IMPROVEMENT OF THE
ALA WAI CANAL

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TABLE OF CONTENTS

SUMMARY OF FINDINGS

I. INTRODUCTION

II. RECOMMENDATIONS

III. SECTIONS

A. Proposed Modification of the Ala Wai Canal

B. Sedimentation Processes of the Ala Wai Canal

C. Water Quality Characteristics of the Ala Wai Canal and the Effects of Mechanical Flushing

D. Well Hydrology, Ala Wai Canal

E. Biological Aspects of the Proposed Mechanical Flushing of the Ala Wai Canal

F. Estimated Costs
SUMMARY OF FINDINGS

1) It has been proposed to improve the water quality of the Ala Wai by introducing a flow of clear salt water at the head of the canal, increasing the flushing rate of water in the canal. Preliminary work by Miller and Chave (1976), Section A, suggested a figure of 10,000 m$^3$/hr. This water might be supplied from off the mouth of the Ala Wai Yacht Harbor and pumped via a large diameter pipe to the head end of the canal, or it might be pumped from salt water wells in the vicinity of the head of the Ala Wai.

2) A survey of the existing data shows that the proposed additional water flow will have little effect on the sedimentation processes now occurring in the Ala Wai (Section B). Specifically, increased sedimentation in the Ala Wai Yacht Harbor would not occur, and the sill would continue to form at the mouth of Manoa-Palolo stream. Periodic dredging to remove this sill would still be necessary.

3) Water in the Ala Wai currently fails to meet Chapter 37-A Water Quality Standards in a number of areas, most conspicuously by its high concentration of total coliform bacteria (Section C). Dilution with clear salt water to meet the standards would require a flow of approximately 18,000 m$^3$/hr, assuming no die-off. However, coliform bacteria are killed by exposure to salt water. The time for 90% disappearance in water of 34.7% salinity was estimated from studies at the Sand Island sewage outfall to be approximately half an hour. This time includes coliform reduction by both die-off and division. Laboratory studies indicate strict (no dilution) die-off periods ranging from 1.5 to 4.5 days. Mixing ports in the Manoa-Palolo stream should assist in reducing the coliform counts both through dilution and increased exposure to salt water. Whether the coliform counts will be reduced to levels meeting current water quality standards is questionable. Certainly flushing will be an improvement in the present situation.

4) Should the water for the Ala Wai improvement project be drawn from salt water wells, it appears possible to provide the necessary volume of water by spacing the wells to achieve an optimum tradeoff between draw-down and radius of influence (Section D). The most serious potential problem appears to be subsidence.
in the area around the wells due to differential compaction of the clay layer. This could seriously affect roads and building foundations in Waikiki. A study should be contracted to an engineering firm to examine subsurface engineering properties of the area and determine the extent of the effect of pumping.

5) The primary effect of additional flushing on the biota of the Ala Wai appears to be through the reaction of the phytoplankton (Section E). The present high concentrations of phytoplankton are maintained by high nutrient levels, which in turn are maintained by the slow water turnover in the head end of the canal. Addition of low-nutrient salt water at 10,000 m³/hr will rapidly reduce nutrient levels in the canal and thus reduce the standing stock of phytoplankton.

6) Providing 10,000 m³/hr of flushing water by pipeline is estimated to cost approximately $1,500,000. Providing the same amount of water from 15 high-capacity wells would cost approximately $700,000-1,000,000. Associated detrimental effects on the aquifer and compaction of the surrounding clay layer may force the drilling of a larger number of smaller wells. The cost of 60 small wells would be approximately $1,900,000. Operational and maintenance costs would be about the same for either the pipeline or the well method. An engineering study of the aquifer and clay layer would cost between $10,000 and $20,000.

7) A clean Ala Wai should rapidly become a great recreational asset for swimming, boating, fishing, and other activities. Thought should be given to improving the banks of the Ala Wai to facilitate recreation, especially on the makai side.
I. INTRODUCTION

The Ala Wai, the 3-km long drainage canal which made possible the development of Waikiki as a major resort area, should also be one of its prime recreational assets. Unfortunately, the Ala Wai is presently an eyesore rather than an asset. Siltation around the mouth of Manoa-Palolo stream, which enters the canal near its midpoint, restricts water circulation and allows the head end of the canal to become stagnant. The resulting murky green, smelly water deters most people from enjoying the canal. The mud bar also restricts boating activities in the Ala Wai, being entirely exposed at low tide as an ugly expanse of mud and discarded junk. A cleaned-up Ala Wai filled with clear water would rapidly become a favorite recreational area, not only for tourists in Waikiki, but more importantly, for the permanent residents of Honolulu. It would also provide a conspicuous example of government action in improving Hawaii's environment.

The Hawaii Department of Land and Natural Resources is preparing to dredge the Ala Wai to a depth of six feet. Such action will remove the mud bar and improve circulation in the canal. However, as this report will demonstrate, removal of the mud bar by itself will not result in a clean Ala Wai. Water in the canal is presently flushed by tidal action and fresh water drainage. The tides do not move water in the head end of the canal to any substantial degree. Stream input moves only the upper one meter of the water. While its volume can be enormous during storms, stream discharge is much smaller under normal conditions. The key to improving water quality in the Ala Wai is the introduction of a constant flow of clean salt water in the head end of the canal to provide constant flushing. This report examines two ways of providing this flow, as well as examining its effects on sedimentation processes, water quality, and biological productivity.

The enhanced recreational potential of a cleaned-up Ala Wai would justify additional improvements along its banks. For example, Ala Wai Boulevard runs so close to the edge of the canal that the intervening grassy strip and sidewalk are quite narrow. A wider margin would allow greater utilization by a larger number of people with less hazard from street traffic.
an artist's conception of a boardwalk extending the margin out over the canal approximately 20 ft. The poor structural quality of the existing canal walls may dictate placement of the pipe adjacent to the wall without dredging. In such case an attractively designed and landscaped boardwalk over the pipeline such as that shown in Figure 1 along the makai side, would add to public enjoyment of the Ala Wai.
II. RECOMMENDATIONS

1) The basic objective of flushing the Ala Wai to produce an esthetically clean appearing canal by introducing a flow of clear salt water, at a rate of 10,000 m$^3$/hr, at its head appears desirable and sound.

2) A firm recommendation as to the source of this water, whether from wells or a pipeline, cannot be made without the engineering studies discussed in Section D.

3) The proposed 10,000 m$^3$/hr, whether from wells or a pipeline, will have no significant adverse affect on sedimentation processes in the canal or boat harbor.

4) High coliform counts will be reduced and thus improved over existing conditions; however, a reduction by dilution sufficient to meet State standards would appear to require an economically unrealistic flow of salt water (18,000 m$^3$/hr.).

5) Cost estimates for the pipeline have been updated and now total approximately $1,500.00.

6) Total cost of well installation is dependent on the results of the engineering study. Estimates range from approximately $700,000 to $1,900,000, including offsite power installation and pump housing.
III. SECTIONS A, B, C, D, E, F

The details of the findings which led to the above cited recommendations are presented in the following six sections.
SECTION A.

PROPOSED MODIFICATION OF THE ALA WAI CANAL

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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>I. Abstract ..........................................................</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acknowledgment ....................................................</td>
<td>1</td>
</tr>
<tr>
<td>II. Introduction ...................................................</td>
<td>1</td>
</tr>
<tr>
<td>III. Possible Modifications ......................................</td>
<td>1</td>
</tr>
<tr>
<td>A. Passive Modification ..........................................</td>
<td>1</td>
</tr>
<tr>
<td>1. Dredging Only ...................................................</td>
<td>2</td>
</tr>
<tr>
<td>2. Biological Solutions ..........................................</td>
<td>3</td>
</tr>
<tr>
<td>B. Active Modification - Pumping ..................................</td>
<td>3</td>
</tr>
<tr>
<td>1. Various Pumping Schemes .......................................</td>
<td>3</td>
</tr>
<tr>
<td>2. Tidal &quot;Pumping&quot; ................................................</td>
<td>5</td>
</tr>
<tr>
<td>3. Alternate Energy Source Pumping ................................</td>
<td>7</td>
</tr>
<tr>
<td>IV. Proposed Pumping .................................................</td>
<td>9</td>
</tr>
<tr>
<td>A. A Brief Description of the Proposed System ..................</td>
<td>9</td>
</tr>
<tr>
<td>B. Computational Method ..........................................</td>
<td>10</td>
</tr>
<tr>
<td>1. Limitation of the Model .......................................</td>
<td>11</td>
</tr>
<tr>
<td>C. Pipe Size and Construction ....................................</td>
<td>14</td>
</tr>
<tr>
<td>V. Benefits ..........................................................</td>
<td>17</td>
</tr>
<tr>
<td>VI. Environmental Impact Statement ..............................</td>
<td>20</td>
</tr>
<tr>
<td>VII. Construction Costs ..........................................</td>
<td>22</td>
</tr>
<tr>
<td>VIII. Summary ......................................................</td>
<td>23</td>
</tr>
<tr>
<td>IX. References .....................................................</td>
<td>24</td>
</tr>
</tbody>
</table>

Figures, Tables

Appendix A

Appendix B
PROPOSED MODIFICATION OF THE ALA WAI CANAL

I. Abstract

The Ala Wai Canal flushes poorly due to its length to width ratio; this, combined with the high nutrient levels discharged into it, causes a persistent problem with water quality. As a result of this study we believe that the most cost effective means of improving the water quality in the Ala Wai Canal is a conventional pumping system. A pipe running the length of the canal fed by an electric powered pump near the entrance will exchange the water sufficiently rapidly so that recreational activities could be conducted in the canal.

Acknowledgment:
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II. Introduction

The Ala Wai Canal was originally dug in its approximate present configuration during 1927. The purpose of the dredging by the Corps of Engineers was to provide an adequate drainage for the Waikiki area. Runoff from the higher Manoa and Makiki area brings in sediment and a heavy nutrient load. This heavy nutrient concentration and the resultant high biological productivity combined with sedimentation effects, tend to spoil what otherwise would be a highly desirable recreation area.

An immediate problem is that an area ewa of the Manoa-Palolo Streams is so heavily sedimented that debris is exposed during much of the tidal cycle. Further, this sedimentation acts as a sill to restrict circulation.

III. Possible Modifications

A. Passive Modification

A simple redredging of the Ala Wai Canal would represent an improvement particularly for some types of boating activity and certainly esthetically. However, the same basic conditions will essentially exist as now do, and the same progression of silting sediments and high concentrations of nutrients and
particulate matter will prevail. The original proposal "Tropical Estuary Ecosystem Analysis and Proposed Modification" under which this proposal was generated outlines some of the background and some of the modifications which were considered. This proposal is Appendix A. This proposed modification of the Ala Wai Canal assumes that a simple clean out and redredging of the Ala Wai Canal will be made. As will be seen later the details of this dredging should take into account the possibility of executing the flushing systems proposed herein.

1. Dredging only

Periodic dredging of the Ala Wai Canal will remove the accumulated sediments to some appropriate location on land or in the ocean where they may perhaps be redistributed. Subsequent runoff from the drainage basin which leads into the Ala Wai will cause sedimentation at approximately the same rate as has been observed in the past, roughly 2,000 m$^3$ per year. The possibility of the concentration of heavy metals and other nutrients and pollutants will remain and the extremely high productivity (Harris, 1975) will continue. The existing sills will reform causing some of the deeper portions of the canal to hold denser water for very long periods of time, so that oxygen depletion occurs in these deeper basins. But simple redredging, while required and desirable, will essentially maintain the status quo and not change the general usage of the Ala Wai Canal significantly.

A second possibility for a completely passive system constructed by dredging would be to complete the loop for the Ala Wai Canal on out to the beach in the Kapiolani-Kuhio Park region. Except in the most cursory way this alternative was not examined in this study. Costs of such a major project would be prohibitively high, if one assigns some reasonable value for the land per square foot which would be used in the construction of an adequately deep open canal, and would be of the order of 12 to 18 acres. Using a figure of half a million square feet of land area to be converted to canal at a dollar cost as low as $20 a square foot, we arrive at a figure of $10 million before questions about keeping the Kapahulu ocean entry to this new canal open would be considered. Further with the exception of some wave-induced pumping action the main mechanism for generating circulation in such a canal would be the wind stress which is unacceptably variable and insufficient to generate velocities throughout the water column great enough to exchange the water within times of the order of a day.
2. Biological Solutions

Although mentioned in Appendix A as a possible approach, purely biological solutions to the high nutrient and pollutant level problems do not seem possible. Numerous consultations with the various staff members of the University of Hawaii and other institutions do not point to any areas where this approach has been successful in a physical situation similar to the Ala Wai Canal. Harris (1975) indicates that for natural waters, outside of a controlled experiment, the Ala Wai Canal is one of the world's most productive bodies of water. The relative instability of conditions tends to disfavor biological solutions because of the population fluctuations which result from the variable input parameters. A possible exception to this would be if there were a type of farming, for example oysters, which could independently become a viable economic enterprise and so support the clean up of the waters of the Ala Wai Canal. This would be a case where the justification for cleaning up the Ala Wai Canal would not be clean water but rather the aquaculture or mariculture of whatever species, or aggregate of species, might exist. Thus any biological agent used to clean up the Ala Wai would have to be as a result of a complete redirection in the use of the Ala Wai Canal in its function partially as a drainage system and perhaps in its use as a waterway for sailboats, outrigger canoes, and other boating activities. The removal of the Ala Wai Canal from its status as a navigable waterway would no doubt be a formidable political task.

B. Active Modification - Pumping

1. Various Pumping Schemes

As indicated in Appendix A various pumping schemes were to be examined. Briefly one could implace a large conduit from Kuhio Beach to the end of the Ala Wai Canal and either pump water from Waikiki into the Ala Wai or from the Ala Wai onto Waikiki Beach. A third alternative, not mentioned in the original proposal, but which became apparently feasible after examining the costs of connecting the Ala Wai to Kuhio Beach by a conduit is to lay a pipe the length of the Ala Wai Canal and to pump water from the entrance of the Yacht Harbor to the Diamond Head end of the Ala Wai Canal. We may first examine the environmental impact of pumping water from the Ala Wai Canal onto Waikiki Beach, and then later use the relative cost of a conduit from the Diamond Head end of the Ala Wai Canal to Waikiki versus the cost of pipe the length of the Ala Wai to distinguish which of these schemes would be preferable. Any method proposed for
the removal of particulate matter from the Ala Wai Canal necessarily implies that that particulate matter will appear somewhere else. The environmental impact of pumping sediment-laden water onto Kuhio Beach is probably totally unacceptable for the reason that it would significantly increase the turbidity of the waters off Waikiki. Further than that the sediment would be deposited in the reef area and a problem situation similar to that of Maunalua Bay would begin to evolve.

At the time of the writing of the original proposal the most likely possibility for cleaning up the Ala Wai Canal was considered to be pumping water from Waikiki Beach to the Diamond Head end of the Ala Wai. A cost analysis was done for the required plumbing for this type of system. The inlet for such a pumping system cannot be in an area where sand is put into suspension by wave action or by any of the other usual nearshore processes which occur in the ocean at the shoreline. Such an inlet pipe from the groin at Kuhio Beach would have to extend seaward to an area where sand in suspension is negligible. All of the sand which is in suspension which would enter such a system would be deposited in the Ala Wai Canal and would cause a problem there in addition to the problem of removal of sand from the Waikiki area.

The cost of such an underwater pipeline extending out from Kuhio Beach may be estimated from other outfall line costs in similar environments on the island of Oahu. The figure for a stormproof line of some 2m cross sectional area extending perhaps a kilometer across the reef may be estimated at $2500 per meter or $2-1/2 million. A further substantial cost ($0.7 to $1.1 million) would be required to construct the type of inlet which would have low enough velocities so that surfers and other debris would not be swept into the intake and the pumping system.

The changing weather, wind, and wave conditions cause sand to migrate on any beach. This has clearly been true in the past at Waikiki and is especially likely to be true as one proceeds with construction projects designed to enlarge the beach. The addition of sand at the shoreline or the establishment of additional groins to hold the beach can change the sand flow pattern.

A second major cost item in a pumping scheme such as this, and applies equally well to pumping water from the Ala Wai to Kuhio Beach, is the cost of a conduit from the end of the canal to Kuhio Beach. In examining this question we assumed that the actual real estate cost would be zero owing to the fact that public lands are available and that
all of the costs would relate to construction costs in rerouting drains, public utilities, removing, repairing, and replacing streets, and the like. A cost figure of $5 million was derived giving a grand total of $8.4 million. The cost of pumping the clean water from the offshore reef through such a system is less than the system proposed herein which would utilize a pipe running the length of the Ala Wai Canal. The pumping cost is less because the pipe length is shorter and because a slightly lesser volume of water would be required. As will be shown in Section IV, this slightly lesser energy cost for pumping from the reef into the Diamond Head end of the Ala Wai is more than offset by the capital cost of the pipeline.

2. Tidal "Pumping"

In these times when the Nation and the State of Hawaii need to conserve energy, one immediately thinks of tidal pumping as one of the natural ways for an estuary to be flushed, as a method which could be enhanced through proper construction, and perhaps achieve the goal of a clean Ala Wai Canal. The tidal range in the Hawaiian Islands is small, of the order of 50 cm. The tides are mixed, that is, both the diurnal and semidiurnal tides are of significant amplitudes so that even at neap tides there is still sufficient tidal range so that significant pumping of the Ala Wai could be achieved. In order for the pumping to be effective, a pipeline would be required. In the present situation as the tide rises the water in the canal is pushed toward the head of the canal so that even though the tidal prism, which is 50 cm thick, could represent one-sixth of the 3 m depth of the dredged canal, far less than one-sixth of the water is exchanged per tidal cycle due to tidal motion. If we imagine a pollutant injected into the Ala Wai Canal at some particular time and then ask what would happen assuming optimal tidal pumping as far as the flushing time for that pollutant is concerned, we see that the optimal time is almost achievable.

If the Ala Wai Canal is to remain a navigable waterway, then the dam across the entrance to the Ala Wai, which would be required for tidal pumping, would have to be equipped with locks which would in turn have to be tended if the free passage of boats were to be permitted. A personnel cost figure including overhead of perhaps $10 per hour might be reasonable. This cost would have to be compared to the electrical energy cost of pumping as will be shown in Section IV. The energy cost would be less than the personnel cost of manning the gates even
assuming the gates are only operational for a fraction of the 24-hour day. Further than that the low head, that is the water height differential available in the tidal pumping scheme, would cause the pipe requirements running the length of the canal to be of very large diameter. Figures for pipe flow for given head and diameter of pipe are given in Table 2 (Anonymous, 1952). The required pipe has an area of $10 \text{ m}^2$, and the alternative to such a pipe would be to build a secondary wall within the canal a few meters from the Waikiki border of the canal. Water would then flow into this channel only on rising tides and out of the main channel only on falling tides. Near the Ala Wai Yacht Harbor this arrangement would have to be modified so that navigation is not affected. The tidal pumping scheme has further against it in that it would not really provide sufficient pumping action and that a marginal volume of water is involved.

To summarize, even the optimal type of tidal pumping scheme for the Ala Wai Canal is not recommended. It would involve implantment of a wall for the length of the Ala Wai Canal parallel with the Waikiki border of the canal. It would require a lock system which would have to be manned at least part time, thereby incurring personnel costs equal to the electrical energy costs required for a pumping system. The water velocities generated would be low; high water velocities have some side benefits as will be seen in Section IV. A tidal pumping scheme would not handle the Manoa-Palolo Drainage Canal in any reasonable fashion, unless a separate arm of the wall and gate system extended partly up that stream. This is a fault that the tidal pumping system has in common with pumping schemes mentioned in Section IIIB1, that is, the Manoa-Palolo Streams represent a complexity which is difficult to handle. Any pumping scheme which functions only between the Diamond Head end of the Ala Wai Canal and Waikiki Beach has the inherent difficulty of not taking adequate care of the Manoa-Palolo Streams. This feature of the canal has an extremely high aspect ratio being only 70 m wide and a kilometer long. A major input into the Ala Wai system is the stream and for a variety of reasons it is desirable that the Manoa-Palolo arm of the Ala Wai Canal system be flushed. Therefore a separate pipeline enhancing this flushing would be required. A small independent flushing system within the Manoa-Palolo Streams could input water from the Ala Wai to the head of the Manoa-Palolo Streams, or alternatively exhaust water from the head of the Manoa-Palolo Streams through a pipe directly into the Ala Wai. In either case the water being
exchanged into this arm of the system would have the water quality which exists at the middle of the Ala Wai Canal at that location. It would thus create a recirculating reservoir which would have a relatively long residence time, hence a high standing population and a very high productivity, creating a source of turbid water in the center of the canal system.

