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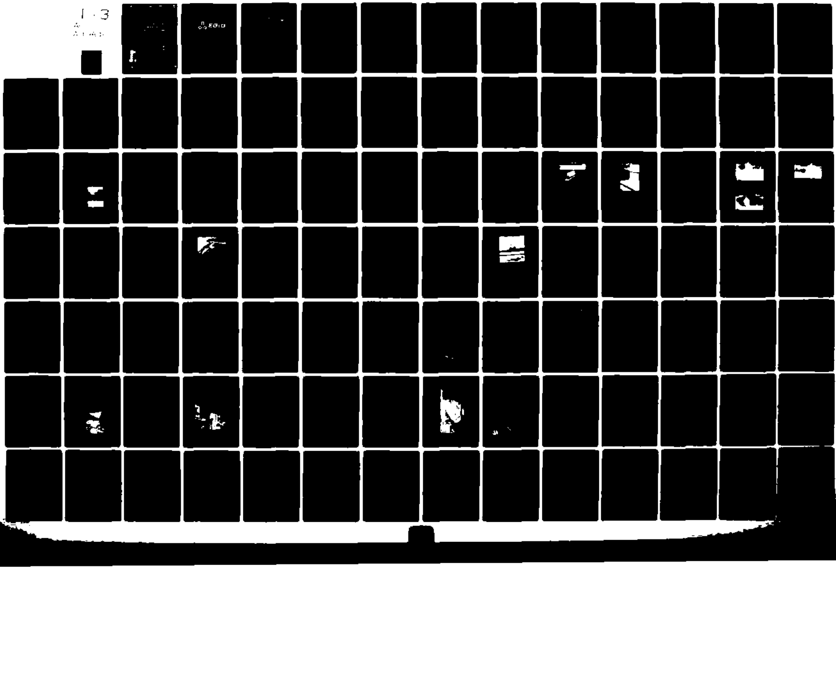

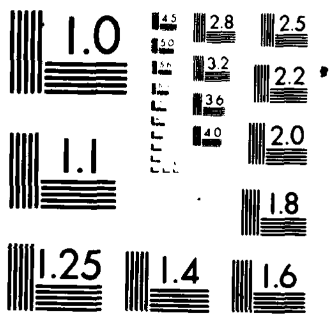

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DREDGING AND SEDIMENTATION CONTROL -- 1980

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WASHINGTON ANALYTICAL SERVICES CENTER, INC.

DREDGING AND SEDIMENTATION CONTROL -- 1980

PROCEEDINGS OF A SYMPOSIUM HELD IN ALEXANDRIA, VIRGINIA, 22-23 APRIL 1980

SPONSORED BY:
NAVAL FACILITIES ENGINEERING COMMAND
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PREFACE

This volume records the proceedings of the First United States Navy Symposium on Dredging and Sedimentation Control. The symposium was held on 22-23 April 1980 at the Guest Quarters Hotel in Alexandria, Virginia under the sponsorship of the Naval Facilities Engineering Command and the Office of Naval Research. Arrangements for the symposium and publication of the proceedings were accomplished by EG&G Washington Analytical Services Center, Rockville, Maryland under contract to the Office of Naval Research.

Attendence at the symposium was by invitation only and, within travel restrictions that were operative in the spring of 1980, most perspectives of Navy interest in the subject area were represented, from the Office of the Deputy Under Secretary through the Fleet. Also only invited presentations were given, and topic areas were assigned to those participants on the program. The intent was to provide a general review of the state-of-the-art and findings obtained to date in the NAVFAC sponsored dredging research, development, test, and evaluation project and to present a compilation of information on topics pertinent to the Navy's problem at Sewell's Point. Subsequent to the symposium, each presentor was requested to provide a written synopsis of his remarks for inclusion in these proceedings. It is hoped that their publication will provide a useful reference by setting forth this material for the record, will provide a vehicle for sharing some of the information presented at the symposium with those who could not attend, and will serve to focus attention on recent advances in our knowledge of the subject area.

In the interest of making this volume available in the shortest possible time, the papers are printed in essentially the form in which they were received from the respective authors, the editorial function being held to a bare minimum. References appearing in the text of this document are listed at the end of each section. The papers herein were not referred nor were they submitted to the authors for review in their final form. Also, the attentive reader will note that the titles of the papers as published in the proceedings do not always coincide exactly with the title of the presentation as listed in the agenda. This is merely a reflection of the author's preference and does not reflect a substantive change in content material.

One of the most important aspects of a symposium of this type is the discussion that follows the presentation of a paper and the informal sharing of ideas and information that takes place outside the formal program. Unfortunately, there is no way that such material can be covered in a proceedings of this type, but its value to the Navy is considered to be positive and permanent.

The success of this symposium has reinforced the earlier thought that it might be the first of a series on the subject area. Consequently, the Second United States Navy Symposium on Dredging and Sedimentation Control is being scheduled for the Spring of 1981, to be hosted by the Scripps Institution of

Oceanography and held in La Jolla, California. Preliminary plans are now being formulated for the third symposium, to be held in the southeast in the Spring of 1982.

PHILIP E. SHELLEY, PH.D.
Editor

ACKNOWLEDGEMENTS

The First Symposium on Dredging and Sedimentation Control could not have been the success that it was without the efforts and cooperation of a number of people, who must go unrecognized by name due to space limitations. However, special recognition and thanks must be extended to those persons on the program who took time from already busy schedules to prepare presentations for the symposium and write papers for inclusion in these proceedings.

Thanks are also due to all those who attended and shared their thoughts and perspectives on the subject during the discussions following each presentation and the informal sessions held outside the regular agenda.

Finally, appreciation is extended to those members of the staff of the Naval Facilities Engineering Command and EG&G Washington Analytical Services Center, Inc. who gave of their time and effort to help with the myriad of details essential to the smooth running of the symposium and the publication of these proceedings.

STEPHEN M. HURLEY
Convenor of the Symposium

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INTRODUCTION

The First United States Navy Symposium on Dredging and Sedimentation Control was held in April 1980 at Alexandria, Virginia. The symposium sought to provide a common forum for workers in the several areas and disciplines that deal, sometimes only secondarily, with the topic area. To set the stage for the reader of these symposium proceedings who has not been active in the Navy's dredging project or has had only peripheral involvement, a brief review of the problem and project as presented in the Development Plan will be given here.

In order to obtain ship access to and from the sea, Naval installations that include harbor and pier facilities have been built in coastal regions, most often in natural harbors and along waterways. National defense requirements mandate that this access of the Fleet to and from the sea be free and unrestricted. However, in many of these Naval facilities, continued sedimentation around berths and docks causes interference and delay in free access of the Fleet. In a few instances, notably Norfolk, Virginia, the situation is compounded by the presence of marine organisms that can cause sea-suction fouling problems. The present solution is to remove and relocate the material by conventional maintenance dredging, which is accomplished through contracts with the Army Corps of Engineers, privately-owned dredging firms, and three Navy-owned hydraulic dredges. At the present time, an estimated 10 million cubic yards of sediment must be removed annually from existing Naval facilities. About half of this material is removed from five Navy harbors: Charleston Naval Station, Alameda Naval Air Station, Mare Island Naval Shipyard, Mayport Naval Station, and Norfolk Naval Station.

Four Federal laws enacted since 1969 that significantly affect the Navy's dredging program are the National Environmental Policy Act (NEPA) of 1969, the Marine Protection Research and Sactuaries Act of 1972 (MPRSA), the Coastal Zone Management Act (CZMA) of 1972, and the Clean Water Act of 1977. Prior to passage of these laws, the business of dredging was primarily one of economics, while concern for the environment played a minor role. These laws and their amendments, agency interpretations, and court decisions indicate that the Navy must consider the environmental impact of dredging. Requirements for conducting environmental studies, monitoring dredge disposal areas and, especially, greatly increased transport to more environmentally suitable spoil areas have, when coupled with inflation, led to two-to-thirteen fold increases in Navy dredging costs since 1971. The total annual cost to the Navy for dredging and disposal of sediment materials is currently estimated to be $30M per year and is projected to continue its upward spiral unless some intervening action is taken. Such expenditures for maintenance dredging in order to assure Fleet readiness represent a diversion of funding that could be used to better advantages elswehere, and it is quite possible that projected continual cost increases will have to be made up by diverting funds from other Navy programs. Thus, there are urgent requirements, economic, environmental, and operational for alternatives to conventional maintenance dredging.

The overall objective of the Navy's dredging research, development, test, and evaluation project as set forth in NDCP Y0817SL is "to maintain free access by surface ships and submarines to and from Navy Berths and docks without interference from sedimentation hazards," and the overall requirement is "to reduce the rising costs associated with the disposal of dredged materials in accordance with USEPA standards and procedures."

