GEOTECHNICAL INVESTIGATION
PIER 94 BACKLAND IMPROVEMENTS
San Francisco, California

Port of San Francisco
Pier 1 – The Embarcadero
San Francisco, California

5 July 2012
Project No. 730509401

T&R / RYCG
A Joint Venture

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Project 730509401

Ms. Carol Bach
Port of San Francisco
Pier 1 – The Embarcadero
San Francisco, California 94111

Subject: Geotechnical Investigation
Pier 94 Backlands Improvements
San Francisco, California

Dear Ms. Bach:

T&R/RYCG, a Joint Venture (T&R/RYCG), is pleased to present this geotechnical investigation report for the proposed improvements to the Pier 94 Backlands in San Francisco, California. Our services were performed in accordance with Task 1 (Alternative Landfill Cover Design) of our proposal for the Pier 94 Backlands Improvements and Amador Street Sanitary Pump Station dated 25 January 2011 and our budget amendment dated 23 May 2011 to drill two additional soil borings.

The Pier 94 Backlands is an irregularly shaped, approximately 47-acre site, generally consisting of the land bound by Amador Street and Cargo Way, extending east to the Amador Street Extension. Previous subsurface investigations identified an approximately 14 - 17 acre portion of the Pier 94 Backlands site where significant quantities of municipal refuse were found in the debris layer (*Data Compilation Report, Pier 94 Solid Waste Disposal Site*, Geo/Resource Consultants, December 1989; *Amended Report of Waste Discharge*, Mark Group, October 11, 1991). This area is identified as the regulated landfill area.

The Port of San Francisco plans to improve approximately 23 acres of the vacant land at the Pier 94 Backlands into 19 acres of leasable property. The area to be improved includes a portion of the regulated landfill area. The site improvements include grading and leveling the site to accommodate leasing and installing new site infrastructure, new water and sanitary sewer utilities for tenant parcels, and a new restroom facility. Site grading will involve placement of up to 18 feet of fill at some locations. To treat the storm water runoff, the site will be graded to collect overland flow around the perimeter of the entire site with vegetated swales; within the regulated landfill area, swales are proposed along Amador Street. Swales through the regulated landfill area will be lined to minimize infiltration. All site flows will be directed to a vegetated swale before being discharged to the San Francisco Bay by means of a new storm water intake structure and outfall pipe and structure.

As part of the design and construction of the Pier 94 Backlands improvements, the Port of San Francisco (Port) seeks to cover the existing regulated landfill area within the Pier 94 Backlands in a manner that would meet the requirements of an engineered alternative landfill cover.

On the basis of our geotechnical investigation, we conclude that the proposed Pier 94 Backlands improvements are feasible from a geotechnical engineering standpoint, provided appropriate landfill closure and post-closure maintenance plans are integrated into the development plans. The primary concerns in developing the site as proposed include:

- seismic hazards
- landfill cover design within regulated landfill area

- water infiltration through the existing and proposed landfill cover within the regulated landfill area
- settlement due to consolidation of underlying Bay Mud and compression and decomposition of refuse
- landfill gas migration, detection, and control systems, and
- foundation design for new restroom facility and storm water intake and outfall structures.

Our conclusions and recommendations for the proposed improvements are presented in the report. Therefore, anyone who relies on this report should read it in its entirety. The conclusions and recommendations presented in this report are derived from limited engineering studies based on our interpretation of the geotechnical conditions existing at the time of the investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, we should be notified and supplemental recommendations should be developed, if necessary.

We appreciate the opportunity of assisting you on this challenging project. If you have questions, please contact us.

Sincerely,
T&R/RYCG – A Joint Venture

Linda H. Liang, GE	John Gouchon, GE
Senior Engineer	Senior Associate

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by T&R/RYCG, a Joint Venture (T&R/RYCG) for the proposed improvements to the Pier 94 Backlands in San Francisco, California. Our services were performed in accordance with Task 1 (Alternative Landfill Cover Design) of our proposal for the Pier 94 Backlands Improvements and Amador Street Sanitary Pump Station dated 25 January 2011 and our budget amendment dated 23 May 2011 to drill two additional soil borings.

2.0 BACKGROUND

The Pier 94 Backlands is an irregularly shaped, approximately 47-acre site, generally consisting of the land bound by Amador Street and Cargo Way, extending east to the Amador Street Extension. The approximate project limits of the proposed Pier 94 Backlands improvements are shown on the Site Location Map and Site Plan, Figures 1 and 2, respectively. The Pier 94 Backlands area was created during the 1960s and 1970s by constructing a perimeter debris dike and placing fill on the inboard side of the dike. The fill consists primarily of dredge spoils and construction debris. After filling ceased in 1975, a soil cap was placed over the construction debris.

Previous subsurface investigations identified an approximately 14 - 17 acre portion of the Pier 94 Backlands site where significant quantities of municipal refuse were found in the debris layer (*Data Compilation Report, Pier 94 Solid Waste Disposal Site*, Geo/Resource Consultants, December 1989; *Amended Report of Waste Discharge*, Mark Group, October 11, 1991). This area is identified as the regulated landfill area. The approximate limits of the regulated landfill area are shown on Figure 2. This area was never operated as a solid waste landfill. However, unauthorized material, including municipal refuse, was placed along with the dredged material and construction debris used as fill to construct the land that comprises the Pier 94 Backlands site. Since 1987, this area has been regulated under Waste Discharge Requirements (WDR) issued by the San Francisco Bay Regional Water Quality Control Board (RWQCB) as a Class III solid waste disposal site. Most of the regulated landfill area has remained vacant and undeveloped, except for the Hanson Aggregates and former Pacific Cement sites. Hanson Aggregates leases and operates a sand and gravel yard on an unpaved area that is partially underlain by the regulated landfill; this area is east of the Pier 94 Backlands boundary. A partially paved, currently unoccupied facility (former Pacific Cement site) was constructed, with RWQCB approval, on another portion of the regulated landfill in 2005. This facility is also east of the Pier 94 Backlands boundary. Bode Concrete uses a small area within the regulated landfill for parking. Unpaved access roads to a concrete crushing facility run through the regulated landfill area. The rest of the regulated landfill is unoccupied and undeveloped.

3.0 PROJECT DESCRIPTION

The Port of San Francisco (Port) plans to improve approximately 23 acres of the vacant land at the Pier 94 Backlands to create 19 acres of leasable property (see project limit on Figure 2). The area to be improved includes a portion of the regulated landfill area. The site improvements include grading and leveling the site to accommodate leasing and installing new site infrastructure, consisting of a paved site access road and a storm water collection and treatment system, new water and sanitary sewer utilities for tenant parcels, and a new restroom facility. Site grading will involve placement of up to 18 feet of fill at some locations and constructing new roadways. To treat the storm water runoff, the site will be graded to collect overland flow around the perimeter of the entire site and direct flow to new vegetated swales. Within the regulated landfill area, vegetated swales are proposed only along Amador Street and Amador Street Extension. All site flows will be directed to the a vegetated swale before being discharged to the San Francisco Bay by means of a new storm water intake structure and outfall pipe and structure. The outfall structure will be about 10 feet by 14 feet in plan and bottom about 8 to 10 feet below grade and will be on the bank adjacent to Islais Creek Channel. The proposed improvements are all within the Pier 94 Backlands area as shown on Figure 2.

4.0 OBJECTIVE AND SCOPE OF SERVICES

As part of the design and construction of the Pier 94 Backlands improvements, the Port seeks to cover the existing regulated landfill portion of the Pier 94 Backlands in a manner that would meet the requirements of an alternative landfill cover. The proposed cover over the landfill area, to be constructed as part of the Pier 94 Backlands improvements, should meet regulatory agency (LEA and RWQCB) standards for alternative landfill cover and support the Port’s request to the RWQCB to rescind the WDR. We understand RWQCB staff has advised the Port that they are amenable to considering a request to rescind the WDRs with adequate technical justification, which demonstrates the proposed improvements reduce infiltration of storm water through the underlying fill material relative to existing conditions.

The geotechnical investigation was performed to evaluate (1) the subsurface conditions of the regulated landfill area within the Pier 94 Backlands and (2) geotechnical issues that may impact the proposed improvements. Our scope of services for Task 1 (Alternative Landfill Cover Design) consisted of a literature review of existing documents pertaining to the landfill and Pier 94 Backlands, including geotechnical investigations, geotechnical laboratory tests, and engineering analyses. We used the results of available documents, our field investigations, laboratory testing, and engineering analyses to develop conclusions and recommendations regarding:

- site history, waste characteristics, and disposal methods
- subsurface soil, refuse, and groundwater conditions
- geologic and seismic hazards
- settlement due to consolidation of Bay Mud and compression and decomposition of the refuse under existing and new loads
- percolation through existing cover, prescription cover, and engineered alternative covers
- landfill gas evaluation and control system
- site grading, subgrade preparation, and fill criteria and compaction requirements
- underground utilities
- foundations for new restroom, storm water intake, and outfall structures
- lateral earth pressures for below-grade walls
- hydrostatic pressures for outfall structure
- vegetated swale engineered alternative cover
- asphalt concrete pavement engineered alternative cover
- floor slabs
- temporary cut slopes and shoring
- construction considerations.

5.0 DOCUMENTS REVIEWED

We reviewed reports and documents pertaining to the site which were provided by the Port. The documents we revised were:

- Geo/Resource Consultants, Inc. "Data Compilation Report & Appendices A through E Data Logs, Volume 1 of 3," 2 January 1990. (GRC 1990)

- Harding Lawson Associates. " Geotechnical Report, Engineering Evaluation of Embarcadero Freeway Construction Debris, Pier 94 Solid Waste Disposal Site, San Francisco, California," 28 May 1993. (HLA 1993)
- The Mark Group. "Air Quality Solid Waste Assessment Test Report, Solid Waste Disposal Sites at Piers 94 and 98, Port of San Francisco, California," 3 January 1989. (TMG 1989)
- The Mark Group. "Phase 1A: Materials Inventory, Closure of Pier 94 Landfill, Port of San Francisco, California," 11 October 1991. (TMG 1991)
- Parsons Brinckerhoff Quade & Douglas, Inc. "Pier 90-94 Backlands Conceptual Development, Feasibility Report" September 2005. (PBQD 2005)

6.0 SITE HISTORY, WASTE CHARACTERISTICS, AND DISPOSAL METHODS

The Pier 94 Backlands was created during the 1960s and 1970s by constructing a perimeter debris dike and placing fill on the inboard side of the dike. The approximate locations of the debris dike and historic shorelines within the Pier 94 Backlands area and vicinity are shown on the Historic Site Plan, Figure 3. Descriptions of site history, fill type and waste characteristics, and fill placement and waste disposal methods are presented in this section: these descriptions are based on information presented in TMG (1989) and GRC (1990).

6.1 Debris Dike

The debris dike was constructed in 1961 to isolate 60 acres of the eastern portion of Pier 94. The debris dike was approximately 200 feet wide at the ground surface, about 5,500 feet long, and about 40 to 50 feet deep. The debris dike bottomed on Bay Mud. The debris fill reportedly consists of construction waste, including wood, paper, brick, plaster, concrete, and other inert materials, mixed with 30 to 40 percent soil. Previous exploratory borings by others have shown the presence of some layers containing no soil component. Mixing and compaction of debris fill was accomplished with a crawler tractor.

6.2 Dredge Spoils

In 1964, about 2.5 million cubic yards of Bay Mud dredged from the Army Street Terminal (Pier 80) construction site was hydraulically placed inboard of the debris dike. The dredge spoils were not compacted. The thickness of the dredge spoils has been estimated to be on the order of 20 to 30 feet.

6.3 Construction Debris

Between 1965 and 1975, an unknown quantity of construction and municipal waste was placed at the landfill over the dredge spoils. Construction debris contents are similar to those described for the debris dike, with the addition of miscellaneous municipal refuse (i.e. furniture, steel containers, discarded appliances, etc.). The degree of mixing and compaction of these materials varied across the site. Soil was occasionally incorporated into areas of the waste to facilitate dump truck access.

6.4 Soil Cap

After waste discharge operations had ended, a layer of soil (soil cap) was placed over the entire landfill in 1977. There is no documentation regarding the compaction of the soil cap.

7.0 FIELD INVESTIGATION AND LABORATORY TESTING

7.1 Borings

We performed a field investigation at the site that consisted of drilling 15 borings, designated borings B-1 through B-15, at the approximate locations shown on Figure 4. The field investigation was performed to develop additional geotechnical information regarding subsurface conditions.

Prior to commencing with the field investigation, we performed the following:

- prepared a Health and Safety (H&S) Plan (dated 19 April 2011) and Work Plan (dated 28 April 2011)
- obtained an encroachment permit from the Port and a drilling permit from the San Francisco Department of Public Health (SFDPH)
- marked the locations of the borings
- checked the boring locations for underground utilities by reviewing documents provided by the Port, contacting Underground Service Alert (USA), and retaining a private underground utility locator, and
- performed a H&S orientation training session conducted by our Site Safety Officer (SSO) with all field personnel.

As requested by the Port, borings B-1 and B-2 are located in the Hanson Aggregates and former Pacific Cement sites, respectively, which are within the regulated landfill area but outside of the Pier 94 Backlands. Borings B-3 and B-5 to B-11 are in the regulated landfill area that is part of the Pier 94 Backlands. Boring B-4 is located near the proposed restroom facility in Pier 94 Backlands and just south of the regulated landfill area. Borings B-12, B-13, and B-15 are near the proposed storm water outfall, intake, and manhole structures, respectively. Boring B-14 is located at the site of the new Amador Street sanitary pump station.

Borings B-1 to B-10 and boring B-13 were drilled on 25 and 26 May 2011 by Exploration Geoservices, Inc. of San Jose, California. These borings were drilled using a truck-mounted drill rig equipped with hollow-stem augers. Borings B-1 to B-10 were drilled to depths between 10 and 16.5 feet below ground surface (bgs) and terminated about five feet into construction debris. Boring B-13 was drilled to 33 feet bgs and terminated in the Bay Mud.

Borings B-11, B-12, B-14 and B-15 were drilled on 23 to 25 May 2011 by Pitcher Drilling Company of East Palo Alto, California. Borings B-11 and B-14 were drilled to depths of about 92 and 77-1/2 feet, respectively, using rotary-wash drilling method. Borings B-11 and B-14 were terminated in very dense sand underlying the Bay Mud. Borings B-12 and B-15 were drilled to depths of 43-1/2 and 31-1/2 feet bgs, respectively, using hollow-stem auger drilling method. Borings B-12 and B-15 terminated in the Bay Mud and dredge spoils, respectively.

Environmental soil samples were obtained from the upper 10 feet of boring B-12. Environmental soil samplers and liners were washed with diluted soap and water and double-rinsed prior to sampling. In addition, augers that were decontaminated in accordance with our Work Plan were used in drilling the upper 10 feet of boring B-12. Environmental samples obtained from the upper 10 feet of boring B-12 were transported to an analytical laboratory under chain of custody procedures and were analyzed as part of the environmental site investigation task of this project.

The borings were drilled under the direction of our field engineers who logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The logs of borings are presented on Figures A-1 through A-15 in Appendix A. Symbols and descriptions used on the logs are presented on Figure A-16.

Soil samples were obtained using four different types of samplers: two driven split-barrel samplers and two pushed thin-walled samplers. They were:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel tubes with an inside diameter of 2.43 inches.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners.
- Shelby Tube (ST) sampler with a 3.0-inch outside diameter and a 2.875-inch inside diameter.
- Dames & Moore Piston Sampler (D&M) with a 2.5-inch outside diameter and a 2.43-inch inside diameter.

The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soil. The ST and D&M samplers were used to obtain relatively undisturbed samples of soft to medium stiff cohesive soil.

For borings drilled by Pitcher Drilling, the SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling 30 inches. For borings drilled by Exploration Geoservices, the SPT and S&H samplers were driven with a 140-pound, downhole, wireline hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration.

For borings drilled by Pitcher Drilling (automatic hammer), the blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy and are shown on the boring logs. For borings drilled by Exploration Geoservices (downhole wireline hammer), S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, if the samplers are above the groundwater: S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.5 and 0.9, respectively, where the samplers are below the groundwater. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

The ST and D&M samplers are both pushed hydraulically into the soil; the pressure required to advance the sampler is shown on the logs, measured in pounds per square inch (psi).

Upon completion, the boreholes were backfilled with grout consisting of cement and water in accordance with the requirements of the SFDPH. The grouting was completed under the intermittent observation of a SFDPH Inspector. The soil cuttings from the borings were collected in 55-gallon drums which were stored temporarily at the site, tested, and eventually transported off-site for proper disposal.

7.2 Health and Safety Plan

Prior to the start of drilling, all field personnel had completed 40-hour H&S training in accordance with Federal OSHA/29 CFR and California OSHA/CCR Title 8 requirements. In addition, a H&S orientation training session for all field personnel was conducted by the SSO. The training session consisted of a review of the H&S Plan and a discussion of potential H&S hazards.

While drilling through the landfill, air monitoring was performed using a meter capable of measuring the lower explosive limit (LEL) of methane, organic compounds in parts per million (ppm), and percent oxygen (O_2) in the atmosphere. Also, a meter equipped with a photo-ionization detector (PID) was used to check for chemical compounds in the air within the work area. When the methane concentration at the borehole locations reached 10 percent of the LEL, our field engineer immediately stopped the drilling activities and monitored the air from a safe distance until it was safe to continue work. In one instance, the drilling work at boring B-4 was temporarily delayed until the methane gas concentrations dissipated to acceptable levels.

7.3 Laboratory Testing

Geotechnical laboratory testing was performed on selected samples obtained from the borings. Laboratory tests included moisture content, dry density, Atterberg limits, particle size distribution, shear strength, compressibility (consolidation), and hydraulic conductivity. Results of the laboratory tests are included on the boring logs and in Appendix B.

8.0 SITE AND SUBSURFACE CONDITIONS

8.1 Site Conditions

Site grade across the project site varies from about Elevation 3 feet[1] at Amador Street (northern limit of the site) to Elevation 22 feet at the southeast limit of the site. Near the location of the proposed storm water outfall structure at the bank of Islais Creek Channel, the site grade is at about Elevation 1 foot. The ground is relatively bare, except for sparse weed and shrubs. There are several mounds of soil stockpiled across the site that are on the order of 20 feet high. The existing Amador Street and Amador Street Extension at the northern and eastern limit of the site are paved roads.

At the western and southern limits of the site, there is an existing swale which supports facultative wetland vegetation, referred to in project drawings as "emerging wetlands", a north swale, and a south swale; the locations of the existing wetlands and swales are similar to the proposed locations shown on Figure 2. The site grades down to about Elevation -1 to -3 feet in the existing emerging wetland, north swale, and south swale. Within the emerging wetlands and north swale and the adjacent banks, the ground is heavily vegetated with shrubs and bushes. In the area of the south swale and adjacent banks, the ground is sparse to moderately vegetated with shrubs and bushes.

8.2 Subsurface Conditions

Three idealized subsurface profiles were prepared for the project site using the information from our borings and borings previously drilled by others and are presented on Figures 5 through 7. The approximate locations of the profiles are shown on Figure 4. Descriptions of the subsurface conditions at the site are presented in this section.

8.2.1 Regulated Landfill Area

The regulated landfill area is located east of the 1961 shoreline and is blanketed by fill (soil cap) consisting of loose to very dense sands and gravels with variable amounts of clay and silt and occasional concrete, brick, and serpentinite fragments. Where explored, the soil cap bottoms 2.5 to 8 feet bgs. Geotechnical laboratory test results for samples of soil cap material indicate hydraulic conductively values ranging between 8.0×10^{-4} and 1.3×10^{-7} centimeters per second (cm/sec).

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[1] All elevations are referenced to San Francisco City Datum.

The soil cap is underlain by construction debris consisting of construction and municipal waste mixed with soil. Where explored, the waste consisted of wood, concrete, asphalt, brick, rock fragments, metal fencing, sheet metal, plastic, and foam. The soil content and composition is highly variable. At borings B-11 and B-15, the bottom of construction debris is 19 to 20 feet bgs.

Beneath the construction debris is dredged spoils consisting of very soft to stiff clay with variable amounts of sand. Where explored, variable amounts of wood, concrete, and brick are embedded within the dredge spoils. At boring B-11 the dredge spoils bottom about 38 feet bgs.

Where explored (borings B-11 and B-15) the dredge spoils are underlain by medium stiff to stiff clay, locally known as Bay Mud. At boring B-11, the Bay Mud extends to about 89 feet bgs and is underlain by medium dense to very dense sand.

8.2.2 Project Site beyond Regulated Landfill Area

The project area west of the 1961 shoreline is outside of the regulated landfill area and is generally blanketed by fill to depths of 25 to 40 feet bgs. The fill, placed prior to 1961, is heterogeneous and consists of variable mixture of clay, silt, sand, and gravel, with occasional brick, concrete, and asphalt debris. The fill is underlain by soft to stiff Bay Mud to depths between 70 and 75 feet bgs (PBQD 2005). Beneath the Bay Mud is about 15 feet of dense to very dense sand underlain by stiff to hard clay (Old Bay Clay) to the maximum explored depth in boring B-1 drilled by Bechtel in 1994 (PBQD 2005).

At boring B-12, located near the bank of the Islais Creek Channel, we encountered debris dike fill to a depth of about 41 feet bgs. The debris dike is underlain by medium stiff Bay Mud to the maximum explored depth of 43-1/2 feet in boring B-12.

8.2.3 Groundwater

Groundwater was measured in borings B-12 and B-13 at a depth of about 10 feet bgs, corresponding to Elevation -8.5 feet; however, these measurements were obtained before the groundwater was allowed to stabilize. Groundwater was measured in boring B-11 at a depth of about 17.5 feet bgs, corresponding to Elevation -5 feet; this measurement was obtained by allowing the groundwater to stabilize overnight. Groundwater monitoring data obtained by GRC (1990) indicates groundwater may vary from Elevation -7 feet at the northern limit of the project site to Elevation -4 feet at the southern limit of the project site.

We expect the groundwater level at the site to fluctuate based on seasonal variations in rainfall. The groundwater level is likely to be influenced by changes in sea level and fluctuations of tides.

9.0 REGIONAL SEISMICITY

The major active faults in the area are the San Andreas, Hayward, and San Gregorio faults. These and other faults of the region are shown on Figure 8. For each of the active faults within 100 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[2] event [2007 Working Group on California Earthquake Probabilities (WGCEP 2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
N. San Andreas - Peninsula	12	West	7.2
N. San Andreas (1906 event)	12	West	8.1
Total Hayward	17	East	7.0
Total Hayward-Rodgers Creek	17	East	7.3
N. San Andreas - North Coast	18	West	7.5
San Gregorio Connected	19	West	7.5
Mount Diablo Thrust	33	East	6.7
Total Calaveras	34	East	7.0
Monte Vista-Shannon	36	Southeast	6.5
Rodgers Creek	38	North	7.1
Green Valley Connected	38	East	6.8
Point Reyes	45	West	6.9
West Napa	48	Northeast	6.7
Greenville Connected	51	East	7.0
Great Valley 5, Pittsburg Kirby Hills	56	East	6.7
Great Valley 4b, Gordon Valley	71	Northeast	6.8
N. San Andreas - Santa Cruz	72	Southeast	7.1
Great Valley 7	75	East	6.9
Hunting Creek-Berryessa	80	North	7.1

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[2] Moment magnitude is an energy-based scale that provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Zayante-Vergeles	82	Southeast	7.0
Great Valley 4a, Trout Creek	94	Northeast	6.6
Monterey Bay-Tularcitos	95	Southeast	7.3
Maacama-Garberville	96	North	7.4

Figure 8 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 9) occurred east of Monterey Bay on the San Andreas Fault.[3] The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6-1/4. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7-1/2. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 430 kilometers long. It had a maximum intensity of XI (MM), a M_w of about 7.9 and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles.
The most recent major earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with a M_w of 6.9. The epicenter of the earthquake was in the Santa Cruz Mountains, approximately 90 km from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 30-year probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area to be about 63 percent. More specific estimates of the probabilities for different faults in the Bay Area are shown in Table 2.

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[3] Toppozada, T.R. and Borchardt G., 1998, Re-Evaluation of the 1836 "Hayward Fault" and the 1838 San Andreas Fault earthquakes, Bulletin of Seismological Society of America, 88(1), 140-159.

TABLE 2

WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward – Rodgers Creek	31
North San Andreas	21
Calaveras	7
San Gregorio Connected	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

10.0 DISCUSSIONS AND CONCLUSIONS

On the basis of our geotechnical investigation, we conclude that the proposed Pier 94 Backlands improvements are feasible from a geotechnical engineering standpoint, provided appropriate landfill closure and post-closure maintenance plans are integrated into the development plans. The primary concerns in developing the site as proposed include:

- seismic hazards
- landfill cover design within regulated landfill area
- water infiltration through the existing and proposed landfill cover within the regulated landfill area
- settlement due to consolidation of underlying Bay Mud and compression and decomposition of the refuse
- landfill gas migration, detection, and control systems, and
- foundation design for new structures.

Our conclusions regarding these concerns are discussed in the remainder of this section.

10.1 Seismic Hazards

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,[4] lateral spreading,[5] and cyclic densification.[6] We used the results of our geotechnical investigation to evaluate these potential hazards.

10.1.1 Fault Rupture

Historically, ground surface displacements closely follow the traces of geologically young faults.
We reviewed published maps and concluded the project site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We concluded the risk of surface faulting at the site is low.

10.1.2 Liquefaction and Associated Hazards

The project site is in a liquefaction seismic hazard zone as defined by the California Division of Mines and Geology (CDMG, now California Geological Survey [CGS]) map titled *State of California Seismic Hazard Zones, City and County of San Francisco, Official Map*, dated 17 November 2001 (Figure 10). This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990. There is no documented historical occurrence of liquefaction within the project limit[7]. We evaluated the potential for liquefaction to occur at the site in accordance with Special Publication 117A, *Guidelines for Evaluating and Mitigating Seismic Hazards Zones in California*, dated 11 September 2008.

