GEOTECHNICAL INVESTIGATION
FOR THE PROPOSED EXPANSION OF
OYSTER POINT MARINA
South San Francisco, California

For

THE CITY OF SOUTH SAN FRANCISCO
c/o DANIEL, MANN, JOHNSON & MENDENHALL
Redwood City, California

By

WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists, and Environmental Scientists
Oakland, California

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Project: 13609A

The City of South San Francisco
c/o Daniel, Mann, Johnson & Mendenhall
611 Veterans Boulevard
Redwood City, California 94063

Attention: Mr. William A. Gissler
Project Manager

Gentlemen:

We are pleased to present our report containing the results of the geotechnical investigation made for the proposed expansion of Oyster Point Marina in South San Francisco. The report contains our recommendations concerning site preparation and grading, leachate and methane control, building foundations, breakwater and mooring systems, and pavements. The report also contains the results of the field soils explorations and laboratory tests performed for the present study by WCC as well as for previous studies done by others at the site.

Donald Treadwell and David Daniel of our staff provided significant input to this study. If you have any questions regarding this report, please contact us. It has been a pleasure to be of service to you on this project.

Sincerely yours,

Edwin M. Hultgren
Edwin M. Hultgren
Senior Staff Engineer

Edward Marg
Edward Margason
Associate

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INTRODUCTION

This report presents the results of our geotechnical investigation at the site of the Oyster Point Marina Expansion in South San Francisco, California. The purpose of this study is basically to provide site specific geotechnical design criteria for expansion of the marina and for landscaping and limited building developments on an extensive sanitary landfill which adjoins the marina.

Our proposed scope of work was submitted to the owner through Daniel, Mann, Johnson & Mendenhall (DMJM) of Redwood City in a letter dated December 10, 1975. As a result of that submittal, an agreement for geotechnical services was sent to our firm on April 20, 1976 by Mr. Ken Wuest of DMJM, and Exhibit A of that submittal gives our final scope of work in detail. Our acceptance of that agreement was returned to Mr. Wuest on April 26, 1976, and work was begun on the 12 tasks enumerated in the scope of geotechnical services; this report presents our findings and recommendations for that scope of work and hereby completes our contract requirements.

PROPOSED DEVELOPMENT

General

Oyster Point Marina is located at the eastern end of Oyster Point Boulevard in South San Francisco adjacent to the north side of a sanitary landfill about 50 acres in size. As the Site Plan (Fig. 1) indicates, the existing on-shore portion of the Oyster Point Marina utilizes less than twenty percent of the available land acreage, and is concentrated on the western half of the north shore of the peninsula. Present facilities include 283 berthing slips, a launching ramp, a fuel dock, a private yacht club, a harbormaster's office, and limited space for boat storage and repair.

The proposed expansion of Oyster Point Marina will provide for a large increase in the number of berthing spaces as well as improved waterfront access for the general public. Selective on-shore commercial development is planned to provide needed services and amenities. The extent of the proposed expansion and development is shown on Figure 1.

Off-Shore

Present berthing capacity will be essentially doubled by the off-shore construction planned for the eastern half of the northern portion of the site. More than 300 new berths are planned in this area as well as an extension of the existing east-west breakwater. A north-south breakwater is also planned to complete the harbor protection system. Additional water-related construction may include the repositioning of the launch ramp, a new boat hoist, and a fuel dock. A fishing pier is included for general public use on the eastern shore of the landfill.

On-Shore

Proposed commercial development of the on-shore acreage includes two restaurants and a specialty shops area, as well as a new yacht club and harbormaster's office. The boat repair and storage facilities will also be improved and expanded. Concurrently, a fairly extensive roadway and parking system will be constructed. General improvement of the site consisting of grading and additional sealing of the rubbish landfill will precede construction of roadways and structures. Extensive landscaping and open space development is also planned.

FIELD INVESTIGATION AND LABORATORY TESTS

Field and laboratory investigations were made as a part of this study to evaluate the extent and nature of the subsurface materials. The field work included detailed visual inspections of the site, discussions with persons familiar with the site history, gathering and testing mudline samples from potential dredging areas, drilling many borings both on-shore and off-shore and excavating inspection pits. The laboratory program included tests to evaluate strength and compressibility under various loading conditions and classification tests. The results of the field and laboratory investigations are presented in Appendix A attached to this report. The logs of borings and trenches made by other organizations in previous years have been reproduced and are presented in Appendix B. All of these field findings are described in detail in the following sections.

SITE AND SOIL CONDITIONS

Site History

Prior to the original development of the Oyster Point landfill in the 1950's, the San Francisco Bay shoreline was located along the far west end of the project site at the approximate location shown on Figure 2, entitled "History of Site Development." Immediately west of the original shoreline there rose a low shale and sandstone bluff which comprised a portion of the original Oyster Point.

Following the 1957 enactment of laws prohibiting open-air burning of rubbish in the Bay Area, plans were made to establish a solid waste disposal site on the submerged lands just east of the original Oyster Point. The resultant landfill developed in three distinct phases as shown on Figure 2. The first area to be filled extended eastward from the original bluff approximately 1500 feet to the vicinity of the existing launching ramping. Filling of the first section began in 1957 and was essentially completed by late 1961.

In 1962, the existing small craft harbor was designed and constructed along the north shore of the landfill. To create a filled breakwater for the east side of the marina, the second phase of landfill was placed in the form of a mole extending from the eastern end of the first fill and north 400 feet into the bay, see Figure 2.

The third and final phase of filling was begun in 1964 and was accomplished by dredging up Bay Mud and forming mud dikes and a dike-enclosed cell in which solid waste was later placed. A final alteration to the peninsular landfill configuration consisted of the Cabot, Cabot and Forbes landfill which was begun to the south in 1965 and now abuts the western half of the southern boundary of the Oyster Point landfill. The present topography of the entire site is shown in Figure 1.

The methods by which the landfill was constructed evolved and became somewhat more sophisticated with time. The first and second phase fills apparently were placed with only a limited effort being made to prevent free mixing of bay waters with the solid waste; such mixing occurred along the south and east sides of phase one and around the entire perimeter of the phase two mole. The northern limit of the phase one fill was primarily constructed of natural soil materials thereby creating a relatively impervious barrier between the solid waste and the existing marina waters.

The mud dike of the third phase fill also formed a relatively impermeable barrier to prevent mixing of waters on either side of the dike. It has been reported that some minor slope instabilities developed during or shortly after placement of the northern mud dike. Based on our field investigation and studies of available aerial photographs taken at the time of filling, it appears that the northern dike has remained intact with the possible exception of a small, localized breach into the cell just east of the mole.

The phase three solid waste operation proceeded quickly and garbage fill was placed up to as much as 24 feet above mean sea level in the easterly half of the site. The rapid filling apparently created unstable mud dike slopes, and some landslide failures occurred into the bay along portions of the south shore and at the southeast corner of the site. Portions of the fill were then removed in these areas and the slopes flattened in a successful effort to stop the landslide movements. The solid waste disposal operations were completed about 1971, and since that time site work has been limited to remedial measures to control leaching around the mole and some local stabilizing of the south slope.

Existing Subsurface Conditions

The extent of the present site is shown on the hydrographic/topographic base map (Fig. 1) prepared by Towill, Inc. for Brian-Kangas-Foulk and Associates and dated March 11, 1976. Of the approximately 57 acres above the high water line, approximately 49 acres are filled land while the remaining 8 acres comprise the natural bluff and original shoreline. The original shoreline and portions of the bluff have also been filled over with rubbish. The primary fill material is refuse placed when the site was an active solid waste disposal site. Utilizing information from exploratory borings and test trenches, aerial photographs from various periods of development and personal communications with persons familiar with the landfill development, a plan has been prepared showing the lateral extent and contours of estimated thicknesses of solid waste. This plan, enclosed as Figure 3, is an interpretation of limited data and was prepared as an aid for predicting future site settlements and for evaluating the extent of proposed leachate and gas control measures.

The refuse fill is primarily composed of domestic and light industrial solid waste such as newspapers, cardboard, cans, some food scraps and the like. The fill is overlain by a thin sandy soil cap of varying thickness, quality and composition. The rubbish is decomposing at a relatively slow rate and is consolidating under both its own weight and the weight of the surface soil cap.

Most of the fill at the site is underlain by soft dark gray silty clays of medium to high plasticity known locally as San Francisco Bay Mud. These clays are quite compressible under loads, and are presently consolidating under the weight of the existing landfill. The thickness of Bay Mud varies considerably across the site from zero near the old shoreline to over 80 feet beneath the eastern half of the landfill. Figure 3 shows the approximate elevation contours of the base of the Bay Mud.

The materials which underlie the fill and Bay Mud vary across the site from rock on the west to stiff clays and dense sands toward the Bay. Borings 76-1 through 76-5 on the west end encountered medium hard shale and sandstone bedrock immediately below the fill. These borings were in the vicinity of proposed Restaurant One (see Fig. 1) and were located near the top and bottom of the original bluff. Boring 76-14 encountered sandstone bedrock below the Bay Mud. The borings in the vicinity of the proposed specialty shops and Restaurant Two encountered very stiff clays and some dense sands beneath the Mud. The two deeper borings in these areas (76-19 and 76-20) encountered bedrock near project Elevation +12 feet (project datum is MSL plus 100 ft). The borings which encountered bedrock suggest that the bedrock surface slopes to the east with an average gradient of approximately five horizontal to one vertical. Eastward from the marina mole, the borings which penetrated the deep Bay Mud typically encountered dense sands although older and stiffer Bay Muds were also encountered below the recent Mud in Boring 76-21.

Idealized profile A-A, enclosed as Figure 4, presents a visual interpretation of the stratigraphic soil conditions along the east-west axis of the site. Profiles B-B and C-C (also shown on Fig. 4) present an idealization of subsurface conditions on north-south sections cut through the easterly and westerly portions of the site, respectively.

Free groundwater was encountered in all the borings which penetrated below mean sea level (MSL = 100 Elev.). The water table is found consistently at an average of 1 to 2 feet above MSL within the fill, and does not appear to be noticeably affected by tides.

Existing Surface Conditions

At present the site surface is largely undeveloped with the existing improvements centered around the present marina along the northwest shore of the landfill. The unimproved portions of the site typically are graded smooth and have grass or ice plant cover. A large soil fill stockpile exists near the southwest corner of the site.

The improved area includes a 283-berth marina, boat launching ramp, paved parking, yacht club building and harbormaster's office. Dry boat storage occupies the mole and a swimming beach is located at the west end of the marina.

DISCUSSION OF FINDINGS

Soil Cap

The materials comprising the present soil cover vary across the site and in many areas are quite sandy. Such material is moderately pervious and, in areas where the soil cover thickness is nine inches or less, may not offer sufficient resistance to surface water percolation during the rainy season.

Due to continued compression within the garbage fill and underlying Bay Mud, some areas of the site have settled differentially forming depressions on the surface of the soil cap. These depressions retain seasonal rains and allow some of the water to percolate through the existing sandy soil cover into the rubbish fill. The resulting higher water levels within the rubbish fill create a greater potential for leachate seeps around the site periphery and local saturation may also accelerate the decomposition rate of the rubbish fill.

Two feet of tight, well-compacted clayey cap material is, in our opinion, likely sufficient to minimize surface water infiltration. Some cracking of the cap is likely to occur as the site continues to settle differentially primarily due to compression within the landfill. Techniques for reducing the cracking include preloading the area of concern to minimize future compression in the solid waste and incorporating filter fabric in the clay cap in selected areas to control the size and vertical propagation of the cracks as recommended later.

Settlement

Surface settlements at this site of as much as 6 to 7 feet are anticipated under existing conditions. The anticipated settlements will be due to the long-term compression of the rubbish fill and the underlying Bay Mud. Approximate total future settlement estimates for various zones at Oyster Point are presented graphically on Figure 5. Such settlement predictions include both fill and mud settlement.

Rubbish fills of the type encountered at Oyster Point will typically compress as much as 15 to 30% of their original height under their own weight. At this site, it is estimated that approximately two-thirds of this settlement has already occurred. In areas of maximum thickness, the rubbish alone could experience from 1-1/2 to 2-1/2 feet of additional settlement in the next 40 years under the existing loading conditions. Each new additional foot of soil fill could cause 1/4 to 1/2 foot of compression for each 10 feet of rubbish thickness; however, approximately half of this additional compression could be expected to occur by the time construction at the site is complete.

The Bay Mud is also consolidating and compressing under the landfill load. Based on our laboratory tests and field measurements and our previous experience with this natural soil material, it is estimated that from 4 to 5 feet of future settlement may occur due to the primary and secondary consolidation of the Bay Mud beneath the interior portions of site in areas where the Bay Mud is thickest.

The above settlement estimates may be in error by as much as 20 percent, primarily due to uncertainties regarding the compressibility characteristics of the rubbish fill. In areas of maximum rubbish fill and Bay Mud thickness, the settlement time rate may be in the range of 3 to 6 inches per year for fifteen years decreasing gradually to less than 3 inches per year thereafter. The proposed addition of up to 4 feet of soil cover across the site could increase the above estimated settlements by as much as 20 percent more than shown on Figure 5.

The magnitude of future settlement will be greater on the eastern half of the site where the Bay Mud is thicker and the rubbish fill has been placed more recently. Another differential effect is that the settlement will be greater along the wide central axis of the site where the rubbish fill is thickest and less along the edges where the rubbish becomes thinner.

Slope Stability

The southern shore of the landfill (see Fig. 3) has had some lateral movements in the past and repair measures have been taken to stop the movements. Based on our observations to date, this slide stabilization work appears to be effective, and no indications of continued movement were noted. We have analyzed the existing slopes using available information to aid in evaluating the possible need for and effects of additional stabilization work. Our analyses indicate that the existing slopes have moderate factors of safety and that they are becoming more stable each year as the Bay Mud continues to drain and consolidate. Our analyses also indicate that the proposed leachate and erosion control measures will have little affect on the existing overall stability of the slopes since they will add little, if any, new surcharge loads.

Two items may affect the stability of slopes significantly: additional fill loads and dredging. Our analyses indicate that adding three to four feet of soil cover (the load the proposed clay cap and topsoil would exert) could reduce the existing factor of safety approximately 15 percent below its present value. Dredging alone could reduce slope stability by 10 percent around the mole in the marina area. The combined affects of filling and dredging could reduce the factor of safety against sliding to 20 to 25 percent below its present value.

Our analyses indicate that the mole has a present static factor of safety of approximately 1.5. Rapid mole filling to Elevation 110 and steep dredging of adjacent muds could reduce this safety factor to about 1.2 which is marginal for long-term loadings. Recommendations for sealing the mole and improving the mole stability to a better value than 1.2 are presented later. On the south slope and at the southeast corner where previous stability problems developed, our analyses indicate the present factor of safety is approximately 1.25.

Some dredging is anticipated in the existing proposed marina areas. Our analyses indicate that for dredged slopes of 6 horizontal to 1 vertical extending down from Mean Lower Low Water to 10 feet below MLLW, the theoretical factor of safety can be maintained above 1.5 in most areas.

The analysis assumes an average slope of 10:1 between MLLW and Mean High Water, MHW. The slopes in the tidal range will be somewhat steeper around the mole. Assuming the slope above MLLW is 5 horizontal to 1 vertical around the mole and that an average of two feet of soil fill will be placed on the top of the mole up to about Elevation 108, our analyses indicate that dredged slopes below MLLW would have to be 10:1 to maintain a 1.3 factor of safety. Cutting the dredged slope to 6:1 around the mole reduces the computed factor of safety by approximately 15 percent.

Leachate

Leachate control has been a major concern at the Oyster Point landfill site and actions have been taken by the Regional Water Quality Control Board to have the City mitigate existing leachate seeps. We have inspected the perimeter of the landfill in detail at low tide on at least three separate occasions, once being at a -1.6 foot tide. The leachate seeps observed during those inspections are in definite areas which correlate well with known specific anomalies (as described later) which occurred in the course of site filling. All other shoreline zones between these specific areas were essentially free of leachate seeps during our detailed inspections.

Mole - The most prominent leachate area is the east side of the mole. In addition, small indications of leachate were observed at many other locations around the mole. Persons familiar with the site development indicate that there was heavy tidal leaching around the west and north sides of the mole prior to placing the existing clay blanket and rip rap in 1974 under the guidance of Harlan Engineers. It is evident to us that this control measure was effective and has greatly reduced leachate on the west side of the mole.

As discussed earlier, the mole was constructed with solid waste in direct contact with the bay waters. Since that time attempts have been made to seal portions of the mole perimeter and these attempts have been successful in reducing the volume of leachate flows. However, some leachate seeps still exist and measures to mitigate this leachate within the marina are necessary, in our opinion.

We believe the two most reasonable approaches for controlling leachate seeps around the mole would be to either expand on the technique used for the earlier work by Harlan or completely excavate the mole making additional space for berthing. Since the city and the design team consider the mole to be a unique feature and an asset to the marina and since preliminary cost estimates indicated no economic advantage to removing the mole, our discussions are henceforth limited to the cover and seal approach on the mole.

The proposed development plans call for placing the harbormaster's office at the north end of the mole and a guest dock on the lee side (east) of the mole. These developments will make future remedial measures difficult and expensive should future problems arise with the leachate seal. Therefore, it would be prudent to install a leachate control system which would minimize as much as practicable the risks of future leachate seeps around the mole. A cover and seal approach calling for removing the existing rubble and rip rap, overexcavating the existing slopes and backfilling with a thick deposit of Bay Mud and blinding with filter fabric and gravel is presented in the recommendation section later in this report.

Slide Area - A second area of observed leachate seeps extends for approximately 1000 feet along the site perimeter from around the southeastern corner, along the south slope to approximately 100 feet west of the eastern limit of the large rip rap, see Figure 3. This is the area where slide movements developed around 1971 and remedial action was taken to control the movements.

Three test trenches were excavated in this area to evaluate the sources and extent of the leachate (refer to Test Trenches 76-T1, -T2, and -T3 in Appendix A). The trenches indicate that the leachate seeps were travelling along specific anomalies beneath the slope such as chunks of concrete rubble, stumps and discarded timber piles. Permeable lenses of sandy fill also were a part of the flow path. The trenches and visible seeps suggested that a shallow clay-filled cutoff trench approximately 25 feet downslope from the high water mark (approximately the 105 ft contour) would adequately seal the existing leachate seeps in this area. Combining the trench with an impermeable surface cover upslope from the trench to above the high water mark should provide an effective seal between the bay waters and the solid waste fill in this area.

Fence Line - A third area in which leachate seeps were observed was near the west end of the rip rap along the south slope. The seeps occur as small point sources spaced approximately 10 feet apart. The test trench revealed that the leachate was travelling vertically along existing timber piles (refer to Trench 76-T4 in Appendix A). A close examination of old aerial photographs revealed an old piled fence line in this area apparently placed in lieu of a mud dike to contain the rubbish. The untrenched seeps line up well with an exposed pile butt and the locations indicated on the aerial photo. The test trench suggested that the fence line could be excavated to some depth below grade, the piles broken off and the excavation backfilled with the excavated Bay Mud spoil. This area is in our opinion the least critical of the four limited seepage areas as the amount of leachate seeping from these point sources is quite small.

Drainage Channel - The fourth area of observed leachate seeps is along the south boundary drainage channel where the solid waste disposal site abuts the Cabot, Cabot & Forbes development. Test Trench 76-T5 and Boring 76-12 indicated two distinctly different conditions. The trench encountered about four feet of loose gravel and clay over uncontaminated clayey sand fill (possibly placed as part of Cabot, Cabot & Forbes development). The boring encountered rubbish fill below the loose clay and gravel.

The best approach for isolating the channel from the solid waste material will likely be a cutoff trench. In the vicinity of Boring 76-12 and possibly in other areas, the trench may have to be constructed very near the channel. The detailed lateral extent of the rubbish may not be precisely defined until the trench is excavated. It may develop that moving the channel to the south may be the most practical solution to cutting off the rubbish in some areas provided that right-of-way and property ownership problems can be solved.

Detailed control schemes for each of the four leachate areas are presented later in the recommendation section of this report. For the three areas on the south side of the site, an option to the more complete schemes recommended later would be to apply corrective measures to the individual leachate seeps observed and accept the likelihood of a higher future maintenance cost if additional seeps appear.

The City and Water Quality Control Board staffs should be aware that some time may be required for the proposed leachate barriers to reach equilibrium especially for the cutoff scheme along the drainage channel. Irregardless of what technique is used or how extensive a leachate control system is installed, there is a risk that a few leachate seeps will develop at some time in the future. The future problems may be repaired as they appear using the same basic schemes recommended for present problem areas.

Erosion

The only serious erosion problem is along the eastern shore of the site which is exposed to the open fetch of bay waters, see Figure 3. The remaining shoreline of the site will either be protected by breakwaters or have short fetches or very flat gravel slopes. A significant quantity of rip rap exists on the site which could be relocated to the east slope. The exposed south shore rip rap serves little purpose and sufficient protection could be provided by coarse gravel and cobbles in this area. Additional rip rap may be buried where the slide stabilization work was performed and could be recovered prior to placing the clay cap. A third on-site source of rip rap is the mole where the rock and rubble would be removed prior to effecting the total mole leachate seal. Detailed recommendations for erosion control are presented later in this report.

Methane Gas

Methane gas control is often encountered at completed disposal sites where development is planned. All persons associated with the planning, design, and construction of such facilities should be cognizant of potential gas problems so that proper attention is given to the details which minimize the methane effects. With proper planning and implementation of the appropriate details, the production of methane gases below grade should not adversely affect the development.

A site the size of Oyster Point likely could generate about 1000 cubic feet of methane per day; this is a relatively small amount and far below that needed to permit economical recycling of the gas as a fuel. Typical problems associated with the methane production include collection in voids in electrical conduits, entrapment in poorly ventilated rooms or buildings, and infrequent surface flares along cracks in pavement (and the potential affect on parked vehicles). Recommendations to prevent the accumulation of gases in structures and control of vent cracks on the surface are presented later.

RECOMMENDATIONS

Site Grading

It is recommended that final site grades be developed realizing that future settlements will occur. (Refer to estimates on Figure 5). It should be noted that the Figure 5 settlement estimates are based on existing fill conditions and that the addition of four feet of new soil fill could increase the estimates by an average of 15 to 20 percent. In addition to providing for the estimated total settlements, it is recommended that a minimum gradient of two percent be incorporated in the final grading scheme to minimize future surface ponding created by differential compression in the rubbish fill. Drainage swales should be located within 300 to 400 feet of each other to minimize surface ponding due to future erratic rubbish compression.

Excavations into the existing rubbish fill will be difficult and costly. Scrapers would not be suitable for such excavations; trucks would probably have to be loaded by clamshells or loaders to efficiently move the garbage materials. It is recommended that excavations into the rubbish or regrading of the rubbish be minimized or avoided wherever possible.

To control surface water infiltration and to aid in controlling methane gas, it is recommended that a minimum two-foot thick compacted clay cap be placed as a seal over the entire site (see Fig. 13). The clay cap should be composed of clayey materials having a Plasticity Index of at least 15 percent and should consist of silty or sandy clays or very clayey sands. The clay cap should be placed in thin lifts not exceeding 8 inches in uncompacted thickness, brought to a moisture content suitable for compacting and compacted to a minimum of 90 percent of California test method 216-G. Where dried and conditioned Bay Mud dredge spoil is used for the clay cap, it is recommended that the minimum relative compaction be 85 percent. Special clay cap details are presented later for parking, roadways, building areas and leachate control. Stockpiling of inert fill materials to precompress selected building areas is discussed as part of the foundation recommendations for the specialty shops.

Slope Stabilization

The results of our stability analyses discussed earlier indicate that the southern shoreline slopes are moderately stable provided no appreciable additional fill load is placed on the slope.

Our primary recommendation for dredging in existing and proposed marina areas is that the dredging start at the existing MLLW line and the dredge slopes be cut at about 6 horizontal to 1 vertical. This recommendation assumes that the eastern half of the site surface will not be raised significantly in elevation and the western half will be raised no more than three feet within 100 feet of the existing high water line.

Assuming that the mole will be raised two feet in elevation and will have average side slopes of 5 horizontal to 1 vertical down to MLLW, it is recommended that future dredged slopes below MLLW be made at 10 horizontal to 1 vertical in the vicinity of the mole.

The mole area of the marina is moderately stable at present; however, the future dredging and measures recommended later for leachate and erosion control will reduce the overall stability somewhat. We estimate the "factor of safety" against sliding at the mole to be approximately 1.3 provided our recommendations for grading, dredging and leachate control are implemented.

Erosion Control

The east or bay end of the peninsula is experiencing some wave erosion problems at present. Prior to placing any protective materials, it is recommended that the east bank be cleared of existing rip rap, surface rubble and other exposed obstructions above project Elevation 100 and smoothed. The new clay cap should then be extended down to the Bay Mud slope on the shingle beach (~Elev.102) to cover any exposed fill and the clay cap covered with plastic filter cloth. It is recommended that on the filter cloth an eight-inch to one-foot-thick blanket of coarse gravel be placed to act as a bedding course for the rip rap. Rip rap should then be placed and should consist of stones or concrete rubble pieces weighing at least 300 to 600 pounds. The above erosion protection is recommended between approximate Elevations 102 and 108 feet around the east end of the site from the new proposed breakwater to the beginning of the south shore as shown on Figure 3.

On the more sheltered southern shoreline, the wave action is likely to be considerably less than on the exposed east end; it is therefore recommended that this erosion protection consist of a smoothly graded site, capped by plastic filter cloth, then eight inches to one foot of well-graded coarse gravel and 25 to 100 pound cobbles or concrete rubble pieces. Our observations along the south slope suggest that the rip-rap has not been needed to date and gravel would have been satisfactory. If the City is willing to assume some possible future maintenance work in exchange for some initial cost savings, we would then recommend eliminating the 25 to 100-pound rip rap along the south slope and relying on the gravel which should be adequate. It is recommended that the south slope erosion protection be applied over the leachate seal work recommended below and cover the portion of the slope between approximate Elevations 102 and 107.

Leachate Control

Mole - It is recommended that a single leachate control system be applied to the entire perimeter of the mole from Mean Lower Low Water up to the top of the mole. The recommended scheme utilizes the previously installed leachate control concept with the addition of a mechanical membrane as a second line of resistance to seepage. The system consists of the following steps and is detailed in Figure 6.