3. Alternate Energy Source Pumping

The energy crisis has focused our attention on alternate energy sources as a possible means of performing various tasks. Two energy sources which are possible for the Ala Wai Canal system are wind power and reverse osmosis, fresh-salt salinity gradient power. One of the methods of generating power which has been mentioned is the reverse osmosis cycle. There is an osmotic pressure difference between fresh and salt water corresponding to a height of 180 m. The outflow from the Manoa Stream would then provide power proportional to that outflow and with the desirable feature that the power available would be greatest when the influx of sediments and nutrients was greatest. The method is in its earliest exploratory stages, is in no way commercial, and it would be impossible to provide cost estimates for it. It is mentioned here only as an engineering curiosity.

Wind power is a technically feasible source of energy for pumping water. Implicit in any wind power scheme would be a row of giant windmills which have their own environmental impact, although it might in itself be a tourist attraction, much as the windmills of Holland, and therefore be viewed as an asset. However, the capital cost of wind power per kilowatt of output is more than three times that for the fossil fuel generating plants at the present time. Maintenance costs on a system of windmills would be higher than that on an electric pump where the maintenance cost of the power plant is already contained in the cost per kilowatt hour, and the wind is highly variable causing two problems. During times of a lack of wind, no pumping can be achieved; during times of hurricane force winds, which will one day come to Oahu, the capital investment of the windmills would likely be destroyed. If we use a recently quoted figure (Dr. James Jones, personal communication) for the capital cost of wind power generation of $1800 per kilowatt capacity (for 20-knot wind speed) and then wish to construct a capacity which would operate the pump 95% of the time then the generating capacity alone is over $2 million and the interest on this amount of money exceeds the cost per hour for a standard electrical pump.

To summarize Section IIIB, a conduit from the Diamond Head end of the Ala Wai Canal into the ocean off of Waikiki has
Ala Wai Canal

an extremely high capital cost. A pumping system on such a conduit would have to pump from the reef with the consequent movement of sand from beach to canal. Several possibilities for systems which would increase the flushing in the Ala Wai Canal system to the point where the water would be esthetically attractive and desirable for recreational purposes were discussed. Of the systems mentioned, dredging only would require the massive conversion of land to canal area which would represent in itself too large a capital investment. Coupled with this the circulation may not be improved that much by cutting the Ala Wai Canal into Kuhio Beach. A purely biological solution seems totally unrealistic except within the possibility of converting the Ala Wai Canal to what would basically be an aquaculture operation. As such it would no longer be available for boating and recreational usage such as envisioned, or is even possible now.

Two pumping schemes, (1) pumping water from the Ala Wai Canal onto Waikiki Beach and (2) pumping water from the reef at Waikiki into the Ala Wai Canal, are examined. The environmental impact of pumping the pollutant and nutrient laden water and the sediments from the Ala Wai Canal onto Waikiki Beach is no doubt unacceptable. No scheme which proposes a drastic reduction of the water quality at Waikiki is viable. The cost of the conduit from the reef into the head of the Ala Wai Canal is substantially higher than the cost of laying the pipe for the length of the Ala Wai Canal. A further difficulty is encountered in that the Manoa-Palolo arm of the Ala Wai Canal system would not be properly flushed by pumping clean seawater only into the head of the canal.

In considering alternatives which would perhaps not be dependent on an electric pumping system, we see that tidal pumping has a high capital cost and a continuing personnel cost to operate locks and the like. In addition it would modify the Ala Wai Canal significantly by requiring an additional wall to divide off the low velocity incoming tidal flow from the rest of the canal. Of the two alternate energy sources (reverse osmosis and wind) reverse osmosis is little known and, at the present time, has too many variables and could not be considered as an operational scheme. Its appeal is that the amount of energy available is proportional to the amount of water and pollutant flowing into the canal. The capital cost of such a system, at least at the present time, is prohibitive. Wind power is at a technical state where it could be utilized, however, the capital cost per kilowatt of
generating capacity is so much higher than that for electricity that the resultant real cost of the energy is higher. A wind generating system would be vulnerable to extreme winds and would require considerable capacity if the pumping system were to be operational during times of very low winds.

IV. Proposed Pumping System

A. A Brief Description of the Proposed System

Before going on to details, it will be well to have in mind the basics of the proposed system. The system would consist of a large diameter pipe laid the length of the Ala Wai Canal with the seaward terminus near the entrance to the Ala Wai Yacht Harbor. A secondary pipe would extend up the Manoa-Palolo Drainage Canal branching off the main pipe. The pump of approximately 400 hp. near the Ala Wai Harbor entrance would force water into the pipe thereby increasing the exchange rate for the Ala Wai Canal system to the point where the waters would be essentially clean although slightly more turbid than the waters off Waikiki in appearance. The configuration, costs, and rationale for this pumping system follow.

The proposed pumping system would have large diameter pipe extending from near the entrance of the Ala Wai Yacht Harbor into the Kapahulu end of the Ala Wai Canal. A smaller branch pipe off the main pipe would extend into the Manoa-Palolo Drainage Canal. A specially controlled gate would be required at this junction serving two purposes: (1) to regulate the flow of the pumping relatively clean seawater and (2) to guide the cleaning mechanism for the interior of the pipe. A large flow, low velocity pumping station could be located anywhere although a location between the Manoa-Palolo Streams and the Makiki Stream would be probably most convenient and accessible. A location as near as possible to the entrance to the Ala Wai Canal would be desirable again because of the problem of cleaning the pipeline. A flow of approximately 10,000 m³ per hour would be achieved by a 400 hp. pumping system. This is sufficient to exchange the water in the Ala Wai Canal at a rate which will flush out the high nutrient and phytoplankton concentrations, thereby greatly improving the water quality. The location of the pipeline along the mauka edge of the canal would require no new right of way. The associated electric pumping station would require very small square footage. Day-to-day manpower costs for maintenance of this system would be minimal. The greatly increased flushing of the canal system would result in the water being desirable and available for various recreational activities. Boating, principally in very
small sailboats, rowboats, kayaks, and canoes could become a popular sport in the area provided that some additional shoreline modifications were made at one or two points; recreational swimming would become quite feasible provided that a small beach were constructed.

B. Computational Method

A variety of computational techniques were used to obtain an estimate of what the effects of various pumping schemes might be. The computational method finally used assumed that the canal was divided into a series of interconnected mixed tanks or units or elements as in Figure 1. Each of these interconnected tanks has a mixing rate associated with it for the adjacent tanks. In addition inflows of fresh water either from storm drains or from streams were assumed and computations were made with varying input flows, typically both the mean flows and extremes of high and zero flows. A mixing coefficient for each interface between the nine elements of the canal was adjusted such that the salinities in the model more or less matched the observed salinities taken from the various reports (Gonzalez, 1971; Miller, 1974). This mixing coefficient is related to the tidal flushing and the turbulent exchange which it generates throughout the length of the canal and to the mixing caused by the winds acting on the surface of the canal. These mixing coefficients for the various elements of the computation are proportional to the area between the boxes. Figure 2 represents elements of the canal with the various inputs, outputs, and mixing coefficients shown schematically. The mixing coefficients are adjusted so that the salinity, which is a balance between the influx of fresh water and the mixing of salt water within the canal, gives a residence time for the water in the canal. This can be compared with the productivity to obtain a check as to the reasonableness of this residence time. From Harris (1975) we see that within the limits of the model and the reliability of the observation that the half life for the water in the canal is approximately 42 hours. The set of equations which go with this simple box model (Figure 2 and Table 1) are given in Appendix B.

The system of equations shown in Appendix B may be solved either for the steady state solution by putting the time derivatives equal to zero, in which case we have a set of algebraic equations, or they may be solved as a function of time in order to examine the effects of a transient. One can imagine an extreme case. Consider that the Ala Wai Canal were totally filled with fresh water. One can then start the time dependent computation using the mixing coefficients which have been adjusted to the present observed steady state. Calculate the amount of time until some fraction, say 90%, of this initial supposed fresh water concentration has been exchanged away due
to the mixing processes caused by the tides and the winds. Figures 3 through 6 illustrate this type of time dependence. We see that the effects of a large perturbation in the salinity of the Ala Wai Canal (due to a storm for instance) should be observable and have an effect for a very few days under present normal conditions. Again this is what is observed after a major rainstorm.

1. Limitations of the Model

Clearly the computer model being discussed is a simplification of what occurs in nature. Some of the more important features of the processes in the Ala Wai Canal are omitted from the model, in particular, the effects of stratification. The fresh water being less dense tends to remain on the surface of the Ala Wai Canal, mixes with the seawater giving the salinity values of approximately 27% which persist more or less regularly at the junction of the Manoa-Palolo Drainage Canal and the main Ala Wai Canal. There is a further stratification which occurs as a result of there being basins within the canal. These basins tend to fill with cool, saline water which remains sometimes for a sufficient period of time to become anoxic. Thus the computer model lacks the feature of stratification which exists in the prototype, and one must be careful in the application of the model in doing any engineering study.

As a first step the computations have assumed that the canal has been dredged to a more or less uniform depth of 3 m. We thereby avoid the problem of there being basins which will capture dense water. In using the results of the computations we have used the more conservative assumption in each case in the application of the results. For example, within the Manoa-Palolo arm of the Ala Wai Canal system we have assumed total mixing at all times. Thus in the presence of the inflow of fresh water from the stream and inflow of salt water from the pumping system even though there is a net mean velocity of water out of that arm of the canal of several centimeters per second, we assume that the mixing occurs instantaneously from the upper to the lower end at all times. At the opposite extreme of this assumption of instantaneous mixing upstream, we could assume no mixing at all, in which case the arm of the canal would be filled with fresh water and salt water in the proportions in which they enter and the mean flow of the sum of these two flows could determine the residence time for a parcel of water in that arm of the canal. This residence time would be unrealistically short and the required pumping rate would be correspondingly low. The addition of the pumped seawater system will in itself promote mixing and reduce the stratification. This in turn will reduce the extremes in the residence times for parcels.
of water in the canal system. This is in addition to the reduction which will be brought about by the dredging and the elimination of basins within the canal system.

At this time one can speculate to make an interesting aside. The productivity of the Ala Wai Canal as indicated by the rate of photosynthesis is currently limited by the light in the water column; that is because the phytoplankton and plant material allow only an insignificant amount of light to penetrate to depths of even a meter. The productivity is thus limited to the upper layers. The stratification tends to keep any parcel of water containing phytoplankton out of the photic zone if it were originally out, and thus we have the light limited productivity as indicated in Harris (1975). The placement of an inadequately small pumping system is the Ala Wai (in an extreme case, one which promoted mixing but generated no substantial flushing) would put the phytoplankters into a situation where the residence time in the photic zone was sufficiently long so that the entire water column would go into production. In this case the productivity of the Ala Wai Canal would actually go up, and the appearance of the Ala Wai Canal might even deteriorate (if that were possible). In particular it might look a lot greener. In any case the computer model does not allow for this, and we will proceed onto a discussion of the results of the model study.

The effects of the addition of the large amount of clean seawater to the upper ends of the canal system will of course be to dilute the nutrient level in the canal system. It will also dilute the standing population of phytoplankton and other particulate matter. In that the productivity in the absence of other limiting factors is related to the standing population, there will be a reduction in productivity over what we now see. The extreme case of this would be if a very large pumping system were introduced such that the canal were flushed rapidly and continuously with pure seawater. Then the standing population of phytoplanktons would be reduced correspondingly. In the course of verification of the computer model a variety of such extreme assumptions was made to insure that the asymptotic solutions agreed qualitatively with what is known as to the present physical situation.

A variety of pumping rates were assumed and the resultant concentration levels computed along with residence times and the steady state solutions. We can examine two of these extremes in terms of the size of the pumping system, one extremely small and one extremely large. Let us suppose for a minute that we use a 1000 hp. pump. This is rather like the shaft horsepower of a sizable oceangoing tug. The volume of water pumped is such that the time scale for the dilution of our
Ala Wai, Canal A-13

reservoir, in this case the Ala Wai Canal, to 32% of its original concentration becomes 6 hours. The outflow through the Ala Wai Canal becomes a noticeable factor for a person swimming and perhaps even rowing, in that the current would be between 1 and 2 knots. The canal would be well flushed in a very short time and except immediately after a large storm the clarity of the water would compare roughly with that at the intake at the Ala Wai Yacht Harbor entrance. At the electrical costs quoted for this type of operation the 1000 hp. pumping system would cost almost $25 per hour. On a yearly basis this approaches a quarter of a million dollars. A similar calculation for a 100 hp. pump cuts the cost by a factor of 10 but boosts the residence time for the water using this definition up to 60 hours. This clearly begins to be at the point where the productivity of the canal would probably be maximized at least for a great fraction of time during the year and while it would represent an improvement in the canal, it would not produce enough improvement to warrant the project.

Within the uncertainty of the currently existing mixing rates and the uncertainty of what the natural mixing rates in the canal will be when it is dredged and balancing electrical costs and initial installation costs off against one another, it appears that a 400 hp. pumping system would be both economically feasible and adequate for the purpose of keeping the canal clean throughout more than 95% of the days of the year. We look at Figure 6 to see that the time required for the dilution of the original concentration, in the worst case to 32% of its original concentration, is roughly 24 hours. A dilution time of 48 hours takes the original concentration of any pollutant down to 10% of its original value. It must be remembered that these are conservative figures assuming a total upstream mixing coefficient comparable to the coefficient that now exists but which might be improved as a result of the dredging. The mean current induced in the canal at this pumping rate would be low enough that it would represent no hazard to either boaters or swimmers. An exception to this would be immediately in the vicinity of the outlet pipes. Depending on the type of outlet there would be a small region of high velocity water flow. In order to minimize this effect, it is recommended that the pipe end in a large L which traverses the end of the canal and that a series of outlet holes from this release water into the canal. The size and spacing of these outlets could be such that it would be difficult for any sediment or other material to settle in the pipe or for any sedimentation to occur in the region of the outlet holes.

In addition to the outlet holes on the diffuser at the ends of the pipes, it will be highly desirable to have a series of inspection ports along the pipeline such that in the event
of an unexpected obstruction of the pipeline a diver would not have to traverse too great a length of pipeline to clear that obstruction. It would be desirable if these inspection ports were of a size and strength such that the inspection covers could be removed to permit a large outflow of water at these various locations along the pipeline. This would be a feature that would be valuable in moving large amounts of totally unconsolidated, low density sediments which may accumulate at one point or another along the pipeline.

C. Pipe Size and Construction

In any flow a certain head or elevation of the water is required and in the proposed system there are several such heads or gradients to be considered. First, we may ignore the head in the head in the canal itself that will be generated by the pumping of water into the ends of the Ala Wai Canal system. The mean flows outward are of such a low velocity that frictional effects can be ignored and need not be considered here. Within the pipelines themselves the flow is proportional to the length of the pipe and the diameter of the pipe for a certain equivalent head. The drag of the water on the inside of the pipe is controlled by its velocity. Higher velocity means relatively more drag and doubling the velocity increases the drag by considerably more than a factor of two. If the velocity in the pipeline is too high, pumping costs will be increased unduly. At the other extreme a very large pipe would be costly but have proportionally smaller pumping costs.

The basic design criterion is that the losses due to the frictional drag of the water inside the pipe not be ridiculously large compared to the other inefficiencies of the system as a whole. Table 2 gives an equivalent head per thousand feet of pipe for different pipe sizes and different roughness coefficients for the interior of the pipe. Later on as we shall see there will have to be a balance between pipe size, pumping costs, and maintenance costs. The main maintenance costs for a concrete pipeline will be keeping the interior of the pipeline free of marine growth. Some studies have been made in connection with large pipelines carrying waters at these velocities in connection with cooling water systems for power generating plants. In all cases it is essential that some provision be made for cleaning unless some other technique such as forcing hot water through the pipeline to kill the organisms can periodically be done. The conditions inside of such a pipeline as is proposed are ideal for the growth for filter feeding organisms such as barnacles, muscles, and so on. These organisms cause hydrodynamic roughness to the interior of the pipe and so greatly increase the drag for any given size pipe. The pipe material choice is between steel and reinforced concrete. Costs are comparable; however, information on the relative durability in this application is very difficult to obtain. Costs of concrete pipe 48 inches in diameter
are approximately $30 to $35 per foot. The effect in the increment of the pipe size is to increase the area and hence reduce the flow velocity and, much more importantly, the equivalent head, which leaves one to favor the larger diameter pipe until the capital investment in the pipeline becomes too large or a limiting factor. Thus a 4-foot diameter pipe will have a cost between $300,000 and $400,000. A 5-foot diameter pipe will have a cost of approximately double this between $600,000 and $800,000, but the pumping cost in the larger pipe is roughly one-third of the cost of the 4-foot pipe. A savings of 100 hp. in the pumping system corresponds to about $25,000 a year in electrical costs. This is to be compared with the difference in costs between the two pipe sizes (between the 4- and 5-foot diameter pipes), and we see that it is in the 8% to 10% of the capital cost range of the difference in pipeline cost. We have combined the data from Table 2 together with the functional relationship in the change in price of pipe as the diameter of the pipe goes up and related it to the electrical costs. Table 1 lets us select for a given flow, the most economical combination of pipeline and pump size assuming the configuration of the pumping to be as shown in Figure 2.

Cleaning requirements on the pipeline tend to favor the larger pipe as indicated above. Further a cleaning method that is not labor intensive needs to be specified. It is not possible to flush the pipeline daily with hot water as is done in some electric plants on the mainland and perhaps in Hawaii. Similarly there is no source of fresh water sufficiently large to fill the pipeline in a short time in order to kill the organisms on the inside. Water from the Manoa-Palolo Streams is not of sufficient quantity or head to ream out the pipes utilizing go-devils or some similar device without a special pumping system to fill the pipe. The introduction of a poison into the pipeline periodically is environmentally unacceptable.

This leaves the only chemical means to be stopping the pumping of water for sufficient time so that the oxygen in the pipeline is used up. After some time anaerobic conditions might be assumed to exist and the pipeline cleared of living organisms. One might expect with the flow cut off in the pipeline that the conditions inside would go anoxic after a short time, however, in a 2 m diameter pipeline the area of the pipeline is 3.14 m², thus the volume per meter of pipeline length is 3,140 liters. This water would be oxygen saturated and therefore would have approximately 7 ml. per liter of oxygen. The amount of organic material in this volume of water which would then have to be oxidized before oxygen levels went low enough to kill the filter feeders on the inside of the pipe would be a good many grams, in particular, approximately a fifth of a gram of carbon equivalent or between 4 and 20 wet grams of organisms depending on the species. It might be possible to utilize this technique provided that some oxidizing material were readily available to inject into the pipeline immediately prior to
shutting down the pumps. However, there ultimately has to be a physical cleaning system for the inside of the pipe and once this system is available, it will probably be most expedient to simply use it periodically at the times that are required to clean the pipe. This will probably be necessary on a yearly basis although the unique usage of the pipeline and its setting does not have any exact correspondence in ordinary industrial situations.

A cleaning device could be made which would be attached to a cable and permitted to go down the pipe pushed by and rotated by the water flowing from the pump. It would be necessary to direct the device at the Y into the appropriate arm of the pipeline and the special cleaning job probably by a scuba diver could be done at the diffusers at the end of the pipeline. A section of pipeline prior to the pump could be cleaned by the device or by a diver. Alternatives to this scheme might be to seal off the pipeline and fill it with air for caisson type workers inside to clean the pipeline. It must be remembered that the interval between cleaning is considerable and these cleaning costs must be measured against the increased electrical costs which are required to overcome the friction generated by the extra roughness or, finally after many many years, the constriction of the pipeline by the growth of sessile animals within it.

As an indication of the nutrient level in the Ala Wai Canal the productivity is over 5 g of carbon fixed per square meter per day which is near the maximum value reported anywhere in a natural environment. This measures out to over a metric ton for the entire canal per day. Using chlorophyll as indicator, the biomass concentration of the surface waters ranges between 9 and 30 mg per cubic meter and between 13 and 48 mg of chlorophyll a per cubic meter in the subsurface waters. We can relate these productivities to the total carbon by way of a productivity index which is given in milligrams of carbon per milligrams of chlorophyll a per hour. Values range between 6 and 17 for this as given by Harris (1975) and reproduced here as Table 3. The corresponding values of carbon fixation range between 120 and 350 mg of carbon per cubic meter per hour.

