GEOTECHNICAL INVESTIGATION

SOUTHEAST OUTFALL SYSTEM MODIFICATIONS
SAN FRANCISCO, CALIFORNIA

FOR

SAN FRANCISCO CLEAN WATER PROGRAM
CITY & COUNTY OF SAN FRANCISCO

Allstate Geotechnical Services
San Francisco • San Ramon • Oakland

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1. INTRODUCTION

1.1 GENERAL STATEMENT

This report presents the results of our geotechnical investigation for the proposed Southeast Outfall System Modifications in San Francisco, California. The project location is shown on Plate 1.1 - Site Location Map. The purpose of this investigation was to explore the subsurface soil conditions and to provide geotechnical findings, conclusions, and recommendations for design and construction of the proposed project.

1.2 PROJECT DESCRIPTION

The Southeast Outfall System discharges effluent from the City of San Francisco's Southeast Sewage Treatment Plant into San Francisco Bay about 700 feet beyond the east end of Pier 80 (Army Street Terminal). The proposed modifications to the system include the placement of a new 60-inch-diameter reinforced concrete sewer force main to replace the existing 54-inch-diameter effluent line which extends from the Third Street Bridge to the offshore diffuser located at the end of Pier 80. It is also planned to make modifications to the existing Southeast Booster Pump Station. The modifications include replacing the existing pipe in the manifold pit with a larger pipe and installing larger pumps to increase effluent outflow capacity.
The length of the proposed new pipeline alignment is approximately 3640 feet and is shown on Plate 1.2. Its invert will be about 4 1/2 to 10 feet below the ground surface, ranging from Elevation -6 feet to Elevation -12.5 feet. All elevations referenced in this report are based on San Francisco City Datum (SFCD).

The new pipeline will connect to the existing manhole structure near the northwest corner of the Third Street Bridge. The pipe will extend east between the Reynolds Aluminum property line and the Islais Creek channel shoreline. Near the southeast corner of the Reynolds Aluminum property, the pipe will turn northeast and extend to just south of a wire fence at the southern edge of a truck trailer parking lot for Pier 80. The pipe will then turn east and run parallel to the fenceline. Near the east edge of the truck trailer parking area, the pipeline will extend eastward parallel to the south edge of the Pier 80 terminal. The pipe is to be placed above a relieving platform located on the north side of the concrete deck. A portion of the alignment will be located below the building designated as Shed A. This building extends over the relieving platform for approximately 1000 feet. It is planned to demolish the eastern 543 feet of Shed A during proposed modifications to Pier 80. Near the east end of Pier 80, the pipe will turn north and continue over the relieving platform along the east edge of the terminal. The new pipe will connect to the existing 54-inch-diameter pipe which extends offshore to the diffuser.
1.3 WORK PERFORMED

The scope of our work was indicated in our proposal dated February 14, 1986. The work performed for this investigation included the following:

1. A review of the available geotechnical, geological, and seismological data of the project area.

2. A field exploration which included drilling 5 borings to depths ranging from 17-1/2 to 146 feet. The locations of the drill holes for this investigation and the locations of previous borings by ourselves and others are shown on Plate 1.2 - Drill Hole Location Map. Bulk samples, Standard Penetration Test samples, Modified California Ring samples, and Shelby Tube samples of the earth materials encountered were obtained for identification and laboratory testing purposes. The logs of drill holes are presented in Appendix A - Supporting Geotechnical Data.

3. Laboratory testing of samples obtained from drill holes to determine the engineering characteristics of the various earth materials encountered. Moisture content, dry density, Atterberg limits, grain size analyses, unconfined compression, torvane shear strength, and consolidation tests were performed. In addition, pH, sulfate content, chloride content, and resistivity tests were performed on soil samples to determine their corrosion potential. Test results and a description of the test procedures are presented in Appendix A.
4. Laboratory testing of soil samples obtained from Drill Hole DH-2 to determine whether the soils are contaminated. Gas chromatography/mass spectrometry for semi-volatiles and oil and grease tests were performed. The test results and recommendations for further study are presented in Appendix B - Soil Contamination Analysis.

5. Development of geotechnical recommendations for the proposed force main, including foundation schemes, pile foundation design parameters, pipe embedment, temporary shoring, earth pressure, backfill requirements, and uplift forces.

6. Recommendations for protection of the pipe against corrosion and galvanic actions.

7. Development of seismic design parameters and response spectra, and evaluation of soil liquefaction potential and seismically induced settlements.

8. Evaluation of pile lateral load capacity and deflection by soil-structure interaction analyses.

9. Review and recommendation of improvements of the City's proposed force main alignment along the wharf and review of the foundation conditions of the existing force main on the south side of the Islais Creek Channel to identify potential soil-related problems.
10. Consultations with the City staff, the Port of San Francisco engineering staff, and the Port's consultant, CH2M HILL.

11. Preparation of this report.

2. FINDINGS

2.1 SITE CONDITIONS

The Southeast Outfall System is located in the Islais Creek basin which was formerly a small bay adjoining San Francisco Bay. Much of this basin was reclaimed from the bay during the late 19th century and the first half of the 20th century by the placement of artificial fill. The original shoreline of Islais Creek basin is illustrated on Plate 2.1 - Pre-1860 Shoreline of San Francisco. Prior to development, the basin was primarily marshland and shallow tidal flats.

Much of the Pier 80 area was reclaimed from the bay in the early to mid-1960's by dredging a keyway trench along the pierhead line through the younger bay mud and into the underlying firm soil and then filling the trench with dredged sand. As the construction of the dike progressed, dredged sand was placed over the younger bay mud in the area enclosed by the dike. The existing 78-foot-wide concrete pier deck along the north, south, and east perimeter of the site was constructed on
prestressed concrete piles ranging up to about 92 feet in length driven into the sand dike. A concrete subgrade relieving platform with a 15-foot-high retaining wall was built at the inboard edge of the wharf.

The Islais Creek Basin area generally is experiencing settlements on the order of one-half inch per year. However, the central and eastern portions of Pier 80 have undergone settlements up to 7 feet since placement of the dredged fill. As a result of the large settlements, repairs were made at the east end of the onshore portion of the existing effluent pipeline last year.

The Southeast Outfall System, which was constructed in 1966 to 1967, begins at the Southeast Booster Pump Station located on the south side of the Islais Creek Channel near the southwest corner of the Third Street Bridge. The pump station is founded on steel pipe piles which take support in sandy soils at about elevation -70 feet. From the pump station, a 36-inch-diameter pipe and a 42-inch-diameter pipe extend across Islais Creek Channel. The pipes are embedded in sand fill and are supported on steel pipe piles over the entire crossing. The piles along the south end of the crossing extend to about elevation -70 feet. Across the center and north end of the crossing, the piles extend to about elevation -145 feet. The pipes are supported on pile-supported trestles on the north and south creek banks. The trestles have some battered piles to resist lateral loads.
From the manhole structure on the north bank of the channel, an existing 54-inch-diameter pipe extends eastward across Third Street. On the east side of Third Street, the pipeline turns northward and continues along Third Street to Marin Street and then eastward through Pier 80 to the sub-marine outfall in San Francisco Bay. The existing alignment is shown on Plate 1.2. The alignment is supported on fill from the manhole structure on the north side of the Islais Creek Channel to the east end of the Pier 80 sand dike. From the end of the sand dike to the end of the diffuser, the alignment is supported in younger bay mud.

The alignment of the proposed new pipeline was described earlier in the Project Description section. Stations were given along this alignment by the San Francisco Clean Water Program. The stations range from 1+00 at the manhole structure on the west side of Third Street to 37+42 at the intersection of the proposed pipeline and the existing pipeline at the east end of the Pier 80 terminal.

At station 1+00, the new pipeline will connect to the existing manhole structure. The pipeline will then cross Third Street to Station 1+86. From Station 1+86 to Station 4+15, the pipeline will pass through a small park area located between the Reynolds Aluminum property and the Islais Creek Channel. A wooden shed exists in the northwest corner of the park. A small retaining wall consisting of concrete panels and wooden piles exists at the south edge of the park. Panels in the vicinity of the Third Street Bridge have tilted and have slipped down the slope. The slope is higher near the Third Street Bridge and is covered with some large pieces of concrete.
From Station 4+15 to about Station 5+26, the new pipeline will cross one set of railroad tracks and will pass adjacent to an abandoned pier. Several pipelines extend from the pier to the shore and continue below the ground surface. It appears that these abandoned lines may cross the proposed pipeline alignment. During our investigation, oil and grease were encountered in the soil near these pipelines. Remedial work to remove the contaminated soil may be required during pipeline construction. Chemical laboratory analyses of these soils and recommendations for further testing and study are contained in Appendix B of this report.

From Station 5+26 to Station 9+27, the pipeline will pass through an undeveloped Islais Creek shoreline area south of a parking lot for container trailers. The ground surface is covered with numerous large pieces of concrete, boulders, and other debris along the shoreline area. The slope from the parking lot to the water line is relatively flat and some vegetation is growing between the concrete and boulders.

At Station 9+27, the pipeline will pass beneath an existing railroad trestle which is located near the Army Street terminal pier deck. On the east side of the trestle, the pipeline will pass through a retaining wall and will continue on top of an existing relieving platform located inboard of the concrete deck at the south edge of Pier 80. The concrete relieving platform is supported on creosoted wood piles about 55 feet in length driven into the sand dike. The relieving platform is about 3 to 3-1/2 feet thick, 25 feet wide, and the top of the platform is located approximately 11 feet below the ground surface according to design drawings.
From Station 11+70 to Station 16+27, the pipeline will pass below the south edge of the existing Shed A at Pier 80. The waterside edge of the shed is supported on short piers ending on the relieving platform. Although design drawings indicate the clearance between the bottom of the shed's floor and the top of the relieving platform is only about 5-3/4 feet, a field measurement on the west end of the shed by S.F. Clean Water Program personnel indicated a clearance of about 7 feet.

The pipeline will continue eastward on top of the relieving platform along the south deck at Pier 80 to Station 34+74 where it turns northward and continues on top of the relieving platform to Station 37+42. At Station 37+42, it will be connected to the existing pipeline which extends offshore to the diffuser.

2.2 GEOLOGY

The San Francisco Bay Area lies within the Coast Range physiographic province of California which is characterized by a series of north-northwesterly trending mountains, valleys, and faults. The geology of the San Francisco Bay Area was mapped by Schlocker (1970). The map is shown on Plate 2.2 - Regional Geology and Fault Map. The San Francisco Bay Area as it is known today came into existence in mid-to-late Pleistocene time. Rocks of the Franciscan Formation were deposited at the foot of the continental slope from late Jurassic to mid-Cretaceous time. The formations were folded and faulted by a series of earth movements into the northwest trending structural patterns of the
central Coast Ranges. During the end of the Pliocene and the Pleistocene epochs, the San Francisco Marin block was tilted toward the east, with the western edge forming the San Francisco and Marin hills, and the depressed eastern edge forming the uneven depression in which the bay now lies. In Pleistocene time, the trough was partially filled with sediments and became San Francisco Bay as it was flooded due to the rising in sea level during the interglacial stages.

San Francisco Bay and the alluvial and estuarine deposits in the Islais Creek area occupy a structurally controlled basin within the Coast Ranges province. Late Pleistocene and Holocene sediments (less than 1.0 million years old) were deposited in this basin as it subsided (Atwater, Hedel, and Helley, 1977). In the Islais Creek area, these sediments rest on the serpentinites and gabbros of the northwest trending Fort Point–Hunters Point shear zone and are locally overlain by artificial fill. Based on the available geologic data, the depth to bedrock along the proposed new pipeline alignment varies from about 200 to 250 feet (Bonilla, 1964). The depth to bedrock under the Southeast Booster Pump Station is about 255 feet.