The level of ground shaking that may occur at the site during future earthquakes is uncertain because the location, recurrence interval, and magnitude of future earthquakes are not known. However, a

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[4] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[5] 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.

[6] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing differential settlement.

[7] Published liquefaction susceptibility map for the central Bay Area from the 1906 San Francisco and 1989 Loma Prieta earthquakes, prepared by USGS in a cooperative project with California Geological Survey. http://geomaps.wr.usgs.gov/sfgeo/liquedfaction/effects.html.

design peak ground acceleration (PGA) of 0.36 times gravity (0.36g) was used in our liquefaction analysis. This PGA was calculated using the procedures specified in the 2010 San Francisco Building Code for the Design Earthquake and soil profile type S_E. We assumed an earthquake magnitude of 8.1, which is the mean characteristic moment magnitude for a repeat of 1906 earthquake on the San Andreas Fault as shown in Table 1.

Based on the data developed from this geotechnical investigation, we conclude there are random and isolated layers of loose to medium dense, saturated sandy soil lenses within the construction debris and debris dike layers that will potentially liquefy during a major earthquake event. Soil liquefaction could result in ground failures, such as lurch cracking, ejection of liquefied soil to the ground surface, and liquefaction-induced ground deformation.

We evaluated the potential for various types of liquefaction-induced ground failures. We estimate the potential for lurch cracking and the ejection of liquefied soil to the ground surface is low because the potentially liquefiable soil layers, which are up to about three-foot thick, are confined beneath about 10 to 15 feet of non-liquefiable fill. However, our evaluation of liquefaction-induced ground surface settlement indicates that settlement can occur as the excess pore pressure in the liquefied soil dissipates. We estimate the ground surface may settle up to one inch after a major earthquake. Differential settlement is estimated to be less than 1/2 inch across a 30-foot distance.

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open cut, or in the direction of a regional slope or gradient. Because the potentially-liquefiable soil at the site is discontinuous, we conclude the potential for lateral spreading at the site is low, except at the debris dike. Our boring B-12, located on the debris dike, encountered loose to medium dense sandy soil below the groundwater that is susceptible to liquefaction. Considering the outboard side of the debris dike is an unsupported face, lateral spreading could occur at the debris dike if the underlying potentially liquefiable soil layers are continuous. We judge lateral spreading, if it occurs at the debris dike, will likely result in lateral ground displacement on the order of several inches to several feet and will likely damage the new storm water outfall pipeline and structure.

10.1.3 Cyclic Densification

We used the results of our field investigation and laboratory testing to evaluate the potential for cyclic densification and liquefaction within the site. The results of our analyses indicate that the soil encountered above the groundwater level is relatively dense or contain relatively high fines content, therefore ground settlement resulting from differential compaction should be small (less than 1/4 inch).

10.2 Settlement Considerations

The project site will undergo settlement caused by the compression and decomposition of refuse (construction debris and dredge spoils), and consolidation of the Bay Mud beneath the site, due to the weight of existing and proposed (new) fill. We have estimated settlement in the next 50 years due to existing and new fill at 17 locations across the site. The 17 locations, designated as settlement points 1 through 17, are shown on Figure 11. Methodology and results of our settlement analyses are presented in Appendix C and summarized in this section.

10.2.1 Bay Mud Consolidation

We evaluated ground settlement as a result of consolidation of the Bay Mud due to existing and new fill. Existing fill consists of historic fill placed west of the 1961 shoreline, debris dike, dredge spoils, construction debris, and soil cap. Placement of the existing fill was completed between 1946 and 1977. Results of our settlement analyses indicate consolidation of Bay Mud due to existing fill is 70 to over 95 percent complete. For the proposed improvements, up to 18 feet of new fill will be placed to reach finish grade. Assuming the new fill will be placed over a one year period, we estimate up to about three feet of ground settlement will occur in the next 50 years as a result of Bay Mud consolidation due to the weight of the existing and new fill.

For each of the 17 locations, the existing ground elevation, existing Bay Mud thickness, existing fill thickness, proposed finish grade and new fill thickness, and future Bay Mud settlement are summarized in Table 3.

TABLE 3

Settlement from Primary Consolidation of Bay Mud due to Existing and New Fill

Settlement Point	Existing Ground Elevation[8] (feet)	Existing Bay Mud Thickness (feet)	Existing Fill Thickness (feet)	Finish Grade Elevation[9] (feet)	New Fill Thickness (feet)	Settlement due to Primary Consolidation of Bay Mud in Next 50 Years (feet)
1	1.5	35.5	41.0	1.5	0.0	0.0
2	5.0	49.5	30.5	5.0	0.0	0.6
3	0.0	50.0	25.0	0.0	0.0	0.5
4	10.0	49.5	35.5	28.0	18.0	3.0
5	20.0	49.5	45.5	32.0	12.0	2.2
6	21.0	39.5	46.5	31.0	10.0	1.4
7	15.0	35.0	35.0	29.0	14.0	1.4
8	0.0	35.0	20.0	12.5	12.5	1.9
9	0.0	40.0	25.0	10.0	10.0	1.8
10	0.0	49.5	25.5	5.5	5.5	1.5
11	0.0	49.5	25.5	7.0	7.0	1.8
12	0.0	49.5	25.5	5.5	5.5	1.5
13	0.0	49.5	25.5	0.0	0.0	0.6
14	10.0	49.5	35.5	10.0	0.0	0.7
15	16.0	40.0	41.0	27.0	11.0	1.6
16	10.0	49.5	35.5	24.0	14.0	2.4
17	5.0	49.5	30.5	5.0	0.0	0.6

The settlements presented in Table 3 are from primary consolidation of Bay Mud only; secondary compression settlement may also occur. Historical data in the vicinity indicates additional settlement of 1/2 to 1 inch may occur every 10 years as a result of secondary compression of Bay Mud.

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[8] Existing ground elevation is based on site topographic survey provided electronically by Port of San Francisco on 8 April 2011.

[9] Proposed finish grade from 30 percent design plan, titled "Pier 94 Backlands Improvements, Proposed Grading and Storm Drainage Plan, Drawing No. R-2" dated December 2009, prepared by Port of San Francisco, Department of Engineering.

10.2.2 Refuse Settlement

The refuse encountered in the regulated landfill area will settle due to the weight of refuse (self-weight), existing soil cap, and new fill. The refuse encountered includes debris dike, dredge spoils, and construction debris. Within the project limit, refuse was encountered in borings drilled near settlement points 1, 2, 4, 5, 6, 14, 15, and 17. We performed settlement analyses at these locations to estimate ground settlement as a result of primary and secondary consolidation of the refuse using two refuse settlement models: (1) Gibson and Lo, and (2) Sowers. Detailed discussion of our refuse settlement analyses is presented in Appendix C. The results of our analyses are summarized in this section.

Gibson and Lo Settlement Model

We estimated settlements using a model developed by Gibson and Lo and refuse settlement parameters provided by Edil et al. (1990). The Gibson and Lo model considers parameters for primary consolidation and secondary compression. Primary consolidation generally includes the bending, crushing, and reorientation of refuse material and the movement of finer grained materials into larger voids. Settlement associated with primary consolidation typically occurs between 1 and 5 years after the initial application of the load. Secondary compression is associated with refuse settlements that occur gradually over time. Secondary compression is generally attributable to the physical-chemical change of the refuse materials, such as corrosion and oxidation, and the bio-chemical decomposition of organic refuse material through aerobic and anaerobic fermentation and decay. The majority of secondary compression is typically completed within 30 to 50 years after the initial application of the load (Sharma and Lewis 1994).

The results of our Gibson and Lo settlement analysis indicate that ongoing landfill settlement due to the self-weight of refuse and weight of the existing fill is essentially complete. Due to the age and low organic content of the refuse constituents, we judge the amount of on-going settlement due to refuse self-weight and existing fill will be relatively small and future settlement due to new fill will occur quickly.

Project plans indicated up to 18 feet of new fill will be placed within the regulated landfill area of the project site. Placement of new fill will initiate a new cycle of consolidation settlement of the refuse. We estimated refuse settlement due to the new fill at settlement points 4, 5, 6, and 15; no new fill will be placed at settlement points 1, 2, 14, and 17.

Edil et al. (1990) concluded the Gibson and Lo model predicts settlement with an accuracy between 2 to 20 percent of the actual settlement. Therefore, we increased our computed settlement using Gibson and Lo model by 20 percent. A summary of refuse settlement calculated using the Gibson and Lo settlement model is presented in Table 4.

TABLE 4

Refuse Settlement Resulting from the Placement of New Fill

Settlement Point	Existing Ground Elevation (feet)	Finish Grade Elevation (feet)	New Fill Thickness (feet)	Gibson and Lo Settlement[10] (feet)	Sowers Settlement (feet)
1	1.5	1.5	0.0	0.0	0.0
2	5.0	5.0	0.0	0.0	0.0
4	10.0	28.0	18.0	2.4	2.9
5	20.0	32.0	12.0	1.9	2.1
6	21.0	31.0	10.0	1.6	1.5
14	10.0	10.0	0.0	0.0	0.0
15	16.0	27.0	11.0	1.7	1.5
17	5.0	5.0	0.0	0.0	0.0

Sowers Settlement Model

We also performed refuse settlement calculations using the Sowers settlement model. Similar to the Gibson and Lo model, the Sowers model requires the selection of appropriate parameters for estimating refuse settlement. The Sowers model assumes the refuse is normally consolidated and the refuse undergoes primary consolidation under self-weight and the weight of new fill.

The Sowers model estimates that ground surface settlement associated with primarily consolidation of refuse will occur quickly during site grading because of the high permeability of landfill debris. Secondary compression occurs more slowly, but accounts for only a relatively small portion (less than five percent) of the total estimated settlement.

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[10] Computed settlement using the Gibson and Lo model has been increased by 20 percent.

We estimated refuse settlement associated with ongoing settlement due to the self-weight of refuse and weight of the existing soil cap. Results of our analyses indicate that ongoing refuse settlement due to the self-weight of refuse and weight of the existing soil cap is essentially complete. We also estimated refuse settlement associated with the placement of new fill across the regulated landfill area within the project site. A summary of the settlement estimates computed using the Sowers model is presented in Table 4.

10.2.3 Settlement and Design Considerations

Total settlement from Bay Mud and refuse due to existing and new fill are presented in Table 5 for settlement points 1 through 17. The total settlement is expected to occur over a period of 50 years from the time of new load application. Actual settlement will depend upon the consolidation history and thickness of Bay Mud and refuse, and thickness of fill placed at any given location. Because of the heterogeneity of the refuse, it is difficult to accurately predict the amount of settlement of a given period of time. These estimates should be used as a guide and could vary several inches.

TABLE 5

Estimated Total Settlement for 50-Year Period from Time of Load Application

Settlement Point	Existing Ground Elevation (feet)	Finish Grade Elevation (feet)	New Fill Thickness (feet)	Settlement from Bay Mud (feet)	Settlement from Refuse, Gibson and Lo Model (feet)	Total Settlement from Bay Mud and Refuse (feet)
1	1.5	1.5	0.0	0.0	0.0	0.0
2	5.0	5.0	0.0	0.6	0.0	0.6
3	0.0	0.0	0.0	0.5	0.0	0.5
4	10.0	28.0	18.0	3.0	2.4	5.4
5	20.0	32.0	12.0	2.2	1.9	4.1
6	21.0	31.0	10.0	1.4	1.6	3.0
7	15.0	29.0	14.0	1.4	0.0	1.4
8	0.0	12.5	12.5	1.9	0.0	1.9
9	0.0	10.0	10.0	1.8	0.0	1.8
10	0.0	5.5	5.5	1.5	0.0	1.5
11	0.0	7.0	7.0	1.8	0.0	1.8

Settlement Point	Existing Ground Elevation (feet)	Finish Grade Elevation (feet)	New Fill Thickness (feet)	Settlement from Bay Mud (feet)	Settlement from Refuse, Gibson and Lo Model (feet)	Total Settlement from Bay Mud and Refuse (feet)
12	0.0	5.5	5.5	1.5	0.0	1.5
13	0.0	0.0	0.0	0.6	0.0	0.6
14	10.0	10.0	0.0	0.7	0.0	0.7
15	16.0	27.0	11.0	1.6	1.5	3.1
16	10.0	24.0	14.0	2.4	0.0	2.4
17	5.0	5.0	0.0	0.6	0.0	0.6

The settlements presented in Table 5 do not include settlement from secondary compression of Bay Mud. As previously discussed, additional settlement of 1/2 to 1 inch may occur every 10 years as a result of secondary compression of Bay Mud. During final design, total settlement from consolidation of Bay Mud and compression of refuse should be re-evaluated based on proposed final grades and construction schedule.

10.3 Hydrologic Evaluation of Landfill Cover Performance

We evaluated the hydrologic performance of the existing soil cap and proposed landfill covers using the computer program titled *Hydrologic Evaluation of Landfill Performance* (HELP-3)[11]. The program was developed for water balance analysis of landfill cover systems for solid waste disposal and containment facilities.

HELP-3 is a quasi-two-dimensional hydrologic model of water movement across, into, through and out of landfills. HELP-3 uses weather and cover design data, and evaluates the amount of percolation through landfill cover systems by taking into account of surface storage, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, and lateral subsurface drainage. Daily infiltration into the landfill is determined indirectly from a surface water balance. Infiltration is assumed to equal the sum of rainfall and surface storage minus the sum of runoff and evaporation of surface water.

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[11] Hydrologic Evaluation of Landfill Performance, HELP Model Version 3.07 (1 November 1997), developed by Environmental Laboratory USAE Waterways Experiment Station for USEPA Risk Reduction Engineering Laboratory. (HELP-3)

The first subsurface processes considered are soil evaporation and plant transpiration from the evaporative zone. The other subsurface processes (vertical drainage, lateral drainage, and percolation) are modeled one layer at a time.

We performed the HELP-3 analyses to estimate the amount of percolation through three landfill cover systems: 1) existing, 2) prescriptive, and 3) engineered alternative, as described below:

- For the existing cover system, we estimated percolation through the existing soil cap at each of the 12 borings located within the regulated landfill area.
- The prescriptive cover is a standard design prescribed in Title 27 of the California Code of Regulations. The prescriptive cover consists of a 24-inch foundation layer, a 12-inch low hydraulic conductivity (less than $1x10^{-6}$ cm/sec) layer, and a 12-inch erosion-resistant layer. We conclude the existing soil cap within the regulated landfill area meets the requirement of the 24-inch foundation layer. The existing cover encountered at borings B-2 and B-10 locations were modeled as the foundation layer for the prescriptive cover; these boring locations represent areas explored with the lowest and highest average annual percolation. Prescriptive 12-inch low conductivity layer and 12-inch erosion-resistant layer were modeled above the existing cover (foundation layer) at borings B-2 and B-10.
- Two engineered alternative covers being considered are: (1) a vegetated swale consisting of 12 inches of vegetation soil layer underlain by a low hydraulic conductivity geomembrane liner that is placed directly over the existing cover, and (2) asphalt concrete pavement section consisting of asphalt concrete over aggregate base underlain by a low hydraulic conductivity geomembrane liner. The vegetated swale engineered alternative was modeled above the existing cover encountered at borings B-3, B-5, and B-6; located along the proposed vegetated swale. The asphalt-concrete pavement engineered alternative was modeled above the existing cover encountered at borings B-2 and B-10; these locations represent areas explored with the lowest and highest average annual percolation through the existing cover.

The existing, prescriptive, and engineered alternative covers were analyzed to determine percolation due to precipitation. Each of the cases was evaluated for a 30-year period. Details of the HELP-3 analyses are presented in Appendix D. A summary of the results of our HELP-3 analyses, including average annual percolation through the existing cover, prescriptive cover, and engineered alternative covers are presented in Table 6.

TABLE 6

HELP-3 Results

Cover	Boring Location	Average Annual Precipitation (inches)	Average Annual Runoff (inches)	Average Annual Evaporation (inches)	Average Annual Percolation (inches)
Existing Cover	B-1	20.3	3.2	6.8	10.4
	B-2	20.3	6.6	11.0	2.7
	B-3	20.3	4.0	3.2	13.1
	B-4	20.3	6.6	11.0	2.7
	B-5	20.3	6.6	10.9	2.9
	B-6	20.3	6.6	10.9	2.9
	B-7	20.3	4.5	10.5	5.4
	B-8	20.3	6.6	11.0	2.7
	B-9	20.3	5.0	11.7	3.6
	B-10	20.3	0.0	7.2	13.1
	B-11	20.3	6.6	10.9	2.9
	B-15	20.3	4.6	10.9	4.8
Prescriptive	B-2	20.3	0.1	11.9	8.4
	B-10	20.3	0.1	11.9	8.4
Vegetated Swale	B-3	20.3	5.2	14.1	1.1
	B-5	20.3	4.6	14.0	1.7
	B-6	20.3	4.6	14.0	1.8
Asphalt Concrete Pavement	B-2	20.3	14.9	3.0	2.3
	B-10	20.3	10.3	2.9	7.2

Results of the HELP-3 analyses indicate that average annual percolation through the existing cover is between 2.7 and 13.1 inches. For the prescriptive cover, average annual percolation through the cover will be approximately 8.4 inches for B-2 and B-10 locations. For existing cover where the upper 18 inches consists of silty or clayey sand material with hydraulic conductivity of 1x10^{-6} cm/sec or less, such as B-2, B-4, B-5, B-6, B-8, and B-11 locations, the average annual percolation through the cover is less than 3 inches, which is lower than those for the prescriptive cover.

For the vegetated swale engineered alternative cover, average annual percolation through the cover will be approximately 1.1, 1.7, and 1.8 inches for B-3, B-5, and B-6, respectively, which are lower than those for the existing conditions. For the asphalt concrete pavement engineered alternative, the average annual percolation through the cover will be approximately 2.3 and 7.2 inches for B-2 and B-10, respectively, which are lower than those for the existing conditions.

The results of the HELP-3 analyses indicate that the average annual percolation through the engineered alternative covers is less that the average annual percolation through the existing and prescriptive covers.

10.4 Landfill Gas Evaluation

A Solid Waste Assessment Test (SWAT) of air quality was conducted at the site in 1988 (TMG 1989 and GRC 1990). The air quality SWAT generally consisted of:

- Wind speed and direction monitoring prior to and during the sampling, in order to provide a basis for the proposed number of samples and sample locations, as well as verify appropriate sampling conditions.
- Collection of subsurface gas samples at five locations within the landfill. Subsurface gas samples were collected about six feet bgs. Subsurface gas samples were collected to characterize the gas stream produced by wastes at the former landfill.
- Collection of an integrated air surface sample at one location within the landfill. The integrated air surface sample was collected about three inches above the ground surface over a 25-minute period. The integrated air surface sample was collected to characterize the gas stream produced by the former landfill wastes and landfill gas (LFG) emissions immediately after having passed through the surface cover.
- Subsurface gas and integrated air surface samples were analyzed for the 10 Calderon[12] specific constituents: the volatile organic compounds (VOCs) vinyl chloride, benzene,

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[12] "Calderon" Air SWAT Program = 1984 amendments to California Health and Safety Code Section 41805.5 which require a SWAT test of air quality at active solid and hazardous waste disposal sites.

dichloromethane, tetrachloroethylene (PCE), trichloroethylene (TCE), tetrachloromethane, trichloromethane, 1,1,1-trichloroethane, 1,2-dibromoethane, and 1,2-dichloroethane.

- Subsurface gas samples were additionally analyzed for methane, total hydrocarbons, and the fixed gases oxygen, carbon dioxide, and nitrogen.

The results of the SWAT investigation (TMG 1989 and GRC 1990) were as follows:

- Subsurface gas samples:
    - Carbon tetrachloride was detected in all five samples, as well as the field blank, at a concentration of 0.3 parts per billion by volume (ppbV), equivalent to 1.89 micrograms per cubic meter (µg/m^3). The shallow soil gas commercial/industrial Environmental Screening Level (ESL[13]) for carbon tetrachloride is 63 µg/m^3.
    - PCE was detected in four of the five samples at concentrations ranging from 1.8 ppbV to 2.89 ppbV (12.21 µg/m^3 to 19.6 µg/m^3). The ESL for PCE is 410 µg/m^3.
    - Benzene was detected in two of five samples at concentrations of 3.4 ppbV (10.86 µg/m^3) and 4.8 ppbV (15.33 µg/m^3). The ESL for benzene is 63 µg/m^3.
    - TCE was detected in one of five samples at a concentration of 3.7 ppbV (19.88 µg/m^3). The ESL for TCE is 1,200 µg/m^3.
    - Concentrations of fixed gases, consisting of oxygen (22.1 ppbV to 22.2 ppbV, or 28.92 µg/m^3 to 29.06 µg/m^3), carbon dioxide (0.037 ppbV to 0.048 ppbV, or 0.0666 µg/m^3 to 0.0864 µg/m^3), and nitrogen (78.8 ppbV to 79.2 ppbV, or 90.27 µg/m^3 to 90.73 µg/m^3) were typical of ambient air concentrations. No ESLs have been established for oxygen, carbon dioxide, or nitrogen.
    - Concentrations of other VOCs, methane, and total hydrocarbons were not detected above method detection limits.
- Integrated air surface sample:
    - Carbon tetrachloride was detected at a concentration of 0.3 ppbV (1.89 µg/m^3).

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[13] ESL values cited from *Table E. Environmental Screening Levels (ESLs), Indoor Air and Soil Gas (Vapor Intrusion Concerns),* in the *Interim Final-Screening for Environmental Concerns at Sites with Contaminated Soil and Groundwater*, by the San Francisco Bay Regional Water Quality Control Board, dated November 2007 and revised May 2008. Values used are the Shallow Soil Gas Screening Levels for Commercial/Industrial Land Use Only.

    - Benzene was detected at a concentration of 14.3 ppbV (45.68 µg/m^3).
    - Other VOCs were not detected above method detection limits.
- Other observations:
    - Wind speed was within the allowable limits during the sampling (less than five miles per hour), and generally blew from the west/northwest.
    - In-situ pressure monitoring during the subsurface gas sampling did not detect a pressure buildup at any sample location.
    - Screening with a portable flame ionization detector (FID) during the subsurface gas sampling did not detect organic gases above 0.0002%.

The TMG (1989) and GRC (1990) reports concluded that:

- Based on the ubiquitous presence of carbon tetrachloride at a concentration of 0.3 ppbV (1.89 µg/m^3) in all samples, including the field blank, it was not likely present in the LFG;
- The benzene concentration identified in the integrated surface sample (14.3 ppbV, or 45.68 µg/m^3) at a greater concentration than the subsurface gas samples (3.4 ppbV and 4.8 ppbV, or 10.86 µg/m^3 and 15.33 µg/m^3) was attributed to mobile sources during the sampling period; and
- Overall, the results indicated an absence of gases generated by anaerobic decomposition.

Based on a review of the available reports, concentrations of detected VOCs were below their respective commercial/industrial ESLs in all samples. The waste disposal site does not appear to be a potential source for VOCs or methane capable of adversely affecting ambient air quality.

11.0 RECOMMENDATIONS

Recommendations for site preparation and fill placement, underground utility design and trench backfill, foundation design, temporary cut-slopes and shoring, hydrostatic and earth pressures, engineered alternative covers, and other geotechnical aspects of this project are presented in this section.

11.1 Site Grading and Fill Placement

In areas to receive new fill, the surface should be stripped of existing pavement and vegetation. The surface exposed by stripping should be scarified to a depth of at least six inches, moisture-conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction[14]. The upper six inches of the subgrade beneath any new proposed hardscape and pavement areas should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction and the subgrade should be non-yielding. The exposed ground surface should be kept moist during subgrade preparation.

General and engineered fill (fill) may consist of onsite or imported material. Imported material and utility trench backfill should be non-hazardous, non-corrosive, free of organic matter, contains no rocks or lumps larger than four inches in greatest dimension, has a liquid limit less than 40 and a plasticity index of 12 or less, and be approved by the geotechnical engineer. Crushed recycled concrete may be used as general fill, provided the concrete is processed to less than four inches in greatest dimension and is acceptable from an environmental standpoint. Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. Fill deeper than five feet, or containing less than 10 percent fines should be compacted to at least 95 percent relative compaction.

All fill material, including onsite fill, should be submitted to the Geotechnical Engineer for approval at least three working days before it is used on site. For imported fill, the grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the proposed fill material is free of hazardous materials at least three days before use at the site. If these data are not provided, up to two weeks may be required to perform any required analytical testing on proposed import soil. Bulk samples of all soil materials should be provided to the Geotechnical Engineer at least three working days before use at the site so a compaction curve and/or gradation analysis can be obtained.

11.2 Underground Utilities

New underground utilities are expected to settle with the ground due to consolidation of Bay Mud and compression of refuse underlying the site. The magnitude of settlement along proposed utility alignments will vary depending upon several factors, including the thickness and compressibility of refuse and Bay Mud, load history, and amount of new fill to be placed. Utility line alignments should be preliminarily designed based on anticipated future settlement estimates presented in Section 10.2 of this

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[14] 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 D1557-09 laboratory compaction procedure.

report. When final grading and utility layout plans become available, we should review the plans and perform additional analyses, as appropriate, to check settlement along underground utility alignments. Utility connections should be designed to accommodate the anticipated settlements.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered with six inches of sand or fine gravel, which should then be mechanically tamped to at least 90 percent relative compaction. Trench backfill should be compacted as recommended in Section 11.1. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements resulting in damage to the pavement section.