Thus we see that for much of the canal a figure of about 500 mg of carbon fixed per hour per square meter is to be compared with the standing crop of 5 g per cubic meter. This high productivity is balanced in two ways: About a third of the organic carbon produced is lost (decomposition) and the remainder is lost to the zooplankton grazing and the bacterial degradation and so is recycled within the canal. Both the biomass and the productivity increase as we move away from the harbor area. This is because the flushing is better near the entrance and the standing crop is correspondingly reduced.
Stratification in the canal leads to there being slightly different plankton populations between the surface and the bottom. Improved mixing and flushing will probably eliminate these differences.

The nitrate and phosphate concentrations are extremely high within the canal and are not limiting the primary productivity. In spite of the extremely high productivity (5 g per square meter per day, on the average), the suspended particulate matter is even greater (making up from 13 to 25 g per cubic meter, or about 50 g per square meter), except in the very deepest areas.

The proposed flushing system, at times when there is a substantial interval (2 to 3 days), after a major rainstorm, will flush out the particulate matter at a rate of about one-third of the instantaneous inventory per day. In the case of the phytoplankton a similar percentage of flushing will occur, but balancing this off will be the productivity which occurs during the 24 hours. This productivity will of course decrease as the parent stock is flushed to sea.

Thus to the extent that the particulate matter is not renewable except mainly from the sediments of the canal, it will be reduced to about 10% of its current value within 2 days. Any transient in the biological productivity of the phytoplankton will have a tendency to last longer, however, it too, within a very few days will assume a new, much lower steady state value.

V. Benefits

An actual computation of benefits resulting from the proposed Ala Wai Canal system is always going to be a highly speculative exercise. At various levels of improvement one will make possible new activities which become suitable for the population of Oahu in general. For some very small segment of the population the canal is now sufficiently clean. (A small boy was once observed to cross the Ala Wai Canal swimming, presumably on his way to the beach at Waikiki.) A small segment of the population utilizes the canal for recreational fishing; however, a much larger fraction of the population considers the waters now to be too polluted for that activity. Certain types of boating activities are pursued in the canal, however, the type of activity where one would be in danger of entering the water, such as sailboating, is not popular in the canal.

Some of the things which might be compared to the daily cost of operating the pumping system, would include boating, swimming, and fishing. For these purposes the daily costs may be taken as approximately $200. A concession which rents small sailboats of the "Sabot" or "El Toro" class as an example, might have 20 such craft each with a gross revenue averaging $10 per day; thus equaling the operating cost of the pumping system in gross revenue. There is a scarcity of such protected "lake"
type water on the island of Oahu, and the average tourist is probably reluctant to put to sea in a small boat, even though they would be inside the reef. Wave action and the irregular bottom topography inside the reef, combined with the fact that if one does tip over he may be carried to sea, makes the hazards of small boat sailing too great for the average tourist. Rowing is another activity which could be pursued in the protected waters of the Ala Wai Canal. Here again a dollar value is easier to make because of the possibility of the rental of rowboats. In addition to rowing, fishing from such boats could be conducted.

Outrigger canoe paddling is currently a popular sport in the Ala Wai Canal, but there is no reason to believe that this activity would increase if the Ala Wai Canal were cleaner.

Swimming in the Ala Wai Canal could become a popular sport. A swimming area typical of those found in lakes might include a roped-off area, a float diving platform and the like. It could be located near the diffuser outlet at the head of the canal, thereby having the cleanest water available. It is difficult to assign any dollar value to the introduction of swimming in the canal but if we assume some number like a 100 people and put any dollar value per person on it then it becomes a significant fraction of the total energy costs for the pumping system.

Fishing in the canal, both for finfish and shellfish, would no doubt be greatly enhanced not that the actual number of fish would necessarily increase significantly, but rather that the desirability of the fish from a food standpoint would be greatly enhanced. The reduction in high pesticide, coliform, and heavy metal concentrations in the canal would in turn lead to reduced concentrations in the shellfish, finfish, and crustacean populations. At the present time even granting that the fish caught in the Ala Wai if thoroughly cooked would probably not cause any individual bodily harm, most people do not prefer to fish there because of the aesthetically undesirable appearance and condition of the canal. Thus fishing in the canal, which has dwindled over the years as the population and contaminant load on the canal has increased, could probably be restored. It would furnish an attractive place for young people to fish, available by bus, and safe enough from wave action with good access on public lands.

In terms of cost effectiveness, it would be compared to other projects designed to enhance recreation. In other states where recreational fishing areas are limited, dams are constructed creating lakes which are then stocked and even though a fishing license is required it will typically have a nominal cost. Boating fees on such lakes may be high enough to generate significant revenue in the operation of the recreational facility. In terms of the number of people which might be expected to fish in the Ala Wai if it were cleaned up, one might expect a factor of 5 over the current 20 people per day fishing. This would put it into the category of being one of the major recreational fishing
areas in the State. There would be a strong dependence on the type of stable ecosystem which evolved as a result of the flushing pumping system being in continuous operation.

Property values along the canal would probably be enhanced. Rather than the present situation with high particulate and phytoplankton concentrations and the resultant discolored water, the scene would be one of relatively clear waters rather like the waters inside the reef at Waikiki. This alternate recreational area to the beach at Waikiki could become a selling point for hotel space along the boulevard which borders the canal. If this indeed were the case and the property values were enhanced by only a very few percentage points then the increased revenue from taxes would eventually pay for the project. A shift in the occupancy rate of the hotels along the canal of 5% would be reflected in the value of the hotel property itself of approximately 5%. Thus 2,000 hotel rooms with mean value of $20,000 each a capital investment of $40 million incremented in value by 5% or $2 million would equal roughly the total cost of the entire project and its operation. Tax revenues from this increased value would then tend to offset the cost.

A less tangible benefit, but perhaps quite significant, would be achieved by the example of action taken in a highly visible location to clean up the environment. The many tourists from all parts of the Nation could be reminded that a positive step had been taken by the State of Hawaii.

Outside the line of ordinary recreational activities that one anticipates and expected with a body of water the size of the Ala Wai Canal, there would be a unique facility especially appropriate to Hawaii. The discharge from the pipe at the head of the canal could be directed into a concrete structure such that a continuous standing wave was created. This standing wave could be surfed and, even though at any one time would only support a limited number of surfers, its persistence would surely make it a tourist and local attraction. Figure 7 shows this schematically. In order to allow for the tides changing level, one would have two, or possibly three, outlet pipes at different elevations. Practically speaking there would be little energy loss in this system, if it were properly designed. The energy contained in the momentum of the water is ultimately lost in turbulence and frictional processes anyway. We have made no serious consideration of constructing such a surfing area, however, it is quite certain that such could be constructed. Some model studies of the concrete structure which produces the flow necessary for a standing wave breaker would be an interesting project for the Ocean Engineering Department of the University of Hawaii, and we are sure one that would be enthusiastically pursued. It might even be possible to construct the concrete structure with multiple wave producing areas so that proportionately more surfers could use the facility. For a variety of reasons the height of the standing wave breaker and hence the energy it is dissipating should be kept small. Thus we do not
envisage a breaker or bore height greater than about 2 feet. This could be sustained over a width of perhaps 20 feet depending on the details of the design.

VI. Environmental Impact Statement

At first glance one might be led to believe that once the new steady state is achieved with the pumping system in operation, there would be no net environmental impact exterior to the canal. Although the analogy is not strictly true, the canal system functions presently to some extent like a sewage treatment plant, which has an effluent that is mixed out by way of the Ala Wai Yacht Harbor into the ocean.

In addition some material (heavy metal for example, see Luoma, 1974) is incorporated into the sediments of the canal. This ultimately is taken out in the dredging process and most likely will end up being hauled out in barges and dumped in deeper water. This has the undesirable effect of remaining in the vicinity of the island of Oahu for very long periods of time with the possibility of entering the food chain. In any case the net effect of operation of the pumping system will be that material will more quickly and more directly enter the nearshore oceanic environment that is now the case. At any one time approximately two-thirds of the organic material within the canal is being recycled within the canal through the food chain. The remaining one-third is being flushed due to tidal action and other water motion.

Bacterial counts within the canal immediately after a storm are high. The relatively long residence time of the water within the canal causes these sudden highs to be averaged out in the present effluent from the canal system. Thus the introduction of the pumping system will occasionally raise the peak bacterial, particulates, pesticides, and other counts and measures of pollutants as compared with what they would be in the absence of the pumping system. This brief increase which results from the storm will be compensated for by the persistent lower levels of contamination brought about by the flushing and water exchange induced by the pumping system. In that sense it will be more comparable to the streams running out of the drainage basins on other parts of the island where there is no comparable storage facility such as the Ala Wai Canal. This lack of averaging out of the peaks and troughs of contamination in the effluent from the canal which will be induced by the pumping system is the biggest change which will be made. As mentioned earlier however there will be a small net increase in those biodegradable contaminants which will not be broken down in the canal but rather will rapidly be injected into the ocean at the entrance of the Ala Wai Yacht Harbor.

Within the Ala Wai Yacht Harbor itself the flushing rate will be enhanced and the residence time of the water significantly reduced depending on the exchange rates of the various arms of the harbor with the main channel. No study has been made of these exchange
rates, however, physical considerations would lead one to believe that the exchange time is small compared to the other time scales associated with the flushing rate of the canal. There will, therefore, be an improvement.

To summarize on the exterior environmental impact, there will be less time averaging of the sediment contaminant load that enters the ocean than there now is. Biodegradation will not have time to occur within the canal to the extent that now happens and there will be some small net increase in the biodegradable pollutants which enter the ocean. These will however be comparable to that for watershed areas on the island, for example, the storm drains that exist at Kuhio Beach.

In this project the impact is mainly interior within the Ala Wai Canal system; the exterior impact noted above is minimal. Within the canal the adverse environmental impact includes the usual ones during construction such as the noise associated with heavy equipment usage, the need to have equipment in the park areas handling pipe, some dredge line work, and perhaps some dredging although for the pipe size envisaged no real dredging should be required other than that which must be done anyway to remove material which has been washed down partly as a result of construction in the drainage basin and partly from natural sources.

There will also be the adverse effects of the increased recreational usage of the Ala Wai Canal system. But this will be offset by the enhancement of the quality of waters in the canal and the positive aspects of its use as a recreational facility. There will be some changes in the composition of the flora and fauna but almost without exception these should be considered as improvement. The trend will be away from the current populations and toward the populations of Honolulu Harbor, for example, *Chanos* (awa) which feed on blue-green algae and diatoms with some zooplankton particularly copepods in their diets would be expected to be relatively reduced in population as compared to *Elope hawaiiensis* (awa-awa) and *Scomberoides sanctipetri* (lae or ulua) which feed more on nehu (*Stolephorus purpureus*) and gobies (*Gambusia* sp.). The dredging which is to be performed anyway will disturb the bottom dwellers such as crabs, tubeworms, and clams of various types, but these will restore to a steady state population determined by the substrate and somewhat similar to what is now present.

To summarize, virtually all of the modifications to the canal which result from the improved flushing brought about by the pumping system are considered to be improvements and thus the resulting environmental impacts are all favorable.

Consideration of what will result with the pumping system off, for example due to mechanical failure, for an extended period of time should be made. Design of the pumping system with two independent
pump units each sharing the load is a distinct possibility. In the event of the failure of one of the units the flow through the system would not be reduced quite to 50% owing to the fact that the turbulent friction within the pipe is reduced by 50%. Some additional costs would be incurred by having two pumps rather than one larger one, but the backup feature would be desirable. A reduction of water flow to 50% doubles the residence time for a parcel of water in the canal which would cause a substantial change in the appearance of the water.

In the event that the pumping system had to be turned off for some reason for an extended period of time the appearance of the water in the canal would revert to the present status in a matter of days. However, the accumulation of contaminants would take longer to reach steady state. From the point of view of the ecology of the canal the long shut off would not be disastrous although there would start the immediate transformation of the population composition from what was present to what is now present.

Noise pollution from the pumping station for a submerged pump system is essentially zero. Atmospheric pollution resulting from the increased production of electricity to run the pump need not be considered here. Visual pollution by the pumping system itself is minimal in that the physical size of the pumping station is small occupying a space for a single pump system approximately 8 feet wide by 50 feet long of ordinary house height. A double pumping system would be proportionately longer or wider depending upon the configuration of the land size available. The pipeline itself would be submerged approximately 4 to 5 feet and would only be prominently visible on days of unusually clear water.

VII. Construction Costs*

Current estimates for the dredging costs for the Ala Wai Canal center around the figure of about $1.5 million. This is a separate figure independent of the cost of the pumping and flushing system. The pumping system will have a cost of approximately $30 to $35 a foot for 4-foot diameter pipe and approaching $50 a foot for the 5-foot diameter reinforced concrete pipe. This puts the piping cost alone in the range of half a million dollars and laying the pipe partially buried will cost approximately another $10 per foot which approaches $100,000 for that construction work. For the required flow of water of 10,000 m³ per hour or approximately 42,000 gallons per minute, a 400 hp. 88% efficient pump capable of delivering 42,000 gallons per minute at a head of 26 feet could be used for the Ala Wai operation. Such a pumping system is available at a currently quoted price of about $80,000. Two years is the expected delivery time for such a pump. Pumping station housing would have about 400 square feet of floor space, and, although the construction is simple, there is a requirement for an exceptionally sturdy foundation and some nearshore embankment and water level construction. Thus construction costs may run as high as an average of $50 per square foot or $20,000 for the

*An updated addendum of construction costs is attached on page A-23A
moderately small pump house required. Electricity costs vary with the cost of fuel and the instantaneous value is always available. Under schedule G 216,000 kw hours per month would cost about $6,300 per month including the current fuel oil adjustment. This is an annual cost of less than $75,000. Under schedule P which would be the more likely rate for the permanent system, the cost would be about $5,500 per month or $66,000 per year.

When the individual costs are summed up we see that we are approaching $1 million capital investment. Many other individual items will appear, for example, the valve that diverts water to the Manoa-Palolo Streams, the manifold which exhausts the water into the head of each of the canal ends, and various servicing ports on the pipeline which should be provided in intervals of perhaps 1,000 feet. The intake manifold near the entrance of the Ala Wai Yacht Harbor must have some safety features which will run the cost up to $1 million at current prices. The cost for a two unit pumping system was not obtained, however, this could be expected to add $40,000 to $50,000.

(In obtaining the various numbers for the pumping system informal discussions were held with various contractors and suppliers throughout the State of Hawaii. Their names and references are not quoted in this report because those interviewed requested that they not be quoted directly for estimates prepared so informally.)

VIII. Summary

When the authors originally considered the problem of writing a proposal for a flushing system for the Ala Wai Canal which would adequately maintain the waters at a safe and aesthetically pleasing level, it was believed that the most straightforward and sensible solution would be a pipeline in from Kuhio Beach. It would be the shortest route and thereby minimize pumping costs. Examination of the capital investment required particularly in the pipeline extending into the ocean, it was determined that the capital costs are so high that that engineering solution could never be justified as compared to the pipeline down the length of the canal. A pipeline the length of the canal has the further advantage that it is possible to put a branch on up the Manoa-Palolo arm of the Ala Wai, thereby flushing it adequately. We believe that this project would be cost effective although there is of course a subjectiveness that comes with evaluating the benefits of the opening up of a new recreational area.

The authors wish to thank Dr. Richard Marland for his encouragement and patience during the production of this report. We thank Ms. Louise Lembeck for editorial assistance. Ms. Jacquelin Miller has contributed suggestions and technical comments throughout the production of this proposal.
Addendum to VII Construction Costs, Section A
Revised Estimates, December 1976, J. N. Miller

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IX. References


Figure 1—Map of the Ala Wai Canal showing the action of nine interconnected units, or boxes, used in the model.
Figure 2.--Elements of the Ala Wai Canal with the various inputs, outputs, and mixing coefficients shown schematically. The mixing coefficients are adjusted so that the salinity, which is a balance between the influx of fresh water and the mixing of salt water within the canal, gives a residence time for the water in the canal.

Legend

- $V_k$ = The nine volumes of this Ala Wai box model.
- $\alpha$ = Pump inflow at Kapahulu.
- $\beta$ = The flow of the Manoa-Palolo Streams.
- $\gamma$ = Pump inflow in the Manoa-Palolo Streams.
- $\delta$ = The flow of the Makiki drainage system.
- $\gamma_k$ = The storm drain inflows.
- $\xi$ = The diffusion coefficients.
- $k$ = The flow from one box to its interbox flow index. $k$:
  1. $\alpha + \gamma_1 + \xi_1$
  2. $\alpha + \gamma_1 + \gamma_2 + \xi_2$

Legend--Continued:

- $3: \alpha + \gamma_1 + \gamma_2 + \gamma_3 + \xi_3$
- $4: \alpha + \gamma_1 + \gamma_2 + \gamma_3 + \gamma_4 + \xi_4$
- $5: \alpha + \gamma_1 + \gamma_2 + \gamma_3 + \gamma_4 + \gamma_5 + \beta + \beta_L + \xi_5$
- $6: \alpha + \gamma_1 + \gamma_2 + \gamma_3 + \gamma_4 + \gamma_5 + \gamma_6 + \beta + \beta_L + \xi_6$
- $7: \alpha + \gamma_1 + \gamma_2 + \gamma_3 + \gamma_4 + \gamma_5 + \gamma_6 + \beta + \beta_L + \xi_7$
- $8: \alpha + \gamma_1 + \gamma_2 + \gamma_3 + \gamma_4 + \gamma_5 + \gamma_6 + \gamma_7 + \beta + \beta_L + \theta + \xi_8$
- $9: \beta + \beta_L + \xi_9$
Figure 3.—Computer model figure showing salt water concentrations as a function of time in hours. At time zero the canal (consisting of nine boxes in this model) is taken as filled with fresh water or zero concentration of seawater. As time progresses the steady state solution to the nine equations is approached.
Figure 4.—Computer model figure showing salt water concentrations as a function of time in hours. At time zero the canal (consisting of nine boxes in this model) is taken as filled with one-third seawater.
Figure 5.—Computer model figure showing salt water concentrations as a function of time in hours. At time zero the canal (consisting of nine boxes in this model) is taken as filled with two-thirds seawater; this is a condition which can occur in some parts of the canal after a heavy storm.
Figure 6.—Computer model figure showing salt water concentrations as a function of time in hours. At time zero the canal (consisting of nine boxes in this model) is taken to represent average salinities as initial concentrations. The high seawater concentrations after a day reflect the expected improvement in water quality.
Figure 7.--Schematic of standing wave which would support a limited number of surfers.
Table 1.—Physical dimensions of the nine-box model of the Ala Wai Canal.

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*Numbers indicate (see Figure 1):
Box No. 1 represents the Kapahulu end of the canal.
Box No. 5 is that part of the canal which intersects with the mouth of the Manoa-Palolo Drainage Canal.
Box No. 8 is the mouth of the canal.
Box No. 9 is the Manoa-Palolo Stream.
Table 2.—Equivalent loss of head in feet for 60,000,000 gallons per day per 1,000 feet.

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Table 3.—Productivity index given in milligrams of carbon per milligrams of chlorophyll a per hour (Harris, 1975: Table 5)

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SECTION B

SEDIMENTATION PROCESSES OF THE ALA WAI CANAL

by

Stephen Wheatcraft
University of Hawaii
I. Introduction

Purpose and Objectives

The proposal to improve the quality of waters in the Ala Wai Canal by introducing a large-scale flushing system will have various effects on the water quality and sedimentation patterns of the canal system. The proposed flushing system will provide a flow rate of approximately 60 million gallons per day (mgd), thus achieving an average residence time of about 2 days for the canal waters (Miller and Chave, 1976). This flow would be sufficient to flush the Ala Wai Canal of nutrients and contaminants producing an appearance similar to that of off-shore waters at the entrance of the Ala Wai Boat Harbor.

Water quality studies for the proposed improvement have been conducted by Miller and Chave (1976). Another problem related to the flushing system which needs to be addressed is the question of what effect the proposed flow will have on sedimentation rates in the canal. Specifically, several of the questions which need to be answered are:

1. Will the flow rate be large enough to cause saltation or transport of suspended load?
2. If sediment transport will occur, how much material will be carried out and where will it be deposited?
3. What effects will the transported sediment (if any) have on the area of deposition?
4. How will the flushing system affect the frequency of dredging the Ala Wai Canal?

Scope of Investigation

Research was limited primarily to the use of existing data from the literature on the nature of sediments in the canal and the Manoa-Palolo stream, which provide the main influx of sediments to the canal.
A limited amount of field work was done to determine the types of sediments that will be encountered during dredging.