The project is designed to meet its overall objective by providing a range of proven sediment and marine organism control concepts that can be synthesized into optimum system solutions for each individual harbor. The specific project objective is to develop the technology for sedimentation control to the extent that, when applied to all candidate Navy harbors, the annual cost of maintenance dredging will be reduced by at least 50 percent. An investment payback period of two years for the majority of installations is also a project objective.

The recommended approach is to expand the present advanced development project to develop and demonstrate new sediment management techniques as viable alternatives to conventional dredging, to consider it as a candidate for transition to engineering development in FY 1983, and to achieve Initial Operational Capability (IOC) by FY 1987. When coupled with the increased understanding of sedimentation processes that will result from the proposed effort, this course of action will allow the implementation of locally-optimum solutions to be effected on a Navy-wide basis.

The papers which follow in these proceedings present some of the findings resulting from project activities, summarize the state-of-the-art as it pertains to project concerns, and address some of the Navy's problems, especially those encountered at the Sewell's Point area in Norfolk, Virginia.

PHILIP E. SHELLEY, PH.D.

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
200 STOVALL STREET
ALEXANDRIA, VA 22332

IN REPLY REFER TO

032P/SMH
Ser: 503
31 MAR 1980

From: Commander, Naval Facilities Engineering Command
To: Distribution

Subj: Dredge Sedimentation and Marine Organism and Control Meeting

Encl: (1) Preliminary Agenda, Dredge Sedimentation and Marine Organism Control Meeting 22-23 April 1980
(2) Guest Quarters Brochure

1. National defense requirements mandate free and unrestricted access by the Fleet to and from the sea. Sedimentation and sea suction problems in berths and harbor installations continue to cause interference and delay in obtaining free access by the Fleet. Alternative solutions for the problem are being developed by this Command.

2. As a part of our efforts to develop alternatives NAVFAC will sponsor a meeting to review and discuss new methods for sedimentation control. The purpose of the meeting is two fold; (1) to review progress by the research program and (2) to discuss new methods for transferring sedimentation control theory into engineering practice. Enclosure (1) is the proposed agenda. NAVFAC has requested each speaker to prepare a paper on the topic listed in the agenda. As a minimum, an outline of the paper will be available for the meeting. The completed paper will be compiled into meeting proceedings for future reference. It is planned to have the proceedings available not later than 1 June 1980.

3. You are invited to send a representative to the meeting. Enclosure (2) provides overnite accommodation information. A block of rooms will be held on a first come first serve basis. Please make reservations directly with Guest Quarters, Phone: (703) 370-9600 or (800) 424-2900. Attendees are invited to join in the scheduled luncheons. Deadline for Guest Quarter reservations is 11 April 1980. For additional information on meeting arrangements contact Mr. S. Hurley, NAVFAC 032P, Phone (703) 325-9044 or Autovon 8-221-9044.

A. A. ARCURI
Assistant Commander for
Research and Development

Distribution List (See next page)

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Subject: Dredge Sedimentation and Marine Organisms Control Meeting

Date: 22-23 APRIL 1980

Meeting Location - Guest Quarters - 100 South Reynolds Street
Alexandria, VA 22304
(703) 370-9600

Sponsored by: Naval Facilities Engineering Command
200 Stoval St.
Alexandria, VA 22332

AGENDA

DATE	TOPIC	SPEAKER	ORGANIZATION
22 April Tuesday AM			
	Welcome	Arcuni	FAC
8:30	Introduction	Hurley	FAC
8:45	. Causes of Sedimentation in Naval Harbors	Inman	SIO
	. Navy Maintenance Problems & Cost	Malloy	CEL
	. Current Dredging Practices	Hoffman	USNA
	. Sedimentation Control Experiments at Mare Island Naval Shipyard	Jenkins	SIO
	. Design Guidelines for Water Jet Arrays	Bailard	CEL
1230	. Lunch Break		
1300	. LUNCHEON (Quarter Pub Restaurant (2nd Floor))	Montoya	OPNAV
22 April Tuesday PM			
1415	. Sedimentation History & Predictions in Norfolk, VA Region - Lower Chesapeake	Ludwick	ODU
	Lower James River	Nichols	VIMS
	. Marine Organisms in Norfolk, VA Region	Diaz	VIMS
	. Ship Cooling Systems	CDR Jones	PCU CARL VINSON CVN 70
1700	. Adjourn		
23 April Wednesday AM			
8:30	. Modeling - How & When to use	MacAnally	WES
	. Alternatives for Sedimentation Control at Norfolk NS Pier 10-11-12	Inman	SIO
	. Mayport Basin Field Test	Inman	SIO
	. Project T&E Plans	Shelley	EG&G
1200	. Break		
1300	. LUNCHEON (Quarter Pub Restaurant (2nd Floor))	-	-
23 April Wednesday PM		Moderator	
1330	. PANEL: Topics of interest to group or extra time for previously presented topics.	Hurley	FAC
3:00	. Adjourn		

DREDGE SEDIMENTATION AND MARINE ORGANISM CONTROL MEETING

22-23 APRIL 1980

ATTENDEES

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Part 9

DREDGING AND SOUNDINGS

7901. Policy

Dredging requirements at base complexes will be consolidated for project accomplishment purposes to the maximum extent feasible. The responsibility for determining requirements, programming, budgeting, and funding shall be assigned to specific shore activities and their major claimants, as shown in Table 7-1.

7902. Designation of Lead Activities

The following activities at the area complexes indicated are designated as "lead activities."

Area Complex	Lead Activity
Newport, Rhode Island	NAVEDTRACEN Newport
Norfolk, Virginia	NAVSTA Norfolk
Guantanamo Bay, Cuba	NAVSTA Guantanamo
Pensacola, Florida	NAS Pensacola
Great Lakes, Illinois	NTC Great Lakes
San Diego, California	NAVSTA San Diego
Pearl Harbor, Hawaii	NAVSTA Pearl Harbor
Guam, Marianas	NAVSTA Guam
Yokosuka, Japan	FLEACT Yokosuka
Subic Bay, Republic of the Philippines	NAVSTA Subic Bay
Charleston, South Carolina	NSY Charleston

7903. Responsibilities

a. Area Coordinator

(1) Establish definitive limits of water areas of responsibility at base complexes located within the United States. Determine "common use" areas for assignment to the "lead activity" and those water areas which are to be the specific responsibility of shore activities.

(2) Designate "lead activity" to be responsible for developing dredging requirements for "common use" water areas at locations not specifically designated in paragraph 7902.

TABLE 7-1
Maintenance Dredging Responsibilities

	SPECIFIC MAINTENANCE (Para 5103 of OPNAVINST 11010.20D)				CONTINUAL MAINTENANCE (Para 5103 of OPNAVINST 11010.20D)			
FACILITY	Est Cost	Requirements	Program & Budget	Funding	Est Cost	Requirements	Program & Budget	Funding
COMMON USE	\$0–10,000	Assigned Lead Activity	Assigned Lead Activity	Assigned Lead Activity from Operating Budget	No Limit	Assigned Lead Activity	Assigned Activity & Major Claimant	Assigned Lead Activity from Operating Budget
	over 10,000	Assigned Lead Activity	Assigned Lead Activity & Major Claimant	Major Claimant on Project Basis				
SPECIFIC USE (ACTIVITY (FACILITY)	\$0–25,000	Assigned Activity	Assigned Activity	Assigned Activity from Operating Budget	No Limit	Assigned Activity	Assigned Activity & Major Claimant	Assigned Activity from Operating Budget
	over 25,000	Assigned Activity	Assigned Activity & Major Claimant	Major Claimant on Project Basis				

(3) Designate shore activity or activities which will be responsible for the "specific use" areas.

(4) Insure that all dredging requirements in the same base complex are coordinated.

(5) Provide CNO (Op-44) a copy of a map showing the water areas of responsibility and the lead activity and/or shore activity to which the dredging responsibility has been assigned.

(6) Coordinate the financing of dredging projects in those cases where funding is the responsibility of more than one major claimant.

b. Fleet Commanders

(1) Same functions as area coordinator (listed above) except that responsibility covers overseas bases only.

c. Lead Activities

(1) Develop dredging requirements (including soundings) for "common use" areas of harbors or waterways assigned.

(2) Develop and sponsor consolidated dredging programs for the base complex.

(3) Budget for "common use" dredging requirements and their pro rata share of the dredging program/projects for any "specific use" areas assigned.

d. Shore Activities having Assigned Water Areas

(1) Determine dredging requirements (including soundings) for those water areas assigned.