1. Excavate the existing soil cover to Elevation 104 and stockpile off the mole.
2. Remove the existing rip rap and stockpile; remove the surface rubble and timber pile breakwater and dispose offsite of mole.
3. Overexcavate slopes down to approximate Elevation 94 benching the excavation as shown on Figure 6.
4. Smooth the exposed subgrade on the slope and place a 45 mil thickness of Hypalon pond liner (available from Burke Industries, San Jose) or equal from Elevation 94 to the top of the mole.
5. Place Bay Mud dredge spoil over mechanical membrane such that the minimum thickness of the Bay Mud is three feet.
6. Cover the Bay Mud with plastic filter fabric.
7. Complete mole seal with one to two feet of well-graded coarse gravel erosion protection. Increase rip rap to 25 to 300 pound stones on mole tip only where boat wakes and inaccessibility around the proposed harbormaster building will make later maintenance difficult.

It is important that the mole overexcavation extend down into uncontaminated Bay Mud at the toe of the proposed leachate control system so that the seepage paths are positively cut off. The Geotechnical Engineer should approve all leachate seal excavations before the Hypalon is placed. It is also recommended that the Bay Mud spoils and erosion protection materials be placed first at the toe area of the excavation to act as a construction counterberm and thereby minimize slumping and deformation within the Bay Mud during spoil placement.

After the peripheral leachate control and erosion protection is in place on the mole, the Hypalon mechanical membrane should be placed over the surface of the mole site and three feet of clayey soil compacted in place as per the clay cap recommendations given under "Site Grading" above to bring the mole site to Elevation 107. Utilities for the harbormaster's office should not be placed deeper than one foot into the clay cap. After construction on the mole has been completed, a maximum of one foot of topsoil may be placed on the mole. Additional top soil may be placed in future years as the mole settles; however, in no case should the fill ever extend above Elevation 108, otherwise slope instability could occur. Figure 6 is an idealized section through the mole leachate barrier and should be carefully followed for best effect.

<u>Slide Area</u> - It is recommended that the existing leachate seeps in the vicinity of the old slides on the south shore and around the southeast corner be controlled by excavating a cut-off trench, placing a mechanical liner from trench to the high water line and covering the membrane with Bay Mud dredge spoil, see Figure 7.

The following steps are recommended to develop the south slope leachate and erosion control.

1. Clean work area of existing rip rap, rubble and other surface irregularities and grade to a smooth surface.
2. Excavate cut-off trench approximately 25 to 30 feet downslope from the high water line. The trench should be made at least 3 to 4 feet deep into uncontaminated Bay Mud, have a minimum base width of 3 feet and have one horizontal to one vertical sideslopes.

3. Place a 45 mil Hypalon pond liner or equal from the bottom of the trench to above the high water line.
4. Backfill the trench and cover the mechanical membrane with a minimum of two feet of Bay Mud dredge spoil.
5. Place plastic filter fabric over the Bay Mud dredge spoil, and
6. Place the erosion protection materials recommended in "Erosion Control" above.

The estimated extent of the south slope leachate control includes approximately 1000 feet of shoreline as shown on Figure 3.

Fence Line - It is recommended that the visible leachate seeps in the vicinity of the buried pile fence line be controlled by systematically trenching along the old fence line to an average depth of three feet and breaking off the piles three feet below existing mudline as shown on Figure 8a. The trench should then be backfilled with the Bay Mud spoil. It is estimated that the total length of buried pile fence line requiring treatment is no more than 200 feet as shown on Figure 3.

Drainage Channel - It is recommended that the existing rubbish fill be isolated from the creek waters by using two cutoff schemes. For the western two thirds of the creek alignment, a vertical-walled cutoff trench extending a minimum of two feet into uncontaminated clayey soils is recommended (see Fig. 8b). The trench should have a minimum width of two feet. The primary sealing material recommended for trench backfill is Bay Mud dredge spoil. In addition to the Bay Mud, a 10 mil visqueen membrane should be draped against the southern side of the trench to provide a temporary cutoff while the Bay Mud consolidates and seals.

For the eastern third of the drainage channel, it is recommended that the channel be relocated further to the south and a cutoff constructed north of the realigned channel as shown in Figure 8c. The cutoff may be constructed by overexcavating the bottom of the existing channel for a minimum of two feet into uncontaminated clayey soils. It is recommended that a 10 mil visqueen membrane be placed on the southside of the cutoff and the cutoff backfilled with clayey soils. The channel relocation may require cofferdams both upstream and downstream to permit working in the dry, and a pumping system may be necessary to allow creek waters to bypass the work area. The channel relocation would involve work on the Cabot, Cabot & Forbes development and may require the purchase of an easement. The suitability of materials excavated from the new channel realignment for reuse as clayey backfill for the cutoff should be evaluated prior to construction.

Methane Gas Control

It is recommended that provisions be made to vent the subsurface methane gases on an approximate 200 foot grid across the site. A simple detail which may be used to vent the gas would be an air-flare pipe.

The pipe could be installed by drilling a twelve-inch-diameter boring in the fill down to the water table, placing a 20-foot-long, four-inch-diameter coated steel or transite pipe approximately ten feet into the drilled hole, and backfilling the boring with gravel to within five feet of grade. The pipe should also be filled with gravel to prevent clogging by passerbys. A small amount of paper wadding should be pressed down into the boring, and the top five feet concreted into place. The addition of slots in the lower three to four feet of the pipe may facilitate gas removal especially in areas where the bottom of the pipe may be near or below the water table. The pipes need not be spaced on a strict triangular grid but may be located such that they blend into the landscaping. The pipes may be incorporated into building walls, hidden in tree groves or used for sign posts and other architectural purposes; however, no lighting or other electrical equipment should be attached to the pipes.

In parking areas, two details are recommended to improve areal control of methane gas. Filter fabric should be incorporated near the midpoint of the clay cap under pavements to control pavement crack propagation. If surface cracks can be kept small, the volume of gas escaping from any one crack in paved areas will be reduced.

It is also suggested that the parking areas be gravelled or armor-coated and not paved for the first few years. The gravel will diffuse the gases thereby avoiding undesirable concentrations of gases. Asphalt and concrete paving over large areas tend to develop distinct cracklines concentrating vented methane and thus increasing the risk of fires at these locations. The gravelled surface recommended for the parking areas would have the added advantages of masking differential ground movements and saving on initial paving costs. In paved roadway areas, it is also recommended that plastic filter fabric be sandwiched into the clay cap to control cracks as discussed above. The details are illustrated in Figure 13.

All public occupancy buildings should have smoothly soffited structural floors above grade to prevent gas entrapment and permit the free flow of air beneath each structure. It is recommended that the bottoms of all floors be finished flat, such as by installing plywood or other sheeting to the undersides of the first floor joists of wood structures. Also, the spaces between the joists should be vented at both ends to allow air to flow through. Rooms within the structures should be constructed with positive ventilation to avoid the accumulation of gases in such areas. Special attention should be given to electrical services at all buildings. Buried electrical conduits are potential concentrated sources of methane, and techniques should be employed which interrupt the continuity of the conduit prior to entering the building.

Asphaltic concrete slabs which will be cast at grade, such as at the boat repair facility, should be constructed on a minimum of one foot of clean coarse gravel and the gravel covered with a double layer of plastic visqueen. Four inches of Class 2 aggregate base should be placed and compacted over the visqueen prior to placing the asphaltic concrete. The clean gravel should be vented by a subdrain system consisting of 4-inch-diameter perforated metal pipes spaced on a 50-foot grid to bleed off gases which might otherwise accumulate beneath the slab. The subdrain system should be connected to an exhaust fan or other positive ventilator. No unventilated rooms should be constructed on the slab, including closets or tool rooms. All rooms and enclosures in the boat repair facility should be well ventilated.

Pavement Section for Roadways and Parking Areas

It has been recommended above that parking areas not be paved but rather be maintained as gravel or armor-coated areas as detailed in Figure 13. It is recommended that eight inches of aggregate base be used in the parking areas and the material be compacted to 95 percent per California Test Method 216-G. In the roadway areas where parking is likely to be a minimum, the flexible pavement section shown on Figure 13c is recommended over the 8 inches of aggregate base and two-foot clay cap seal. It is anticipated that the clay cap seal will require a layer of filter fabric to control cracks and gas seepage in paved areas. This detail is also shown on Figure 13. Pavements may be constructed over any materials satisfying the requirements for site seal, including Bay Mud dredge spoil, provided it is properly compacted as recommended. Any pavements at the site should be expected to crack and deform with settlements of the landfill surface and higher than normal maintenance should be expected. The City's experience with the paved areas in the existing marina area are good indications of future maintenance requirements which can be expected.

Foundations

Restaurant One - The 6000-square-foot restaurant at the west end of the site will be situated over the original bluff; the west side of the building will be bearing on bluff bedrock, while the east side of the building will be beyond the bluff and underlain by up to about 25 feet of rubbish fill. The details of these conditions are shown in Figure 9.

It is recommended that the east portion of this restaurant, which will be over fill, be supported on drilled pier foundations extending through all fill and to bedrock; the piers should be designed to develop their support through peripheral shear (skin friction) in the rock. End bearing piers socketed into the rock may be considered; however, we believe it may be unsafe due to the presence of methane gases to send workers into a pier cased through garbage to clean the bearing surface. Therefore, our primary recommendation is to design the piers for peripheral shear only ignoring end bearing. Design skin friction values to be used in the bedrock should not exceed 2000 psf for combined dead plus live loads and 3000 psf for transient downward loads. All piers should be drilled a minimum of five feet into the relatively friable bedrock to provide a firm pier socket which would properly key the pier base into the originally sloping face of the bluff. This Geotechnical Engineer should inspect all piers thus placed to be sure proper bedrock penetration is obtained.

The pier holes should then be cased with Sonotubes through the rubbish fill, and the Sonotubes would become a permanent part of the foundation system. Prior to placing concrete, the pier hole should be pumped dry, or if there is an obvious inflow of leachate as opposed to normal groundwater, the pier hole should be backfilled with fresh water and the concrete placed by tremie methods. Type V sulfate-resistant cement should be used for the pier concrete and all other embedded foundation concrete at the site to resist sulphates which would likely develop from leachate over a period of time.

It is recommended that the pier tops be tied into reinforced concrete grade beams to connect all piers together near the site surface. The top of the grade beam in fill area should be cast about one foot below grade and designed for long-term fill downdrag load of 400 pounds per lineal foot. The one-foot depression over the grade beam should be lightly backfilled with non-cohesive sand or gravel. The purpose of the initially depressed grade beam is primarily for visual effects, i.e. to keep it below grade as the filled part of the site settles around the pier foundations and thereby minimize "mole hills" in later years.

It is recommended that the restaurant floor slab be supported clear above grade 2 to 3 feet by short structural columns extending between the soffit of the structural slab and the piers. The resulting air space beneath the structural slab will permit the prevailing winds to easily flush any traces of methane gas from beneath the building before flammable concentrations occur. The bottom of the slab should be flat to prevent gas concentrating.

Shallow spread footings may be used in lieu of drilled piers where little or no rubbish exists and where the depth to rock is shallow. Permissible bearing pressures of 8000 psf for dead plus live loads and 12,000 psf for total wind or seismic loads are recommended for design of spread footings on rock. The footings should be tied to the drilled piers by grade beams, and the structural slab should be cast above grade even in footing areas to provide wind-methane flushing from beneath the slab.

Restaurant Two - A second restaurant of approximately 10,000 square feet will be sited near the harbor in the area presently occupied by the yacht club. As Figure 10 indicates, it is recommended that this restaurant structure be supported on prestressed concrete piles driven through the Bay Mud and the underlying very stiff to hard clays below and several feet into bedrock. Prior to driving piles through fill, it is recommended that any rubbish or dike fill be predrilled or spudded to remove obstacles which possibly could damage piles during driving.

In addition to structural building loads, piles should be designed to resist a total of 7 tons of downdrag per foot of pile circumference (e.g., a 12-inch square pile should be designed for a total of 28 tons of downdrag). The downdrag loads will be caused by downward movement of the fill and Bay Mud as the supporting Bay Mud consolidates under the existing fill and mud surcharge.

It is recommended that the piles be designed as combined friction and end bearing elements with the skin friction developed in the very stiff to hard clays underlying the Bay Mud. An average design skin friction value of 1200 psf is recommended in the very stiff to hard clays for combined dead, live and downdrag loads. This value may be increased by one-third for wind or seismic loadings. An additional end-bearing value of the order of 50 tons is recommended for piles which are driven to refusal a few feet into bedrock.

The depth to bedrock in the vicinity of Restaurant Two is presently known at the location of Boring 76-19. The length of piles could vary a few feet because the depth to bedrock is probably variable over the site; therefore, piles should be cast with moderate extra length and then cut off after driving.

Because of the expected fill settlements, batter piles should not be used. Instead, it is recommended that all piles be driven as vertical elements and that lateral building loads be resisted by bending or moment resistance in the piles. It is estimated that piles supporting an elevated restaurant structure and having ultimate moments between 120 and 200 kip-ft will have ultimate lateral capacities each of 8 to 13 tons, respectively.

More precise embedded pile lateral capacity estimates can be computed after the pile sections are selected. Due to the possibility of lurching and slumping of the ground surface during a strong earthquake, it is recommended that the upper half length of all restaurant piles be provided with extra longitudinal reinforcing bars inside spiral cages or closer spiral steel. The piles should frame into a strong monolithic floor slab to tie the building together and distribute the lateral loads induced by the piles. The main floor slab should be cast above grade with a flat bottom so that methane gas may be flushed by prevailing winds, see Figure 10.

over the cap and left in place at full height for approximately three months or until rapid settlement due to rubbish compression ceases; after that the stockpile may be removed and placed over other areas as capping material where suitable. It is recommended that the top of the surcharge fill extend 30 feet beyond the building limits to precompress the entire specialty shops area. The material for the surcharge could be either clay cap fill or stockpiled topsoil as is convenient.

It is recommended that the buildings be supported only at their four corners to make maintenance jacking simple and to induce less distortion into the structures. The individual footings should be embedded not more than 18 inches into the compacted clay cap and should be connected to one another by stiff grade beams. It is recommended the footings be designed to exert maximum bearing pressures of 1500 psf for combined dead and live loads, and 4000 psf for seismic. The individual jacking points should be designed such that the structure is connected to the foundation by vertical guides and capable of easy jacking and releveling yet also capable of transmitting design lateral seismic shear to the foundation without falling off the jacks.

To enhance the overall appearance of the shop complex, it is recommended that all building and roof lines be offset or staggered between each shop such that the casual observer cannot easily detect differential movements between the structures. The structures should all be elevated above grade two to three feet and smooth sheeting applied to the bottoms of the floor joists to minimize gas entry into the buildings, as detailed in Figure 11. The ends of joist spaces should be well-vented and the joists oriented east-west such that the prevailing westerly winds may flush out gases which could otherwise accumulate in the flooring system. It is suggested that all structures be connected by articulating ramps with gravel or preferably ice plant at grade between all structures to obscure expected signs of differential settlements.

Harbormaster's Office - It is recommended that the harbormaster's office be constructed on grade with relevelling jacks similar to the shops, see Figure 11, with the exception that no temporary surcharge should be placed on the mole. The office should be kept at least ten feet back from the top of the slope to minimize risks of local slope instabilities. All other details should be the same as those recommended for the specialty shops complex.

Yacht Club - It is our understanding that the yacht club will have a large floor area and may be located near the edge of the landfill. It is recommended that piled foundations be used to support the yacht club and that the design details be similar to those of Restaurant Two. The only exceptions are that all piles should be designed for 8 tons downdrag per foot of pile perimeter and that the piles be designed as skin friction elements only. It is recommended that the design skin friction values be 1000 psf for dead plus live loads and 1300 psf for all loads including seismic. The estimated lateral capacity of the piles are the same as those for Restaurant Two.

Boat Repair Shop - The proposed boat repair facility will include an enclosed work area. It is anticipated the enclosure will be a flexible steel building and will have a slab on grade. As shown on Figure 12, it is recommended that three feet of compacted clay cap and one foot of clean gravel for a gas vent be placed under the floor pavement. An impermeable membrane should be placed on top of the gravel followed by 4 inches of aggregate base and 4 inches of A.C. The gravel should have 4-inch perforated metal or plastic pipes spaced about 50 feet apart and running the length of the building in the gravel,and as Figure 12 indicates, the pipes should connect to an industrial exhaust fan to aid in flushing gases out of the gravel.

It should be recognized and understood that the repair shop pavement will deform with the differentially settling ground surface and may crack with time. The amount of differential settlement can be reduced significantly by surcharging the building area as recommended for the specialty shops area.

Shallow footings may be used to support the building columns. It is recommended that the column lines be overexcavated two feet deep and eight feet wide prior to placing the clay cap. The cap should then be placed and compacted providing a total cap thickness of four feet beneath footings. The footings should be embedded through the gravel and cast on the clay cap, as shown in Figure 12. Footings so placed should be designed for bearing pressures of 1500 psf for dead plus live loads and 4000 psf for all loads including wind and seismic. Provision should be made at the base of each column for periodic relevelling.

As discussed earlier, it is recommended that all facilities be well ventilated and special attention should be given to service and tool storage rooms which often have very limited ventilation. It is also strongly recommended that no concrete slabs be used on grade since the predicted settling and cracking will create very difficult maintenance problems over the years. Flexible asphaltic pavements on grade which can be easily resurfaced are far more desirable from a maintenance point of view and the garage areas of the Watergate Complex in Emeryville are a good example of the advisability of using asphalt on settling subgrades.

Fishing Pier - A fixed fishing pier is planned which will extend into the Bay beyond the bulkhead line at the east end of the site. The pier concept has not been finalized yet; however, it is assumed that it will be designed with simple spans between supporting pile bents.

It is recommended that the pier be supported by prestressed concrete piles driven through the Bay Mud and into the underlying firm soils. The westerly piles which extend through fill should be structurally designed to withstand the combination of the fill downdrag loads recommended for Restaurant Two piles and the structural loads from the pier. To estimate the downdrag load on the pile for each bent, it is recommended that the full 7 tons per foot of pile perimeter be applied to all piles west of the MLLW line and that no downdrag load be applied to piles more than 50 feet east of the MLLW line. The downdrag on piles between these two areas may be estimated by linear interpolation.

To resist downward loads, it is recommended that a skin friction value of 1000 psf be assumed for the soils beneath the Bay Mud. Since no off-shore borings were authorized by the City in the fishing pier area, the conditions of the soil materials below the base of mud have been assumed to be similar to those encountered in Boring 76-21 on the mole and in the off-shore borings in the marina area. The ultimate lateral capacities for 12- and 14-inch-square prestressed pier piles are estimated to be 4 kips and 7 kips, respectively.

Berthing and Breakwater - It is recommended that prestressed concrete piles be used to moor the proposed berthing floats in the new marina addition. Our analysis indicates that 12- and 14-inch-square prestressed concrete piles will resist allowable lateral loads of

1.5 and 2.2 kips, respectively. The mooring piles should be tapped into the Bay Mud with tips at about Elevation 40 (60 feet below mean sea level).

Due to the much longer life expectancy of prestressed concrete, we recommend the breakwater be comprised of prestressed concrete sheet piles with tips placed to about Elevation 35. The sheet piles may be either cantilevered or braced with batter piles on the harbor side of the bulkhead. Assuming the resultant of wave forces will be located approximately six to eight feet above mudline, we estimate that 12-inch-thick prestressed cantilevered sheet piling will resist an allowable lateral force of 1.5 kips per lineal foot of wall. It is recommended that the tops of the sheet piles be connected by a continuous reinforced concrete bond beam to aid in maintaining alignment and distributing wave forces. Batter piles may be necessary to reduce moments in the sheet piles if the design wave forces exceed the allowable lateral force on the cantilevered walls.

Utilities

It is recommended that utility corridors be overexcavated and the clay cap thickened such that a minimum of two feet of clay cap will remain between the rubbish fill and the individual utility lines. Conduit and piping should be designed to tolerate marked differential settlements which may develop due to compression in the rubbish. Localized differential movements of the order of 8 to 12 inches in ten feet are possible in rubbish fills. Connections between utilities and buildings should be made flexible to tolerate differential settlements, especially at pile supported structures where differential movements will be equal to the total ground settlement. Utilities should also be designed to connect through the sides of structures rather than through the floors wherever possible to minimize points where methane gases may collect beneath the buildings.

Bedding courses for buried pipes can collect methane gas and direct the gases into manholes and other junction points. Therefore, it is recommended that a clay seal be placed around such access points to cut off the gases. Electrical conduits may also collect gases and should be sealed at junctions.

SEISMIC SITE RESPONSE TO EARTHQUAKES

Potential Seismic Events

The project site is located approximately 4.5 miles east of the San Andreas Fault and 15 miles west of the Hayward Fault, the two fault systems considered most seismically active in the San Francisco Bay Area. For the purposes of this investigation, it is assumed that a Magnitude 7+ event represents a 50-year recurrence interval and a Magnitude 8+ event represents a 100-year recurrence interval for earthquakes on the San Andreas Fault. Events of lesser magnitude, respectively, would be assumed for earthquakes along the Hayward Fault. Considering the proximity to the San Andreas Fault and the higher magnitudes of potential earthquakes, an earthquake on the San Andreas Fault would predominate in seismic design consideration.

Soil Profile

A generalized soil profile was developed to represent subsurface conditions beneath the proposed structures. The 110-foot-deep generalized profile consists of 3 feet of cover fill, 30 feet of rubbish fill, 35 feet of soft Bay Mud, and 42 feet of stiff to hard clayey soils with occasional sand lenses, overlying bedrock. The actual stratigraphy beneath a particular structure may contain thicker or thinner layers of these materials or even only bedrock. The thickness of the rubbish fill and soft Bay Mud layers dominate the seismic response of the soil profile.

Estimates of fundamental period for the generalized site profile given above provide an expected range of periods between 0.5 and 1.5 seconds, depending on the deformation parameters selected and on the severity of ground shaking. Shallower and/or stiffer profiles would have shorter fundamental periods, while deeper and/or softer profiles would exhibit longer fundamental periods, relative to the range given above.

Ground Response Spectrum

Considering the variability of profile stratigraphy and the uncertainties in modeling the dynamic properties of the rubbish fill, a generalized smooth spectrum of absolute acceleration versus period was developed, based upon spectra of recorded motions on soft sites published in the literature. It should be noted that in considering either a Magnitude 7+ or 8+ seismic event, the proximity to the San Andreas Fault and the characteristics of the soft site would lead to essentially very similar absolute acceleration spectra for motions at the ground surface. The potential for severe ground deformations or structural damage would be greater for the larger magnitude earthquake due to such factors as duration of strong motion, which are not directly related to the ground motion response spectra.

Based on our study results, an estimate of the peak bedrock acceleration for a nearby Magnitude 7+ event would be approximately 0.6g. The attenuation of peak acceleration, for vertically propagating shear waves through the soft profile, results in approximately 0.3g peak acceleration at the ground surface, consistent with published results. Based upon normalized acceleration spectra of accelerograms measured on soft sites, modified to 10 percent spectral damping, an amplification factor of approximately 2.1 is obtained over a period range between 0.2 and 1.25 seconds. The resulting estimate of the surface ground motion response spectrum is given in Figure 14.

The spectrum given in Figure 14 may be used as a basis for preliminary seismic design of the proposed structures. It represents an "upper average" spectrum for anticipated ground motions typical for the marina site. It does not include consideration of any factors which are considered in the structural seismic response analysis, such as out-of-phase motions, duration of strong shaking, and ductility.

For structures founded on a significantly different hard soil or bedrock profile, an adjusted response spectrum would be necessary to reflect site-specific conditions.

Liquefaction Potential and Related Problems

Soil borings at the site indicate that the sandy and silty soils beneath the site are generally lense-like, discontinuous deposits, and/or contain sufficient cohesion to suggest that risk of liquefaction or potential for flow failure of these materials is low. It is likely, however, that lenses of loose sandy, or silty soils and loosely compacted fill, both above and below the water table, may tend to densify due to strong ground shaking, resulting in differential settlements at the ground surface after a large event.

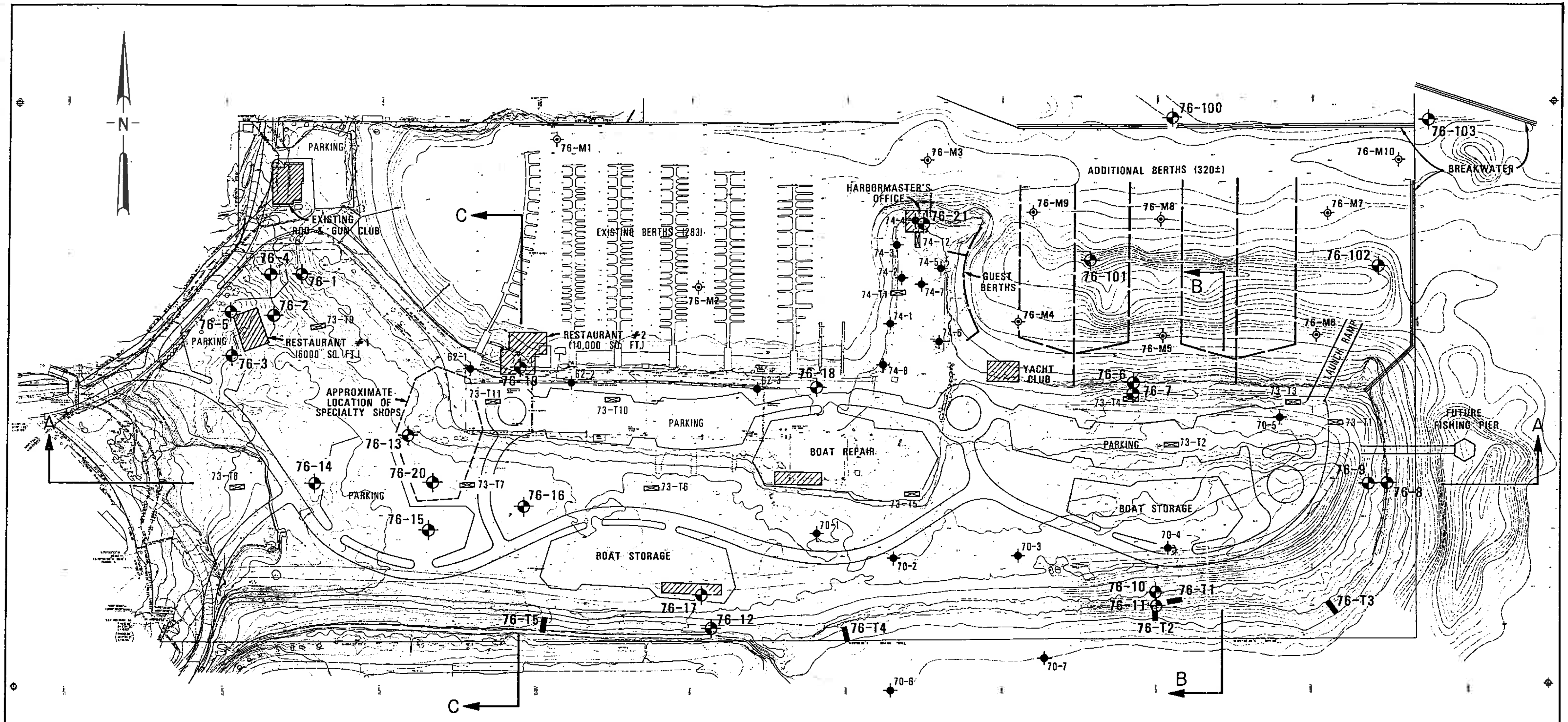
In response to a nearby, strong earthquake, the layer of soft Bay Mud may lose significant strength. This tendency will result in additional differential settlement of the ground surface. The more significant consequences of soil shear strength losses due to a nearby strong earthquake may be relatively high potential for detrimental ground movements near the unrestrained lateral boundaries of the site. Lateral spreading, surface cracking and lurching of the soil mass may develop in the vicinity of the "free face" boundaries of the site underlain by deep soft Bay Mud.