Review of Literature

Research on the Ala Wai Canal has primarily been limited to studies of water quality and physical oceanography and therefore very little information is available on the canal sediments or the sediments carried into the canal by the Manoa-Palolo stream. Gonzales (1971) did a descriptive study of the physical oceanography of the canal but included no sediment data. Similarly, Cox and Gordon (1970) discuss various biological aspects of water quality but includes no sediment data. The only useful sediment data that is available is from Ching (1972) which gives a considerable amount of data on suspended solids in Manoa stream from August 1970 to May 1971. This data will be employed in the current analysis because it provides good knowledge of the daily sediment loads being carried into the Ala Wai Canal by the Manoa stream.

II. Sedimentation Rates in the Ala Wai Canal

Drainage Area and Runoff

The drainage area for the Ala Wai Canal includes the watersheds for both the Manoa and Palolo streams. Runoff from natural and urbanized areas enter the canal through a combination of natural tributaries and man-made diversions such as catchbasins, storm drains and drainage ditches. The watershed areas for these two streams are shown in Figure 1.

Records of streamflow have been kept for both Manoa and Palolo streams for many years. Cox and Gordon (1970) gave a figure of combined (Manoa and Palolo) mean flow into the canal of 12.7 cubic feet per second (cfs), or 0.360 cubic meters per second (m³/sec). This figure is considered low, however, and records from Ching (1972) will be used in this report, primarily because the sediment data presented by Ching corresponds to the streamflow measurements for his
study time period. The average combined flow for this period is given to be 24.34 cfs (0.689 m³/sec). This is the figure which will be used for calculating sedimentation rates.

**Suspended Load Data for the Canal**

Ching (1972) performed suspended solids analyses for Manoa Stream for a nine-month period beginning August 6, 1970 and ending May 4, 1971. Samples were taken from six sample stations which ranged in location from upper Manoa Valley to a location quite near the Ala Wai Canal. Ching's sample station 6 is located in the combined Manoa-Palolo Stream just 100 yards from the mouth which empties into the canal. Since this station represents virtually the total flow and sediment load for the Manoa-Palolo watersheds, the data from station 6 will be used here to compute sediment loads. This data is presented in Table 1. Ching arrived at an average value of suspended solids for Station 6 of 30.3 mg/l.

However, this figure appears to be in error because this worker has repeatedly averaged the same data and arrived at a value for average suspended solids of 60.38 mg/l.

By knowing the average flow rate and the average concentration of suspended solids, one can calculate the mass flow rate of suspended solids into the canal from the stream. This value turns out to be $1.31 \times 10^9$ gm/year. If an average density of 2.0 gm/cm³ is assumed, then the volumetric flow rate of sediments into the canal is 656 m³/year. Miller and Chave (1976) report a sedimentation rate of 2000 m³/year. It is reasonable that this figure would be larger than that of the suspended solids alone because waste product from phytoplankton production in the canal itself must also contribute to the sedimentation rate. Furthermore, it will be shown later that the suspended solids from Manoa and Palolo Streams are most likely entirely separate from the 2000 m³/yr. sedimentation rate reported by Miller and Chave (1976).

Sediment rating curves (Ekern, 1976) exist for several Oahu streams but unfortunately, not for Manoa or Palolo streams. The nearest stream that has a sediment rating curve is Kalihi Stream. Kalihi Stream would produce about 38,000 m³/year of sediment at a flow rate similar to the average flow rate for the Manoa-Palolo streams that was used in these calculations. There is a marked difference in the sediment flow rates of Kalihi Stream and the Manoa-Palolo
stream, but their watersheds are also very different in nature (primarily degree of urbanization) and there is no particular reason to think their sedimentation rates should be similar.

III. Effects of the Proposed Flushing System on Canal Sedimentation

Stream Hydraulics

The proposed flow rate in the canal would be approximately 60 million gallons per day (mgd) (10,000 m$^3$/hr). This design flow rate will reduce original concentrations to 10% in 48 hours and was chosen to provide optimum flushing versus the cost of pumping. An average velocity (specific discharge) can be calculated by dividing the flow rate by the cross-sectional area of the canal. Table 2 provides data on the geometry of the canal which was used for these calculations. In reality, the particle velocity is a function of position with respect to the channel, but as a rough rule of thumb, velocities in the center of the channel will be about twice the specific discharge and velocities near the bottom and sides of the channel will be about 1/2 the specific discharge. The average cross-sectional area of the canal (after dredging) is 90 m$^2$ (50m wide by 1.8m deep) and the resulting specific discharge is then about 3 cm/sec. Hjulstrom (1935) provides an erosion and deposition curve for velocity versus grain size (figure 2). A velocity of 3 cm/sec will transport particles with grain sizes of up to about 0.2mm. Recent sediment samples taken from the Ala Wai Canal indicate that most of the sediment is silt-size (less than approximately 0.05mm). It therefore seems reasonable to assume that if the sediment particles in suspension are this small when they reach the canal, they will then remain in suspension and be transported through the canal and into the ocean.

There is some evidence to suggest that when the fresh water from the stream mixes with the salt and brackish water, flocculation of the sediments will occur, producing relatively large particles which would, at these velocities, be deposited in the canal. A considerable research effort would be required to gain a handle on the possibility of this flocculation occurring and is beyond the scope of this report.
Effects on the Canal

Miller and Chave (1976) state that the sediment deposition rates in the canal are approximately $2000 \text{ m}^3/\text{yr}$. Data from Ching (1972) indicate that a similar volume of material is being provided by the normal daily flow of suspended sediment from the Manoa and Palolo Streams. Since this material is small enough to remain in suspension at the design flow rate of the canal, it appears likely that most of the sediment will be carried on through by the flushing system and will not be deposited in the canal. It must be emphasized, however, that this conclusion is based on the supposition that the sediment load will remain small in particulate size (less than 0.1mm) after it mixes with the brackish water in the canal.

However, it is likely that this suspended load is already being carried intact through the canal and into the ocean. The average combined total discharge for Manoa and Palolo streams during the same period of time of Ching's study is 24.34 cfs (0.689 m$^3$/sec). This figure yields a specific discharge through the canal of 0.77 cm/sec. It should be noted that the actual velocity at any point where the water is moving in the canal would be higher than 0.77 cm/sec. The velocity that will occur under the proposed flushing system will be nearly four times this much. From figure 2, it can be seen that velocities of 0.77 cm/sec can suspend particles of up to 0.1mm in diameter while velocities of 3 cm/sec (the velocity of the flushing system) will suspend particles of up to 0.3mm in diameter. Since the suspended load is probably considerably smaller than 0.1mm in size, it is most likely that the present flow rate in the canal is already carrying these sediments out to sea. This is a strong indication that the 2,000 m$^3$/yr sedimentation rate is caused by factors other than the suspended sediments carried by the Manoa and Palolo Streams. Since the sediments are highly organic in nature (69% organic), they may be caused primarily by the biological activity in the canal (phytoplankton growth, etc.).

It should be noted also that neither the current natural flow rate nor the proposed flushing rate are capable of picking up sediments from the canal bottom. The velocities are (and will be) much too low for bed erosion to occur. This can be seen from Figure 2.
IV. Conclusions

Under the proposed flushing systems, the flow rate will be large enough to transport suspended sediments assuming that flocculation does not occur, but this condition will not be any different from the present, since the current flow rates in the canal already carry the suspended sediments through the canal and out to sea. However, the flow rates will not be anywhere near large enough to cause bed erosion (saltation). The flushing system will therefore have little if any effect on the nature of sediment transported in the canal or in the Ala Wai Boat basin.

Sediment deposition in the canal may be reduced considerably if most of the 2,000 m$^3$/hr. sedimentation rate is being caused by biological activity, because this activity will be greatly reduced under the flushing system. One result of this would be to reduce the frequency with which the canal would need to be dredged.

V. References Cited


Ekern, P., 1976. Information communicated prior to publication.


Table 1
(from Ching, 1972)

TEST: Suspended Solids mg/l

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|        | AVE   | 9.64     | 11.5      | 13.6      | 13.7      | 22.8      | 30.3*     |
| STORM AVE | 108.5 | 99.0     | 86.5      | 140.0     | 128.0     | 176.5     |
| AVE F | 151.0 | 50.0     | 146.0     | 252.0     | 438.0     | 800.0     |
| AVE P | 95.5  | 217.5    | 219.0     | 160.0     | 74.0      | 16.0      |
| AVE T | 209.0 |          |           |           |           |           |

| URBAN AREA | 0.0 | 5.0 | 27.2 | 27.9 | 30.8 | 31.4 |

* The value given by Ching of 30.3 appears to be in error. I have calculated the average to be 60.38.
TABLE 2  
(from Gonzalez, 1971)  
AREA, VOLUME, MEAN DEPTH, AND DIMENSIONS OF THE ALA WAI CANAL

<table>
<thead>
<tr>
<th>Area</th>
<th>Length</th>
<th>Volume</th>
<th>Surface area</th>
<th>Mean depth</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Ala Wai Canal:</strong></td>
<td>$3.1 \times 10^3$ m</td>
<td>$3.6 \times 10^5$ m$^3$</td>
<td>$2.0 \times 10^5$ m$^2$</td>
<td>1.8 m</td>
</tr>
<tr>
<td><strong>Basin:</strong></td>
<td>$1.3 \times 10^3$ m</td>
<td>76 m</td>
<td>$1.9 \times 10^5$ m$^3$</td>
<td>$8.4 \times 10^4$ m$^2$</td>
</tr>
<tr>
<td><strong>Sill:</strong></td>
<td>$9.7 \times 10^2$ m</td>
<td>76 m</td>
<td>$7.3 \times 10^4$ m$^3$</td>
<td>$7.4 \times 10^4$ m$^2$</td>
</tr>
<tr>
<td><strong>Small Basin:</strong></td>
<td>$2.5 \times 10^2$ m</td>
<td>50 m</td>
<td>$2.3 \times 10^4$ m$^3$</td>
<td>$1.2 \times 10^4$ m$^2$</td>
</tr>
</tbody>
</table>
TABLE 2 (Continued) AREA, VOLUME, MEAN DEPTH, AND DIMENSIONS OF THE ALA WAI CANAL

<table>
<thead>
<tr>
<th>Channel:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length:</td>
<td>$6.4 \times 10^2$ m</td>
</tr>
<tr>
<td>Width:</td>
<td>50 m</td>
</tr>
<tr>
<td>Volume:</td>
<td>$7.8 \times 10^4$ m$^3$</td>
</tr>
<tr>
<td>Surface area:</td>
<td>$3.2 \times 10^4$ m$^2$</td>
</tr>
<tr>
<td>Mean depth:</td>
<td>2.5 m</td>
</tr>
</tbody>
</table>
SECTION C

WATER QUALITY CHARACTERISTICS
OF THE ALA WAI CANAL AND
THE EFFECTS OF MECHANICAL FLUSHING

by
David Bills
University of Hawaii
TABLE OF CONTENTS

OBJECTIVE

BACKGROUND 1

PART I - PRESENT WATER QUALITY EVALUATION

Introduction 5
Microbiological Requirements 6
pH Units 9
Nutrient Material 10
Dissolved Oxygen 12
Total Dissolved Solids, Salinity, and Currents 13
Temperature 14
Turbidity 15
Sulfide 16
Carbon Dioxide 16
Suspended Solids 16
Metals 17
Pesticides 18
Summary 18

PART II - EFFECTS OF FLUSHING ON WATER QUALITY 19

Summary 34

REFERENCES 37

APPENDIX A

APPENDIX B
OBJECTIVE

The objective of this report is two-fold:

1) To establish pre-dredging and pre-flushing water quality background information in the Ala Wai Canal and immediate offshore waters.

2) To determine the effects of the proposed flushing system on these existing water quality parameters.

The first objective has been met by compiling all existing water quality data available for the Ala Wai Canal proper as well as data for the Ala Wai Boat Harbor and the Diamond Head end of Ala Moana Beach Park. Specific interest was directed at parameters cited in the State Water Quality Standards (Hawaii Public Health Regulations, Chapter 37-A). Additional parameters, where data existed, were included in this report. Part I deals solely with this first part of the study objective.

Part II is an evaluation through a mixing and flushing analysis of the Canal with an additional flow of 10,000 m³/hr entering the Canal at its Diamond Head end.
BACKGROUND

For simplicity in presenting Water Quality data from a varied group of sampling programs, Figures 1, 2, and 3 reference all major testing programs. Four testing schemes representing the Department of Health (State of Hawaii), Frank Gonzalez (1971), Carol Harris (1975), and Jacquelin N. Miller (1975) (University of Hawaii - Oceanography) have been located on Figure 1. The Department of Health sampling stations (320, 321, and 360) are permanent stations. Monitoring stations established by Frank Gonzalez, Carol Harris, and Jacquelin Miller were for short term sampling periods.

Figure 2 represents five stations utilized in an evaluation of the Ala Wai Boat Harbor. Lawrence P. Raymond of Oceanic Institute authored this report entitled the Final Environmental Statement, Administrative Action for Improvement at the Ala Wai Boat Harbor (1972). Monitoring at these stations was on a short term basis.

Figure 3 represents monitoring stations on Manoa Stream. This work was conducted in conjunction with a Water Resources Research Center (State of Hawaii) project principally investigated by L. Stephen Lau. This project was published as Chun, Young, and Anderson (1972), WRRC Technical Report no. 63.

These six sources represent the significant amount of Water Quality data available for compilation. Additional sources consulted for specific water quality parameters were Alan Eschleman (University of Hawaii, Thesis on Lead and Mercury, 1973) and Cynthia Schultz's Hawaii Institute of Marine Biology report (1971) on pesticides.
Figure 1. Chart showing the location of stations on the Ala Wai Canal.
Fig. 2  Current measurement stations, Ala Wai Boat Harbor

Oceanic Institute (1972)
LEGEND:
- SAMPLING SITES
- STUDY AREA

FIGURE 3
LOCATION OF STUDY AREA AND SAMPLING SITES, MANOA WATERSHED, OAHU.
PART I

WATER QUALITY BACKGROUND

This portion of this report is directed solely at summarizing water quality background information for the Ala Wai Canal, the Ala Wai Boat Harbor and "near shore waters" immediately adjacent to the confluence of the Ala Wai Canal system and Ala Moana Point. All monitoring reports are included in Appendix A of this report. These specific monitoring reports are referenced throughout the text to allow more coherent examination of specific data. The text portion of this report represents, section by section, the parameters cited in Chapter 37-A Water Quality Standards. Prefacing each section is a quantitative statement of Chapter 37-A limitations. Following, is a discussion of available data and an evaluation, respectively.
Microbiological Requirements

A. Limitations

<table>
<thead>
<tr>
<th></th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Coliform</strong></td>
<td>1000/100ml</td>
<td>2400/100ml</td>
</tr>
<tr>
<td>(Class A, 1, and 2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Fecal Coliform</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Class A)</td>
<td>200/100ml</td>
<td>400/100ml</td>
</tr>
<tr>
<td><strong>Fecal Coliform</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Class B)</td>
<td>400/100ml</td>
<td>1000/100ml</td>
</tr>
</tbody>
</table>

B. Discussion

The Department of Health has been sampling the Ala Wai Canal, Boat Harbor and "near-shore waters" (Ala Moana Beach Park) since 1955. Sampling frequency has varied between a minimum of ten annual samples to fifty annual samples. Sampling sites have been limited to three sites in the Canal and Basin and three sites off Ala Moana Beach Park (See Figure 1). Based on Department of Health records, waters at station 320 (Ala Moana Bridge) and station 321 (McCully Street Bridge) have violated the median total coliform count on an annual basis in sixteen of the last twenty-one years of recorded data. As a general trend violations have been greater at station 321. The mean total coliform counts at stations 321, 320, and 360 have been 21,480, 13,413, and 8715 per 100ml sample, respectively (Appendix A-1, 2, and 3).

Department of Health records regarding fecal coliform bacteria have only been kept since 1969. As with total coliform monitoring, the number of annual samples has fluctuated between ten and fifty. The median fecal coliform count at stations 320, 321, and 360 has approximated 300 per 100ml sample.
Stations 321 and 360 have exceeded the 200/100 ml limitation in every year but one on an annual basis. Station 320 has never achieved the average fecal coliform limitation on an annual basis since monitoring began in 1969. The range of fecal coliform counts at all interior monitoring stations has been 0-240,000/100 ml. The range of annual means for all three stations has fluctuated from 616/100 ml to 12,350/100 ml (Appendix A-4,5, and 6).

In contrast to high bacteriological counts within the canal and basin, the near-shore waters represented by Ala Moana Beach Park stations, exhibit virtually no coliform bacteria. Department of Health records indicated no violations at any of the three Beach Park stations on an annual basis.

Since Department of Health data is limited to the lower third of the Canal, additional data reflecting the upper portion of the Canal (Diamond Head of McCully Street Bridge) was located. The only direct information available was reported by Gonzalez (1971). This sampling was limited to one date. Monitoring stations utilized were presented in Figure 1. The results indicated a minimum of approximately 1000/100 ml total coliform bacteria immediately upstream of the Manoa-Palolo junction with the remainder of the Canal exhibiting counts not less than 3000/100 ml (Appendix A-7). Gonzalez' work illustrated similar findings of the same magnitude for fecal coliform monitoring. Fecal coliform monitoring surpassed 2,500/100 ml by the Manoa-Palolo outfall (Appendix A-8). This report while only exerting one day's monitoring, did conclude that Manoa-Palolo Streams as well as Apukehau Stream were major factors in excessive bacteria levels in the Ala Wai Canal system.
Verification of Gonzalez conclusion was published in 1972 by the University of Hawaii Water Resources Research Center (Wastewater Effluents and Surface Runoff Quality, Technical Report Number 63). This study monitored the Manoa Stream at six points illustrated in Figure 3. Monitoring revealed a total and fecal coliform count of approximately 20,000 and 900/100 ml, respectively. These counts further appeared to be uniform regardless of dry and wet weather conditions. Total bacteria counts were also measured along the length of the Stream. The average count approached 100,000/100 ml (Appendix A-9,10, and 11).

The final source of bacteriological data available was prepared by Lawrence P. Raymond in 1972 (Final Environmental Statement for Administrative Action for Improvements at Ala Wai Boat Harbor, Oceanic Institute). This study dealt in depth as to the status of the Boat Harbor only. Top and bottom samples were taken for one month at all stations (Figure 2) as a function of different tidal stages. The range of total coliform density was between 50 and 4000/100 ml. Fecal coliform density was between 0 and 600/100 ml. Since the monitoring period was relatively short it is difficult to reach firm conclusions. Noticeable though was the fact that both total and fecal coliform counts consistently met Class A Water Quality Standards. With one exception the only violations occurred on outgoing tides when brackish water flow from the drainage canals is accentuated (Appendix A-12, 13, 14, 15, and 16).

C. Evaluation

Based on the existing data the Ala Wai Canal violates Class A standards nearly 100 percent of the time by an approximate factor of seven. The major source of contamination appears to be all connecting canals and outfalls collecting runoff typical of urbanized areas. Contamination further appears
to be concentrated in the upper one meter of the canal due to fresh water-salt water stratification (Gonzalez, 1971). The portion of the canal exhibiting the least abundance of bacteriological activity coincides with the Diamond Head end of the Canal upstream of all drainage systems.

Based on the combined monitoring of the Department of Health and Lawrence Raymond (1972), the Boat Harbor appears to meet Class B Standards 50 per cent of the time. The coliform data is influenced by sampling point as well as the direction of tidal flow.

Near shore waters reveal no violations and meet Class AA Standards of 70/100 ml, average and 230/100 ml, maximum, for total coliform bacteria, respectively.

**pH Units**

A. Limitations

Classes A, B, and 1

\[ 7.0 - 8.5 \]

B. Discussion

Virtually no pH data is available for the Ala Wai Canal system. One random sample at the Department of Health station 321 exhibited a pH of 7.8. Other than this the only data available is for Manoa Stream and not the Canal (WRRC, Technical Report Number 63). Stream sampling revealed a pH range of 6.9 to 7.6 with a mean of 7.25. One reason for scarce pH monitoring probably is due to all inputs to the canal are "natural" (no industrial inputs). Therefore any pH monitoring would reveal natural conditions (Appendix A-17).

C. Evaluation

Based on little conclusive monitoring it appears the Canal meets Class A Standards in all sections. The same is assumed to hold true in the boat Harbor.
Nutrient Material

A. Limitations

<table>
<thead>
<tr>
<th>Nutrient</th>
<th>Limitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Phosphorus</td>
<td>0.025 mg/l</td>
</tr>
<tr>
<td>(Class A)</td>
<td></td>
</tr>
<tr>
<td>Total Phosphorus</td>
<td>0.030 mg/l</td>
</tr>
<tr>
<td>(Class B)</td>
<td></td>
</tr>
<tr>
<td>Total Nitrogen</td>
<td>0.15 mg/l</td>
</tr>
<tr>
<td>(Class A)</td>
<td></td>
</tr>
<tr>
<td>Total Nitrogen</td>
<td>0.20 mg/l</td>
</tr>
<tr>
<td>(Class B)</td>
<td></td>
</tr>
</tbody>
</table>

B. Discussion

Three sources of available data pertaining to phosphorus and forms of nitrogen have been located for this study. These are: Frank Gonzalez (1971), Oceanic Institute (1972), and WRRC (Tech. Report No. 63, 1972). The sum of all monitoring periods was less than two months. The more significant monitoring occurred in the Boat Harbor and Manoa Stream with only one day's sampling available for the Canal proper.