2.3 EARTH MATERIALS

Earth materials encountered during this investigation consist of 8 to 24 feet of artificial fill (Qaf) underlain by younger bay mud (Qyb). Three drill holes, DH-1, DH-3, and DH-5 were extended below the younger bay mud into the underlying bay
side sands (Qbs). The thickness of younger bay mud at these drill holes was 80, 121, and 43 feet, respectively. Drill hole DH-5 was extended through the bay side sands layer into older bay mud (Qob). The thickness of the bay side sand layer in drill hole DH-5 was 20 feet.

Two idealized soil profiles included herein on Plates 2.3 and 2.4 have been drawn from drill hole logs for this investigation and from boring information from a previous investigation by ourselves. The profiles represent generalized soil sections interpreted from widely spaced borings. Soil deposits may vary in type, strength and many other important properties between points of observation and exploration.

2.3.1 Artificial Fill (Qaf). Fill materials encountered in the exploratory program consist of layers of sand (SP), clayey sand (SC), silty sand (SM), gravelly sand (SP), sandy gravel (GP), and silty clay (CH), with varying amounts of cobbles, brick, and wood. Also, the fill in drill hole DH-2 contains some oil and grease, possibly from leakage from nearby pipelines. Chemical tests performed on the contaminated soil and recommendations for further testing and study are included in Appendix B of this report.

The relative density of the fill materials ranges from very loose to medium dense, with most fill being very loose to loose. Standard Penetration blow counts in the fill range from 2 to 31 blows per foot. Moisture contents range from 5 to 52 percent.
2.3.2 **Younger Bay Mud (Qyb).** Younger bay mud is the youngest deposit in San Francisco Bay and it consists of soft unconsolidated sediments generally containing more than 90 percent clay and silt size particles. Deposition of the mud extends from approximately 9,000 years ago to the present (Atwater, et. al., 1977). Available information indicates that normally, the composition of the younger bay mud ranges from 30 to 60 percent clay, 30 to 65 percent silt, and 1 to 10 percent sand. Lenses and irregular segregations of 1/2 inch to several feet thickness of fine to coarse sand or mollusk shells are common in younger bay mud in some places. Younger bay mud is, generally, soft to firm and typically it exhibits a highly plastic, highly compressible behavior.

The younger bay mud encountered during the exploration program is primarily dark gray, soft to firm, high plasticity, silty clay (CH) with local sand lenses and occasional shell layers. In addition, a dark brown, stiff, peat (Pt) and peaty clay (OH) layer was encountered at the bottom of the younger bay mud deposit in drill holes DH-1 and DH-3. The thickness of the peat layer ranged from 4 to 23 feet in the 2 drill holes. Also, a 4-foot-thick dark brown, stiff peat layer was encountered at top of the younger bay mud deposit in drill hole DH-2.

Torvane and unconfined compression triaxial tests were used to determine the shear strength of the younger bay mud samples. The shear strength test values range from 260 pounds per square foot (psf) at the top of the younger bay mud deposit to 1740 psf in the peat layer in drill hole DH-3. The shear strength of the
peat is primarily attributable to the stiff, compressed, wood-like nature of the organics and the higher shear strength values are not representative of the overall shear strength of the younger bay mud.

The tested moisture content of the silty clay layer of the younger bay mud ranges from 53 to 90 percent. The moisture content of the peat layers ranges from 38 to 155 percent. The tested dry density of the younger bay mud for this investigation ranges from 49 to 65 pcf.

2.3.3 Bay Side Sand (Qbs). The bay side sands may be comprised of wind blown and alluvial sands deposited during a low sea level associated with the Wisconsinian glaciation. The bay side sands are normally bounded above by younger bay mud and below by older bay mud. Occasionally bay side sands may be underlain directly by bedrock. The bay side sands are typically medium- to fine-grained, dense to very dense sand. Induration is generally slight and attributable to the presence of silt and clay. The maximum thickness of bay side sands in the San Francisco Bay is approximately 50 feet.

During this investigation, bay side sand was encountered in drill holes DH-1, DH-3, and DH-5. The layer is typically a green-gray to gray, dense to very dense, medium to fine-grained clayey sand (SC). The tested dry density and moisture content of the bay side sand samples obtained ranged from 107 to 118 pcf and 11 to 28 percent, respectively.
2.3.4 **Older Bay Mud (Qob).** The texture of older bay mud is similar to that of younger bay mud. The older bay mud unit, however, has been overconsolidated by several thousands of pounds per square foot due to both repeated dessication and the weight of overlying soils which were subsequently eroded. Older bay mud is believed to be primarily an estuarine clay with subordinate alluvial sediments that were deposited during sea level fluctuations associated with the Sangamon Interglacial period.

Typically older bay mud comprises a light to dark gray-green, stiff to very stiff, moderately to highly plastic silty clay (CH). Layers and lenses of dark gray to green-gray, dense to very dense sand (SP-SM), gravelly sand (SW), clayey sand (SC), and sandy clay (CL) are common throughout the unit. The shear strength for the older bay mud layer in drill hole DH-5, as determined by torvane tests, was 1700 psf. The tested dry density and moisture content of the sample taken from this layer was 64pcf and 62 percent, respectively.

### 2.4 Subsurface Conditions

#### 2.4.1 Booster Pump Station

Drill hole DH-5 was drilled on the fill embankment near the northeast corner of the Southeast Booster Pump station. The drill hole was 10 feet north of the retaining wall along the north edge of the pump station and about 40 feet east of the existing force main. The thickness of fill in the drill hole was about 14 feet. The fill is generally
poorly compacted sand and clayey sand with gravel and brick fragments. The fill is underlain by about 43 feet of younger bay mud which is underlain by a 20-foot-thick layer of dense to very dense bay side sands. Based on our findings in this drill hole, the pipe piles supporting the booster pump station and the southern portion of the force main crossing Islais Creek are embedded about 10 feet into this bay side sand layer.

2.4.2 Proposed Pipeline. The pipe invert will be between 5 and 10 feet below the existing ground surface from Station 1+00 to the relieving platform inboard of the Pier 80 deck at about Station 10+75. An idealized subsurface soil profile which is presented on Plate 2.3 has been drawn between the above stations based on drill hole logs for this investigation and from boring information from previous investigations by ourselves and others. The profile represents generalized soil sections interpreted from widely spaced borings. Soil deposits may vary in type, strength and many other important properties between points of observation and exploration.

Based on our exploration, the pipeline will be founded mainly in artificial fill between Stations 1+00 and 10+75. However, between 6+00 and 8+75, the pipeline may be founded on or slightly below the boundary between younger bay mud and artificial fill. The fill thickness encountered during our investigation varies from about 24 feet in drill hole DH-1 to about 8 feet in drill hole DH-4. The fill encountered is generally poorly compacted heterogeneous granular fill with varying amounts of silt, clay, gravel, bricks, and wood debris.
The heterogeneous nature of the fill makes it impossible to predict the composition or material properties of the fill at any given point between Station 1+00 and the edge of the sand dike at Pier 80 at about Station 8+75. From Station 8+75 to 9+75, the artificial fill slopes down to the east at approximately 5:1 (horizontal:vertical). From Station 9+75 to 10+75 the fill slope steepens to about 1:1. The fill between Stations 8+75 and 10+75 is anticipated to consist of a dense to very dense, clayey sand or sandy clay for approximately the upper 10 feet. Below this upper fill layer, the fill is mainly a hydraulically-placed medium to fine-grained sand. The hydraulic fill is generally medium dense to dense with localized zones of loose and very dense sand.

The artificial fill from Station 1+00 to 8+75 is underlain by approximately 80 to 120 feet of younger bay mud. The top of the younger bay mud is mainly very soft to soft, silty clay. From Station 8+75 to 10+75 the thickness of the younger bay mud decreases from about 120 feet to approximately 45 feet at Station 10+75. It is anticipated that a dense bay side sand layer underlies the younger bay mud along the proposed alignment from Station 1+00 to 10+75. However, between Station 1+00 and about 2+00, the bay side sand layer is only about 5 feet thick. The bay side sand layer is underlain by older bay mud which is underlain by bedrock.

The pipeline will be founded on the relieving platform inboard of the Pier 80 deck from Station 10+75 to the intersection with existing pipeline at Station 37+42. As discussed under Section 2.1 - Site Conditions, the relieving
platform is founded on wood piles driven into the perimeter containment dike which exists around the north, south, east edges of Pier 80. The bottom of the sand dike is founded in stiff older bay mud or dense bay side sands. The bottom of the sand dike along the proposed pipeline alignment varies from about elevation -100 to elevation -147. The inboard and outboard slopes between the sand dike and the younger bay mud are 1:1. A typical section view of the sand dike along the proposed pipeline alignment is shown on Plate 2.3.

2.5 GROUNDWATER

Groundwater was observed during the drilling operations for this investigation at a depth of 5 to 6 feet below the ground surface, corresponding to Elevation -4-1/2 to Elevation -7-1/2 feet. Due to the close proximity of the pipeline alignment to the Islais Creek Channel and the granular nature of the fill materials, it is expected that the groundwater elevation will be approximately the same as the water level in the channel.

2.6 FAULTS AND SEISMICITY

As part of the Coastal Ranges geologic province, the San Francisco Bay Area is within a region of high seismic activity. Epicenters for the recorded earthquakes greater than Richter Magnitude 2.5 of the area are shown on Plate 2.6 - Earthquake Epicenter Map. The map also shows the major active faults in the region.
The San Andreas fault dominates the tectonism, geology, and physiography of the San Francisco Bay region. This fault, which extends more than 700 miles northwestward from the Gulf of California to Point Arena, represents the boundary between the North American and Pacific Ocean crustal plates. Other major active faults in the region include the Hayward fault, Calaveras fault, and the Seal Cove-San Gregorio fault. The Hayward fault trends northwestward along the base of the hills behind the East Bay cities from Fremont northwest to Richmond. North of San Pablo Bay, the Rogers Creek and Healdsburg faults continue along much the same trend. The Calaveras fault diverges northward from the San Andreas fault south of Hollister and continues northward along the eastern margin of the Santa Clara Valley and into the Diablo Range. The San Gregorio fault trends northward across the mouth of Monterey Bay and along the coast of San Mateo County. The Seal Cove and associated faults north of Half Moon Bay may represent a northward continuation of the San Gregorio fault.

All these faults trend northwesterly and display a similar right-lateral, primarily horizontal movement. The proximity of the site with respect to active faults is presented in Table 2.1 - Active Faults (Kiremidjian and Shah, 1978; Borcherdt, 1975). Numerous other smaller active faults are present throughout the region, but are farther from the site and not believed to be capable of causing significant earthquake shaking within the project area.
<table>
<thead>
<tr>
<th>Fault</th>
<th>Distance to Site (miles)</th>
<th>Fault Length (miles)</th>
<th>Maximum Richter Magnitude (assigned)</th>
<th>Maximum Richter Magnitude (recorded)</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Andreas</td>
<td>7</td>
<td>745</td>
<td>8.3</td>
<td>8.3</td>
</tr>
<tr>
<td>Hayward</td>
<td>11</td>
<td>45</td>
<td>7.7</td>
<td>6.7</td>
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<td>San Gregorio</td>
<td>16</td>
<td>84</td>
<td>7.5</td>
<td>6.1</td>
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<tr>
<td>Concord</td>
<td>23</td>
<td>12</td>
<td>6.3</td>
<td>5.4</td>
</tr>
<tr>
<td>Green Valley</td>
<td>28</td>
<td>24</td>
<td>6.6</td>
<td>5.0</td>
</tr>
</tbody>
</table>

There have been at least 17 recorded earthquakes since 1800 that have caused damage to structures in the San Francisco Bay Area (Tocher, 1957). The five largest are discussed below.