The effects of these potential deformations on shoreline structures, pavements, buried utilities, and other facilities need to be considered in design. The avoidance of structures having fundamental periods approaching those of the subsurface profile is advised. For pile-supported structures, seismically-induced bending stresses and pile cap details may deserve special structural design attention. Finally, important structures near or on any "free face" boundaries of the site should be conservatively designed, as these areas have the greatest potential for detrimental ground movements caused by a strong earthquake.

LIMITATIONS

The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the borings and trenches. If variations or unanticipated undesirable conditions are encountered during construction, or if the proposed development will differ significantly from that described in this report, the Geotechnical Engineer should be notified so that supplemental recommendations can be made.

This report completes our assignment for the geotechnical phase of this project. As designs are finalized, you and your design team may wish to consult with us on unanticipated problems or questions beyond the scope of your study and the review of plans and specifications. The work assignments for the above services will be subject to your authorization prior to our undertaking them.



N O T E S :

1) BASE MAP REDUCED FROM "HYDROGRAPHIC/TOPOGRAPHIC MAP OF OYSTER POINT MARINA" PREPARED BY TOWILL, INC. FOR BRIAN-KANGAS-FOULK & ASSOCIATES DATED MARCH 11, 1976

2) SITE DEVELOPMENT PLANS TAKEN FROM "PRELIMINARY MASTER PLAN - OYSTER POINT MARINA" BY ROYSTON, HANAMOTO, BECK, & ABEY DATED JUNE 15, 1976.

L E G E N D :

⊕	76-1	1976 BORINGS BY WOODWARD-CLYDE CONSULTANTS FOR PRESENT STUDY (25)
▮	76-T1	1976 TEST TRENCHES BY WOODWARD-CLYDE CONSULTANTS FOR PRESENT STUDY (5)
⊕	76-M1	1976 MUDLINE SAMPLES BY WOODWARD-CLYDE CONSULTANTS FOR PRESENT STUDY (10)
✛	74-1	1974 BORINGS BY HARLAN ENGINEERS
✛	70-1	1970 BORINGS BY GRIBALDO, JONES & ASSOCIATES
✛	62-1	1962 BORINGS BY JOHN A. BLUME & ASSOCIATES, ENGINEERS
▨	74-T1	1974 TEST TRENCHES BY HARLAN ENGINEERS
▨	73-T1	1973 TEST TRENCHES BY BERLOGAR, LONG & ASSOCIATES

0 100 200 300 400 500 600

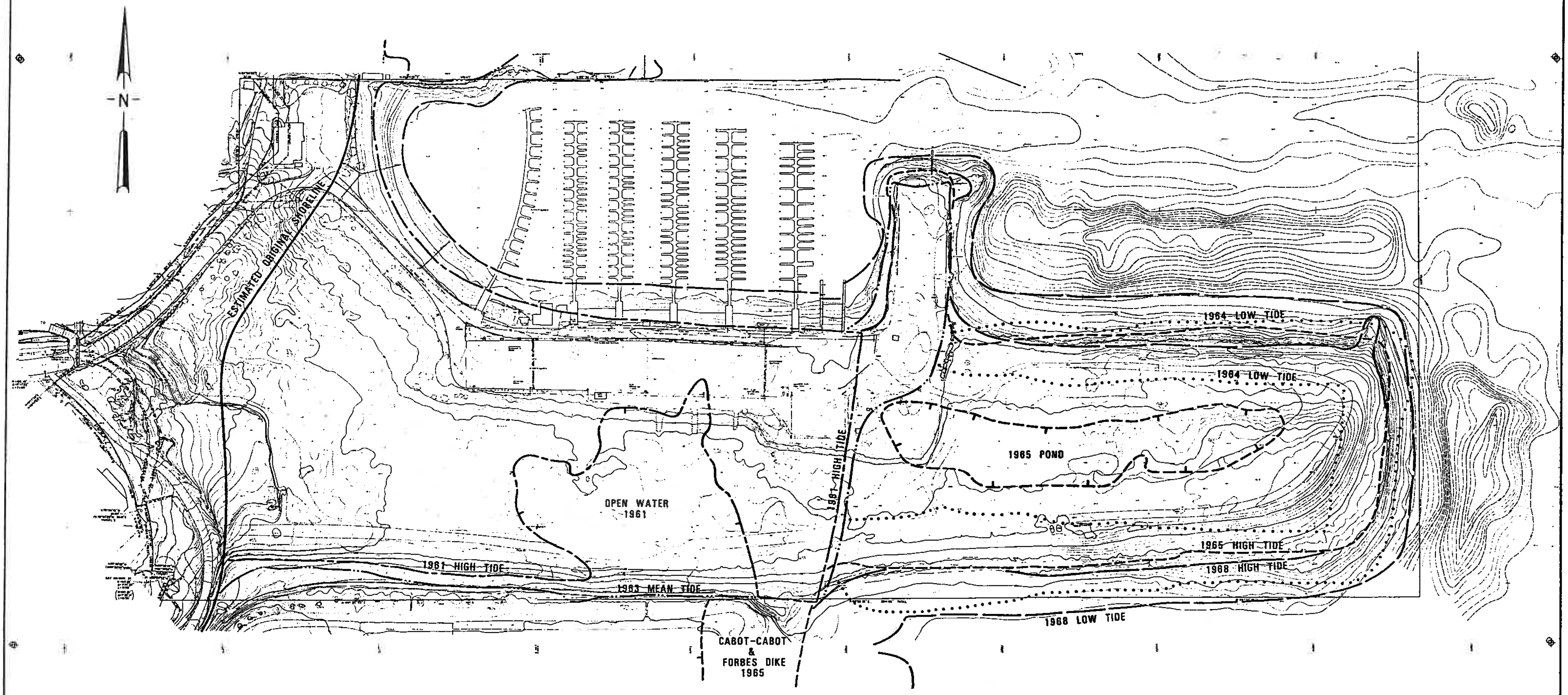
Scale in Feet

1" = 250'

SITE EXPLORATION PLAN
AND PROPOSED DEVELOPMENT
OYSTER POINT MARINA
South San Francisco, California

Project No. 13609A
WOODWARD-CLYDE CONSULTANTS

Figure 1



N O T E S :

1) EXTENT OF LANDFILL AT VARIOUS STAGES OF SITE DEVELOPMENT BASED ON AERIAL PHOTOGRAPHS TAKEN BY PACIFIC AERIAL SURVEYS ON THE DATES NOTED.

2) BASE MAP REDUCED FROM "HYDROGRAPHIC/TOPOGRAPHIC MAP OF OYSTER POINT MARINA" PREPARED BY TOWILL, INC. FOR BRIAN-KANGAS-FOULK & ASSOCIATES DATED MARCH 11, 1976.

3) PRECISION OF LINES DENOTING EXTENT OF LANDFILL APROXIMATELY ±20 FEET.

LEGEND OF LINES DENOTING EXTENT OF LANDFILL AT VARIOUS STAGES OF DEVELOPMENT:

SYMBOL	DATE OF AERIAL PHOTOGRAPH	CONDITION(S) SHOWN
——— - ———	JUNE 20, 1961	HIGH WATER LINE (PHOTO NEAR MSL)
-·-·-··-·-·-	JULY 9, 1963	NEAR MEAN SEA LEVEL
• • • • • • • • • • •	JUNE 18, 1964	NEAR MEAN LOWER LOW WATER
— — — — — —	JUNE 29, 1965	NEAR HIGH TIDE
———— ————	JUNE 12, 1968	BELOW MEAN LOWER LOW WATER AND HIGH WATER LINE

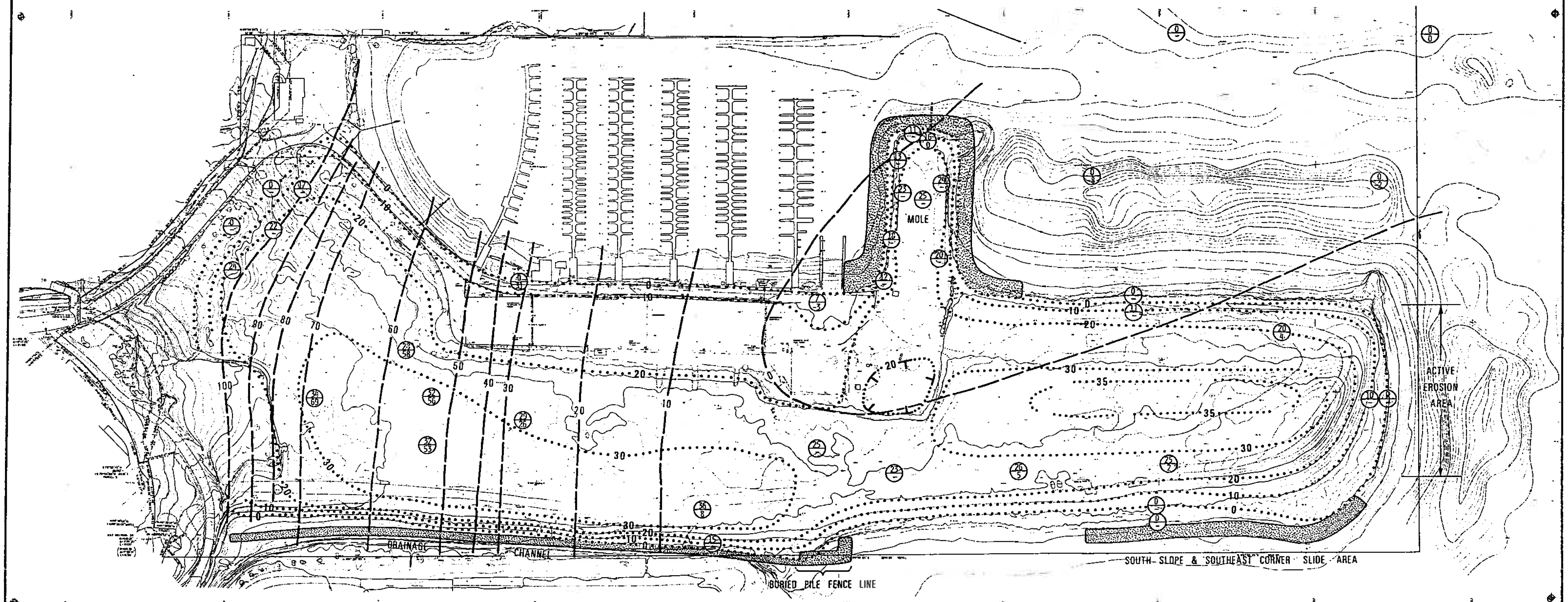
0 100 200 300 400 500 600
Scale in Feet

HISTORY OF SITE DEVELOPMENT

OYSTER POINT MARINA
South San Francisco, California

Project No. 13609A
WOODWARD-CLYDE CONSULTANTS

Figure 2



NOTES:

1) BASE MAP REDUCED FROM "HYDROGRAPHIC/TOPOGRAPHIC MAP OF OYSTER POINT MARINA" PREPARED BY TOWILL, INC. FOR BRIAN-KANGAS-FOULK & ASSOCIATES DATED MARCH 11, 1976.

LEGEND:

••••••20•••••• ESTIMATED THICKNESS OF RUBBISH (FEET)

——40—— ESTIMATED ELEVATION OF BASE OF BAY MUD (FEET)

26/5 APPROXIMATE RUBBISH THICKNESS (FEET) / ELEVATION OF BASE OF BAY MUD (FEET) } AT BORING LOCATIONS

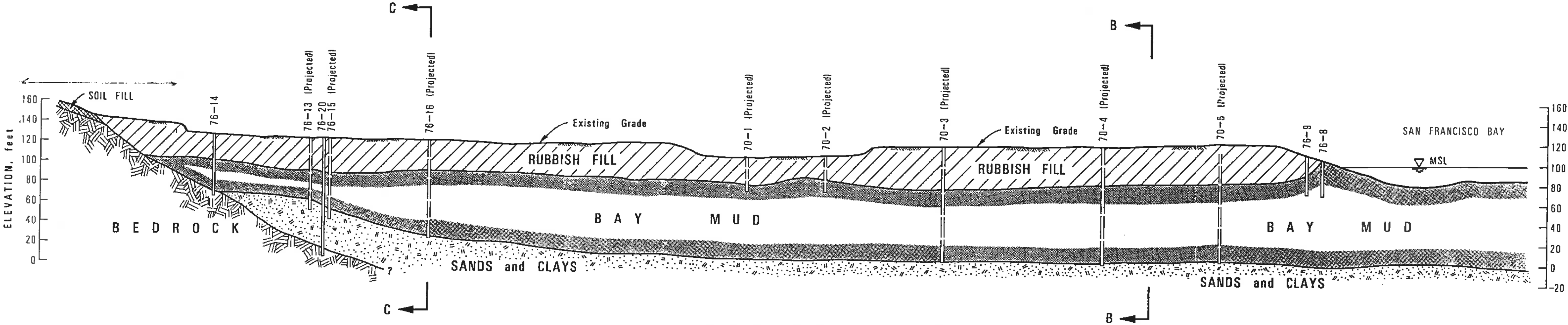
▨ ZONES IN WHICH LEACHATE SEEPS WERE OBSERVED OR ARE LIKELY TO OCCUR

0 100 200 300 400 500 600
Scale in Feet

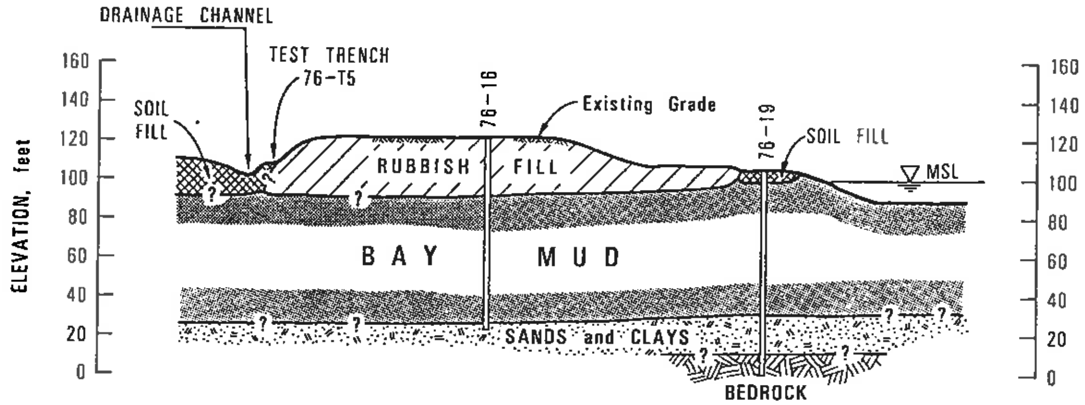
EXTENT OF RUBBISH AND BASE OF BAY MUD
OYSTER POINT MARINA
South San Francisco, California

Project No. 13609A
WOODWARD-CLYDE CONSULTANTS

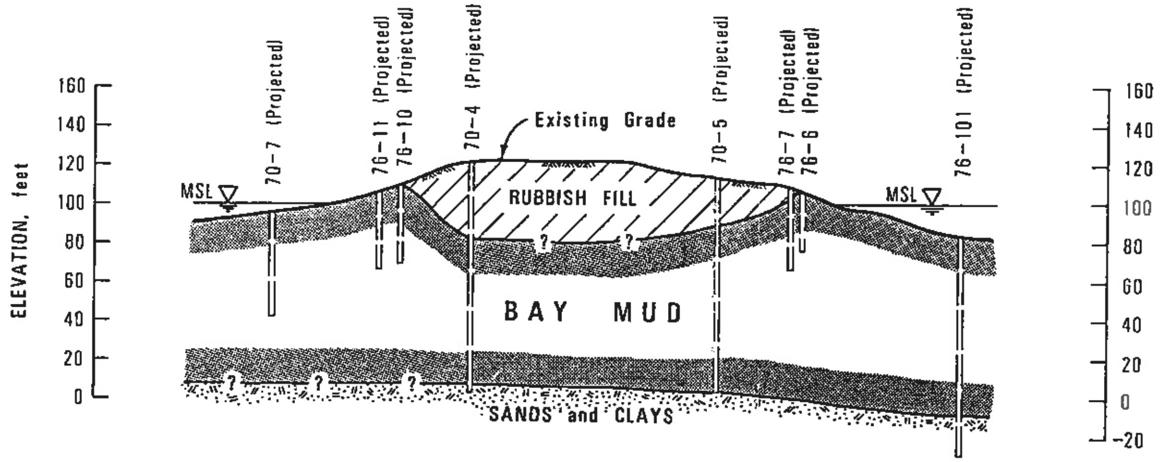
Figure 3



SECTION A - A



SECTION C - C



SECTION B - B

NOTES:

1) REFER TO FIGURE 1 FOR LOCATION AND ORIENTATION OF SECTIONS.

2) VERTICAL EXAGGERATION = 100 / 40 = 2.5x

3) THE RUBBISH FILL IS OVERLAIN BY A SOIL CAP OF VARYING THICKNESS, QUALITY AND COMPOSITION.

4) EXISTING SURFACE ELEVATIONS TAKEN FROM THE "HYDROGRAPHIC/TOPOGRAPHIC MAP OF OYSTER POINT MARINA" PREPARED BY TOWILL, INC. FOR BRIAN-KANGAS-FOULK & ASSOCIATES DATED MARCH 11, 1976.

5) THE IDEALIZED SOIL PROFILES ARE CONSTRUCTED BY DIRECT INTERPOLATION BETWEEN TEST BORINGS DRILLED AT VARYING SPACINGS. THE LINES DELINEATING THE VARIOUS SOIL AND ROCK TYPES WERE DONE FOR SCHEMATIC ILLUSTRATION PURPOSES ONLY. THE PROFILES SHOULD NOT BE CONSTRUED AS ACCURATE REPRESENTATIONS OF ACTUAL FIELD CONDITIONS.

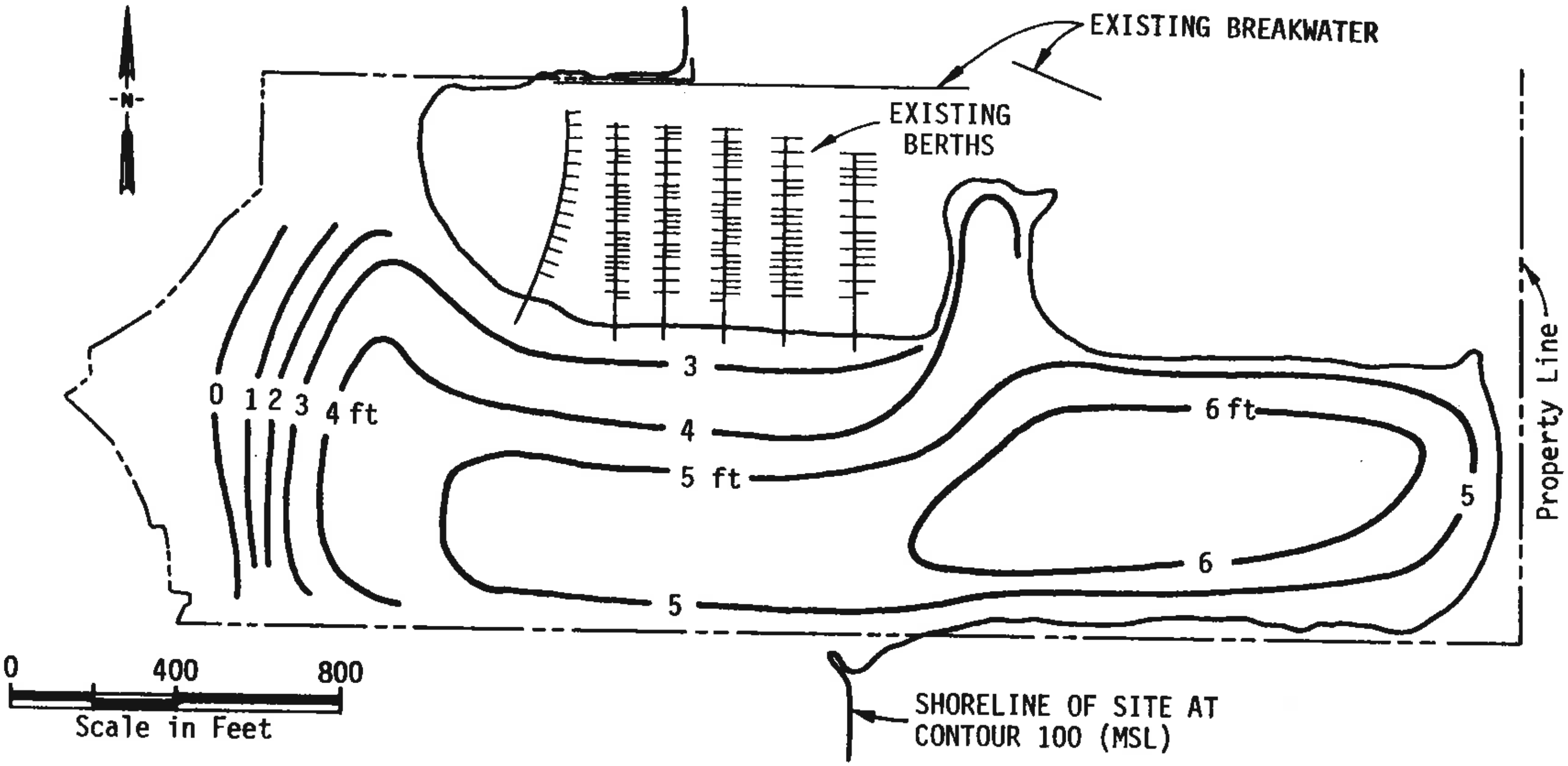
LEGEND:

▨	RUBBISH FILL POORLY COMPACTED DOMESTIC AND LIGHT INDUSTRIAL SOLID WASTE INCLUDING NEWSPAPERS, CARDBOARD, CANS, ETC.
▩	BAY MUD SOFT, DARK GRAY SILTY CLAY
░	SANDS and CLAYS MEDIUM DENSE TO DENSE SANDS AND SILTS AND MEDIUM STIFF TO HARD CLAYS
▧	BEDROCK SOFT TO MEDIUM HARD SHALE AND SANDSTONE
▦	SOIL FILL MODERATELY COMPACTED CLAYEY AND SILTY SANDS

IDEALIZED SOIL PROFILES
SECTIONS A-A, B-B, and C-C
OYSTER POINT MARINA
South San Francisco, California

Project No. 13609 A
WOODWARD-CLYDE CONSULTANTS

Figure 4



NOTE: This figure depicts zones of future settlement potential. Because of the uncertainties involved in the prediction of settlements in rubbish fills and partially-consolidated compressible clays, the values shown here may be in error by 20 percent or more. Also, if 4 feet of additional fill is placed at the site, post-construction settlements may be as much as 20 percent greater than the values shown.