Oceanic Institute's sampling of the Boat Harbor reported an average total phosphorus concentration of 9.042 mg/l and a nitrate-nitrite concentration of 0.206 mg/l. Data indicates higher concentrations of both parameters closer to the Canal with lower concentrations nearer the ocean-side of the Basin. It must be kept in mind that the nitrate-nitrite levels do not include ammonia and organic-nitrogen determinations and therefore the 0.206 mg/l average would in all probability be higher on a total nitrogen basis. The Oceanic Institute study suggested a 90% reduction in total phosphorus would bring both the Canal and Boat Harbor within existing limitations (Appendix 18, 19, and 20).

The 1972 WRRC Technical Report No. 63 evaluated total phosphorus and
nitrates in Manoa Stream. The range of phosphorus was 0.23 - 0.43 mg/l and 0.56 - 0.65 mg/l in dry and wet weather, respectively. The range of nitrates (NO$_3$ - N) was 0.74 - 1.78 and 1.9 - 3.5 mg/l, respectively (Appendix A-17). These results are graphically illustrated in Appendix A-21 and 22. Assuming Manoa Stream runoff is typical of all similar Canal runoff the Canal receives a substantial load of both phosphorus and nitrogen from secondary drainage systems connected to the Canal. Based on an average annual stream runoff of 0.4m$^3$/sec (Gonzalez, 1971) the Ala Wai Canal receives 30 and 75 pounds daily of phosphorus (total) and nitrogen (NO$_3$ -N), respectively. At peak wet weather conditions this nutrient loading rate can be increased by a factor of 'SO.

Frank Gonzalez's monitoring of the Ala Wai Canal included total phosphate, nitrate, and nitrite. Longitudinal sections of the Canal are presented in Appendices A-23, 24, 25, 26, and 27. For all parameters the highest concentrations are found in the surface portions of the Canal and nearer drainage canal junctions. Below 2 meters depth the concentration of these nutrients drops by a factor of ten as compared to surface samples. The ranges of phosphate, nitrate, and nitrite are 0.09 - 0.019 mg/l, 3.1 - 0.3 mg/l, and 0.37 - 0.092 mg/l, respectively. Once again for complete understanding of this work it must be kept in mind that these samples represent only a portion of the total phosphorus and nitrogen loads.

C. Evaluation

The Boat Harbor and Canal, based on the data of this report exceed Chapter 37-A Standards nearly 100% of the time. Violations can be as high as by a factor of 100. Manoa-Pa'alelo Streams and Apukehau Stream are major sources of these nutrient loads along with the Ala Wai Golf Course located
at the Diamond Head end of the Canal (Gonzalez, 1971). The major load of phosphorus and nitrogen exist in the upper one meter of the Canal's water.

**Dissolved Oxygen**

*A. Limitations*

<table>
<thead>
<tr>
<th>Class A and 2</th>
<th>Not less than 5.0 mg/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class B</td>
<td>Not less than 4.5 mg/l</td>
</tr>
</tbody>
</table>

*B. Discussion*

Results of monitoring by Oceanic Institute (1972) indicate that at all depths and at any tidal flow the Harbor Achieves Class A Standards. This data is in Appendix A-28 through 35.

The work done by Miller and Gonzalez in the canal indicates the top two meters of water comply with Class A standards. Rarely does the dissolved oxygen drop below 6.0 mg/l. Below a depth of two meters though, the dissolved oxygen can fluctuate between 0.0 mg/l and 6.0 mg/l. Since the major portion of the Canal exceeding a depth of two meters exists Diamond Head of the Manoa-Palolo Drainage Canal most of the near anoxic conditions occur in this area. Both authors cite the major cause of this condition as due to poor circulation and entrapment of denser saline water in this upstream basin. Appendix A-36, 37, and 38 numerically compare surface and bottom water dissolved oxygen samples (Miller, 1975). Appendix A-39 through 44 graphically portray longitudinal dissolved oxygen samples compiled by Gonzalez (1971).
C. Evaluation

The Ala Wai Canal system conforms to Chapter 37-A dissolved oxygen standards nearly 100% of the time in all areas except one. This area exists Diamond Head of the Manoa-Palolo Drainage Canal junction. In this region and at a depth greater than 2 meters anoxic conditions exist, conservatively, 50% of the time. Poor circulation in this Canal section appears to be the major factor for this condition.

Total Dissolved Solids, Salinity, and Currents

A. Limitations

Class AA (Only)

No changes in channels, in basin geometry of the area, or in freshwater influx shall be made which would cause permanent changes in isohaline patterns of more than ± 10% of naturally occurring variation or which would otherwise affect biological and sedimentiological situation. Total dissolved solids shall not be below 28,000mg/l from other than natural causes.

B. Discussion

Of the three parameters mentioned above, salinity is the only parameter for which enough information is available to evaluate. As expected the Canal system exhibits a salinity range from that typical of seawater (34.7 °/oo) to that typical of fresh water. Salinity data can be found in Appendix A-28 through 35 as well as Appendix A-45 through 56. Salinity, as indicated by Chapter 37-A limitations is not a major factor in dictating water quality in the Canal system. Salinity on the other hand, lends itself as an ideal parameter to measure displacements of water as a function of tides. Appendix A-48 through 56 are particularly geared to this type of study.
C. Evaluation

Stratification occurs at approximately the one meter depth. Rarely does water above this depth achieve a salinity of 31°/oo. Water below this level fluctuates between 32.5°/oo to 34.5°/oo. This stratification is a major cause of stagnation of water in the deeper basin upstream of the Manoa-Palolo Drainage Canal. Water below two meters can only escape from this basin when more dense water spills over a sill built up by sedimentation from Manoa and Palolo Streams. Thus while salinity is not violating Water Quality Standards, it is an indirect cause of dissolved oxygen violations in this part of the Canal.

Temperature

A. Limitations

Classes AA, A, B, 1, and 2

Temperature of the receiving waters shall not change more than 1.5° C from natural conditions.

B. Discussion

All discharges into and resulting from the Ala Wai Canal system must be referred to as "natural" (assuming only runoff discharges), therefore this section only attempts to document characteristic temperatures of the Canal and no evaluation will be included.

Temperature in the Ala Wai Canal fluctuates between 21°C and 28°C. Typical seawater infiltrating the Canal system approximates 27.5°C while runoff originating in the cooler mountain regions approximates 21°C thus accounting for this temperature range. Winds, tidal flows and fluctuations in ambient air temperatures affect daily variations as would be expected. As with salinity, temperature profiles (Appendix A-60 through 69) provide
insights as to mass water displacements. Additional temperature data can be found in Appendix A-28 through 36 and A-57 through 59.

**Turbidity**

**A. Limitations**

Classes AA, A, B, 1, and 2

Secchi disc or secchi equivalent as "extinction coefficient" determinations shall not be altered from natural conditions more than 5% for Class AA or 1 waters, 10% for Class A or Class 2 waters, or 20% for Class B waters.

**B. Discussion**

Information available discussing turbidity was done in conjunction with the WRRC Technical Report No. 63 and the work of Harris (1975). Appendix A-17 cites the average turbidity (APHA TU) as 3.1 and 51 in Manoa Stream under dry and wet weather conditions, respectively. Average extinction coefficients (k) reported by Harris at 3 stations (Appendix A-17) yielded values ranging from 1.64 to 2.53 (k = 1.7/D, where k is the extinction coefficient and D is the secchi depth in meters).

**C. Evaluation**

Turbidity in the Ala Wai is largely related to excessive phytoplankton production and quantities of suspended solids (Harris, 1975). Inasmuch as all discharges into the Ala Wai result from "natural" runoff it is not possible to apply the Department of Health turbidity standard which states that "extinction coefficient" (turbidity) "determinations shall not be altered from natural conditions more than 5% for Class AA or Class 1 waters, 10% for Class A or Class 2 waters, or 20% for Class B waters."
The following section includes parameters other than those cited in Chapter 37-A but are available as a result of this review.

**Sulfide**

Oceanic Institute monitored H₂S in the Ala Wai Boat Harbor. Median levels ranged from 60 ug/l to 0.0 ug/l (as S) at stations 1 through 5 (See Figure 2). The Institute indicated the source as originating in the Canal or outside the Harbor (Appendix A-18, 19, and 20).

**Carbon Dioxide**

Oceanic Institute monitored CO₂ levels in the Boat Harbor and found a median concentration of approximately 2.0 m-moles/l CO₂ and 0.60 m-moles/l CO₂ uniformly dispersed in the Harbor on rising and falling tides, respectively. The data indicate that high natural productivity is responsible for the low concentration of carbon dioxide; not the reverse (Oceanic Institute, 1972) (Appendix A-18, 19, and 20).

**Suspended Solids**

Two days of suspended solids sampling (Gonzalez, 1971) indicated a range of 30 to 90 mg/l suspended solids in a longitudinal section of the Canal (Appendix A-70 and 71). WRRC Technical Report No. 63 indicates the input from Manoa Stream approximates 20 mg/l and 110 mg/l in dry and wet weather conditions, respectively (Appendix A-72).

**Metals**

The majority of metal analysis available for the Ala Wai Canal system was conducted by Alan M. Fshleman's, *A Preliminary survey of Lead and Mercury in the Hawaiian Environment* (August, 1973).
Metals examined were lead, zinc, copper, cadmium, and mercury. The sampling stations utilized in this survey are presented in Appendix A-73 and 74. Sampling was conducted on both water and the sediments. Out of three sampling dates lead and cadmium never exceeded detectable limits of 0.01 and 0.005 μg/ml, respectively. Mercury surpassed detectable limits only once with a sample registering 0.0007 μg/ml. Sampling points for these water analyses were the Texaco Dock, Ala Moana Bridge, and the Manoa-Palolo Drainage Canal (Appendix A-75).

Sediment analysis on lead revealed a mean content of approximately 66.0 ppm. As a general trend lead sediment concentrations increased in the landward portion of the Canal.

Zinc and copper monitoring, performed only one day, extracted mean sediment content of approximately 181.0 and 50.0 ppm, respectively. Zinc concentrations appear to be higher in the Harbor while copper monitoring showed the same such general trend (Appendix A-75 and 76).

Cadmium monitoring was conducted on three dates. Average sediment content approached 2.0 ppm. Both highest and lowest readings occurred in the Harbor thus making source location difficult (Appendix A-75 and 76).

Mercury monitoring for three days yielded a mean sediment content of 0.26 ppm (July 7 and 14) and 0.56 ppm (March). All sampling indicated highest levels in the Harbor (Appendix A-75 through 80).
Pesticides

Pesticide monitoring of the Ala Wai Canal was conducted by Cynthia Schultz (1971). No monitoring of the Boat Harbor was done in conjunction with this report. Pesticides analyzed were DDE, Dieldrin, DDT, and DDD. Water samples, sediment samples, and assorted fish and algae samples were tested. A summary of this work is tabulated in Appendix A-80. Results of this study indicated that pollution of the Canal by organochlorine pesticides does not occur to any significant degree. (Bevenue and Hylin, 1972).

Summary

The following table summarizes the findings of section I.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Present Range</th>
<th>Maximum Limitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Coliform</td>
<td>8,000-20,000/100 ml</td>
<td>2,400/100 ml</td>
</tr>
<tr>
<td>Fecal Coliform</td>
<td>616-12,000/100 ml</td>
<td>400/100 ml</td>
</tr>
<tr>
<td>pH</td>
<td>7.0 - 8.5</td>
<td>8.5</td>
</tr>
<tr>
<td>Phosphorus (as P)</td>
<td>0.04 - 0.1 mg/l</td>
<td>0.025 mg/l</td>
</tr>
<tr>
<td>Nitrogen (as N)</td>
<td>0.20 - 1.0 mg/l</td>
<td>0.20 mg/l</td>
</tr>
<tr>
<td>Dissolved Oxygen</td>
<td>1.0 - 7.0 mg/l</td>
<td>&gt;4.5 mg/l</td>
</tr>
<tr>
<td>Temperature</td>
<td>21.0 - 27.5 °C</td>
<td>Ambient</td>
</tr>
<tr>
<td>Sulfide (as S)</td>
<td>0 - 60 ug/l</td>
<td>-</td>
</tr>
<tr>
<td>CO₂ (as CO₂)</td>
<td>0.6 - 2.0 m-moles/l</td>
<td>-</td>
</tr>
<tr>
<td>Suspended Solids</td>
<td>30 - 60 mg/l</td>
<td>-</td>
</tr>
<tr>
<td>Cadmium</td>
<td>&lt;0.01 mg/l</td>
<td>-</td>
</tr>
<tr>
<td>Lead</td>
<td>&lt;0.005 mg/l</td>
<td>-</td>
</tr>
<tr>
<td>Mercury</td>
<td>&lt;0.001 mg/l</td>
<td>-</td>
</tr>
<tr>
<td>Pesticides</td>
<td>Insignificant</td>
<td>-</td>
</tr>
</tbody>
</table>
PART II

It has been proposed that the addition of sea water (10,000 m$^3$/hr) at the head end of the Ala Wai Canal will produce water quality in the Canal of the same order as the sea offshore of the Ala Wai Boat Harbor. This portion of this report examines this hypothesis.

The Ala Wai Canal is a partially mixed, moderately stratified estuary. Waters of the Canal can be categorized as surface water, channel water, and basin water (Gonzalez, 1971). The surface water consists of a mass of brackish water residing in the upper one meter of the Canal. The source of this water is runoff. Channel water is that mass of water from the Ala Wai Boat Harbor which intrudes into the Canal as a function of tide. This mass is more characteristic of sea water in respect to salinity as well as all other water quality parameters. Mixing with surface water does increase the pollutant load in the channel water. Basin water is essentially channel water that spills over a sill generated by sedimentation from the Manoa-Palolo Stream drainage. The reference for basin water instead of channel water results from this mass of water, once trapped by the sill, exhibiting poor recycling in the seaward direction (Gonzalez, 1971). Figure 4 represents a longitudinal section of the Canal system illustrating both the sill area and the basin area. By previous definition basin water would not exist in the absence of the sill.

In the classical approach to water quality improvement the objective is to eliminate or, as a minimum, reduce the pollutant load. In the case of the Ala Wai Canal system this objective does not appear feasible. The substantial load of all water quality parameters cited in Part I of this report
are non-point sources which enter the Canal through various drainage canals. In lieu of this condition it has been proposed to supplement the Canal flow with an additional flow of 10,000 m³/hr (63 MGD) of high water quality sea water. Based on the inference that pollutant loadings will remain similar to present conditions, the only mechanisms available for water quality improvement are dilution and flushing.

Dilution refers to mixing of the supplemental 10,000 m³/hr sea water flow with water in the Canal that exceeds Water Quality Standards. This mixing will allow the pollutant load (by weight) to occupy a larger volume, thus lowering the concentration. Flushing on the other hand, indicates a mass transfer of one mass of water with another. The theory involved with flushing assumes denser seawater will displace lighter Canal water thus redistributing the Canal water for increased mixing possibilities.

With use of these two mechanisms, dilution and flushing, it is necessary to hypothesize how water present in the Canal will mix with the proposed sea water source. At that point in time where probable mixing has been established a range of Water Quality parameter concentration reductions can be assigned to segments of the Canal. For this reason a series of figures (Figure 5 through 10) have been prepared suggesting mixing conditions possible in the Canal. These figures indicate mixing with sea water at pre-dredging and post-dredging conditions. Since it has been established that sedimentation has occurred at 20cm/yr in portions of the Canal (Gonzalez, 1971) pre-dredging mixing conditions will be valuable with time as bottom topography approaches that of Figure 4.

It should be kept in mind that Figures 5 through 10 are not statistical
models. These Figures represent the author's interpretation, based on available literature, of existing conditions and future conditions of water mass distribution. Figure 5 indicates present Canal conditions with no additional water sources. The three distinct water masses described by Gonzalez are present in three distinct areas. Surface water occupies the region of the Canal above a one meter depth. Channel water (C) is characteristically found below the one meter depth and seaward of the sill. Basin water (B) is located Diamond Head of the Sill and below the one meter depth. The only substantial mixing of water masses occurs approximately at the one meter depth between less dense surface water and more dense (salinity) channel water. This mixing is a result of interfacial shear (Gonzalez, 1971).

Figure 6 is a representation of post-dredging conditions with no additional source of sea water. The distribution of water masses is identical to that of Figure 5 with the absence of all basin water. With the sill absent channel water is no longer trapped at the landward end of the Canal. This interpretation is significant on a water quality basis in terms of dissolved oxygen. The only portion of the Canal system that presently violates dissolved oxygen standards is the basin area. With the basin gone water stagnation in the Canal would be essentially eliminated.

Figure 7 illustrates pre-dredging conditions with the 10,000 m$^3$/hr sea water source applied to the basin region of the Canal. As with Figures 5 and 6 the portion of the Canal seaward of the Manoa–Palolo Drainage Canal remains unchanged with surface water mainly occupying the upper levels of the Canal and channel water residing below the one meter depth. The additional sea water source does eliminate the basin water. Under present conditions the residence time of basin water is more accurately described in days or weeks.
FIGURE 5

- B = Basin water
- C = Channel and sea water
- S = Surface water

MEAN LOWER LOW WATER

DEPTH IN METERS
FIGURE 6

MEAN LOWER LOW WATER

Apukahau Stream

Drainage Ditch (1)

Drainage Ditch (2)

Mona - Palolo Drainage Canal

Drainage Ditch (3)

DEPTH IN METERS
Assuming the sea water source more typically approaches the salinity of channel water, this water would seek the lower depths in the Canal basin and systematically force more mixed channel water up and seaward of the sill. At a flow rate of 10,000 m³/hr the retention time in the basin would approach 19 hours. This relatively short retention is typical of other portions of the Canal where no oxygen deficiencies have been noted. Thus, the reduction of residence time in the basin hopefully rationalizes the absence of basin water from this figure.

Figure 8 demonstrates post-dredging water mass distribution with the sea water source located upstream of Manoa-Palolo Drainage Canal. It is this author's interpretation that surface water will predominate above the one meter depth and channel water will occupy most of the volume below 1.5 meters. A larger region of interfacial shear has been assigned to the 1.0 to 1.5 meter depth due to increased sea water flow toward the Boat Harbor under average conditions. This conclusion might be disputed based on two lines of thought. It has been demonstrated that during periods of high surface water runoff the depth of surface water remains unchanged but the mean velocity of the less dense surface water increases to accommodate larger flows. Conversely, it is true for increased flow of channel water, mixing might not be significant. Secondly 10,000 m³/hr can only generate a velocity of 0.024 meters/sec through a two meter section. This velocity is insignificant as to interfacial shear generation capabilities when compared to tidal velocities and surface water velocities at periods of high runoff.

Figure 9 is the representation of pre-dredging conditions with the sea water source artificially mixed with Manoa-Palolo runoff in the Manoa-Palolo Drainage Canal. This is the first illustration where there exists a substantial
change from present conditions seaward of the Manoa-Palolo Drainage Canal. With complete mixing of surface water and sea water, the upper 1.5 meters of the Canal forms a homogeneous layer of water. This is the only case where substantial dilution has occurred between these two water masses. The result of this mixing will lower the concentration levels of the surface 1.5 meters of the Canal as a function of the ratio of surface water to sea water. While improving the water quality condition of the major portion of the Canal no improvement of the basin will be noticed. This premise is based on the fact that completely mixed water now exiting from the Manoa-Palolo Drainage Canal will not be dense enough to transfer stagnant channel water (basin water) from this area.

The final interpretive model (Figure 10) results from complete mixing of sea water and surface water in the Manoa-Palolo Drainage Canal under post-dredging conditions. This Figure represents the most advantageous longitudinal section for the Canal. Surface water appears to be diluted to the maximum degree and all basin water has been eliminated with the absence of the sill. It must be kept in mind that the sea water flow does not generate enough additional velocity to minimize sedimentation of a new sill. For this reason in a period of a few years a new sill layer will again produce stagnation above the Manoa-Palolo Drainage Canal.

