill ranging from approximately 10 to 24 feet in depth shows a high potential for liquefaction when saturated and subjected to strong ground shaking. At depths of 24 feet or greater, soils (e.g., Younger Bay Mud, Bayside Sand deposits, and bedrock) show a low potential for liquefaction. Liquefaction of the fill could cause loss of bearing capacity, large seismic total and differential settlement, or heaving of shallow foundations resulting from densification, sand boils, or lateral spreading during seismic shaking.

In our analysis, all soils identified in our borings were assumed saturated in order to evaluate liquefaction potential based on material type and relative density alone. However, as soil saturation is required for liquefaction to occur, it should be noted that at the time of our exploration, groundwater levels in B-3 and B-4 were encountered at approximately 22 and 13 feet below the ground surface, respectively. Therefore, under the groundwater conditions encountered at the time of our exploration, a zone from 22 to 24 feet in boring B-3 and a zone from 13 to 22 feet deep in boring B-4 would be prone to liquefaction. These groundwater elevations may change seasonally. Although loose unsaturated fill above the groundwater table will not liquefy, it may undergo seismic settlement resulting from densification.

In conclusion, a zone from approximately 10 to 22 feet below the ground surface in the Deep Fill Area west of the existing Station A Building should be considered susceptible to liquefaction and/or seismic settlement for design purposes. Methods for the mitigation of liquefiable loose fill include:

- Removal and replacement of loose fill materials,
- Stone columns (pipe pile or vibro-replacement type)
- Chemical grouting
- Jet grouting (cement slurry)

In lieu of the soil improvement methods listed above, we recommend structures within the Deep Fill Area be supported on a deep foundation scheme consisting of driven H-piles that derive their support from end bearing on the underlying bedrock. It should be recognized, however, that the pile installation would not significantly reduce the potential for liquefaction of loose fill within the Deep Fill Area. Additionally, consideration should be given to the reduction in lateral load support during liquefaction of loose fill along the pile axis.

2.4 **Design Acceleration Response Spectra.** If the proposed structures are designed using, or checked against, the Uniform Building Code Static Force Procedure (ICBO, 1997), the seismic parameters shown on Table 5 – UBC Seismic Design Criteria may be used in developing the site response.

TABLE 5 – ICBO (1997) SEISMIC DESIGN CRITERIA

UBC Seismic Parameter	Bedrock Site	Deep Fill Site
Seismic Zone Factor, Z:	0.4	0.4
Soil Profile Type:	S_C	S_E
Seismic Coefficient, C_a	0.4	0.36
Seismic Coefficient, C_v	0.6	1.06
Near-Source Factor, N_a:	1.0	1.0
Near-Source Factor, N_v:	1.1	1.1

3.0 GROUNDWATER

Our exploration indicates that two groundwater-bearing zones exist beneath the project site. The zones consist of phreatic groundwater in the Deep Fill Area on the southern half of the project site, and confined groundwater in rock fractures in the Shallow Bedrock Area.

Based on our data review from groundwater monitoring performed by others (Geomatrix, 2000), potentiometric elevation of the groundwater confined within in the Shallow Bedrock Area is on the order of +21 feet above mean sea level (msl). Based on a working ground surface elevation of +31 feet (SFCD), groundwater confined in bedrock fractures may enter into foundation excavations on the order of 10 feet deep or greater, subsequently stabilizing at a phreatic surface approximately 9 feet below the ground surface. The rate at which groundwater may accumulate in foundation excavations will likely occur slowly but could vary depending on the size and orientation of exposed rock fractures within the excavation.

The phreatic groundwater within the Deep Fill Area experiences a steep south- to southwest-trending gradient that approximately follows the bedrock/fill interface. This steep gradient was reflected in our groundwater measurements in borings B-4 and B-3, where groundwater depth increased from 10 to 23 feet below the relatively level ground surface. The gradient likely represents transition flow from the elevated groundwater in the bedrock fractures to the lower groundwater in the bayside fill. The Groundwater in the Deep Fill Area should be anticipated at depths ranging from 5 feet below ground surface along the northern Deep Fill Area boundary to as deep as 20 feet in the southwestern corner of the Deep Fill Area.

Groundwater elevations at the project site may fluctuate seasonally, particularly within the fractured shallow bedrock. Because the groundwater within the project site is generally higher than the high tide level of the adjacent San Francisco Bay (approximately 1000 feet east of the site), tidal fluctuations of groundwater are likely to be minimal. In the southwest corner of the site, where the deepest groundwater is on the order of 20 feet deep, some tidal influence may occur, but should only result in groundwater fluctuations on the order of a few feet.

4.0 EARTHWORK

4.1 **Site Preparation.** Site preparation will consist of major demolition of existing abandoned buildings, structures, and utilities as well as excavation and removal of on-site materials to desired foundation grades. Evaluation and identification of potential contamination of subsoils, if any, and the need to excavate such contaminated subsoils are not part of our scope of work.

Excavations should likely be accomplished with conventional equipment. All areas to support new foundations should be stripped of any debris or other unsuitable material. The location of live underground utilities should be determined and, if affected by construction activities, relocated or protected as appropriate.

The soil surface exposed by the site preparation work should be scarified to a depth of at least six inches, conditioned with water or allowed to dry to achieve a soil water content near or slightly above the optimum value, and mechanically compacted to at least 90 percent relative compaction, as determined by Standard Test Method ASTM D1557. The site may then be brought to design grade with engineered fill.

For structures supported on shallow serpentinite bedrock, unsuitable shallow fill and highly weathered serpentinite soils should be removed to expose firm natural rock and, where over-excavated to remove such soils, backfilled with compacted engineered fill.

To provide firm and uniform support for at-grade structures supported on spread footings, at least two feet of compacted engineered fill, or suitable native material should underlie the foundations. This may require over-excavation of the existing in-place soils to a depth of two feet below the foundation bottom. The exposed surface should be examined by a Geotechnical Engineer or his representative to determine whether additional over-excavation is required to remove any remaining unsuitable material. The limits of this over-excavation should extend a minimum of two feet beyond the projected foundation lines.

4.2 **Excavations.** Weathered serpentinite and granular fills, such as those encountered in our investigation, can be excavated with conventional power equipment. Excavations for the proposed improvements are anticipated to be on the order of ten feet deep, or less, and will allow for open unshored sloped excavations, or vertical walled shored or braced excavations.

As a minimum, excavations should be constructed in accordance with the current California Occupational Safety and Health Administration (OSHA) regulations (Title 8, California Code of Regulations) pertaining to excavations. Temporary cut slopes for shallow foundation excavations (20 feet or less) in the Shallow Bedrock Area are expected to be stable for configurations described in Title 8 and Appendix A of the California Department of Transportation (DOT) Shoring Manual (DOT, 1990) for Type A soils and should be cut back no steeper than ½:horizontal to 1:vertical (1/2:1) for excavations 12 feet deep, or less, and ¾:1 for deeper excavations up to 20 feet. Temporary cut slopes for shallow excavations in the Deep Fill Area are expected to be stable for configurations described in Title 8 and Appendix A of the DOT Shoring Manual for Type C soils and should be cut back no steeper than 1½:horizontal to 1:vertical (1½:1) for excavations up to 20 feet deep. The proposed excavation plans should be reviewed by a geotechnical engineer prior to construction. All excavations should be closely monitored during construction to detect any evidence of instability and distress to adjacent structures.

Over-excavation may be required if unsuitable soils are encountered at the bottom of the foundation excavation or if the soils at the excavation bottom are loosened by construction operations. The unsuitable material should be removed and replaced by Class 2 aggregate base rock, engineered fill as specified in Section 4.3, or other approved fill. We should be given the opportunity to observe the excavations during construction.

4.3 **Engineered Fill.** New foundations should be supported on compacted engineered fill, or dense undisturbed soils of the Franciscan Complex. If engineered fill is required to achieve foundation grade or replace over-excavated unsuitable soils, it should be at least one foot thick beneath concrete slabs and two feet thick beneath footings. Material for engineered fill should be inorganic, well graded, free of rocks or clods greater than 4 inches in greatest dimension, and have a low potential for expansion. The material should have a liquid limit less than 35, a plasticity index less than 15 and no more than 25 percent passing the No. 200 sieve. Because the on-site fills are poorly graded and contain variety of debris, we conclude that they are unsuitable for use as engineered fill. We also recommend that excavated weathered serpentinite not be used for engineered fill because of its potential for high plasticity when highly weathered.

4.4 **Engineered Fill Placement and Compaction.** Engineered fill should be placed in layers no greater than 8 inches in uncompacted thickness, conditioned with water or allowed to dry to achieve a soil water content near or slightly above optimum, then mechanically compacted to at least 90 percent relative compaction based on ASTM D1557. All engineered fill placed to support footings and the upper 6 inches of engineered fill supporting slabs-on-grade should be mechanically compacted to at least 95 percent relative compaction as determined by ASTM D1557. All compaction should be performed using mechanical compaction means; flooding or jetting should not be used as a means to achieve compaction. The ASTM D1557 laboratory compaction tests should be performed at the time of construction to provide a proper basis for compaction control

4.5 **Structural Backfill.** All structures extending below grade should be backfilled with structural fill to a minimum width of two feet beyond the foundation footprint. Structural backfill should meet the following gradation:

Sieve Size	Percent Passing
3 inches	100
11/2 inch	80 to 100
#4	50 to 100
#16	40 to 90
#50	10 to 60
#200	0 to 10

Backfill should be moisture conditioned to within two percent above optimum, placed in layers not exceeding 8 inches in uncompacted thickness, and mechanically compacted to 90 percent relative compaction per ASTM D1557. Once backfill candidate materials are determined, compaction curves should be developed per ASTM D1557 prior to placement.

5.0 FOUNDATION DESIGN

Our foundation recommendations in the following sections are based on the assumption that the surface grades at the site are not going to significantly change after demolition of existing facilities. As we do not know the final foundation locations, elevations and anticipated loads at this time, the foundation recommendations presented here should be considered preliminary in nature. They should be reviewed, and revised if necessary, once design foundation elevations and loads are established.

5.1 **Shallow Bedrock Area.** Proposed power plant structures within the Shallow Bedrock Area can be supported on slabs, mat foundation, or conventional spread footings, provided our findings, conclusions, and recommendations are used in preparing the project plans and specifications. We recommend that overburden fill be excavated to expose competent serpentinite bedrock or to a minimum depth of two feet below foundation grade and, if necessary, backfilled with engineered fill to bring up to foundation grade. In cases where structures require resistance to high lateral and/or uplift loads, we recommend consideration of concrete drilled shafts socketed in rock. Required drilled shaft diameter and depth of socket would depend on the magnitude of lateral load demands.

5.2 **Deep Fill Area.** Because of the existence of loose and highly variable artificial fill overlying soft marine sediments in the Deep Fill Area, we recommend that critical structures that cannot tolerate significant differential settlement be supported on deep foundations, such as H-piles end-bearing on bedrock. This conclusion is based on the following considerations:

- Unpredictable settlements resulting from soil compression caused by structural loading may occur if the facilities are supported on the existing uncertified fill overlying marine deposits of unknown thickness, and

- Uncontrolled settlements may occur due to potential liquefaction or dynamic compaction under seismic forces if the facilities are supported above the depth interval where liquefaction or settlement of loose fill occurs.

Other deep foundation schemes including concrete drilled shafts, driven pipe piles, or driven pre-cast concrete piles were considered, but driven steel H-piles are most preferable. H-piles derive their compression resistance from end bearing in bedrock and have the most flexibility in terms of cutting off or adding length to reach the varying depths of bedrock. Driving criteria for H-piles should be developed during field design.