Figure 5 - FUTURE SETTLEMENT POTENTIAL UNDER EXISTING CONDITIONS

OYSTER POINT MARINA
South San Francisco, California

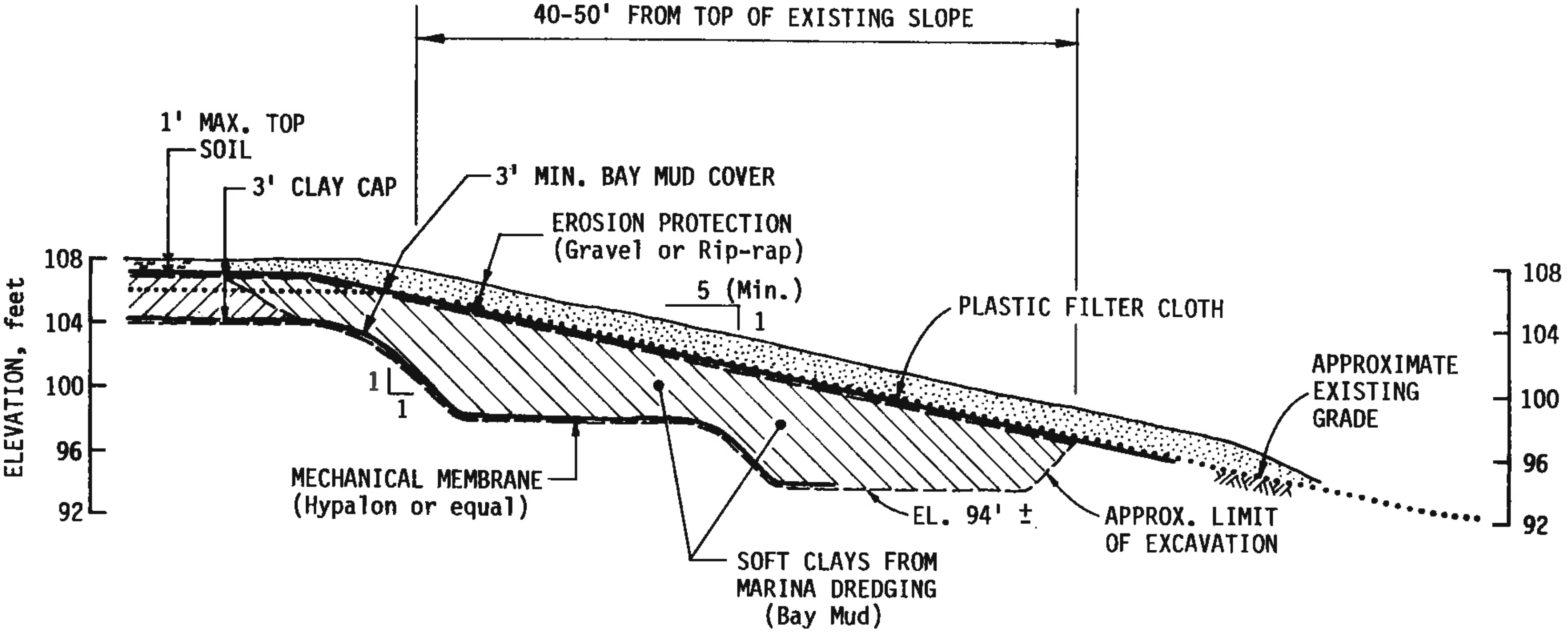


Figure 6 - RECOMMENDED DETAILS FOR LEACHATE AND EROSION CONTROL FOR MOLE AREA

OYSTER POINT MARINA

Approx. Scale : 1" = 10'

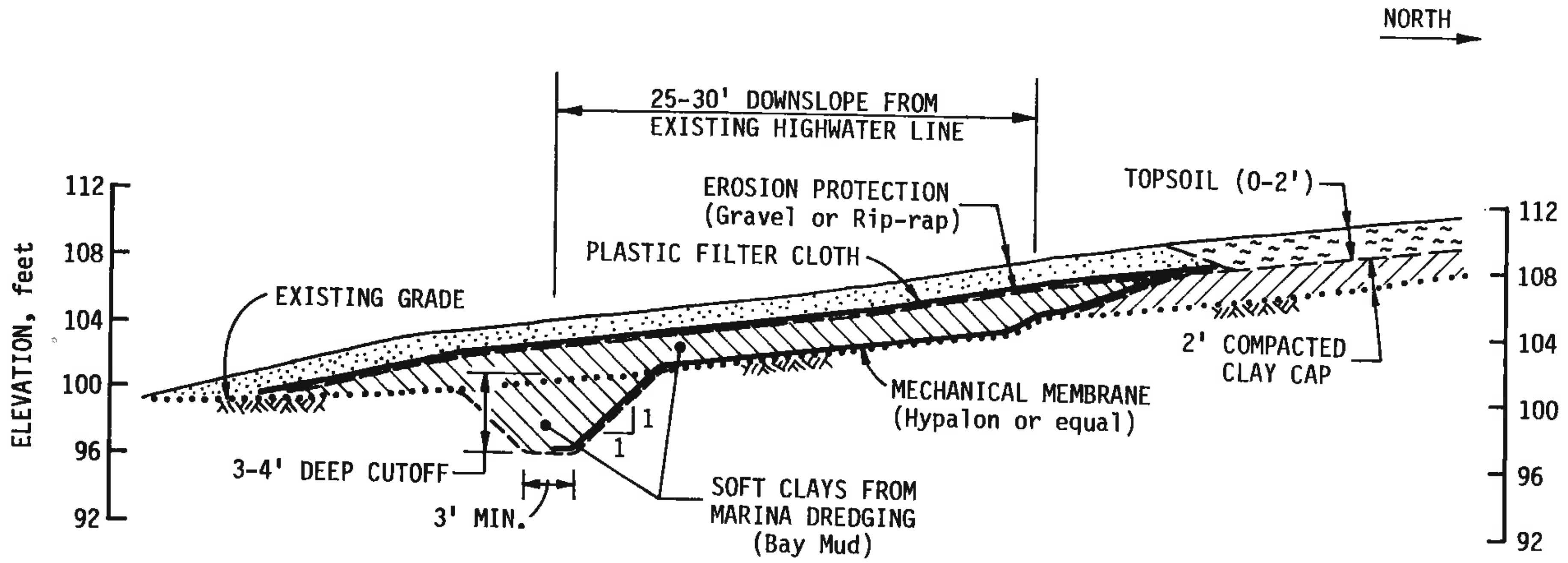
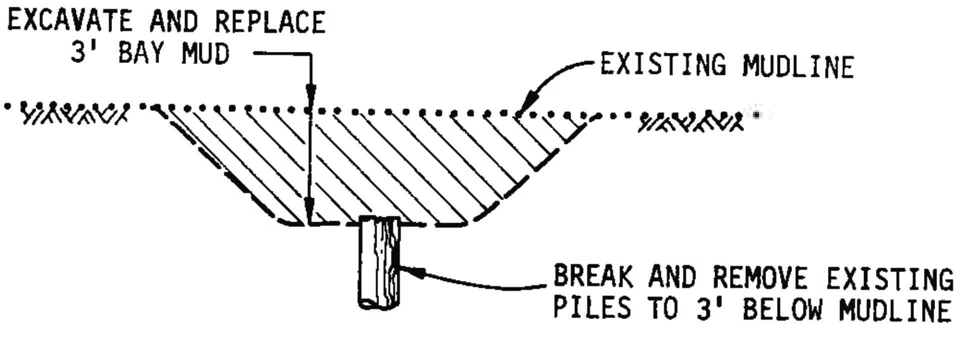
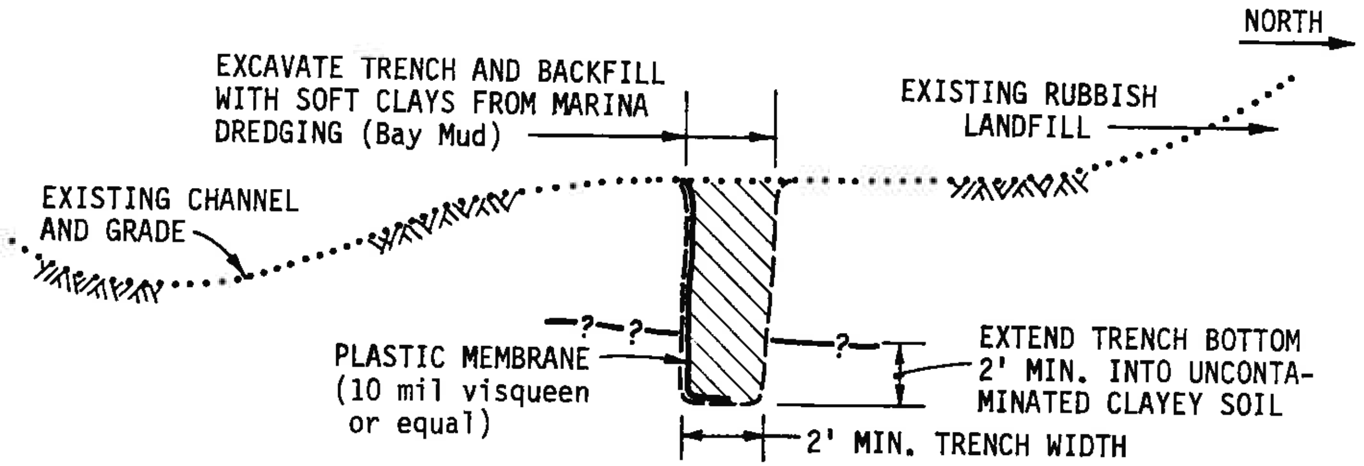


Figure 7 - RECOMMENDED LEACHATE AND EROSION CONTROL DETAILS FOR SOUTH SLOPE AND SOUTHEAST CORNER

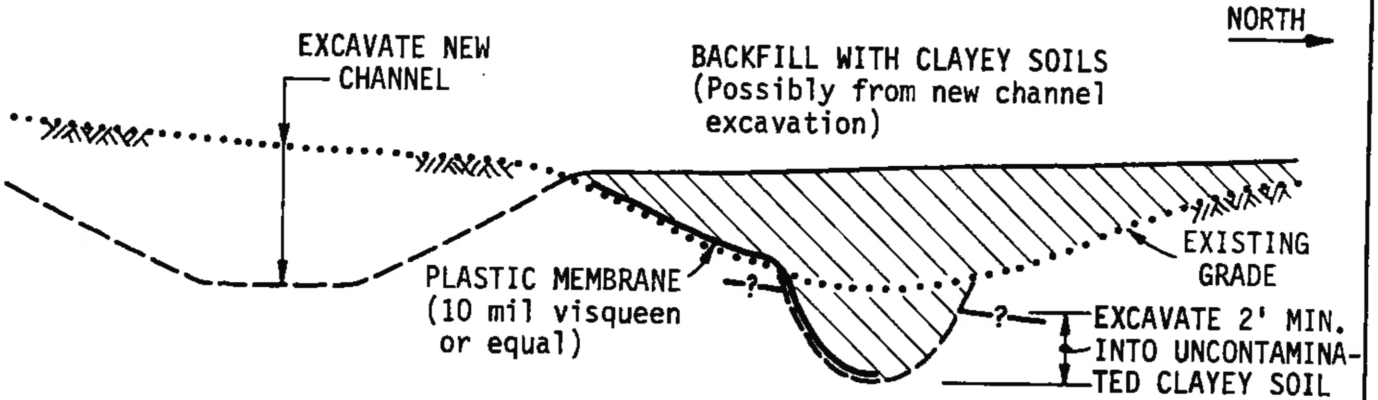
Approx. Scale : 1" = 10'



(a) Buried Pile Fence Area



(b) Western Two-Thirds of Drainage Channel



(c) Eastern Third of Drainage Channel

RECOMMENDED DETAILS FOR LEACHATE CONTROL

OYSTER POINT MARINA

Approx. Scale : 1" = 5'

Figure 8

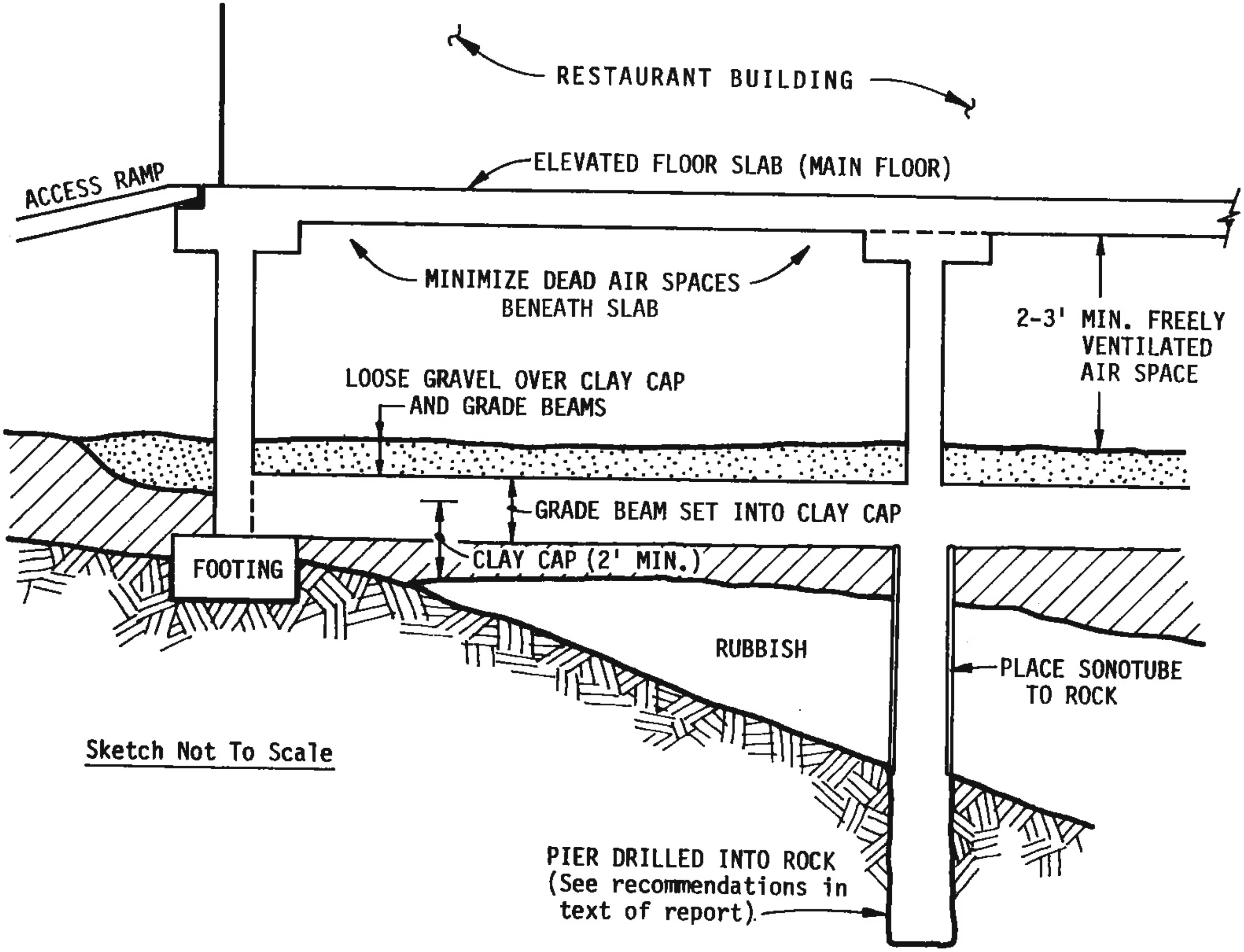


Figure 9 - TYPICAL FOUNDATION DETAILS FOR RESTAURANT ONE

OYSTER POINT MARINA

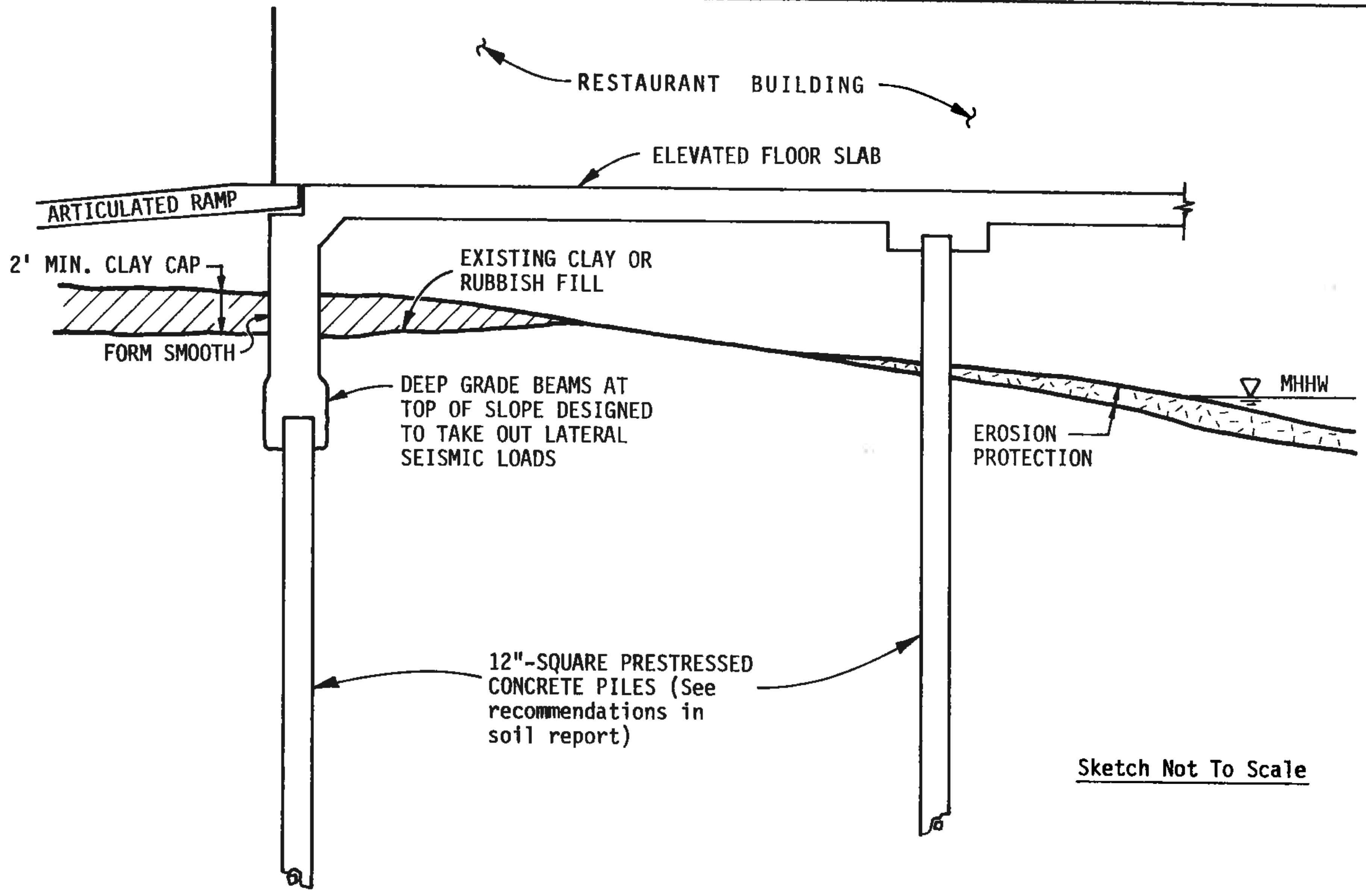
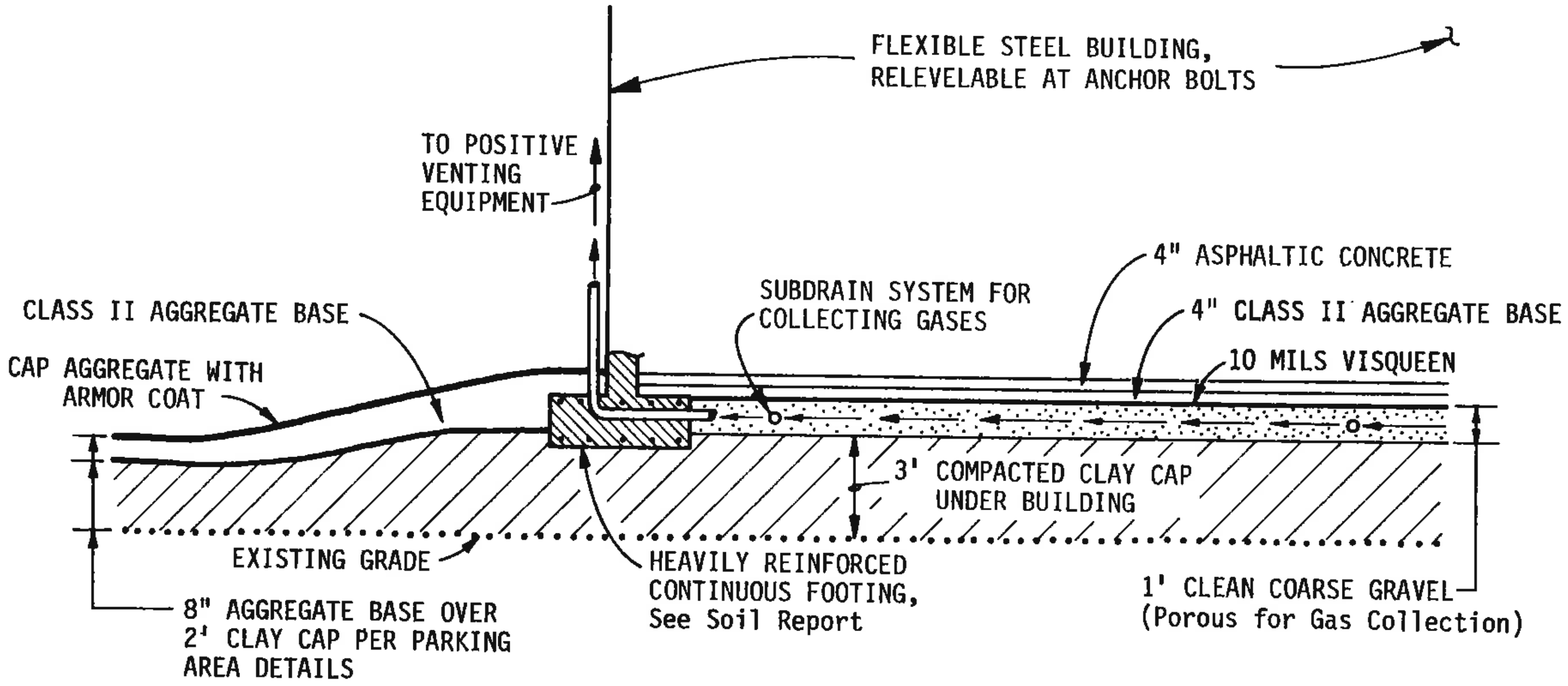


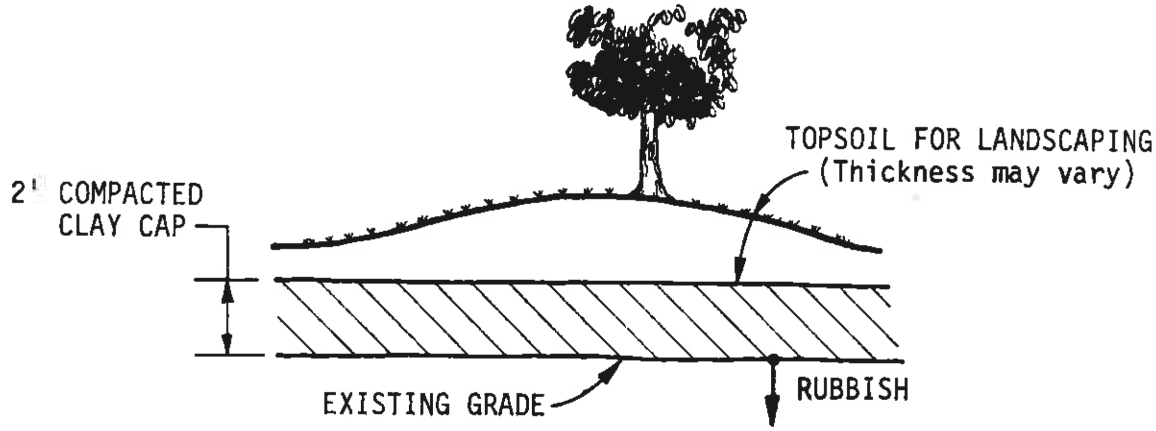
Figure 10 - TYPICAL FOUNDATION DETAILS FOR RESTAURANT TWO
OYSTER POINT MARINA



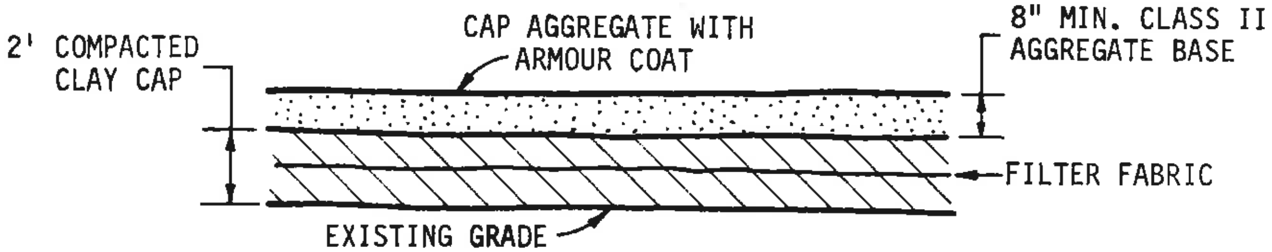
Sketch Not To Scale

Figure 12 - TYPICAL FOUNDATION DETAILS FOR BOAT REPAIR BUILDING

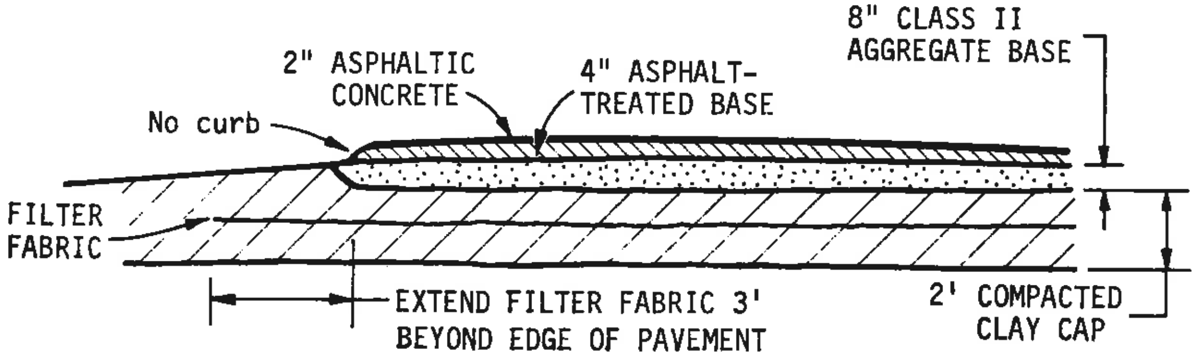
OYSTER POINT MARINA



(a) Typical Clay Cap Detail Over Existing Site Surface
Approx. Scale: 1" = 5'



(b) Typical Parking Area Detail
Approx. Scale: 1" = 5'