11.3 Foundation Design and Settlement

11.3.1 New Restroom Facility

Current design plan shows about 12 feet of new fill to be placed to reach finish grade at the new restroom facility. Placement of 12 feet new fill will result in up to about four feet of ground settlement over a 50-year period due to consolidation of the underlying refuse and Bay Mud. To mitigate the detrimental effects of erratic settlement across the footprint of the new restroom structure, we recommend the structure be supported on a mat foundation bearing on engineered fill. A mat foundation can reduce the anticipated potential differential settlement across the building by distributing static loads more evenly. The amount of differential settlement within the building will depend on the stiffness of the foundation system and its ability to redistribute the load across the foundation.

We recommend the mat be designed for an allowable bearing pressure of 1,500 pounds per square foot (psf) for dead plus live load conditions; the allowable bearing pressure may be increased by one third for total loads including wind/seismic loads. For design of the mat using the subgrade modulus method, we recommend using a subgrade modulus of 12 kips per cubic foot (kcf).

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the mat foundation and friction along the base of the mat foundation. We recommend passive resistance be calculated using an equivalent fluid weight of 300 pounds per cubic foot (pcf). The upper one foot of soil should be ignored unless it is confined by a concrete slab. We recommend frictional resistance should be computed using a base friction coefficient of 0.3. The passive pressure and frictional resistance values include a factor of safety of at least 1.5.

11.3.2 Storm Water Structures

The proposed storm water conveyance system includes an intake at the north end of the proposed emerging wetlands, a manhole at the intersection of Amador Street and Amador Street Extension, and an outfall structure at the shoreline adjacent to Islais Creek Channel. Current grading plans show that no new fill will be placed at the locations of the intake, manhole, and outfall structures; therefore, we anticipate future settlement at these locations will from consolidation of Bay Mud due to the weight of existing fill. We estimated primary consolidation of the Bay Mud of about 6 to 7 inches at the proposed storm water intake and manhole structures, and to be less than one inch at the outfall structure. When final grading plans becomes available, we should confirm final site grades and perform analyses, as appropriate, to check settlement at these locations.

We recommend the intake and outfall structures be supported on a mat bearing on soil subgrade where the upper 12 inches is compacted to at least 95 percent relative compaction. Allowable bearing pressure and subgrade modulus provided in Section 11.3.1 for the new restroom facility may be used for design of the storm water intake and outfall structures.

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the mat foundation and friction along the base of the mat foundation. We recommend passive resistance be calculated using an equivalent fluid weight of 240 and 140 pcf above and below the groundwater table, respectively (see Section 11.5 for design groundwater table). The upper one foot of soil should be ignored unless it is confined by a concrete slab. We recommend frictional resistance should be computed using a base friction coefficient of 0.3. The passive pressure and frictional resistance values include a factor of safety of at least 1.5.

We anticipate the outfall structure will be underlain by debris dike material consisting of clayey sand with variable amounts of concrete, brick and wood debris. The intake structure will likely be underlain by fill consisting of medium dense clayey gravel or soft clay. If soft clay, weak soil, or other unsuitable material (i.e. wood or plastic) is encountered at the bottom of intake or outfall structures, the weak soil and unsuitable material should be removed and the overexcavation should be backfilled with engineered fill or lean concrete.

11.4 Seismic Design

As discussed in Section 10.1 of this report, the site is underlain by random and isolated layers of loose to medium dense, saturated sandy soil that will potentially liquefy during a major earthquake event. For design in accordance with the 2010 San Francisco Building Code (SFBC), the soil profile type would be a S_F. However, if the proposed structure has a period of 0.5 seconds or less, FEMA 368 and ASCE 7-05 do not require site-specific evaluations. Hence, for a structure with a period of 0.5 seconds or less, such as the proposed restroom facility, we recommend a soil profile type S_E be used. We also recommend the following seismic design parameters:

- spectral acceleration values S_s and S_1 of 1.500 and 0.647, respectively
- site coefficients F_a and F_v of 0.9 and 2.4, respectively
- Maximum Considered Earthquake (MCE) spectral acceleration values S_{Ms} and S_{M1} of 1.350 and 1.553, respectively, and
- Design Earthquake (DE) spectral acceleration values S_{Ds} and S_{D1} of 0.900 and 1.036, respectively.

11.5 Design Groundwater and Hydrostatic Uplift

Groundwater was encountered between Elevations -4 and -7 feet bgs during our investigation and previous investigations by GRC (1990). We recommend a design groundwater at Elevation -5 feet for evaluating hydrostatic pressures on storm water intake, manhole, and outfall structures.

Where new structures will extend below the groundwater level, they will need to be designed to resist hydrostatic uplift loads associated with a design groundwater level. We recommend a factor of safety of at least 2.0 be used for permanent uplift. Uplift loads may be resisted by the weight of the structure and any overlying soil. We recommend the soil weight be calculated using unit weights of 63 pcf for soil below the design groundwater table and 120 pcf for soil above the design groundwater table. If additional uplift resistance is needed, tiedown anchors may be used. We can provide recommendations for tiedowns should it be determined they are needed.

11.6 Below-Grade Walls

Below-grade walls, including the walls for the outfall structure, are anticipated to be less than 12 feet in height. Wall less than 12 feet in height should be designed for at-rest earth pressures using an equivalent fluid weight of 60 pcf. A backdrain should be provided to prevent the buildup of hydrostatic pressure. Where retaining walls are not backdrained, the walls should be designed for at-rest earth plus hysdrostatic pressures using an equivalent fluid weight of 95 pcf. Where walls are within 10 feet of adjacent roadways, the wall should be designed for additional traffic surcharge consisting of a uniform (rectangular distribution) lateral pressure of 100 psf, applied to the portion of the wall within 10 feet of the ground surface.

Walls may be supported on continuous footings at least 18 inches wide or a mat. To limit total settlement of walls supported on footings to less than one inch, we recommend the wall footings be designed using an allowable bearing pressure of 500 psf. Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of footings and the supporting soil. For passive resistance we recommend using an equivalent fluid weight of 240 and 140 pcf for above and below groundwater table, respectively. Frictional resistance should be computed using a base friction coefficient of 0.3. These values include a factor of safety of 1.5.

11.7 Temporary Cut Slope and Shoring

Excavations of varying depths will be required for the different elements of the project; e.g. new water and sanitary utility excavations will be on the order of 3 to 5 feet, and storm water intake and outfall pipeline and structures will be on the order of 5 to 10 feet. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). The shoring designer should be responsible for the design. The contractor should be responsible for the construction and safety of the temporary shoring.

The sides of excavation may be sloped where space permits and where the excavation is above groundwater. To prevent sloughing of surficial soils, we recommend temporary construction slopes not exceed inclinations greater than 1.5:1 (horizontal to vertical).

Where sloping of the excavation is not feasible, temporary shoring will be required to laterally restrain the sides of the excavation. For water and sanitary sewer utility trenches, the most appropriated shoring system for this project will likely be trench plates. For the storm water intake and pipelines, we recommend temporary shoring to consist of trench plates, trench boxes, or sheet piles. Because of the loose nature of the surficial fill, sheet piles, if used, should not be vibrated into place because of the potential for vibration-induced settlement. Instead, they should be pushed down.

Construction of the outfall structure will also require temporary sloping or shoring. If the excavation for the outfall structure is above the water level in Islais Creek Channel, we recommend the outfall structure be excavated with temporary slopes. Where sloping is not feasible, shoring consisting of sheet piles may be used to laterally restrain the sides of the excavation for the outfall structure. Installation of the sheet piles may first require the removal of rip-rap, if present.

Sheet piles should be designed to resist active earth pressures using an equivalent fluid weight of 40 pcf. Where the excavation extends below the design groundwater table (Elevation -5 feet), the sheet piles should be designed to resist active earth plus hydrostatic pressures using an equivalent fluid weight of 85 pcf.

For lateral resistance below the bottom of the excavation, we recommend passive pressure be evaluated using an equivalent fluid weight of 240 pcf and 140 pcf above and below the groundwater, respectively. The passive pressure values include a factor of safety of about 1.5.

The shoring system should be designed by a licensed structural engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring designer should evaluate the required embedment depth of the sheet piles. Furthermore, the designer should determine the type and size of shoring members required to resist the lateral earth and hydrostatic pressures presented in this section. Control of ground movement will depend as much on the timeliness of installation of lateral restraint on the design. We should review shoring plans and calculations to check that they conform to our recommendations.

11.8 Floor Slabs

If water vapor moving through the slab (i.e. new restroom facility) is considered detrimental, we recommend installing a capillary moisture break and a water vapor retarder beneath the floor. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 7.

TABLE 7

Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90-100
3/4 inch	30-100
1/2 inch	5–25
3/8 inch	0-6
Sand	
No. 4	100
No. 200	0-5

The sand overlying the membrane should be dry at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the restroom expansion slab should have a low water/cement (w/c) ratio – less than 0.5. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured.

We recommend the specifications for slabs-on-grade floors require the moisture emission tests be performed in accordance with the requirements of ASTM F 1869 on the slab prior to the installation of flooring, if any. No flooring should be installed until safe moisture emission levels are recorded for the type of flooring to be used.

11.9 Engineered Alternative Covers

As presented in Section 10.3, the results of the HELP-3 analyses indicate the average annual percolation through existing covers that consist of 18 inches of low hydraulic conductivity material (less than $1x10^{-6}$ cm/sec) is less than the average annual percolation through the prescriptive covers; the average annual percolation through the vegetated swale and asphalt concrete pavement is less than the average annual percolation through the prescriptive covers. Therefore, we conclude the 18-inch low hydraulic conductivity soil cover, vegetated swale, and asphalt concrete pavement may be used as engineered alternative covers within the regulated landfill area, provided they are designed following the recommendations presented in this section and are approved by the RWQCB.

11.9.1 Soil Cover

Current project plans indicate additional fill will be placed within the regulated landfill area to reach final grades. The fill may consist of imported soil or existing material that is stockpiled onsite. Where a soil cover is to be used as an engineered alternative cover, we recommend the upper 18 inches of soil consists of sandy or clayey sand with at least 30 percent fines, and no more than 5 percent gravel, has liquid limit less than 40 and a plasticity index of 12 or less, be compacted to at least 90 percent relative compaction, and has a hydraulic conductivity of $1x10^{-6}$ cm/sec or less (when compacted to 90 percent relative compaction). The soil subgrade underlying the 18-inch soil cover should be striped of vegetation and compacted to at least 90 percent relative compaction prior to placing the soil cover.

11.9.2 Vegetated Swale

Current project plans indicate vegetated swale will be constructed within the regulated landfill area along the south side of Amador Street and the east and west sides of Amador Street Extension.

In areas that will be covered with vegetated swale, we recommend the upper six inches of soil subgrade be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction, as described in Section 11.1. The soil subgrade of the vegetated swale should be graded with a minimum one percent slope and be covered with a low hydraulic conductivity geomembrane liner. The geomembrane liner should have maximum hydraulic conductivity of $1x10^{-9}$ cm/sec, maximum transmissivity of 0.3 cm^2/sec, and maximum total defect of 11 holes per acre. Delivery, storage, and placement of geomembrane liner, including overlapping and taping seams, should meet manufacturer requirements.

The geomembrane liner should be covered with at least 12 inches of vegetation soil layer. Vegetation should be established on this soil layer and the surface of the soil layer should be graded to a minimum one percent slope and to direct runoff to the proposed emerging wetlands and new storm water structures. Irrigation on the vegetated swale should not be permitted.

11.9.3 Asphalt Concrete Pavement

The State of California resistance value (R-value) method for flexible pavement design was used to develop recommendations for asphalt concrete pavement sections. However, since locations for asphalt pavement areas were not identified when this report was prepared, and the soil subgrade is variable across the site, we conservatively assumed an R-value of 10 for the sandy clay encountered during our exploration.

We evaluated pavement sections for traffic index (TI) of 3 through 9. Table 8 presents our pavement section recommendations for asphalt concrete pavement.

TABLE 8

Recommended Asphalt Pavement Sections (inches)

	Asphalt Concrete	Class 2 AB
TI = 3	2.5	6.0
TI = 4	2.5	7.0
TI = 5	3.0	9.0
TI = 6	3.5	11.5
TI = 7	4.0	14.5
TI = 8	5.0	16.5
TI = 9	5.5	19.0

When plans for asphalt concrete pavement becomes available, we can obtain soil samples of the subgrade soil to determine the R-value and revise our pavement section recommendations for design traffic index and R-value, as appropriate.

In areas that will be covered with asphalt pavement, we recommend the upper six inches of the subgrade be scarified, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction, as described in Section 11.1. Aggregate base should conform to Section 26-1.02A of the current Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction.

The asphalt pavement within the regulated landfill area should have a minimum slope of one percent and the pavement section should be underlain by a geomembrane liner. The geomembrane liner should have maximum hydraulic conductivity of $1x10^{-9}$ cm/sec, maximum transmissivity of 0.3 cm^2/sec, and maximum total defect of 11 holes per acre. Delivery, storage, and placement of geomembrane liner, including overlapping and taping seams and protection of liner, should meet manufacturer requirements.

11.10 Landfill Gas Control System Recommendation

Landfill gas control at waste disposal sites is subject to Title 27 of the California Code of Regulations. In Article 6 Section 20921 of that regulation, landfill gas controls are required to maintain the following three conditions:

- Concentrations of methane gas shall not exceed 1.25 percent by volume in air within on-site structures.
- The concentration of methane gas migrating from the landfill property shall not exceed 5 percent by volume in air at the facility property boundary.
- Trace gases shall be controlled to prevent adverse acute and chronic exposure to toxic and/or carcinogenic compounds.

Since methane gas was not detected during the SWAT air quality investigation in 1988 and concentrations of detected VOCs were below applicable health risk criteria cited above, landfill gas controls are not necessary as part of the proposed site improvements. Also, on-site structures for human occupancy are not part of the current plans for the proposed site improvements. However, should the re-use plans change to include enclosed structures, a soil gas investigation should be performed at the proposed structure location to confirm the absence of methane and VOCs in soil gas at the location of the proposed structure.

12.0 GEOTECHNICAL SERVICES DURING FINAL DESIGN AND CONSTRUCTION

When final grading and utility layout plans become available, we should review the plans and perform additional analyses, as appropriate, to check settlement along underground utility alignments.

In addition, we should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing services during subgrade preparation, fill placement, utility trench backfill, and shoring installation. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

13.0 LIMITATIONS

We performed this assessment in accordance with our scope of services described in Task 1 of our proposal dated 25 January 2011 and budget amendment dated 23 May 2011. Reasonable effort has been made to check that information obtained from others is factual and reliable, however, we assume no responsibility for the completeness or the accuracy of the information. Hazardous substance or condition may exist at the site that is not identified due to the limited scope of this study. No warranty or guarantee is either expressed or implied with regard to conditions at the site. We assume no responsibility or liability for errors in the information used or statements from sources other than T&R/RYCG. All conclusions and recommendations in this report concerning the subject property are our professional opinion; this report should not be considered a legal interpretation of existing environmental regulations. Opinions presented herein apply to site conditions existing at the time of our assessment, and may not apply if the site conditions change or if conditions exist that we are not aware of and has not had the opportunity to evaluate.

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FIGURES

Base map: The Thomas Guide
San Francisco County
2002

0 2000 Feet

Approximate scale

N

PIER 94 BACKLANDS IMPROVEMENTS
San Francisco, California

T&R / RYCG
A Joint Venture

SITE LOCATION MAP

Date 07/14/11	Project No. 730509401	Figure 1

Reference: Base map from a drawing and electronic file provided by the Port of San Francisco, delivered 04/08/11 and Google Earth Pro, 2011.

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EXPLANATION

— — Project limit

▨ RWQCB regulated landfill area

0 300 Feet

Approximate scale

PIER 94 BACKLANDS IMPROVEMENTS

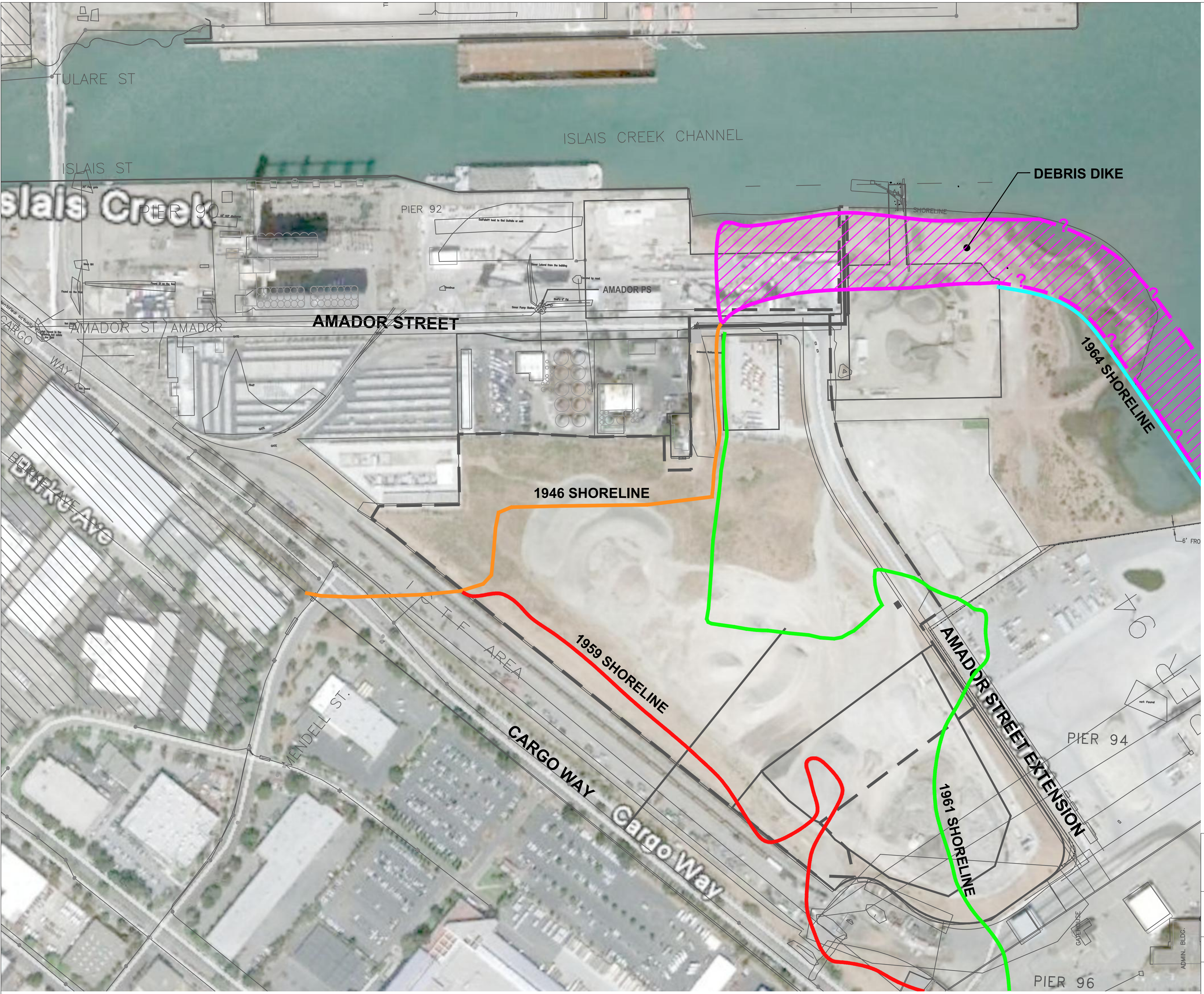
San Francisco, California

SITE PLAN

Date 09/28/11	Project No. 730509401	Figure 2

T&R / RYCG

A Joint Venture



EXPLANATION

- — — Project limit
- 🟧 1946 shoreline
- 🟥 1959 shoreline
- 🟩 1961 shoreline
- 🟦 1964 shoreline
- ▨ Debris Dike

0 300 Feet
Approximate scale

Reference: Base map from a drawing and electronic file provided by the Port of San Francisco, delivered 04/08/11 and Google Earth Pro, 2011.

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PIER 94 BACKLANDS IMPROVEMENTS
San Francisco, California

HISTORIC SITE PLAN

Date 10/11/11	Project No. 730509401	Figure 3

T&R / RYCG
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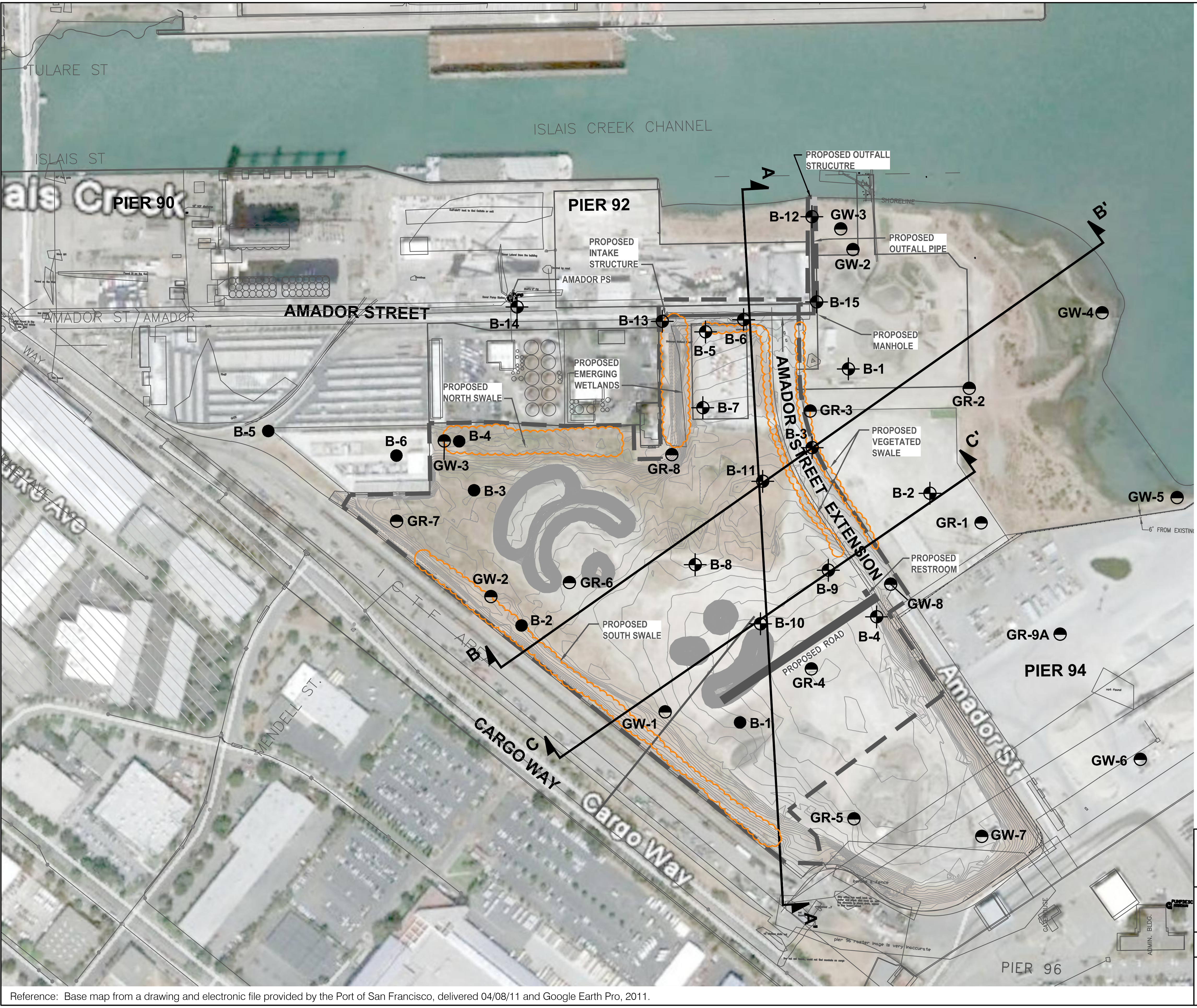

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EXPLANATION

B-1 ⊕	Approximate location of boring by T&R/RYCG, May 2011
B-1 ●	Approximate location of boring by Bechtel, 1994
GR-1 or GW-1 ◓	Approximate location of boring or groundwater monitoring well by Geo/Resources Consultants, Inc., January 1989
A A'	Idealized subsurface profile
— —	Project limit

0 300 Feet

Approximate scale

PIER 94 BACKLANDS IMPROVEMENTS

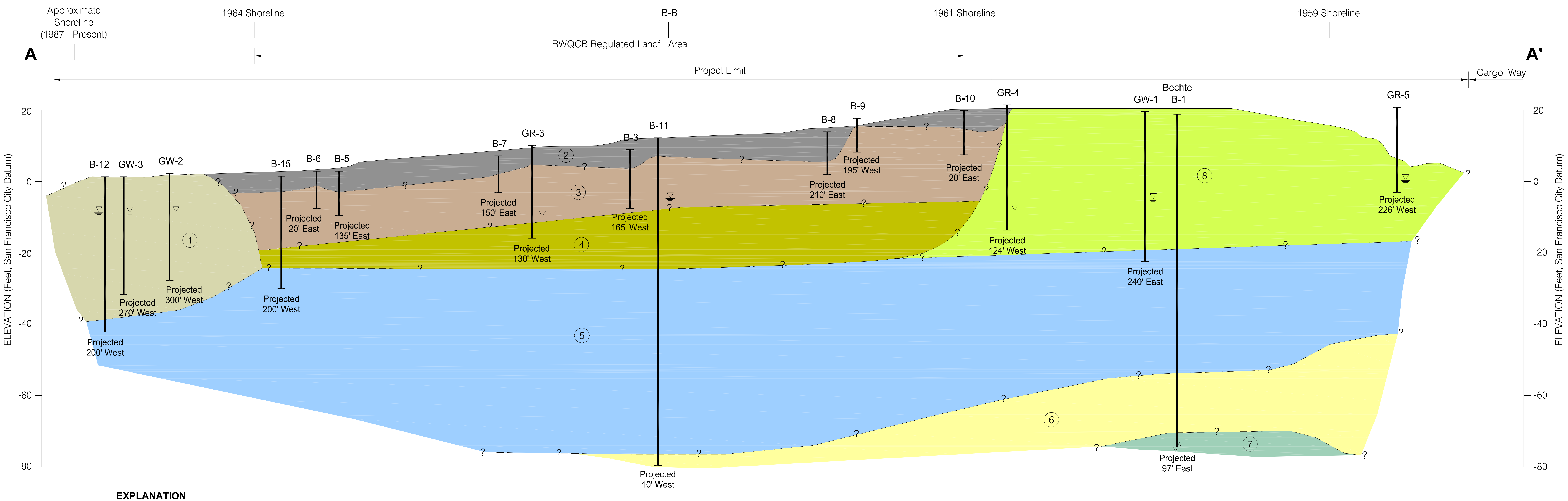
San Francisco, California

BORING LOCATIONS

Date 07/01/12 | Project No. 730509401 | Figure 4

T&R / RYCG

A Joint Venture



EXPLANATION

①	Debris Dike - The debris dike is constructed with fill consisting of construction wastes, including wood, paper, brick, plaster, concrete, and other inert materials, mixed with 30 to 40 percent soil.
②	Soil Cap - The soil cap consists of heterogeneous mixtures of sand and gravel with variable amounts of clay and silt.
③	Construction Debris - The construction debris consists of construction and municipal waste mixed with 30 to 40 percent soil. Construction waste consists of wood, paper, brick, plaster, concrete, and other inert materials. Municipal waste consists of furniture, steel containers, discarded appliances, etc.
④	Dredge Spoils - The dredge spoils consists of Bay Mud dredged from the Army Street Terminal (Pier 80) construction site. The dredge spoils was hydraulically placed inboard of the debris dike.
⑤	Bay Mud - The Bay Mud consists of gray, soft to stiff clay.
⑥	Dense to very dense sand.
⑦	Old Bay Clay - Stiff to hard clay with layers of silty sand.
⑧	Historic Fill - Heterogeneous fill containing clayey soil and rock fragments placed prior to 1961.
⊻	Measured groundwater level

Notes:

1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

2. Elevation base on site topographic survey provided by Port of San Francisco, delivered on 04/08/11.

3. Shoreline limits based on Geo/Resource Consultants, Inc. report titled "Data Compilation Report & Appendices A through D Data Logs, Vol. 1 of 3," dated 01/02/90.