Removal of uncertified fill and underlying marine sediments and replacement with compacted engineered fill was evaluated as a possible alternative. However, considering the depth of removal and associated dewatering requirements, we conclude that removal and replacement is not a practical alternative. Other considerations included in-situ soil improvement methods to densify loose fill, thereby mitigating liquefaction potential and increasing bearing capacity. Such methods included dynamic compaction, stone columns, and injection or jet grouting. These methods were determined cost prohibitive when compared to a driven H-pile foundation scheme.

Light load, non-critical, structures that can tolerate moderate to high settlements can be supported on spread footings within the Deep Fill Area provided that our recommended bearing capacity for the fill is used in design.

5.3 **Foundation Bearing Capacity.** For design of footings constructed on competent subgrade, allowable bearing capacities shown on Table 6 – Allowable Bearing Capacity may be assumed for dead plus normal duration live loads. These allowable capacities have a factor of safety of at least 3.0 against bearing failure. When considering additional short-term seismic or wind loading, the allowable bearing capacities may be increased by one-third. Allowable bearing capacities recommended herein are applicable to newly constructed footings with minimum widths of 24 inches and footing bottoms embedded at least 18 inches below lowest adjacent grade.

TABLE 6 – ALLOWABLE BEARING CAPACITY

Soil Type	Allowable Bearing Capacity (psf)
Franciscan Bedrock (Competent Serpentinite)	5,000
Engineered Fill (Shallow Bedrock Area)	3,000
Engineered Fill (Deep Fill Area)	2,000

5.4 **Total and Differential Settlements.** Settlement of mats, slabs, and footings will depend upon their dimensions as well as the imposed loads. For estimation purposes, footings that are constructed on subgrades as described herein should settle less than half an inch in the Shallow Bedrock Area. Less than half an inch of differential settlement between footings may be expected. Most of the settlement should occur during or immediately after construction. Long-term settlements are not anticipated in the Shallow Bedrock Area.

Because the Deep Fill Area contains uncertified fill of varying composition, density, and thickness, unpredictable short and long-term settlements and differential settlements of mats, slabs, and footings may occur.

5.5 **Unequal Support Conditions.** Shallow foundations that bear within both the Shallow Bedrock Area and the Deep Fill Area are may experience differential settlement because of unequal support conditions. It is recommended that whenever possible, structures be located to bear within a single zone (i.e. shallow bedrock or deep fill areas). Where structures must bear within both areas, considerations should be given to the use of combination foundation utilizing footings founded on rock and driven H-piles to reduce the risk for damaging differential settlement.

5.6 **Modulus of Subgrade Reaction.** The modulus of subgrade reaction addresses the relationship between foundation pressure and deflection. It is also generally influenced by the size of the footing. For design purposes, the modulus of subgrade reaction of undisturbed foundation material at the project site may be taken as 500 kips per cubic foot.

6.0 LATERAL EARTH PRESSURES

Structural components that extend below ground surface, such as foundations and below-grade walls, will experience lateral earth pressure from the soil and hydrostatic pressure from any existing groundwater. Recommendations for the active, at-rest, passive, and seismic earth pressures, and coefficient of base friction to resist active and at-rest loads are provided in the following sections and summarized below in Table 7 - Lateral Earth Pressures and Base Friction for Native Serpentine Rock and Engineered Fill. Surcharges from adjacent structures should be evaluated separately during design.

6.1 **Active Earth Pressure.** Active earth pressures are imposed by the soil on walls that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. Active earth pressure may be calculated using a design Equivalent Fluid Pressure (EFP) of 15 pcf in native undisturbed serpentine rock or 30 pcf in engineered fill or structural backfill. In submerged conditions, these values should be reduced to 10 pcf and 15 pcf, respectively. In addition, hydrostatic loads should be considered in the design.

6.2 **At-Rest Earth Pressure.** At-rest pressures should be used for design of walls that are restrained such that the deflections required to develop active earth pressures cannot occur or are undesirable. The at-rest earth pressure may be calculated using a design EFP of 25 pcf in native undisturbed serpentine rock or 50 pcf in engineered fill or structural backfill. In submerged conditions, these values should be reduced to 15 pcf and 25 pcf, respectively. In addition, hydrostatic loads should be considered in the design.

6.3 **Passive Earth Pressure.** Lateral loads on structures can be resisted by passive pressures that develop against the sides of below-grade structures such as walls or footing keys. On level ground, the passive earth pressure may be calculated using a design EFP of 400 pcf in native undisturbed serpentine rock or 300 pcf in engineered fill or structural backfill. In submerged conditions, these values should be reduced to 210 pcf and 150 pcf, respectively. The passive earth pressure values recommended here include a reduction factor of 1.5 to limit deflections. Passive pressures may be combined with the base friction mobilized at the concrete-soil interface to resist lateral loading.

6.4 **Surcharge Loading.** Additional surface applied live and dead surcharge loads may also impose an increase to active and at-rest lateral earth pressures. For design, we recommend the additional lateral earth pressure imposed by a surcharge to be calculated as 0.25***q**, where **q** is the surcharge pressure applied on a relatively level surface near the edge of the lateral earth retaining structure, provided the earth retaining structure is 10 feet high or less. The lateral pressure increase due to surcharge loading should be distributed continuously (i.e., "rectangular" distribution) for the entire depth of the earth retaining structure. For sloping ground surfaces, higher earth retaining structures, or other unanticipated or unusual surcharge loads, the magnitude and distribution of increased lateral earth pressures should be evaluated on a case-by-case basis.

6.5 **Seismic Active Earth Pressure.** In addition to the active and at-rest pressures, walls extending below grade should be designed to consider additional earth pressures due to earthquake loading. In addition to the static active earth pressure, the increment in active earth pressure due to seismic loading may be calculated using an EFP of 20 pcf. This seismically induced increment in active earth pressure should be applied to the walls as an inverted triangular distribution.

6.6 **Base Friction.** The passive earth pressure and base friction mobilized at the concrete-soil interface may be combined to resist lateral loading. A coefficient of friction of 0.5 may be used for estimating the resistance due to base friction for mass concrete interfacing with clean, undisturbed, serpentinite. A coefficient of friction of 0.4 may be used for estimating the resistance due to base friction for mass concrete interfaced with compacted engineered fill.

TABLE 7 - LATERAL EARTH PRESSURES AND BASE FRICTION FOR NATIVE SERPENTINE ROCK AND ENGINEERED FILL

Soil Type	Active Equivalent Fluid Pressure (pcf)[1]	At-Rest Equivalent Fluid Pressure (pcf)[1]	Passive Equivalent Fluid Pressure (pcf)[1]	Coefficient of Base Friction
Undisturbed Native Serpentine Rock	15 (10)	25 (15)	400 (210)	0.5
Engineered Fill or Structural Backfill	30 (15)	50 (25)	300 (150)	0.4

[1] Values for submerged ground given in parenthesis. Hydrostatic pressure should be also evaluated for submerged earth pressures.

7.0 CORROSION TESTING

Corrosion testing was performed on samples from borings B-3 and B-4, which were representative of sandy fill materials in the Deep Fill Area, and boring B-7, which consisted of weathered serpentine bedrock material. The testing results are summarized in Appendix A.

The measurement of pH ranged from 6.8 to 8.4, indicating neutral to mildly alkaline conditions, but not buffered enough to cause sustained reactions with buried structures.

Sulfates, which can have deleterious effects on buried concrete structures, were detected in concentrations of 400 to 4,000 ppm in the sandy fill, and 100 ppm in the weathered serpentinite. These results indicate that concrete structures placed in granular fill materials in the Deep Fill Area could be subject to long-term degradation. Structures placed in serpentine bedrock in the Shallow Bedrock Area should not experience long-term degradation from sulfate attack.

The measurement of electrical resistivity is a major factor in determining soil corrosivity. The test results for resistivity ranged from 1,050 to 21,750 ohm-centimeters (ohm-cm) in the "as-received" condition, but dropped significantly to 220 to 1,850 ohm-cm under saturated conditions. Resistivities in the "as-received" state are considered mildly corrosive to corrosive to ferrous materials. In the saturated state, the resistivities were corrosive to severely corrosive resulting from the increased water content to saturate the samples. Based on the test results, the designer should consider adopting a suitable means of corrosion protection including protective coating and cathodic protection.

For steel pile foundations in a corrosive environment, several corrosion protective design options can be considered, including:

- Use of a heavier pile steel section than required to provide extra sacrificial thickness,
- Use of epoxy coating (e.g., paint epoxy, fusion-bonded epoxy, coal-tar epoxy). Coating should be durable enough to withstand damage during pile driving,
- Use of ASTM A690 grade (i.e., high strength-low alloy) steel in the pile.

8.0 ENVIRONMENTAL ANALYSIS

During our investigation, soil samples were collected by CH2M HILL for an environmental conditions analysis. The samples were collected from our borings, B-1 through B-8. It is our understanding that CH2M HILL will provide the laboratory results to the SFPUC under separate cover.

9.0 CONSTRUCTION CONSIDERATIONS

Excavations may encounter some isolated zones of relatively hard rock that may require additional means for removal, though measures such as heavy ripping or blasting are not anticipated. In the Shallow Bedrock Area, excavations that are greater than 5 feet deep may encounter confined groundwater that may inundate the excavation, requiring dewatering measures. Dewatering measures should be implemented to provide a relatively dry environment for the placement, moisture conditioning, and compaction of engineered fill and backfill, and to provide a firm working surface at foundation grades for construction of footings or other soil load bearing structures. Design and implementation of any dewatering scheme should be the responsibility of the contractor.

To verify that the intent of our recommendations is incorporated into the final design, we should be given the opportunity to review the geotechnical aspects of project plans, specifications, shoring, excavation, and other geotechnical aspects, prior to construction.

We should be retained during construction to provide site observation and consultation concerning the condition of the bottom of excavations pertaining to foundation construction. Foundation grades should be observed and, where necessary, tested under the direction of a qualified geotechnical engineer to verify compliance with our recommendations. All site preparation work and excavations should also be observed to compare the generalized site conditions assumed in this report with those found on site at the time of construction.

10.0 CLOSURE

The conclusions and recommendations presented herein are professional opinions based on geotechnical and geologic data and the project as described. A review by this office of any foundation, excavation, grading plans and specifications, or other work product that relies on the content of this report, together with the opportunity to make supplemental recommendations is considered an integral part of this study. Should unanticipated conditions come to light during project development or should the project change from that described, we should be given the opportunity to review our recommendations.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied.

REGISTERED PROFESSIONAL ENGINEER
JOSEPH N. SEIBOLD
No. 2600
Exp. 12-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

Submitted by:
GEOTECHNICAL CONSULTANTS, INC.

Joseph N. Seibold, P.E., G.E.
Geotechnical Engineer 2600

REGISTERED PROFESSIONAL ENGINEER
AMY L. KILLEEN
C 61634
Expires 6-30-05
CIVIL
STATE OF CALIFORNIA

Amy Killeen, P.E.
Civil Engineer 61634

REFERENCES

Andrews, D.C. and Martin, G.R. (2000) "Criteria for Liquefaction of Silty Soils." 12th World Conference on Earthquake Engineering, Proceedings, Auckland, New Zealand.

Blake, T.F., 2000, Computer Program: "EQFAULT", Version 3.0 - A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults.

California Department of Transportation (DOT), 1990, "Trenching and Shoring", Appendix A, Revision No. 5, April 1992.

California Geologic Survey, San Francisco Quadrangle Seismic Hazards Map, 2003.

Camp Dresser and McKee, October 1997, "Phase I Environmental Site Assessment, Potrero Power Plant".