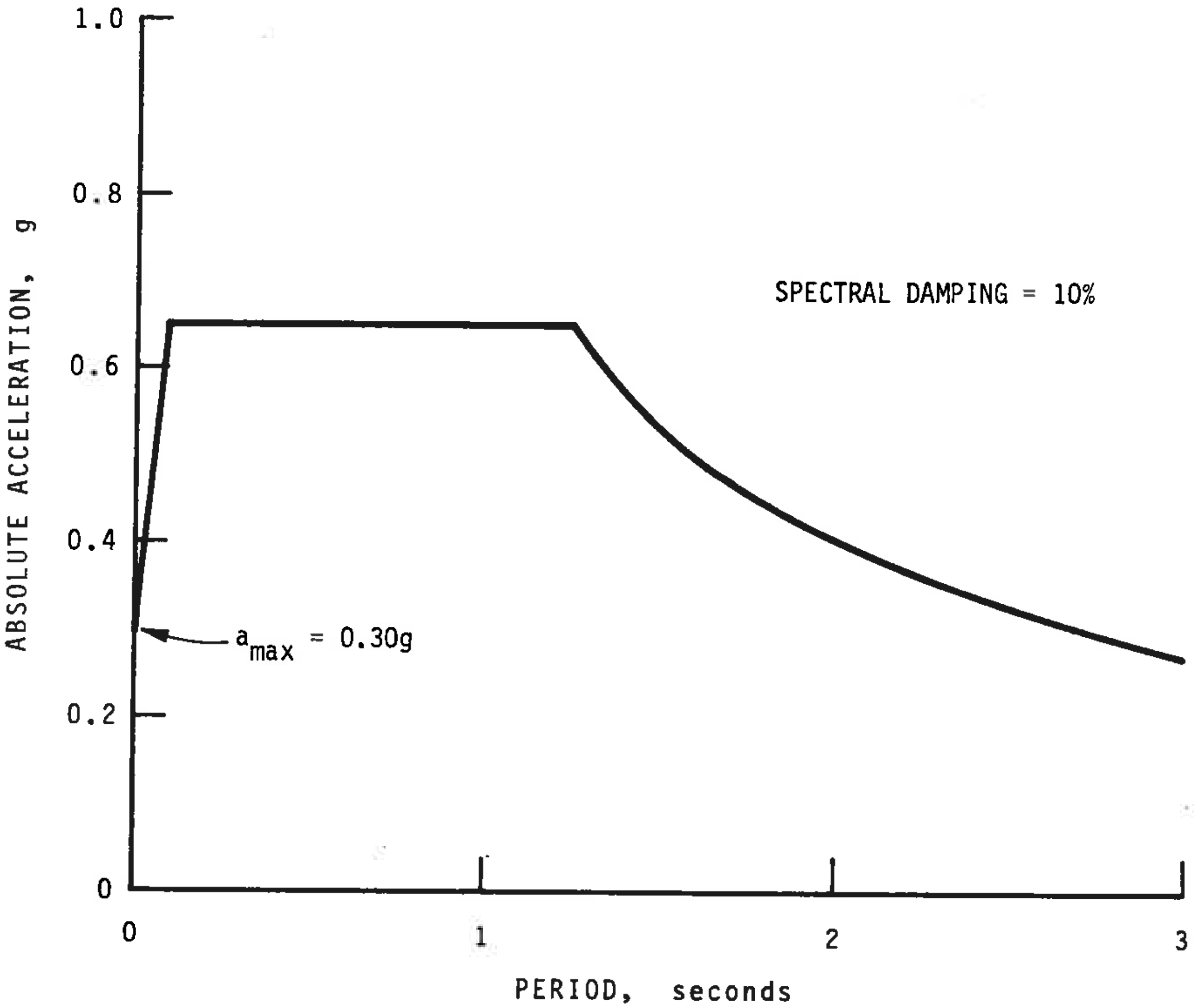


(c) Typical Roadway Pavement Section
Approx. Scale: 1" = 5'

CLAY CAP AND PAVEMENT DETAILS

OYSTER POINT MARINA

Figure 13



GROUND MOTION RESPONSE SPECTRUM FOR A TYPICAL SOIL PROFILE
OYSTER POINT MARINA

Figure 14

APPENDIX A

RESULTS OF FIELD AND LABORATORY INVESTIGATIONS

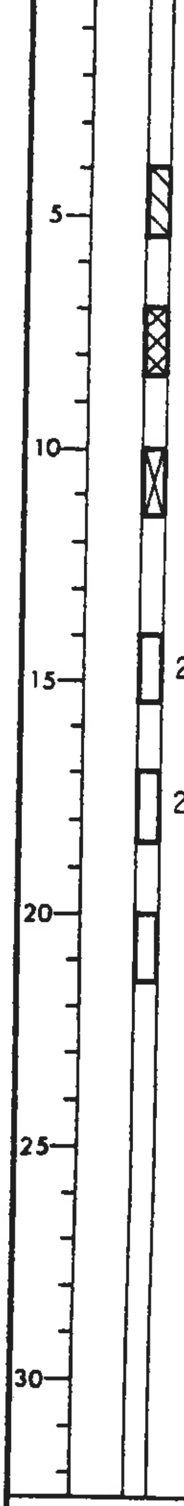
Table of Contents

Project: OYSTER POINT MARINA
South San Francisco, California

BORING LOG LEGEND SHEET

Date Drilled: ____________________ **Hammer Weight:** ____________________
Type of Boring: ____________________ **Remarks:** ____________________

Depth, Ft	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
			Surface Elevation:			



← 2-INCH I.D. MODIFIED CALIFORNIA SAMPLER

← 2½-INCH I.D. MODIFIED CALIFORNIA SAMPLER

← 3-INCH O.D. SHELBY TUBE SAMPLER

29 ← BLOW COUNT WITH A 140-LB. HAMMER FALLING 30 INCHES

29* ← BLOW COUNT WITH A DOWNHOLE HAMMER FALLING 12 INCHES THROUGH DRILLING FLUID

P ← SAMPLER ADVANCED BY PUSHING

▽ WATER LEVEL MEASURED:
ATD ← At Time of Drilling
6-4-76 ← On Date Indicated

Job No. 13609A

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Figure A-1

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-1

Date Drilled: June 4, 1976

Type of Boring: 6" Auger

Hammer Weight: 140 lbs.

Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp. Strength, psf
			Surface Elevation: 115			
			SANDY SILT FILL Poorly compacted, dry, dark brown			
	1	7		-	-	-
5			RUBBISH FILL Includes tin cans, paper, organic matter			
	2	7		-	-	-
10						
	3	5		-	-	-
15						
	4	30	(FILL)	-	-	-
20			SHALE Soft to medium hard, dark gray			
	5	70/4½"		-	-	-
25			Medium hard			
			BOTTOM OF HOLE @ 28' Water Table Not Encountered			
30						
35						
40						

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Figure A-2

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-2

Date Drilled: June 4, 1976

Type of Boring: 6" Auger

Hammer Weight: 140 lbs.

Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp Strength, psf
			Surface Elevation: 125			
			SANDY SILT FILL			
5			RUBBISH FILL			
10						
15						
	1 ▨	8		-	-	-
20						
			(FILL)			
			SHALE Medium hard, dark gray			
25			BOTTOM OF HOLE @ 25' Water Table Not Encountered			
30						
35						
40						

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Figure A-3

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-3

Date Drilled: June 15, 1976

Type of Boring: 6" Auger

Hammer Weight:

Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp. Strength, psf
			Surface Elevation: 130			
			SILTY SAND FILL			
5			RUBBISH FILL Mixed with silty sand in places			
10						
15			SILTY CLAY (CH-OH) Medium stiff, moist to wet, dark gray, mixed with rubbish			
20			(FILL)			
25			SHALE Medium hard, gray-brown			
30			BOTTOM OF HOLE @ 27' Water Table Not Encountered			
35						
40						

Job No. 13609A

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Figure A-4

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-4

Date Drilled: June 4, 1976
Hammer Weight: 140 lbs.
Type of Boring: 6" Auger
Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp. Strength, psf
			Surface Elevation: 125			
			SANDY SILT FILL Poorly compacted, dry, gray			
	1 ▨	47/9"	SHALE Medium hard, gray	-	-	-
5						
10						
			BOTTOM OF HOLE @ 13' Water Table Not Encountered			
15						

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-5

Date Drilled: June 15, 1976
Hammer Weight:
Type of Boring: 6" Auger
Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp. Strength, psf
			Surface Elevation: 133			
			SILTY SAND FILL : Poorly compacted, dry to damp, dark gray, with gravel			
			With rubbish			
			SHALE: Medium hard to hard, gray			
5			BOTTOM OF HOLE @ 5.5' Water Table Not Encountered			
10						

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-6

Date Drilled: April 9, 1976

Type of Boring: 6" Auger

Hammer Weight:

Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp Strength, psf
			Surface Elevation: 107			
			CLAYEY GRAVEL FILL: Poorly compacted, contains large rock fragments			
5			SILTY CLAY FILL Poorly compacted, wet, dark gray, with trace of rock fragments ▽ 5-28-76 } Large rock fragments with clay (FILL)			
10 15 20 25			SILTY CLAY (CH) Soft, saturated, dark gray (BAY MUD)			
30 35 40			BOTTOM OF HOLE @ 30'			

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Figure A-6

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-7

Date Drilled: April 9, 1976
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:

Surface Elevation: 111

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp Strength, psf
			SILTY CLAY FILL: Moderately compacted, moist, dark gray, with trace of rock fragments			
	1	7	RUBBISH FILL	-	-	-
5						
	2	11/9"	← Metal spring	-	-	-
			▽ 5-28-76			
10						
	3	6		-	-	-
			SANDY CLAY (CL-CH) Medium stiff, saturated, gray, with rock fragments (FILL)			
15						
	4	2	SILTY CLAY (CH) Soft, saturated, dark gray	No Recovery		
20						
	5	2		-	-	-
25						
	6	2		No Recovery		
30			(BAY MUD)			
	7	2		-	-	-
35						
40						
	8	6		-	-	-
			BOTTOM OF HOLE @ 42'			

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Figure A-7

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-8

Date Drilled: April 8, 1976

Type of Boring: 6" Auger

Hammer Weight: 140 lbs.

Remarks:

Surface Elevation: 107

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp. Strength, psf
	1	9	SILTY CLAY FILL Poorly compacted, moist, gray, with rock fragments	-	-	-
5						
	2	5	▽ 5-28-76 Very poorly compacted, saturated, dark gray	No Recovery		
10						
			With large rock fragments			
	3	4		No Recovery		
15						
	4	P		-	-	-
20			SILTY CLAY (CH) Soft, saturated, dark gray, with trace of fine shells			
25						
			(BAY MUD)			
30						
35			BOTTOM OF HOLE @ 35'			
40						

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Figure A-8

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-9

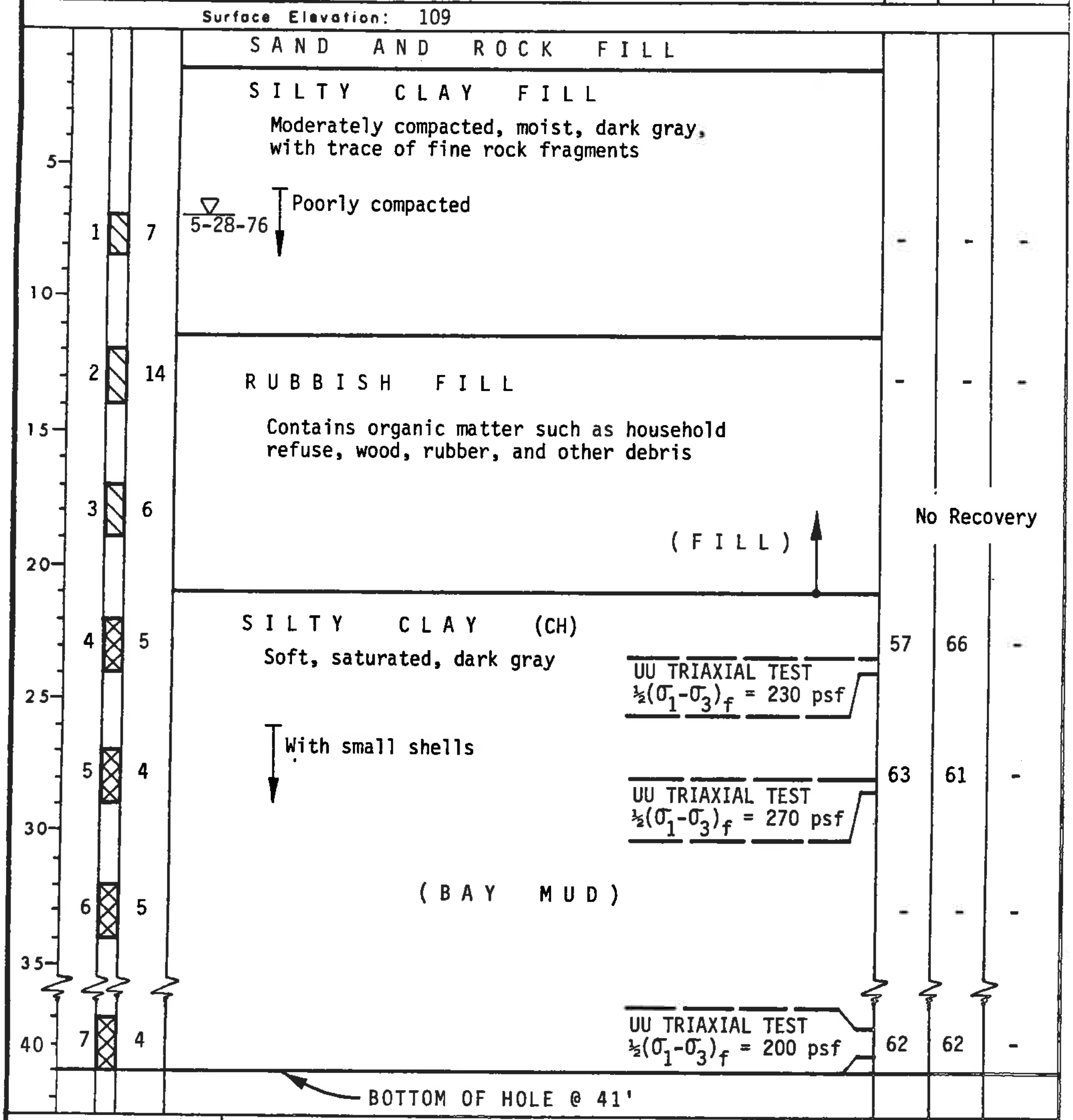
Date Drilled: April 8, 1976

Hammer Weight: 140 lbs.

Type of Boring: 6" Auger

Remarks:

Surface Elevation: 109



SAND AND ROCK FILL

SILTY CLAY FILL

Moderately compacted, moist, dark gray, with trace of fine rock fragments

Poorly compacted

RUBBISH FILL

Contains organic matter such as household refuse, wood, rubber, and other debris

(FILL)

SILTY CLAY (CH)

Soft, saturated, dark gray

With small shells

(BAY MUD)

UU TRIAXIAL TEST
$\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 230 psf

UU TRIAXIAL TEST
$\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 270 psf

UU TRIAXIAL TEST
$\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 200 psf

BOTTOM OF HOLE @ 41'

Samples	Blows/Ft.	Moisture Content, %	Dry Density pcf	Unc. Comp Strength, psf
1	7	-	-	-
2	14	-	-	-
3	6	No Recovery		
4	5	57	66	-
5	4	63	61	-
6	5	-	-	-
7	4	62	62	-

Job No. 13609A

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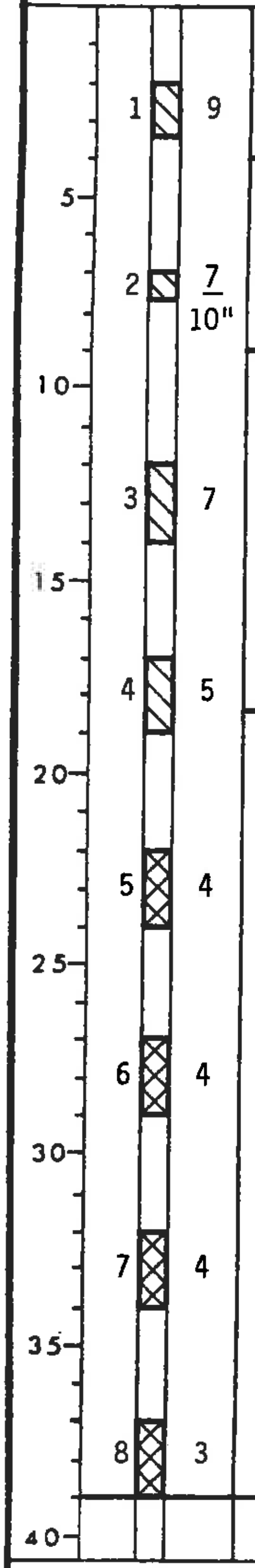
Figure A-9

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No.76-10

Date Drilled: April 9, 1976
Hammer Weight: 140 lbs.
Type of Boring: 6" Auger
Remarks:

Surface Elevation: 110



Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp. Strength, psf
1	9	SILTY CLAY FILL Moderately compacted, moist, with fine rock fragments	-	-	-
2	7/10"	SILTY CLAY FILL Poorly compacted, wet, dark gray ▽ 5-8-76 With rock fragments	-	-	-
3	7	CLAYEY SILT FILL Poorly compacted, saturated, dark gray, with some fine rock fragments and trace of refuse	-	-	-
4	5	(FILL)	-	-	-
5	4	SILTY CLAY (CH) Soft, saturated, dark gray UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 190 psf	69	59	-
6	4	UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 240 psf	62	62	-
7	4		-	-	-
8	3	UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 80 psf	-	-	-

BOTTOM OF HOLE @ 39'

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Figure A-10

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No.76-11

Date Drilled: April 9, 1976
Hammer Weight:
Type of Boring: 6" Auger
Remarks:

Surface Elevation: 105

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp Strength, psf
			SILTY CLAY FILL Poorly compacted, moist, dark brown, with rock fragments			
			▽ 5-8-76			
5						
			} Large rock fragments			
10			SILTY CLAY FILL Poorly compacted, saturated, dark gray, with trace of rock fragments and rubbish			
15						
20			(FILL) ↑			
25			SILTY CLAY (CH) Soft, saturated, dark gray			
30						
			(BAY MUD)			
40			BOTTOM OF HOLE @ 40'			

Job No. 13609A

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Figure A-11

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No.76-12

Date Drilled: April 12, 1976

Hammer Weight: 140 lbs.

Type of Boring: 6" Auger

Remarks:

Surface Elevation: 107

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unc. Comp Strength, psf
			SILTY CLAY FILL Moderately compacted, moist, brown			
	1	10	SILTY CLAY FILL Poorly compacted, wet, dark gray, with large rock fragments	-	-	-
5			▽ ATD			
			} Boulder			
	2	13	With rubbish ↓	-	-	-
10						
	3	8		-	-	-
15						
	4	13/8"	(FILL) ↑	-	-	-
20						
	5	2		-	-	-
25			SILTY CLAY (CH) Very soft to soft, saturated, dark gray, with trace of shells			
	6	3		-	-	-
30						
			(BAY MUD)			
	7	2		-	-	-
35						
40	8	4		-	-	-

BOTTOM OF HOLE @ 41'

Job No. 13609A

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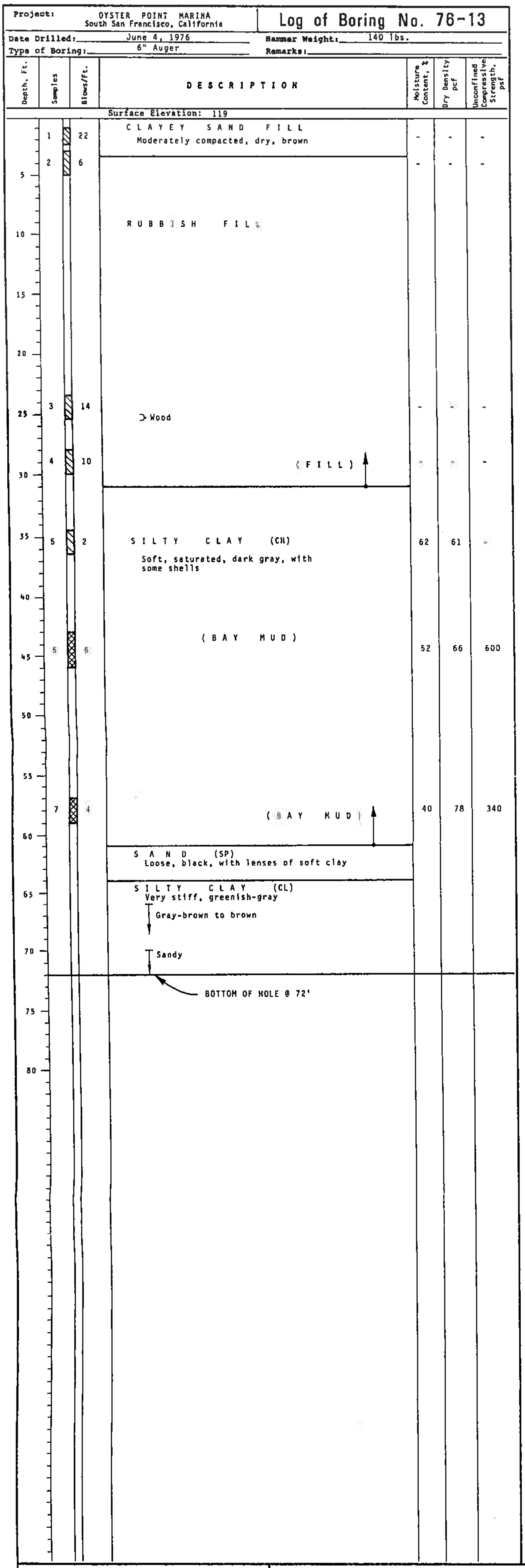
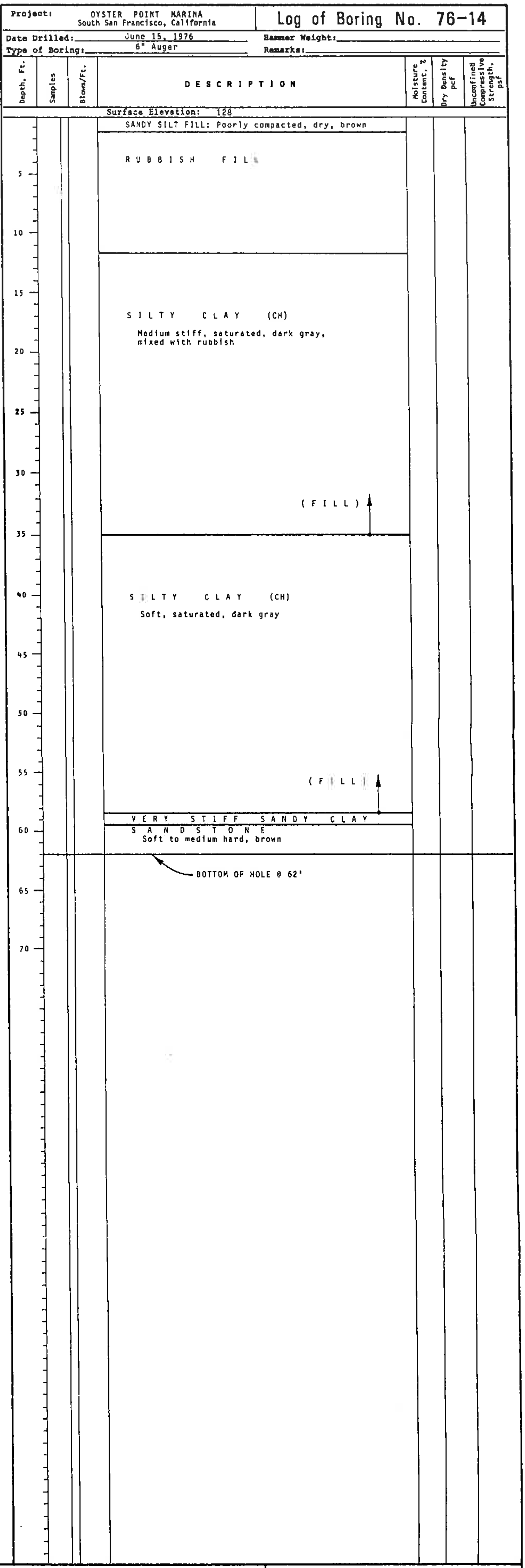
Figure A-12



Project: OYSTER POINT MARINA, South San Francisco, California

Log of Boring No. 76-13

Date Drilled: June 4, 1976
Hammer Weight: 140 lbs.
Type of Boring: 6" Auger
Remarks:



Project: OYSTER POINT MARINA, South San Francisco, California

Log of Boring No. 76-14

Date Drilled: June 15, 1976
Hammer Weight:
Type of Boring: 6" Auger
Remarks:

Proj. No. 13609A

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Figure A-13

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-15

Date Drilled: June 4, 1976
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:

Surface Elevation: 123

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
	1	18	SILTY CLAY FILL: Well compacted, dry, brown	-	-	-
	2	17/8"	SANDY CLAY FILL Poorly compacted, mixed with rubbish	-	-	-
5 10 15 20			RUBBISH FILL Rubbish mixed with soft clay			
25 30	3	5	(FILL)	-	-	-
35 40 45	4	4	SILTY CLAY (CH) Soft, saturated, gray, with trace of shells	53	69	-
50 55	5	3		No Recovery		
60 65	6	4	Medium stiff (BAY MUD) Grades with depth to clayey sand	49	69	740
70	7	37	CLAYEY SAND (SC) Medium dense, saturated, green	-	-	-
75 80	8	34	FINE SANDY CLAY (CL) Very stiff, saturated, brown	15	118	8400
85			BOTTOM OF HOLE @ 80'			

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-16

Date Drilled: June 5, 1976
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:

Surface Elevation: 124

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
	1	14	CLAYEY SAND FILL Poorly compacted, dry, brown, with rock fragments	-	-	-
5 10 15 20 25 30			RUBBISH FILL (FILL)			
35 40 45 50 55 60	2	2	SILTY CLAY (CH) Very soft, saturated, dark gray	64	61	-
65 70 75 80 85 90 95	3	4	Soft to medium stiff (BAY MUD)	No Recovery		
	4	P	SILTY CLAY (CL): Stiff, green	-	-	-
100 105			BOTTOM OF HOLE @ 99'			

Proj. No. 13609A

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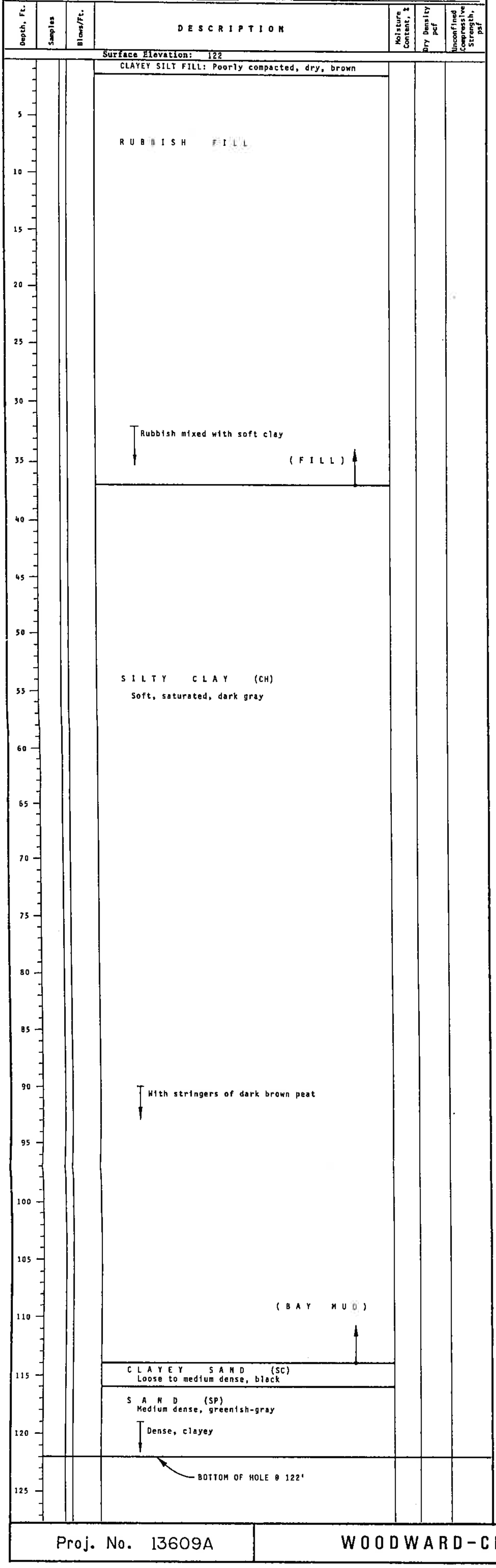
Figure A-14