Based on the previous interpretive model study a compromise between Figure 7 and 10 apparently will most enhance water quality in the Canal for the period between dredgings. Sea water must be added to the basin area to insure flushing of stagnant water and sea water must also be completely mixed with surface water in Manoa-Palolo Drainage Canal to insure the greatest dilution of water quality parameter concentrations.
With the most attractive mixing conditions described it is now possible to approximate improvements of water quality in the Canal. Runoff from Manoa and Palolo Streams was monitored by Gonzalez in 1969. Runoff rates varied from 11.30 $\text{m}^3/\text{sec}$ to 0.22 $\text{m}^3/\text{sec}$. (Appendix B-1) The median of eleven observations was 0.25 $\text{m}^3/\text{sec}$. This median corresponds to the average monthly discharge of the combined streams for the ten year period, 1959-1969/ (Appendix B-2). Therefore to investigate average dilution conditions a runoff of 0.25 $\text{m}^3/\text{sec}$ has been assigned to the Manoa-Palolo Stream discharge to the Ala Wai Canal. The dilution factor thus applying in the Manoa-Palolo Drainage Canal is the ratio of the average runoff flow rate and the 10,000 $\text{m}^3$/hr input. This dilution factor is approximately eleven ($10,000/ .25 \times 3600$). Therefore if the assumption is made that the additional source of sea water is devoid of significant concentrations of water quality parameters now violating standards, the complete mixing of both water masses would reduce all water quality parameters in this area. This would apply to approximately a depth of 1.5 meters under average conditions.

It is difficult to assign dilutions below 1.5 meters since mixing in this region is caused by interfacial shear. One fact that can be concluded is, the concentrations in this region cannot be greater than the water mass above. This is true if the source of the parameter had been the stream runoff. Fortunately the parameters that appear to be scrutinized to the highest degree (bacteria, nitrogen, phosphorus, metals) are basically attributable to runoff.

Seaward of the Manoa-Palolo Drainage Canal there will be a deterioration of water quality resulting from other drainage canals depositing high concentrations of pollutants into the surface levels of the Canal. Most notable of these drainage canals is Apukehau Stream draining Makiki Valley. This
water will be less dense than the mixed water of Manoa-Palolo Streams and the bottom channel water. It might be expected that water quality will remain unchanged from present conditions through a maximum depth of one meter.

On a monthly average basis during the period 1959 through 1969, the highest Manoa-Palolo Stream runoff was approximately 0.7 M³/sec. This still computes to a dilution factor of four in the drainage canal. Thus over a range of typical average runoffs in Manoa-Palolo Streams a substantial dilution is available assuming good mixing can be attained.

From Part I of this report the water quality parameters most often exceeding Water Quality Standards were bacteriological parameters. On a yearly mean basis average exceedence of Total Coliform limitations was by a factor of seven. It is therefore the conclusion of this report that a minimum dilution factor of seven will bring a substantial portion of the Canal within appropriate bacteriological limitations as well as all other parameters causing violation through Stream runoff. Therefore approximately 18,000 M³/hr of sea water would be necessary to be completely mixed with Manoa-Palolo Stream runoff to attain monthly average compliance with all water quality parameters. But even with this flow it is likely that the farther seaward future sampling is conducted, there is a substantially increased possibility of surface water quality exceedences.

Since dissolved oxygen is the only parameter applicable to flushing, the basin area will be discussed separately. To alleviate stagnation of channel water landward of the sill a correspondingly greater volume of sea water will be needed with time to flush an ever increasing basin volume. For descriptive purpuses a maximum retention time of one day has been assigned to the basin
area to prevent stagnation. When and if the basin volume approaches present conditions, 7,500 m$^3$/hr would be needed to sustain one day residence. Therefore to maintain water quality in the largest part of the Canal for the greatest length of time, approximately 25,000 m$^3$/hr (160MGD) of sea water pumping capability should be supplied based on the assumptions of this section.

**Summary**

The table on the following page represents possible water quality that can be achieved by complete mixing of sea water with channel water. A dilution factor of seven was used in the formulation of this table. This factor represents a seawater flow of 10,000 m$^3$/hr and a Manoa Stream runoff of approximately 0.4 m$^3$/sec.
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Addendum to Water Quality Section

Saltwater Toxicity Effects on Coliforms in the Ala Wai Canal

The Water Quality section of the Improvement of the Ala Wai Canal project attempted to evaluate the water quality effects of the addition of 10,000 m$^3$/hr to the head waters of the Ala Wai Canal. Under the most ideal mixing conditions the canal waters would be expected to meet the levels expressed in the Summary Table (Section C, pg. C-34). These levels are attainable immediately downstream of the Manoa-Palolo Drainage Canal and reflect the dilution effect of 10,000 m$^3$/hr of sea water and the average Manoa-Palolo Drainage Canal discharge of approximately 1,440 m$^3$/hr (0.4 m$^3$/sec).

The coliform levels will fluctuate due to other factors than strict dilution. Carlucci and Pramer (Savage, Hanes, 1971) cite factors for coliform reduction as: 1) destructive action of sunlight, 2) bacteriophages and predators, 3) adsorption and sedimentation, 4) toxic substances, and 5) lack of nutrients. The ability to quantitatively predict what additional effects will be exerted on the coliform population by these factors during and after the initial sea water dilution is a function of the inability to mathematically model these factors.

The synergistic and composite effects of these factors are circumvented by measurement of $t_{90}$ levels. This number is a measure of the time to reduce the coliform population by 90 per cent. These times are evaluated as disappearance times and decay times. Disappearance times refer to organism reduction through dilution as well as death. Decay times refer to death resulting from toxicity of salt water to the coliform organisms. A review of the literature found typical $t_{90}$ (decay) times of 1.5 to 4.5 days. A lag time from 0.5 to 2.5 days
accompanied these $t_{90}$ decay times (Pearson, 1963). This lag time refers to the time period preceding first order equation death rates. During the lag phase it is possible for the actual coliform population to continue to grow.

A locally cited $t_{90}$ time (disappearance) is 0.5 hours. This value has been used to describe coliform disappearance at the Sand Island sewage outfall. It must be kept in mind that this value represents the composite effects of dilution as well as death resulting from salt toxicity and all other decay variables. The salinity involved at this outfall is approximately $34.7^{\circ}/\text{o}$. Reduced salinity will have an effect on this toxicant's effect on coliform populations.

In conclusion the findings of the Summary Table (page C-34) represent the levels of water quality parameters expected in the Canal after complete mixing at the Ala Wai Canal and Manoa-Palolo Drainage Canal confluence. There is insufficient data available to properly predict the additional reduction in coliform levels through decay. An inherent lag period prior to logarithmic decay combined with decay $t_{90}$'s exceeding the residence time of water in the canal indicate possible minimal increased coliform reduction.
REFERENCES


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* COURTESY OF THE STATE OF HAWAII DEPARTMENT OF HEALTH
TOTAL COLIFORM DATA

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* COURTESY OF STATE OF HAWAII DEPARTMENT OF HEALTH
## TOTAL COLIFORM DATA

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* COURTESY OF STATE OF HAWAII DEPARTMENT OF HEALTH*
Fecal Coliform Data

Station 321-McCully Bridge

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*Courtesy of State of Hawaii Department of Health.*
FECAL COLIFORM DATA

STATION 320-ALA MOANA BRIDGE

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* COURTESY OF THE STATE OF HAWAII DEPARTMENT OF HEALTH
Total coliform distribution. December 9, 1969.
(Frank Gonzalez)
(Frank Gonzalez)
WATER QUALITY RANGE

- --- DRY WEATHER CONDITIONS
- --- WET WEATHER CONDITIONS

SAMPLING STATION LOCATION

STATE WATER QUALITY STANDARD
OF 1000/100 ML FOR CLASS 2 WATERS

**Total Coliforms / 100 mL**

**Length of Stream (Feet)**

RANGE IN TOTAL COLIFORM COUNT OBSERVED IN MANOA STREAM, 1970-71.
(WRRC Tech Report No. 63, 1972)
WATER QUALITY RANGE

- DRY WEATHER CONDITIONS
- WET WEATHER CONDITIONS

SAMPLING STATION LOCATION

---

STATE WATER QUALITY STANDARD
OF 200/100 ML FOR CLASS Z WATERS

RANGE IN FECAL COLIFORM COUNT OBSERVED
IN MANOA STREAM, 1970-71.
(WRSC Tech Report No. 63, 1972)
RANGE IN TOTAL BACTERIA COUNT OBSERVED IN MANOA STREAM, 1970-71.
(WRRC Tech. Report No. 63, 1972)
Bacteriological Data (Number/100 ml)
Ala Wai Yacht Harbor
(Oceanic Institute, 1972)

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(Oceanic Institute, 1972)

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(Oceanic Institute, 1972)

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Bacteriological Data (Number/100 ml) - Ala Wai Yacht Harbor
(Oceanic Institute, 1972)

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Median

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|       | 600     | 4000    | 38      | 600          | 70        |
|       | 560     | 3033    | 27      | 520          | 57        |
|       |         |         |         |              |           |
| 2     | 120     | 600     | 22      | 300          | 64        |
|       | 124     | 600     | 28      | 324          | 108       |
|       | 108     | 600     | 24      | 305          | 72        |
|       |         |         |         |              |           |
| 3     | 110     | 1400    | 100     | 130          | 78        |
|       | 130     | 1450    | 104     | 160          | 84        |
|       | 107     | 1177    | 100     | 137          | 78        |
|       |         |         |         |              |           |
| 4     | 14      | 124     | 6       | 100          | 40        |
|       | 20      | 136     | 10      | 120          | 44        |
|       | 15      | 119     | 5       | 101          | 36        |
|       |         |         |         |              |           |
| 5     | 8       | 48      | 4       | 10           | 0         |
|       | 22      | 60      | 6       | 10           | 2         |
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### Bacteriological Data (Number/100 ml) Ala Wal Yacht Harbor

(Oceanic Institute, 1972)

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TABULATION OF MEAN ANALYTICAL RESULTS AT EACH SAMPLING STATION ON MANOA STREAM.

(WRRC Tech. Report No. 63, 1972)

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**Average Extinction Coefficients (k) (from Harris, 1975)**

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*HIGHEST POLLUTION RATIOS, HUMAN SOURCES OF CONTAMINATION, NOT INCLUDED IN ARITHMETIC MEAN.
Chemical Analyses: Ala Wai Harbor Study
(Oceanic Institute, 1972)

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<th>Sulfide (µg-at/l)</th>
<th>Total CO₂ (m mole/l)</th>
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| 1 m | " | 0.465 | 1.316 | 0.00 | 1.415 |
| 1 Bottom | " | 1.161 | 0.790 | 0.17 | 1.333 | 2.530 |
| 2 Surface | " | 0.619 | 3.685 | 1.550 |
| 2 m | " | 1.703 | 0.526 | 0.17 | 1.569 |
| 2 Bottom | " | 1.084 | 0.792 | 0.17 | 1.610 | 1.150 |
| 3 Surface | " | 1.006 | 5.700 | 1.439 |
| 3 m | " | 0.852 | 1.841 | 0.17 | 1.458 |
| 3 Bottom | " | 0.852 | 1.053 | 0.17 | 1.155 | 5.700 |
| 4 Surface | " | 1.161 | 1.569 | 1.426 |
| 4 m | " | 0.619 | 1.316 | 0.17 | 1.575 |
| 4 Bottom | " | 1.703 | 1.841 | 0.17 | 1.550 | 3.370 |
| 5 Surface | " | 0.387 | 8.949 | 1.290 |
| 5 m | " | 0.774 | 3.685 | 0.17 | 1.568 |
| 5 Bottom | " | 0.929 | 3.948 | 0.17 | 1.554 | 0.910 |

* Nitrate plus nitrite
### Chemical Analyses: Ala Wai Harbor Study - Incoming Tide

(Oceanic Institute, 1972)

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<th>Sulfide (μg-atS/l)</th>
<th>Total CO₂ (m mole/l)</th>
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*Nitrate + nitrite
## Chemical Analyses: Ala Wai Harbor Study

(Oceanic Institute, 1972)

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1 Surface 8/4/72 1.050 12.803 0.422
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1 Bottom | " | 0.450 | 0.671 | 0.000 | 0.332 | 3.507 |
2 Surface | " | 0.300 | 2.036 | 1.270 | | |
2 m | " | 0.225 | 0.886 | 0.000 | 1.070 | |
2 Bottom | " | 0.600 | 1.677 | 0.000 | 0.485 | 1.910 |
3 Surface | " | 1.125 | 6.826 | 0.378 | | |
3 m | " | 0.225 | 0.671 | 0.000 | 1.180 | |
3 Bottom | " | 0.375 | 0.431 | 0.000 | 0.101 | 1.382 |
4 Surface | " | 0.825 | 0.192 | 0.683 | | |
4 m | " | 0.150 | 0.934 | 0.000 | 0.645 | |
4 Bottom | " | 0.075 | 0.407 | 0.000 | 1.050 | 1.677 |
5 Surface | " | 0.075 | 0.647 | 0.590 | | |
5 m | " | 0.000 | 0.719 | 0.000 | 0.583 | |
5 Bottom | " | 0.000 | 0.216 | 0.000 | 0.696 | 0.811 |

* Nitrate + Nitrite
RANGE IN TOTAL PHOSPHORUS CONCENTRATION
OBSERVED IN MANOA STREAM, 1970-71.
(WREC Tech. Report No. 63, 1972)
RANGE IN TOTAL NITROGEN CONCENTRATION OBSERVED IN MANOA STREAM, 1970-71.

(WRRC Technical Report No. 63, 1972)
Longitudinal section of phosphate in ug-at/l. December 9, 1969.
(Frank Gonzalez)
Longitudinal section of phosphate in ug-at/l. October 20, 1969.
(Frank Gonzalez)
Longitudinal section of nitrate in ug-at/l. October 20, 1969.
(Frank Gonzalez)
Longitudinal section of nitrite in ug-at/l. December 9, 1969.
(Frank Gonzalez)
Longitudinal section of nitrite in µg-at/l. October 20, 1969.
(Frank Gonzales)
Salinity, temperature, and dissolved oxygen profiles for incoming tides.
(Oceanic Institute, 1972)

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<th>Salinity (^{0/00}) 7/24/72</th>
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* using optical salinometer

(continued - next page)
(Oceanic Institute, 1972)

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1 Shaded areas indicate periods of observation on rising tides of July 10 and 24, 1972.
Salinity, temperature, and dissolved oxygen profiles for equilibrium tide.

(all measurements made July 17, 1972)

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Salinity (‰), Temperature (°C), and Dissolved O₂ (mg/l) data for August 2 and 4, 1972, at various stations and depths.

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1 Shaded areas indicate periods of observation on falling tides of August 2 and 4, 1972.
Twenty-four hour temperature and dissolved oxygen profiles.

(Oceanic Institute, 1972)

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Twenty-four hour temperature and dissolved oxygen profiles (cont't)

(Oceanic Institute, 1972)

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Average Surface and Bottom Oxygen (ppm)

(Jacquelin Miller, 1975)

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## Monthly Average Surface and Bottom Oxygen (ppm), Standard Deviation, and Number of Observations in Sections I-IV

*(Jacquelin Miller, 1979)*

### Table: Monthly Average Surface and Bottom Oxygen (ppm), Standard Deviation, and Number of Observations in Sections I-IV

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| **BOTTOM** | | | | |
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| Mar   | 5.60     | 6.87     | 1.06     | 6.37     |
| Apr   | 6.09     | 7.62     | 3.62     | 8.83     |
| May   | 6.51     | 5.52     | 1.23     | 5.16     |
| Jun   | 7.02     | 5.46     | 1.14     | 2.85     |
| Jul   | 4.25     | 6.19     | 1.65     | 0.79     |
| Aug   | 5.10     | 6.16     | 1.96     | 2.61     |
| Sep   | 3.76     | 4.21     | 1.69     | 1.19     |
| Oct   | 4.40     | 4.17     | 0.97     | 1.09     |
| Nov   | 5.55     | 2.10     | 0.85     | 0.80     |
| Dec   | 5.52     | 4.87     | 2.64     | 2.43     |
| Jan   | 4.33     | 5.90     | 4.66     | 2.10     |
| Feb   | 2.77     | 4.63     | 1.80     | 1.29     |
| Mar   | 5.70     | 3.10     | 0.0      | 0.70     |
| Apr   | 4.35     | 5.50     | 0.0      | 0.30     |
Average Surface and Bottom Oxygen (ppm) on an Incoming and Outgoing Tide at Each Station
(Jacquelin Miller, 1973)

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**Jacquelin Miller, 197$)**

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**Jacquelin Miller, 1975**

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| **Bottom** |           |           |           |                  |           |           |           |           |           |           |           |           |
| FEB       | 24.90     | 0.0       | 1         | 25.90           | 3.35      | 6         | 24.00      | 0.0      | 0         | 26.27    | 0.66      | 6   |
| MAR       | 24.37     | 0.15      | 4         | 26.85           | 0.65      | 8         | 25.51      | 1.57      | 9         | 27.08    | 0.86      | 3   |
| APR       | 25.26     | 0.45      | 10        | 25.73           | 1.52      | 7         | 25.69      | 0.45      | 6         | 27.14    | 0.50      | 16  |
| MAY       | 25.91     | 0.32      | 7         | 26.46           | 0.22      | 12        | 26.84      | 0.50      | 16        | 27.35    | 0.74      | 6   |
| JUNE      | 26.50     | 0.89      | 6         | 27.21           | 0.48      | 12        | 27.49      | 0.61      | 16        | 27.59    | 0.47      | 9   |
| JULY      | 27.22     | 0.27      | 6         | 28.04           | 0.41      | 7         | 28.05      | 0.42      | 17        | 23.97    | 0.74      | 4   |
| AUG       | 27.30     | 0.41      | 6         | 27.94           | 0.51      | 9         | 27.86      | 0.31      | 9         | 28.53    | 0.59      | 7   |
| SEPT      | 27.12     | 0.33      | 5         | 28.57           | 1.16      | 9         | 28.25      | 0.56      | 10        | 29.25    | 1.02      | 4   |
| OCT       | 27.20     | 0.41      | 7         | 27.56           | 0.57      | 7         | 27.03      | 0.34      | 20        | 27.53    | 0.33      | 3   |
| NOV       | 26.30     | 0.30      | 2         | 26.85           | 0.49      | 4         | 26.62      | 0.11      | 4         | 24.01    | 0.70      | 3   |
| DEC       | 25.22     | 0.51      | 6         | 26.28           | 0.33      | 6         | 25.54      | 0.60      | 10        | 25.02    | 0.61      | 5   |
| JAN       | 24.00     | 0.36      | 5         | 23.73           | 2.02      | 6         | 24.29      | 1.10      | 8         | 22.85    | 1.51      | 2   |
| FEB       | 25.36     | 0.50      | 5         | 26.02           | 0.82      | 8         | 25.68      | 0.48      | 9         | 28.25    | 1.65      | 2   |
| MAR       | 34.20     | 0.0       | 1         | 25.80           | 0.0       | 1         | 25.15      | 0.15      | 2         | 25.30    | 0.0      | 1   |
| APR       | 25.70     | 0.20      | 2         | 26.50           | 0.0       | 1         | 26.40      | 0.0      | 1         | 26.90    | 0.0      | 1   |
Average Surface and Bottom Temperature (°C) on an Incoming and Outgoing Tide at Each Station (Jacquelin Miller, 1976)

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<th>MEAN BOTTOM</th>
<th>MEAN SURFACE</th>
<th>MEAN BOTTOM</th>
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Longitudinal section of temperature in °C. December 9, 1969.
(Frank Gonzalez)
Longitudinal section of temperature in °C. October 20, 1969.
(Frank Gonzalez)
Longitudinal section of temperature in °C. July 12, 1969.
(Frank Gonzalez)
Longitudinal section of temperature in oC. July 12, 1969.
(Frank Gonzalez)

(Frank Gonzalez)
Longitudinal section of temperature in oC. June 10, 1969.
(Frank Gonzalez)
Longitudinal section of temperature in °C. May 9, 1969,
(Frank Gonzalez)
Longitudinal section of temperature in °C, April 1, 1969.
(Frank Gonzalez)
Longitudinal section of temperature in °C. March 29, 1969.
(Frank Gonzalez)
Longitudinal section of temperature in °C. March 16, 1969.
(Frank Gonzalez)
Longitudinal section of suspended load in mg/l. October 20, 1969.
(Frank Gonzalez)
Longitudinal section of suspended load in mg/l. December 9, 1969.

(Frank Gonzalez)
RANGE IN SUSPENDED SOLIDS CONCENTRATION OBSERVED IN MANOA STREAM, 1970-71.
(WRRC Tech. Report No. 63, 1972)
(Eshleman, 1973)
- Sampling Stations in the Ala Wai Canal and Yacht Harbor.