**June 10, 1836.** The earthquake was attributed to movement on the Hayward fault in the East Bay. Ground rupturing took place along the base of the Berkeley Hills from Mission San Jose to San Pablo. The Richter Magnitude of this earthquake is believed to be about 7.

**June 1838.** The earthquake originated on the San Andreas fault and resulted in surface rupturing from Santa Clara to San Francisco. The Richter Magnitude of this earthquake is believed to be about 7.
October 8, 1865. The earthquake was centered on the San Andreas fault in the Santa Cruz Mountains and resulted in severe damage to the entire Bay Area. The Richter Magnitude of this earthquake is also believed to be about 7.

October 21, 1868. The earthquake was centered on the Hayward fault. Surface faulting was about 20 miles. The maximum horizontal displacement was 3 feet. Damage to buildings occurred in San Francisco and San Jose, along with 30 fatalities. The Richter Magnitude of this earthquake is also believed to be about 7.

April 18, 1906. This earthquake was probably California's greatest shock. Rupture occurred along 270 miles of the San Andreas fault from San Juan Bautista to southern Humboldt County. The resultant maximum vertical displacement was 6 to 8 feet at Shelter Cove, and the maximum horizontal displacement was about 20 feet at the south end of Tomales Bay. Structures in the San Francisco Bay Area were severely damaged and over 700 fatalities occurred. It had a Richter Magnitude of 8.3.

The Southeast Outfall System lies within the Fort Point - Hunters Point shear zone. The Fort Point - Hunters Point shear zone was first described by Schlocker (1974) from outcrops of sheared serpentinite in Potrero Hill and Hunters Point. It forms an indistinct boundary between the graywacke, chert, greenstone and shale of the Central highlands belt and the massive graywacke and shale of the Nob Hill belt to the east. The shear zone comprises a 5,000- to 8,000-foot wide belt of serpentinite and cataclasite, with inclusions of sandstone, that extends from Fort Point southeastward through Potrero Hill and the Hunters
Point area. This shear zone, like the San Bruno Fault, Hillside Fault, and City College fault zone, does not display any evidence of recent movement or activity, and thus is not considered active.
3. CONCLUSION AND RECOMMENDATIONS

3.1 PROJECT FEASIBILITY

Based on the field exploration, laboratory testing, and geotechnical analyses performed, it is feasible to construct the proposed Southeast Outfall System modifications, provided the recommendations presented in this report are considered in the project design and construction.

3.2 SEISMIC DESIGN CONSIDERATIONS

3.2.1 Maximum Credible Earthquake. A maximum credible earthquake is the largest earthquake that a given fault appears capable of generating. The maximum credible earthquake that could affect the site would be one with Richter Magnitude 8.3 occurring along the San Andreas fault at approximately 7 miles from the project site. The estimated return period of a Richter Magnitude 8.3 earthquake on the San Andreas fault is approximately 230 years (Kiremidjian and Shah, 1978). The duration of strong shaking for this earthquake would be approximately 80 seconds. The maximum credible earthquake would have a predominant period of approximately 0.4 seconds at bedrock.
Correlations between distance from a causative fault and peak bedrock accelerations have been developed and are shown on Plate 3.1 - Earthquake Acceleration Correlations (Seed and Idriss, 1983). Based on these correlations, a maximum credible earthquake occurring on the San Andreas fault would generate a peak bedrock acceleration of 0.55 g.

3.2.2 Risk Analysis and Design Earthquake. In order to evaluate the statistical probability of a given earthquake occurring during the life of the project, an analysis was performed based on the procedures and data developed by Kiremidjian and Shah (1975).

The number of earthquake events for each of the major faults in the general vicinity of the project site are listed in Table 3.1. Because the time period of the data may not be long enough to include the greatest possible magnitude earthquakes that may have occurred in the past, the maximum Richter magnitude that is assigned is different and in most cases higher than the maximum observed value.
TABLE 3.1

Number Of Seismic Events For Each Source

<table>
<thead>
<tr>
<th>Fault Source</th>
<th>Number of Records</th>
<th>Maximum Richter Magnitude Assigned</th>
<th>Maximum Richter Magnitude Recorded</th>
<th>Focal Depth (Km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Gregorio-Palo Colorado</td>
<td>106</td>
<td>7.5</td>
<td>6.1</td>
<td>10</td>
</tr>
<tr>
<td>San Andreas North</td>
<td>280</td>
<td>8.3</td>
<td>8.3</td>
<td>10</td>
</tr>
<tr>
<td>Hayward-Calaveras</td>
<td>321</td>
<td>7.7</td>
<td>6.7</td>
<td>10</td>
</tr>
</tbody>
</table>

The return periods for earthquakes of various magnitudes on the above faults were calculated by a regresional analysis. The results are presented in Table 3.2 and on Plate 3.2.
TABLE 3.2

Return Periods of Earthquakes On The Major Faults
Near The Project Site

<table>
<thead>
<tr>
<th>Richter Magnitude</th>
<th>San Andreas Fault-North</th>
<th>Hayward-Calaveras Faults</th>
<th>San Gregorio Fault</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.3</td>
<td>230</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.0</td>
<td>163</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.7</td>
<td></td>
<td>800</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>91</td>
<td>570</td>
<td>400</td>
</tr>
<tr>
<td>7.0</td>
<td>50</td>
<td>242</td>
<td>190</td>
</tr>
<tr>
<td>6.5</td>
<td>28</td>
<td>103</td>
<td>92</td>
</tr>
<tr>
<td>6.0</td>
<td>16</td>
<td>44</td>
<td>44</td>
</tr>
<tr>
<td>5.5</td>
<td>9</td>
<td>19</td>
<td>21</td>
</tr>
<tr>
<td>5.0</td>
<td>5</td>
<td>8</td>
<td>10</td>
</tr>
</tbody>
</table>

The above data present the recurrence of specific magnitudes of earthquakes with respect to their causative faults. It is also necessary to determine the probability that a given number of specific magnitude earthquakes will occur anywhere on a particular fault during the life of the structure.
Seismic hazard evaluation of the project was based on procedures and data developed by Kiremidjian and Shah (1975). Using the return periods for various peak bedrock accelerations and a correlation between service life of structures, return period, and risk level, also by Kiremidjian and Shah (1975), the probability of occurrence for various peak bedrock accelerations was computed for structures with a service life of 50 years. The computed values are presented graphically on Plate 3.3 - Seismic Hazard Analysis.

However, the attenuation relationship for bedrock acceleration used in the Kiremidjian and Shah analysis is that developed by Esteva (1974). The Esteva attenuation relationship tends to have higher attenuation for small earthquakes but lower attenuation for large earthquakes, compared with other commonly used attenuation functions. Another seismic hazard analysis was performed utilizing the computer program EQRISK (McGuire, 1976) and using the attenuation relationship developed by Schnabel and Seed (1972). The probability of exceedence for a 50-year design life is also shown on Plate 3.3. In addition, probability of exceedence for the San Andreas fault as a single source was computed and is also presented on Plate 3.3.

As shown on Plate 3.3, the probability of exceedence curve determined by the computer program EQRISK shows higher bedrock acceleration than that determined by Kiremidjian and Shah for a probability of exceedence greater than 20 percent. The upper envelope of the curves by EQRISK and Kiremidjian and Shah was used for site response analyses.
For the type of the structure under consideration, it seems to be appropriate to select a peak bedrock acceleration with a 50 percent probability of exceedence during a 50-year structure life as its operating level design earthquake. Based on the above recommended probability of exceedence curves, the peak bedrock acceleration for this level of earthquake is 0.35 g.

The significant parameters for the maximum credible earthquake and recommended design earthquake are presented in Table 3.3.

### Table 3.3

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Richter Magnitude</th>
<th>Peak Bedrock Acceleration at The Site (g)</th>
<th>Predominant Period (Seconds)</th>
<th>Approximate Epicentral Distance (Miles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Credible Earthquake</td>
<td>8.3</td>
<td>0.55</td>
<td>0.40</td>
<td>7</td>
</tr>
<tr>
<td>Design Earthquake</td>
<td>6.5</td>
<td>0.35</td>
<td>0.28</td>
<td>7</td>
</tr>
</tbody>
</table>
3.2.3 **Site Response Analyses.** One-dimensional, free-field site response analyses were conducted for two generalized soil profiles. Profile 1 represents the average soil conditions beneath the existing pier. Profile 2 represents the soil conditions in the undeveloped area. The analyses were conducted using the computer program SHAKE (Schnabel et al., 1972) which determines the response of a horizontally layered, linear viscoelastic system to input earthquake rock motions by the method of wave propagation. Non-linear soil behavior is approximated by the use of strain-dependent, equivalent linear soil properties (Seed and Idriss, 1970). The response of the soil profiles was obtained for the maximum credible earthquake and design earthquake.

The maximum accelerations and equivalent cyclic shear stress ratios at various depths were calculated. The results of these analyses are presented on Plates 3.4 and 3.5. The results show that the maximum accelerations at the base of the pipe are as follows:

<table>
<thead>
<tr>
<th></th>
<th>Profile 1</th>
<th>Profile 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Credible Earthquake</td>
<td>.11g</td>
<td>.17g</td>
</tr>
<tr>
<td>Design Earthquake</td>
<td>.07g</td>
<td>.09g</td>
</tr>
</tbody>
</table>

Response spectra at the base of the pipe were developed for both earthquake levels and for damping ratios of 2, 5, and 10 percent for Profile 1. The results are shown on Plate 3.6.
The corresponding response spectra recommended by ATC (Applied Technology Council) for soft to medium stiff clays and sands normalized to the maximum accelerations listed above are shown on Plate 3.7.

3.2.4 Fault Rupture. No known faults are known to cross the alignment of the proposed effluent line. Therefore, the potential risk of damage due to fault rupture is minimal.

3.2.5 Liquefaction. Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils can be subject to a temporary loss of strength due to the build-up of excess pore water pressure, especially during cyclic loadings such as induced by earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical movements, if not confined. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded, fine-grained sands. Silty sands may also liquefy during strong ground shaking.

The exploratory borings show that the fill materials in the undeveloped area are generally very loose with Standard Penetration blow counts mostly below 10 blows per foot. Therefore, we believe that the artificial fill in this area has a moderate to high potential of liquefaction. Liquefaction of the artificial fill could result in significant ground movement, possibly damaging portions of the proposed pipeline.
3.2.6 **Lateral Spreading.** Lateral spreading of the artificial fill may occur as a result of the design earthquake. Ground movement of this type was the cause of nearly all major pipeline breaks during the 1906 San Francisco earthquake (Youd and Hoose, 1978). The shoreline slopes in the undeveloped area of the project site have a moderate to high potential for lateral spreading and provisions should be made to allow for repair of damaged facilities if a major earthquake should occur in the future.