Log of Boring No. 76-17

Project: OYSTER POINT MARINA
South San Francisco, California
Date Drilled: June 15, 1976
Type of Boring: 6" Auger
Hammer Weight:
Remarks:



Surface Elevation: 122
CLAYEY SILT FILL: Poorly compacted, dry, brown

RUBBISH FILL

Rubbish mixed with soft clay

(FILL)

SILTY CLAY (CH)
Soft, saturated, dark gray

With stringers of dark brown peat

(BAY MUD)

CLAYEY SAND (SC)
Loose to medium dense, black

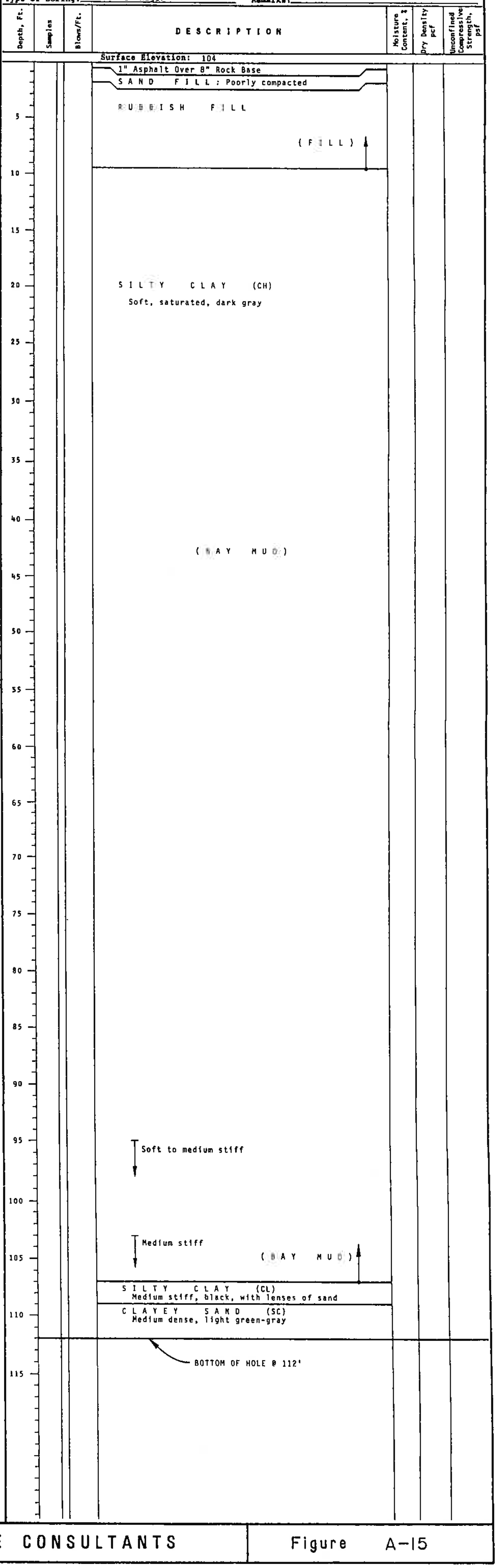
SAND (SP)
Medium dense, greenish-gray

Dense, clayey

BOTTOM OF HOLE @ 122'

Log of Boring No. 76-18

Project: OYSTER POINT MARINA
South San Francisco, California
Date Drilled: June 16, 1976
Type of Boring: 6" Auger
Hammer Weight:
Remarks:



Surface Elevation: 104
1" Asphalt Over 8" Rock Base
SAND FILL: Poorly compacted

RUBBISH FILL

(FILL)

SILTY CLAY (CH)
Soft, saturated, dark gray

(BAY MUD)

Soft to medium stiff

Medium stiff

SILTY CLAY (CL)
Medium stiff, black, with lenses of sand

CLAYEY SAND (SC)
Medium dense, light green-gray

BOTTOM OF HOLE @ 112'

Proj. No. 13609A | WOODWARD-CLYDE CONSULTANTS | Figure A-15

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-19

Date Drilled: June 10, 1976
Type of Boring: 6" Auger
Hammer Weight: 140 lbs.
Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 105			
			2" Asphalt Over 5" Rock Base			
	1	21	C L A Y E Y S A N D F I L L Moderately compacted, moist, dark gray, with rock fragments and some rubbish	-	-	-
	2	12		-	-	-
5			(F I L L) }Soft clay and rubbish			
	3	1		69	58	-
10						
15	4	2*	S I L T Y C L A Y (CH) Very soft to soft, saturated, dark gray	58	64	200
20						
25						
30	5	2		59	61	500
35			(B A Y M U D)			
40						
	6	2*	LL = 67%, PI = 40%	60	60	380
45						
50						
55						
	7	4*		52	64	570
60						
65						
70			Medium stiff (B A Y M U D)			
	8	18*		21	106	-
75			S I L T Y C L A Y (CH) Stiff, saturated, green UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)$ = 800 psf			
	9	32		19	111	6870
80			S I L T Y C L A Y (CL) Very stiff, saturated, brown			
	10	19	Grades with depth to Fine Sandy Clay UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 1350 psf	23	103	-
85						
			S A N D (SP) Medium dense, saturated, brown			
	11	32	With lenses of silty sand and sandy silt	-	-	-
90						
	12	68	Grades with depth to sandy silt (ML)	-	-	-
95			S H A L E Soft, saturated, dark gray Soft to medium hard			
100			Medium hard Medium hard to hard			
105			BOTTOM OF HOLE @ 105'			
110						

Project: OYSTER POINT MARINA
South San Francisco, California

Log of Boring No. 76-20

Date Drilled: June 11, 1976
Type of Boring: 4-3/4" Rotary
Hammer Weight: 140 lbs.
Remarks:

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 123			
	1	43	C L A Y E Y S A N D F I L L Moderately compacted, dry, brown	-	-	-
	2	16/9"		-	-	-
5						
10			R U B B I S H F I L L			
15						
20						
25						
30			Rubbish fill mixed with soft clay and wood (F I L L)			
35						
	3	6		-	-	-
40			S I L T Y C L A Y (CH) Soft, saturated, dark gray			
45	4	1		No Recovery		
50						
	5	3	LL = 72%, PI = 46%	54	64	590
55						
60	6	6		51	66	470
65			(B A Y M U D)			
			F I N E C L A Y E Y S A N D (SC)			
	7	29	S I L T Y C L A Y (CL) Very stiff, saturated, green	22	104	5390
70						
	8	78	S A N D (SP) Very dense, saturated, dark green-gray	-	-	-
75						
	9	41	F I N E S A N D Y C L A Y (CL) Very stiff, saturated, brown UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 1910 psf	16	119	-
80						
	10	80	UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)$ = 4830 psf	16	117	-
85			S I L T Y C L A Y (CL) Hard, saturated, gray-brown			
	11	80		15	118	11,600
90			Very stiff, more plastic (CL-CH) with some sand			
	12	40	UU TRIAXIAL TEST $\frac{1}{2}(\sigma_1-\sigma_3)_f$ = 2330 psf	18	113	-
95						
100			S I L T Y S A N D (SM) Dense, saturated, brown			
			S A N D Y C L A Y (CL-SC) Very stiff, saturated, mottled gray-brown, with rock fragments, probably highly weathered bedrock			
105	13	73/9"		-	-	-
110			S A N D S T O N E Hard, damp, reddish-brown, with claystone inclusions			
115						
	14	90/6"	BOTTOM OF HOLE @ 117'	-	-	-
120						

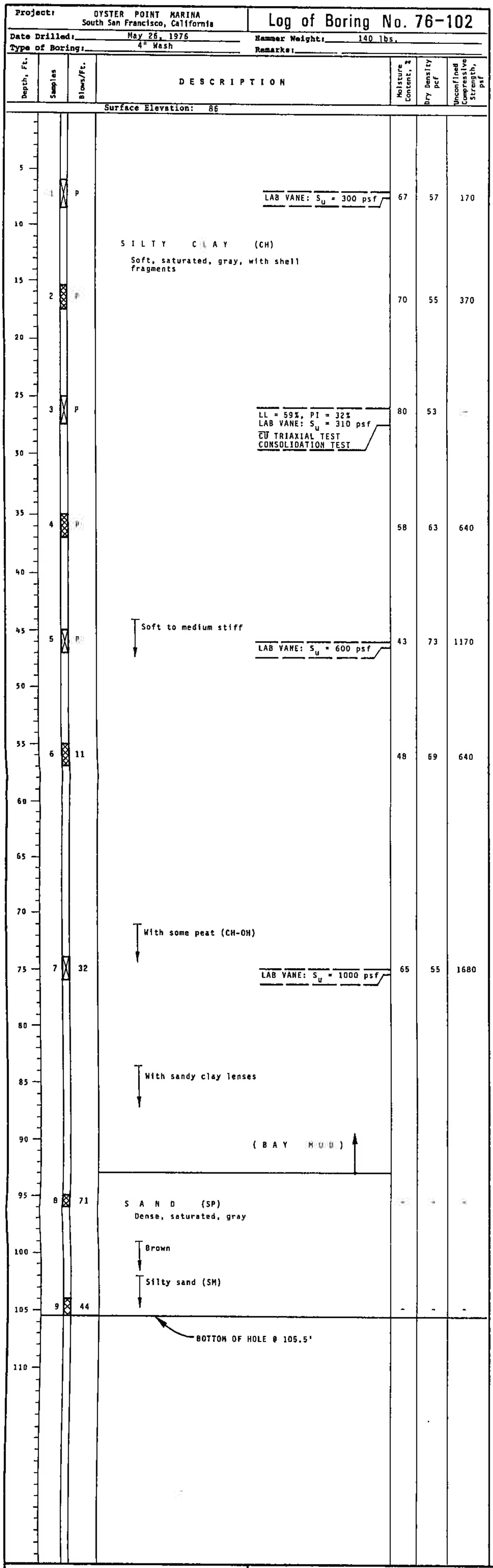
Proj. No. 13609A

WOODWARD-CLYDE CONSULTANTS

Figure A-16

Log of Boring No. 76-102

Project: OYSTER POINT MARINA, South San Francisco, California
Date Drilled: May 26, 1976
Type of Boring: 4" Wash
Hammer Weight: 140 lbs.
Remarks:
Surface Elevation: 86



SILTY CLAY (CH)
Soft, saturated, gray, with shell fragments

Soft to medium stiff

With some peat (CH-OH)

With sandy clay lenses

(BAY MUD)

SAND (SP)
Dense, saturated, gray

Brown

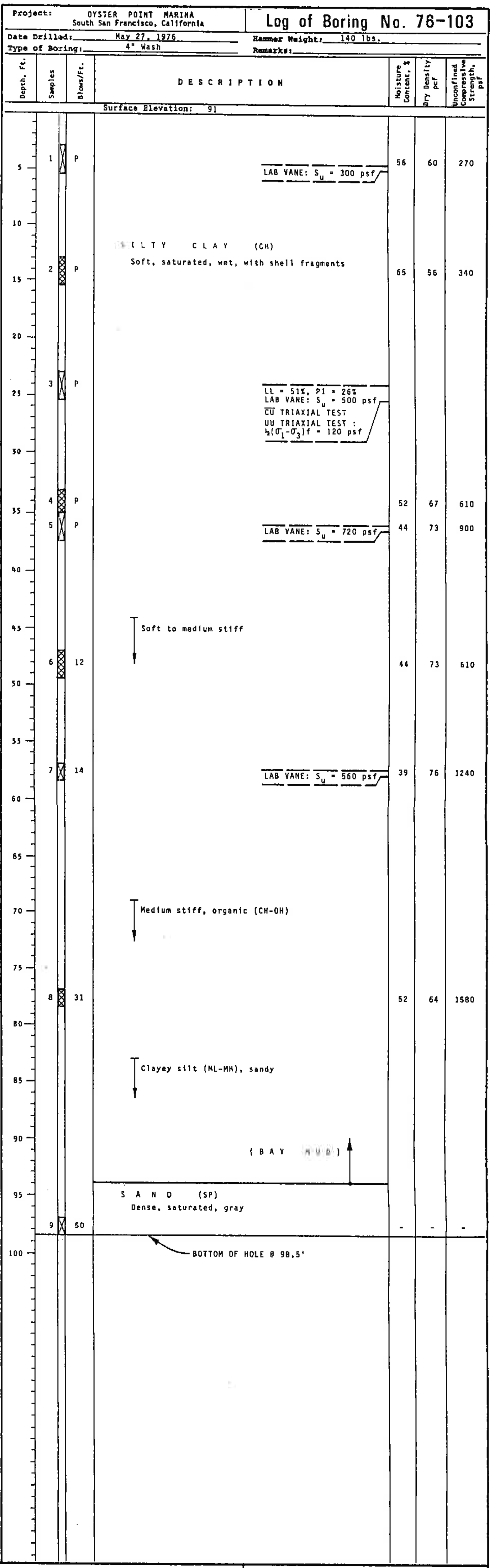
Silty sand (SM)

BOTTOM OF HOLE @ 105.5'

Samples	Blows/Ft.	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf	Remarks
1	P	67	57	170	LAB VANE: S_u = 300 psf
2	P	70	55	370	
3	P	80	53	-	LL = 59%, PI = 32% LAB VANE: S_u = 310 psf $\overline{CU}$ TRIAXIAL TEST CONSOLIDATION TEST
4	P	58	63	640	
5	P	43	73	1170	LAB VANE: S_u = 600 psf
6	11	48	69	640	
7	32	65	55	1680	LAB VANE: S_u = 1000 psf
8	71	-	-	-	
9	44	-	-	-	

Log of Boring No. 76-103

Project: OYSTER POINT MARINA, South San Francisco, California
Date Drilled: May 27, 1976
Type of Boring: 4" Wash
Hammer Weight: 140 lbs.
Remarks:
Surface Elevation: 91



SILTY CLAY (CH)
Soft, saturated, wet, with shell fragments

Soft to medium stiff

Medium stiff, organic (CH-OH)

Clayey silt (ML-MH), sandy

(BAY MUD)

SAND (SP)
Dense, saturated, gray

BOTTOM OF HOLE @ 98.5'

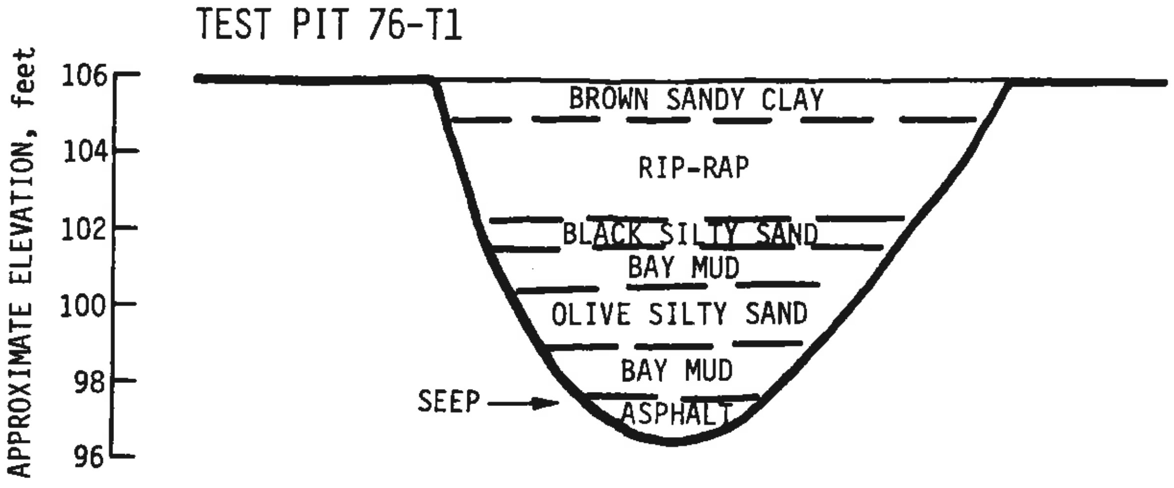
Samples	Blows/Ft.	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf	Remarks
1	P	56	60	270	LAB VANE: S_u = 300 psf
2	P	65	56	340	
3	P				LL = 51%, PI = 26% LAB VANE: S_u = 500 psf $\overline{CU}$ TRIAXIAL TEST UU TRIAXIAL TEST : $\frac{1}{2}(\sigma_1-\sigma_3)f$ = 120 psf
4	P	52	67	610	
5	P	44	73	900	LAB VANE: S_u = 720 psf
6	12	44	73	610	
7	14	39	76	1240	LAB VANE: S_u = 560 psf
8	31	52	64	1580	
9	50	-	-	-	