20
0
0 100
SCALE IN FEET

PIER 94 BACKLANDS IMPROVEMENTS
San Francisco, California

IDEALIZED SUBSURFACE PROFILE
A-A'

Date 08/31/11	Project No. 730509401	Figure 5

T&R / RYCG
A Joint Venture

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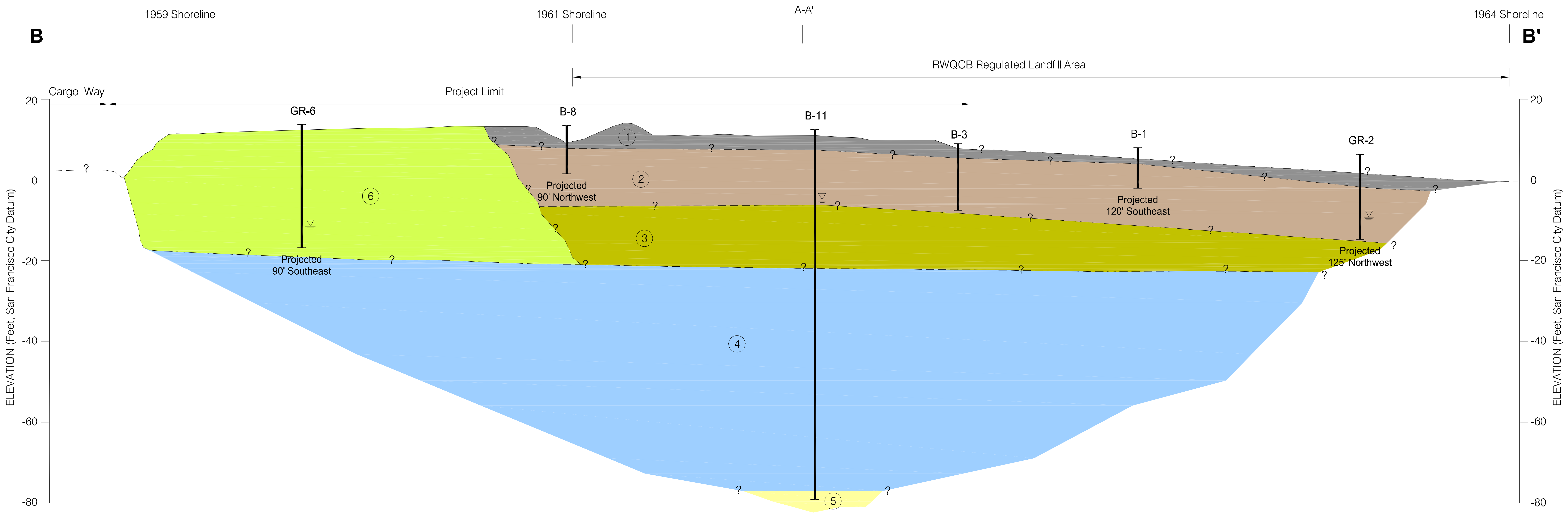


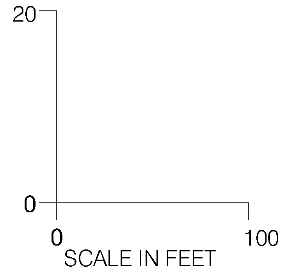
EXPLANATION

①		Soil Cap - The soil cap consists of heterogeneous mixtures of sand and gravel with variable amounts of clay and silt.
②		Construction Debris - The construction debris consists of construction and municipal waste mixed with 30 to 40 percent soil. Construction waste consists of wood, paper, brick, plaster, concrete, and other inert materials. Municipal waste consists of furniture, steel containers, discarded appliances, etc.
③		Dredge Spoils - The dredge spoils consists of Bay Mud dredged from the Army Street Terminal (Pier 80) construction site. The dredge spoils was hydraulically placed inboard of the debris dike.
④		Bay Mud - The Bay Mud consists of gray, soft to stiff clay.
⑤		Dense to very dense sand.
⑥		Historic Fill - Heterogeneous fill containing clayey soil and rock fragments placed prior to 1961.
▽		Measured groundwater level

Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

2. Elevation base on site topographic survey provided by Port of San Francisco, delivered on 04/08/11.

3. Shoreline limits based on Geo/Resource Consultants, Inc. report titled "Data Compilation Report & Appendices A through D Data Logs, Vol. 1 of 3," dated 01/02/90.



PIER 94 BACKLANDS IMPROVEMENTS
San Francisco, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 08/31/11 | Project No. 730509401 | Figure 6

T&R / RYCG
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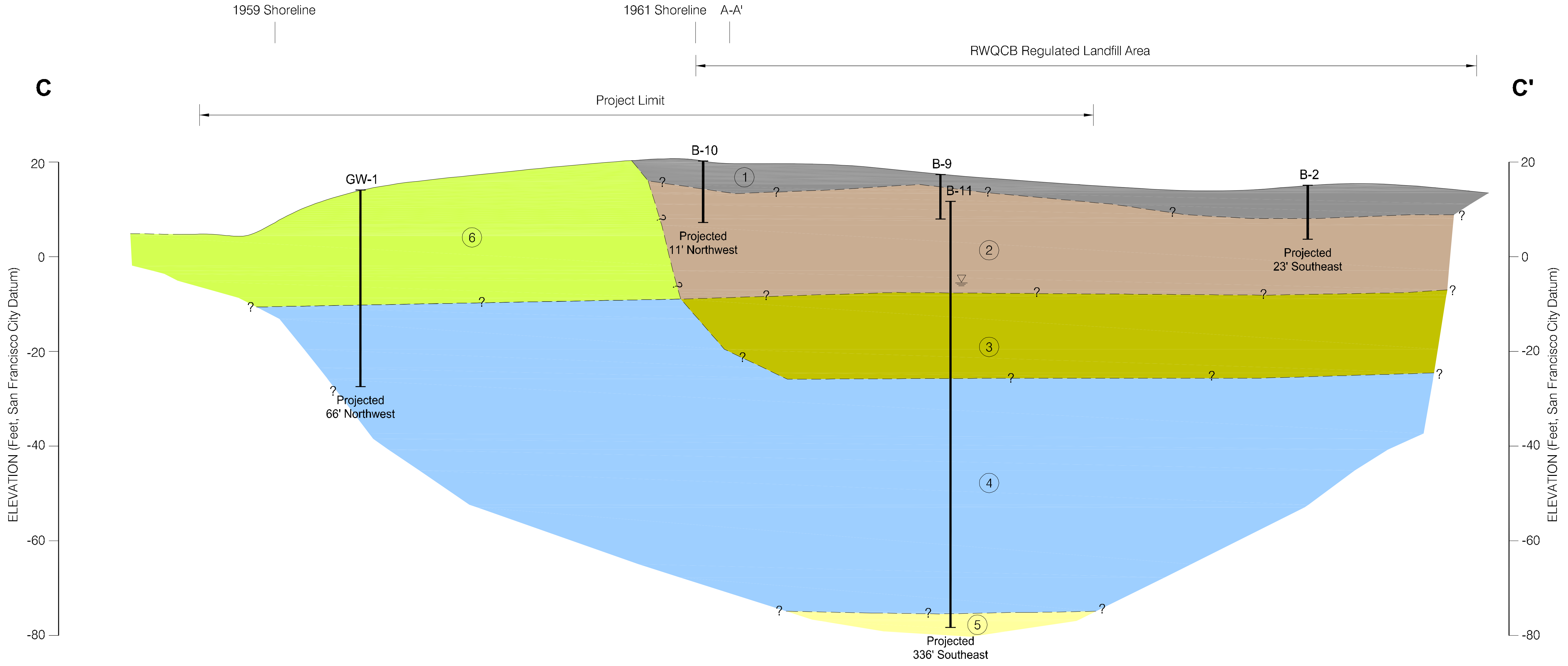


EXPLANATION

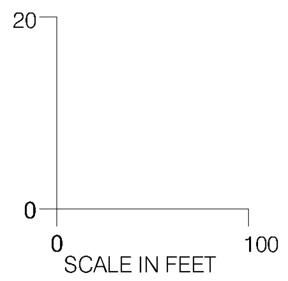
- ① Soil Cap - The soil cap consists of heterogeneous mixtures of sand and gravel with variable amounts of clay and silt.
- ② Construction Debris - The construction debris consists of construction and municipal waste mixed with 30 to 40 percent soil. Construction waste consists of wood, paper, brick, plaster, concrete, and other inert materials. Municipal waste consists of furniture, steel containers, discarded appliances, etc.
- ③ Dredge Spoils - The dredge spoils consists of Bay Mud dredged from the Army Street Terminal (Pier 80) construction site. The dredge spoils was hydraulically placed inboard of the debris dike.
- ④ Bay Mud - The Bay Mud consists of gray, soft to stiff clay.
- ⑤ Dense to very dense sand.
- ⑥ Historic Fill - Heterogeneous fill containing clayey soil and rock fragments placed prior to 1961.
- ▽ Measured groundwater level

Notes:

1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

2. Elevation base on site topographic survey provided by Port of San Francisco, delivered on 04/08/11.

3. Shoreline limits based on Geo/Resource Consultants, Inc. report titled "Data Compilation Report & Appendices A through D Data Logs, Vol. 1 of 3," dated 01/02/90.



PIER 94 BACKLANDS IMPROVEMENTS
San Francisco, California

IDEALIZED SUBSURFACE PROFILE
C-C'

Date 07/01/12 | Project No. 730509401 | Figure 7

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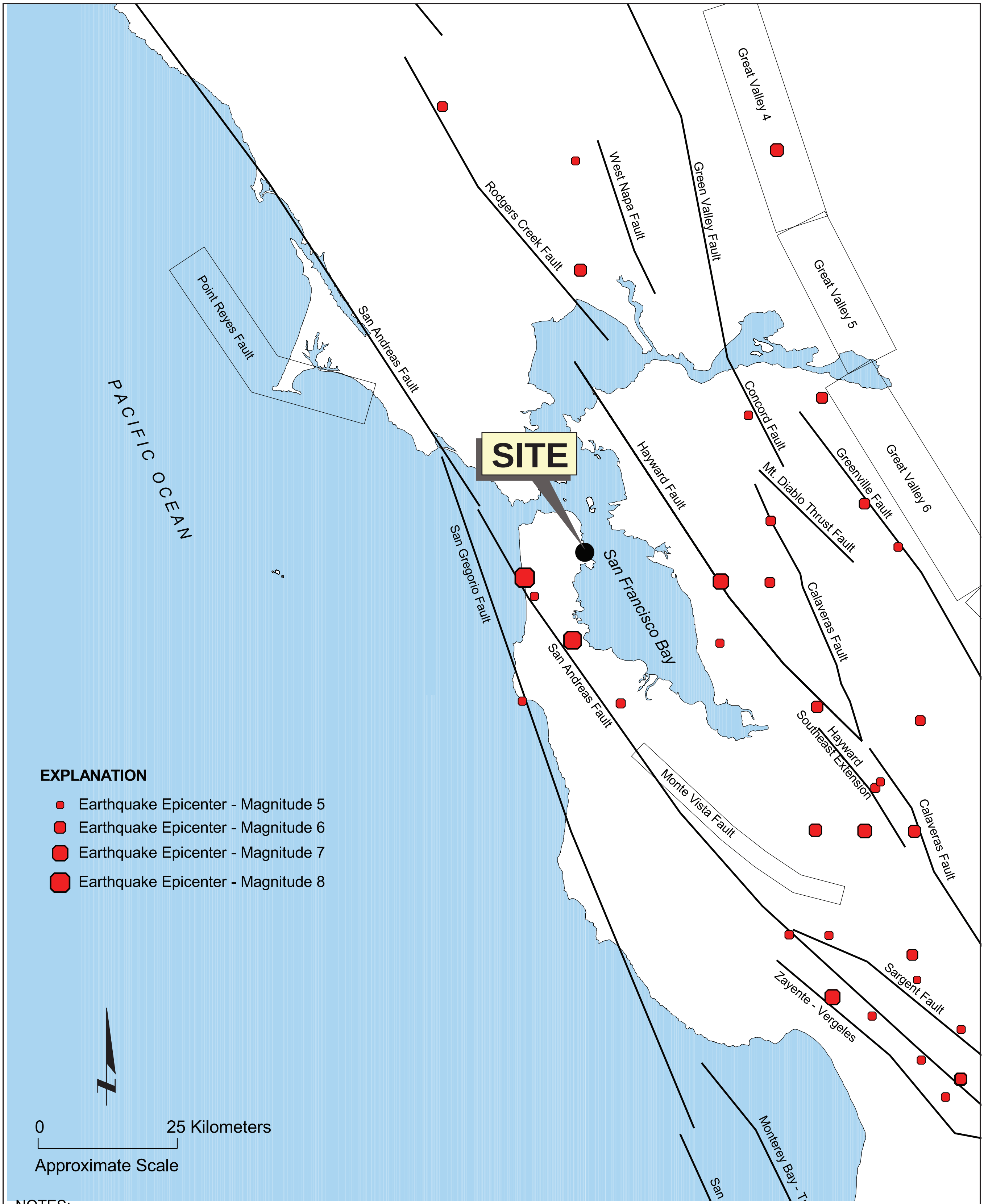

NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

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MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 07/14/11 | Project No. 730509401 | Figure 8

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**

Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**

As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**

Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**

Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**

Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**

Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**

People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**

Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX **Panic is general.**

Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**

Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**

Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**

Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

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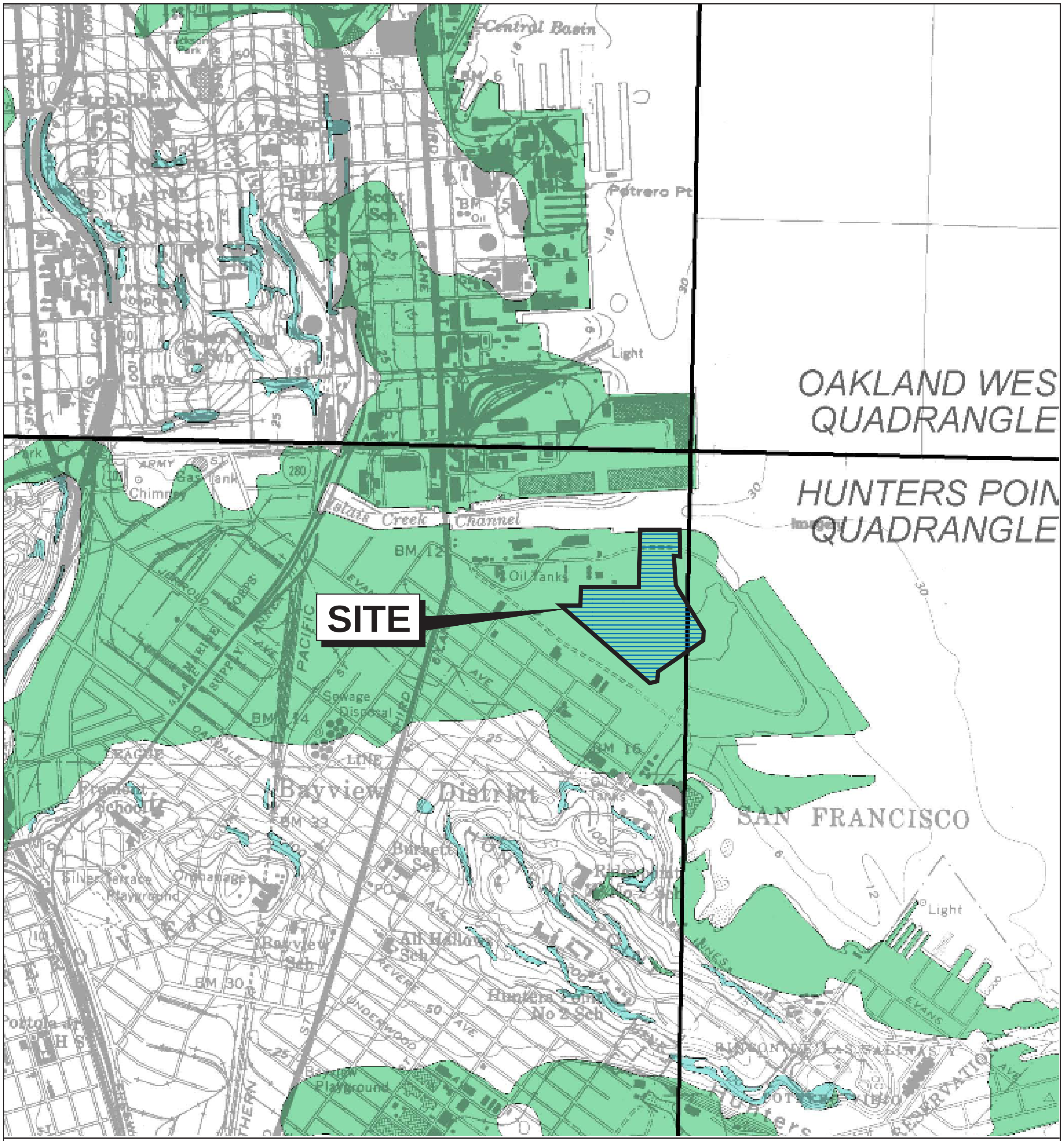
MODIFIED MERCALLI INTENSITY SCALE

Date 07/14/11 | Project No. 730509401 | Figure 9



EXPLANATION

🟩 Zone of Liquefaction

🟦 Earthquake-induced Landslides

0 2000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones" City and County of San Francisco Released on November 17, 2001

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REGIONAL SEISMIC HAZARD ZONES MAP

Date 07/20/11 | Project No. 730509401 | Figure 10

Reference: Base map from a drawing and electronic file provided by the Port of San Francisco, delivered 04/08/11 and Google Earth Pro, 2011.

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EXPLANATION

1 ◉	Approximate location of settlement point

0 300 Feet

Approximate scale

PIER 94 BACKLANDS IMPROVEMENTS

San Francisco, California

SETTLEMENT POINTS

Date 03/27/12	Project No. 730509401	Figure 11

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APPENDIX A
Logs of Borings

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-1

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 5/25/11
Date finished: 5/25/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: M. McKee

DEPTH (feet)	SAMPLES Sampler Type	 Sample	 Blows/6"	 SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 8 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
1						SAND (SP) olive-brown, moist, trace silt and wood	SOIL CAP						
2	GRAB	☒			SP								
3													
4	S&H	■	29 50/5"	30/5"		trace concrete fragments							
5	SPT	◪	21 38 27	65	SP-SM	SAND with SILT (SP-SM) olive-gray, very dense, moist, fine-grained, cemented, trace fine-grained gravel Hydraulic Conductivity Test, see Figure B-6						8.9	110
6							CONSTRUCTION DEBRIS						
7					SM	SILTY SAND (SM) olive, very dense, moist, trace brick, wood, and asphalt							
8													
9	S&H	■	6 8 10	11	SC	CLAYEY SAND with GRAVEL (SC) dark bluish-gray, medium dense, moist, serpentinite and chert fragments, brick, wood							
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
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21													
22													
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24													
25													
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27													
28													
29													
30													

Boring terminated at a depth of 10 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401
Figure: A-1

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-2

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 5/26/11
Date finished: 5/26/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: T. Shu

LABORATORY TEST DATA

Ground Surface Elevation: 15 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[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
	GRAB				SM	SILTY SAND with GRAVEL (SM) light gray, very dense, dry, trace asphalt	SOIL CAP						
1	S&H		50/ 3.5"	30/ 3.5"									
2	SPT		16 8 6	14	GP	GRAVEL with SAND (GP) brownish-gray, medium dense, moist, trace brick							
3													
4													
5													
6	S&H		11 14 19	20	SC	CLAYEY SAND with GRAVEL (SC) brown, olive, and gray, medium dense to very dense, moist, gravel size serpentinite fragments Hydraulic Conductivity Test, see Figure B-7					17.1	17.3	111
7	SPT		17 26 50/ 0.5	75/ 6.5"		with brick at 6.5 feet	CONSTRUCTION DEBRIS						
8													
9						plastic, string, and sheet metal at 8 feet, chain-link fencing pieces at 9.5 to 10 feet							
10	S&H		50/ 2"	30/ 2"									
11	SPT		50/ 3"	50/ 3"		dark gray-brown sandy clay, wet, with wood, and brick							
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 10.8 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401
Figure: A-2

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-3

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 5/26/11

Date finished: 5/26/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: T. Shu

Ground Surface Elevation: 9 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/6"	SAMPLES: SPT N-Value[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
	GRAB					CLAYEY SAND with GRAVEL (SC) dark brown, medium dense, moist, some roots, trace gravel						
1												
2	S&H		12 18 23	25		Hydraulic Conductivity Test, see Figure B-8 concrete fragments at 2.5 feet					10.9	119
3	SPT		6 8 11	19	SC	**SOIL CAP**				22.4		
4						grades brown with bluish-gray and yellow specks, some serpentinite fragments						
5						dense						
6	S&H		13 34 36	42		SANDY CLAY (CL) dark brown, gray, and black, stiff, moist, trace gravel						
7	SPT		7 6 4	10								
8						wood, metal, plastic, brick, and foam at 8 feet						
9	SPT		9 11 4	15	CL							
10	S&H		16 20 28	29		**CONSTRUCTION DEBRIS**						
11						grades brown, trace brick fragments						
12	SPT		7 13 29	42		CLAYEY SAND with GRAVEL (SC) bluish-gray, dense, moist, with serpentinite gravel fragments and brick						
13												
14					SC							
15												
16	S&H		6 7 10	10	SP-SM	SAND with SILT (SP-SM) gray, loose to medium dense, moist, with wood						
17												
18												
19												
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22												
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26												
27												
28												
29												
30												

Boring terminated at a depth of 16.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-3

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-4

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 5/26/11 | Date finished: 5/26/11

Drilling method: Hollow Stem Auger

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

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: T. Shu

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 19.5 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
	GRAB					SILTY SAND with GRAVEL (SM) brown, dense, dry	SOIL CAP						
1			31		SM								
2	S&H		38 40	47		grades bluish-gray and green, dense, moist Hydraulic Conductivity Test, see Figure B-9					22.3	20.3	90
	SPT		50/ 6"	50/ 6"									
3						SILTY SAND with GRAVEL (SM) dark grayish-brown, very dense, moist, wood, trace concrete and metal	CONSTRUCTION DEBRIS						
4													
5	S&H		50/ 5"	30/ 5"		increasing wood							
6													
7					SM								
8													
9						metal fencing pieces							
10	S&H		50/ 3"	30/ 3"		wood, plastic, and fibrous debris at 10 feet							
	SPT		12 50/ 4"	50/ 4"									
11													
12													
13													
14													
15													
16													
17													
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28													
29													
30													

Boring terminated at a depth of 11 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401 | Figure: A-4

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-5

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 5/25/11

Date finished: 5/25/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H)