Campbell, K. W., and Bozorgnia, Y., 1994, Near-Source Attenuation of Peak Horizontal Acceleration from Worldwide Accelerograms Recorded from 1957 to 1993, in Proceedings: Volume III, Fifth U.S. National Conference on Earthquake Engineering, pp 283 - 289.

Division of Occupational Safety and Health, "Excavation Trenches Earthwork", Title 8, California Code of Regulations, 1992.

Department of Conservation, Division of Mines and Geology, 1997, "Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California", March.

Fluor Daniel GTI, December 1997, "Field Sampling Plan, Pacific Gas and Electric Company, Potrero Power Plant".

Fluor Daniel GTI, June 1998, "Phase II Site Assessment, Pacific Gas and Electric Company, Potrero Power Plant".

Geomatrix, April 2000, "Report of Results, Additional Site Characterization, Potrero Power Plant Site, Volume I of II".

ICBO (International Conference of Building Officials), 1997, Uniform Building Code, Volume 2, Structural Engineering Design Provisions, Chapter 16, Division V.

Idriss, I.M. (1987), "Lecture Notes, Presentation at the EERI Course on Strong Ground Motion, April 10-11, 1987, Pasadena, California," in, Singh, J.P., Course Organizer, Strong Ground Motion, Seismic Analysis, Design and Code Issues, Earthquake Engineering Research Institute, 14 pp.

Kramer, S.L., 1996, "Geotechnical Earthquake Engineering", 1st Edition, Prentice-Hall.

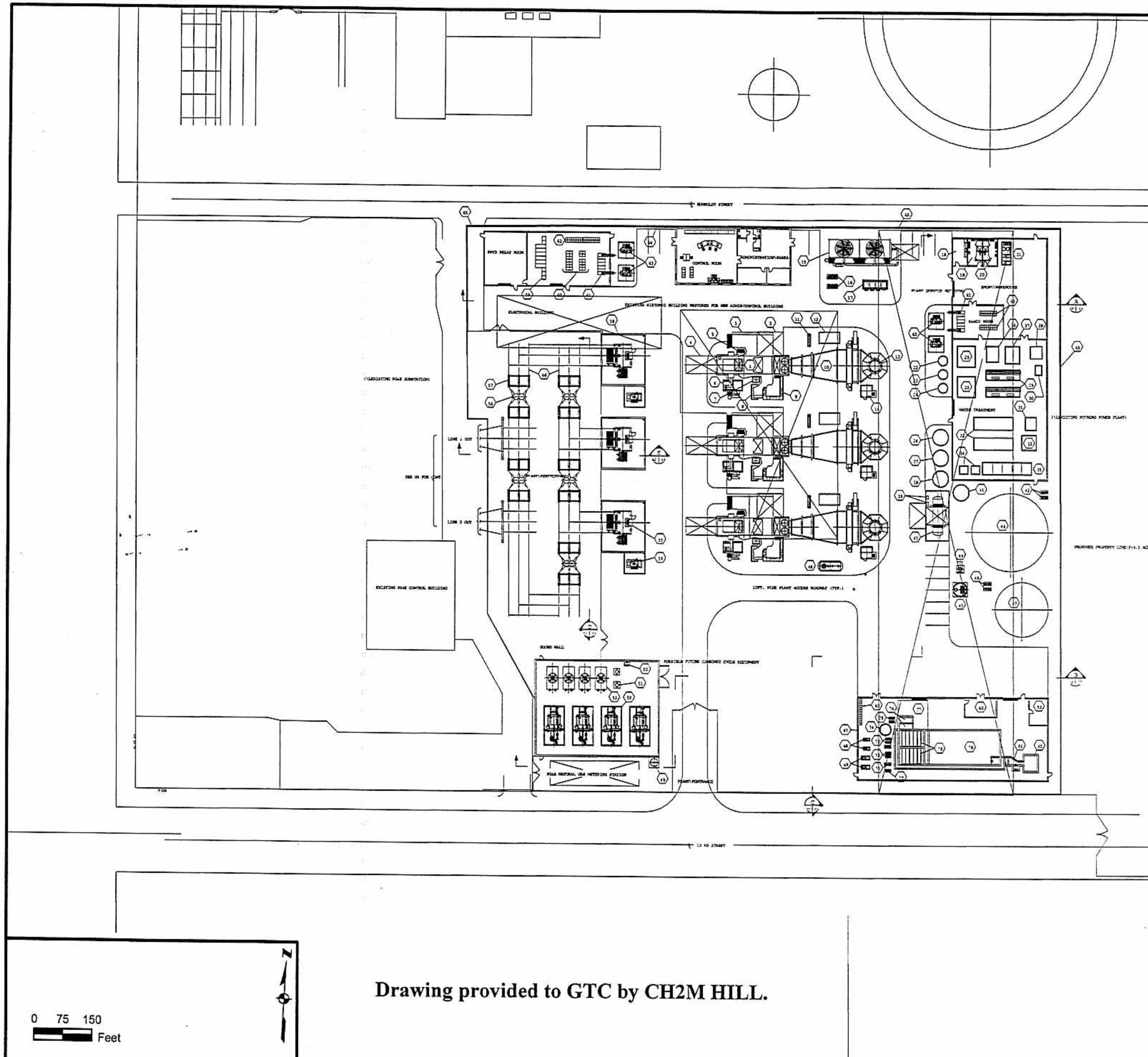
M.J. Schiff and Associates, 1997, "Soil Corrosivity Study, Grant Park Reservoir", Letter report prepared for Geotechncial Consultants, Inc., November.

Mualchin, L., 1996, A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1966 (Based on Maximum Credible Earthquakes), Caltrans Engineering Service Center, Office of Earthquake Engineering.

Naval Facilities Engineering Command (NAVFAC), 1982, "Soil Mechanics," Design Manual 7.1, pp. 7.1-329, May.

Sadigh et al., 1987, "Attenuation Relations for Shallow Crustial Earthquakes Based on California Strong Motion Data," Seismological Research Letters, Vol. 68, No. 1, pp. 180-189.

Seed, H.B. and Idriss, I. M., 1982, "Ground Motions and Soil Liquefaction During Earthquakes": Earthquake Engineering Research Institute Monograph, Oakland, CA.



LEGEND:

1. LM6000 COMBUSTION TURBINE GENERATOR
2. TURBINE REMOVAL/MAINTENANCE AREA
3. CTG AIR INTAKE FILTER SYSTEM
4. GENERATOR ROTOR REMOVAL AREA
5. CTG FIRE PROTECTION SKID
6. GENERATOR BREAKER SWITCHGEAR
7. SPRINT SYSTEM SKID
8. NOx WATER INJECTION SKID
9. AUXILIARY SKID
10. SCR/CO CATALYST SYSTEM
11. AMMONIA FLOW BALANCE SKID
12. AMMONIA VAPORIZATION SKID
13. STACK
14. CEMS
15. DUPLEX CHILLER/COOLING TOWER PACKAGE
16. AUXILIARY COOLING PUMPS
17. COOLING TOWER CHEMICAL SYSTEM
18. LUNCH/BREAK ROOM
19. MEN'S LOCKERS/SHOWERS
20. WOMEN'S LOCKERS/SHOWERS
21. PLANT AIR COMPRESSOR PACKAGE
22. BULK CAUSTIC STORAGE (IF REQUIRED)
23. BULK ACID STORAGE (IF REQUIRED)
24. BULK SODIUM HYPOCHLORITE TANK
25. EDI TRAIN
26. EDI FEED PUMP SKID
27. RO CLEAN IN PLACE SKID
28. RO FEED PUMP SKID
29. RO TRAINS
30. RO CARTRIDGE FILTERS
31. ULTRA FILTRATION SYSTEM WASTE SKID
32. ULTRA FILTRATION SYSTEM TRAINS
33. ULTRA FILTRATION SYSTEM PUMP SKID
34. AIR BLOWERS
35. CHEMICAL METERING SYSTEMS
36. EQUALIZATION TANK
37. BIO REACTOR
38. ULTRA FILTRATION PERMEATE TANK
39. AQUEOUS AMMONIA FORWARDING PUMPS
40. AQUEOUS AMMONIA STORAGE TANK
41. RO PERMEATE TANK
42. TREATED WATER PUMPS
43. TREATED WATER STORAGE TANK
44. OIL/WATER SEPARATOR (UG)
45. WASTE WATER SUMP AND LIFT STATION
46. DI WATER PUMPS
47. DI WATER STORAGE TANK
48. TURBINE WASH WATER DRAIN TANK (UG)
49. NATURAL GAS INLET SCRUBBER
50. HYDROCARBON DRAIN TANK
51. DISCHARGE FILTER SCRUBBER (TYP. 2)
52. FUEL GAS COMPRESSOR (TYP. 4)
53. FUEL GAS COOLING RADIATOR (TYP.4)
54. 13.8kV/115kV GSU (TYP. 4)
55. 5KV AUXILIARY TRANSFORMER (TYP. 2)
56. 115 kV BREAKER (TYP.5)
57. 115kV SWITCH (TYP. 10)
58. FIRE/BLAST WALL (TYP.)
59. 5kV SWITCHGEAR
60. BATTERIES
61. 480V SWITCHGEAR
62. 480V MCC'S
63. 480V STATION SERVICE TRANSFORMERS
64. EXISTING RETAINING WALL
65. NEW RETAINING WALL
66. SWITCHYARD BUSWORK
67. RECLAIMED WATER TREATMENT BLDG
68. SUPLEMENTAL AERATION BLOWERS
69. MEMBRANE AIR SCOUR BLOWERS
70. DRAIN PUMP
71. PERMEATE PUMP (TYP. 2)
72. MIXED LIQUOR RECIRCULATION PUMPS
73. CIP/BACKPULSE PUMPS
74. CIP/BACKPULSE TANK
75. DIP TANK RECIRC/DRAIN PUMPS
76. DIP TANKS
77. CASSETTE LAYDOWN AREA
78. MEMBRANE TANKS
79. AEROBIC ZONE
80. CHEMICAL FEED/STORAGE ROOM
81. FEED CHANNEL
82. COMBINED INLET SYSTEM
83. OFFICE/CONTROL ROOM

Drawing provided to GTC by CH2M HILL.