(2) Budget for their pro rata share of any consolidated dredging program/project developed by the lead activity.

e. Public Works Centers and Public Works Lead Activities (Located at Base Complexes)

(1) Render technical assistance and advice to the "lead activity" and/or shore activities in the development of dredging requirements, schedules, and programs/projects.

(2) Execute dredging programs/projects when such programs/projects are authorized and funded.

f. Naval Facilities Engineering Command/Engineering Field Divisions

(1) Render technical advice and assistance on dredging matters as required and as necessary.

(2) Maintain liaison with the Corps of Engineering for the purpose of coordinating Navy dredging requirements and programs with the dredging operations of the Army.

(3) Advise and assist the CNO on overall Navy dredging matters.

g. Major Claimant

(1) Approve for execution and funding the O&MN portion of dredging programs/projects at Naval activities under their command.

(2) Insure that appropriate requests are included in their budget submissions.

7904. Funding

a. Maintenance dredging will be financed from the Operations and Maintenance, Navy, Appropriation; Research, Development, Test and Evaluation Appropriation; and/or the Navy Industrial Fund, as applicable, consistent with the assigned responsibilities.

b. Dredging for excess depths and widths of channels or basins, where the Navy Department and not the Corps of Engineers is responsible for the additional depth or width, shall be financed from appropriations available for construction.

c. In those cases where maintenance dredging and excess depth or width dredging projects are combined for contract accomplishment purposes, the categories of work shall be approved and funded on a pro rata basis; i.e., maintenance from O&MN and excess depth/widening from appropriations available for construction.

SECTION 1

OPENING AND MAINTAINING TIDAL LAGOONS AND ESTUARIES

by

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and

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INTRODUCTION

Historically the Navy has sited her port facilities within tidal lagoons and estuaries where natural quiet water was found compatible with ships of 150 to 200 years ago. Sedimentation in tidal lagoons and estuaries has now become an acute problem, particularly where there has been over-dredging to accommodate modern deep draft warships (15m). The rise in mean sea level ≅0.03 cm/yr, which created natural estuaries, has not kept pace with the Navy's demand for greater draft ships. The limited number of navigable lagoons confronting a demand for certain strategic inland waterways has encouraged reconstruction dredging of some closed or partially filled lagoons.

Man's efforts to deepen existing or relict estuarine systems have disturbed the steady state equilibrium of the systems. Sedimentation acts continuously to restore this equilibrium by either of two processes. One is flocculation of fine-granted fluvial born sediments, a process accelerated when an estuary is deepened thereby allowing greater salt wedge instrusion. The other is the interception of the longshore transport of coarse-grained beach sediments by the lagoon inlet. The larger tidal prism of an enlarged lagoon draws a greater percentage of the longshore transport into the lagoon from the adjacent surf zone. In the absence of wave suspension within the lagoon, very little of this sediment is carried back out of the lagoon on ebbing tides.

These sedimentation processes confront NAVFAC with several distinct problems. A particular set of countermeasures are needed against accumulations of cohesive fines from the rivers, another set against accretion of cohesionless coarse-grained sediments from the beaches. Still other methods are needed to reopen a closed lagoon.

Dredging has been the most widely practiced solution to all of these problems for the past 150 years. It is a solution the Navy may not be able to afford indefinitely. In addition to intrinsic high rates of energy consumption and equipment wear, there are ever-growing costs associated with dredge spoils disposal. Ninety percent of the material annually dredged by the Navy is contaminated by heavy metals, concentrated in the flocs from either natural erosion of country rock or industrial sources (Malloy, 1980). More contamination results the longer fine sediments remain immobile on the harbor bottom because of chemical and oil spills, sand blasting, paint removers, and other ship and industrial waterfront activities. Sand is rarely contaminated because it is chemically inert. The contaminated sediments pollute the disposal sights, imposing additional costs to measure the pollution and minimize its effect. Recent climatic cycles and real estate developments have accelerated erosion and sediment runoff, ultimately bringing many spoils ponds to full capacity before their expected lifetime (N.E.S.O., 1976). Ocean dumping of poisoned spoils is environmentally unsound and strictly limited by environmental protection laws. Alternative spoils ponds are generally available only at a great distance from the lagoon, requiring expensive booster stations and additional pumping capacity to transport the dredge spoils (Little, 1975).

This paper reports on five separate prototype scale field experiments that test alternative measures to dredging. Two of these experiments evaluate techniques of resuspension and exclusion for reducing fine sediment accumulations in quiet water berths, where the observed shoaling rates are greatest and dredging most difficult. The fine sediment control studies were performed in and around berths at Mare Island Naval Shipyard. Another two experiments involved by-passing sand around the inlet of Agua Hedionda

Lagoon, California, using fluidized trenches funnelling into a crater sink. A final experiment used open trench fluidization to reopen Penasquitos Lagoon, California.

SEASONAL AND EPISODIC MUC ACCUMULATION IN BERTHS

There are abundant data showing that the deposition rates of settled flocs in berthing areas and around structures within a tidal estuary exceed those in unobstrcted navigation channels (N.E.S.O., 1978; Van Dorn, Inman, McElmury, 1977; 1978). This was dramatically shown in Mare Island Straits, California, during the record flood, winter and spring of 1978, that brought an end to the California drought which began in 1945. Figure 1-1 compares the average bottom shoaling rates along a line extending across the navigation channel with the shoaling rates at two stations within the finger pier complex at Mare Island Naval Shipyard, pictured in Figure 1-2. Mare Island Straits are over-dredged to 10m and situated at the confluence of the Napa and Sacramento Rivers on the eastern side of San Pablo Bay in the San Francisco Estuary. The shoaling at both stations within the finger pier berths at Mare Island were found to be three times the mean sedimentation rates across the navigation channel. Mud was found to accumulate at nearly uniform rates in both the channel and berthing areas through the wettest months of February and March, when the combined fresh water discharge of the

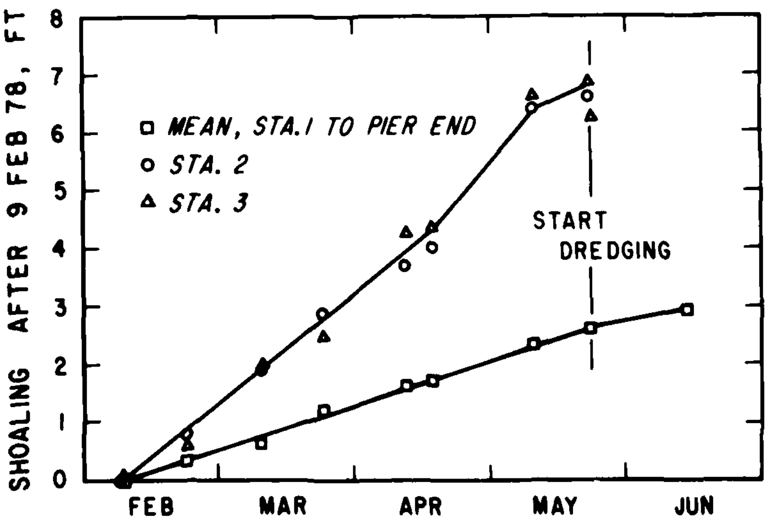

Figure 1-1. Shoaling History Within a Berth and Across a Navigation Channel

Napa and Sacramento Rivers ran as high as 2.12×10^3 m^3/sec, about 1/2 the tidal flux in Mare Island Strait. When the river discharge dropped to 1.13×10^3 m^3/sec by the end of April with subsidence of Pacific storms, the sedimentation rate in the finger pier berths abruptly increased.

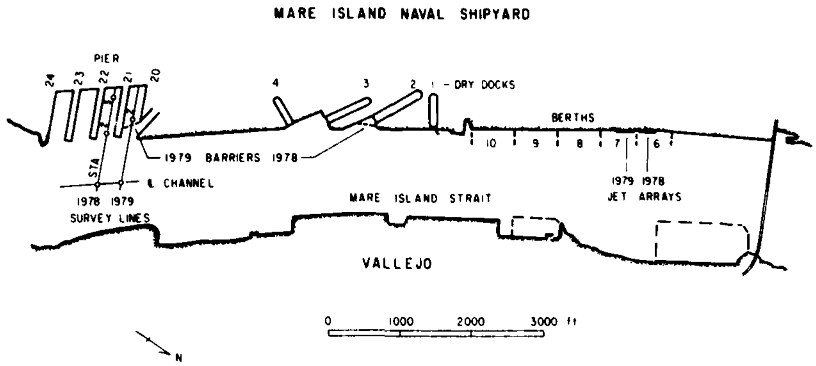

Figure 1-2. Mare Island Strait, Ebb Current and River Flow to the Left

By this time 2 meters of new deposition had occurred, four times the shoaling during the previous drought year of 1977 (Van Dorn, Inman, and McElmury, 1978).