Station codes are TMC, midchannel opposite Texaco Fuel Dock; TD, at the Texaco Fuel Dock; NYC, midchannel opposite the Hawaii Yacht Club; SDH, midchannel opposite the dock with a large sign reading "Skindivers Hawaii"; AMB, midchannel by the Ala Moana Bridge; KPOI, midchannel opposite the transmitting tower of radio station KPOI; KB, midchannel by the Kalakaua Bridge; AS, the entrance of Apukehau Stream; MD, midchannel by the McCully Bridge; KUA, midchannel by Kuamoo St., MPD, the confluence of the Manoa-Palolo Drainage Canal and the Ala Wai Canal; MPDD, a dock on the Manoa-Palolo Canal about 100 m upstream of the Ala Wai; KAN, midchannel opposite Kanekapolei Street; PAO, midchannel opposite Paokalani St., END, the extreme landward end of the Canal near Kapahulu Blvd.

(Eshleman, 1973)
On all sampling dates water samples from the Texaco Dock, Ala Moana Bridge and Manoa-Palolo Drainage Canal Stations were analyzed for Pb, Cd and Hg. No detectable Pb or Cd was present in any samples (limit of detection: Pb 0.01 ug ml\(^{-1}\); Cd 0.005; Hg 0.0001). Detectable mercury was present in the March 6 samples (0.0007 ug ml\(^{-1}\) at the Texaco Dock).

C. Discussion

**Pb:** Lead in Ala Wai sediments may not originate from a source in the harbor. Dredging has probably distorted the distribution pattern in the Canal and Harbor. Substrate differences may also be important. For example, the bottom off the Texaco Dock is largely crushed coral whereas other stations are largely a high organic anaerobic sludge. The fact that the highest levels of lead in the March sampling run were recorded for the Manoa-Palolo Drainage Canal station suggests that some of the lead in the Ala Wai may originate externally to the Canal-Harbor area. Terrestrial soils on Oahu in regions of high motor vehicle traffic density may contain amounts of lead one or two magnitudes higher than the highest values reported for the Ala Wai sediments.

**Zn and Cu:** These two metals correlate well (\(r = 0.78, p = 0.01, 10\) d.f.) and occur at highest levels in the harbor area, suggesting that they may have been leached from antifouling paints. The highest recorded Zn and Cu levels on Oahu are 480 and 165 ppm respectively in the Kapalama Canal\(^2\).

**Cd:** Cadmium does not correlate well with any of the other metals. Levels of this metal are higher than "natural" but no source has been identified.

**Hg:** Mercury is probably released as a contaminant in the Harbor area. This is supported by the observations that high values were always associated with proximity to the harbor and that on one occasion, when relatively high amounts of Hg was present in harbor waters, the sediments showed a corresponding high level of Hg.

(Oceanic Institute, 1972)

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A. Heavy metals in Ala Wai Boat Harbor Sediments and Water
   (Information and analysis courtesy of Mr. Allen Eshleman
   and Dr. Doak C. Cox)

   On three occasions, July 7, 1971, July 14, 1971, and March 6, 1972,
   sediments and waters from the Ala Wai Canal and Yacht Harbor were sampled for
   heavy metals. The July samplings were assumed to represent the condition of
   the waterway during a period of low rainfall; the March sample series was
   obtained during a period of heavy rainfall. Sample sites are shown in Figure 1.
   All analyses are performed using standard atomic absorption spectrophotometric
   techniques.

B. Results

Sediments

Lead (Pb) - Using data from all three sampling dates, the mean Pb content of
sediments sampled was 65.7 ± 39.9 ppm \(^1\) (19-193; n=30). The highest values
for the July samples were recorded at the extreme landward end of canal near
Kapahulu Boulevard (193, 141 ppm respectively). The highest value encountered
in the March sampling was 92 ppm at the juncture of the Ala Wai and the Manoa-
Palolo Drainage Canal. Lead in harbor sediments (taken here as all samplings
from the Ala Moana Bridge seaward) never exceeded 67 ppm (July 7). Samples taken by
the Texaco Fuel Dock were uniformly low (20-29 ppm).

Zinc (Zn) - Zn was determined at 12 sampling stations on March 6 only. The mean
Zn content was 181.0 ± 49.5 ppm. Values were generally higher in the Harbor area.
The highest value, 252 ppm, was recorded at the Ala Moana Bridge station.

Copper (Cu) - Cu analyses was performed on March 6 samples. The mean copper
content of sediments was 48.7 ± 16.7 ppm (30-76; n=12).

Cadmium (Cd) - For samples from all three dates, \( \bar{X} = 1.8 \pm 1.1 \) ppm (0.8-5.6; n=30).
The highest and lowest values were recorded at the same general location, midchannel
off the Texaco dock (0.8 ppm July 7; 5.6 ppm March 6).

Mercury (Hg) - The mean Hg content of sediments obtained on July 7 and 14 was 0.26 ppm;
the mean Hg content of the March samples was 0.56 (0.09-1.34). On all sampling dates
the highest values for Hg were obtained in the harbor area.

(Oceanic Institute, 1972)

\(^1\) Dry weight.
(Eshleman, 1973)

Heavy Metals* in Sediments from the Ala Wai Yacht Harbor and the Ala Wai Canal, Honolulu, Oahu

<table>
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<th>Pb**</th>
<th>Zn***</th>
<th>Cu***</th>
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<tr>
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<td>--</td>
</tr>
<tr>
<td>Texaco Dock</td>
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<td>33</td>
<td>113</td>
</tr>
<tr>
<td>Texaco Midchannel</td>
<td>67</td>
<td>59</td>
<td>226</td>
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<tr>
<td>Hawaii Yacht Club</td>
<td>--</td>
<td>55</td>
<td>176</td>
</tr>
<tr>
<td>Skindivers Hawaii</td>
<td>--</td>
<td>67</td>
<td>231</td>
</tr>
<tr>
<td>Ala Moana Bridge</td>
<td>78</td>
<td>56</td>
<td>252</td>
</tr>
<tr>
<td>KFOI</td>
<td>126</td>
<td>85</td>
<td>168</td>
</tr>
<tr>
<td>Kalakaua Bridge</td>
<td>69</td>
<td>38</td>
<td>159</td>
</tr>
<tr>
<td>McCully Bridge</td>
<td>108</td>
<td>19</td>
<td>146</td>
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<tr>
<td>Kuamoo Street</td>
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<td>74</td>
<td>251</td>
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<td>Manoa-Palolo Drainage Canal</td>
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<td>92</td>
<td>189</td>
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<td>Manoa-Palolo**** Canal Dock</td>
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<td>70</td>
<td>--</td>
</tr>
<tr>
<td>Kanekapolei St.</td>
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<td>111</td>
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<td>Paoakalani St.</td>
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<td>150</td>
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<tr>
<td>Kapahulu Blvd.</td>
<td>193</td>
<td>141</td>
<td>--</td>
</tr>
</tbody>
</table>

* Expressed as µg·g⁻¹ dry matter.

** A: July 7, 1971; B: July 14, 1971; C: March 6, 1972.

*** March 6, 1972.

**** Sediments sampled directly next to this location; other site designations refer to midchannel samplings opposite the named landmark.
(Ishleman, 1973)
Mercury* in the Ala Wai Canal and Yacht Harbor

<table>
<thead>
<tr>
<th>Site</th>
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<th>July 14</th>
<th>March 6</th>
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<tr>
<td>Texaco Dock***</td>
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<td>460</td>
<td>1,340</td>
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<tr>
<td>Texaco Midchannel</td>
<td>300</td>
<td>440</td>
<td>590</td>
</tr>
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<td>Hawaii Yacht Club</td>
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<td>730</td>
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<tr>
<td>Skindivers Hawaii</td>
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<td>---</td>
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<td>520</td>
<td>570</td>
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<td>KPOI</td>
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<td>620</td>
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<td>McCully Bridge</td>
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<td>Kuamoo Street</td>
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<td>Pacakalani St.</td>
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<tr>
<td>Kapahulu Blvd.</td>
<td>110</td>
<td>120</td>
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</table>

* Expressed as μg·kg⁻¹ dry matter.

** Expressed as μg·kg⁻¹ fresh weight (dry weights unavailable, but water content of these sediments was probably ca. 50 percent).

*** Sediments samples at stated location; other site designations refer to midchannel samplings opposite the named landmark.
C(A-79)

(Eshleman, 1973)
Mercury in the Hawaiian Environment

**Water**

<table>
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<th>Collection Site and Date</th>
<th>Mercury Content (µg·l⁻¹)</th>
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<tr>
<td>Kaehi Lagoon, Hon. January 6, 1971</td>
<td>0.2</td>
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<tr>
<td>Kapalama Canal, Hon. January 5, 1971</td>
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<td>Ala Wai Canal and Yacht Harbor, Hon. January 7, 1971</td>
<td>0.1</td>
</tr>
<tr>
<td>July 14, 1971</td>
<td>0.1</td>
</tr>
<tr>
<td>March 6, 1972</td>
<td>0.3--0.6</td>
</tr>
<tr>
<td>*Sewage Water, Sand Island Outfall, Hon. December 15, 1970</td>
<td>10.0</td>
</tr>
<tr>
<td>**Kaukonahua Stream at Waialua, Oahu October 7, 1970</td>
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<td>**Waikele Stream at Waipahu, Oahu October 7, 1970</td>
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<tr>
<td>**Nuuanu Stream near Honolulu October 7, 1970</td>
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<tr>
<td>**Manoa-Palolo Drainage Canal, Hon. October 7, 1970</td>
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<tr>
<td>**Kawainui Drainage Canal, Kailua, Oahu October 7, 1970</td>
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<td>**Wailuku River at Piilanihulua, Maui October 13, 1970</td>
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<tr>
<td>**Honolii Stream near Papaikou, Hawaii October 13, 1970</td>
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<tr>
<td>**Kalihi Stream at Nimitz Bridge, Hon. October 7, 1970</td>
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D. Pesticides

Shultz (1971) conducted a survey of chlorinated pesticide residues for the waters of the Ala Wai Canal. Table 1 excerpted from this report, generalizes the results of this survey.

**COMPARISON OF PESTICIDE LEVELS OF WATER, SEDIMENT, ALGAE AND FISH FROM THE ALA WAI CANAL AND MANOA AND PALOLO STREAMS**

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<tr>
<th>Sample</th>
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<th>Dieldrin</th>
<th>DDT</th>
<th>DDD</th>
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<tbody>
<tr>
<td>Water*</td>
<td>0.2</td>
<td>11.1</td>
<td>1.8</td>
<td>2.2</td>
<td>1.22</td>
</tr>
<tr>
<td>Sediment*</td>
<td>40,000</td>
<td>40,000</td>
<td>70,000</td>
<td>120,000</td>
<td>2.39</td>
</tr>
<tr>
<td>Algae*</td>
<td>10,000</td>
<td>40,000</td>
<td>40,000</td>
<td>30,000</td>
<td>1.00</td>
</tr>
<tr>
<td>Mollies*</td>
<td>60,000</td>
<td>240,000</td>
<td>130,000</td>
<td>120,000</td>
<td>1.38</td>
</tr>
<tr>
<td>Guppies*</td>
<td>70,000</td>
<td>220,000</td>
<td>170,000</td>
<td>160,000</td>
<td>1.35</td>
</tr>
<tr>
<td>Elops (muscle)</td>
<td>140,000</td>
<td>110,000</td>
<td>90,000</td>
<td>400,000</td>
<td>6.00</td>
</tr>
<tr>
<td>Chanos (muscle)</td>
<td>250,000</td>
<td>410,000</td>
<td>120,000</td>
<td>130,000</td>
<td>3.17</td>
</tr>
</tbody>
</table>

* Combined for all locations.

Although some individual samples of fish yielded significant amounts of these pesticides, the average values were found to be within the limits of acceptability.

It must be assumed that pesticide concentrations within canal waters and sediments are equal to or higher than those for harbor waters. No samples were taken within the harbor by Shultz, and funds were not sufficient to allow for sample collection by the Oceanic Institute.

(Oceanic Institute, 1972)
### TABLE 2

**Estimated Mean Fresh Water Runoff Into the Ala Wai Canal During Surveys**

<table>
<thead>
<tr>
<th>Date of Survey</th>
<th>Runoff Recorded by Stream Gauges (m³/sec)</th>
<th>Runoff Estimated from Rain Data (m³/sec)</th>
<th>Total Estimated Runoff (m³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16 March</td>
<td>11.30</td>
<td>3.70</td>
<td>15.00 ± 1.85</td>
</tr>
<tr>
<td>29 March</td>
<td>0.45</td>
<td>0.04</td>
<td>0.49 ± 0.02</td>
</tr>
<tr>
<td>01 April</td>
<td>1.80</td>
<td>3.60</td>
<td>5.40 ± 1.80</td>
</tr>
<tr>
<td>09 May</td>
<td>0.50</td>
<td>0.19</td>
<td>0.69 ± 0.09</td>
</tr>
<tr>
<td>10 June</td>
<td>0.86</td>
<td>2.48</td>
<td>3.34 ± 1.24</td>
</tr>
<tr>
<td>25 June</td>
<td>0.22</td>
<td>0.33</td>
<td>0.55 ± 0.16</td>
</tr>
<tr>
<td>29 June</td>
<td>0.22</td>
<td>0.68</td>
<td>0.90 ± 0.34</td>
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<tr>
<td>30 June</td>
<td>0.25</td>
<td>0.17</td>
<td>0.42 ± 0.08</td>
</tr>
<tr>
<td>12 July</td>
<td>0.25</td>
<td>0.04</td>
<td>0.29 ± 0.02</td>
</tr>
<tr>
<td>20 October</td>
<td>0.23</td>
<td>0.17</td>
<td>0.40 ± 0.08</td>
</tr>
<tr>
<td>09 December</td>
<td>0.22</td>
<td>0.47</td>
<td>0.69 ± 0.23</td>
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</table>
SECTION D

WELL HYDROLOGY
ALA WAI CANAL

by
Stephen Wheatcraft
University of Hawaii
A. Objectives

An alternative way of producing the required flushing rate for the canal is to install an array of wells at the mauka-Diamond Head end of the canal. Even for a very large number of wells, this method of providing the water for the flushing system would probably be considerably cheaper than the pipeline, and would require less maintenance. In addition, pumping costs would probably be considerably smaller with wells as opposed to the pipeline.

In order to evaluate this alternative, several questions must be addressed:

1) Can the required quantity of water (60 mgd) be developed at this site, and if so, how many wells will be required, at what depths, location, and costs?

2) Will the array of wells cause degradation of groundwater quality in terms of salinity, and if so, how will it affect surrounding wells?

3) What will be the extent of dewatering and possible shifting of groundwater gradients in the area?

4) What is the likelihood that a significant amount of subsidence could occur as a result of aquifer dewatering?

B. Effects of the Flushing System on the Flow Regime

1) Available data

It is assumed that the wells will be shallow, no more than 100 feet in depth and that they will not penetrate into the basal aquifer, that is, flow will take place entirely within the caprock. There are two reasons for this assumption: (1) the added expense of drilling into the basal aquifer (at least three-fold) and (2) the Board of Water Supply would probably not issue a permit to tap the basal aquifer (Lao, 1976).

The flow regime will only be analyzed here for the steady-state case, since we are interested in the long-term effects that pumping might have on the aquifer. Also, a transient analysis would be considerably more difficult and beyond the scope of this study.
In doing a well analysis, it is necessary to have a handle on some reasonable value for the hydraulic conductivity for the aquifer. The availability of information regarding conductivity depends upon there being other wells in the area where well analysis has already been performed and values obtained. Unfortunately, there are no wells in the area which have been pump tested and consequently, there are no good estimates for local conductivities. Ferrall (1976) provides geologic cross-sections of the area and some reasonable guesses can be made knowing the type of material that is present. The cross-section indicates that the top 5-10 feet are fill and alluvium, followed by 10-20 feet of lagoonal clay with some coralline debris and below that is found alternating layers of coralline debris and sand. This type of material could range in conductivity value anywhere from 1000 feet/day to 10,000 feet/day (Todd, 1976, p. 53). The clay layers themselves could be much lower, but for a 100-foot deep well they would probably average out. This range of values is quite inaccurate and the best way to determine the actual hydraulic conductivity would be to conduct pump tests in the area, however, it is all there is to go on at present.

2) Possible methods of analysis

As mentioned earlier, it is possible to treat this situation as transient or as steady-state but the steady-state case is the most useful here because of the primary concern about long-term effects of pumping. Under steady-state conditions, the standard Thiem equation for a free-surface aquifer can be employed:

$$h^2(x,y) = \frac{Q}{2\pi K} \ln \left[ \frac{(x-x_0)^2 + (y-y_0)^2}{(x-x_1)^2 + (y-y_1)^2} \right] + C$$

where: $h(x,y)$ = head or piezometric surface,
$Q$ = pumping rate,
$K$ = hydraulic conductivity,
$(x_0, y_0)$ = coordinates of the well,
$C$ = constant to be determined by boundary conditions.

The use of a single well for pumping rates this high would lead to excessively high drawdown in the vicinity of the well. One way of avoiding this is to use an array of $n$ wells, where the Thiem equation would then be given by:

$$h^2(x,y) = \frac{1}{2\pi K} \sum_{i=1}^{n} Q_i \ln \left[ \frac{(x-x_i)^2 + (y-y_i)^2}{(x-x_0)^2 + (y-y_0)^2} \right] + C$$
The number of wells employed depends on several factors and is, at least at this point, highly speculative. By increasing the number of wells and/or distributing them over a larger area, one can reduce the maximum drawdown resulting from a given flow rate to an acceptable level.

A third method of analysis is essentially a refinement of the second and is called image well theory. The Thiem equation employs assumptions which require that the aquifer be of infinite areal extent. This is actually seldom the case, and when certain regular types of boundary conditions exist, image well theory is employed. These types of boundaries include impermeable boundaries such as dikes, and infinite recharge boundaries, such as perennial streams or rivers and canals. A description of image well theory can be found in Davis and DeWiest (1966) and a complete description can be found in Hantush and Jacob (1954).

The method of analysis used in this report will be the second one, that is, to examine the effects that multiple wells have on the flow regime. It is expected that the technique of image well theory will be added as a refinement if results of the multiple well analysis are favorable.

3) Multiple Well Analysis

The non-linear Thiem equation for free surface aquifers gives computational problems when converting the equations to a computer program that can best be avoided by treating the problem as a confined aquifer. This approach makes the equation linear with the form:

\[ h(x,y) = \frac{1}{4\pi Kb} \sum_{i=1}^{n} Q_i \ln \left[ \frac{(x-x_i)^2 + (y-y_i)^2}{b^2} \right] + C \]

where \( b \) = depth of the aquifer (or in this case, depth of the well), and all other variables are previously defined. Another justification for taking the confined aquifer approach is that most of the study area is underlain by a 10-15 foot clay layer which may be considered an aquiclude compared to the coral rubble material beneath it.

By using a field of four wells placed at the corners of a rectangle 400 feet by 200 feet, drawdowns were about 15 feet within a 15-foot radius of each well and as much as 3 feet within 300 feet of the wells. These figures employed a hydraulic conductivity of 10,000 ft/day which is rather optimistic. Therefore,
In reality drawdowns would be much greater if the conductivity of the aquifer is lower.

If the number of wells is increased to 16, all at equal spacing of 200 feet, the drawdown within 15 feet of each well is reduced to 8 feet and drawdown at 300 feet is reduced to 2 feet. However, the effect of the wells is "felt" at a greater distance from the center of the well spread (that is, the radius of influence is increased). It is clear that the drawdown can be reduced to an acceptable level by increasing the number of wells, but at the expense of increasing the radius of influence.

Once accurate values for hydraulic conductivity of the area have been established, it will be relatively simple matter to determine the number and positioning of wells to yield acceptable levels of drawdown with minimum radius of influence. The critical question will be whether this radius of influence is small enough (criteria for determining what is small enough will be discussed in the section on subsidence).

4) Water Quality
The primary question with respect to water quality is whether the pumping will affect wells in the vicinity by salting them up. The only wells near the study area are the wells that the golf course uses for irrigation and the well at the Marco Polo condominium. These are all on the up-gradient side of the study area and at considerable distance (2,000 or 3,000 feet) and it is very unlikely that they will be affected by the proposed pumping scheme (Lao, 1976).

The flux through the entire caprock from upper Manoa to Diamond Head is very low (about 4 mgd) and as a result, would add only a very small percentage to the 60 mgd flow required from the well system. Since this natural flux is not currently being exploited, for all practical purposes, it can be ignored.