3.2.7 **Seismically Induced Strain.** Underground structures may be damaged by seismic ground shaking or vibration, even though an actual fracture or slip of the ground surface does not occur. The axial and bending strains induced in a pipeline by ground shaking can be estimated from the following equations (Newmark, 1967):

\[
\text{Axial Strain: } \varepsilon_m = \frac{v_m}{c} + \frac{a_m}{c} \frac{1}{k}
\]

\[
\text{Bending Strain: } K = \frac{a_m}{c^2}
\]

Where \( \varepsilon_m \) = Axial strain of the pipe
\( v_m \) = Soil particle velocity
\( a_m \) = Soil particle acceleration
\( c \) = Wave propagation velocity in the direction of the pipe axis
\( k \) = Reciprocal of the radius of curvature
The wave velocities in the direction of the pipe axis can be evaluated using the relationship presented in Coats (1980). It is recommended that the compressive wave velocity \((V_p)\) be used for the axial wave propagation velocity and the shear wave velocity \((V_s)\) be used for the curvature wave propagation velocity. Peak ground surface particle velocities were evaluated based on the empirical relationship between peak acceleration and velocity developed by Newmark (1973), which suggests a velocity/acceleration ratio of 48 in/sec/g for alluvium.

For the design earthquake of Richter Magnitude 6.5, the following values may be used:

\[
V_m = 14 \text{ inches per second (1.2 feet per second) in soil}
\]

\[
a_m = 0.3g \text{ in soil}
\]

\[
V_p = 2,500 \text{ feet per second in soil}
\]

\[
V_s = 1,000 \text{ feet per second in soil}
\]

Using the above equations and the velocity and acceleration values, seismically induced pipe strain was computed and is presented in Table 3.4.
TABLE 3.4

Seismically Induced Pipe Strain During The Design Earthquake

<table>
<thead>
<tr>
<th>Pipe Location</th>
<th>Axial Strain (feet/foot)</th>
<th>Bending Strain (radians/foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>4.8 x 10^{-4}</td>
<td>1.0 x 10^{-5}</td>
</tr>
</tbody>
</table>

3.2.8 Seismic Earth Pressures. The increase in earth pressures that would be produced by the design earthquake has been estimated using the procedures suggested by Seed and Whitman (1970) for earth retaining walls. A rectangular pressure distribution of 6 pounds per cubic foot (pcf) times the height of the buried wall can be used for design.

3.3 BOOSTER PUMP STATION

Our visual observation during the field exploration indicates that the dual force main at the booster pump station at the south side of Islais Creek appears to be stable. However, slope stability evaluation of the structure is not within the scope of this study. We will investigate the stability further at your request.
The lateral capacity of the existing piles was evaluated based on the method outlined in the Navy's NAVFAC DM-7.2 publication dated May 1982. The computations were performed by first assuming level ground conditions. The resulting lateral loads were then multiplied by a factor of 0.3 to account for the reduction of passive resistance due to the 2:1 slope near the piles. The fill condition as determined by our recent boring is very loose. The coefficient of variation of subgrade reaction ("f" factor) for this condition was assessed to be about 5 tons/ft$^3$.

The results of the analysis show that, for a recommended maximum lateral deflection of 1/2 inch at the top of the pile, the maximum lateral capacity of the existing steel pipe piles is about 1.5 kips per pile. It should be noted that the batter piles which were incorporated in the existing foundation system also provide a significant lateral resistance.

3.4 PIPE DESIGN

3.4.1 General. A 60-inch diameter pipe will be installed by the cut-and-cover method in trenches about 10 feet deep. The pipe will be located in fill, younger bay mud, and on the relieving platform. External loads on the pipe will include loads due to the overlying earth materials, loads due to construction activities, traffic loads, loads due to nearby
structures and the overlying railroad, and earthquake induced loads. It is recommended that the pipe be designed to resist the imposed loads with a minimum factor of safety of 1.5 and/or an acceptable amount of deflection as recommended by the manufacturer.

3.4.2 Foundation Design. Based on our understanding of the soil conditions along the proposed pipeline alignment, we believe that it is feasible to support the pipe on either the existing fill or piles for the pipe section between the Third Street and the concrete deck of Pier 80. Under Pier 80, the pipe will be supported on the relieving platform. The two foundation schemes are discussed below.

Pipe Supported on Existing Fill - To support the pipe on the existing fill, a special pipe bedding must be provided and anticipated large settlements must be considered in the pipe design. Specific pipe bedding recommendations are presented in Section 3.5.

Available settlement data in the Islais Creek Basin area indicates that areal settlement is occurring at a rate on the order of one-half inch per year due to consolidation of the younger bay mud under the weight of the artificial fill. Based on this settlement data and the results of the consolidation test in drill hole DH-1, settlement of the pipe section in the undeveloped area east of Third Street is estimated to be about 2 feet in 50 years.
As indicated in the Subsurface Conditions section earlier in this report, the edge of the sand dike begins at about Station 8+75. From Station 8+75 to 10+75, the fill thickness beneath the pipeline alignment increases from approximately 8 to 130 feet. The younger bay mud thickness in this interval decreases from about 120 feet at Station 8+75 to 45 feet at Station 10+75. The relieving platform begins at about Station 10+75. It is estimated that the settlement of a pipe supported on the fill between Station 8+75 and 10+75 will vary from about 1 to 1-1/2 feet in 50 years.

The settlement for the pipe section inboard of Pier 80 will be the same as that of the relieving platform. Based on the settlement monitoring data at Pier 80, we anticipate that additional future settlements of the pipe inboard of Pier 80 will be about 1 foot in 50 years.

Because of the variation in the anticipated settlement for the various sections of the pipe, differential settlements of the pipeline for the different supporting conditions may be as much as 1 foot. In addition, because the fill materials beneath and around the pipeline are not homogeneous, both in terms of material type and density, local differential settlements of several inches may occur over a 50 year period. We recommend that flexible joints be incorporated to accommodate such differential settlements. Bedding material recommendations for the pipeline-on-existing-fill alternative are included later in this report.
Pipe Supported on Piles. The pipe section between Third Street and the Pier 80 concrete deck may be supported on piles taking support in the dense bay side sands. We recommend that prestressed precast concrete square piles with a minimum width of 12-inch and a minimum of 5 feet of embedment in the bay side sands be used. It may be preferrable to use 14-inch-square piles due to pile length and other structural reasons. For these conditions and provided downdrag loads on the piles are reduced as discussed in the following paragraph, an allowable downward capacity of 50 and 70 tons may be used for 12-inch-square and 14-inch-square piles, respectively. This allowable capacity may be increased by one-third for seismic and/or wind loading.

Downdrag or negative skin friction due to the consolidation of the younger bay mud under the existing fill should be minimized to avoid further reduction of the pile capacities. It is recommended that the entire pile length with the exception of the lower 5 feet be coated with bitumen to reduce downdrag loads. In addition, it is recommended that a slightly oversized hole be drilled in the fill to protect the bitumen coat during driving.

Our borings indicate that the depth of the bay side sand layer in this area varies from about 100 feet to 140 feet. Therefore, we anticipate that the pile lengths will be between 105 feet and 145 feet. However, in the vicinity of the existing manhole structure at Station 1+00 and under Third Street, the bay side sand layer is only a few feet thick and cannot provide adequate pile support. We recommend that piles in this area be driven to a similar tip elevation as the piles supporting the manhole structure. Based on previous borings and pile driving records by others, we anticipate pile tip elevations of about -145 feet.
As discussed previously, the edge of the sand dike begins at about Station 8+75. We recommend that the piles between Station 8+75 and Station 10+00 be extended into the bay side sand layer as discussed previously. We recommend predrilling through the entire fill depth between these stations. Between Station 10+00 and the relieving platform, the piles may be terminated in the sand dike at about elevation -50 feet. The piles terminating in the sand dike should not be coated with bitumen. Some predrilling may be needed to reach the recommended tip elevation.

Settlements of the piles terminating in the bay side sand layer are anticipated to be less than 1/2 inch. As discussed earlier in the report, it is expected that the sand dike will settle about 1 foot in the next 50 years. Therefore, differential settlements on the order of 1 foot should be expected in the vicinity of Station 10+00. We recommend that flexible pipe joints be used in this area.

Alternatively, shorter piles extending into the lower portion of the younger bay mud may be used to reduce the amount of differential settlement of the pipe. Recommendations will be provided if this alternative is to be considered.

Lateral load capacities were computed for 12-inch-square piles supporting the pipe between Station 1+00 and the relieving platform. The lateral load capacity of individual piles should be limited to 5.7 kips for 1/2 inch lateral deflection. Bending moments along the length of the pile caused by different horizontal forces applied at the top of the pile were calculated and the results are shown on Plate 3.8 - Bending Moments. The curves are only applicable for a hinged and condition.
3.4.3 Earth Loads

Pipes Supported on Piles. Pipes supported on piles should be designed to resist full overburden pressures. Overburden pressures may be calculated as the pressure exerted by a fluid weighing 125 pcf above ground water level and 60 pcf below ground water level. Downdrag loads can be taken as 200 pounds per square foot (psf) acting on vertical planes extending from the ground surface to the springline of the pipe.

Pipes Supported on Fill and Soil. Loads on the pipe due to the overlying soil will be dependent upon the depth of placement, the backfill type and placement, and the type of pipe. It is likely that the pipe will be placed in trenches with near vertical sides. No major fill is contemplated above the existing ground surface and the pipe will thus be in a "trench" condition.

The earth load on the pipe may be calculated using formulas developed by Marston (1930). For a rigid pipe in a "trench" condition, the formula is:

\[ W_c = C_d W B_d^2 \]

Where: \( W_c \) = Vertical load on the pipe in pounds per unit length

\( C_d \) = An empirical coefficient described by Marston (1930)
W = Unit weight of the trench backfill material in pcf

\[ B_d = \text{Width of the trench at the top of the pipe in feet} \]

When using this equation, the empirical coefficient, \( C_d \), can be obtained from Plate 3.9 - Earth Load Coefficient \( C_d \), and the unit weight of the trench backfill, \( W \), can be assumed as 130 pcf.

If the pipe is flexible and the bedding surrounding the pipe is placed as recommended in this report, then the bedding and earth materials surrounding the pipe will carry a portion of the earth load and the above equation can be modified to:

\[ W_c = C_d W B_c B_d \]

Where: \( B_c = \text{Outside diameter of the pipe in feet} \)

It is believed that most of the pipe will be placed in the "trench" condition. However, if the width of the trench is greater than twice the diameter of the pipe, then the pipe may be in an "embankment" condition and the above equations will not apply. The earth load should then be calculated on the basis of Marston's formula for "embankment" conditions (Marston, 1930).
3.4.4 **Surcharge Pressures.** For any surcharge applied on the pipe, the pressures on the pipe may be estimated using the pressure diagrams presented on Plate 3.10 for vertical surcharge pressures and Plate 3.11 for lateral surcharge pressures.

3.4.5 **Modulus of Soil Reaction.** Flexible and semi-rigid pipes are typically designed to limit deflections caused by the applied loads. These deflections can be estimated with the aid of equations developed by Spangler (1941). A modulus of soil reaction of 100 pounds per square inch (psi) can be assumed for pipe supported in artificial fill and younger bay mud.

3.4.6 **Thrust Resistance.** Where the proposed pipeline changes direction abruptly, resistance to thrust forces can be provided by mobilizing frictional resistance between the pipe and surrounding soil, by the use of a thrust block, or by a combination of the two.