Initially, new deposits of mud are fairly mobile, fluid mud, with a low threshold of motion of typically 15 cm/sec depending upon the time of immobility of the mud (Le Mer, 1963; Van Dorn et al., 1977). In the navigation channels these threshold stress levels ($\cong$ 1 dyne/cm^2) are exceeded more frequently, particularly if not over-dredged. Some fraction of the newly deposited fluid mud is resuspended and flushed out by tidal circulation. The remaining fraction of resuspended floc settles again next slack water, eventually reaching quiet water. The quiet water of the berthing areas allows for much less resuspension of fluid mud and greatly restricts circulation that might remove the resuspended material. Low density fluid mud 10-15 gm/l) which is not resuspended within 24-72 hours begins to compact under gravity, driving out interstitial water. After a week it becomes a high density (1.22 gms/cm^3) anerobic mud (Krone, 1962), which can then only be moved by mechanical means. Eddies from pier piles, ship hulls, and other vertical structures in and around the berth increase particle collisions and mix the higher salinity bottom water into the remainder of the water column, further promoting flocculation. These factors all play a role in contributing to the higher siltation rates of berthing areas, although the relative importance of each may vary from place to place.

The seasonal variations of runoff in turn cause seasonal variation in sediment abundance and water properties. Figure 1-3 shows the vertical distributions of suspended sediment concentration (C), salinity (S), and temperature (T) from mid winter through summer 1978 at Station 3 inside the berth at Pier 23. Salinity and temperature are scaled on the horizontal axis in $^{o}/_{oo}$ and °C along with concentration in gm/l. These data were collected during spring tides at four phases of tidal elevation labeled H, L, F, and E for high, low, flood, and ebb respectively. The salinity profiles show that the river discharge during the maximal rainfall months, from February until mid April, produced a thick lens of fresh water on the surface and rather low bottom salinities of 1-7 $^{o}/_{oo}$. The salinity was uniform over depth at low tide indicating total retreat of the salt wedge from the berths by the end of the ebbing tide. By mid-June most of the Sierra snow pack had melted, and the discharge of the Sacramento and Napa had fallen to 1.7×10^{2} m^{3}/sec. The fresh water surface lens then grew thinner into summer until finally an isohaline vertical distribution of $20^{o}/_{oo}$ results from the tidal flux. The temperature distribution remains isothermal, since thermal diffusivities ($\cong 10^{-3}$ cm^{2}/sec) are large compared with diffusivities of salts ($\cong 10^{-5}$ cm^{2}/sec).

With the declining river discharge in late March, the salt wedge intrudes more freely into the berthing areas and bottom salinities reach the flocculation threshold of $7\text{-}10^{o}/_{oo}$ (see Krone, 1962). From this time on through summer, suspended sediment profiles show a high concentration toe ($\cong$ 100 mg/l) in the lower meter of the water column. The toe is due to the settling of flocs toward the bottom boundary and the resuspension of those flocs by the tidal and eddy motion over pier piles, dredge banks, and other bottom obstructions. However during periods of subthreshold salinities, e.g., following a train of local storms, 9 March 1978, the sediment remains uniformly distributed through the water column in spite of high sediment abundance.

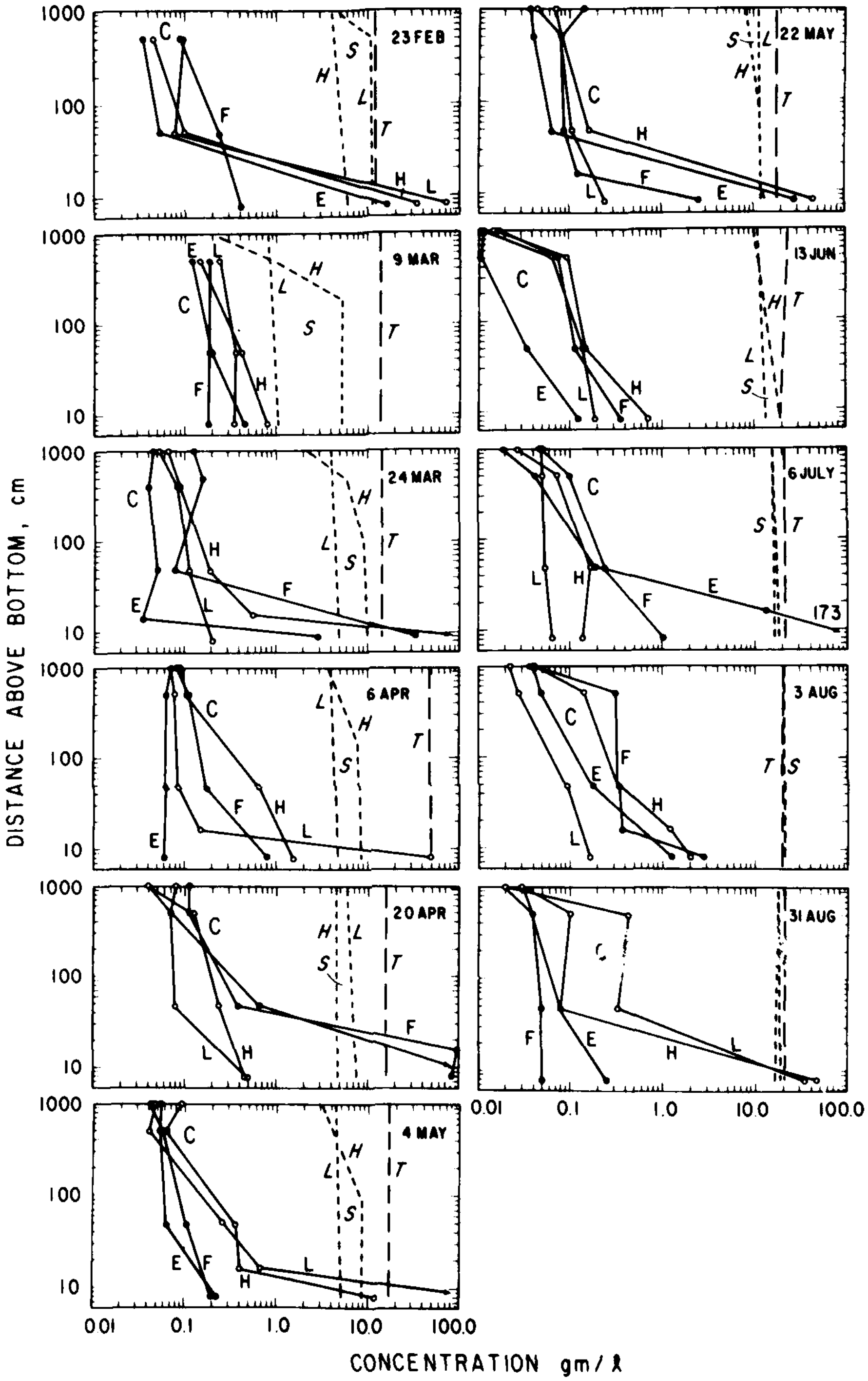

Figure 1-3. Vertical profiles of: Concentration, gm/l; salinity, $^{o}/_{oo}$; and temperature, °C

The bi-weekly profiles of discrete properties in Figure 1-3 give a good resolution of the seasonal trends but may be somewhat aliased by weekly or daily variations. These short-term fluctuations are most prevalent during the wet months as river discharge fluctuates daily in response to local storms passing over the watershed. As a result of these fluctuations, fine sediment deposition will exhibit episodes of extremely rapid build up or "mud storms" when bottom salinities are near the flocculation threshold (7-10$^{\circ}/_{oo}$) while rivers remain laden with charged clay particles from recent runoff. Under these conditions four such mud storms, each depositing 0.6m or more of new fluid mud at a time, were observed in the entrance to Pier 21N at Mare Island during a near record flood winter in 1980 (see Figure 1-4). This echogram was taken using a 40 kHz echo transceiver. Each dated horizon