FIGURE 2-2
SITE LAYOUT
SAN FRANCISCO ELECTRIC RELIABILITY PROJECT
CH2MHILL

PLATE 1
JUNE 2004
SF03015

RDD \\LOKI\PROJECTS\RDDGIS\SFPUC\MXDS\1-4_SITE_LAYOUT.MXD 1-4_SITE_LAYOUT.PDF 03/04/2004 15:51:25

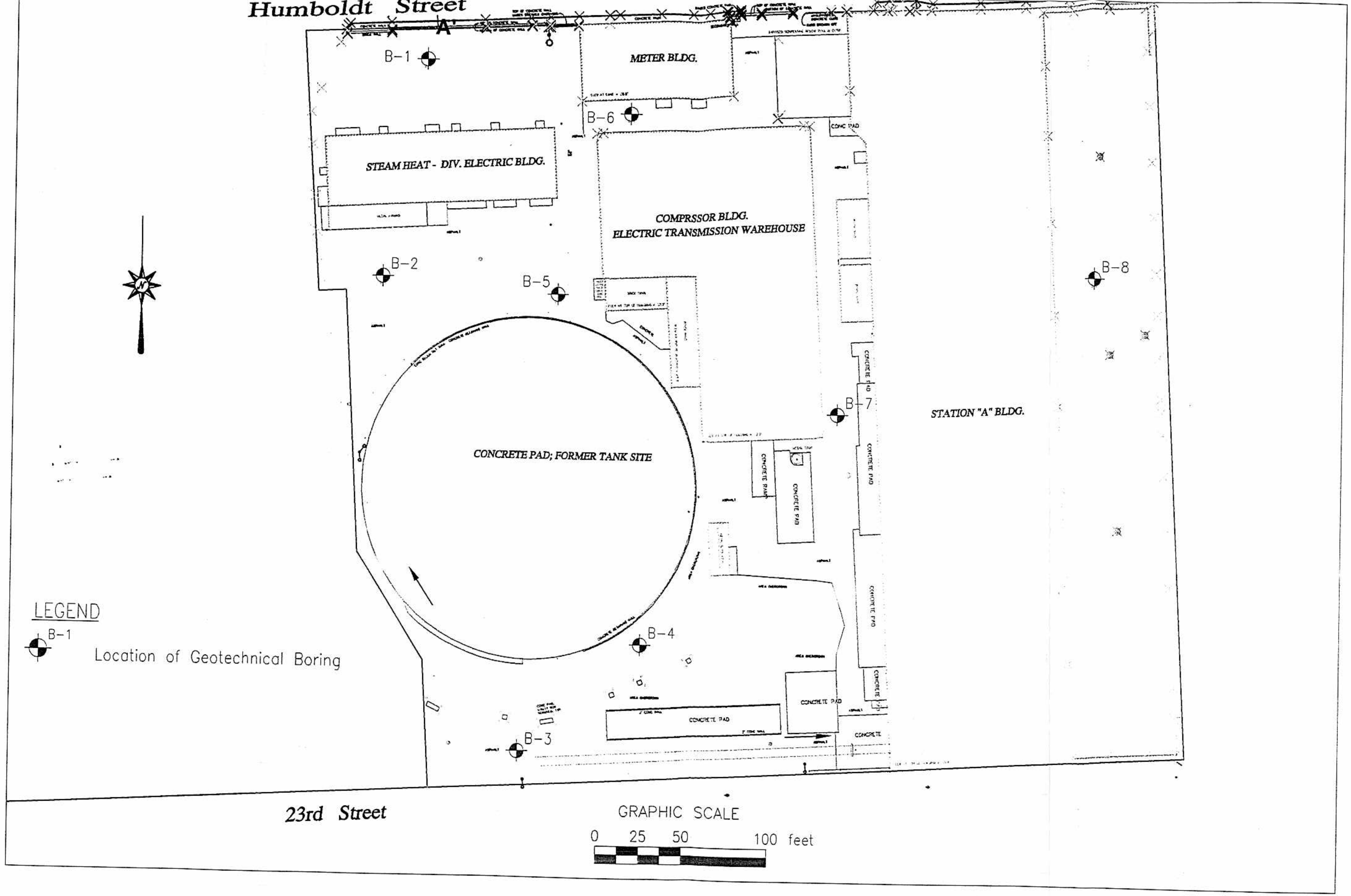
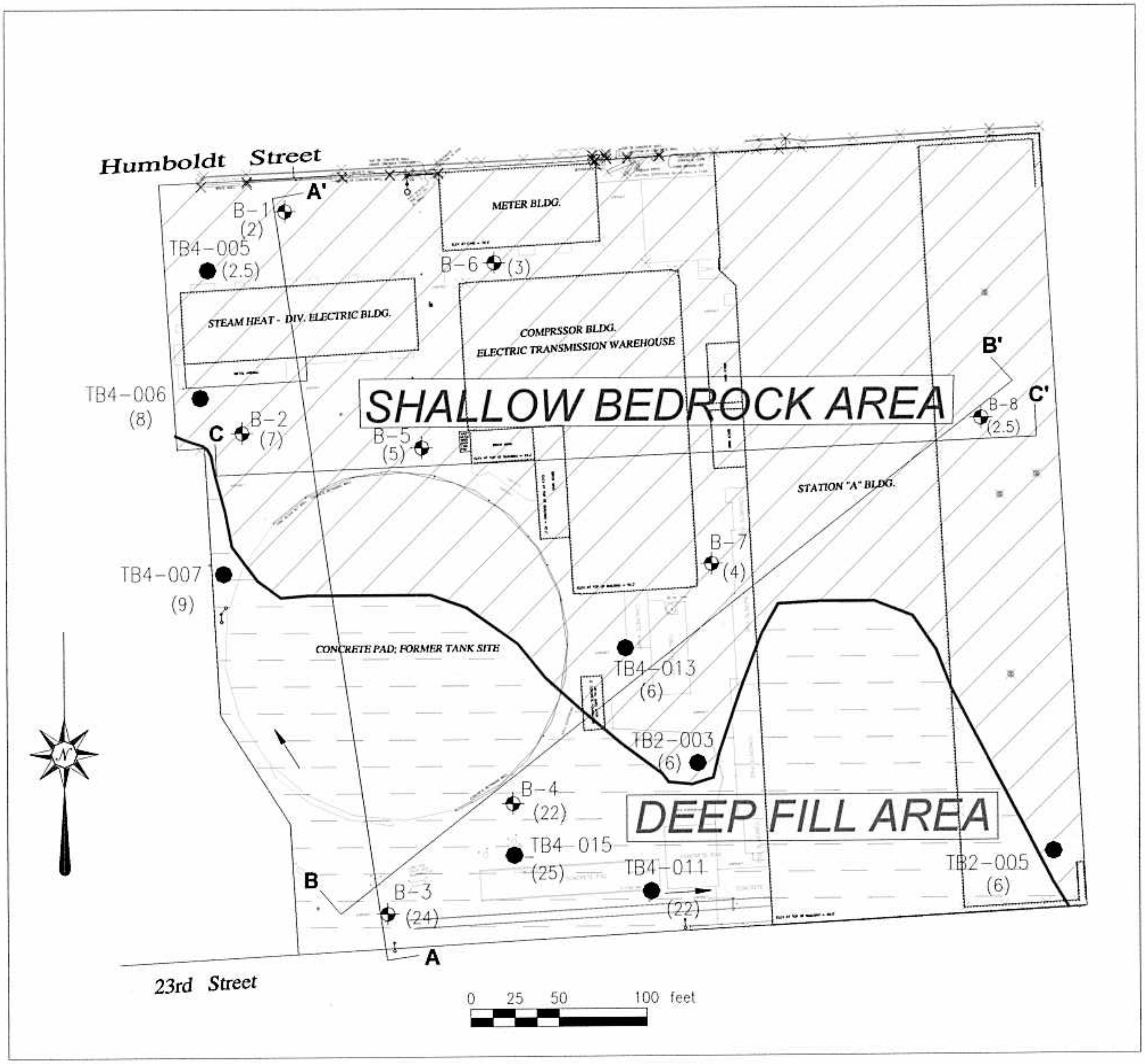


PLATE 2 - BORING LOCATION MAP
JUNE 2004 SF03015



LEGEND

Area underlain by Shallow Bedrock (7 feet or less)

Area underlain by Deep Fill (greater than 7 feet)

B-3 (22) GTC Boring Location (depth of fill in feet)

TB4-011 (22) Approximate Locations of Borings by Others (GTI, 1998) (approximate depth of fill in feet)

C C' Line of Geotechnical Cross Section

PLATE 3 - MAP OF CHARACTERISTIC SUBSURFACE PROFILES

JUNE 2004

SF03015

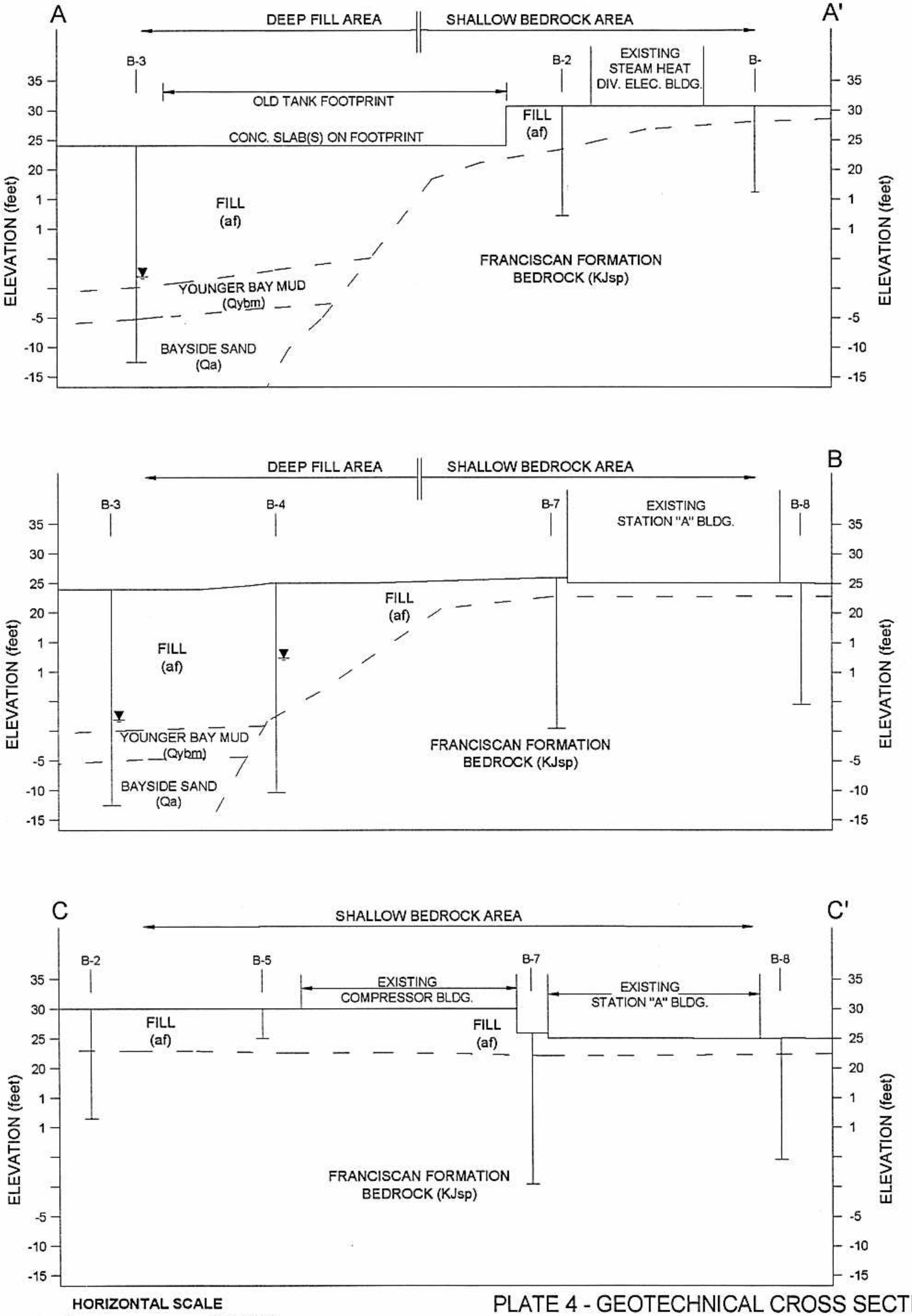


PLATE 4 - GEOTECHNICAL CROSS SECTIONS
JUNE 2004
SF03015

APPENDIX A
SUPPORTING GEOTECHNICAL DATA

SUBSURFACE EXPLORATION

Subsurface exploration for the Potrero Power Plant took place on February 24 and 25, 2004, and consisted of drilling eight hollow stem auger borings, B-1 through B-8. B-5 was drilled to a depth of 5 feet before hitting refusal from what may have been a concrete slab. All of the borings were backfilled with cement grout. The following table shows the depths and surface elevations of the borings. The boring locations and elevations were surveyed by the San Francisco Public Utilities Commission. The project datum is San Francisco City Datum.