Logged by: T. Shu

LABORATORY TEST DATA

Ground Surface Elevation: 3 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/6"	SAMPLES: SPT N-Value[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
1													
2	S&H	▨	14 6 7	8	SM	SILTY SAND with GRAVEL (SM) gray and brown, loose, moist, trace glass, concrete, and chert fragments Particle Size Analysis, see Figure B-1 Hydraulic Conductivity Test, see Figure B-10	**SOIL CAP**				22.9	13.3	119
3													
4													
5													
6	S&H	▨	2 4 8	7									
7					SM	SILTY SAND with GRAVEL (SM) gray, loose, moist, with wood, concrete, and glass	**CONSTRUCTION DEBRIS**						
8					GP	GRAVEL (GP) bluish-gray, medium dense, moist, serpentinite and brick fragments and wood debris							
9	S&H	▨	18 16 15	19									
10					SM	SILTY SAND with GRAVEL (SM) grayish-brown, medium dense, wet, some brick fragments							
11					SC	CLAYEY SAND (SC) dark gray, loose, wet, with wood debris							
12	S&H	▨	8 8 5	8	CH	CLAY (CH) gray, medium stiff to stiff, wet, trace wood fragments, high plasticity							
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 12.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-5

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-6

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: T. Shu

Date started: 5/25/11

Date finished: 5/25/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Ground Surface Elevation: 3 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	
0–2	S&H	31 50/ 5"	30/ 5"	SM	SILTY SAND with GRAVEL (SM) brown, dry black, very dense, moist, with gravel and asphalt fragments	SOIL CAP
2.5–4	S&H	12 17 11	17	SP	SAND (SP) olive, medium dense, moist	
4–8.5	S&H	21 26 17	26	SC	CLAYEY SAND (SC) bluish-gray and brown, medium dense, moist, trace gravel (siltstone fragments) and wood grades with wood, brick, and pieces of sheet metal at 4.5 feet	CONSTRUCTION DEBRIS
8.5–10.5	SPT	26 36 9	45	SM	SILTY SAND (SM) black, dense, with wood and concrete	

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

Boring terminated at a depth of 10.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-6

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-7

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 5/25/11

Date finished: 5/25/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H)

Logged by: T. Shu

Ground Surface Elevation: 8 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ 6"	SAMPLES SPT N-Value[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
1	S&H	▨	50/ 4"	30/ 4"	SP-SM	SAND with GRAVEL and SILT (SP-SM) olive-brown, dry	**SOIL CAP**						
2						SILTY GRAVEL with SAND (GM) brown and gray, very dense, moist, wood and brick fragments							
3	GRAB	☒			GM	Particle Size Analysis, see Figure B-1					14.8		
4													
5													
6	S&H	▨	18 20 24	26		CLAYEY SAND (SC) grayish-brown, olive, and gray , medium dense, moist, with gravel, brick, wood, and asphalt fragments, and plastic bag	**CONSTRUCTION DEBRIS**						
7													
8					SC								
9	S&H	▨	8 8 8	10		grades loose to medium dense, with pockets of olive-yellow clayey sand							
10													
11													
12													
13													
14													
15													
16													
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27													
28													
29													
30													

Boring terminated at a depth of 10 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-7

TEST GEOTECH LOG 730509401.GPJ TR.GDT 3/28/12

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-8

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 5/25/11
Date finished: 5/25/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: T. Shu

Ground Surface Elevation: 13.5 feet^2

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/6"	SAMPLES SPT N-Value[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
	GRAB	☒			SM	SILTY SAND (SM) brown, dry, trace brick, concrete, and gravel	SOIL CAP						
1													
2	S&H	■	19 15 14	17	SC	CLAYEY SAND with GRAVEL (SC) gray and olive-brown, medium dense, moist, serpentinite gravel fragments							
3	SPT	◩	5 5 6	11									
4													
5						CLAYEY SAND with GRAVEL (SC) gray, medium dense, moist, trace brick fragments Hydraulic Conductivity Test, see Figure B-11							
6	S&H	■	30 17 20	22	SC						20.5	13.1	120
7													
8	S&H	■	9 12 14	16		grades brownish-gray to olive-gray							
9	SPT	◩	6 6 7	13		CLAYEY SAND with GRAVEL (SC) brown and gray, medium dense, moist, with wood fragments	CONSTRUCTION DEBRIS						
10					SC								
11	S&H	■	24 50/ 6"	30/ 6"		grades with wood debris, trace gray clayey sand							
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 12 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401
Figure: A-8

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-9

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 5/25/11

Date finished: 5/25/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: T. Shu

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/6"	SPT N-Value[1]		Ground Surface Elevation: 17.5 feet[2]		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					ML	SANDY SILT (ML) brown, dry, some gravel, trace wood debris	SOIL CAP						
1													
2	S&H	▨	24 27 32	35	SC	CLAYEY SAND with GRAVEL (SC) brown and gray mottled, dense, moist, brick							
3						CLAYEY SAND with GRAVEL (SC) dark grayish-brown to black, medium dense, moist, wood debris concrete fragments at 4.5 to 5 feet	CONSTRUCTION DEBRIS						
4	SPT	○	7 8 12	20									
5													
6	S&H	▨	7 11 20	12	SC								
7	SPT	○	20 50/ 1"	50/ 1"		fibrous mesh debris at 6.5 feet							
8													
9	S&H	▨	32 50/ 5"	30/ 5"									
10						wood debris at 9.5 feet							
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 9.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Figure: A-9

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-10

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 5/25/11

Date finished: 5/25/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: T. Shu

Ground Surface Elevation: 19.5 feet[2]

SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	
GRAB	☒			GP-GM	GRAVEL with SILT and SAND (GP-GM) brown, dense, dry, some brick and concrete fragments	SOIL CAP
S&H	▨	18 32 41	44			
SPT	◩	8 17 19	36	SM	SILTY SAND with GRAVEL (SM) gray and olive-brown, mottled, dense, moist, trace concrete and brick	
S&H	▨	6 7 10	10	CL	SANDY CLAY (CL) brown, stiff, very moist, trace wood and shells	
SPT	◩	16 21 16	37	SC	CLAYEY SAND (SC) gray, dense, moist, trace wood fragments	CONSTRUCTION DEBRIS
					plastic, wood, metal at 10.5 to 11 feet	
SPT	◩	31 37 33	70		wood in sampler	

DEPTH (feet): 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

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

Boring terminated at a depth of 12.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Figure: A-10

TEST GEOTECH LOG 730509401.GPJ TR.GDT 3/28/12

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-11

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2
Logged by: T. Shu
Date started: 5/23/11
Date finished: 5/24/11
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)

LABORATORY TEST DATA

Ground Surface Elevation: 12.5 feet^2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[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
1					SM	SILTY SAND with GRAVEL (SM) brown, moist, with asphalt	SOIL CAP						
2	S&H	▨	6 9 20	20	GP-GM	GRAVEL with SILT and SAND (GP-GM) reddish-brown, medium dense, slightly moist, trace clay and asphalt							
3					SM	SILTY SAND (SM) dark brown, medium dense, dry, trace gravel							
4	S&H	▨	7 11 11	15	SC	CLAYEY SAND with GRAVEL (SC) bluish-gray and olive, medium dense, slightly moist, trace wood Hydraulic Conductivity Test, see Figure B-12						13.5	105
6	S&H	▨	5 7 9	11	GC	CLAYEY GRAVEL (GC) dark bluish-gray, medium dense, moist, trace wood							
9	S&H	▨	5 10 14	17	SC	CLAYEY SAND (SC) dark grayish-brown, medium dense, moist, with gravel, trace wood debris	CONSTRUCTION DEBRIS						
11					SC	CLAYEY SAND with GRAVEL (SC) black, medium dense, moist, wood and brick glass, concrete, plastic, brick, and wood debris at 12 to 12.5 feet							
16	S&H	●	22 18 17	25									
17	SPT	◩	5 11 9	24	▽	wood debris, wet (05/24/11, 7:15 a.m.)							
20	S&H	▨	4 6 7	9	CH	SANDY CLAY (CH) gray, stiff, wet, with shells, trace wood, highly plastic	DREDGE SPOILS	PP		1,500- 2,200		21.1	105
25	S&H	▨	17 50/ 2"	35/ 2"		Concrete and wood debris with black clayey silt wood, brick, shells							
30	S&H	●		8									

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Project No.: 730509401
Figure: A-11a

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-11

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value¹	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		16 6 6	8		concrete and wood debris with black clayey silt (continued)	DREDGE SPOILS						
32													
33													
34						CLAY (CH) gray, very soft, wet, highly plastic							
35													
36	S&H		0 0 1	1	CH	trace wood fragments at 36.5 feet							
37													
38						CLAY (CH) gray, stiff, wet, highly plastic	BAY MUD						
39													
40													
41	D&M			300 psi		Triaxial Test, see Figure B-2 Consolidation Test, see Figure B-3		TxUU TV	4,000	1,070 1,160		47.4 50.6	74 72
42													
43													
44													
45													
46													
47													
48													
49													
50					CH	grades silty, soft to medium stiff							
51	D&M			200 psi				TV		800			
52													
53													
54													
55													
56													
57													
58													
59													
60						grades medium stiff to stiff							
61	D&M			400 psi				TV		1,200			
62													

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Figure: A-11b

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-11

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DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[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
						CLAY (CH) (continued)						
63												
64												
65												
66						shell bed from 66 to 68 feet						
67												
68												
69												
70												
71	D&M			500 psi			TV		1,130			
72												
73												
74												
75					CH	BAY MUD						
76												
77												
78												
79												
80												
81	D&M			375 psi		trace shell fragments Consolidation Test, see Figure B-4	TV		1,170		56.4	67
82												
83												
84												
85												
86												
87												
88												
89			6			SAND (SP)						
90	S&H		13 24	26		olive-gray, medium dense to very dense, wet, fine-grained						
91	SPT		9 22 30	62	SP							
92												
93												

Boring terminated at a depth of 92 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 17.6 feet below ground surface on 05/24/11.
PP = pocket penetrometer, TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Figure: A-11c

TEST GEOTECH LOG 730509401.GPJ TR.GDT 3/28/12

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-12

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 5/25/11 | Date finished: 5/25/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)

Logged by: M. McKee

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 1.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1				SM	SILTY SAND with GRAVEL (SM) brown, loose, moist, with fine-grained gravel, trace wood, brick, and concrete fragments						
2	GRAB										
3	S&H	9 7 7	10	GP-GC	GRAVEL with SAND and CLAY (GP-GC) bluish-gray and green, loose to medium dense, moist, serpentinite fragments						
4					CLAYEY SAND (SC) greenish-gray, loose to medium dense, moist, trace angular fine-grained gravel trace asphalt						
5											
6	S&H	4 5 9	10								
7	SPT	2 4 4	10		grades olive, trace concrete at 6.5 feet grades bluish-gray at 7 feet						
8											
9											
10					▽ (05/25/11) grades dark gray to black, loose to medium dense, with clasts of dark gray clay, brick and wood, trace glass, wet						
11	S&H	3 5 6	8								
12	SPT	2 6 9	18	SC							
13											
14											
15											
16	S&H	24 16 20	25								
17	SPT	10 21 36	68		wood (65%)						
18											
19											
20				SP	SAND (SP) gray, wet, fine-grained						
21	S&H	12 11 12	16		SAND with CLAY (SP-SC) black and olive, medium dense, wet						
22											
23											
24											
25				SP-SC	wood						
26	S&H	18 7 9	11								
27					grades to black clayey sand and wood at 27 feet						
28											
29											
30											

DEBRIS DIKE

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Figure: A-12a

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
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Log of Boring B-12

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DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[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
31	S&H	▨	12 7 9	11	SP-SC	SAND with CLAY (SP-SC) (continued)	DEBRIS DIKE						
32	SPT	◪	1 2 1	4		very loose to loose, trace glass fragments, less clayey							
36	S&H	▨	20 14 12	18		medium dense, black sand and wood							
41	S&H	▨	7 4 3	5		loose							
					CH	CLAY (CH) gray, medium stiff, wet, high plasticity, some shells	BAY MUD	TV		1,000			
43	D&M	▨	14	200 psi				TV		1,000			

Boring terminated at a depth of 43.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater measured at 10 feet below ground surface during drilling.
TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Figure: A-12b

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-13

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/25/11
Date finished: 5/25/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: T. Shu

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: -0.5 foot[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
1					SM	SILTY SAND with GRAVEL (SM) brown, medium dense, moist, trace brick and plastic debris, asphalt fragments, and clay FILL						
2	S&H		18 22 24	28							7.6	103
3					GC	CLAYEY GRAVEL (GC) gray, olive, and black, medium dense, wet, trace asphalt						
4	S&H		8 20 9	17			PP		1,000	28.1	23.6	93
5												
6												
7					CH	CLAY (CH) gray, very soft to soft, moist, with shell fragments						
8	S&H		1 1 2	2							46.0	73
9												
10						⊻ (05/25/11) wet						
11	S&H		7 16 24	20			TV		700			
12	SPT		17 23 28	46	GC	CLAYEY GRAVEL with SAND (GC) bluish-gray, dense, wet, fine- to coarse-grained gravel				16.0		
13												
14												
15	SPT		23 50/ 6"	45/ 6"								
16												
17												
18												
19												
20												
21	S&H		20 25 21	23		medium dense, trace clay, wet						
22												
23												
24												
25					CH	CLAY with SILT (CH) gray, medium stiff to stiff, wet, high plasticity BAY MUD						
26	S&H		5 8 11	10			TV		640		60.2	63
27												
28												
29												
30												

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Figure: A-13a

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-13

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DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[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
						CLAY with SILT (CH) (continued)						
31					CH	BAY MUD						
32	S&H	▨	7 7 8	8		grades with shell fragments at 32 feet	TV		520			
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 33 feet below ground surface.
Boring backfilled with cement grout.
Groundwater measured at a depth of 10 below ground surface during drilling.
PP = pocket penetrometer, TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy above groundwater, and 0.5 and 0.9, respectively, below groundwater.

[2] Elevations based on San Francisco City Datum.

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Figure: A-13b

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-14

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2
Date started: 5/24/11 Date finished: 5/24/11
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by: T. Shu

LABORATORY TEST DATA

Ground Surface Elevation: 1.5 feet^2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^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
1						10 inches concrete over 14 inches cement-treated aggregate base							
2	GRAB	⊠				SILTY SAND with GRAVEL (SM) reddish-brown, moist, fine- to coarse-grained gravel							
3					SM								
4													
5					GP	GRAVEL with SAND (GP) black and olive-green, medium dense, angular serpentinite fragments							
6	S&H	■	10 7 11	12	CL	SANDY CLAY (CL) gray and olive-brown, stiff, moist, trace gravel (05/24/11, 1:05 p.m.)							
7	SPT	◩	2 3 8	13	SC CL ▽		FILL				42.9	15.4	117
8						CLAYEY SAND (SC) gray and olive-brown, medium dense, moist, trace gravel							
9					CL- CH								
10						SANDY CLAY (CL) dark gray and brown, stiff, wet, trace gravel							
11	S&H	■	2 5 13	13		CLAY with SAND (CL-CH) bluish-gray, stiff, wet, trace gravel							
12	SPT	◩	3 2 1	4	SC	CLAYEY SAND with GRAVEL (SC) gray, very loose to medium dense, wet					20.1		
13	D&M	●		400 psi									
14						CLAY (CH) gray, soft, wet, high plasticity, trace shell fragments							
15													
16													
17													
18						soft to medium stiff							
19	D&M	◨		50 psi				TV		800			
20													
21													
22					CH		BAY MUD						
23													
24													
25													
26	D&M	◨		350 psi				TV		800			
27													
28													
29													
30													

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Project No.: 730509401
Figure: A-14a

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-14

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[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
						CLAY (CH) (continued)							
31													
32													
33													
34													
35		▨											
36	D&M	▨		350 psi		medium stiff, trace shell fragments Triaxial Test, see Figure B-2 Consolidation Test, see Figure B-5		TxUU TV	3,500	600 800		56.5 63.2	67 62
37													
38													
39													
40													
41													
42													
43													
44													
45		▨			CH	grades with shell fragments	BAY MUD						
46	D&M	▨		300 psi									
47													
48													
49													
50													
51													
52													
53													
54													
55													
56													
57													
58													
59													
60													

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San Francisco, California

Log of Boring B-14

PAGE 3 OF 3

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[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	D&M	▨		400 psi		CLAY (CH) (continued) grades stiff, no shell fragments	TV		1,000			
62												
63												
64												
65												
66												
67						BAY MUD						
68					CH							
69												
70												
71												
72												
73						grades sandy at 73 feet						
74												
75			10			SAND (SP)						
76	S&H	▨	26 50/ 5"	53/ 11"	SP	gray, very dense, wet, fine-grained						
77	SPT	▨	18 50/ 5"	60/ 5"								
78												
79												
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 77.4 feet below ground surface.
Boring backfilled with cement grout.
Groundwater measured at 7.4 feet during drilling.
TV = torvane

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-14c

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-15

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/25/11
Date finished: 5/25/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)
Logged by: M. McKee

Ground Surface Elevation: 1.5 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/6"	SAMPLES SPT N-Value[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
0–5						SAND with SILT and GRAVEL (SP-SM) olive, loose, slightly moist, trace concrete fragments	SOIL CAP						
2–3	GRAB	☒			SP-SM						11.7		
5–10	S&H	■	6 6 6	8	SM	SILTY SAND with GRAVEL (SM) brown and olive-gray, loose, moist, with serpentinite fragments, concrete, brick, and wood debris	CONSTRUCTION DEBRIS					12.7	118
10–11	S&H	■	3 2 2	3		very loose, abundant wood fragments at 10 feet							
11–12.5	SPT	◪	7 17 19	43	SC	CLAYEY SAND (SC) black, dense, wet, with asphalt concrete, brick, and wood							
15–16.5	S&H	■	8 10 7	12		predominantly wood at 15 feet							
16.5–18	SPT	◪	3 2 4	7		wood, clayey sand, trace brick and PVC at 16.5 feet							
20–25	S&H	■	0 1 0	1	CH	CLAY (CH) gray, very soft, wet, highly plastic, with shells	DREDGE SPOILS						
25–30	S&H	■	1 2 4	4	CH	CLAY (CH) gray, soft to medium stiff, wet, highly plastic, shells	BAY MUD					34.4	86

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Project No.: 730509401
Figure: A-15a

PROJECT: **PIER 94 BACKLANDS IMPROVEMENTS**
San Francisco, California

Log of Boring B-15

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[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
31	S&H	▨	2 2 3	4	CH	CLAY (CH) (continued) soft						
32												
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 31.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not measured during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on San Francisco City Datum.

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Project No.: 730509401

Figure: A-15b

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

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◪	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample, hand auger
⊡	Sampling attempted with no recovery
◫	Core sample
▣	Analytical laboratory sample
▥	Sample taken with Direct Push sampler

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

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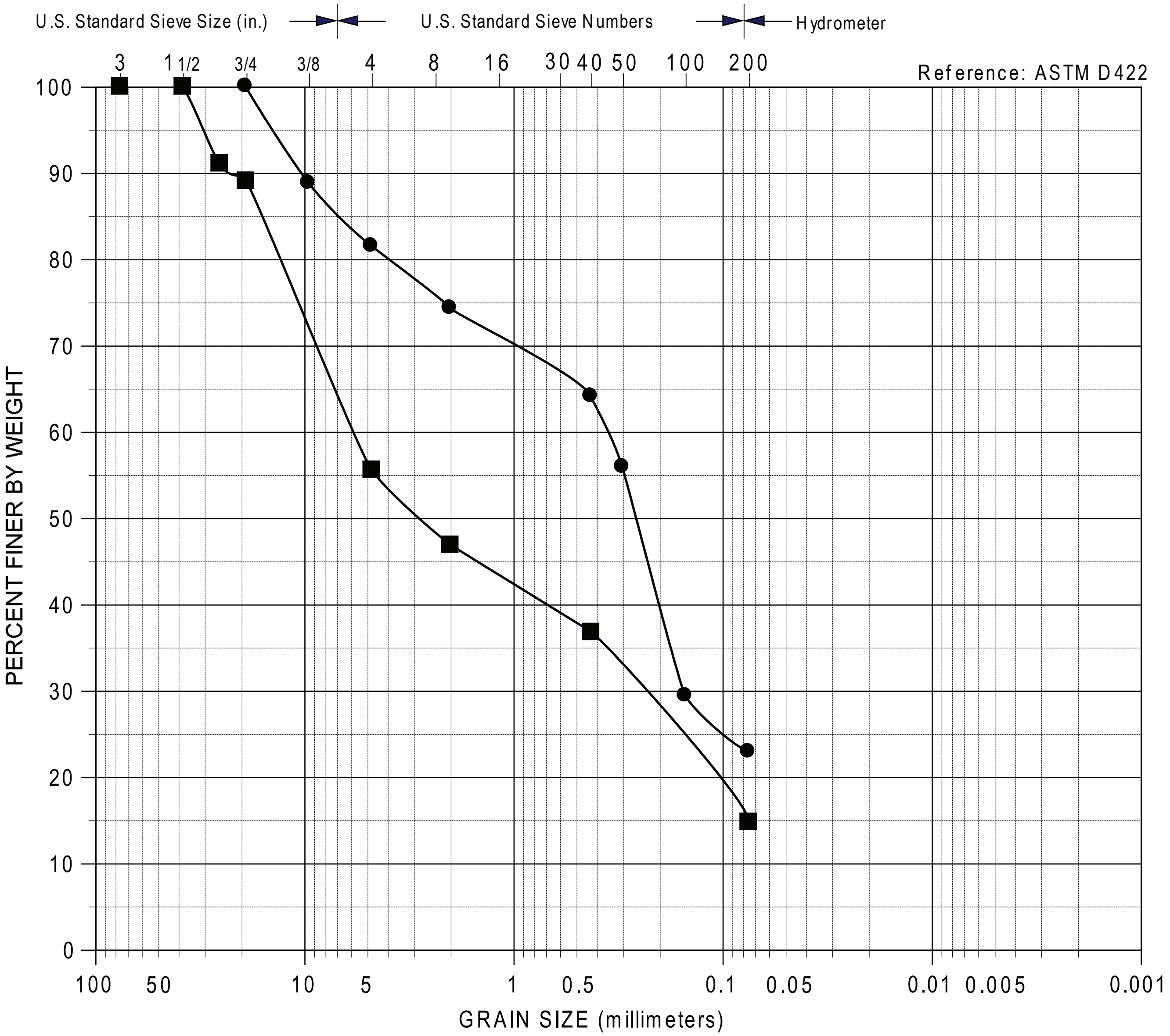
CLASSIFICATION CHART

Date 06/30/11 | Project No. 730509401 | Figure A-16

APPENDIX B
Laboratory Testing Results



Sample	% Gravel	%Sand	% Fines
B-5	18	59	23
B-7	45	40	15

Symbol	Sample Source	Classification
●	B-5 at 2.0 feet	SILTY SAND with GRAVEL (SM), gray and brown
■	B-7 at 2.0-4.5 feet	SILTY GRAVEL with SAND (GM), brown and gray

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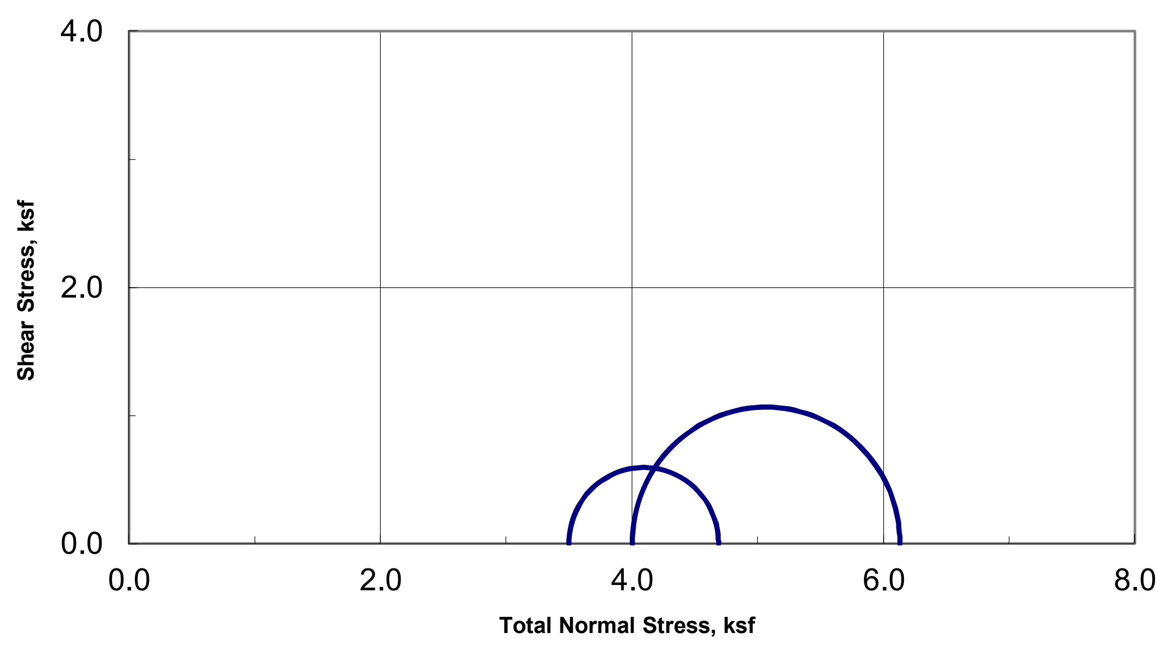
PARTICLE SIZE ANALYSIS