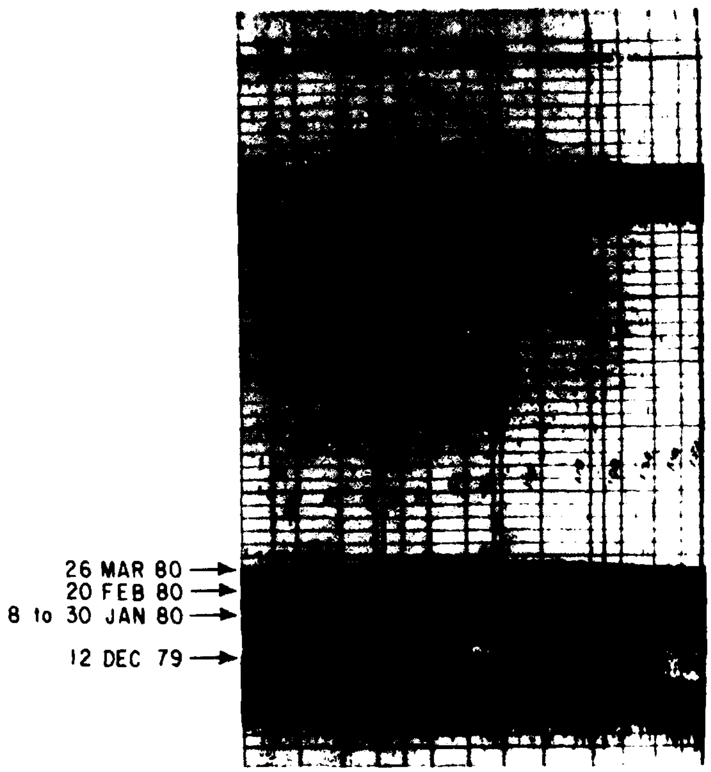

Figure 1-4. Echogram of Entrance to Pier 21N at Mare Island During Winter 1980

is a veneer of loose floc trapped between thick layers of denser more consolidated fluid or anerobic mud. The trapping of these floc layers implies that the subsequent build-up of mud occurred so rapidly that insufficient time had lapsed for the water content of the floc layer to diminish. The accumulations of these mud storms are found between the dates of the horizons following the initial pulse of runoff from series of Pacific storms. The rise in bottom salinity in between these pulses triggers flocculation and a subsequent abrupt build-up in fluid mud. The flocs themselves appear as a speckled pattern in the water column between the surface reflection and the first bottom horizon on the echogram.

To a lesser degree the river discharge fluctuates during drier summer months, principally in response to manipulation of containment reservoirs and dam diversions. These fluctuations do not produce the episodes of rapid sediment accumulation found during the wet winter months. Shoaling in the summer months is characterized by a nearly steady state build up of about 0.25 cm/day. Whereas the controlling variable for shoaling in the winter is a bottom salinity of sufficient magnitude to induce flocculation in the presence of high sediment abundance, the summer shoaling rate in the presence of higher salinities appears strictly limited by the small amounts of suspended sediment brought in by the rivers. Krone (1959, 1960) has shown by tracer studies that suspended loads in the lower 80 percent of the water during the summer are in fact resuspended by wind waves over permanent shoals elsewhere in the estuary and carried subsequently into berthing areas by the density-stratified tidal flows.

Although these seasonal and episodic variations of water properties and fine sedimentation were observed at a single place, the mechanisms of flocculation and resuspension are at work in almost every tidal lagoon and estuary. The fact that these observations were made during an extreme in the weather cycle and covered maximum ranges of temperature, salinity, suspended sediment, and shoaling rate, should make them a useful design guide for sediment control measures anywhere.

CONTROL OF MUD ACCUMULATION BY RESUSPENSION

The first attempt to control mud accumulation by resuspending newly deposited layers of fluid mud date back to the Chinese in the fifth century A.D. Figure 1-5 shows a rolling suspensifier, the hun chiang lung, first illustrated in the Ho Kung Chhi Chii Thu Shuo and reproduced here from Needham (1974). This device was drawn along the bottom by a vessel or team of horses proceeding upstream. The teeth on the roller raised clouds of silt which were carried away on the ebbing tide.

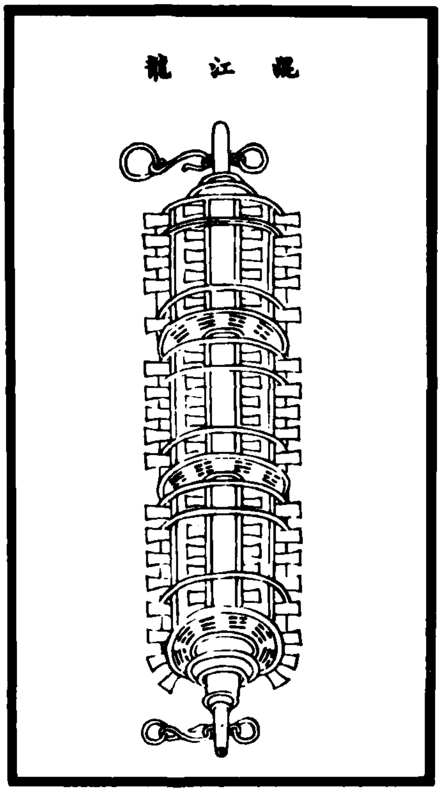

Figure 1-5. Fifth Century A. D. Rolling Suspensifier

The modern equivalent of the Chinese suspensifier is the tide actuated water jet array shown in Figures 1-6 and 1-7 tested on the water front of Mare Island at Berth 7 shown in Figure 1-1. The fundamental environmental constraint in the successful application of any such resuspending device is

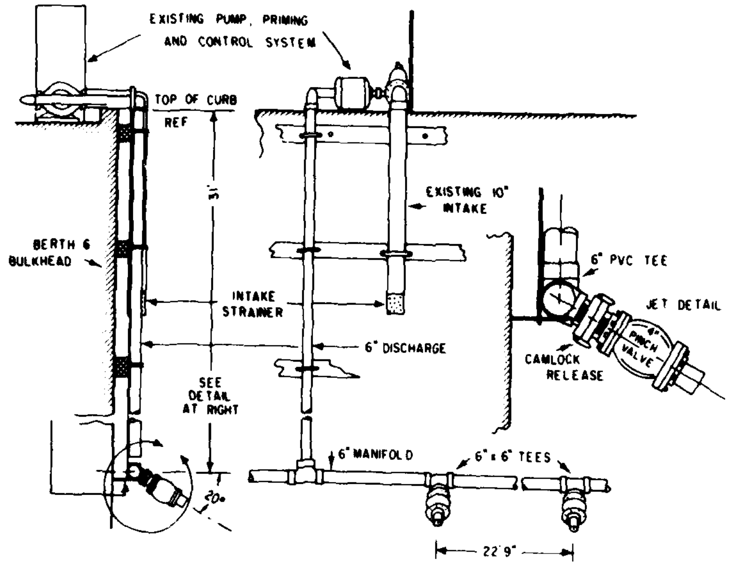

Figure 1-6. Quay Wall Installation of Water Jet Array

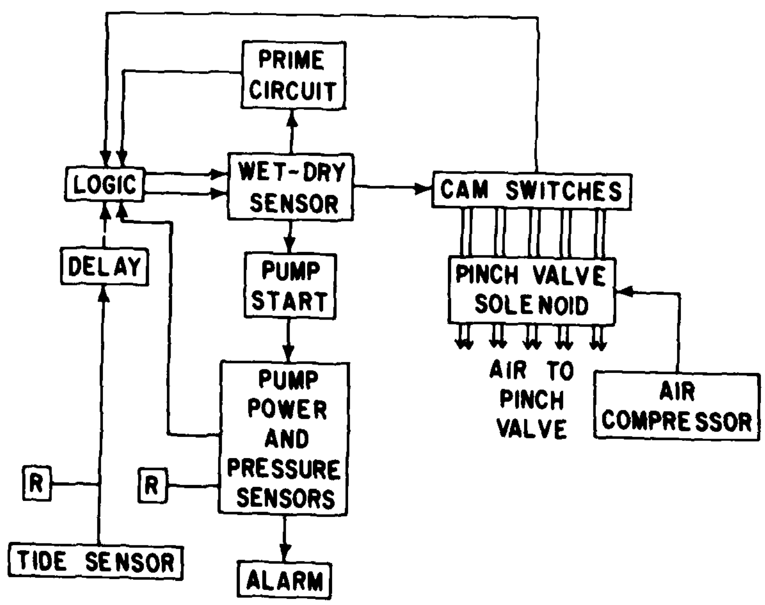

Figure 1-7. Tide Actuated Duty Cycle Control System for Jet Array

the presence of unidirectional currents for a sufficient period to advect away the material which has been resuspended. Berth 7 at mare Island is ideal in this respect. This berth rests along the streamlined western shore of Mare Island Straight where 1200m of unobstructed concrete quay wall stabilizes the bank against over-dredging to a depth of 10m. Here bottom ebb current commences about 2.5 hours before high tide and persists for about 5 hours giving a rather long window to transport resuspended material. The weakest ebb current amplitudes at low river discharge (10^2 m^3/sec) still produce near threshold stresses for newly deposited fluid mud, rising from 20 cm/sec at 0.5 meters above the bottom to 80 cm/sec on the surface. The tide actuated switching circuit entered in Figure 1-7 was synchronized for a 4 hour duty cycle through the period of maximum ebbing bottom currents.