Proj. No. 13609A

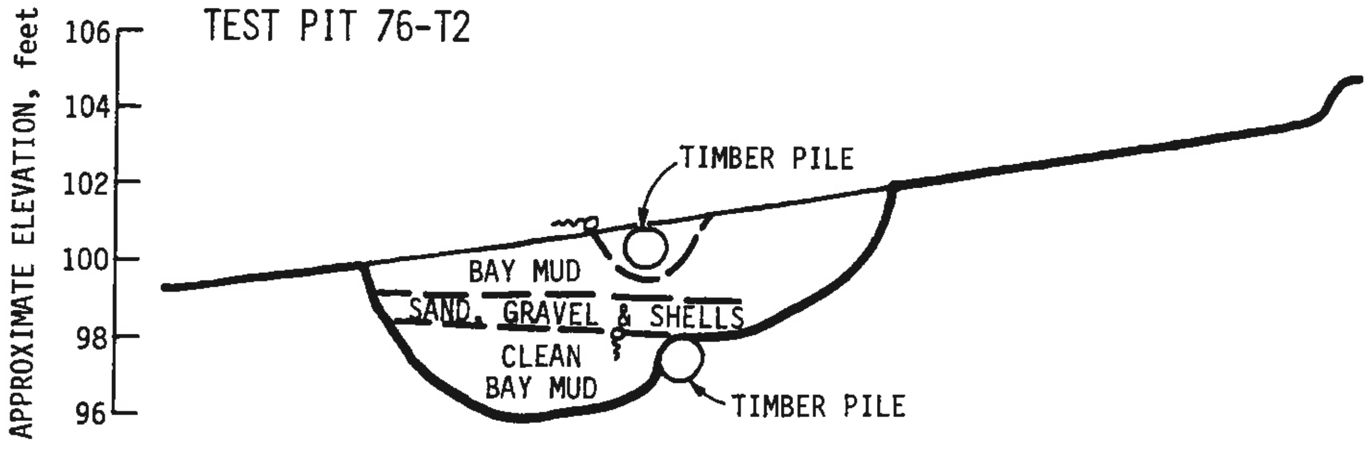
WOODWARD-CLYDE CONSULTANTS

Figure A-19

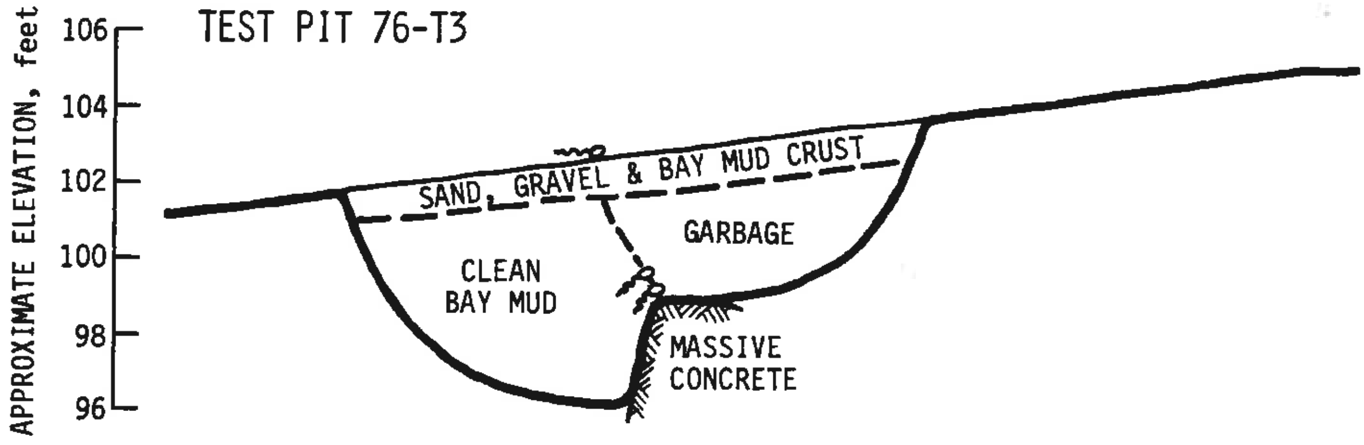
TEST PIT 76-T1



TEST PIT 76-T2



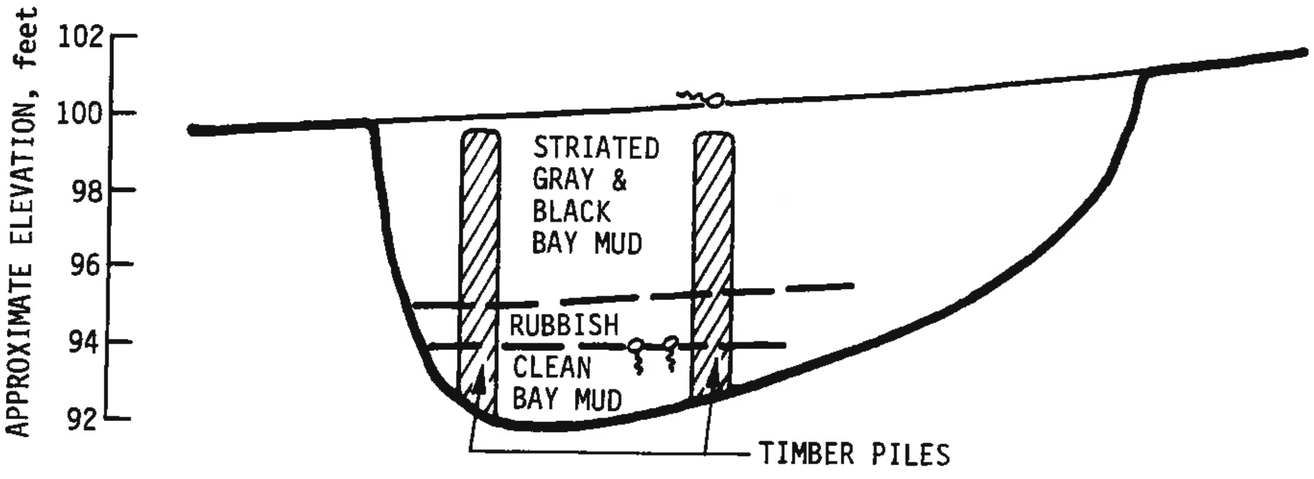
TEST PIT 76-T3



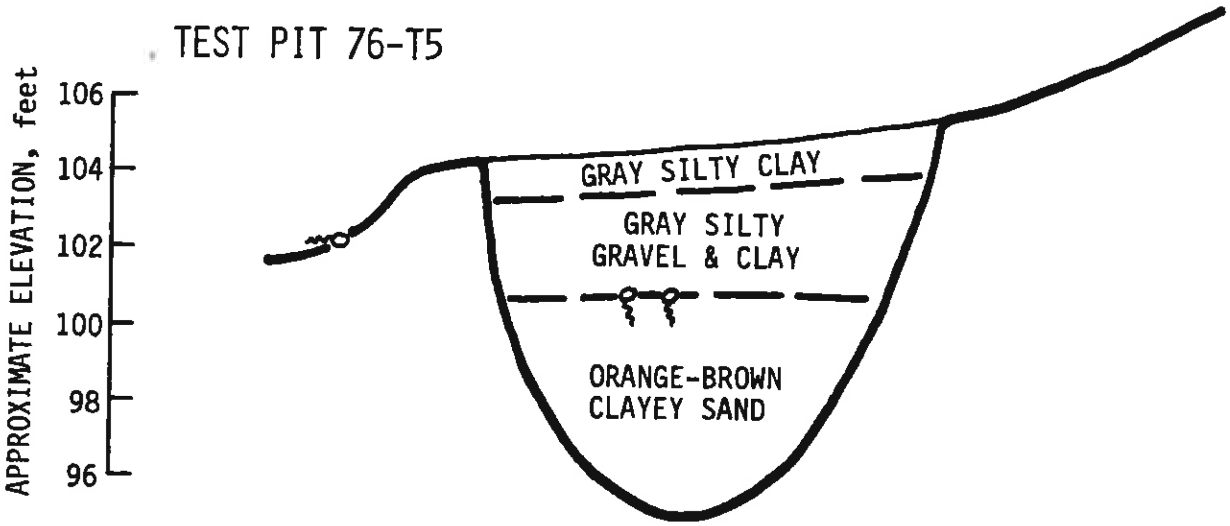
TEST PITS 76-T1, 76-T2, 76-T3

OYSTER POINT MARINA
South San Francisco, California

TEST PIT 76-T4



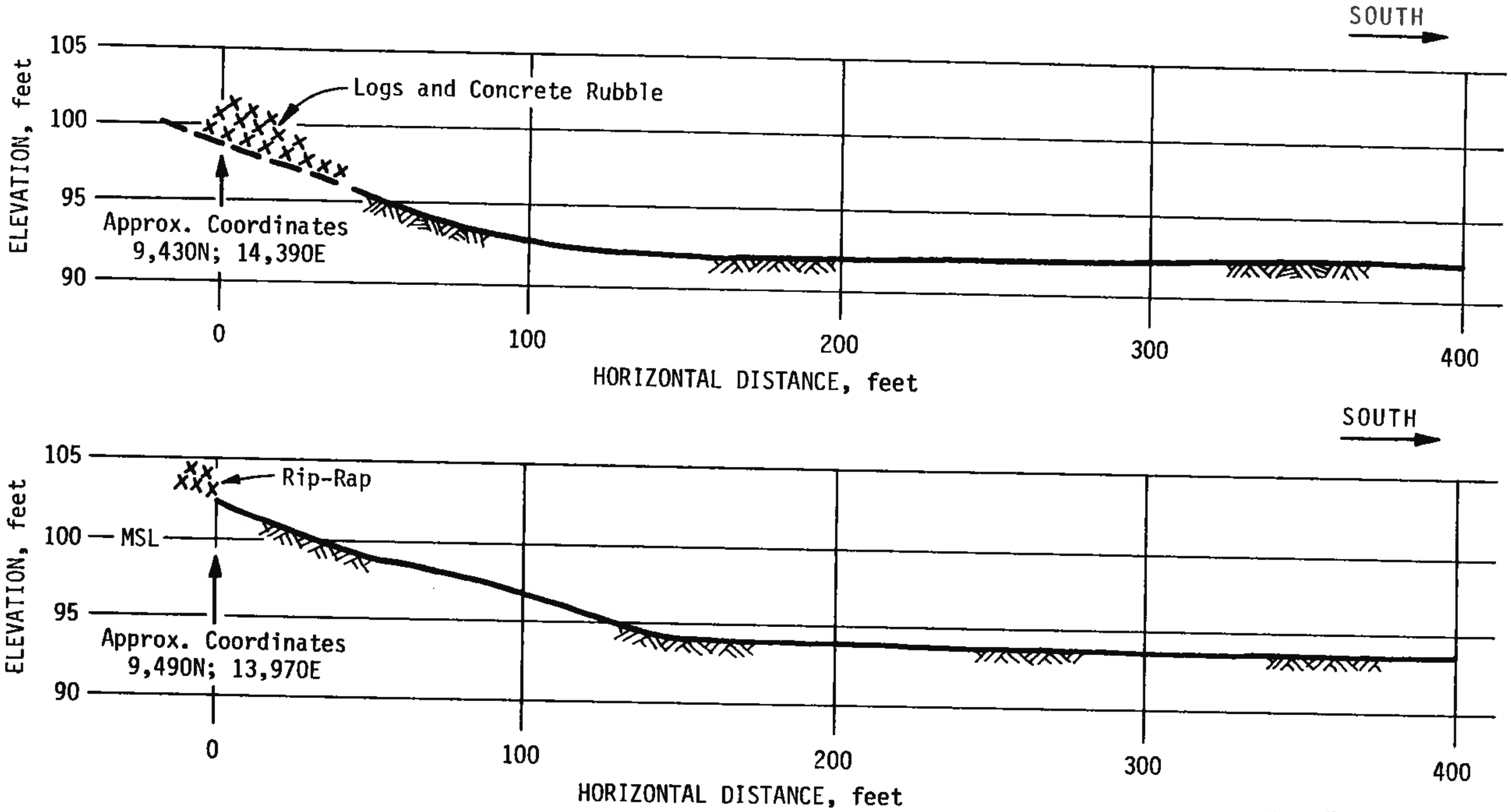
TEST PIT 76-T5



TEST PITS 76-T4 AND 76-T5

OYSTER POINT MARINA
South San Francisco, California

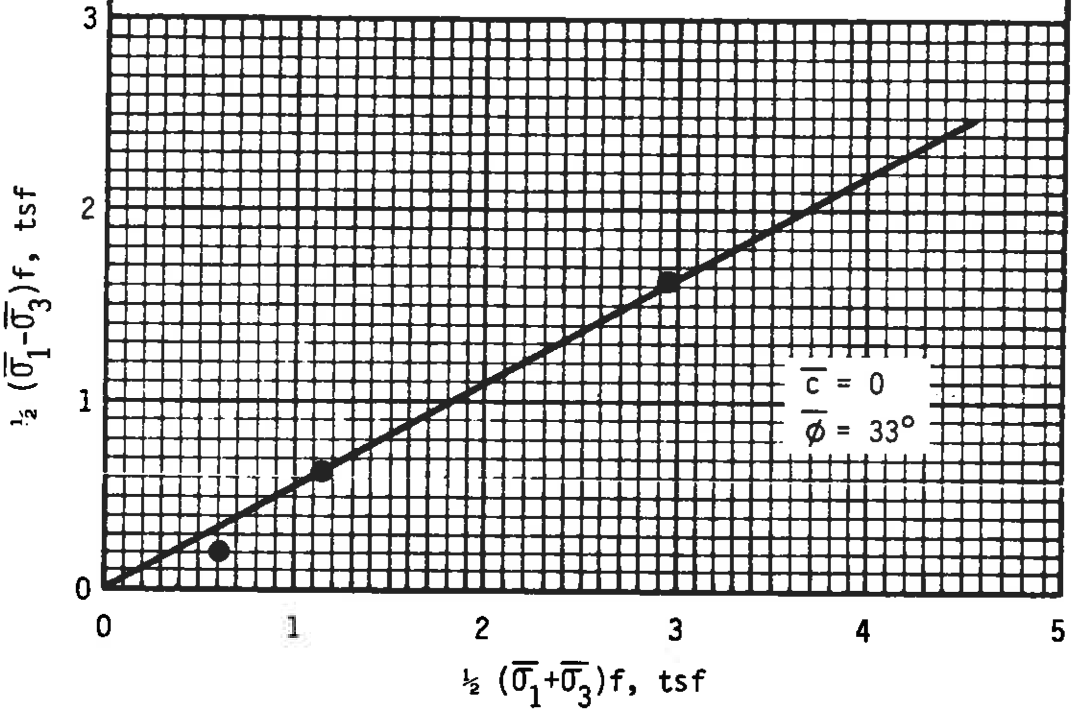
Figure A-21



A total of 5 bottom profiles were made in the field. Three of the profiles overlapped the hydrographic survey made by Brian-Kangas-Foulk & Associates and verified the resulting map (See Figure 1). Two bottom profiles were made along the southshore and are presented above.

Figure A-22 - BOTTOM PROFILES

SPECIMEN NO.	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (PCF)	CONFINING PRESSURE, (TSF)	DEVIATOR STRESS AT FAILURE (TSF)	PORE PRESSURE AT FAILURE (TSF)
76-103-3	59	64	1.0	0.80	0.80
76-101-2	48	69	2.0	1.24	1.49
76-102-3	46	71	5.0	3.26	3.68



NOTES:

FAILURE DEFINED AT MAXIMUM PRINCIPAL STRESS RATIO

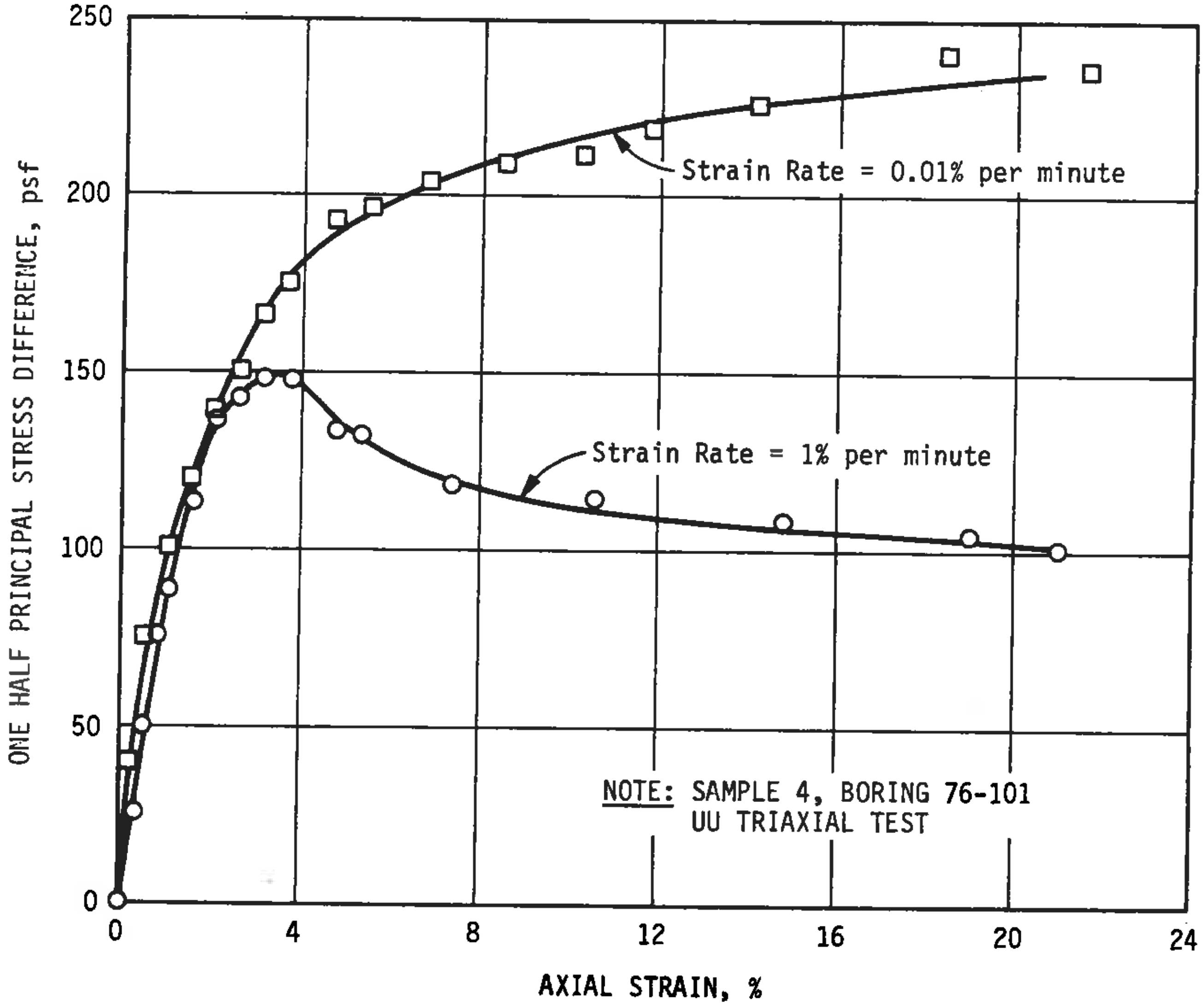
$\overline{\sigma}_1$ = MAJOR PRINCIPAL EFFECTIVE STRESS

$\overline{\sigma}_3$ = MINOR PRINCIPAL EFFECTIVE STRESS

Project No. 13609A
WOODWARD–CLYDE CONSULTANTS

SUMMARY OF CU TRIAXIAL TEST RESULTS

Figure A-23



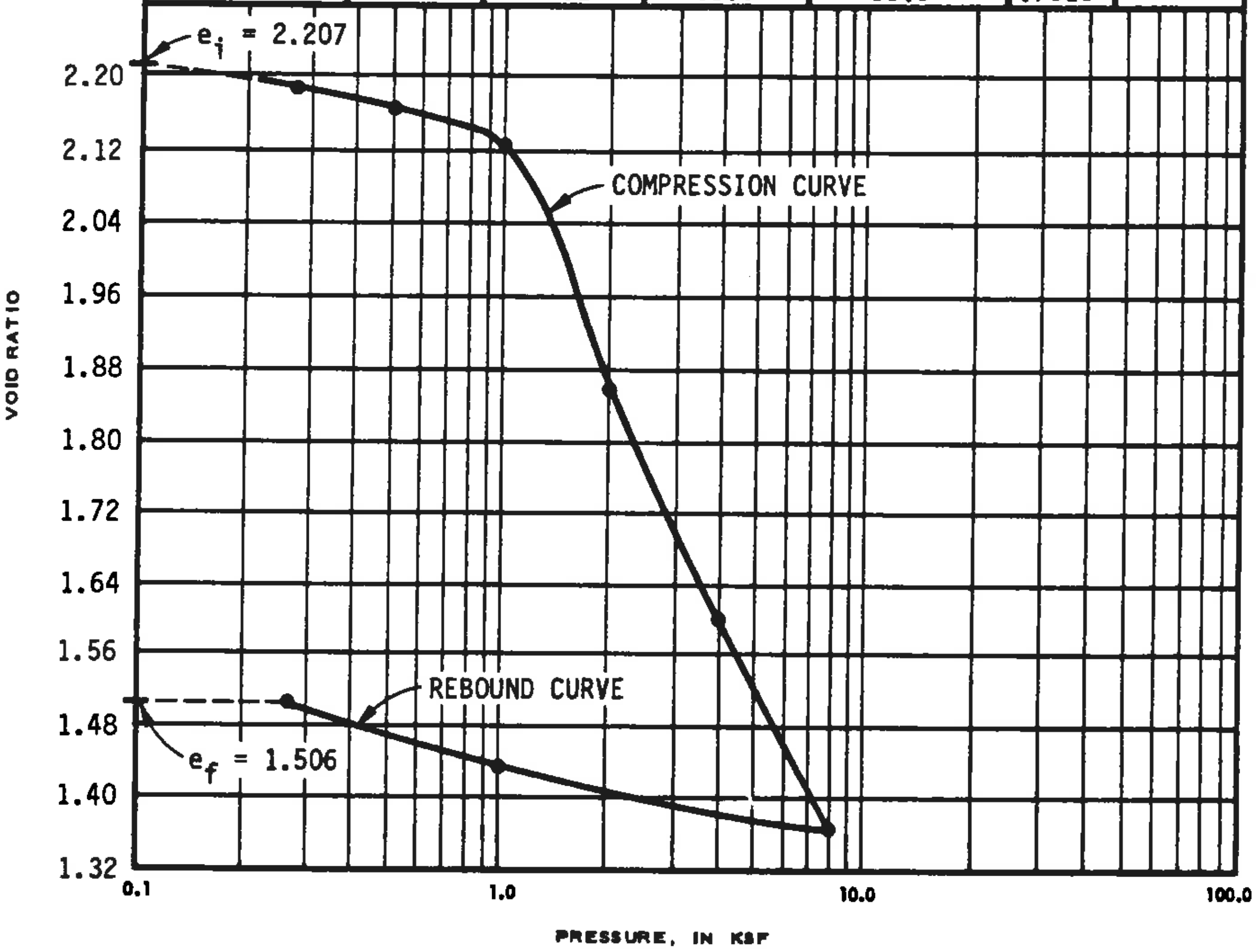
NOTE: SAMPLE 4, BORING 76-101
UU TRIAXIAL TEST

Project No. 13609A
WOODWARD-CLYDE CONSULTANTS

STRESS STRAIN CURVES FROM
SUSTAINED LOADING UU TRIAXIAL TESTS

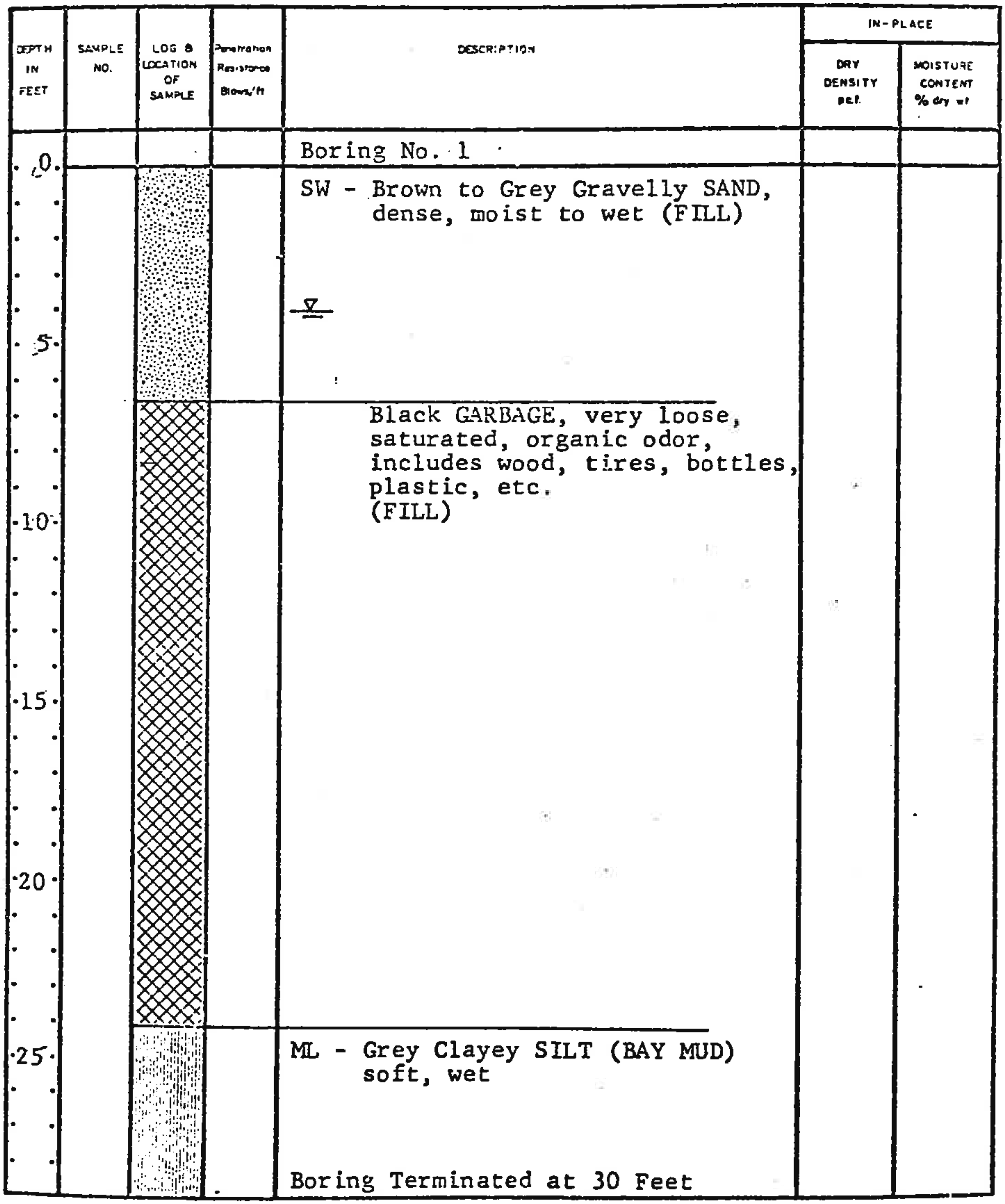
Figure A-24

SAMPLE NO.	SUMMARY OF TEST RESULTS					
102-3	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY, (PCF)	PERCENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.73 Assumed	80.3	53.1	99.4	.8968	2.43
FINAL		55.1	68.0	99.9	.7010	



CONSOLIDATION TEST

1974 BORINGS BY HARLAN ENGINEERS



Boring No. 1

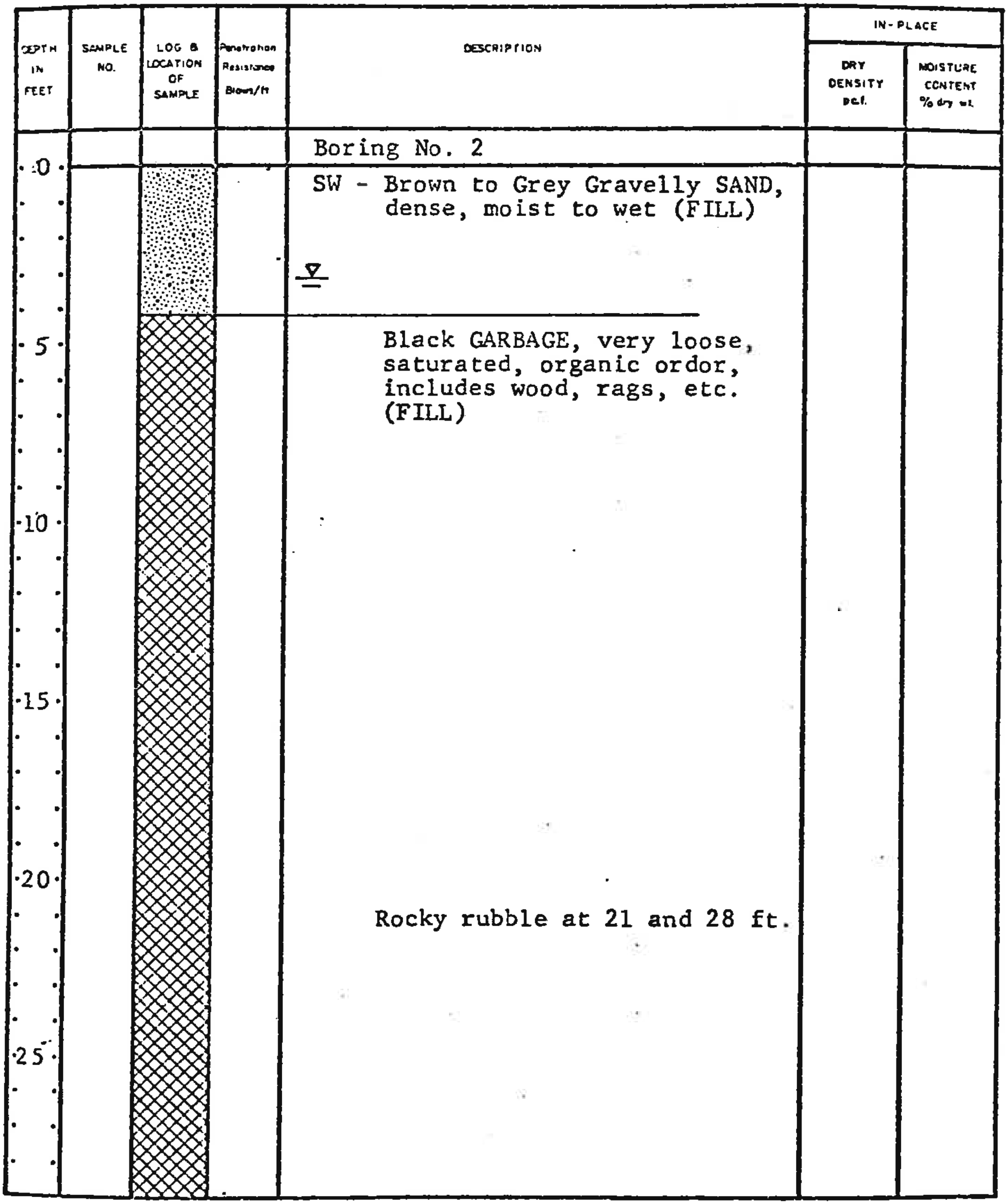
SW - Brown to Grey Gravelly SAND, dense, moist to wet (FILL)

Black GARBAGE, very loose, saturated, organic odor, includes wood, tires, bottles, plastic, etc. (FILL)

ML - Grey Clayey SILT (BAY MUD) soft, wet

Boring Terminated at 30 Feet

Figure No. 2 - Log of Test Boring No. 1



Boring No. 2

SW - Brown to Grey Gravelly SAND, dense, moist to wet (FILL)

Black GARBAGE, very loose, saturated, organic ordor, includes wood, rags, etc. (FILL)

Rocky rubble at 21 and 28 ft.

Figure No. 3 - Log of Test Boring No. 2

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt
				Boring No. 2 (Continued)		
30		▨		ML - Grey Clayey SILT (BAY MUD), soft, wet		
				Boring Terminated at 32 Feet		

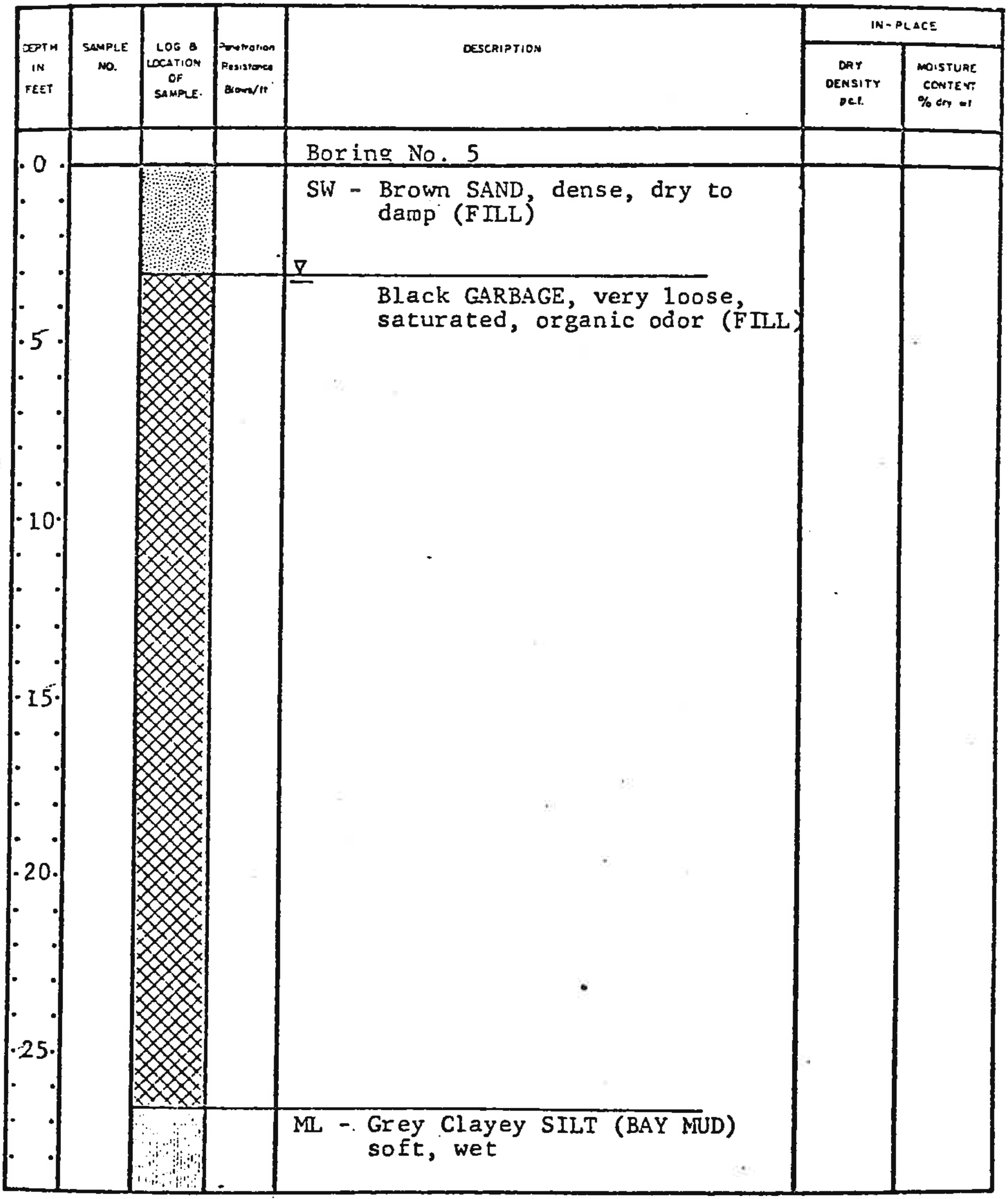
Figure No. 4 - Log of Test Boring No. 2

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt
0				Boring No. 3		
		▒		ML - Grey SILT, loose, dry (STOCK-PILED FILL) with scattered shells		
		▒		SW - Brown Gravelly SAND dense, moist to wet (FILL)		
5		▨		∇ Black GARBAGE, very loose, saturated, saturated organic ordor, wood, rags, etc.		
10		▨				
15		▨				
		▒		ML - Grey Clayey SILT (BAY MUD) soft, wet		
20		▒				
	3-1	▒				
				Boring Terminated at 23 Feet		
25						