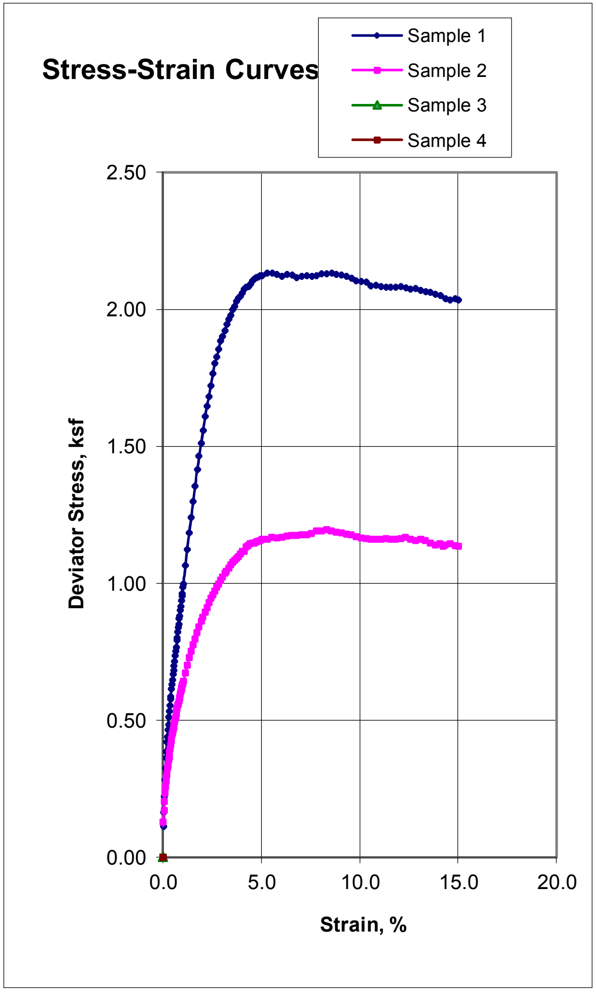
Date 07/14/11	Project No. 730509401	Figure B-1

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D-2850





	Sample Data			
	1	2	3	4
Moisture %	47.4	56.5		
Dry Den,pcf	73.8	66.5		
Void Ratio	1.284	1.536		
Saturation %	99.7	99.3		
Height in	5.02	5.01		
Diameter in	2.40	2.40		
Cell psi	27.8	24.3		
Strain %	5.30	8.30		
Deviator, ksf	2.133	1.196		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	**092-005**			
Client:	**T&R/RYCG**			
Project:	**Pier 94 Backlands/ 730509401**			
Boring:	B-11	B-14		
Sample:	11	8		
Depth ft:	40(Tip-4")	35(Tip-4")		

Visual Soil Description

Sample #	
1	Gr CLAY, trace brn org & Sa pockets (Bay Mud)
2	Gray CLAY, trace shell fragments (Bay Mud)
3	
4	

Remarks:

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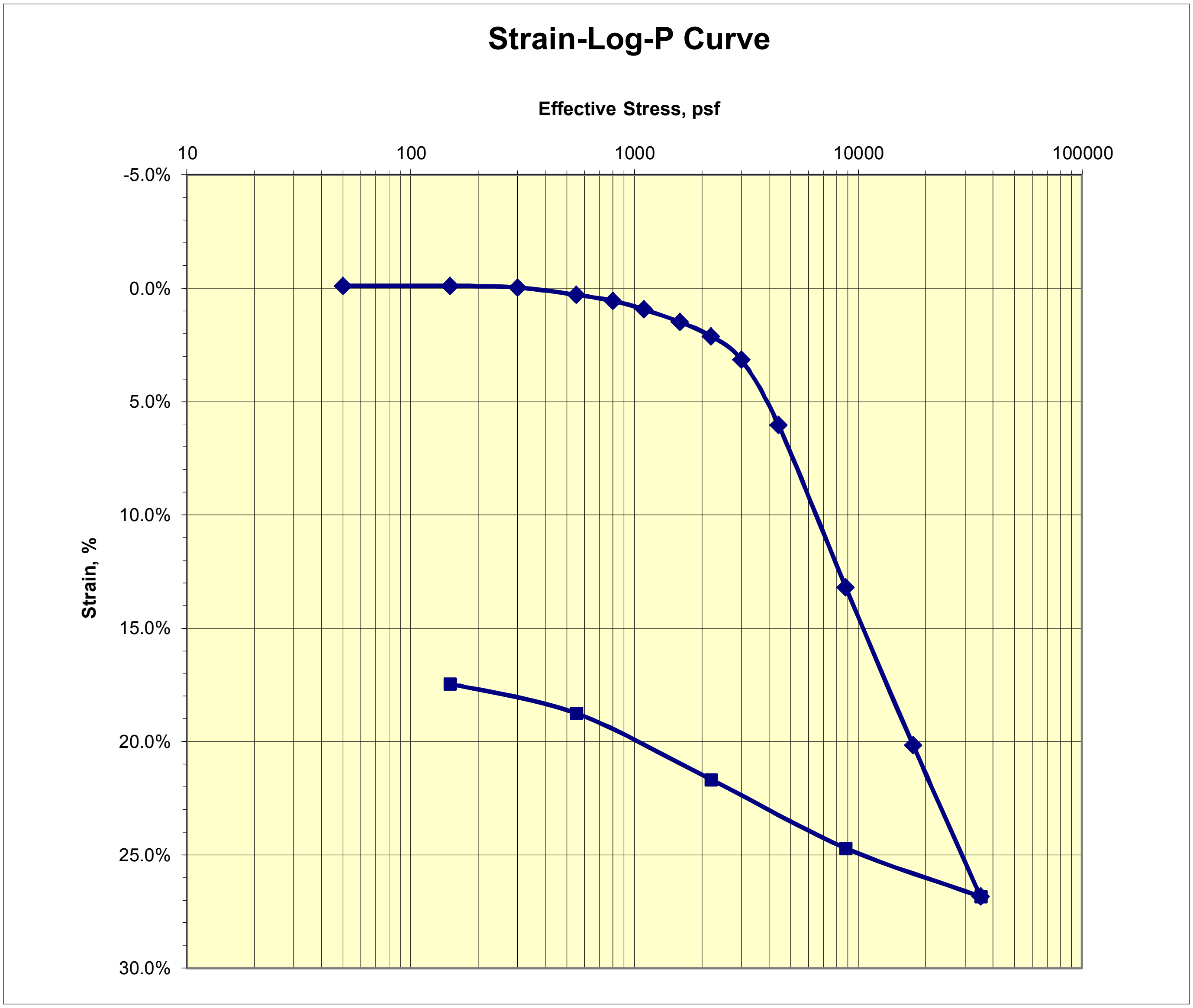
Figure B-2

COOPER TESTING LABORATORY

Consolidation Test

ASTM D2435

Job No.:	092-005	**Boring:**	B-11	**Run By:**	MD
Client:	Robert Y Chew Geotechnical	**Sample:**	11	**Reduced:**	PJ
Project:	Pier 94 Backlands - 2011-003	**Depth, ft.:**	40(Tip-3.5")	**Checked:**	PJ/DC
Soil Type:	Gray CLAY (Bay Mud) stiff			**Date:**	7/6/2011



Ass. Gs = 2.8	**Initial**	**Final**
Moisture %:	50.6	38.4
Dry Density, pcf:	71.7	84.3
Void Ratio:	1.437	1.074
Saturation:	98.7%	100%

Remarks:

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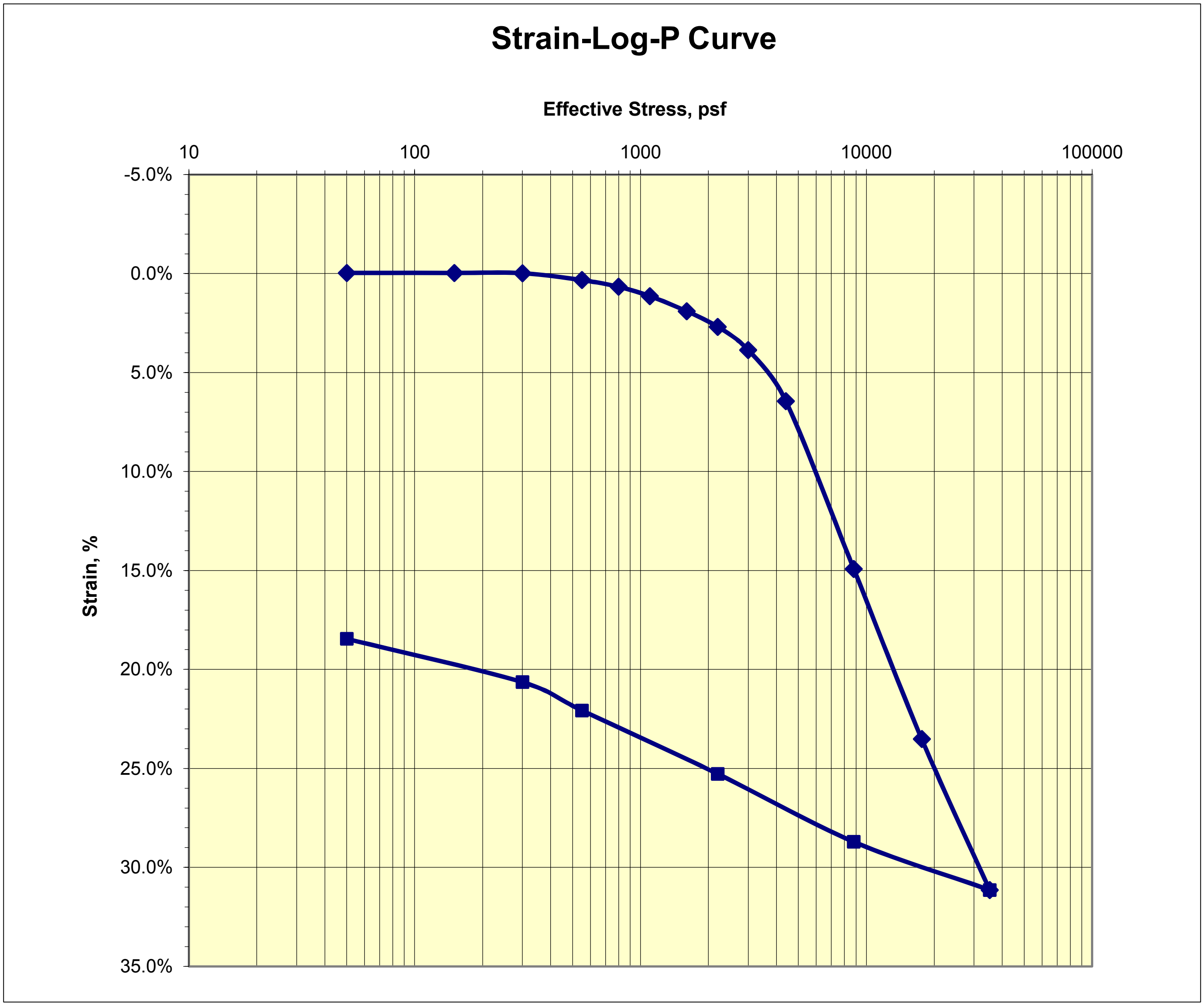
Figure B-3

COOPER
TESTING LABORATORY

Consolidation Test
ASTM D2435

Job No.:	092-005	**Boring:**	B-11	**Run By:**	MD
Client:	Robert Y Chew Geotechnical	**Sample:**	15	**Reduced:**	PJ
Project:	Pier 94 Backlands - 2011-003	**Depth, ft.:**	80(Tip-3.5")	**Checked:**	PJ/DC
Soil Type:	Gray CLAY (Bay Mud) stiff			**Date:**	7/6/2011

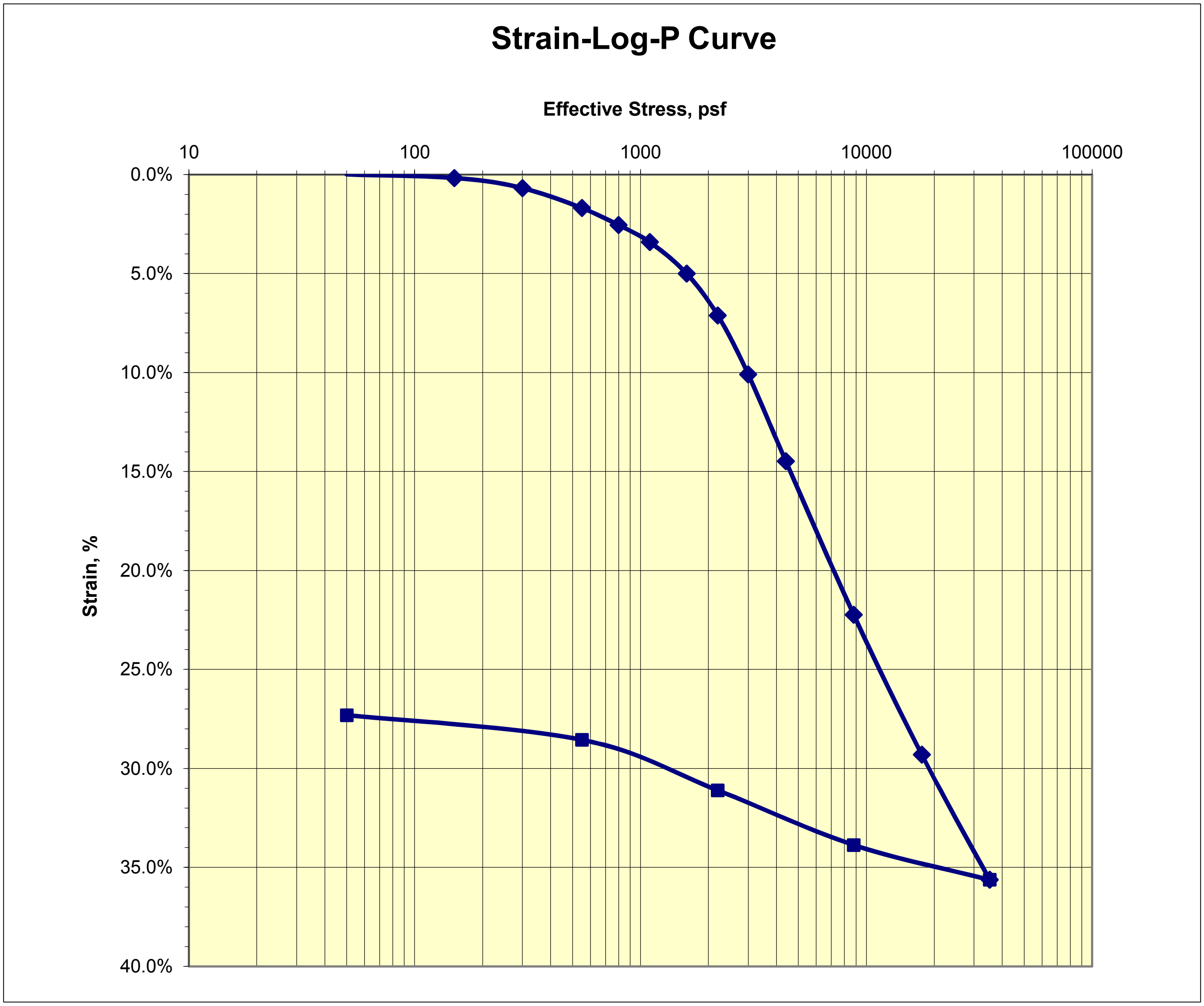


Ass. Gs = 2.8	**Initial**	**Final**
Moisture %:	56.4	42.6
Dry Density, pcf:	67.2	79.7
Void Ratio:	1.600	1.192
Saturation:	98.7%	100%

Remarks:

Consolidation Test
ASTM D2435

Job No.:	092-005	**Boring:**	B-14	**Run By:**	MD
Client:	Robert Y Chew Geotechnical	**Sample:**	8	**Reduced:**	PJ
Project:	Pier 34 Backlands - 2011-003	**Depth, ft.:**	35(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Gray CLAY (Bay Mud)			**Date:**	7/6/2011



Ass. Gs = 2.7	**Initial**	**Final**
Moisture %:	63.2	42.0
Dry Density, pcf:	61.6	79.0
Void Ratio:	1.734	1.133
Saturation:	98.3%	100%

Remarks:

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Figure B-5

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	092-005	**Boring:**	B-1	**Date:**	06/29/11
Client:	T&R/RYCG	**Sample:**	2B	**By:**	MD/PJ
Project No:	730509401	**Depth, ft.:**	4.5	**Remolded:**	

Visual Classification: Olive Gray SAND w/ Silt (Cemented)

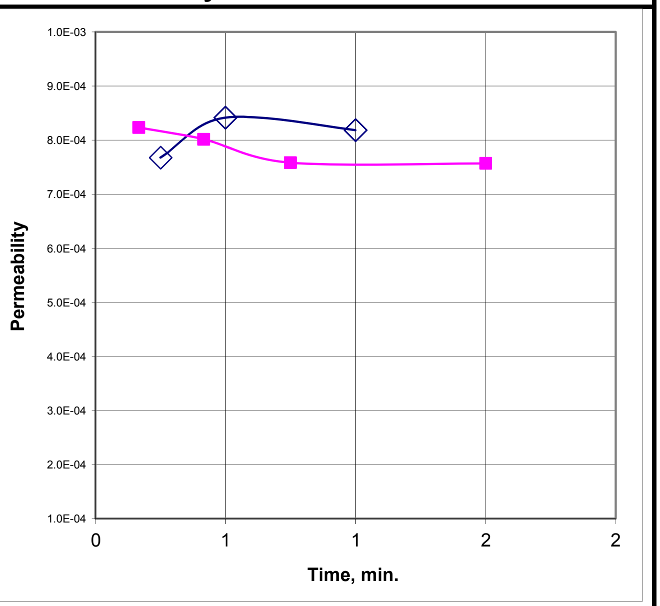
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
51.5	48.5	48.5	3

B: = >0.95 ("B" is an indication of saturation)

Max Hydraulic Gradient: = 6

Date	Minutes	Head, (in)	K,cm/sec
6/27/2011	0.00	14.80	Start of Test
6/27/2011	0.25	12.60	7.7E-04
6/27/2011	0.50	10.40	8.4E-04
6/27/2011	1.00	7.45	8.2E-04
6/28/2011	0.17	13.40	8.2E-04
6/28/2011	0.42	11.40	8.0E-04
6/28/2011	0.75	9.40	7.6E-04
6/28/2011	1.50	5.90	7.6E-04



Average Hydraulic Conductivity: 8.0E-04 cm/sec

Sample Data:	Initial (As-Received)		Final (At-Test)
Height, in	2.50		2.50
Diameter, in	2.41		2.43
Area, in2	4.56		4.64
Volume in3	11.40		11.59
Total Volume, cc	186.9		190.0
Volume Solids, cc	124.6		124.6
Volume Voids, cc	62.3		65.4
Void Ratio	0.5		0.5
Total Porosity, %	33.3		34.4
Air-Filled Porosity (θa),%	17.7		0.8
Water-Filled Porosity (θw),%	15.6		33.6
Saturation, %	46.9		97.7
Specific Gravity	2.65	Assumed	2.65
Wet Weight, gm	359.4		394.1
Dry Weight, gm	330.2		330.2
Tare, gm	0.00		0.00
Moisture, %	8.9		19.4
Wet Bulk Density, pcf	120.0		129.4
Dry Bulk Density, pcf	110.2		108.4
Wet Bulk Dens.ρb, (g/cm^3)	1.92		2.07
Dry Bulk Dens.ρb, (g/cm^3)	1.77		1.74

Remarks: The final dimensions and associated values are approximate because the sample slumped after the confining pressure was released.

Figure B-6

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	092-005	**Boring:**	B-2	**Date:**	06/29/11
Client:	T&R/RYCG	**Sample:**	4A	**By:**	MD/PJ
Project:	730509401	**Depth, ft.:**	5.5	**Remolded:**	

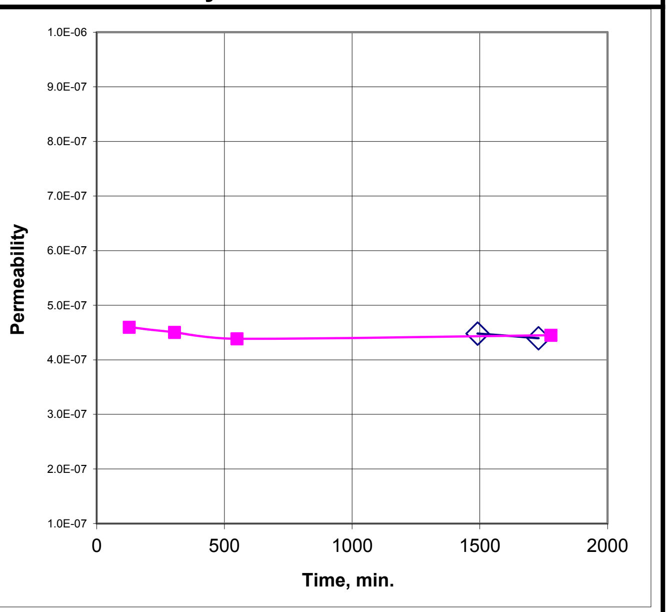
Visual Classification: Brown Olive and Gray Clayey SAND w/ Gravel

B: = >0.95 ("B" is an indication of saturation)

Max Hydraulic Gradient: = 17

Max Sample Pressures, psi:			
Cell:	**Bottom**	**Top**	**Avg. Sigma3**
62.5	59	58	4

Date	Minutes	Head, (in)	K,cm/sec
6/22/2011	0.00	15.00	**Start of Test**
6/23/2011	1491.00	8.60	4.5E-07
6/23/2011	1730.00	7.95	4.4E-07
6/24/2011	127.00	40.69	4.6E-07
6/24/2011	304.00	38.19	4.5E-07
6/24/2011	549.00	34.99	4.4E-07
6/25/2011	1778.00	22.39	4.5E-07



Average Hydraulic Conductivity: 4.5E-07 cm/sec

Sample Data:	**Initial (As-Received)**		**Final (At-Test)**
Height, in	2.53		2.49
Diameter, in	2.38		2.38
Area, in2	4.45		4.46
Volume in3	11.25		11.13
Total Volume, cc	184.3		182.4
Volume Solids, cc	121.1		121.1
Volume Voids, cc	63.2		61.3
Void Ratio	0.5		0.5
Total Porosity, %	34.3		33.6
Air-Filled Porosity (θa),%	3.6		0.8
Water-Filled Porosity (θw),%	30.7		32.8
Saturation, %	89.6		97.8
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	383.7		387.0
Dry Weight, gm	327.1		327.1
Tare, gm	0.00		0.00
Moisture, %	17.3		18.3
Wet Bulk Density, pcf	129.9		132.4
Dry Bulk Density, pcf	110.7		111.9
Wet Bulk Dens.ρb, (g/cm^3)	2.08		2.12
Dry Bulk Dens.ρb, (g/cm^3)	1.77		1.79

Remarks: Consolidated to 4 PSI. * 2"+/- brick in middle of sample may have impacted test.

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Figure B-7

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	092-005	**Boring:**	B-3	**Date:**	06/30/11
Client:	T&R/RYCG	**Sample:**	2B	**By:**	MD/PJ
Project:	730509401	**Depth, ft.:**	2	**Remolded:**	

Visual Classification: Dark Brown Clayey SAND w/ Gravel

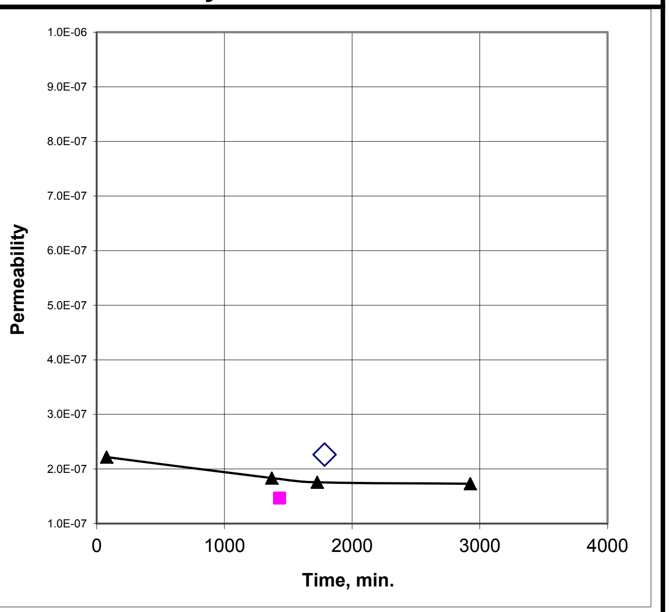
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
62	59.5	58.5	3

B: = >0.95 ("B" is an indication of saturation)

Max Hydraulic Gradient: = 17

Date	Minutes	Head, (in)	K,cm/sec
6/24/2011	0.00	42.69	Start of Test
6/25/2011	1784.00	30.59	2.3E-07
6/26/2011	1431.00	35.89	1.5E-07
6/27/2011	78.00	42.09	2.2E-07
6/28/2011	1371.00	34.89	1.8E-07
6/28/2011	1726.00	33.29	1.8E-07
6/29/2011	2925.00	28.59	1.7E-07



Average Hydraulic Conductivity: 1.9E-07 cm/sec

Sample Data:	Initial (As-Received)		Final (At-Test)
Height, in	2.53		2.51
Diameter, in	2.41		2.41
Area, in2	4.57		4.54
Volume in3	11.57		11.40
Total Volume, cc	189.6		186.8
Volume Solids, cc	136.1		136.1
Volume Voids, cc	53.5		50.7
Void Ratio	0.4		0.4
Total Porosity, %	28.2		27.1
Air-Filled Porosity (θa),%	7.4		0.3
Water-Filled Porosity (θw),%	20.8		26.9
Saturation, %	73.8		99.0
Specific Gravity	2.65	Assumed	2.65
Wet Weight, gm	400.1		410.8
Dry Weight, gm	360.6		360.6
Tare, gm	0.00		0.00
Moisture, %	10.9		13.9
Wet Bulk Density, pcf	131.7		137.2
Dry Bulk Density, pcf	118.7		120.5
Wet Bulk Dens.ρb, (g/cm^3)	2.11		2.20
Dry Bulk Dens.ρb, (g/cm^3)	1.90		1.93

Remarks: Consolidated to 3 PSI.