TABLE A-1 – BORING DEPTHS AND ELEVATIONS

Boring	Surface Elevation (feet)	Depth (feet)
B-1	31	14.5
B-2	30	18.5
B-3	23	30.5
B-4	25	35.5
B-5	30	5.0'
B-6	31	20.5
B-7	26	25.5
B-8	25	20.5

Locations of the borings are shown on Plate 1. Logs of the borings are presented as Plates A-1.1 through A-1.8.

The stratification lines shown on the boring logs represent the approximate boundaries between soil types; the actual transition may be gradual. Boring locations and elevations were surveyed by the San Francisco Public Utility Commission. The locations and elevations of borings should be considered accurate only to the degree implied by the method used.

SOIL SAMPLING METHODS

Two soil sampling methods were used during the exploration program. A split barrel sampler was driven a total of 18 inches or until refusal per ASTM D1586. The rock was driven into three six-inch long, 2½-inch inside diameter brass liners and the sampler shoe. The sampler was driven by repeatedly dropping a 140-pound hammer approximately 30 inches into the drill rod to which the sampler was attached. The number of blows required to drive the sampler the last 12 inches of a total of 18-inch interval is referred to as the blow count and is recorded on the boring logs. Blow counts were recorded for the purpose of estimating relative soil densities.

Standard penetration tests (SPT's) were performed to evaluate the in-place density of the rock. A 2-inch outside diameter, 1.38-inch inside diameter steel sampler was driven into the rock by repeatedly dropping a 140-pound hammer approximately 30 inches onto the sampling rod to which the sampler was attached. The number of blows required to drive the sampler the last 12 inches of a total 18-inch interval is referred to as the standard penetration test blow count or N-value, and is recorded on the drill hole logs.

LABORATORY TESTING

Laboratory tests were performed on representative soil samples in order to define the engineering properties of the earth materials. Testing procedures followed accepted practice where possible. Where ASTM Standards were used, the latest edition or revision for each test procedure was employed.

MOISTURE AND DENSITY DETERMINATIONS

Moisture content and dry density determinations were performed on representative undisturbed samples to evaluate the natural water content and dry density of the soils encountered. The results are presented on the boring logs.

GRAIN SIZE DISTRIBUTION DATA

Grain-size distribution tests were conducted on 10 samples from B-2, B-3, B-4, B-6, and B-8. The tests were performed in accordance with Standard Test Method ASTM D422 - Standard Method for Particle-Size Analysis of Soils.

These analyses are illustrated on Plates A-3.1 through A-3.4 - Grain Size Distribution Data.

DIRECT SHEAR TEST DATA

Direct shear tests were conducted on 2 samples from B-4 and B-7. The tests were performed in accordance with Standard Test Method ASTM D3080 - Direct Shear Tests for Soils Under Consolidated Drained Conditions. These analyses are illustrated on Plates A-4.1 and A-4.2 – Direct Shear Test Data.

ATTERBERG LIMITS

Atterberg limits were performed on a sample from B-2. Testing was performed in accordance with ASTM D4218 - Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Results of this test are presented on the boring log.

CORROSION TESTING

Corrosion testing was performed on samples from B-3, B-4, and B-7. Testing was performed in accordance with Cal – Test 301 procedures. The results are summarized in the following table.

TABLE A-2 – CORROSION TESTING SUMMARY

Boring	Depth (feet)	Resistivity (ohm-cm)		pH	Sulfates (ppm)	Chlorides (ppm)	Moisture Content (%)
		As received	Saturated				
B-3	5.0	1,046	218	6.8	4,000	130	19.4
B-4	5.0	21,788	1,847	8.4	400	19	6.0
B-7	5.0	8,217	880	8.1	100	122	14.4

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: W of Meter Bldg.
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-1
DRILLING DATE: 2/24/2004
ELEVATION: 31 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							3 inches Asphalt.						
					0.8		"ARTIFICIAL FILL (af)" GRAVELLY SAND (SP) dark gray to black, damp.						
							Concrete block?						
		60					"FRANCISCAN FORMATION (KJsp)" WEATHERED SERPENTINITE (R) greenish gray to dark greenish gray, damp to moist, dense.	119	10.5				
5		28			11.8		Becoming wet, free petroleum product (NAPL) at 4-4.5 feet in sample, strong aromatic/hydrocarbon odor. Fractured, wet, medium dense, strong hydrocarbon odor.						
10		50/6"					Light green to dark blue-gray, moist, very dense, moderate hydrocarbon odor.						
		50/5.5"					Light green to dark greenish gray.						
15							1) Bottom of boring at 14.5 feet. 2) No Groundwater encountered. 3) Boring backfilled with cement grout.						
20													
25													
30													
35													

LOG_DRILL_HOLE POTRERO.GPJ GTC.GDT 3/26/04

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: NW Corner of Large Circular Tank Pad
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-2
DRILLING DATE: 2/24/2004
ELEVATION: 30 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							2 inches Asphalt.						
							"ARTIFICIAL FILL (af)" SILTY SAND (SM) red-brown, moist, loose, fine to medium grained sand.						
		11					Medium dense.						GS
5							Rock (serpentinite) caught in shoe.						
							"FRANCISCAN FORMATION (KJsp)" DECOMPOSED SERPENTINITE weathered to Clay (CL) with sand, dark greenish gray, moist, very stiff, very fine grained sand, slight sulfur odor.						
		24								47	20		
10							Decreasing sand content.						
		72											
							SERPENTINITE (R) light greenish gray, damp, very dense, weathered, fractures to coarse grained sand particles.						
15													
		50/6"					Dark greenish gray, damp to moist.	83	32.5				
20							1) Bottom of boring at 18.5 feet. 2) No Groundwater encountered. 3) Boring backfilled with cement grout.						
25													
30													
35													

LOG_DRILL_HOLE POTRERO.GPJ GTC.GDT 3/26/04

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: SW Corner of Property, N of Railroad Spur
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-3
DRILLING DATE: 2/25/2004
ELEVATION: 23 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							2 inches Asphalt.						
							"ARTIFICIAL FILL (af)" POORLY GRADED GRAVEL (GP) with sand, black, damp, subangular to subrounded gravel, medium to coarse grained sand. Grains are vesicular and appear melted - slag?						
		41					Becoming dense.						GS
5		26					POORLY GRADED SAND (SP) with gravel, black, damp, medium dense, medium to coarse grained sand with subangular to subrounded gravel.						
10		15					Damp to moist.						
							Becoming wet.						
15		2					SILTY SAND (SM) with gravel, olive-gray, wet, very loose, fine to coarse grained sand, subangular to subrounded gravel to 1/2 inch, slight sulfur odor.						GS
20		4					Dark olive-gray, gravel to 3/8 inch, scattered shell fragments, moderate sulfur odor.						GS
25		1					"YOUNGER BAY MUD (Qybm)" CLAY (CL) with sand, dark greenish gray, wet, very soft, very fine to fine grained sand, trace to minor shell fragments (predominantly clam), trace gravel to 1/4-inch, strong sulfur odor.						
30		50/6"					"DUNE OR BAYSIDE SAND (Qa)" POORLY GRADED SAND (SP) dark gray, wet, very dense, fine grained sand, trace to minor shell fragments. Red-brown.	106	22.4				GS
35		41					Trace shell fragments, very fine to fine grained sand.						

1) Bottom of boring at 36.5 feet.
2) Groundwater measured at 22.0 feet through auger.
3) Boring backfilled with cement grout.

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: SE of Circular Tank Pad
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-4
DRILLING DATE: 2/25/2004
ELEVATION: 25 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							3 inches Asphalt.						
							"ARTIFICIAL FILL (af)" SANDY GRAVEL (GP) dark gray to black, damp.						
							SILTY SAND (SM) dark red-brown, moist, fine grained sand.						
		16					Medium dense, minor brick fragments to 1/2-inch.						DS
5		6					Damp, loose.						GS
							Light red-brown.						
10		3					CLAYEY SAND (SC) banded layers of light gray to white and black, damp, very loose. Interlayers of ash and charcoal. Well graded sand with scattered fine gravel.						GS
15		1					Becoming moist.						
							Becoming wet.						
20		3					dark gray to black. Oil sheen on sampler and on water. Moderate hydrocarbon odor.						
							POORLY GRADED SAND (SP) with gravel, light gray to white, wet, very loose. Disintegrated concrete, oil staining.						
							"FRANCISCAN FORMATION (KJsp)" SERPENTINITE (R) dark greenish gray to black, moist, very dense. Very hard drilling at 22 feet.						
25		50/3"					Oil staining in fractures.						
30		50/4"											
35		50/6"											

1) Bottom of boring at 35.5 feet.
2) Groundwater at 12.76 feet measured through auger.
3) Backfilled boring with cement grout.

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: NW Edge of Large Circular Tank Pad
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-5
DRILLING DATE: 2/24/2004
ELEVATION: 30 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							3 inches Asphalt.						
							"ARTIFICIAL FILL (af)" GRAVELLY SAND (SP) greenish gray, dry.						
							POORLY GRADED SAND WITH SILT (SP-SM) red-brown, moist, fine to medium grained sand. Loose.						
		7											
		50/6"					Concrete chunk in shoe of sampler.						
5													
							1) At 5 feet auger "grabbed" on to something, copper wire wrapped around auger bit. Boring location near concrete wall/footing. Moved boring 3 feet east and reattempted drilling. Same results. 2) Depth of boring 5 feet. 3) No Groundwater encountered. 4) Boring backfilled with cement grout.						
10													
15													
20													
25													
30													
35													

LOG_DRILL_HOLE POTRERO.GPJ GTC.GDT 3/26/04

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: Between Meter and Compressor Bldgs.
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-6
DRILLING DATE: 2/25/2004
ELEVATION: 31 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							2 inches Asphalt.						
							"ARTIFICIAL FILL (af)" GRAVEL WITH SAND (GP) dark gray to black, damp, angular gravel to 1-inch, fine to coarse grained sand.						
		4					POORLY GRADED SAND WITH SILT (SP-SM) red-brown, damp, fine to medium grained sand.						GS
5		4					"FRANCISCAN FORMATION (KJsp)" DECOMPOSED SERPENTINITE completely weathered to Clayey Sand (SC) with gravel and sand, marbled light red-brown, dark green, and black, moist, very loose. At 5 feet dark greenish olive-gray, loose, minor subangular gravel sized rock remnants. At 6 feet oil in sample, slight hydrocarbon odor.						
10		7					Dark greenish gray to blue-green, moist to wet, loose, minor subangular gravel sized rock remnants. Becoming less weathered. Roots up to 3/4-inch in diameter.	86	41.4				GS
							SERPENTINITE (R) blue-gray, moist, very dense, slightly to moderately weathered. Very hard drilling at 12 feet.						
15		60											
20		50/6"					Dark greenish gray to black.	107	22.0				
							1) Bottom of boring at 20.5 feet. 2) Groundwater measured at 17.96 feet through auger after 10 minutes. 3) Boring backfilled with cement grout.						
25													
30													
35													

LOG_DRILL_HOLE POTRERO.GPJ GTC.GDT 3/29/04

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: W Side of Station "A" Bldg.
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

DRILL HOLE NO.: B-7
DRILLING DATE: 2/25/2004
ELEVATION: 26 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							2 inches Asphalt.						
							"ARTIFICIAL FILL (af)" GRAVELLY SAND (SP) dark gray to black, damp, angular gravel to 3/4-inch, medium to coarse grained sand.						
		46					Increasing Sand, dark red-brown, damp, loose to medium dense, subangular gravel, abundant brick fragments and serpentinite fragments to 1 1/2-inches.						DS
5		58					"FRANCISCAN FORMATION (KJsp)" SERPENTINITE (R) light greenish gray to dark greenish gray and black, damp, very dense, moderately weathered. At 5 feet dark greenish gray.						
10		62					Dark greenish gray to black, wet, pervasively sheared.						
15		50/5"					Moist.						
20		54					Greenish gray to dark greenish gray, pervasively sheared.						
25		50/5.5"											
							1) Bottom of boring at 25.5 feet. 2) Groundwater not encountered. 3) Boring backfilled with cement grout.						
30													
35													