The frictional resistance can be calculated utilizing coefficients of friction between the pipe and adjacent backfill of 0.30 for the steel pipe. Pipe segments may be connected by tension joints capable of transmitting the required thrust forces if more than one segment of pipe is needed.
Passive resistance at a thrust block may be used instead of, or in conjunction with, frictional resistance to resist pipe thrust. For thrust blocks that are used in artificial fill or younger bay mud, an equivalent fluid pressure of 190 pcf above the water table and 95 pcf below the water table should be used for design. These low values are dictated by the soft and yielding nature of the soil.

3.4.7 Pipeline Passing Below Shed A. As described in Section 2.1, a test pit excavated under the direction of the S.F. Clean Water Program showed a clearance of about 7 feet between the bottom of Transit Shed A's floor and the top of the relieving platform near the west end of the Shed. The clearance between the bottom of the Transit Shed A's floor and the top of the relieving platform near the west end of the Shed. The clearance at this location is adequate for the proposed 73-1/2 inch O.D. pipe. However, the original design drawings indicate that the clearance is only 5-3/4 feet. If it is found that there is inadequate clearance for the proposed pipe at other locations, the following alternatives may be considered:

1. Use dual pipes and reduce the pipe size.

2. Change the pipe shape to rectangular.

3. Modify the foundation of the existing shed. One approach is to drill short piers and have this part of the structure span over the pipe and be supported on the relieving platform.
For the eastern portion of the shed, CH2M-Hill and AGS will raise the new foundation in the modification design to provide sufficient clearance.

3.5 EARTHWORK

3.5.1 Excavation Characteristics. Excavation for the pipeline will encounter primarily artificial fill. The artificial fill is composed of a wide range of materials including gravelly sand, silty clay, silty sand, cobbles, glass, and brick fragments. However, we do not expect that excavation of these materials will present major difficulties.

3.5.2 Trench Width. Minimum trench widths should be provided in order to ensure uniform support and to minimize external loads on the pipe. The width of the backfill, as measured from the side of the pipe to the side of the trench, should be a minimum of 12 inches for rigid pipes. For flexible or semi-rigid pipes, the width of the backfill should be a minimum of 18 inches in artificial fill, and 24 inches in younger bay mud.

3.5.3 Trench Backfill. Trench backfill may consist of excavated on-site soils provided they are free of organics and debris, have been screened to remove particles larger than six inches in diameter, have a plastic limit less than 12 percent
and a liquid limit less than 35 percent, and have been properly
blended and dried to near optimum moisture content. Imported
fill may also be used provided that the material is approved by
a qualified geotechnical engineer prior to placement.

Trench backfill should be placed in layers not exceeding
eight inches in uncompacted thickness and should be compacted to
90 percent relative compaction as determined by standard test
method ASTM D1557. The upper three feet of trench backfill
should be compacted to 95 percent relative compaction in areas
where traffic or structural loads are anticipated. An
appropriate compaction method should be used to avoid damaging
the pipe.

3.5.4 Pipe Bedding. The proposed pipeline should be completely
surrounded with bedding material to provide uniform support and
protection. Pipe bedding should consist of crushed rock ranging
in size from 1/4 inch to 1-1/2 inches in diameter for the pipe
sections founded on gravelly miscellaneous fill and should
consist of medium to coarse-grained sand, pea gravel, or crush
of rock of less than 1/2 inch in size for the sections of pipe
founded on younger bay mud and fine to medium-grained sand fill.
Where the pipe is to be supported on the gravelly miscellaneous
fill, sand should not be used as bedding because it may be
subjected to migration into the surrounding porous fill, leaving
the pipe unsupported. The material should be of uniform
gradation, and should contain less than three percent by weight
passing the No. 200 sieve.
Pipe bedding should be placed beneath the pipe with a minimum thickness of 18 inches over artificial fill and younger bay mud. The material should be placed to cover the proposed pipe by a minimum of six inches.

All bedding material should be placed carefully to achieve uniform contact with the pipe and a minimum relative compaction of 90 percent as determined by standard test method ASTM D1557. Compaction by jetting or flooding should not be permitted.

3.6 CONSTRUCTION CONSIDERATIONS

3.6.1 Temporary Earth Pressures. The use of sloping sides in the pipeline excavation may not be practical at all locations. Excavation at these locations will require a shoring system for support.

Temporary, internally braced and shored excavations will be subjected to the generalized earth pressures depicted on Plate 3.12 - Lateral Pressures on Temporary Structures. Lateral pressures due to surcharge loading, including construction equipment, should also be considered in design of the temporary bracing system.
3.6.2 Groundwater Control. The lowest invert elevation of the pipe is at -12.5 feet in the undeveloped area, which will be lower than the mean lower low water level (MLLW). Therefore, groundwater will likely be present in the trench and dewatering will be required. Dewatering will be difficult in some areas due to the highly permeable nature of the artificial fill and the close proximity of the Islais Creek Channel.

To avoid the problems associated with dewatering in the undeveloped area, it is recommended that the excavation be isolated from the groundwater by an impervious barrier such as steel sheet piling. The barrier should extend into the underlying younger bay mud. Steel sheet piling will act as a ground water barrier only if it extends into an impermeable layer and maintains the interlock between sheets. Where the sheet pile interlock fails to retain the groundwater, well points and sump pumps may be used to alleviate local accumulation of ground water.

The invert elevation of the pipe under Pier 80 is -7.5 feet and we anticipate a minor amount of groundwater flowing into the excavations. Under this condition, groundwater control may be handled by pumps placed at the base of the excavation.

3.6.3 Excavation Base Stability. Stability of the excavation base will be dependent upon the success of the groundwater control system, the proximity of the soft younger bay mud to the excavation base, and the dimensions of the trench. When the excavation is in granular materials, it is recommended that the
groundwater level be maintained a minimum of two feet beneath the bottom of the excavation throughout construction in order to minimize the chance of base failure due to high seepage gradients.

Basal heave may occur where zones of very soft younger bay mud are encountered during construction. A qualified geotechnical engineer should be consulted at that time for recommendations on how to improve conditions prior to proceeding with the excavation.

3.6.4 Pile Driving. Pile driving criteria should be determined by an indicator pile program before casting of the production piles. It is recommended that piles be driven with followers prior to excavation to avoid surcharging the excavation. Also it may be desirable to predrill the piles through the heterogeneous artificial fill. Both the hammer and the piles should be supported on fixed leads.

3.6.5 Construction Loads. Significant loads may be imposed on the pipe during compaction of the trench backfill by heavy equipment. No heavy construction equipment should be allowed to operate within 2-1/2 feet of the top of the pipe.

3.6.6 Construction Induced Settlements. Caution should be taken to minimize settlements of the ground surface surrounding the trench excavation as a result of construction activities such as excavation, dewatering, and sheetpile driving.
It is recommended that the surrounding structures be monitored during construction to ensure that construction activities do not result in detrimental settlements. Should settlements be observed, a qualified geotechnical engineer should be consulted for recommendation of appropriate alternative construction techniques.

3.7 CORROSION

Analysis of soil samples taken during this study indicates that the artificial fill has a pH ranging from 7.2 to 10.0, a resistivity of $1.5 \times 10^3$ to $2.2 \times 10^4$ ohm centimeters, a sulfate content of 53 to 160 parts per million (ppm), and a chloride content of 34 to 2700 ppm. Clay zones within the artificial fill may exhibit lower resistivity values than the granular zones. This information indicates that the artificial fill may be moderately corrosive to the buried pipeline. Damage to the pipe can be prevented by application of a protective coating or by other protective methods.
4. QUALITY CONTROL

The conclusions and recommendations presented herein are based upon interpretations of subsurface conditions encountered at the boring locations. Should subsurface conditions encountered during construction differ from those encountered at the boring locations, a qualified geotechnical engineer should review the conclusions and recommendations presented herein for their applicability and completeness in light of the new information.

The placement of pipe bedding and trench backfill should be observed in the field by a qualified geotechnical engineer or his representative. Periodic density tests should be made to verify that construction is in conformance with the specifications. Also, pile driving should be observed and recorded by a qualified geotechnical engineer. The pile driving criteria should be developed following driving 4 to 5 indicator piles.

It is recommended that a qualified geotechnical engineer review the pipeline design, excavation support system, and ground water control scheme to determine conformance with the recommendations presented in this report.
5. CLOSURE

This report was prepared in accordance with generally accepted geotechnical engineering standards for the exclusive use of the San Francisco Clean Water Program's Southeast Outfall System Modifications Project. No warranty is made as to the accuracy or completeness of information obtained during previous investigations by others, and no other warranty is either expressed or implied.

Respectfully Submitted,
ALLSTATE GEOTECHNICAL SERVICES

Chris Y.K. Lin
Civil Engineer 26084

Robert T. Wong
Civil Engineer 20107
REFERENCES


Brown, R.D., Jr., 1970, "Faults that are Historically Active or that Show Evidence of Geologically Young Surface Displacement," San Francisco Bay Region: U.S. Geological Survey/Housing and Urban Development Basic Data Contribution No.7.


California Division of Mines and Geology, 1973, "Geologic Map of California, San Francisco Sheet."


Dames and Moore, 1982, "Geotechnical Investigation, Army Street Terminal (Pier 80), San Francisco, California," Report for San Francisco Port Commission.


LEGEND

DH-2 - LOCATION AND DESIGNATION OF DRILL HOLES BY AGS FOR THIS INVESTIGATION

DH-3 - LOCATION OF DRILL HOLES BY AGS FOR PREVIOUS INVESTIGATION

DH-4 - LOCATION OF DRILL HOLES BY OTHERS

SEE PLATE 2.3 FOR IDEALIZED SOIL PROFILE FROM STATION 1+00 TO STATION 11+00 ALONG PROPOSED PIPELINE ALIGNMENT.
Franciscan assemblage
KJf-sandstone and shale
chert, metamorphic rocks,
limestone, sheared rocks
melange)
KJv - volcanic rocks

Contact
Dashed where approximately
located.
Fault known to be active
Dotted where concealed
Fault not known to be active
Dotted where concealed Only selected
faults shown. Query indicates
uncertainty as to existence of fault
IDEALIZED SOIL PROFILE
SOUTHEAST OUTFALL SYSTEM MODIFICATIONS

JOB NO. SF51002  DATE: MAY 1986  PLATE - 2.3
PLATE 2.4 - IDEALIZED TRANSVERSE PROFILE - PIER 80 SOUTH DECK
LEGEND

QAF  Artificial fill

QYB  Younger bay mud, soft to firm, highly plastic silty clay

QBS  Bay side sands, poorly graded sand, silt, and clay mixture

PT   Peat and peaty clay, stiff to very stiff, highly plastic

QOB  Older bay mud, stiff to very stiff, moderately to highly plastic sandy and silty clays and clayey sands

Geologic contact

Proposed pipeline

DH-2  Location of exploratory drill hole by AGS for this investigation

TB-9  Location of exploratory boring by others for previous investigations

PLATE 2.5 - LEGEND FOR SOIL PROFILES
### Location of Epicenter vs. Richter Magnitude

<table>
<thead>
<tr>
<th>Location of Epicenter</th>
<th>Corresponding Richter Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.5-3.4</td>
</tr>
<tr>
<td></td>
<td>3.5-4.4</td>
</tr>
<tr>
<td></td>
<td>4.5-5.4</td>
</tr>
<tr>
<td></td>
<td>5.5-6.4</td>
</tr>
<tr>
<td></td>
<td>6.5-7.4</td>
</tr>
<tr>
<td></td>
<td>7.5-8.4</td>
</tr>
</tbody>
</table>

Data obtained from the University of California at Berkeley.