Figure B-8

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	092-005	**Boring:**	B-4	**Date:**	06/29/11
Client:	T&R/RYCG	**Sample:**	2B	**By:**	MD/PJ
Project:	730509401	**Depth, ft.:**	2	**Remolded:**	

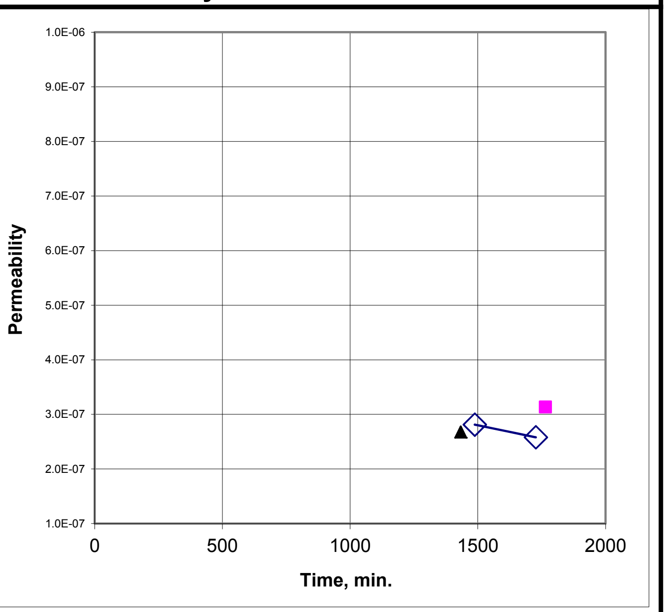
Visual Classification: Bluish-Gray and Green Silty SAND w/ Gravel

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 17

Max Sample Pressures, psi:			
Cell:	**Bottom**	**Top**	**Avg. Sigma3**
62	59.5	58.5	3

Date	Minutes	Head, (in)	K,cm/sec
6/22/2011	0.00	15.00	**Start of Test**
6/23/2011	1488.00	10.50	2.8E-07
6/23/2011	1727.00	10.25	2.6E-07
6/25/2011	1764.00	26.89	3.1E-07
6/26/2011	1433.00	30.99	2.7E-07



Average Hydraulic Conductivity: 2.8E-07 cm/sec

Sample Data:	**Initial (As-Received)**		**Final (At-Test)**
Height, in	2.50		2.49
Diameter, in	2.42		2.41
Area, in2	4.60		4.56
Volume in3	11.50		11.36
Total Volume, cc	188.4		186.1
Volume Solids, cc	100.9		100.9
Volume Voids, cc	87.5		85.2
Void Ratio	0.9		0.8
Total Porosity, %	46.4		45.8
Air-Filled Porosity (θa),%	17.1		2.0
Water-Filled Porosity (θw),%	29.3		43.8
Saturation, %	63.1		95.6
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	327.8		354.0
Dry Weight, gm	272.6		272.6
Tare, gm	0.00		0.00
Moisture, %	20.3		29.9
Wet Bulk Density, pcf	108.6		118.7
Dry Bulk Density, pcf	90.3		91.4
Wet Bulk Dens.ρb, (g/cm^3)	1.74		1.90
Dry Bulk Dens.ρb, (g/cm^3)	1.45		1.46

Remarks: Consolidated to 3 PSI.

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Figure B-9

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	092-005	**Boring:**	B-5	**Date:**	06/29/11
Client:	T&R/RYCG	**Sample:**	1B	**By:**	MD/PJ
Project:	730509401	**Depth, ft.:**	2.0	**Remolded:**	

Visual Classification: Gray and Brown Silty SAND w/ Gravel

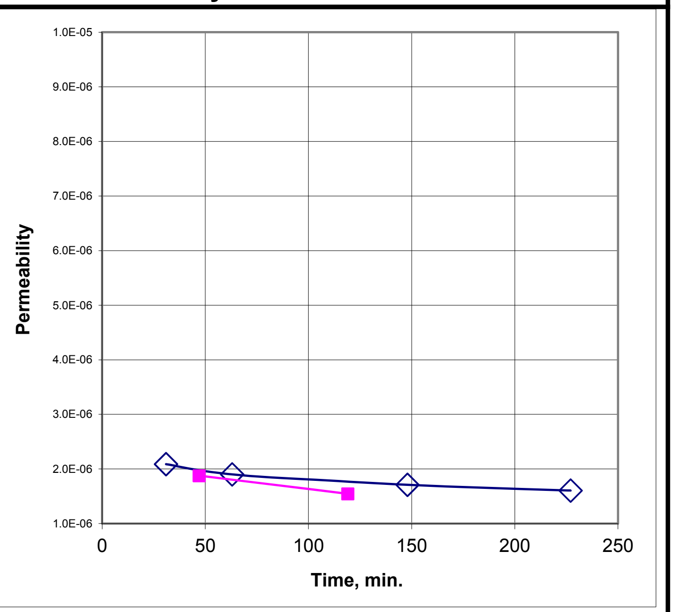
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
52	49.5	48.5	3

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 17

Date	Minutes	Head, (in)	K,cm/sec
6/23/2011	0.00	15.00	**Start of Test**
6/23/2011	31.00	14.20	2.1E-06
6/23/2011	63.00	13.55	1.9E-06
6/23/2011	148.00	12.10	1.7E-06
6/23/2011	227.00	11.00	1.6E-06
6/24/2011	47.00	39.64	1.9E-06
6/24/2011	119.00	36.59	1.5E-06



Average Hydraulic Conductivity: **1.8E-06** **cm/sec**

Sample Data:	**Initial (As-Received)**		**Final (At-Test)**
Height, in	2.46		2.50
Diameter, in	2.38		2.41
Area, in2	4.45		4.56
Volume in3	10.94		11.40
Total Volume, cc	179.3		186.9
Volume Solids, cc	128.5		128.5
Volume Voids, cc	50.8		58.4
Void Ratio	0.4		0.5
Total Porosity, %	28.4		31.2
Air-Filled Porosity (θa),%	3.2		1.3
Water-Filled Porosity (θw),%	25.2		29.9
Saturation, %	88.9		95.7
Specific Gravity	2.65	Assumed	2.65
Wet Weight, gm	385.7		396.4
Dry Weight, gm	340.5		340.5
Tare, gm	0.00		0.00
Moisture, %	13.3		16.4
Wet Bulk Density, pcf	134.2		132.4
Dry Bulk Density, pcf	118.5		113.7
Wet Bulk Dens.ρb, (g/cm^3)	2.15		2.12
Dry Bulk Dens.ρb, (g/cm^3)	1.90		1.82

Remarks: Consolidated to 3 PSI.

Figure B-10

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

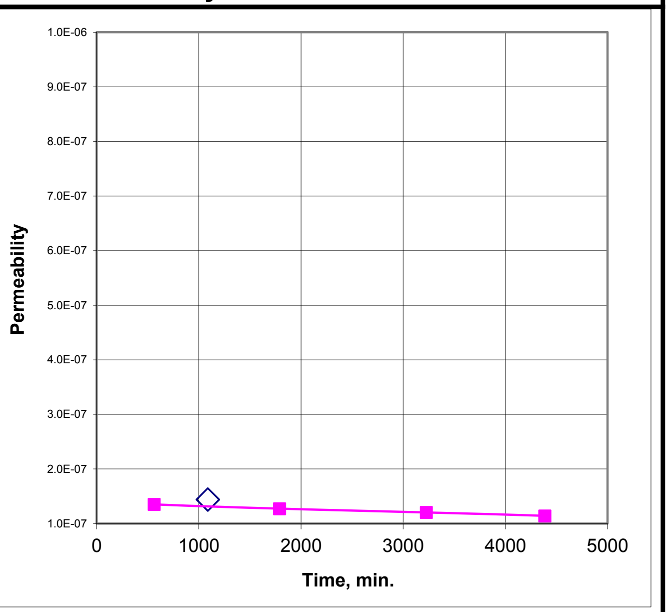
Job No: 092-005 **Boring:** B-8 **Date:** 06/29/11
Client: T&R/RYCG **Sample:** 4A **By:** MD/PJ
Project: 730509401 **Depth, ft.:** 5.5 **Remolded:**
Visual Classification: Gray SAND w/ Gravel

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 17

Max Sample Pressures, psi:			
Cell:	**Bottom**	**Top**	**Avg. Sigma3**
63	59.5	58.5	4

Date	Minutes	Head, (in)	K,cm/sec
6/23/2011	0.00	15.00	**Start of Test**
6/24/2011	1087.00	13.10	1.4E-07
6/24/2011	562.00	39.99	1.4E-07
6/25/2011	1788.00	35.19	1.3E-07
6/26/2011	3225.00	30.69	1.2E-07
6/27/2011	4386.00	27.99	1.1E-07



Average Hydraulic Conductivity: 1.3E-07 cm/sec

Sample Data:	**Initial (As-Received)**	**Final (At-Test)**
Height, in	2.51	2.48
Diameter, in	2.41	2.43
Area, in2	4.56	4.62
Volume in3	11.45	11.46
Total Volume, cc	187.6	187.9
Volume Solids, cc	133.5	133.5
Volume Voids, cc	54.1	54.3
Void Ratio	0.4	0.4
Total Porosity, %	28.8	28.9
Air-Filled Porosity (θa),%	3.6	0.7
Water-Filled Porosity (θw),%	25.2	28.2
Saturation, %	87.5	97.5
Specific Gravity	2.70 Assumed	2.70
Wet Weight, gm	407.9	413.5
Dry Weight, gm	360.5	360.5
Tare, gm	0.00	0.00
Moisture, %	13.1	14.7
Wet Bulk Density, pcf	135.7	137.4
Dry Bulk Density, pcf	119.9	119.8
Wet Bulk Dens.ρb, (g/cm^3)	2.17	2.20
Dry Bulk Dens.ρb, (g/cm^3)	1.92	1.92

Remarks: Consolidated to 4 PSI.

Figure B-11

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No: 092-005 **Boring:** B-11 **Date:** 06/29/11
Client: T&R/RYCG **Sample:** 2A **By:** MD/PJ
Project: 730509401 **Depth, ft.:** 3.5 **Remolded:**
Visual Classification: Bluish-Gray Clayey SAND w/ Gravel

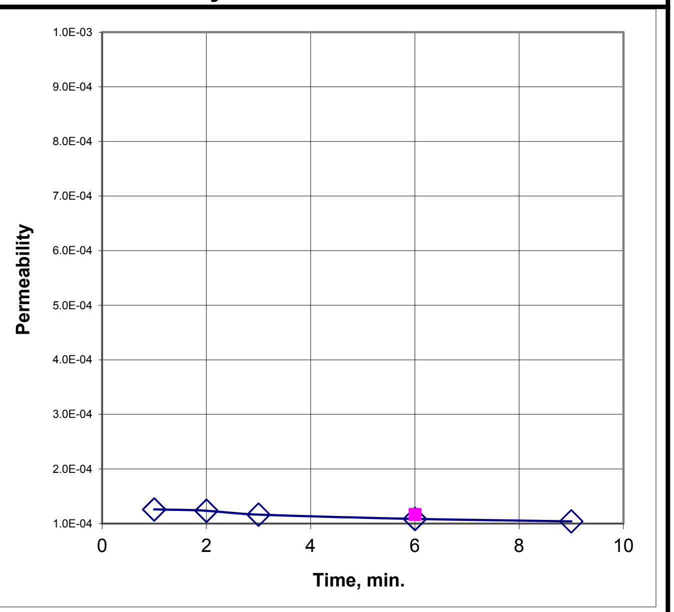
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
52	49	49	3

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 6

Date	Minutes	Head, (in)	K,cm/sec
6/23/2011	0.00	15.00	**Start of Test**
6/23/2011	1.00	13.50	1.3E-04
6/23/2011	2.00	12.20	1.2E-04
6/23/2011	3.00	11.20	1.2E-04
6/23/2011	6.00	8.70	1.1E-04
6/23/2011	9.00	6.85	1.0E-04
6/24/2011	6.00	8.40	1.2E-04



Average Hydraulic Conductivity: **1.2E-04** **cm/sec**

Sample Data:	**Initial (As-Received)**	**Final (At-Test)**
Height, in	2.52	2.52
Diameter, in	2.40	2.40
Area, in2	4.52	4.52
Volume in3	11.40	11.38
Total Volume, cc	186.8	186.5
Volume Solids, cc	119.0	119.0
Volume Voids, cc	67.8	67.5
Void Ratio	0.6	0.6
Total Porosity, %	36.3	36.2
Air-Filled Porosity (θa),%	13.4	1.5
Water-Filled Porosity (θw),%	22.9	34.6
Saturation, %	63.0	95.7
Specific Gravity	2.65 Assumed	2.65
Wet Weight, gm	358.1	380.0
Dry Weight, gm	315.4	315.4
Tare, gm	0.00	0.00
Moisture, %	13.5	20.5
Wet Bulk Density, pcf	119.6	127.1
Dry Bulk Density, pcf	105.3	105.5
Wet Bulk Dens.ρb, (g/cm^3)	1.92	2.04
Dry Bulk Dens.ρb, (g/cm^3)	1.69	1.69

Remarks: Consolidated to 3 PSI.

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A Joint Venture

Figure B-12

APPENDIX C
SETTLEMENT ANALYSES

C1.0 INTRODUCTION

The Pier 94 Backlands was created during the 1960s and 1970s. The western portion of Pier 94 Backlands is outside the regulated landfill area and was created by placing fill prior to 1961 (historic fill). The eastern portion of Pier 94 Backlands is within the regulated landfill area and was constructed in the 1960s and 1970s by constructing a perimeter debris dike and placing fill within the dike. Current plans show up to about 18 feet of new fill will be placed for the proposed Pier 94 Backlands improvements.

The project site will undergo settlement caused by the compression and decomposition of the refuse (construction debris and dredge spoils) and consolidation of the Bay Mud due to the weight of existing and new fill. Settlement of underlying refuse and Bay Mud will impact the design of underground utilities, surface drainage, and new buildings and structures.

We have estimated the anticipated settlement due to existing and proposed new fill at 17 locations across the site. The 17 locations, designated as settlement points 1 through 17, are shown on Figure 10. Methodology and results of our settlement analyses are presented in this appendix.

C2.0 SETTLEMENT ANALYSES

The existing fill and refuse are underlain by highly compressible clay, locally known as Bay Mud. Based on our investigation, the Bay Mud is 35 to 50 feet thick at the site. The Bay Mud will settle as external loads are placed.

Refuse settlement under its self-weight and external loads consists of primary settlement and secondary compression. Primary settlement, due to consolidation of refuse under self-weight and external loads has been reported to occur anywhere between 1 and 5 years after load application. Secondary compression, due to decomposition and creep processes, decreases with time and depth of waste fill. The majority of secondary compression is completed within 50 years after load application (Sharma and Lewis 1994).

We performed settlement analyses to estimate future settlement at 17 locations, designated settlement points 1 through 17. Settlement was calculated at each of the 17 locations for three loading conditions:

1) refuse self-weight, if present, 2) existing fill, and 3) new fill. A discussion of soil profiles, loading conditions, settlement model, and selection of settlement parameters is presented below.

C2.1 Generalize Soil Profiles

Generalized soil profiles were developed for each of the 17 settlement locations based on information presented on boring logs from our field investigation and previous borings drilled by others (GRC 1989 and PBQD 2005).

C2.2 Load Cases

Settlement was calculated at each of the 17 settlement locations from the weight of existing fill and proposed (new) fill. We considered settlement of refuse and Bay Mud under existing conditions (load of existing fill and refuse). Existing fill consists of fill placed prior to 1961 (west of the 1961 shoreline), debris dike, dredge spoils, construction debris, and soil cap. Placement of the existing fill was completed between 1946 and 1977. We concluded settlement of Bay Mud under the weight of existing fill is 70 to over 95 percent complete. In addition, we concluded primary and secondary settlement of refuse under self-weight and existing soil cap is essentially complete (over 95 percent complete).

For the proposed improvements, we anticipated up to 18 feet of new fill will be placed on the refuse and Bay Mud. We calculated settlement from consolidation of Bay Mud and compression of refuse between 2012 and 2062 (50 years after load application).

C2.3 Methodology

Settlement of Bay Mud due to the weight of existing and new fill was calculated using conventional consolidation theory. Primary compression index and preconsolidation pressures were obtained from laboratory tests of representative soil samples.

Settlement of refuse due to self-weight and the weight of existing and new fill was evaluated using (1) Gibson and Lo and (2) Sowers settlement models.

C2.3.1 Bay Mud Consolidation

We evaluated ground settlement as a result of consolidation of the Bay Mud due to existing and new fill. Existing fill consists of historic fill placed prior to 1961 (west of the 1961 shoreline), debris dike, dredge spoils, construction debris, and soil cap. Placement of the existing fill was completed between 1946 and 1977. Results of our settlement analyses indicate consolidation of Bay Mud due to existing fill is 70 to over 95 percent complete. For the proposed improvements, up to 18 feet of new fill will be placed to reach finish grade. Assuming the new fill will be placed over a one year period, we estimate up to about three feet of ground settlement will occur in the next 50 years as a result of Bay Mud consolidation due to the weight of the existing and new fill.

C2.3.2 Refuse Settlement

Gibson and Lo Settlement Model

We evaluated the settlement of refuse using the Gibson and Lo settlement model (Edil et al. 1990). The Gibson and Lo model is a rheological model found to be useful in predicting the settlement of peat. The Gibson and Lo model consists of two springs in a series. When a stress increment acts on the model, the first spring in the series compresses instantaneously; this is analogous to primary compression. The initial compression of the second spring in the series is retarded by the dashpot. The sustained load is transferred progressively from the dashpot to the second spring; this is similar to the continuous process of secondary compression under a sustained effective stress. After a long period of time (i.e. in the secondary compression range), the full effective stress will be taken by the two springs, thus the dashpot will sustain no load.

The equation for refuse settlement based on the Gibson and Lo model is:

$$S(t) = H(\Delta\delta)\{a+b[1-\exp(-\lambda/bt)]\}$$

where "S(t)" is settlement at time "t", "H" is the initial height of refuse, "Δδ" is change in overburden pressure, "a" is primary compressibility parameter, "b" is secondary compressibility parameter, "λ/b" is the rate of secondary compression, and "t" is time since load application.

Settlement parameters a, b, and λ/b were selected based the type of landfill constituents described on the logs of borings drilled at the site, and the refuse site settlement data used in the Gibson and Lo model (Edil et al. 1990). We estimated settlement parameters based on settlement data from a site in

Wisconsin (Site A) where refuse placement condition consists of fresh refuse with active filling. For computing the settlement of refuse under self-weight, settlement parameters were selected for the placement of fresh refuse with active filling. The values of settlement parameters a, b, and λ/b selected for our settlement analysis are 0.000126 (1/kPa), 0.000464 (1/kPa), and 0.001816 (1/day), respectively.

Edil et al. (1990) estimates the Gibson and Lo model predicts settlement with an accuracy of between 2 and 20 percent of the actual settlement. Therefore, the computed refuse settlement has been increased by 20 percent to accommodate the predictive uncertainties associated with the Gibson and Lo analyses procedure.

Sowers Settlement Model

We estimated refuse settlement due to primary and secondary compressions due to the weight of new fill using the Sowers method. Sowers method is based on consolidation theory and has separate equations to calculate primary settlements and secondary compression. To calculate primary settlements, the following equation is used:

$$S = (H)C_{ec}\{\log[(P_o+dP)/ P_o]\}$$

Where "S" is primary compression occurring in the layer under consideration, "H" is the initial thickness of the waste layer under consideration, "C_{ec}" is the primary compression ratio, "P_o" is the existing overburden pressure acting at the midlevel of the layer, and "dP" is the increment of overburden pressure at the midlevel of the layer.

For older landfills (10 to 15 years) which are subjected to external loads, primary compression ratio values range from 0.1 to 0.4 (U.S. Dept. of the Navy, 1982). The primary compression ratio values of 0.1 and 0.4 are representative of older landfills with low and high organic content, respectively. For settlement analyses, we selected primary compression ratio value of 0.1, 0.2, and 0.3 for low, medium, and high estimates of refuse settlement due to new fill, respectively.

Secondary compression is estimated using

$$S_s = H[C_\alpha/(1+e_o)][\log(t_2/ t_1)]$$

where "S_s" is the secondary compression occurring in layer under consideration, "H" is the initial thickness of waste layer under consideration, "C_α" is the secondary compression index, "e_o" is the initial void ratio of the layer, "t_1" is the starting time for the long-term time period under consideration, and "t_2" is the ending time for the long-term time period under consideration.

NAVFAC DM7.3 (U.S. Dept. of the Navy 1982) recommends secondary compression index values ranging from 0.02 to 0.07 for landfills between 10 and 15 years old. Oweis and Khera (1990) recommend secondary compression index values between 0.01 and 0.04. For settlement analyses, we selected secondary compression index values of 0.02, 0.03, and 0.04 for low, medium, and high estimates of refuse settlement due to new fill, respectively.

C3.0 CONCLUSIONS

Results of our settlement analyses indicate consolidation of Bay Mud due to existing fill is 70 to over 95 percent complete. For the proposed improvements, up to 18 feet of new fill will be placed to reach finish grade. Assuming the new fill will be placed over a one year period, we estimate up to about three feet of ground settlement will occur in the next 50 years as a result of Bay Mud consolidation due to existing and new fill.

For each of the 17 locations, the existing ground elevation, existing Bay Mud thickness, existing fill thickness, proposed finish grade and new fill thickness, and future Bay Mud settlement are summarized in Table C-1.

TABLE C-1

Settlement from Primary Consolidation of Bay Mud due to Existing and New Fill

Settlement Point	Existing Ground Elevation [15] (feet)	Existing Bay Mud Thickness (feet)	Existing Fill Thickness (feet)	Finish Grade Elevation [16] (feet)	New Fill Thickness (feet)	Settlement due to Primary Consolidation of Bay Mud in Next 50 Years (feet)
1	1.5	35.5	41.0	1.5	0.0	0.0
2	5.0	49.5	30.5	5.0	0.0	0.6
3	0.0	50.0	25.0	0.0	0.0	0.5
4	10.0	49.5	35.5	28.0	18.0	3.0
5	20.0	49.5	45.5	32.0	12.0	2.2
6	21.0	39.5	46.5	31.0	10.0	1.4
7	15.0	35.0	35.0	29.0	14.0	1.4
8	0.0	35.0	20.0	12.5	12.5	1.9
9	0.0	40.0	25.0	10.0	10.0	1.8
10	0.0	49.5	25.5	5.5	5.5	1.5
11	0.0	49.5	25.5	7.0	7.0	1.8
12	0.0	49.5	25.5	5.5	5.5	1.5
13	0.0	49.5	25.5	0.0	0.0	0.6
14	10.0	49.5	35.5	10.0	0.0	0.7
15	16.0	40.0	41.0	27.0	11.0	1.6
16	10.0	49.5	35.5	24.0	14.0	2.4
17	5.0	49.5	30.5	5.0	0.0	0.6

The settlements presented in Table C-1 are from primary consolidation of Bay Mud only; secondary compression settlement may also occur. Historical data in the vicinity indicates additional settlement of 1/2 to 1 inch may occur every 10 years as a result of secondary compression of Bay Mud.

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[15] Existing ground elevation is based on site topographic survey provided electronically by Port of San Francisco on 8 April 2011.

[16] Proposed finish grade from 30 percent design plan, titled "Pier 94 Backlands Improvements, Proposed Grading and Storm Drainage Plan, Drawing No. R-2" dated December 2009, prepared by Port of San Francisco, Department of Engineering.

Results from the settlement analyses using the Gibson and Lo model and Sowers model indicate primary and secondary settlement of refuse from self-weight and existing fill is essentially complete. Placement of new fill will initiate a new cycle of consolidation settlement of the refuse. We calculated refuse settlement due to the new fill at settlement points 4, 5, 6, and 15; no new fill will be placed at settlement points 1, 2, 14, and 17.

Edil et al. (1990) estimates that the Gibson and Lo model predicts settlement with an accuracy between 2 to 20 percent of the actual settlement. Therefore, the computed landfill settlement using the Gibson and Lo model has been increased by 20 percent. A summary of refuse settlement calculated using the Gibson and Lo model and Sowers model is presented in Table C-2.

TABLE C-2

Refuse Settlement Resulting from the Placement of New Fill

Settlement Point	Existing Ground Elevation (feet)	Finish Grade Elevation (feet)	New Fill Thickness (feet)	Gibson and Lo Settlement[17] (feet)	Sowers Settlement (feet)
1	1.5	1.5	0.0	0.0	0.0
2	5.0	5.0	0.0	0.0	0.0
4	10.0	28.0	18.0	2.4	2.9
5	20.0	32.0	12.0	1.9	2.1
6	21.0	31.0	10.0	1.6	1.5
14	10.0	10.0	0.0	0.0	0.0
15	16.0	27.0	11.0	1.7	1.5
17	5.0	5.0	0.0	0.0	0.0

Total settlement from Bay Mud and refuse due to existing and new fill are presented in Table C-3 for settlement points 1 through 17. The total settlement is expected to occur over a period of 50 years from the time of new load application.

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[17] Computed settlement using the Gibson and Lo model has been increased by 20 percent.