LOG OF DRILL HOLE

JOB NO.: SF03015
PROJECT: Potrero Power Plant
LOCATION: E Side of Station "A" Bldg.
DRILLING METHOD: Hollow Stem Auger, 6-inch

LOGGED BY: D. Herold
CHECKED BY: J. Seibold

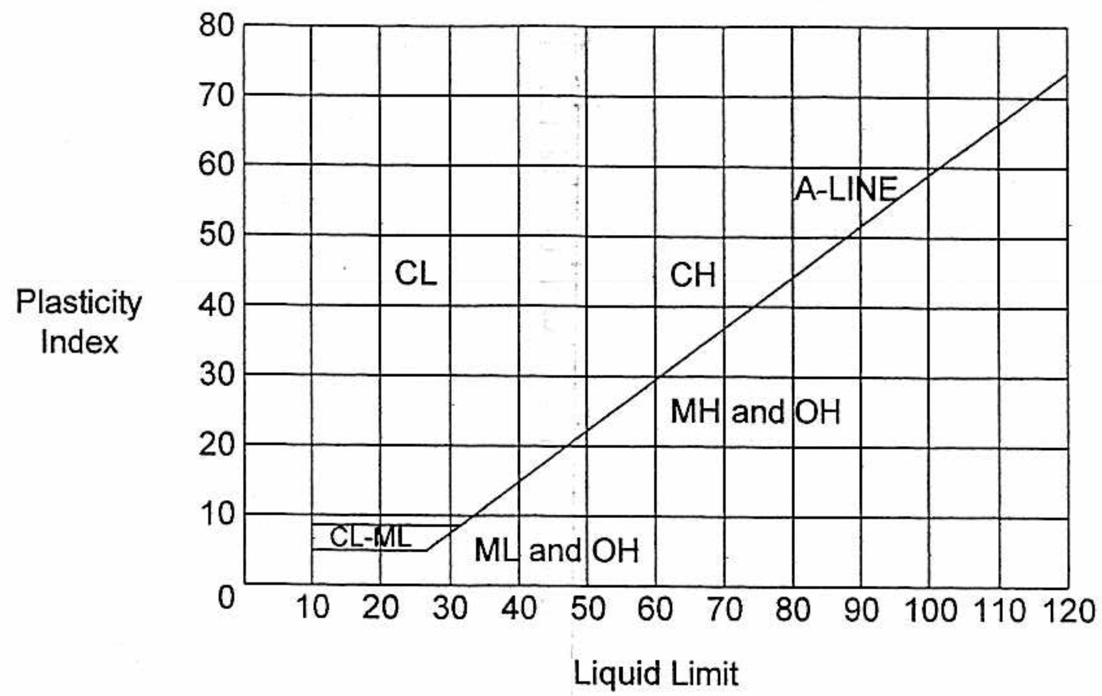
DRILL HOLE NO.: B-8
DRILLING DATE: 2/24/2004
ELEVATION: 25 feet
DATUM: San Francisco City Datum

DEPTH (FEET)	SAMPLE	BLOW COUNT	TORVANE SHEAR STRENGTH (PSF)	POCKET PENETROMETER COMP. STRENGTH (TSF)	PHOTOVAC TIP READING (PPM)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	UNCONFINED SHEAR STRENGTH (PSF)	ADDITIONAL TESTS
							2 inches Asphalt.						
							"ARTIFICIAL FILL (af)" GRAVELLY SANDY CLAY (CL) dark red-brown, damp, gravel clasts are crushed brick fragments to 1/2-inch, medium to coarse grained sand.						
		50/5"					"FRANCISCAN FORMATION (KJsp)" HIGHLY WEATHERED SERPENTINITE weathered to Silty Sand (SM), light olive-gray, damp, very dense.						GS
5		38					WEATHERED SERPENTINITE (R) light greenish gray, dry, dense, fragments into fine to coarse grained sand and fine gravel.						
							Dark greenish gray, becoming moist.						
10		50/3"					Greenish gray to black, moist, very dense.						
15		50/5.5"					Wet at 14.5 feet. Dark greenish gray to black, wet, fractures into medium sized gravel.						
							Becoming dark blue-gray, moist.						
20		50/5"											
							1) Bottom of boring at 20.5 feet. 2) Groundwater not encountered. 3) Boring backfilled with cement grout.						
25													
30													
35													

LOG_DRILL_HOLE POTRERO.GPJ GTC.GDT 3/26/04

LEGEND TO LOGS ON PLATE A-2

PLASTICITY CHART - Used for Classification of Fine Grained Soils



BLOW COUNT - The number of blows required to drive the sampler the last 12 inches of an 18-inch drive. When the sampler is not advanced the last 12 inches, i.e. 100 blows in 9 inches, the notation is 100/9. Symbols designating various hammer weights, drop heights, and sampling methods are shown below. A number not enclosed by one of the following symbols indicates a Standard Penetration Test (SPT) using a 140-pound hammer and 30-inch drop height.

No. of blows	Driving Weight (pounds)	Drop Height (inches)	Sampling Method
( )	______	______	______
[ ]	______	______	______
{ }	______	______	______
< >	______	______	______

ADDITIONAL TESTS -

C: Consolidation
CL: Chloride
CORR: Corrosion
CP: Compaction
DS: Direct Shear
EL: Elasticity Index
EX: Expansion
GS: Grain Size Distribution
pH: Hydrocarbon Ion Concentration
PM: Permeability
R: R-Value
RS: Resistivity
S: Swell
SE: Sand Equivalent
SP: Specific Gravity
SU: Sulphate
TD: Triaxial Compression, Drained
TDy: Triaxial Compression, Dynamic
TU: Triaxial Compression, Undrained
TRPH: Total Recoverable Petroleum Hydrocarbons

GTC

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISION			GROUP SYMBOL	DESCRIPTION	GRAPHIC LOG
COARSE GRAINED SOILS Over 50% By Weight Coarser Than No.200 Sieve Size	GRAVELLY SOILS OVER 50% OF COARSE FRACTION LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELLY SOILS LITTLE OR NO FINES	GW	well graded gravels or gravel-sand mixtures	
			GP	poorly graded gravels or gravel-sand mixtures	
		GRAVELLY SOILS WITH FINES OVER 12% FINES	GM	silty gravels or gravel-sand-silt mixtures	
			GC	clayey gravels or gravel-sand-clay mixtures	
	SANDY SOILS OVER 50% OF COARSE FRACTION SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDY SOILS LITTLE OR NO FINES	SW	well graded sands or gravelly sands	
			SP	poorly graded sands or gravelly sands	
		SANDY SOILS WITH FINES OVER 12% FINES	SM	silty sands or sand-silt mixtures	
			SC	clayey sands or sand-clay mixtures	
FINE GRAINED SOILS Over 50% By Weight Finer Than No.200 Sieve Size	SILTY AND CLAYEY SOILS LIQUID LIMIT LESS THAN 50		ML	inorganic silts, very fine sands, silty fine sands, clayey silts with slight plasticity	
			CL	inorganic clays, gravelly, sandy, silty, or lean clays, of low to medium plasticity	
			OL	organic clays or organic silts of low plasticity	
	SILTY AND CLAYEY SOILS LIQUID LIMIT GREATER THAN 50		MH	inorgainc silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
			CH	inorganic clays of high plasticity, fat clays	
			OH	organic clays or organic silts of medium to high plasticity	
HIGHLY ORGANIC SOILS			Pt	peat or other highly organic soil, organic content greater than 60%	
				trash fill-landfill refuse (not a part of unified soil classification system)	

SAMPLE TYPES:

- UNDISTURBED SLEEVE
- DISTURBED
- UNSUCCESSFUL ATTEMPT
- STANDARD PENETRATION
- STANDARD PENETRATION NO RECOVERY
- SOIL CORE

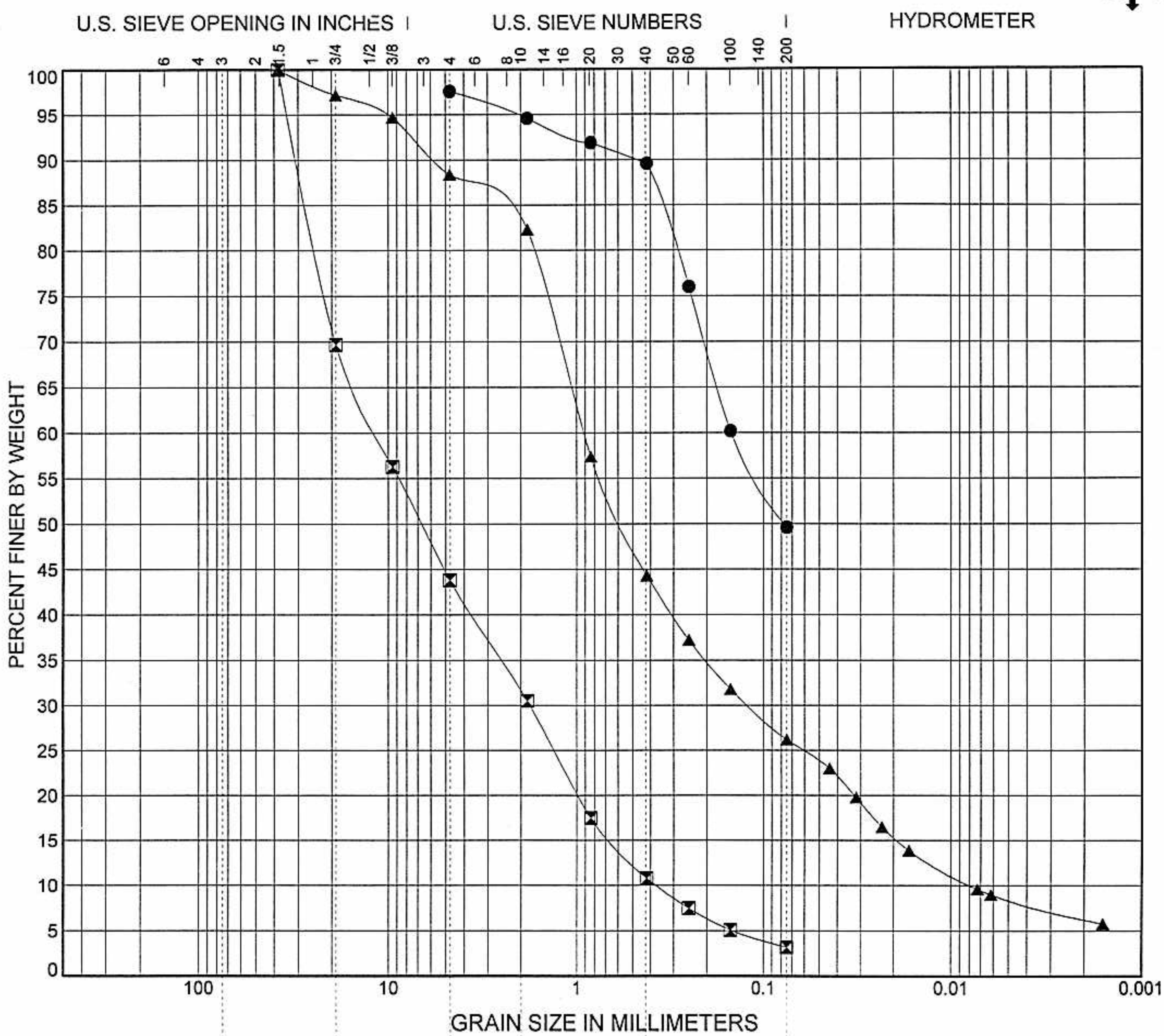
CAVING:

- LIGHT CAVING
- HEAVY CAVING

WATER LEVEL:

PLATE A - 2
LEGEND TO LOGS

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

PARTICLE SIZE DISTRIBUTION

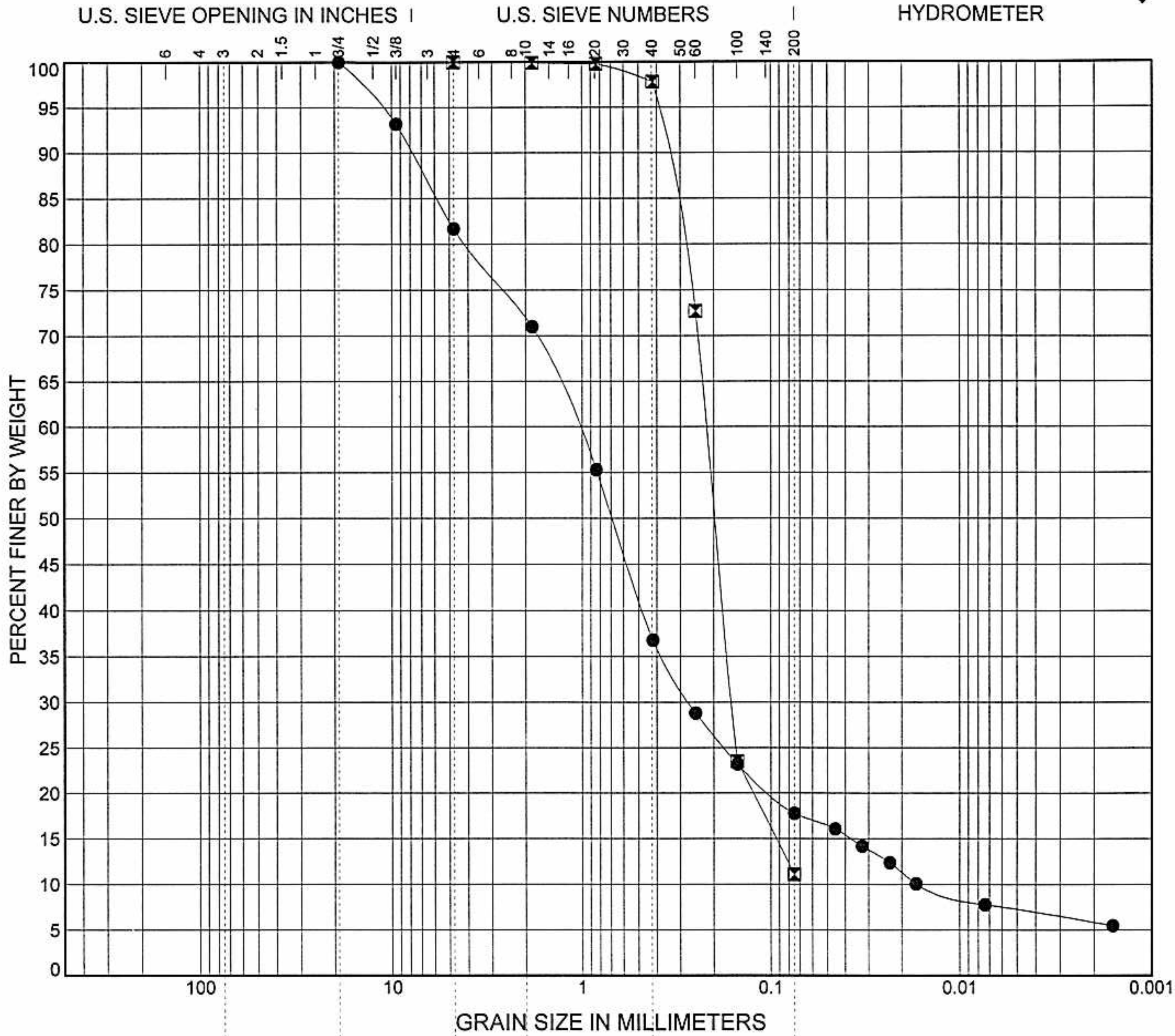
	Specimen Identification	Depth (feet)	Classification
●	**B-2**	**3.5**	**SM**
⊠	**B-3**	**3.0**	**GP**
▲	**B-3**	**15.0**	**SM**

TEST PERFORMED BY:	PROJECT:
Soil Mechanics Lab 8378 Baldwin St. E. Oakland, CA 94621	**Potrero Power Plant**
	JOB NUMBER: **SF03015**

US_GRAIN_SIZE POTRERO.GPJ US_LAB.GDT 3/24/04

PLATE A-3.1

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

PARTICLE SIZE DISTRIBUTION

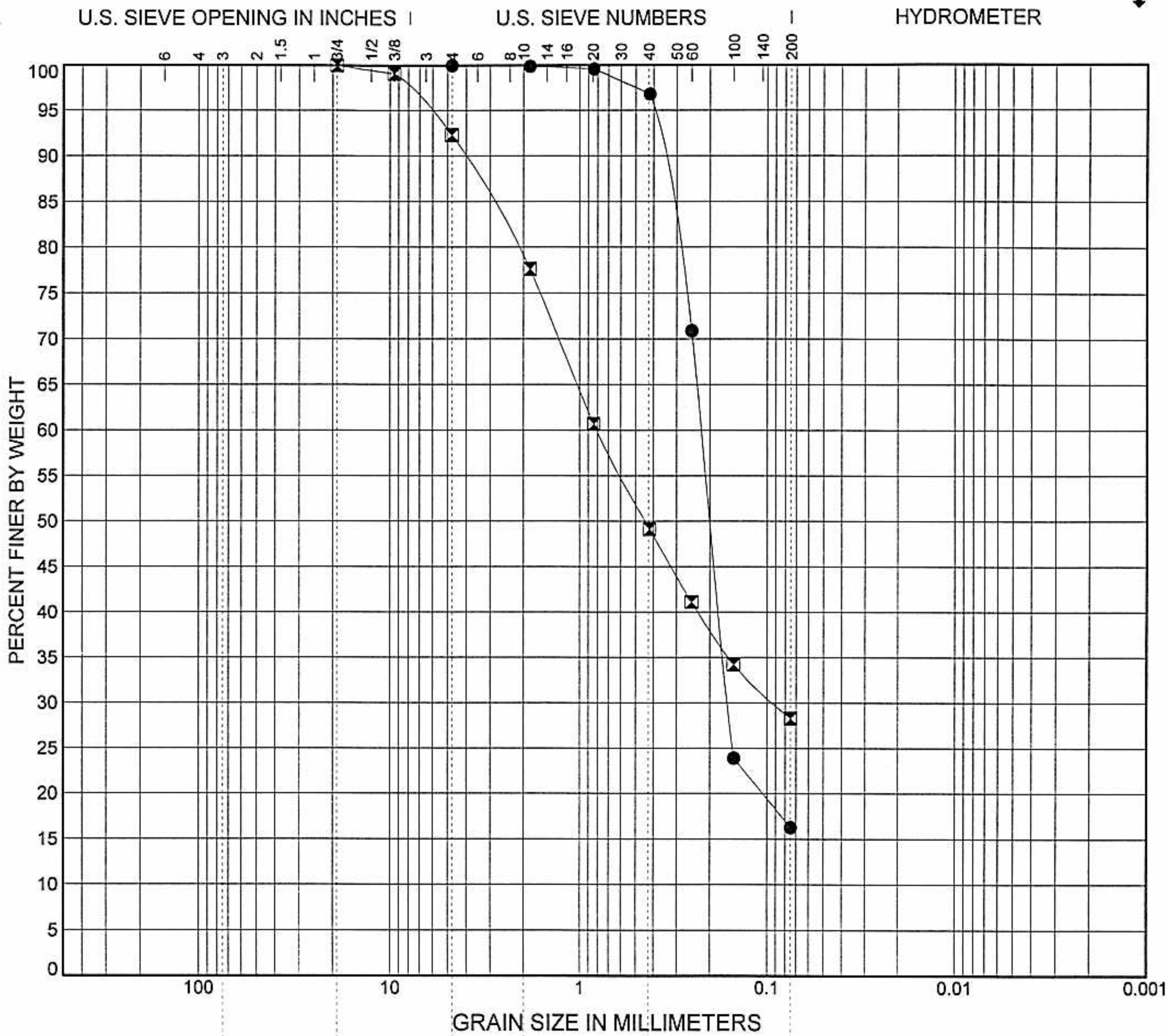
	Specimen Identification	Depth (feet)	Classification
●	B-3	20.0	SM
⧗	B-3	30.5	SP-SM

TEST PERFORMED BY: Soil Mechanics Lab 8378 Baldwin St. E. Oakland, CA 94621	PROJECT: **Potrero Power Plant** JOB NUMBER: **SF03015**

US_GRAIN_SIZE POTRERO.GPJ US_LAB.GDT 3/24/04

PLATE A-3.2

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

PARTICLE SIZE DISTRIBUTION

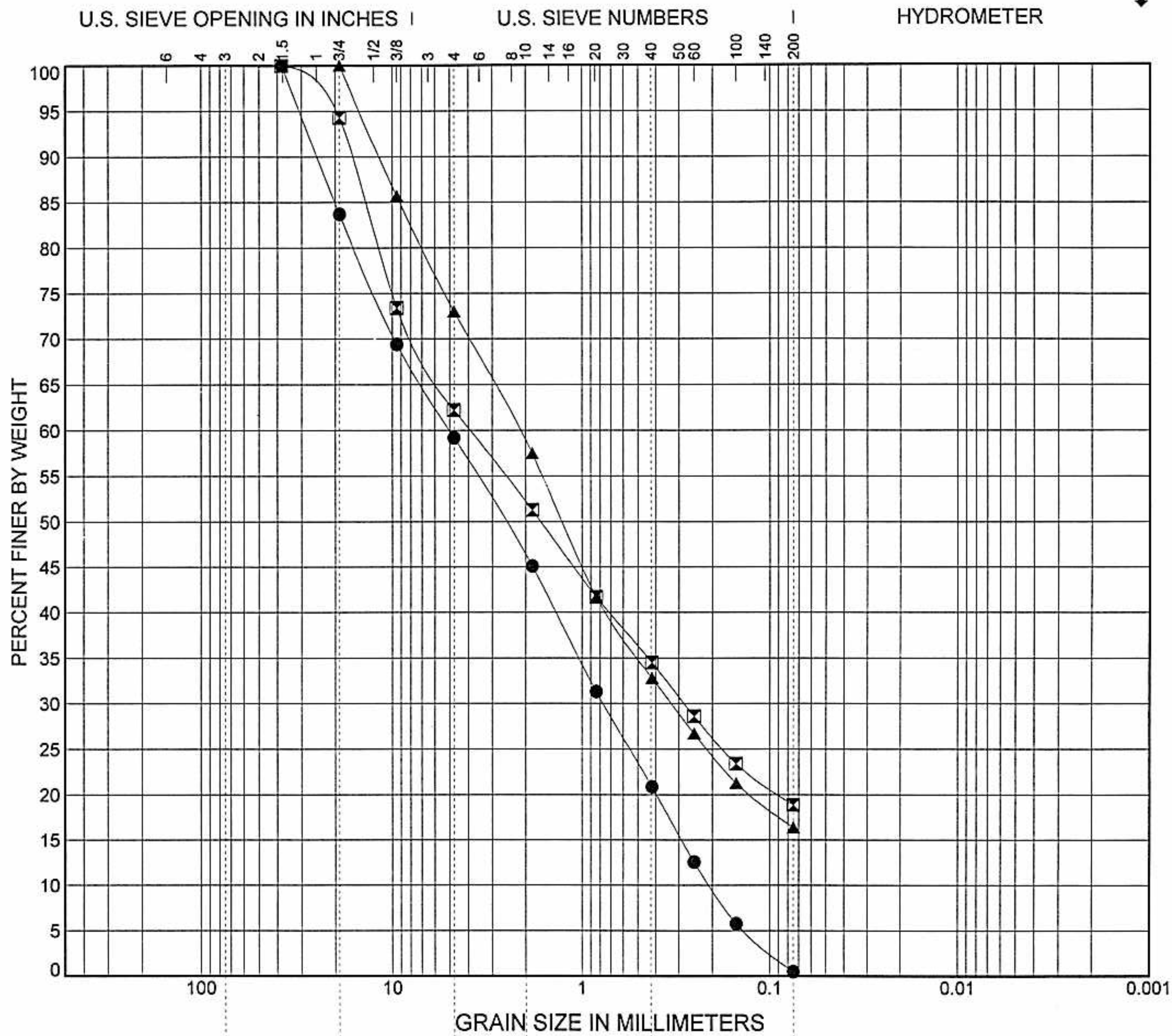
	Specimen Identification	Depth (feet)	Classification
●	**B-4**	**5.0**	**SM**
⊠	**B-4**	**10.0**	**SM**