---

**Explanation**

- **1.** Mud
- **2.** Unconsolidated sediments
- **3.** Moderately consolidated to well-consolidated sedimentary rocks
- **4.** Tertiary volcanic rocks
- **5.** Jurassic and Cretaceous
- **6.** Franciscan assemblage
- **7.** Serpentinite and peridotite
- **8.** Granite

**Contact**

- Dashed where approximately located
- Dotted where concealed
- Fault known to be active
- Fault not known to be active
- Faults shown where concealed
- Only selected faults shown where uncertainty as to existence of fault

**Scale in Miles**

---

**Plate 28 - Earthquake Epicenter Map**
AVERAGE VALUES OF MAXIMUM ACCELERATIONS IN ROCK

APPROXIMATE RELATIONSHIPS BETWEEN MAXIMUM ACCELERATIONS ON ROCK & OTHER LOCAL SITE CONDITIONS

PLATE 3.1 - EARTHQUAKE ACCELERATION CORRELATIONS
PLATE 3.2 - EARTHQUAKE RECURRENCE CURVES
PLATE 3.4 - SITE RESPONSE FOR PROFILE 1
PLATE 3.6 - RESPONSE SPECTRA AT THE BASE OF PIPELINE - PROFILE 1
DAMPING = 5%

PROFILE 1

MAXIMUM CREDIBLE EARTHQUAKE

DESIGN EARTHQUAKE

PROFILE 2

MAXIMUM CREDIBLE EARTHQUAKE

DESIGN EARTHQUAKE

PLATE 3.7 - ATC RESPONSE SPECTRA
USE CURVE C FOR DESIGN OF PIPELINES FOR THIS PROJECT

PLATE 3.9 - EARTH LOAD COEFFICIENT - $C_d$
Note:
\[ m = \frac{B}{z}, \quad n = \frac{L}{z} \]
\[ m \text{ and } n \text{ are interchangeable} \]
\[ \sigma_z = q \eta \]

Poisson's ratio = 0

PLATE 3.10 - VERTICAL SURCHARGE PRESSURES
PLATE 3.11 - LATERAL SURCHARGE Pressures

L (LINE LOAD) OR Q (CONCENTRATED LOAD)

VALUES OF PH/L

VALUES OF PH^2/Q

VALUES OF N

M = 0.7
M = 0.6
M = 0.4
M = 0.4
M = 0.7
M = 0.5
NOTES

1) $H_1, H_2, D_0$ are to be in feet. Pressures are in pounds per square foot.

2) $\gamma_0 = \frac{\gamma_m H_1 + \gamma_b H_2}{H_1 + H_2}$, $\gamma_m = 120$ pcf (unit weight of fill above water table) $\gamma_b = 63$ pcf (unit weight of fill below water table) $q_U = 600$ psf (unconfined compression strength of Qyb)

3) Safety factor to be included by designer.
Exploration for this investigation was performed from March 31, 1986 through April 3, 1986, and consisted of drilling of 5 drill holes, DH-1 through DH-5. The drill holes ranged in depth from 17-1/2 feet to 146 feet with a total aggregate depth of 398-1/2 feet. The locations of the drill holes are shown on Plate 1.2 - Drill Hole Location Map. A Failing 750 rotary wash rig, provided by Pitcher Drilling Company of Palo Alto, California, was used. Both casing and driller's mud were used in advancing the drill holes.

The soils encountered were logged continuously in the field by an engineer during drilling operations. The soils were logged using the Unified Soil Classification System. Logs of borings presented on Plates A-1.1 through A-1.14 give descriptions of the earth materials encountered, show a graphic representation of the soil profile found, give the depth at which soil samples were obtained, indicate water levels, if encountered, and field and laboratory tests performed. A legend to the logs is presented on Plate A-2. Drill hole locations at the Southeast Outfall System Modifications site were determined by tape measurements from known points along the alignment. The location of the drill holes should be considered accurate only to the degree implied by the method used. Ground surface elevations at the drill hole locations were determined using a profile drawing of the ground surface along the alignment supplied by the San Francisco Clean Water Program.
SAMPLING

Three different types of samplers were utilized for soil sampling in this investigation which are tabulated below:

<table>
<thead>
<tr>
<th>Sampler</th>
<th>Method of Advance</th>
<th>Sample Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Split Spoon (2&quot; O.D.)</td>
<td>Driven by 140-pound hammer falling</td>
<td>1-3/8-inch diameter x 18 inches</td>
</tr>
<tr>
<td>or Standard Penetration Test</td>
<td>30 inches</td>
<td></td>
</tr>
<tr>
<td>Sampler</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Split Spoon Sample Barrel (3&quot; O.D.) or</td>
<td>Driven by 400-pound hammer falling</td>
<td>2.4-inch diameter x 18 inches</td>
</tr>
<tr>
<td>Modified California Sampler</td>
<td>18 inches</td>
<td></td>
</tr>
<tr>
<td>Thin-Wall Shelby Tube Sampler</td>
<td>Pushed into the soil. 2.8-inch diameter</td>
<td>Sampler head contains x 30 inches</td>
</tr>
<tr>
<td></td>
<td>ball check to prevent water pressure from forcing the sample out of tube during withdrawal.</td>
<td></td>
</tr>
</tbody>
</table>
The samples were obtained generally by using the Standard Penetration Test Sampler and the Modified California Sampler on the fill, the bay side sands, and the older bay mud. The Shelby Tube Sampler was used in the younger bay mud.

LABORATORY TESTING

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

MOISTURE CONTENT AND DENSITY TESTS

Moisture content tests were performed on 38 samples. The moisture content was determined in accordance with ASTM D2216, Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Density determinations were performed on 13 undisturbed samples. The results of the moisture and density tests are presented on the drill hole logs.

CONSOLIDATION TEST

A one-dimensional consolidation test was performed on 1 selected undisturbed sample in accordance with Standard Test Method ASTM D2435-Standard Test Method for One Dimensional Consolidation Properties of Soils.
The test sample was 1.0 inches in height and 2.875 inches in diameter. Loads were applied to the samples by the use of air pressure regulators feeding into the consolidometers. Accuracy was maintained throughout the loading range by the use of sensitive oil and mercury manometers for the lower loads and psi gauges for the higher loads. Sample deformation was measured to 0.0001 inch. The sample was allowed to consolidate for approximately 24 hours under each load increment. Time readings were taken at selected loads. Rebounding was done at twice the rate of loading and the final specimen data was calculated at the last rebound increment. Results of the consolidation test, in the form of consolidation versus log of pressure, is presented on Plate A-3. Time readings, in the form of dial readings versus log of time, are presented on Plate A-4.

UNCONFINED COMPRESSION TESTS

Unconfined compression tests were performed on 2 undisturbed samples of younger bay mud. The tests were run in accordance with Standard Test Method ASTM D2166-Standard Test Method for Unconfined Compression Strength of Cohesive Soil.

Prior to testing, the specimen's dimensions were measured and recorded. The tests were performed at field moisture conditions. During the tests, the laterally unsupported cylindrical specimens were subjected to a gradually increased axial compression load at a rate of strain of 2 percent per minute until failure occurred. Displacements and their corresponding loads were measured and recorded throughout the tests. Results of the unconfined compression tests are presented on Plates A-5.1 and A-5.2.
SIEVE ANALYSIS TESTS

Five wash sieve analysis tests were performed on the obtained samples to assist in the classification of subsurface soils and determination of the liquefaction potential. The tests were performed in accordance with ASTM D422. The results of the wash tests are shown on the drill hole logs.

TORVANE SHEAR STRENGTH TESTS

Torvane shear strength tests were performed on 18 undisturbed cohesive soil samples. The test results are presented on the drill hole logs.

CHEMICAL TESTS

In order to evaluate the potential corrosiveness of the artificial fill, two samples were submitted for chemical tests. The results of these tests are presented in Table A-1 - Chemical Tests.

TABLE A-1
CHEMICAL TESTS

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (feet)</th>
<th>Soil Type</th>
<th>pH at 25°C</th>
<th>Resistivity (ohm-cm)</th>
<th>Sulphate SO₄ (mg/kg)</th>
<th>Chloride Cl (mg/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>SP</td>
<td>7.2</td>
<td>1500</td>
<td>160</td>
<td>34</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>SM</td>
<td>10.1</td>
<td>22000</td>
<td>53</td>
<td>2700</td>
</tr>
</tbody>
</table>
# LOG OF DRILL HOLE

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**DRILL HOLE NO.:** DH-1  
**CHECKED BY:** CL  
**DRILLING METHOD:** Rotary Wash  
**DRILLING DATE:** 3/31/86  
**REFERENCE EL.:** 1 ½ ft. (Approx  
**DATUM:** San Francisco City Datum (SFCD)

## GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

<table>
<thead>
<tr>
<th>ELEVATION (FEET)</th>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNTS/FT.</th>
<th>GRAPHIC LOG</th>
<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>PLASTIC LIMIT (%)</th>
<th>ADDITIONAL TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td></td>
<td></td>
<td>&quot; ARTIFICIAL FILL (Qaf) &quot; SAND (SP), gray-brown, fine-grained PEAT (Pt) lens at 2 ½ to 3 feet</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-3½</td>
<td>2</td>
<td>5</td>
<td></td>
<td>&quot; ARTIFICIAL FILL (Qaf) &quot; GRAVELLY SAND (SP), with silt, glass, and brick fragments, dark gray to gray-brown, very loose to loose, medium-grained 3/31 Interbedded with SILTY CLAY (CH) at 5 ft. /86</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-13½</td>
<td>3</td>
<td>3</td>
<td></td>
<td>&quot; ARTIFICIAL FILL (Qaf) &quot; GRAVEL AND COBBLES mixed with SILTY CLAY (CH), with glass and bricks, soft, high plasticity</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-23½</td>
<td>4</td>
<td>4</td>
<td></td>
<td>&quot; YOUNGER BAY MUD (Qyb) &quot; SILTY CLAY (CH), gray, very soft to soft, high plasticity</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-33½</td>
<td>5</td>
<td></td>
<td></td>
<td>[ TORVANE = 400 psf ]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**TORVANE = 400 psf**
<table>
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<tr>
<th>ELEVATION (FEET)</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNTS/FT.</th>
<th>GRAPHIC LOG</th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>PLASTIC LIMIT (%)</th>
<th>ADDITIONAL TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>38 1/2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>48 1/2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>58 1/2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td>65</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>59</td>
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<td>CN</td>
</tr>
<tr>
<td>68</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

"YOUNGER BAY MUD (Qyb)"
SILTY CLAY (CH), gray, soft to firm, high plasticity

TORVANE = 320 psf
<table>
<thead>
<tr>
<th>ELEVATION (FEET)</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNTS/FT.</th>
<th>GRAPHIC LOG</th>
<th>GEOTECHNICAL DESCRIPTION AND CLASSIFICATION</th>
<th>DRY DENSITY (PSF)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>ADDITIONAL TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td></td>
<td></td>
<td></td>
<td>&quot;YOUNGER BAY MUD (Qyb)&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SILTY CLAY (CH), gray, firm,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>high plasticity</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>73 ½</td>
<td>7</td>
<td></td>
<td></td>
<td>TORVANE = 560 psf</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>7</td>
<td></td>
<td></td>
<td>&quot;BAY SIDE SAND (Qbs)&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>83 ½</td>
<td></td>
<td></td>
<td></td>
<td>&quot;YOUNGER BAY MUD (Qyb)&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td>SANDY PEAT (Pt), dark brown,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>stiff, high plasticity</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>93 ½</td>
<td></td>
<td></td>
<td></td>
<td>&quot;BAY SIDE SAND (Qbs)&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>103 ½</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Geotechnical Description and Classification**