The water jet array itself, Figure 1-6, was designed to operate from a fixed mounting on the quay wall where vulnerability to damage from dragging anchors and channel dredging activities is minimized. The most protruding components, the jet nozzles themselves, are secured to the discharge manifold by a quick-release Camlock clamp for easy diver replacement in the event of damage or malfunction. Ten equally spaced 7.3 cm diameter jet nozzles comprise the linear array 63 meters in length at a depth of 8.2 meters below MLL sea level. The jet array was driven by a 1910 gpm water pump at 92 psi powered by a 140 hp, 220 vac electric motor. The automatic switching circuit, Figure 1-7, sequences the entire pump discharge through each individual nozzle one at a time beginning from the upstream side of the array. This was accomplished through an arrangement of pneumatically operated pinch valves operating on an inlet pressure of 67 psi. In this way the jet is able to produce a discharge velocity of 760 cm/sec, or nearly 1.65 x 10^8 dynes of static thrust. Wall jet experiments by Poreh et al. (1967) and Sforza and Herbst (1969) indicate that the bottom stress on an immobile boundary will decay with distance, r, from the jet as $r^{-2.3}$. These data suggest that each jet will exert a super-critical bottom stress of 4.6 dynes/cm^2 out to the design scour radius of 15.24m, while not decaying to the threshold stress of fluid mud, 1 dyne/cm^2, until 30m out from the quay wall. These ranges of coverages are sufficient to protect draft to beam ratios of most modern vessels.

The jet nozzles were inclined downward to allow the jet flow to be directed around the bottom of the hull of a moored ship, given the constraint of mounting the array above the dredged depth. A number of deflection angles were tested ranging from 20° to 45° of downward inclination. Figure 1-8 shows three time staggered bottom profiles along three separate range lines measured out from the concrete quay wall. Those for the control were taken downstream in Berth 8 (see Figure 1-1). Another control area was monitored on the upstream side of the area in Berth 6. Curves for Jet 3 and Jet 8 compare the effects of different degrees of downward deflection. Jet 3 used 29° of downward deflection while Jet 8 was inclined 35° downward from horizontal. Bottom profiles labeled 30 March were taken just after the completion of dredging in Berth 7. Curves labeled 17 May show the build up of mud over 1-1/2months while the jet remained inoperative pending Coastal Commission permits. The jet array became operational on 30 May. Therefore, the curves labeled 9 August indicate the scour and protection provided by the array over a 2-1/3 month period. The most nearly optimum coverage appears to have resulted from the29° downward deflection provided by Jet 3. The larger downward deflection of Jet 8 is shown in Figure 1-8 to have excavated a 1.2m deep impact crater extending from 3.05 to 12.19m out from the quay wall, but depositing a mound beyond 12.19 out to 21.33m. The lower curve of Figure 1-9 plots the net change dut to Jet 3, showing scour out to 21.33m and little new accumulation out to 30m during the 2 month operational period. Integrating over all survey contours for the entire array, and comparing with the accumulations at the upstream control in Berth 6, it was determined that the jet array prevented or scoured 1200 cubic meters of deposition in Berth 7 from 30 May to 9 August.

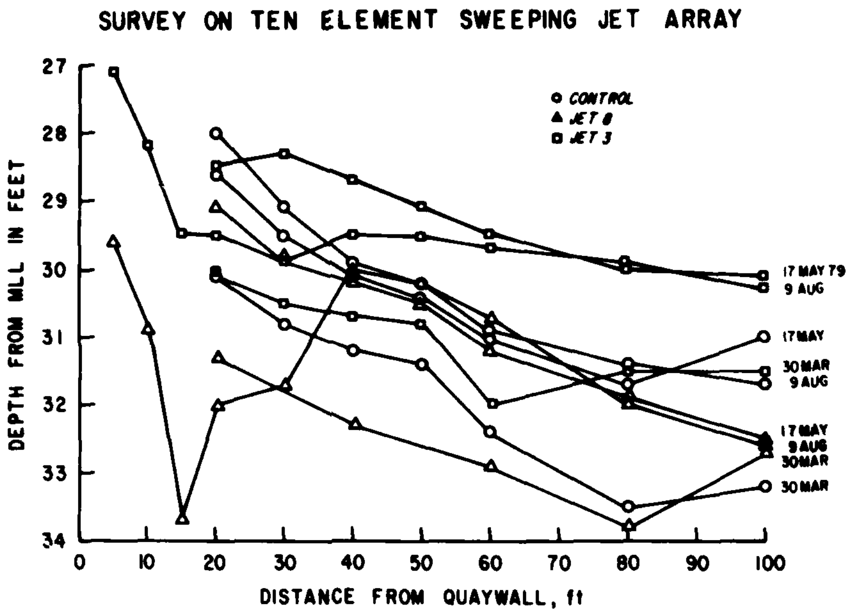

Figure 1-8. Survey on Ten Element Sweeping Jet Array

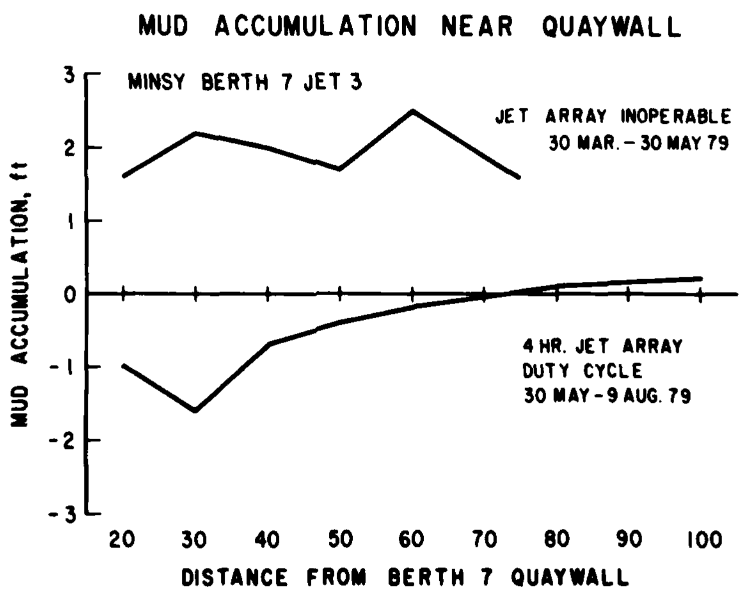

Figure 1-9. Mud Accumulation Near Quay Wall

There was also an additional downstream infuence from the jet array that can be found in the control contours from Berth 8 in Figure 1-8. Here only about 15 cm of new deposition was observed over the 2-1/3 month operational period as compared to an average of 45 cm during the same period at the upstream control in Berth 6. Factors which may have contributed to this apparent downstream influence were the presence of a 13m diameter submarine and a highly stratified and stable water column with a Richardsons number, $R_i = g(d\rho/dz)/(du/dz)^2 \cong 10^3$, through May and June. Both these factors inhibit vertical mixing and keep the turbulent jet effluent confined near the bottom where it resuspends sediment while being advected downstream with the ebb flow.

The jet array resuspension technique is an attractive alternative to bucket and scow dredging presently used near structures. The jet array was also found to effectively prevent the new accumulations of mud along channel-side quay walls during hopper dredging of the navigation channel.

CONTROL OF MUD ACCUMULATION BY EXCLUSION

Finding that the preponderance of suspended sediment is in the lower portions of the water column (Figure 1-3) led to the hypothesis that a flexible barrier, or curtain, could block these sediments from continuously circulating and settling into a berth by either tidal density-stratified currents or eddy motions. The finger pier complex notched out of the banks of Mare Island Strait (Figure 1-2) is well suited for this application. The entrance to the berth in between Pier 20 and 21 was partitioned off from the main channel by a 82.9m long Hypalon curtain that extended from the bottom up to MLL sea level (see Figure 1-10). The lateral seal on the berth was provided by an undredged mud bank extending up to MLL water under Pier 21, and by a concrete quay wall on the Pier 20 side. The 1.2-2.1m gap over the curtain and mud bank between MLL and MHH water allows Berth 20-21 to equilibrate any tidal cycle. The lens of water overtopping the curtain and mud bank would be expected to transport a negligible amount of suspended sediment, even for the nearly isotropic conditions of peak rainfall periods, as during the 9 March extreme shown in Figure 1-3.