TABLE C-3

Estimated Total Settlement for 50-Year Period from Time of Load Application

Settlement Point	Existing Ground Elevation (feet)	Finish Grade Elevation (feet)	New Fill Thickness (feet)	Settlement from Bay Mud (feet)	Settlement from Refuse, Gibson and Lo Model (feet)	Total Settlement from Bay Mud and Refuse (feet)
1	1.5	1.5	0.0	0.0	0.0	0.0
2	5.0	5.0	0.0	0.6	0.0	0.6
3	0.0	0.0	0.0	0.5	0.0	0.5
4	10.0	28.0	18.0	3.0	2.4	5.4
5	20.0	32.0	12.0	2.2	1.9	4.1
6	21.0	31.0	10.0	1.4	1.6	3.0
7	15.0	29.0	14.0	1.4	0.0	1.4
8	0.0	12.5	12.5	1.9	0.0	1.9
9	0.0	10.0	10.0	1.8	0.0	1.8
10	0.0	5.5	5.5	1.5	0.0	1.5
11	0.0	7.0	7.0	1.8	0.0	1.8
12	0.0	5.5	5.5	1.5	0.0	1.5
13	0.0	0.0	0.0	0.6	0.0	0.6
14	10.0	10.0	0.0	0.7	0.0	0.7
15	16.0	27.0	11.0	1.6	1.5	3.1
16	10.0	24.0	14.0	2.4	0.0	2.4
17	5.0	5.0	0.0	0.6	0.0	0.6

The settlements presented in Table C-3 do not include settlement from secondary compression of Bay Mud. As previously discussed, additional settlement of 1/2 to 1 inch may occur every 10 years as a result of secondary compression of Bay Mud.

APPENDIX D

HYDROLOGIC EVALUATION OF LANDFILL COVER PERFORMANCE

D1.0 INTRODUCTION

The objective of our hydrologic study was to evaluate the performance of existing and proposed landfill cover systems during post-closure operations. We performed the evaluation using the computer program titled *Hydrologic Evaluation of Landfill Performance - Version 3.07* (HELP-3). The computer program was developed by Environmental Laboratory USAE Waterways Experiment Station for USEPA Risk Reduction Engineering Laboratory. The program was developed for water balance analysis of landfill cover systems for solid waste disposal and containment facilities.

HELP-3 is a quasi-two-dimensional hydrologic model of water movement across, into, through and out of landfills. HELP-3 accepts weather and cover design data, and evaluates the amount of percolation through landfill cover systems by taking into account of surface storage, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, and lateral subsurface drainage.

D2.0 HELP ANALYSIS

Hydrologic processes modeled by HELP-3 are divided into two categories: surface processes and subsurface processes. Surface processes include interception of rainfall by vegetation, surface runoff, and surface evaporation. Subsurface processes include evaporation from the soil profile, plant transpiration, unsaturated vertical drainage, saturated lateral drainage, and barrier soil liner percolation.

Daily infiltration into the landfill is determined indirectly from a surface water balance. Infiltration is assumed to equal the sum of rainfall, surface storage, and snowmelt, minus the sum of runoff, additional storage in snowpack, and evaporation of surface water.

The first subsurface processes considered are soil evaporation and plant transpiration from the evaporative zone. The other subsurface processes (vertical drainage, lateral drainage, and percolation) are modeled one layer at a time, from top to bottom.

We performed HELP-3 analyses to evaluate the amount of percolation through three landfill cover designs: 1) existing, 2) prescriptive, and 3) engineered alternative. The existing, prescriptive, and engineered alternative covers were analyzed to determine percolation due to precipitation, and each of the cases was evaluated for a 30-year period.

D2.1 Existing Cover

Generalized existing cover profiles were developed for each of the 12 borings (B-1 through B-11 and B-15) drilled within the regulated landfill area by T&R/RYCG in May 2011. The soil cap (existing cover) consists of loose to very dense sands and gravels with variable amounts of clay and silt and occasional concrete, brick, and serpentinite fragments. Where explored, the existing cover bottoms 2.5 to 8 feet below the ground surface (bgs). Generalized existing cover layers for HELP-3 are presented in Table D-1.

Table D-1

Existing Cover Profile

Boring Location	Layer Description (top to bottom)	Layer Type	Thickness (inches)	Soil Texture	Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity (cm/sec)
B-1	Sand	1	51	23	0.333	0.300	0.203	8.0×10^{-4}
	Sand with Silt	1	21	23	0.333	0.300	0.203	8.0×10^{-4}
B-2	Silty Sand with Gravel	1	18	22	0.374	0.307	0.180	1.0×10^{-6}
	Gravel with Sand	1	48	21	0.397	0.032	0.013	3.0×10^{-1}
	Clayey Sand with Gravel	1	30	24	0.343	0.305	0.202	4.5×10^{-7}
B-3	Clayey Sand with Gravel	1	72	27	0.282	0.281	0.280	1.9×10^{-7}
	Sandy Clay	1	30	25	0.437	0.373	0.266	3.6×10^{-6}
B-4	Silty Sand with Gravel	1	30	22	0.374	0.307	0.180	1.0×10^{-6}
B-5	Silty Sand with Gravel	1	72	24	0.374	0.305	0.202	1.0×10^{-6}
B-6	Silty Sand with Gravel	1	21	24	0.374	0.305	0.202	1.0×10^{-6}
	Sand	1	6	22	0.419	0.307	0.180	1.9×10^{-5}
	Clayey Sand	1	24	24	0.304	0.303	0.202	2.6×10^{-7}

Boring Location	Layer Description (top to bottom)	Layer Type	Thickness (inches)	Soil Texture	Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity (cm/sec)
B-7	Sand with Silt and Gravel	1	12	22	0.419	0.307	0.180	1.9x10^{-5}
	Gravel with Sand	1	54	21	0.397	0.032	0.013	3.0x10^{-1}
	Clayey Sand	1	12	24	0.304	0.303	0.202	2.6x10^{-7}
B-8	Silty Sand	1	12	22	0.374	0.307	0.180	1.0x10^{-6}
	Clayey Sand with Gravel	1	42	27	0.288	0.287	0.286	1.3x10^{-7}
	Clayey Sand with Gravel	1	42	27	0.288	0.287	0.286	1.3x10^{-7}
B-9	Sandy Silt	1	21	23	0.461	0.360	0.203	9.0x10^{-6}
	Clayey Sand with Gravel	1	6	24	0.304	0.303	0.202	2.6x10^{-7}
	Clayey Sand with Gravel	1	27	24	0.304	0.303	0.202	2.6x10^{-7}
B-10	Gravel with Sand and Silt	1	24	21	0.397	0.032	0.013	3.0x10^{-1}
	Silty Clayey Sand with Gravel	1	36	27	0.400	0.366	0.288	7.8x10^{-7}
	Sandy Clay	1	18	26	0.445	0.393	0.277	1.9x10^{-6}
B-11	Silty Sand with Gravel	1	18	24	0.374	0.305	0.202	1.0x10^{-6}
	Gravel with Sand and Silt	1	9	21	0.397	0.032	0.013	3.0x10^{-1}
	Silty Sand	1	9	23	0.461	0.360	0.203	9.0x10-^{6}
	Clayey Sand with Gravel	1	12	24	0.365	0.305	0.202	2.7x10^{-6}
B-15	Sand with Silt and Gravel	1	60	22	0.419	0.307	0.180	1.9x10^{-5}

D2.2 Prescriptive Cover

Prescriptive cover designated by the State Water Resources Control Board (SWRCB) and the California Integrated Waste Management Board (CIWMB) Regulations Division 2, Title 27, and Section 21090 for Class III landfills consists of the following layers, from top to bottom:

- Erosion-resistant layer (via vegetative layer): at least one foot of soil that contains no waste and is capable of sustaining native or other plant growth.
- Low hydraulic conductivity layer: at least one foot of soil containing no waste or leachate and compacted to attain a hydraulic conductivity of $1x10^{-6}$ cm/sec or less.
- Foundation layer: at least two feet of soil, contaminated soil, incinerator ash, or other waste materials, provided that such materials have appropriate engineering properties to be used for a foundation layer.

The existing cover at the site is at least two feet thick and meets the requirements for foundation layer. We modeled the prescriptive cover at boring B-2 and B-10 locations by modeling a one-foot-thick layer of low hydraulic conductivity material over the existing cover, and a one-foot-thick layer of vegetative fill over the low hydraulic conductivity layer. Boring B-2 and B-10 locations represent areas explored with the lowest and highest average annual percolation, respectively. Generalized prescriptive cover layers for HELP-3 are presented in Table D-2.

Table D-2

Prescriptive Cover Profile

	Layer Description (top to bottom)	Layer Type	Thickness (inch)	Soil Texture	Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity (cm/sec)
Prescriptive Cover (Boring B-2)	Erosion resistant layer	1	12	7	0.473	0.222	0.104	$5.2x10^{-4}$
	Low conductivity layer	1	12	28	0.452	0.411	0.311	$1.0x10^{-6}$
	Silty Sand with Gravel	1	18	22	0.374	0.307	0.180	$1.04x10^{-6}$
	Gravel with Sand	1	48	21	0.397	0.032	0.013	$3.0x10^{-1}$
	Clayey Sand with Gravel	1	30	24	0.343	0.305	0.202	$4.5x10^{-7}$
Prescriptive Cover (Boring B-10)	Erosion resistant layer	1	12	7	0.473	0.222	0.104	$5.2x10^{-4}$
	Low conductivity layer	1	12	28	0.452	0.411	0.311	$1.0x10^{-6}$
	Gravel with Sand and Silt	1	24	21	0.397	0.032	0.013	$3.0x10^{-1}$
	Silty Clayey Sand with Gravel	1	36	27	0.400	0.366	0.288	$7.8x10^{-7}$
	Sandy Clay	1	18	26	0.445	0.393	0.277	$1.9x10^{-6}$

D2.3 Engineered Alternative Cover

Post-closure development plans for the project site may include two engineered alternative covers: (1) a vegetated swale consisting of 12 inches of vegetation soil layer underlain by a low hydraulic conductivity geomembrane liner over the existing cover, and (2) asphalt concrete pavement section consisting of asphalt concrete and aggregate base underlain by a low hydraulic conductivity geomembrane liner. The vegetated swale engineered alternative was modeled above the existing cover encountered at borings B-3, B-5, and B-6; these borings are located along the proposed vegetated swale. The asphalt concrete pavement engineered alternative was modeled above the existing cover encountered at borings B-2 and B-10; these locations represent areas explored with the lowest and highest average annual percolation. Generalized engineered alternative cover layers for HELP-3 are presented in Table D-3.

Table D-3
Engineered Alternative Cover Profile

	Layer Description (top to bottom)	Layer Type	Thickness (inch)	Soil Texture	Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity (cm/sec)
Vegetated Swale at Boring B-3	Vegetated layer	1	12	7	0.473	0.222	0.104	5.2x10^{-4}
	Geomembrane liner	4	0.03	35	N/A	N/A	N/A	1.0x10^{-9}
	Clayey Sand with Gravel	1	72	27	0.282	0.281	0.280	1.9x10^{-7}
	Sandy Clay	1	30	25	0.437	0.373	0.266	3.6x10^{-6}
Vegetated Swale at Boring B-5	Vegetated layer	1	12	7	0.473	0.222	0.104	5.2x10-^{4}
	Geomembrane liner	4	0.03	35	N/A	N/A	N/A	1.0x10^{-9}
	Silty Sand with Gravel	1	72	24	0.374	0.305	0.202	1.0x10^{-6}
Vegetated Swale at Boring B-6	Vegetated layer	1	12	7	0.473	0.222	0.104	5.2x10^{-4}
	Geomembrane liner	4	0.03	35	N/A	N/A	N/A	1.0x10^{-9}

	Layer Description (top to bottom)	Layer Type	Thickness (inch)	Soil Texture	Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity (cm/sec)
	Silty Sand with Gravel	1	21	24	0.374	0.305	0.202	$1.0\text{x}10^{-6}$
	Sand	1	6	22	0.419	0.307	0.180	$1.9\text{x}10^{-5}$
	Clayey Sand	1	24	24	0.304	0.303	0.202	$2.6\text{x}10^{-7}$
Asphalt Concrete Pavement at Boring B-2	Asphalt concrete	1	3	21	0.040	0.032	0.013	$1.0\text{x}10^{-6}$
	Aggregate base	1	6	21	0.170	0.032	0.013	$5.0\text{x}10^{-5}$
	Geomem-brane liner	4	0.03	35	N/A	N/A	N/A	$1.0\text{x}10^{-9}$
	Silty Sand with Gravel	1	18	22	0.374	0.307	0.180	$1.0\text{x}10^{-6}$
	Gravel with Sand	1	48	21	0.397	0.032	0.013	$3.0\text{x}10^{-1}$
	Clayey Sand with Gravel	1	30	24	0.343	0.305	0.202	$4.5\text{x}10^{-7}$
Asphalt Concrete Pavement at Boring B-10	Asphalt concrete	1	3	21	0.040	0.032	0.013	$1.0\text{x}10^{-6}$
	Aggregate base	1	6	21	0.170	0.032	0.013	$5.0\text{x}10^{-5}$
	Geomem-brane liner	4	0.03	35	N/A	N/A	N/A	$1.0\text{x}10^{-9}$
	Gravel with sand and silt	1	24	21	0.397	0.032	0.013	$3.0\text{x}10^{-1}$
	Silty clayey sand with gravel	1	36	27	0.400	0.366	0.288	$7.8\text{x}10^{-7}$
	Sandy clay	1	18	26	0.445	0.393	0.277	$1.9\text{x}10^{-6}$

D3.0 HELP INPUT PARAMETERS – WEATHER DATA

Weather data required by HELP-3 are evapotranspiration, precipitation, temperature, and solar radiation data. Weather data used in our analyses are discussed below.

D3.1 Evapotranspiration Data

Evapotranspiration data include the following:

- location (nearby city and state)
- latitude
- evaporative zone depth
- maximum leaf area index
- growing season start and end dates
- average wind speed, and
- quarterly relative humidity.

A summary of evapotranspiration data for our analyses is presented in Table D-4.

Table D-4

Evapotranspiration Data

	Existing Cover				Prescriptive Cover and Vegetated Swale	Asphalt Concrete Pavement
	B-1, B-7, B-10	B-15	B-2, B-4, B-5, B-6, B-8, B-9, B-11	B-3		
Evaporative Zone Depth	8 inches	10 inches	12 inches	24 inches	12 inches	2 inches
Maximum Leaf Area Index	0	0	0	0	2	0
Growing Season Start Date	day 78	day 78	day 78	day 78	day 78	day 78
Growing Season End Date	day 328	day 328	day 328	day 328	day 328	day 328
Average Annual Wind Speed	10.6 mph	10.6 mph	10.6 mph	10.6 mph	10.6 mph	10.6 mph
First Quarter relative humidity	75.9%	75.9%	75.9%	75.9%	75.9%	75.9%
Second Quarter relative humidity	71.4%	71.4%	71.4%	71.4%	71.4%	71.4%
Third Quarter relative humidity	73.3%	73.3%	73.3%	73.3%	73.3%	73.3%
Fourth Quarter relative humidity	74.3%	74.3%	74.3%	74.3%	74.3%	74.3%

D3.1.1 Evaporative Zone Depth

The evaporative zone depth is the maximum depth from which water may be removed from the soil due to evapotranspiration processes. The value specified influences the storage of water near the surface and therefore directly affects the computations for evapotranspiration and runoff. Where surface vegetation is present, the evaporative depth should at least equal the expected average depth of root penetration and capillary suction of roots. In general, the depth of capillary draw to the surface without vegetation or to the root zone may be only several inches in gravels; 4 to 8 inches in sand, 8 to 18 inches in silts, and 12 to 60 inches in clays.

An evaporative zone depth of 8 to 24 inches was selected for existing cover, 12 inches for prescriptive cover and vegetated swale engineered alternative cover, and 2 inches for asphalt concrete pavement engineered alternative cover.

D3.1.2 Maximum Leaf Area Index

The maximum leaf area index (LAI) is a dimensionless ratio of the leaf area of actively transpiring vegetation to the nominal surface area of the land on which the vegetation is growing. The maximum LAI for bare ground is zero; for a poor strand of grass, the LAI approaches 1.0; for a fair strand of grass, the LAI approaches 2.0; for a good strand of grass, the LAI approaches 3.5; and for an excellent strand of grass, the LAI approaches 5.0.

We specified a LAI of 0 for existing cover and asphalt concrete pavement engineered alternative cover to be representative of the bare ground condition. Based on the assumption that the vegetative layer can support a fair strand of grass, we specified a LAI of 2 for the prescriptive and vegetated swale engineered alternative covers.

D3.1.3 Growing Season, Average Wind Speed, Relative Humidity

HELP-3 has default evapotranspiration data for selected U.S. cities, including San Francisco, California. Default evapotranspiration data included start and end dates of growing season, average wind speed, and relative humidity. HELP-3 default evapotranspiration data were adjusted to match published climate data[18] for San Francisco, California, based on a 1961 to 1990 record period.

The start of the growing season is based on mean daily temperature and plant species. Typically, the start of the growing season for grasses is the Julian date (day of the year) when the normal mean daily temperature rises above 50 to 55 degrees Fahrenheit. The growing season ends when the normal mean daily temperature falls below 50 to 55 degrees Fahrenheit. HELP-3 default start and end of growing season dates are 78 and 328, respectively for San Francisco. Days 78 and 328 are within the months where the mean monthly temperature (see Section D3.3) rises above and falls below 53 degrees Fahrenheit, respectively.

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[18] Published climate data for San Francisco, California reference in this appendix is obtained from website http://www.wrcc.dri.edu/summary/lcd.html.

We specified average annual wind speed for the site to be 10.6 miles per hour (mph). For relative humidity, we specified 75.9, 71.4, 73.3, and 74.3 percent, for the first, second, third, and fourth quarter, respectively, which is an average annual relative humidity of 73.7 percent.

D3.2 Precipitation Data

Thirty years of daily precipitation data was generated for the site by using HELP-3 synthetic weather generator for San Francisco. Normal mean monthly precipitation and daily irrigation values used in our HELP-3 analyses are presented in Table D-5.

Table D-5

Precipitation Data

	Normal Mean Monthly Precipitation (inches)
January	4.35
February	3.17
March	3.06
April	1.37
May	0.19
June	0.11
July	0.03
August	0.05
September	0.20
October	1.22
November	2.86
December	3.09

D3.3 Temperature Data

Thirty years of daily temperature data was generated using HELP-3 synthetic weather generator. HELP-3 has synthetic temperature data for San Francisco, and therefore, synthetic temperature data of San Francisco was selected to generate daily temperature data. Normal mean monthly temperature generated by HELP-3 is adjusted to match temperature from published climate data, as shown in Table D-6.

Table D-6

Normal Mean Monthly Temperature

	Temperature from Published Climate Data (°F)
January	48.7
February	52.2
March	53.3
April	55.6
May	58.1
June	61.5
July	62.7
August	63.7
September	64.5
October	61.0
November	54.8
December	49.4

D3.4 Solar Radiation Data

Thirty years of solar radiation data was generated using HELP-3 synthetic weather generator. HELP-3 has synthetic solar radiation data for San Francisco, and therefore, synthetic solar radiation data of San Francisco was selected to generate daily solar radiation data for our analyses.

D4.0 HELP INPUT PARAMETERS – DESIGN DATA

The design data required by HELP-3 include layer data, soil characteristics, and site characteristics, as discussed in this section.

D4.1 Layer Data

Layer data required by HELP-3 include layer type and layer thickness. There are four layer types permitted in HELP-3. The four layer types are: 1) vertical percolation, 2) lateral drainage, 3) barrier soil liner, and 4) geomembrane liner. Flow in vertical percolation layers are by unsaturated vertical drainage downward due to gravity, and upward flux due to evapotranspiration. Lateral drainage layers are directly above liners that are designed to promote lateral drainage to a collection and removal system. Barrier soil liners are intended to restrict vertical drainage; these layers should have saturated hydraulic conductivities substantially lower than those of the other types of layers. Geomembrane liners are virtually impermeable synthetic membranes.

All soil and asphalt concrete layers for analyses were specified as vertical percolation layers. The geomembrane liners underlying the vegetated swale and asphalt concrete pavement engineered alternative covers were specified as geomembrane liner.

Layer types and thicknesses for each cover system analyzed are presented in Tables D-1 to D-3.

D4.2 Soil Characteristics

Soil characteristics may be assigned to each layer using the default option or user defined option.
Soil characteristics required include total porosity, field capacity, wilting point, and saturated hydraulic conductivity. Total porosity is the volume of soil water storage at saturation as a fraction of total volume of soil. Field capacity is the volume of soil water storage after a prolonged period of gravity drainage at saturation and a soil suction of 1/3 bar. The field capacity value is less than total porosity value for a soil type. The wilting point is the lowest volume of soil water storage that can be achieved by plant transpiration or air-drying, that is the moisture content where a plant will be permanently wilted at a soil suction of 15 bars. Saturated hydraulic conductivity is the rate at which water drains through a saturated soil under a unit pressure gradient.

HELP-3 has 42 default characteristics for 42 soil/material types. The default characteristics of soil types 1 through 15 are typical of surficial and disturbed agricultural soil, which may be less consolidated and more aerated than soil typically placed in landfills. Clays, silts, and other soil at the site would generally be compacted except within the vegetative layer, which might be tilled to promote vegetative growth. Soil texture types 22 through 29 are for compacted soils. Soil/material types 16, 17, 20, and 21 pertain to barrier soils, bentonite mats, drainage nets, and gravel, respectively. Types 18, 19, and 30 through 33 refer to a variety of municipal waste, fly ash, and slag materials. Soil/material types 34 through 42 relate to geosynthetic materials, such as drainage nets, and HDPE, PVC, and neoprene liners. The soil type and characteristics specified for our analyses are summarized in Tables D-1 to D-3.

For the analyses, we specified appropriate default soil/material types for each layer. The total porosity and saturated hydraulic conductivity of each soil/material type were adjusted, as appropriate, to match the laboratory test results or published laboratory test results. Saturated hydraulic conductivity and total porosity for asphalt concrete was selected based on asphalt concrete pavement permeability studies Westerman (1998) and Prowell (2001). Saturated hydraulic conductivity and total porosity for compacted aggregate base was selected based on hydraulic performance of base material study performed by Paara and Blanco (2002). For the engineered alternative covers, we modeled the geomembrane liners using a saturated hydraulic conductivity of 1×10^{-9} cm/sec.

We performed seven laboratory permeability tests on representative cover soil material collected from the site during our geotechnical investigation. The results of the permeability tests, including total porosity and saturated hydraulic conductivity are summarized in Table D-7. Default soil characteristics were adjusted to match total porosity and saturated hydraulic conductivities of representative soil samples, as appropriate.

TABLE D-7

Laboratory Permeability Test Results

Location	Description	Total Porosity (vol/vol)	Saturated Hydraulic Conductivity (cm/sec)
B-1 at 4.5 feet	Sand with Silt	0.333	8.0×10^{-4}
B-2 at 5.5 feet	Clayey Sand with Gravel	0.343	4.5×10^{-7}
B-3 at 2.0 feet	Clayey Sand with Gravel	0.282	1.9×10^{-7}
B-4 at 2.0 feet	Silty Sand with Gravel	0.464	2.8×10^{-7}
B-5 at 2.0 feet	Silty Sand with Gravel	0.284	1.8×10^{-6}
B-8 at 5.5 feet	Clayey Sand with Gravel	0.288	1.3×10^{-7}
B-11 at 3.5 feet	Clayey Sand with Gravel	0.363	1.2×10^{-4}

D4.3 Site Characteristics

Input parameters for site characteristics include acreage, ground surface slope, slope length, soil texture of the top layer, and the vegetative cover. We assumed the site encompasses 20 acres. For the existing cover, we assumed ground surface slope of 1.85 percent and slope length of 800 feet. For the evaluation of the prescriptive and engineered alternative covers, we assumed a ground surface slope of one percent and slope length of 600 feet. Furthermore, we assumed bare ground for existing cover and asphalt concrete pavement engineered alternative cover, and fair strand of grass for prescriptive cover and vegetated swale engineered alternative cover. Based on the input site characteristics, a runoff curve number was computed by HELP-3 (see Tables D-1 to D-3).

D5.0 DISCUSSION AND CONCLUSION

Results of our HELP-3 analyses indicate that average annual percolation through the existing cover is between 2.7 and 13.1 inches. For the prescriptive cover, average annual percolation through the cover will be approximately 8.4 inches for boring B-2 and B-10 locations. For the vegetated swale engineered alternative cover, average annual percolation through the cover will be approximately 1.1, 1.7, and 1.8 inches for B-3, B-5, and B-6, respectively. For the asphalt concrete pavement engineered alternative cover, the average annual percolation through the cover will be approximately 2.3 and 7.2 inches for boring B-2 and B-10 locations, respectively. The results of HELP-3 analyses are presented in Table D-8.

TABLE D-8

HELP-3 Results

Cover Design	Boring	Average Annual Precipitation (inches)	Average Annual Runoff (inches)	Average Annual Evaporation (inches)	Average Annual Percolation (inches)
Existing Cover	B-1	20.3	3.2	6.8	10.4
	B-2	20.3	6.6	11.0	2.7
	B-3	20.3	4.0	3.2	13.1
	B-4	20.3	6.6	11.0	2.7
	B-5	20.3	6.6	10.9	2.9
	B-6	20.3	6.6	10.9	2.9
	B-7	20.3	4.5	10.5	5.4
	B-8	20.3	6.6	11.0	2.7
	B-9	20.3	5.0	11.7	3.6
	B-10	20.3	0.0	7.2	13.1
	B-11	20.3	6.6	10.9	2.9
	B-15	20.3	4.6	10.9	4.8
Prescriptive	B-2	20.3	0.1	11.9	8.4
	B-10	20.3	0.1	11.9	8.4
Vegetated Swale	B-3	20.3	5.2	14.1	1.1
	B-5	20.3	4.6	14.0	1.7
	B-6	20.3	4.6	14.0	1.8
Asphalt Concrete Pavement	B-2	20.3	14.9	3.0	2.3
	B-10	20.3	10.3	2.9	7.2

The results of the HELP-3 analyses indicate that the average annual percolation through existing covers that consist of 18 inches of low hydraulic conductivity material (less than 1x10^{-6} cm/sec), such as B-2, B-4, B-5, B-6, B-8, and B-11 locations, is less than the average annual percolation through the prescriptive covers; and the average annual percolation through the engineered alternative covers is less that the average annual percolation through the existing cover and prescriptive cover.

DISTRIBUTION

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