<table>
<thead>
<tr>
<th>Elevation (Feet)</th>
<th>Sample No.</th>
<th>Blown Counts/ft.</th>
<th>Graphic Log</th>
<th>Dry Density (pcf)</th>
<th>Moisture Content (%)</th>
<th>Plastic Limit (%)</th>
<th>Additional Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>105</td>
<td>8</td>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&quot;Bay Side Sands (Qbs)&quot;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Clayey Sand (SC), gray-green, dense, fine to medium-grained</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sandy Clay (CH), green-gray, very stiff, high plasticity at 104½ to 106½ feet</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Torvane = 2200 psf on Silty Clay (CH) at 105 feet</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>108½</td>
<td>9</td>
<td>78</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Less clay content and color change to gray-brown at 120½ feet (5.1% Passing No. 200 Sieve)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>115</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bottom of boring at 121 feet below the ground surface at approximately elevation -119½ feet (SFCD)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>118½</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>125</td>
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</tr>
<tr>
<td>128½</td>
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</tr>
</tbody>
</table>

**Reference EL.:** 1½ ft. (Approx: DATUM: SFCD)
**LOG OF DRILL HOLE**

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**DRILL HOLE NO.:** DH-2  
**CHECKED BY:** CL  
**REFERENCE EL.:** -3 ft. (Approx)  
**DRILLING METHOD:** Rotary Wash  
**DRILLING DATE:** 4/1/86  
**DATUM:** SFCD  

### GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

<table>
<thead>
<tr>
<th>ELEVATION (FEET)</th>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>GRAPHIC LOG</th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>ADDITIONAL TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>3</td>
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<td>-20</td>
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<tr>
<td>-28</td>
<td>5</td>
<td>5</td>
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<td>Brick and wood in cuttings</td>
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</table>

**" ARTIFICIAL FILL (Qaf) "**  
CLAYEY SAND (SC), with gravel, brown,  
fine to medium-grained  
Color change to gray and oil odor at 3 ft.

**" ARTIFICIAL FILL (Qaf) "**  
SANDY GRAVEL (GP), with clay and  
oil odor, loose  
SILTY CLAY (CH), gray, firm at 5½-6 ft.

**" ARTIFICIAL FILL (Qaf) "**  
SILTY SAND (SM), with gravel  
and wood, black, loose, fine-grained  
(25.4% Passing No. 200 Sieve)

**" YOUNGER BAY MUD (Qyb) "**  
PEAT (Pt), dark brown, stiff,  
high plasticity with SILTY SAND (SM) lenses

**" YOUNGER BAY MUD (Qyb) "**  
SILTY CLAY (CH), with shells, gray, soft,  
high plasticity TORVANE = 470 psf

Bottom of boring at 27½ feet below the  
ground surface at approximately  
elevation -30½ feet (SFCD)
# LOG OF DRILL HOLE

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**CHECKED BY:** CL  
**REFERENCE EL.:** -2.5 ft. (Approx)  
**DRILLING DATE:** 4/1-2/86  
**DATUM:** SFCD

## GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

<table>
<thead>
<tr>
<th>ELEVATION (FEET)</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNTS/FT.</th>
<th>GRAPHIC LOG</th>
<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>ADDITIONAL TESTS</th>
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</table>

- "ARTIFICIAL FILL (Qaf)"
  - "SAND (SP), brown, medium dense," fine to medium-grained, 4/1/86 Saturated below 5 ft.
  - "SAND (SP), brown, medium dense," fine to medium-grained, 4/1/86 Saturated below 5 ft.
  - "ARTIFICIAL FILL (Qaf)"
  - "CLAYEY SAND (SC), with gravel, fine to coarse grained" 5 inches asphaltic concrete at surface
  - "YOUNGER BAY MUD (Qyb)"
  - "SILTY CLAY (CH), with shells, gray, soft, high plasticity"  

**TORVANE = 330 psf**
# LOG OF DRILL HOLE

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**DRILL HOLE NO.:** DH-3  
**CHECKED BY:** CL  
**DRILLING METHOD:** Rotary Wash  
**DRILLING DATE:** 4/1-2/86  
**REFERENCE EL.:** -2½ ft. (Approx)  
**DATUM:** SFC

## GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

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<th>ELEVATION (FEET)</th>
<th>SAMPLE NO.</th>
<th>GRAPHIC LOG</th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
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"YOUNGER BAY MUD (Qyb)"

Silty Clay (CH), gray, soft, high plasticity

TORVANE = 430 psf

Grading to very soft at 55 ft.

TORVANE = 200 psf

70  TV
**LOG OF DRILL HOLE**

*PROJECT*: Southeast Outfall  
*LOGGED BY*: CS  
*DRILL HOLE NO.:* DH-3  
*CHECKED BY*: CL  
*REFERENCE EL.:* -2½ ft. (Appro)  
*DRILLING METHOD*: Rotary Wash  
*DRILLING DATE*: 4/1-2/86  
*DATUM*: SFCD

### GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

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<th>LIQUID LIMIT (%)</th>
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- "YOUNGER BAY MUD (Qyb)"
- SILTY CLAY (CH), gray, firm, high plasticity

- TORVANE = 520 psf

- TORVANE = 700 psf

- 69
- TV

- 58
- TV

*SHEET 3 OF 5*  
*PLATE A-1.8*
### LOG OF DRILL HOLE

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**DRILL HOLE NO.:** DH-3  
**CHECKED BY:** CL  
**REFERENCE EL.:** -2½ ft. (Approx)  
**DRILLING METHOD:** Rotary Wash  
**DRILLING DATE:** 4/1-2/86  
**DATUM:** SFCD

#### GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

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<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
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"YOUNGER BAY MUD (Qyb)"

SILTY CLAY (CH), gray, firm, high plasticity

TORVANE = 1340 psf

"YOUNGER BAY MUD (Qyb)"

PEATY CLAY (OH), dark brown to black, stiff, high plasticity

TORVANE = 1740 psf

Interbedded PEATY CLAY (OH) and CLAYEY SAND (SC) from 120 feet to 136 feet

Increased sand content at 131 to 135 feet

PEATY CLAY (OH) with sand lens at 135 to 136 feet

"BAY SIDE SANDS (Qbs)"

SILTY SAND (SM), green-brown, dense, fine-grained

**SHEET 4 OF 5**
<table>
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**GEOTECHNICAL DESCRIPTION AND CLASSIFICATION**

"BAY SIDE SANDS (Qbs)"
GRAVELLY SAND (SP), with clay, green-gray, very dense, interbedded with CLAYEY SAND (SC), dark brown, very dense, fine-grained

Bottom of boring at 146 feet below the ground surface at approximately elevation -148.5 feet (SFCD)

**ADDITIONAL TESTS**

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<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (%)</th>
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# LOG OF DRILL HOLE

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**DRILL HOLE NO.:** DH-4  
**CHECKED BY:** CL  
**REFERENCE EL.:** -2 ft. (Approx)  
**DRILLING METHOD:** Rotary Wash  
**DRILLING DATE:** 4/2/86  
**DATUM:** SFC

## GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

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<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNTS/FT.</th>
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<tr>
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<td></td>
<td>CLAYEY SAND (SC), with gravel, fine to coarse-grained</td>
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<td></td>
<td>5 inches asphaltic concrete at surface</td>
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<td>&quot; ARTIFICIAL FILL (Qaf) &quot;</td>
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<td>SAND (SP), brown, dense, fine to medium-grained</td>
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<td>&quot; YOUNGER BAY MUD (Qyb) &quot;</td>
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<td>CLAYEY SILT (OH), with organics and sand</td>
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<td>SILTY CLAY (CH), interbedded with organic CLAYEY SILT (OH), soft, high plasticity</td>
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<td></td>
<td>TORVANE = 300 psf</td>
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<td>Extensive shells at 17 feet</td>
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<td>Bottom of boring at 17½ feet below the ground surface at approximately elevation -19½ feet (SFCD)</td>
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**DENSITY (PCF):**  
**MOISTURE CONTENT (%):**  
**LIQUID LIMIT (%):**  
**PLASTIC LIMIT (%):**  
**ADDITIONAL TESTS:** TV, UC

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**SHEET 1 OF 1**  
**PLATE A-1.11**
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<th>ELEVATION (FEET)</th>
<th>SAMPLE NO.</th>
<th>BLOW COUNTS/FT.</th>
<th>GRAPHIC LOG</th>
<th>GEOTECHNICAL DESCRIPTION AND CLASSIFICATION</th>
<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>ADDITIONAL TESTS</th>
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<td>&quot; ARTIFICIAL FILL (Qaf) &quot; SAND (SP), brown, fine-grained, loose Chunk of clay at 3 feet</td>
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<td>5</td>
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<td>&quot; ARTIFICIAL FILL (Qaf) &quot; CLAYEY SAND (SC), with gravel, shells and brick fragments, gray-brown and brown, very loose to loose, fine to medium-grained Saturated below 8 feet (24.1% Passing No. 200 Sieve) Extensive gravel and color change to gray-brown and black at 10 feet</td>
<td>16</td>
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<td>GS</td>
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<td>&quot; YOUNGER BAY MUD (Qyb) &quot; SILTY CLAY (CH), with shells and some organics, dark gray, soft to firm, high plasticity TORVANE = 500 psf</td>
<td>63</td>
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<td>Organic odor at 25 feet TORVANE = 430 psf</td>
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LOG OF DRILL HOLE

PROJECT: Southeast Outfall  
LOGGED BY: CS

DRILL HOLE NO.: DH-5  
CHECKED BY: CL

DRILLING METHOD: Rotary Wash  
REFERENCE EL.: 0 ft. (Approx)

DRILLING DATE: 4/2-3/86  
DATUM: SFCD

GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

<table>
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<td>60</td>
<td>9</td>
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"YOUNGER BAY MUD (Qyb)"
Silty clay (CH), gray, soft to firm, high plasticity
TORVANE = 500 psf

"BAY SIDE SANDS (Qbs)"
Clayey sand (SC), gray, dense to very dense, fine-grained
Decreased clay content (SP-SC) and fine to medium-grained at 65 feet
**LOG OF DRILL HOLE**

**PROJECT:** Southeast Outfall  
**LOGGED BY:** CS  
**DRILL HOLE NO.:** DH-5  
**CHECKED BY:** CL  
**REFERENCE EL.:** 0 ft. (Approx)  
**DRILLING METHOD:** Rotary Wash  
**DRILLING DATE:** 4/2-3/86  
**DATUM:** SFCD

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<td></td>
<td></td>
</tr>
<tr>
<td>-95</td>
<td></td>
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<td></td>
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</tr>
</tbody>
</table>

**GEOTECHNICAL DESCRIPTION AND CLASSIFICATION**

- "BAY SIDE SAND (Qbs)"
  - CLAYEY SAND (SP-SC), gray, very dense, fine to medium-grained

- SAND (SP), brown, fine-grained, very dense at 75 feet
  - Moisture Content: 111, Plastic Limit: 21

- "OLDER BAY MUD (Qob)"
  - SILTY CLAY (CH), gray-green, stiff, moderate to high plasticity

- TORVANE = 1700 psf
  - Moisture Content: 64, Plastic Limit: 62
  - TV

Bottom of boring at 86½ feet below the ground surface at approximately elevation -86½ feet (SFCD)
### Unified Soil Classification System (ASTM D-2487)