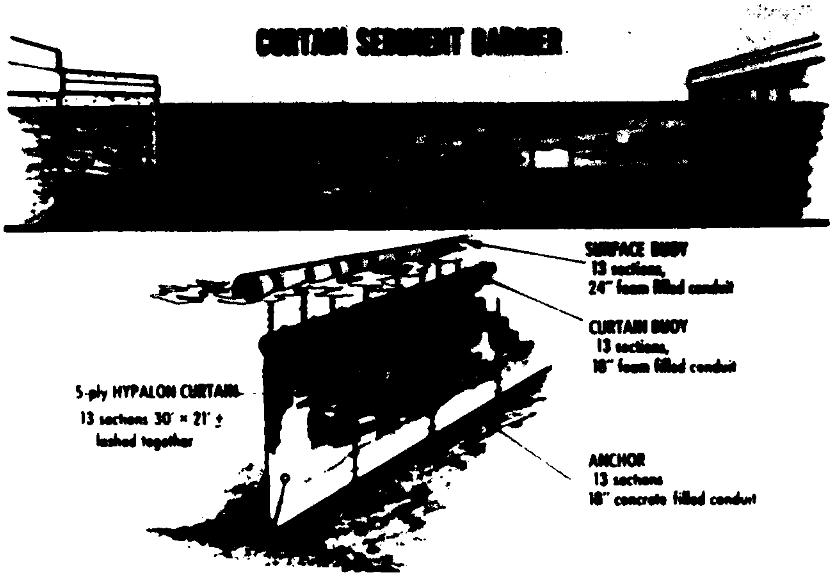

Figure 1-10. Hypalon Curtain Sediment Barrier

The curtain was constructed in 13 sections, each 6.4m in length. The Hypalon curtain material on each section was anchored to the bottom by an 18 inch storm drain conduit filled with concrete aggregate weighing 8000 lbs. The 9.14m high curtain sections were supported vertically in the water by curtain buoys constructed from 18 inch storm drain conduit filled with polyurethane foam. A pair of air-filled 10-inch diameter PVC pipes were retrofitted to the curtain buoys, seen in Figure 1-11, to trim the buoyancy against additional water absorption by the foam and concrete aggregate.

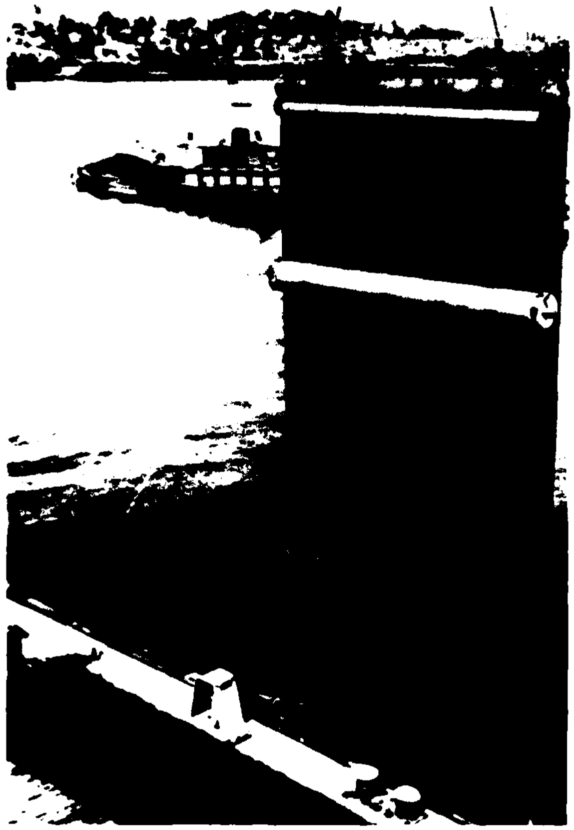

Figure 1-11. Two Air-filled 10 inch diameter PVC pipes retrofitted to curtain buoys

To raise the curtain as shown schematically in Figure 1-12, a second set of buoys at the surface is connected by a length of chain sufficient to extend down to the submerged buoy floats at a higher high water during spring tides. At the low tide preceding curtain opening, the surface floats and buoy floats are at nearly the same level, and the slack chain is drawn up and secured by a stopper arrangement. With the ensuing flooding tide, the additional buoyancy of the 24-inch diameter foam-filled surface buoys is sufficient to gradually overcome the bearing stress of the fluid mud, about 20 dyne/cm^2, seen in progress in Figure 1-13. At high tide the curtain floats free of the bottom illustrated in Figure 1-11, whence it can be swung open into the berth by towing behind a small tug boat as shown in Figure 1-14. The end about which the curtain pivots was anchored to Pier 21 by cutting a notch in the slope of the mud bank and allowing the first curtain sections to become buried. The curtain rotates about tongue and pin joints between the second and third sections. Consequently, it was not necessary to raise the first two sections allowing surface floats adjacent to Pier 21 to be removed. This gave access to the berth by shallow draft barges while the curtain still remains closed and anchored (Figure 1-15).

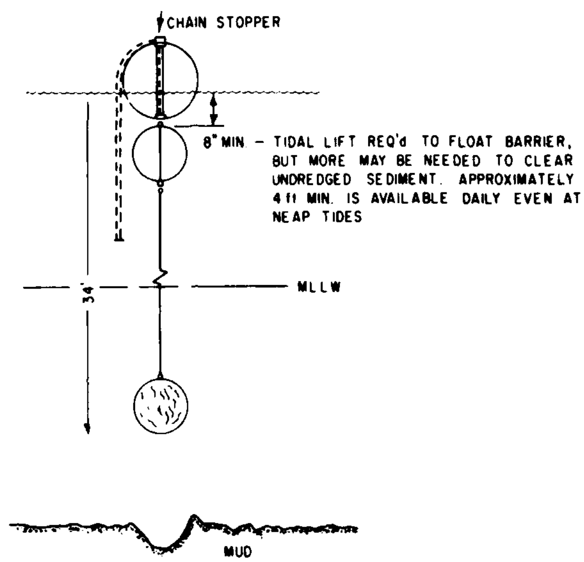

Figure 1-12. Curtain Cross Section in Raised Configuration.

Figure 1-13. Surface Buoys While Raising Anchored Sections From the Mud During a Rising Tide

Figure 1-14. The Curtain Being Opened by the Sea Mule Yard Tug

Figure 1-15. The Curtain in the Closed and Anchored Configuration As Seen Looking into the Pier 20-21 Berth

The shoaling with time, averaged across the Pier 20-21 berth, appears in Figure 1-16, comparing mean shoaling on two survey ranges inside the curtain with another two outside. The range lines were taken across the berth perpendicular to Pier 21 at positions measured from the shoreward end of the 228.6m length Pier. The shoaling along these range lines is shown in Figure 1-17 to have been fairly uniform. Four months of operational testing with six curtain opening cycles showed that only 0.46-0.76m of new deposition occurred inside the curtain while 1.52-2.59m accumulated in the unprotected waters outside the curtain. This represents a 70 percent effectiveness during high depositional conditions with several protracted periods in the open configuration necessitated by ship movements. An apparent dredging savings of between 18,000 to 30,000 cubic meters was achieved. This 4 month savings payed for the material costs of the curtain considering dredging costs at the present local value of $\$2.50/yds^3$. Comparing the dates in Figures 1-16 and 1-17 with the mud storm events in Figure 1-4, it is concluded that most of the deposition behind the curtain occurred during February when the curtain was open for only one 3-day period. Only about 6 in. of deposition inside the curtain occurred in the months of December, January and March when there were 5 opening cycles adding up to 36 open days. Therefore the timing between mud storm events and curtain opening seems critical in achieving maximum protection of a berth by exclusion techniques. It was fortunate in this experiment that the curtain happened to be closed during the mud storms in December and early March when large mud accumulations are shown in Figure 1-16 to have occurred outside the curtain.