TEST PERFORMED BY:	PROJECT:
Soil Mechanics Lab 8378 Baldwin St. E. Oakland, CA 94621	**Potrero Power Plant**
	JOB NUMBER: **SF03015**

US_GRAIN_SIZE POTRERO.GPJ US_LAB.GDT 3/24/04

PLATE A-3.3

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

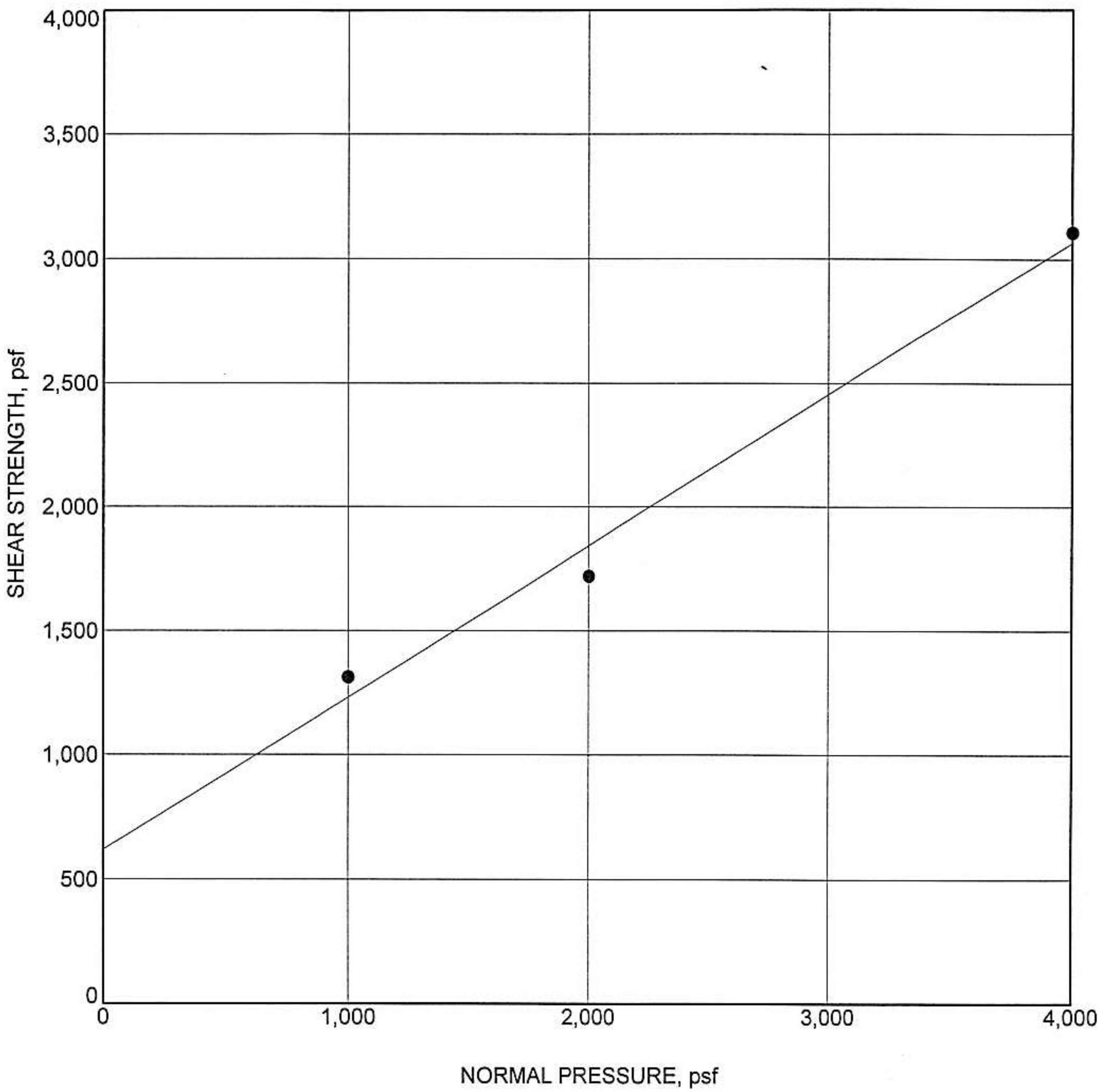
PARTICLE SIZE DISTRIBUTION

	Specimen Identification	Depth (feet)	Classification
	B-6	**3.5**	**SP**
⊠	**B-6**	**10.0**	**SM**
▲	**B-8**	**3.0**	**SM**

TEST PERFORMED BY:	PROJECT:
Soil Mechanics Lab 8378 Baldwin St. E. Oakland, CA 94621	**Potrero Power Plant**
	JOB NUMBER:
	SF03015

PLATE A-3.4

DIRECT SHEAR TEST

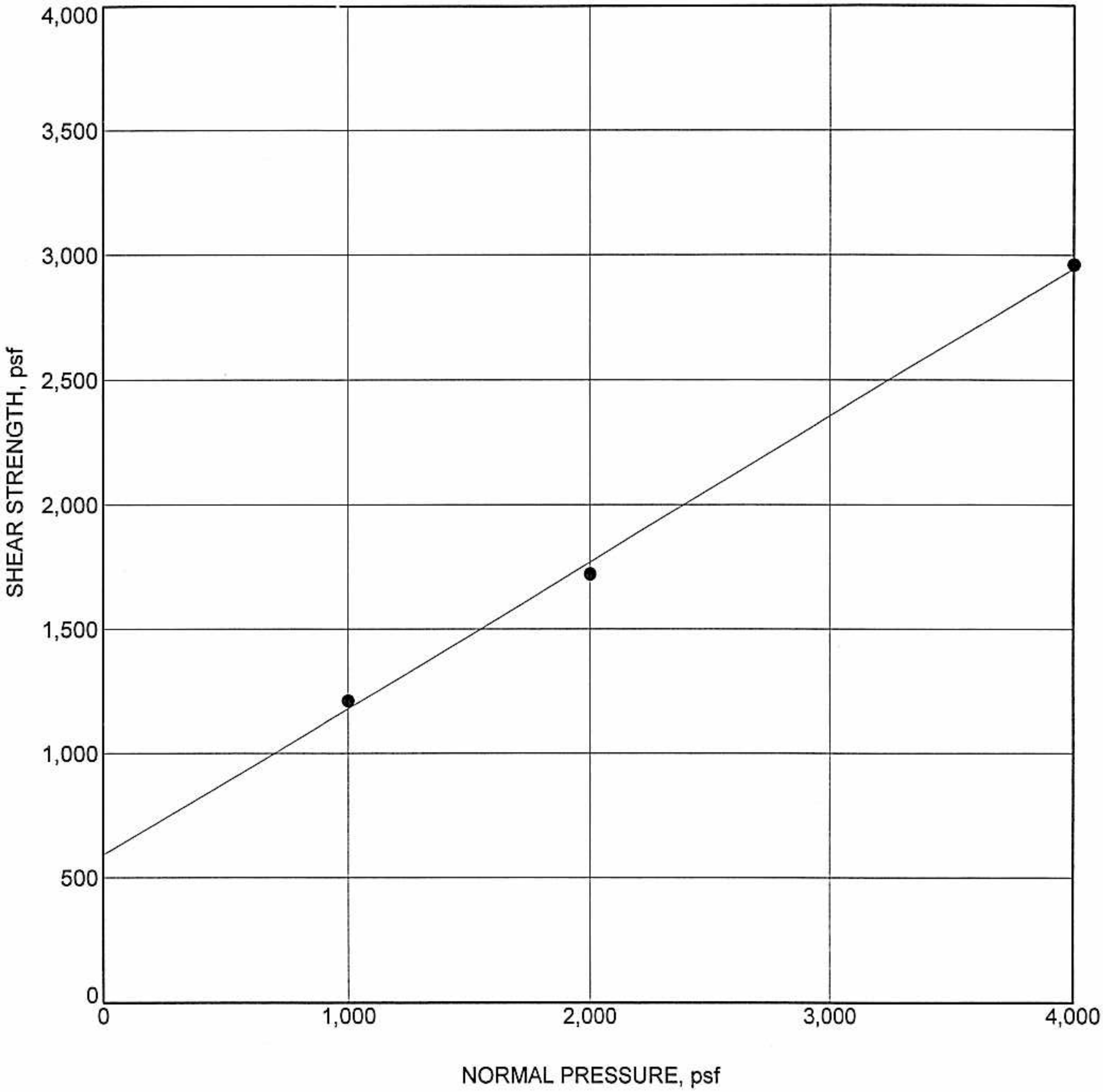


	Specimen Identification	Depth (feet)	Classification	γ_d	MC%	c	ϕ
●	B-4	3.5	SM	101	19	430	34

TEST PERFORMED BY:	PROJECT:
Soil Mechanics Lab 8378 Baldwin St. E. Oakland, CA 94621	**Potrero Power Plant**
	JOB NUMBER:
	SF03015

US_DIRECT_SHEAR POTRERO.GPJ US_LAB.GDT 3/29/04

DIRECT SHEAR TEST



	Specimen Identification	Depth (feet)	Classification	γ_d	MC%	c	ϕ
●	**B-7**	**3.5**	**SP**	**95**	**23**	**375**	**35**

TEST PERFORMED BY:	PROJECT: **Potrero Power Plant**
Soil Mechanics Lab 8378 Baldwin St. E. Oakland, CA 94621	JOB NUMBER: **SF03015**

US_DIRECT_SHEAR POTRERO.GPJ US_LAB.GDT 3/29/04

PLATE A-4.2

GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

TRANSMITTAL

DATE: June 9, 2004

OUR JOB: SF03015

ATTENTION: Tom Lae
CH2M HILL

SUBJECT: Potrero Power Plant Final Geotechnical Report

WE ARE SENDING THE FOLLOWING:

Two bound copies and one unbound copy of the Potrero Power Plant Geotechnical Report.

REMARKS: Please call Amy Killeen with any questions you may have.

COPIES TO: Ralph Hollenbacher, SFPUC, 2 bound copies

3004 16th Street, Suite 204 • San Francisco, CA 94103 • (415) 565-7366