<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Group Symbols</th>
<th>Typical Names</th>
<th>Laboratory Classification Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>GW</td>
<td>Well-graded sands, gravel-sand mixtures, little or no fines</td>
<td>$G_o &gt; 0.4$, between 1 and 3 $G_s$ $D_{40}$ not meeting all gradation requirements for GW</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td>Poorly graded sands, gravel-sand mixtures, little or no fines</td>
<td>$G_o &gt; 0.5$, between 1 and 3 $G_s$ $D_{40}$ not meeting all gradation requirements for GW</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>Silty sands, sand-silt mixtures</td>
<td>$G_o &gt; 0.5$, less than 4 $G_s$</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>Clayey sands, gravel-clay mixtures</td>
<td>$G_o &gt; 0.5$, less than 7 $G_s$</td>
</tr>
<tr>
<td></td>
<td>S</td>
<td>Well-graded sands, gravel-sand mixtures, little or no fines</td>
<td>$G_o &gt; 0.5$, less than 7 $G_s$</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>Poorly graded sands, gravel-sand mixtures, little or no fines</td>
<td>$G_o &gt; 0.5$, less than 7 $G_s$</td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Silty sands, sand-silt mixtures</td>
<td>$G_o &gt; 0.5$, less than 7 $G_s$</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>Clayey sands, sand-clay mixtures</td>
<td>$G_o &gt; 0.5$, less than 7 $G_s$</td>
</tr>
<tr>
<td></td>
<td>ML</td>
<td>Inorganic clays and very fine sands, silt, silt or clayey fine sands, or clayey silts with slight plasticity</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, clayey silts, sandy clays, clayey clays, lean clays</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>Organic clays and organic silty clays of low plasticity</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
<tr>
<td></td>
<td>MH</td>
<td>Inorganic clays, micaeous or diatomaceous fine sands or silty soils, elastic soils</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
<tr>
<td></td>
<td>CH</td>
<td>Inorganic clays of high plasticity, stiff clays</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic soils</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
<tr>
<td></td>
<td>PI</td>
<td>Paste and other highly organic soils</td>
<td>Limitation in plasticity chart with P.I. between 4 and 7 and borderline cases requiring use of dual symbols</td>
</tr>
</tbody>
</table>

### Soil Classification Subdivisions

Division of GM and SM groups may be subdivided into d and u for roads and airfields. Suffix d is used when liquid limit is 28 or less and plastic index is 6 or less whereas suffix u is used when liquid limit is greater than 28.

### Sample Types

- Bulk Sample
- Punched Shelby Tube
- Pitcher Barrel
- Standard Penetration
- Modified California

### Blow Count

The number of blows required to drive the sampler the last 12 inches of an 18 inch drive. The notation 100/9 indicates 9 inches of penetration were achieved in 100 blows.

### Additional Tests

- UC: Unconfined Compression
- TD: Drained Triaxial Compression
- TU: Undrained Triaxial Compression
- GS: Grain Size Distribution
- SF: Specific Gravity
- CP: Compaction
- CN: Consolidation
- TV: Torvane Shear
- DS: Direct Shear
- FM: Permeability
- EX: Expansion
- SW: Swell
- BS: Resistivity
- CBR: California Bearing Ratio
- RV: K-Value
- PP: Pocket Penetrometer

---

PLATE A-2 UNIFIED SOIL CLASSIFICATION & LEGEND TO DRILL HOLE LOGS
NOTE: Consolidation curves are for 24 hour load increments.

<table>
<thead>
<tr>
<th>HOLE NO.</th>
<th>SAMPLE NO.</th>
<th>DEPTH (ft)</th>
<th>INITIAL SPECIMEN DATA</th>
<th>FINAL SPECIMEN DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>DRY DENSITY (pcf)</td>
<td>WATER CONTENT (%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>VOID RATIO</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>DRY DENSITY (pcf)</td>
<td>WATER CONTENT (%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DEGREE OF SATURATION (%)</td>
</tr>
<tr>
<td>1</td>
<td>1-6</td>
<td>65</td>
<td>65.3</td>
<td>58.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.676</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>77.8</td>
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<td></td>
<td>44.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>100</td>
</tr>
</tbody>
</table>

Southeast Outfall System Modifications
San Francisco, California

CONSOLIDATION TEST
PROJECT NO. | DATE   | FIGURE NO.
SF51002     | MAY 1986 | A-3
Southeast Outfall System Modifications
San Francisco, California

CONSOLIDATION TEST
TIME - COMPRESSION CURVES

PROJECT NO.  DATE  FIGURE NO.
SF51002  MAY 1986  A-4

KEY
1. 2000-4000 psf
2. 4000-8000 psf

Boring: DH-1
Sample No.: 1-6
Sample Depth: 65 ft.
Southeast Outfall System Modifications
San Francisco, California

UNCONFINED COMPRESSION TEST

<table>
<thead>
<tr>
<th>SPECIMEN NO.</th>
<th>4-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drill Hole No.</td>
<td>4</td>
</tr>
<tr>
<td>Depth (ft)</td>
<td>10</td>
</tr>
<tr>
<td>Water Cont. %</td>
<td>85.0</td>
</tr>
<tr>
<td>Dry Density,pcf</td>
<td>49.5</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>2.53</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>94</td>
</tr>
</tbody>
</table>

NOTE:

AXIAL STRAIN, %

UNCONFINED COMPRESSIVE STRENGTH, psi

800
700
600
500
400
300
200
100
0
0 3 6 9 12
During our field investigation, the soils encountered in drill hole DH-2 at approximately 3 to 8 feet below the ground surface had an "oily" appearance and odor. Samples of these soils were taken to a chemical testing laboratory to determine if they contained hazardous substances. A gas chromatography/mass spectrometry (GC/MS) test for semi-volatile compounds (U.S. EPA Method 8270) was performed on a drive sample and an oil and grease analyses was performed on a bag sample. The results of the GC/MS test for semi-volatiles are shown on Plate B-1. The results indicate that the soil does not contain priority or non-priority pollutant compounds in excess of the limits determined by the EPA. However, the oil and grease analyses showed that the soil materials at a depth of 6 to 7 feet contained 870 mg/kg of oil and grease.

The EPA does not have specific upper limits for the amount of oil and grease a soil may contain. However, the presence of oil and grease may indicate that the soil contains volatile priority pollutant compounds and/or the soil may have a flashpoint below the upper limit determined by the EPA. Specialized sampling and storage equipment is needed to obtain soil samples for laboratory analyses for volatile compounds and flashpoint testing. Since this equipment was not available at the time drill hole DH-2 was drilled, the soil samples taken were not sampled or stored adequately and tests for volatile compounds and flashpoint were not performed.
In conclusion, the soil encountered in drill hole DH-2 contains oil and grease but does not contain semi-volatile pollutants above the limits determined by the EPA. However, due to the unavailability of sampling and storage equipment during drilling, it has not been determined if the soil contains volatile pollutant compounds or has a flashpoint below the upper limit determined by the EPA. Recommendations for further study are contained in our proposal to the San Francisco Clean Water Program dated June 11, 1986.
Lab.No. 862106  Date Collected -  Collected by Allstate Geotechnical Services  
Source S.E. Outfall SF51002  Time Collected - Sample Type: Sediment: DH-2 5 ft

**PRIORITY POLLUTANT COMPOUNDS**

<table>
<thead>
<tr>
<th>Compounds</th>
<th>mg/kg(ppm)</th>
<th>Compounds</th>
<th>mg/kg(ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>phenol</td>
<td>&lt;0.5</td>
<td>2,6-dinitrotoluene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2-chlorophenol</td>
<td>&lt;0.5</td>
<td>acenaphthene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2-nitrophenol</td>
<td>&lt;0.5</td>
<td>2,4-dinitrotoluene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2,4-dimethylphenol</td>
<td>&lt;0.5</td>
<td>fluorene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2,4-dichlorophenol</td>
<td>&lt;0.5</td>
<td>diethyl phthalate</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>4-chloro-3-methylphenol</td>
<td>&lt;0.5</td>
<td>4-chlorophenylphenyl ether</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2,4,6-trichlorophenol</td>
<td>&lt;0.5</td>
<td>N-nitrosodiphenyamine</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2,4-dinitrophenol</td>
<td>&lt;2.5</td>
<td>1,2-diphenylhydrazine (azobenzene)</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2-methyl-4,6-dinitrophenol</td>
<td>&lt;2.5</td>
<td>4-bromophenyl ether</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>4-nitrophenol</td>
<td>&lt;0.5</td>
<td>hexachlorobenzene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>pentachlorophenol</td>
<td>&lt;0.5</td>
<td>phenanthenene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>bis(2-chloroethyl)ether</td>
<td>&lt;0.5</td>
<td>anthracene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>1,3-dichlorobenzene</td>
<td>&lt;0.5</td>
<td>di-n-butyl phthalate</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>1,4-dichlorobenzene</td>
<td>&lt;0.5</td>
<td>fluoranthene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>1,2-dichlorobenzene</td>
<td>&lt;0.5</td>
<td>benzidine</td>
<td>&lt;2.5</td>
</tr>
<tr>
<td>bis(2-chloroisopropyl)ether</td>
<td>&lt;0.5</td>
<td>pyrene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>hexachloroethane</td>
<td>&lt;0.5</td>
<td>benzyl butyl phthalate</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>N-nitrosodi-n-propylamine</td>
<td>&lt;0.5</td>
<td>benz[a]anthracene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>nitrobenzene</td>
<td>&lt;0.5</td>
<td>3,3'-dichlorobenzidine</td>
<td>&lt;1</td>
</tr>
<tr>
<td>isophorone</td>
<td>&lt;0.5</td>
<td>chrysene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>bis(2-chloroethoxy) methane</td>
<td>&lt;0.5</td>
<td>bis(2-ethylhexyl)phthalate</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>1,2,4-trichlorobenzene</td>
<td>&lt;0.5</td>
<td>di-n-octyl phthalate</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>naphthalene</td>
<td>&lt;0.5</td>
<td>benzo[b]fluoranthene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>hexachlorobutadiene</td>
<td>&lt;0.5</td>
<td>benzo[k]fluoranthene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>hexachlorocyclopentadiene</td>
<td>&lt;0.5</td>
<td>benzo[a]pyrene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>2-chloronaphthalene</td>
<td>&lt;0.5</td>
<td>indeno[1,2,3-cd]pyrene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>dimethyl phthalate</td>
<td>&lt;0.5</td>
<td>dibenz[a,h]anthracene</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>acenaphthylene</td>
<td>&lt;0.5</td>
<td>benzo[ghi]pyrrole</td>
<td>&lt;0.5</td>
</tr>
</tbody>
</table>

**NON-PRIORITY POLLUTANT COMPOUNDS**

- benzoic acid <5 4-chloroaniline <2.5
- 2-methylphenol <0.5 2-methylnaphthalene <0.5
- 4-methylphenol <0.5 2-nitroaniline <2.5
- 2,4,5-trichlorophenol <0.5 3-nitroaniline <2.5
- aniline <0.5 dibenzofuran <0.5
- benzyl alcohol <1 4-nitroaniline <2.5

Analysis by U.S. EPA Method 8270, reported in milligrams per kilogram, wet (as received) weight basis.

Analyst  JW  Manager  [Signature]

This report applies only to the sample investigated and is not necessarily indicative of the quality of apparently identical or similar samples. The liability of the laboratory is limited to the amount paid for the report by the issuer. The issuer assumes all liability for the further distribution of this report or its contents and by making such distribution agrees to hold the laboratory harmless against all claims of persons so informed of the contents hereof.