C. Possible Subsidence and Compaction Due to Pumping
1) Available data
Ferrall (1976) provides several geologic cross-sections compiled from various boreholes in Waikiki. The topmost 5 feet (0-5 feet above sea level) of the cross-section is comprised of fill and alluvium in a 20-foot thick (0 to
-20 feet below sea level) layer described as "lagoonal clays." This clay layer is also widespread beneath Waikiki and beneath the study area. These clays are potentially highly susceptible to compaction upon dewatering. If the cone of drawdown extends across Ala Wai Boulevard and underneath the buildings in Waikiki, then it is likely that these clays will be dewatered. If the clays are dewatered, compaction may take place causing differential subsidence of the area. The degree of compaction that might occur is a function of the engineering properties of the clay, something which would have to be determined.

Some evidence is available that subsidence takes place upon dewatering the local aquifer. In 1963, a ditch was dewatered for a sewer installation by pumping it at about 4 mgd for about 6 months, while the sewer was being installed. This situation essentially amounted to the installation of a horizontal well with drawdown in the well of 3 to 4 feet. That fact that subsidence actually took place was reflected in the cracked foundations and walls of many buildings in the vicinity (Lao, 1976). This subsidence took place in response to a pumping rate that was more than an order of magnitude smaller than the proposed pumping rate required for the flushing system.

2) Possible effects on the clay layer due to the pumping station

Compaction of the clay layer in the vicinity of the canal, the well array and the golf course is not considered a problem. If the radius of influence could be limited to this area, then concern regarding possible subsidence could be minimized. However, it is at least fairly reasonable to think that significant drawdown, and thus clay layer dewatering, may extend beneath some of the buildings in Waikiki. If the engineering properties of the clay layer are such that dewatering will cause compaction, then the subsidence problem will be very real.

D. Discussion and Summary

An analysis of well drawdown for a multiple well situation shows that drawdowns in the vicinity of the well array can be reduced to acceptable levels through a trade-off of an increase in radius of influence. Whether this radius of influence is acceptable or not is a function of the true value of hydraulic
conductivity for the aquifer which is not at present known. The rate of natural flux through the caprock is so small compared to the required pumping rate (4 mgd of natural flux compared to 60 mgd needed for pumping) that it contributes negligibly to the pumping system and can be ignored.

The possibility that salting up of wells in the area would occur is very small because the nearest wells (golf course and the Marco Polo condominium) are at least several thousand feet away and are out of the possible radius of influence.

The most serious problem associated with the pumping well scheme appears to be the possibility that differential compaction of the underlying clay layer may cause problems of subsidence in Waikiki. Subsidence due to pumping a horizontal well has been documented in one case, although it was not proved to be due to dewatering the clay layer. However, this problem of subsidence took place at a pumping rate that was more than an order of magnitude less than the current proposed pumping scheme. This is an indication that the problem could be potentially much more serious for a pumping rate of 60 mgd.

E. Recommendations

It is not yet possible to make a final recommendation concerning the ultimate fate of the well pumping scheme due to potential complications that might occur as a result of its installation. Nevertheless, the system of pumping wells would probably be considerably cheaper than the alternative pumping scheme (a large diameter pipeline from the ocean) both in capital outlay and in maintenance and pumping costs. For this reason, the multiple-well pumping system deserves to be analyzed in greater detail so that its potential impact can be fully and accurately assessed.

In order to do this, a major study should be contracted to an engineering firm which is well equipped to conduct a study on subsurface engineering properties of the area and a study of the groundwater hydraulics of the area to determine the maximum areal and vertical extent that the pumping system will affect.

The cost of a study of this nature would be negligible compared to the potential difference in cost between a pipeline and a pumping well system.
F. References Cited


Lao, Chester, 1976. Hydrologist with the Honolulu Board of Water Supply, personal communication.

SECTION E

BIOLOGICAL ASPECTS OF THE PROPOSED MECHANICAL FLUSHING OF THE ALA WAI CANAL

by

John Walters
University of Hawaii
Introduction

There have been two studies of the biology of the Ala Wai, a study of primary productivity by Harris (1975) and an ecological study by J. N. Miller (1975). Harris measured primary productivity at intervals of 10 to 14 days over a 13-month period in 1970 and 1971. Three stations (H1-H3, fig. 1) one in each of the major sections of the canal were studied at two depths, 0.2 and 1.2 m. Miller measured physical parameters (temperature, salinity, and dissolved oxygen), zooplankton composition, benthic macrofauna, and nekton, using a series of 30 stations in the three sections of the canal proper and another eight stations in the lower Manoa-Palolo stream (1-38, fig. 1). This study extended over approximately the same time period as Harris's study.

Phytoplankton

Harris found the Ala Wai to be one of the most highly productive bodies of water on record, with an average daily production of 5.26gC/m², or approximately 1.1x10⁶ g/day for the entire canal. Nutrient concentrations were very high, and appeared to be largely regenerated within the canal itself. The limiting factor for phytoplankton growth was light intensity, with nearly all productivity occurring in the upper 2m of the canal. Productivity and phytoplankton standing crop increased from the mouth to the head of the canal (table 1). Productivity in the surface layer varied greatly with salinity. Large flows of low-salinity water after storms diluted the plankton population, while slow water movement in this layer during times of low runoff allowed the phytoplankton concentration to increase sharply. Subsurface water moved more regularly under the influence of the tides. Tidal flushing removed about 33% of the organic carbon produced between 0.7 and 1.7m, where about 50 per cent of the total productivity occurred. The waters near the mouth of the canal were better flushed. Productivity and standing crop were reduced in this area, but production per unit of biomass was higher than in the sections of the canal farther upstream (table 2). Harris concluded that the higher flushing rate near the mouth of the Ala Wai resulted in selection for a species composition with higher growth rates than those found in other parts of
the Ala Wai, although she did not directly examine the species composition of phytoplankton in the Ala Wai. Productivity varied throughout the year, being highest from April to July and lowest from August to November (table 3).

**Zooplankton**

J. N. Miller found 28 kinds of zooplanktonic organisms in the Ala Wai (table 4). Of these, the most important as potential members of the food chains of recreationally important fish species were copepods and their nauplii, both in numbers and in frequency of occurrence. Other important types included cirripede larvae, crab zoeae, medusae, and larval polychaetes. Chaetognaths and Oikopleura were often abundant, but were evidently not eaten by fish or crabs. Abundance of the various groups was relatively independent of location in the canal or season of the year. No attempt was made to determine zooplankton biomass or secondary productivity.

**Benthos**

J. N. Miller found five species of benthic macrofauna in the Ala Wai, all crustacea: Thalamita crenata (blue claw crab), Podophthalmus vigil (Hawaiian swimming crab), Scylla serrata (Samoan crab), Portunus sanguinolentus (white crab), and Squilla oratoria (mantis shrimp). Only the first two of these two species occurred in large enough numbers to be recreationally important. T. crenata occurred throughout the canal and lower stream. P. vigil avoided the head end of the canal (east of station 20) and the lower stream (stations 31-38), being most abundant at stations 8-13. Miller concluded that low concentrations of dissolved oxygen in the head of the Ala Wai and low salinity in the lower Manoa-Palolo stream limited the distribution of P. vigil. While T. crenata was less abundant than usual in the low-oxygen areas, it was very abundant in the stream stations and seemed more tolerant of variations in salinity than P. vigil.

Benthic infauna was not sampled.
Nekton

J. N. Miller found 19 species of fishes in the Ala Wai (table 5). Eleven of these were considered recreational species, but only six were abundant enough to be considered an important recreational resource. These included Chanos chanos (awa), Elops hawaiensis (awa awa), Mugil cephalus (mullet), Scomberoides sancti-petri (papio), Sphyraena barracuda (kaku, barracuda), and Tilapia mozambique (tilapia). Of these species, Elops is the most frequently fished for, using hook and line baited with live Molliesia. Chanos, while equally abundant and a desirable food fish, is rarely caught, as it is an algae eater and seldom takes a baited hook. Both species were present from June to December, disappearing between March and May. Bottom-living fishes, such as Apogon brachygrammus and Saurida gracilis, were absent in areas of low dissolved oxygen. Tilapia occurred primarily in regions of reduced salinity such as lower Manoa-Palolo stream. The carnivorous fishes (Elops, Scomberoides, and Sphyraena) fed largely on small fishes: gobids, nehu (Stolephorus purpureus), and probably apogonids, with only small amounts of crustaceans and other types of animal food. The herbivorous fishes (Chanos and Mugil) ate blue-green algae and diatoms; in addition, Chanos often ate zooplankton, particularly copepods.

Discussion

From the standpoint of the biota of the Ala Wai, the reaction of the phytoplankton is the most important consideration. Preliminary study of phytoplankton growth rates suggested that 10,000 m³/hr would be insufficient to dilute populations of small, rapidly-growing phytoplankton species. Species of phytoplankton from Kaneohe Bay grown in the laboratory have been found to have a doubling time as short as 8 hours (J. Caperon, personal communication), far faster than could possibly be flushed by any conceivable system. Further investigation has shown that two factors will greatly reduce growth rates in the Ala Wai from these laboratory levels.

First, flushing will greatly reduce nutrient levels in the canal. The present high levels, similar to the laboratory system, are maintained by regeneration from bacterial action on dead phytoplankton sinking to the bottom of the
canal. Flushing will remove the source of these nutrients by carrying away the dead phytoplankton. Thus the high-nutrient water presently characteristic of the canal will quickly be replaced by low-nutrient water either from offshore or from underground. Phytoplankton growth in low-nutrient water is slow, and flushing will maintain standing stocks at very low levels.

Second, phytoplankton growth rates are greatly reduced in flowing water. G. Prowse (personal communication) has found that phytoplankton in rivers are inhibited from reproducing by the moving water. The present high growth rates in the stagnant head end of the Ala Wai should therefore be reduced as the residence time of the water decreases. The proposed flushing rate of 10,000 m³/hr will be adequate to inhibit phytoplankton growth.

Other effects of a clear Ala Wai on its biota should be less drastic. Herbivorous fish will not necessarily be adversely affected by the drop in the phytoplankton population, as increased light penetration will produce an increase in attached benthic algae. Since the zooplankton do not presently appear to be food limited, their biomass will probably not change greatly. Zooplankton-feeding fish in turn should maintain much the same biomass as at present. Increased circulation in the head of the canal would prevent the buildup of anoxic areas, allowing the extension of benthic crustacea such as Podophthalmus vigil and bottom-living fishes like Apogon brachygrammus into areas they do not now occupy. Other fish species not now abundant might prosper in a clear Ala Wai, but conversations with fishermen indicate that the main species will probably be those present now, species adapted to shallow water of variable salinity with a soft bottom. However, clean water in the Ala Wai would greatly increase the desirability of its fish, so that recreational fishing will undoubtedly become very popular.

Summary

1) The Ala Wai supports an extremely productive phytoplankton community, which is limited by light rather than nutrients.

2) A variety of zooplanktonic species inhabit the Ala Wai, the most abundant group being copepods.

3) The most important species of benthic macrofauna are two species of crabs. One is found throughout the canal, the other avoids areas of low salinity and dissolved oxygen.
4) A number of fish species inhabit the canal. The recreationally important species feed either on small fishes or on algae and phytoplankton. 

5) Flushing the Ala Wai at 10,000 m³/hr will inhibit phytoplankton growth. Nutrient levels will be greatly lowered, resulting in a much less favorable habitat for phytoplankton, and the moving water will itself inhibit phytoplankton reproduction.

Literature Cited


Table 1
Primary Production, Average, March 1970-March 1971 (mg C/m³/hr)

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<tr>
<th>Station</th>
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<td>216.72</td>
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<td>H2</td>
<td>153.97</td>
<td>177.07</td>
</tr>
<tr>
<td>H3</td>
<td>354.15</td>
<td>260.14</td>
</tr>
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</table>

Table 2
Productivity per Unit Biomass, Average, March 1970-March 1971 (mgC/mg chla/hr)

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<tbody>
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<td>12.60</td>
<td>6.96</td>
</tr>
<tr>
<td>H3</td>
<td>12.22</td>
<td>5.81</td>
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Table 3
Primary Production-Seasonal Averages (mg C/m³/hr)

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<td>April-July</td>
<td>296.48</td>
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<td>August-November</td>
<td>112.22</td>
<td>134.78</td>
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<td>December-March</td>
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Fig. 1. Ala Wai Canal station locations.
<table>
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<td>Chaetognaths</td>
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<tr>
<td>Lucifer Protozoa</td>
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<td>472</td>
<td>828</td>
<td>104</td>
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<td>Nudibranchs</td>
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<td>222</td>
<td>3707</td>
<td>175</td>
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<td>Nitzes</td>
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<td>-</td>
<td>-</td>
<td>32</td>
</tr>
<tr>
<td>Molluscs</td>
<td>283</td>
<td>82</td>
<td>237</td>
<td>72</td>
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<tr>
<td>Notohids</td>
<td>32</td>
<td>-</td>
<td>-</td>
<td>16</td>
</tr>
<tr>
<td>Nematodes</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>16</td>
</tr>
<tr>
<td>Dikeypleura</td>
<td>200</td>
<td>40</td>
<td>64</td>
<td>64</td>
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<td>Ostreacods</td>
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<td>27</td>
<td>61</td>
<td>21</td>
</tr>
<tr>
<td>Polykstes</td>
<td>464</td>
<td>284</td>
<td>1896</td>
<td>412</td>
</tr>
<tr>
<td>Pseudodiactiums</td>
<td>28405</td>
<td>17</td>
<td>1837</td>
<td>132</td>
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<tr>
<td>Shrimp</td>
<td>268</td>
<td>94</td>
<td>284</td>
<td>768</td>
</tr>
<tr>
<td>Stomatopods</td>
<td>42</td>
<td>-</td>
<td>160</td>
<td>-</td>
</tr>
<tr>
<td>Number of tots</td>
<td>26</td>
<td>32</td>
<td>33</td>
<td>20</td>
</tr>
</tbody>
</table>
Table 5. Number of Fishes Captured and Distribution of Standard Lengths

<table>
<thead>
<tr>
<th></th>
<th>Number of Fish</th>
<th>Mean</th>
<th>Median</th>
<th>Range (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Albula vulpes</td>
<td>8</td>
<td>14.65</td>
<td>12.60</td>
<td>6.25-35.00</td>
</tr>
<tr>
<td>Apogon brevirostris</td>
<td>288</td>
<td>3.19</td>
<td>2.96</td>
<td>0.92-5.31</td>
</tr>
<tr>
<td>Arothorn hispidus</td>
<td>77</td>
<td>9.07</td>
<td>7.85</td>
<td>0.89-20.50</td>
</tr>
<tr>
<td>Bathygobius fuscus</td>
<td>32</td>
<td>4.23</td>
<td>4.07</td>
<td>2.16-6.75</td>
</tr>
<tr>
<td>Bothus pantherinus</td>
<td>1</td>
<td>12.70</td>
<td>12.70</td>
<td>12.70</td>
</tr>
<tr>
<td>Caranx sp.</td>
<td>1</td>
<td>11.65</td>
<td>11.65</td>
<td>11.65</td>
</tr>
<tr>
<td>Chanos chamos</td>
<td>25</td>
<td>41.60</td>
<td>43.50</td>
<td>28.00-52.40</td>
</tr>
<tr>
<td>Electric sandvicensis</td>
<td>27</td>
<td>7.67</td>
<td>8.40</td>
<td>4.00-12.00</td>
</tr>
<tr>
<td>Elops hawaienses</td>
<td>51</td>
<td>32.03</td>
<td>34.50</td>
<td>6.20-43.00</td>
</tr>
<tr>
<td>Mugil cephalus</td>
<td>29</td>
<td>24.70</td>
<td>25.00</td>
<td>10.50-34.00</td>
</tr>
<tr>
<td>Oxyurichthys longicu tus</td>
<td>68</td>
<td>5.11</td>
<td>5.32</td>
<td>1.80-7.42</td>
</tr>
<tr>
<td>Polydactylus sexilis</td>
<td>1</td>
<td>9.58</td>
<td>9.58</td>
<td>9.58</td>
</tr>
<tr>
<td>Saurida gracilis</td>
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<td>12.30</td>
<td>13.32</td>
<td>2.93-17.00</td>
</tr>
<tr>
<td>Scorberoides sancti-petri</td>
<td>12</td>
<td>26.86</td>
<td>30.00</td>
<td>13.00-35.50</td>
</tr>
<tr>
<td>Sphyraena barracuda</td>
<td>14</td>
<td>33.55</td>
<td>36.33</td>
<td>20.00-42.00</td>
</tr>
<tr>
<td>Stolephorus purpureus</td>
<td>21</td>
<td>4.97</td>
<td>5.09</td>
<td>3.46-6.52</td>
</tr>
<tr>
<td>Tilapia mozambique</td>
<td>26</td>
<td>20.11</td>
<td>20.00</td>
<td>14.00-27.00</td>
</tr>
<tr>
<td>Upponeus arga</td>
<td>8</td>
<td>6.66</td>
<td>6.03</td>
<td>3.49-10.20</td>
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<tr>
<td>Vitraria clarescens</td>
<td>1</td>
<td>3.47</td>
<td>3.47</td>
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</tr>
</tbody>
</table>
SECTION F

ESTIMATED COSTS
Estimated Costs

Well-supply alternatives

Miller and Chave (1976) estimated that a pipeline and pumping system capable of delivering 10,000 m³/hr. (approximately 60 mgd) to the head of the Ala Wai would require a capital investment of about $1,000,000. Current estimates (December 1976) place this figure at approximately $1,500,000.

The potential for supply of this same discharge from wells has been considered in relation to the approximate cost and efficacy of several alternative well fields, including clusters of wells of roughly 100-ft. depth with various numbers and individual capacities, lines of wells of roughly 100-ft. depth along the canal with various numbers and individual capacities, and a line of wells of roughly 300-ft. depth along the canal.

The consideration is extended to the lines of shallow wells and eventually the line of deeper wells because of the possibly great detriment of compaction of the lagoon silts and similar sediments underlying Waikiki that might result from their dewatering if within the drawdown cones of the wells. The drawdown in the Waikiki area would probably be greatest with the well-cluster scheme, less with the line of shallow wells if on the mauka side of the canal, and least with the line of deeper wells. Available information suggests that the sediments (and interbedded basalt) in the depth range between about 100 and 200 feet below sea level are relatively impermeable. Hence estimation for the deeper-well scheme assumed well depths of 300 feet.

Rough costs of the wells and pumps in the alternative schemes are estimated as indicated below. Operating power costs would be the same for any of the well schemes and may be somewhat less than for the pipeline scheme.

<table>
<thead>
<tr>
<th>No. of wells</th>
<th>Depth, ft.</th>
<th>Casing id., in.</th>
<th>Well cap. mgd.</th>
<th>Costs, $ per well</th>
<th>Costs, $ per pump</th>
<th>Sub-total</th>
<th>Elec. connection &amp; pump housing (15,000/well)</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>100</td>
<td>18</td>
<td>4</td>
<td>11,000</td>
<td>20,000</td>
<td>465,000</td>
<td>225,000</td>
<td>690,000</td>
</tr>
<tr>
<td>60</td>
<td>100</td>
<td>12</td>
<td>1</td>
<td>7,500</td>
<td>9,000</td>
<td>990,000</td>
<td>900,000</td>
<td>1,890,000</td>
</tr>
<tr>
<td>15</td>
<td>300</td>
<td>20</td>
<td>4</td>
<td>36,000</td>
<td>20,000</td>
<td>840,000</td>
<td>225,000</td>
<td>1,065,000</td>
</tr>
</tbody>
</table>
Consolidation and hydraulic conductivity studies

The estimation of the importance of consolidation of the sediments underlying Waikiki will require a study of the consolidation characteristics of these sediments. An engineering firm has provided an unofficial rough estimate for such a study of $3,000. This would include the cost of case drilling two 30-ft. test holes for consolidation test samples as well as compilation and analysis of geologic information from previous test-hole drilling in the area.

The consolidation estimation will also require estimation of the extent of drawdown in areas of importance. Such estimation will require pumping tests on test wells to determine hydraulic conductivity. Two 100-ft. 6 in. i.d. test wells could probably be drilled and cased for such tests for about $8,000 in total, and the pump tests would probably cost on the order of $4,000.

Investigation of the feasibility and effects of drawing the water from greater depths will require drilling a couple of test wells to depths of at least 300 feet, which would cost on the order of $25,000. The cost of pump tests on these deeper test wells would be about the same as that for the shallower test wells.

Note

None of the costs estimates presented above are intended for use beyond general planning purposes. Detailed estimates should be made for the schemes that appear most attractive.