Figure No. 5 - Log of Test Boring No. 3

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 4		
0				SW - Brown to Grey Gravelly SAND, dense, mois t to wet (FILL)		
5				▽ Black GARBAGE, very loose, saturated, organic ordor, includes wood etc. (FILL)		
10						
15				ML - Grey Clayey SILT (BAY MUD), soft, wet		
	4-1					
20				Boring Terminated at 20 Feet		

Figure No. 6 - Log of Test Boring No. 4



Boring No. 5

SW - Brown SAND, dense, dry to damp (FILL)

Black GARBAGE, very loose, saturated, organic odor (FILL)

ML - Grey Clayey SILT (BAY MUD) soft, wet

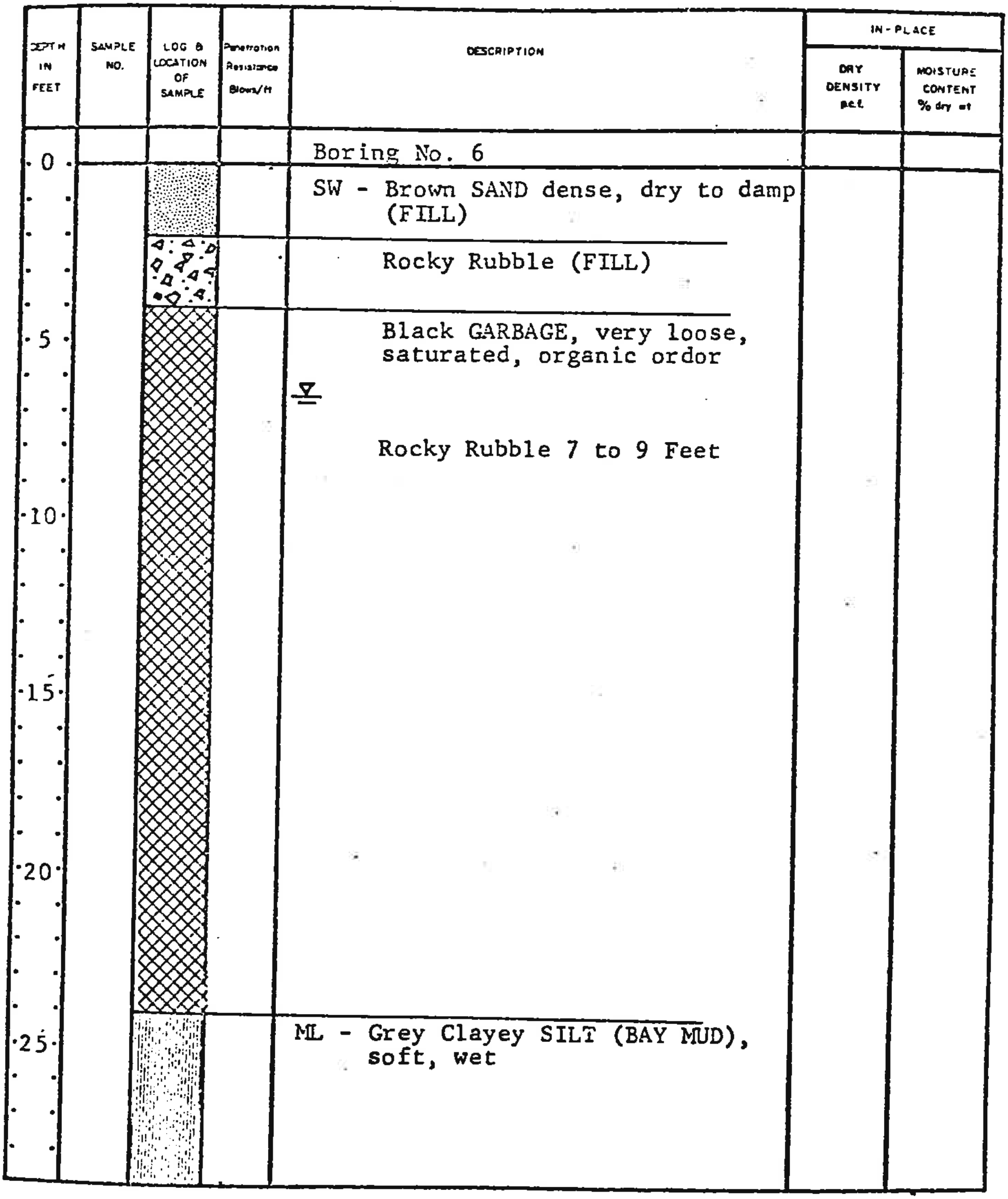
Figure No. 7 - Log of Test Boring No. 5

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY pcf	MOISTURE CONTENT % dry wt.
				Boring No. 5 (Continued)		
30		▨		BAY MUD (Continued)		
				Boring Terminated at 30 Feet		

Figure No. 8 - Log of Test Boring No. 5

File No. H4-0406-F2

June 3, 1974



Boring No. 6

SW - Brown SAND dense, dry to damp (FILL)

Rocky Rubble (FILL)

Black GARBAGE, very loose, saturated, organic ordor

Rocky Rubble 7 to 9 Feet

ML - Grey Clayey SILT (BAY MUD), soft, wet

Boring Terminated at 30 Feet

Figure No. 9 - Log of Test Boring No. 6

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt.
				Boring No. 7		
0		░		SW - Brown Gravelly SAND, dense, dry to wet (FILL)		
5		░		▽		
		▨		Black GARBAGE, very loose, saturated, organic odor, wood, metal, etc. (FILL)		
10		▨				
15		▨				
20		▨				
25		▨				

Figure No. 10 - Log of Test Boring No. 7

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY pcf.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 7 (Continued)		
30		▩		Garbage (Continued)		
		▒		ML - Grey Clayey SILT (BAY MUD), soft, wet		
35				Boring Terminated at 35 feet		

Figure No. 11 - Log of Test Boring No. 7
(Continued)

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 8		
0		░		SW - Brown SAND, dense, moist (FILL)		
5 10		▩		▽ Black GARBAGE, very loose, saturated, organic odor, wood, paper, etc. (FILL)		
15 20		▒		ML - Grey Clayey SILT (BAY MUD), soft, wet		
25				Boring Terminated at 25 Feet		

Figure No. 12 - Log of Test Boring No. 8

1973 TEST PITS BY BERLOGAR, LONG & ASSOCIATES

TABLE A
TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
1	0 - 1½	Soil, sandy silt and some angular rock fragments, light brown, dry
	1½- 15	Garbage, very fresh, slightly damp, no decomposition, lots of wood and metal strapping, material encountered varies from household to commercial or industrial waste products, large percent of paper and plastic, no intermixing of soil
		Total Depth 15', no water.
2	0 - 1.0	Soil, sandy silt, brown, dry
	1.0-2.5	Layered excavated rock and garbage, rock is a blue gray serpentine type material very densely compacted with thin layers of garbage
	2.5-14	Garbage, household to industrial, lots of waste paper and phone books including a large amount of plastic items, wood and tires, slightly damp (newspaper from 10' deep dated March 4, 1969)
		Total Depth 14', no water.
3	0 - 0.5	Soil, sandy silt with rock fragments, brown, dry
	0.5 - 4.0	Garbage, very fresh, no intermixed soil, slightly moist, no decomposition
		Total Depth 4', no water.
4	0 - 1	Soil, sandy silty clay, gray brown, dry
	1 - 4	Garbage, fresh, slightly moist, top 1' of garbage starting to decompose, mostly paper and plastic items with some scrap wire and steel
		Total Depth 4', no water.
5	0 - 1.3	Soil, sandy silt with some rock fragments
	1.3 - 3.0	Garbage, slightly decomposed, moist, no soil mixed in garbage
		Total Depth 3', no water.

TABLE A
TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
6	0 - 1.0	Soil, sand and silt, dry, brown, loose
	1 - 3	Decomposed garbage, very moist
	3 - 4	Fresh garbage, slightly damp
		Total Depth 4', no water.
7	0 - 1.5	Soil, silty sand, brown dry, very hard, (probable road area)
	1.5-3	Silty clay, gray, dry, very hard, (probable road area)
	3 - 5	Decomposed garbage, turning dark gray black, moist
		Total Depth 5', no water.
8	0 - 1	Soil, silty sand and rock fragments to 6"
	1 - 4	Garbage, decomposed, turning black, moist
		Total Depth 4', no water.
9	0 - 1	Soil, silty sand with rock fragments to 6", dry
	1 - 3	Garbage, fair[illegible] well decomposed, turning black, mixed [illegible]h lumber and bricks and broken concre[illegible] garbage is only slightly moist
		Total Depth 3', no water.
10	0 - ½ "	½" asphalt
	½"- 5½"	5" crushed aggregate subbase
	5½"- 1.5	Sand, fine, gray, moist to wet, smells of garbage leachate
		Total Depth 1.5', no water.
11	0 - ½"	Asphalt
	½"- 6"	Crushed aggregate subbase
	6" - 1'	Gray fine sand
	1' - 1.5'	Brown clay
		Sand smelled of garbage leachate.
		Total Depth 1.5', no water.

1970 BORINGS BY GRIBALDO, JONES & ASSOCIATES

SUMMARY OF STANDARD LABORATORY TEST RESULTS

BOREHOLE No.	SAMPLE No.	MATERIAL DESCRIPTION	PHYSICAL CONSTANTS		Shear tests		CONSOLIDATION		COMMENTS
			γ_D	W	C	ϕ	C_c	t_{90}	
2	1	Gray silty CLAY (Bay mud)	59.1	64.5					
4	1	"	50.7	76.2	160	-			Unconfined comp.
5	1	"	60.3	65.4	162	-			"
5	2	"	57.9	70.2					
5	3	"	59.0	66.2	135	-			"
5	4	"	54.6	70.8	370	-			"
6	1	"	50.8	80.7					
6	2	"	51.4	79.0	250	-			TX/UU 450 p.s.f.
6	3	"	56.7	70.0					
6	4	"	54.4	75.7					
6	5	"	54.5	68.8	145	-			TX/UU 1100 psf
6	6	"	58.2	65.5	600	-			Unconfined comp.
7	1	"	55.0	73.4	130	-			TX/UU 450 psf
7	2	"	54.4	70.3	190	-			TX/UU 700 psf
7	3	"	58.4	65.8					
7	4	"	48.2	87.8	480	-			Unconfined comp.
7	5	"	60.6	62.3					

TX/UU = Triaxial compression test, unconsolidated, undrained, at given confining pressure

File No. E8706-S1
1 June 1970

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f	IN-PLACE MOISTURE CONTENT % dry wt
0					
			Sanitary fill (wood, paper, trash, etc. dry, loose		
5					
15					
20					
25			Gray silty CLAY (Bay Mud) and sanitary fill (wood, paper, trash), mixed, wet, very loose		
28					
41					
45			Gray silty CLAY (Bay Mud)		
			Boring Terminated at 47 feet Bay Mud not penetrated		

Figure No. 3 - Log of Test Borings No. 1

File No. E8706-S1
1 June 1970

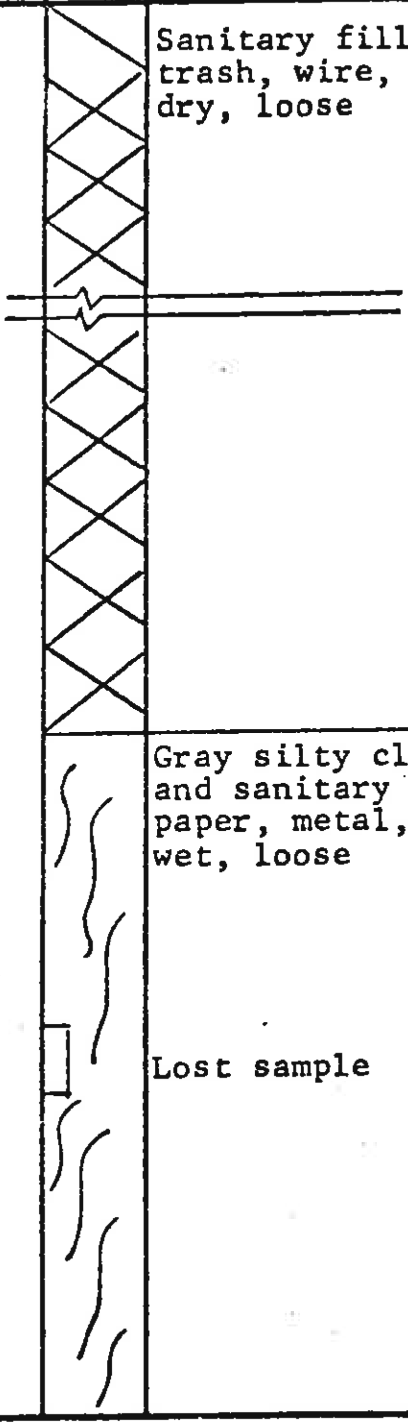
						IN-PLACE	
DEPTH IN FEET	SAMPLE NO	LOG & LOCATION OF SAMPLE	DESCRIPTION	N Stand. Pen. Blows/ft.	qu U.C.S. t.s.f.	DRY DENSITY pcf	MOISTURE CONTENT % dry wt.
0							
			Sanitary fill (wood, paper, trash, wire, tires, etc.), dry, loose				
5							
15							
20							
23			Gray silty clay (Bay Mud) and sanitary fill (wood, paper, metal, etc.), mixed, wet, loose				
25							
30			Lost sample				
35							

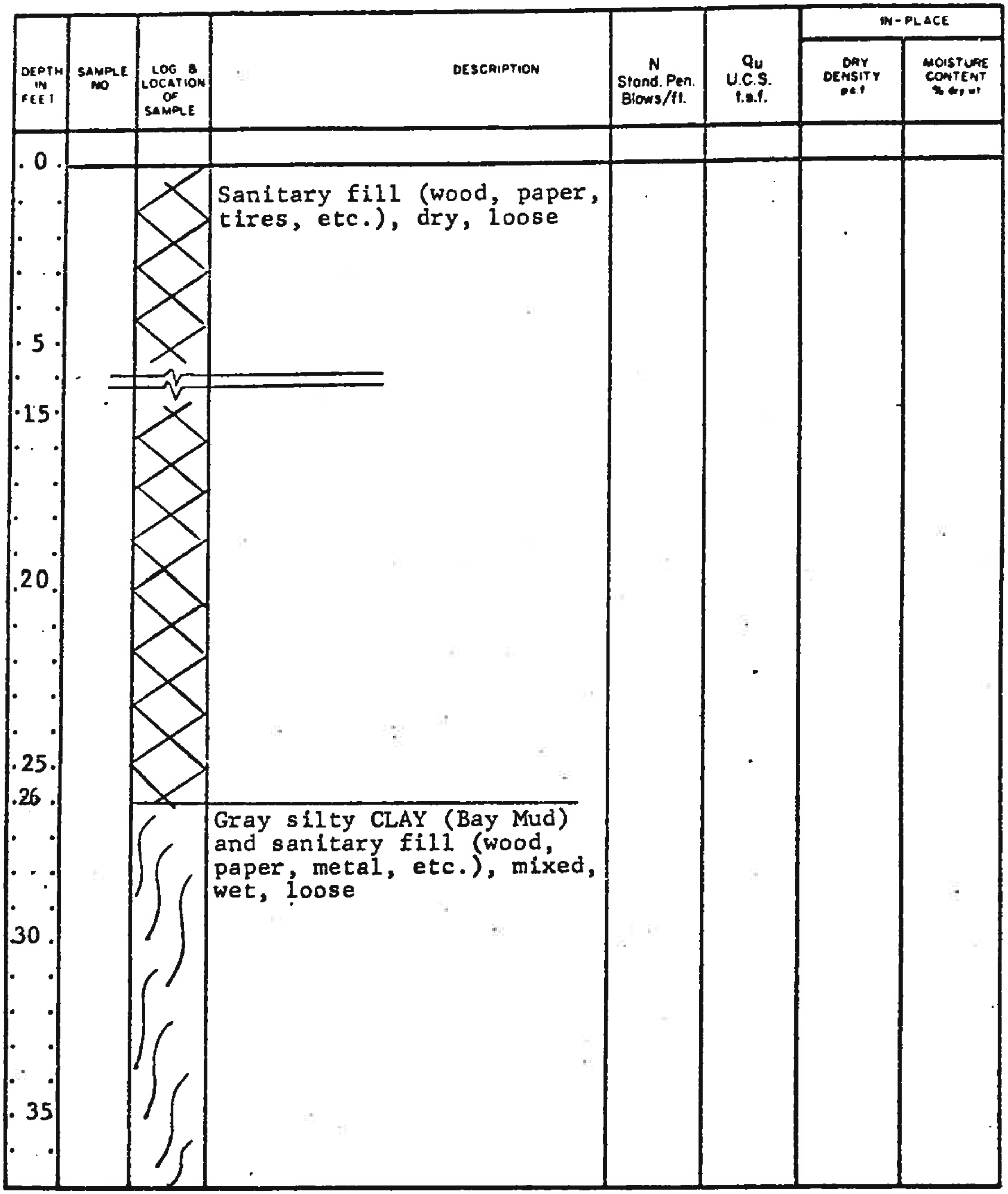
Figure No. 4 - Log of Test Borings No. 2

File No. E8706-S1
1 June 1970

DEPTH IN FEET	SAMPLE NO	LOG & LOCATION OF SAMPLE	DESCRIPTION	N Stand. Pen. Blows/ft.	qu U.C.S. t.s.f.	IN-PLACE	
						DRY DENSITY pcf	MOISTURE CONTENT % dry wt
			Boring No. 2 continued				
38		▨	Gray silty clay (Bay Mud) organic, soft, wet, some shells				
40		▨					
45		▨					
	2-1	▨				59.1	64.5
48			Boring Terminated at 48 feet Bay Mud not penetrated				

Figure No. 5 - Log of Test Boring No. 2

File No. E8706-S1
1 June 1970



Sanitary fill (wood, paper, tires, etc.), dry, loose

Gray silty CLAY (Bay Mud) and sanitary fill (wood, paper, metal, etc.), mixed, wet, loose

Figure No. 6 - Log of Test Borings No. 3

File No. E8706-S1
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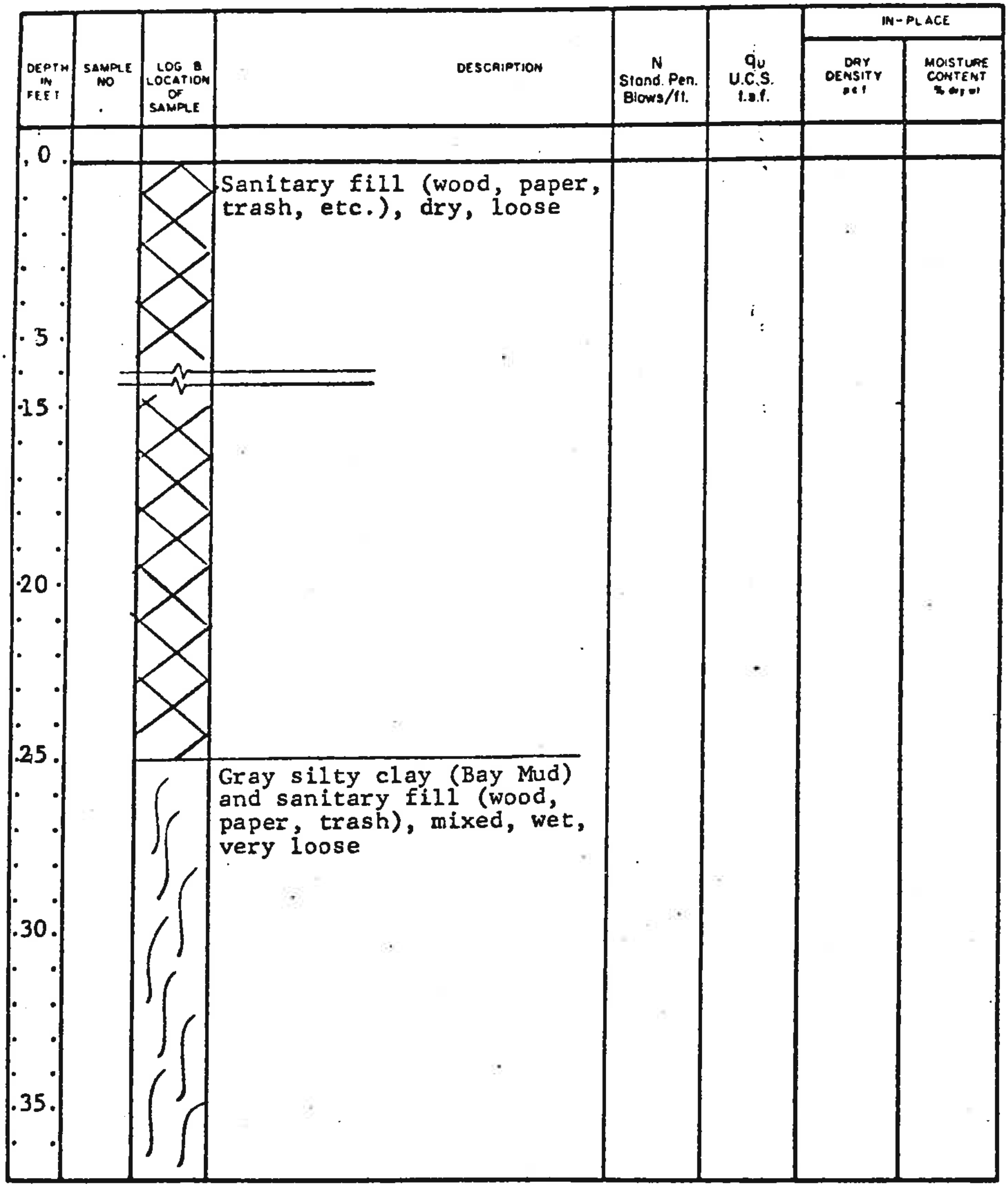
DEPTH IN FEET	SAMPLE NO	LOG & LOCATION OF SAMPLE	DESCRIPTION	N Stand. Pen. Blows/ft.	Qu U.C.S. t.s.f.	IN-PLACE DRY DENSITY pcf	IN-PLACE MOISTURE CONTENT % dry wt
			Gray silty Clay (Bay Mud) and sanitary fill, mixed				
40			Grading with less sanitary fill material				
43		▨	Gray silty clay (Bay Mud) organic, soft, wet				
45		▨					
		▨	Numerous shells				
50		▨					
106		▨					
110		▨					
115		▨	Firm material at 115 feet				
			Boring Terminated at 115 feet				

Figure No. 7 - Log of Test Borings No. 3



Sanitary fill (wood, paper, trash, etc.), dry, loose

Gray silty clay (Bay Mud) and sanitary fill (wood, paper, trash), mixed, wet, very loose

Figure No. 8 - Log of Test Borings No. 4

File No. E8706-S1
1 June 1970

						IN-PLACE	
DEPTH IN FEET	SAMPLE NO	LOG & LOCATION OF SAMPLE	DESCRIPTION	N Stand. Pen. Blows/ft.	qu U.C.S. t.s.f.	DRY DENSITY pcf	MOISTURE CONTENT % dry wt
			Boring No. 4 continued				
			Gray silty CLAY and sanitary fill (Bay Mud)				
40			Gray silty CLAY (Bay Mud), very soft, organic, wet				
43							
53							
55			Soft to moderately firm				
60							
	4-1					50.7	76.2
62							
115			Sandy clay, firm				
118							
			Boring Terminated at 118 feet				

Figure No. 9 - Log of Test Borings No. 4

File No. E8706-S1
1 June 1970

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE	
				DRY DENSITY p.c.f	MOISTURE CONTENT % dry wt
0			Sanitary fill (wood, paper, trash, etc.), dry, loose		
5					
15					
20			Gray silty CLAY (Bay Mud), very soft, numerous shell fragments		
25			soft, fewer shells		
30	5-1			60.3	65.4
35	5-2			57.9	70.2

Figure No. 10 - Log of Test Borings No. 5

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f	IN-PLACE MOISTURE CONTENT % dry wt.
			Boring No. 5 continued		
		▨	Gray silty CLAY (Bay Mud), soft, some shells		
40		▨			
		▨	Soft - moderately firm		
45		▨			
	5-3	▨		59.0	66.2
50		▨			
55		▨			
	5-4	▨		54.6	70.8
58		▨			
107		▨			
110		▨	Firm materials at 110		

Figure No. 11 - Log of Test Borings No. 5

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE	
				DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt.
0					
		▨	gray silty CLAY (Bay mud), soft, some shells		
5		▨			
	6-1	▨		50.8	80.7
10		▨			
15	6-2	▨		51.4	79.0
20	6-3	▨		56.7	70.0
25	6-4	▨		54.4	75.7

Figure No. 12 - Log of Test Boring 6

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f	IN-PLACE MOISTURE CONTENT % dry wt.
			Boring No. 6 continued		
30			very soft (highly disturbed in place)		
			grading moderately firm		
35	6-5			54.5	68.8
40					
45					
	6-6			58.2	65.5
50					
55					
58					
			Boring Terminated at 58 feet Bay mud not penetrated		

Figure No. 13- Log of Test Boring No. 6

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE	
				DRY DENSITY p.c.f	MOISTURE CONTENT % dry wt.
0					
		▨	Gray silty CLAY (Bay Mud), sanitary fill, organic, some shells		
5		▨			
10		▨			
15		▨			
	7-1	▨		55.0	73.4
20		▨			
	7-2	▨		54.4	70.3
25		▨	Grading moderately firm		
	7-3	▨		58.4	65.8

Figure No. 14 - Log of Test Borings No. 7

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	DESCRIPTION	IN-PLACE	
				DRY DENSITY p.c.f	MOISTURE CONTENT % dry wt.
			Boring No. 7 continued		
		▨	Gray silty CLAY (Bay Mud), moderately firm, few shells		
30		▨			
33	7-4	▨		48.2	87.8
35		▨			
40		▨			
44		▨			
45	7-5	▨		60.6	62.3
47		▨			
51		▨			
	7-6	▨			
55		▨			
			Boring Terminated at 55 feet Bay Mud not penetrated		

Figure No. 15 - Log of Test Borings No. 7