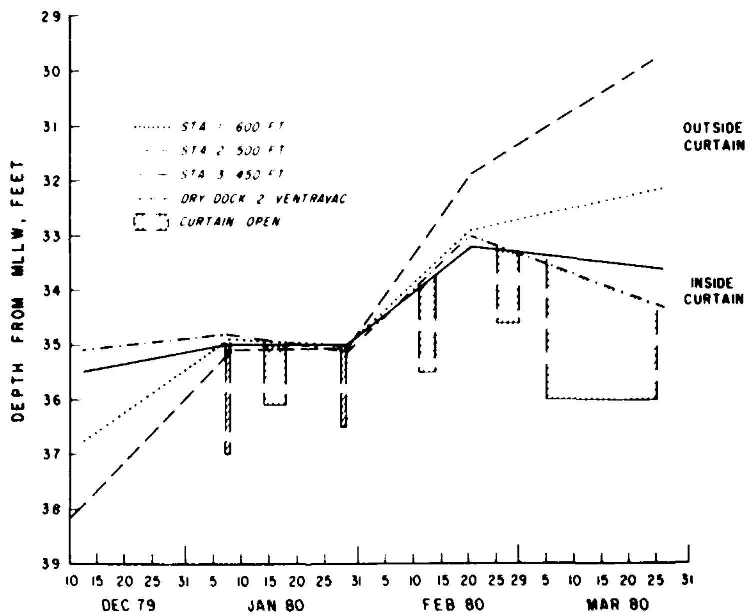

Figure 1-16. Shoaling History Averaged Across the Berth

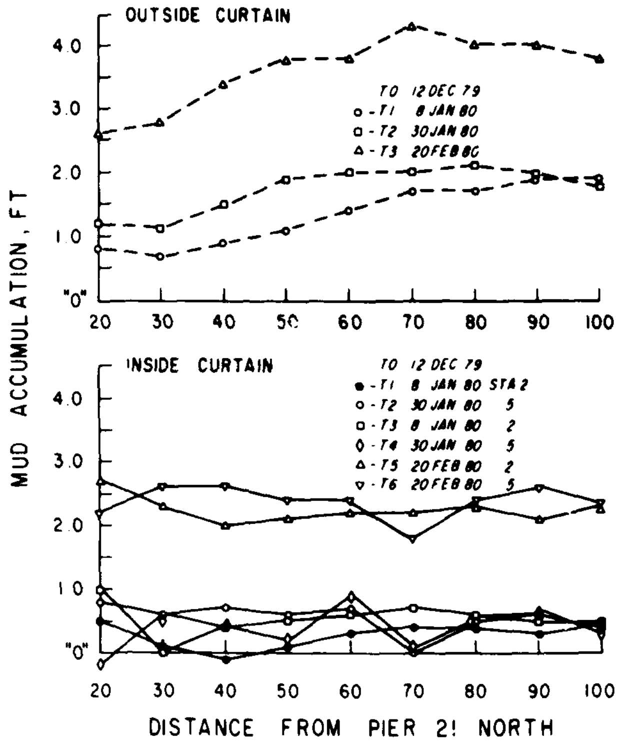

Figure 1-17. Mud Accumulation Across the Berth Beginning from 12 December 1979

CONTROL OF SAND ACCUMULATION BY CRATER-SINK/FLUIDIZATION

The crater-sink sand bypassing concept, which uses a crater-shaped depression in the channel bed to capture sand, was first proposed by Inman and Harris (1970). Several systems based on this concept have been tested in both the laboratory and the field (Harris, et al., 1976; McNair, 1976), and full scale systems are currently operating at Mexico Beach, Florida (Pekor, 1977), and at Rudee Inlet, Virginia.

One of the limitations of the crater-sink concept has been the relatively small trapping radius as compared with depositional patterns. Harris, et al, (1976) found that the trapping radius of a crater could be enlarged by feeding the crater with a fluidized trench cut across the depositional area. These trenches are themselves sinks to the sediment flux. The trench is both cut and maintained by a fluidizer pipe, having a line of downward axially slanted water jets along its length. The jets fluidize the neighboring sand and impart momentum to the resulting slurry in the direction of the jet effluent. A momentum survey of the process in closed duct configurations is discussed in Bailard and Inman (1975).

The Agua Hedionda Lagoon selected for these experiments is located 30 miles north of San Diego, California. The lagoon consists of an outer, middle, and inner section with a total area of 1.04×10^6 m^2. The tidal prism is approximately 1.68×10^6 m^3, passing through a stabilized inlet channel with a cross sectional area of 33.4 m^2. One third of the tidal prism is removed from the lagoon each tide cycle and diverted through the cooling condensers of a power plant and discharged into the sea. The corresponding reduction in ebbing currents through the inlet which intercepts the longshore transport of sand results in an average influx of beach sands of about 11,000 m^3/month. This influx is deposited entirely in the outer lagoon where the utility company operates a suction dredge on a yearly or bi-yearly basis.

A crater-sink/fluidization sand bypassing system was designed to intercept the inflow of sand and return it to the downdrift (south) beach face (Figure 1-18). The system was sited at two different locations between 1978 and 1979 near the outer bank of the curved inlet where the centrifugal accelerations on a flooding tide direct the influx, depositing a sand bar. The latest of several system designs for Agua Hedionda Lagoon consisted of a 1-4m deep crater excavated by a 6" x 6", 150 hp centrifugal dredge pump connected to a 270m long, 6-inch diameter discharge pipeline. A 50m long fluidizer trench cut across the primary depositional bar, and was powered by a 6-inch 100-hp water pump discharging into the crater on the interior downslope side of the bar. The fluidizer pipe was 4 inches in diameter with 0.282-cm diameter jets drilled at 45° angles and spaced 6.25 cm apart. The flow rate from the fluidizer drive water pump was 784 gpm at 100 ft total head pressure. One third of this flow rate was diverted to a 2-inch diameter liquefier jet in the bottom of the crater, dropping the pressure to the fluidizer pipe down to 22 psi. The liquefier jet fluidized the sand at the crater

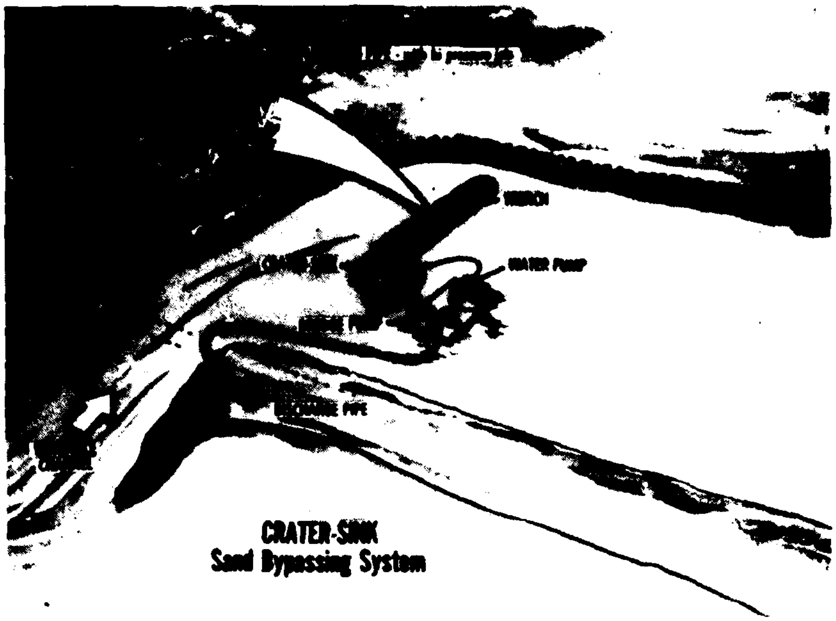

Figure 1-18. Crater-Sink/Fluidization

suction inlet to minimize suction losses on the dredge pump. The flow rate through the dredge pump and sand discharge pipeline varied between 890 and 1000 gpm at 180 ft of total head, depending upon the amount of vertical lift required to remove the sand. To minimize the required suction lift, which varied between 2-6m depending upon crater depth, tidal phase, and sand bar level, the pumping systems were operated from a moored barge.

To evaluate the ability of the system to capture and bypass sand, the flow rate and sand concentration in the discharge line was monitored through the period of maximum flooding tide. Figure 1-19 shows the bypassing history during the 1978 experiment using a 60 cm deep fluidizing trench. The mean capture rate of 2.75 l/sec accounts for about 1/2 of the average sand influx rate, 4-6 l/sec. Flood currents during this period peaked at 130 cm/sec on the surface above the fluidizer trench. If the sediment transport rate is taken to vary as the cube of the velocity according to Bagnold (1966), then 1/2 the sand influx would be suspended load at these current speeds. Hence, the capture efficiency of the system is in proportion to the bed load. Suspended load is transported over a shallow fluidizer trench. Furthermore, underwater observations discovered that the initially sharp lip of the trench became rounded (Figure 1-20) after about 55 minutes of operation under currents in excess of 1 m/sec. With this round-off the flow no longer separated at the top of the trench, allowing the flood current to sweep into the trench and carry away sand, as indicated in Figure 1-19 by the decline in sand capture with the onset of trench breakdown.

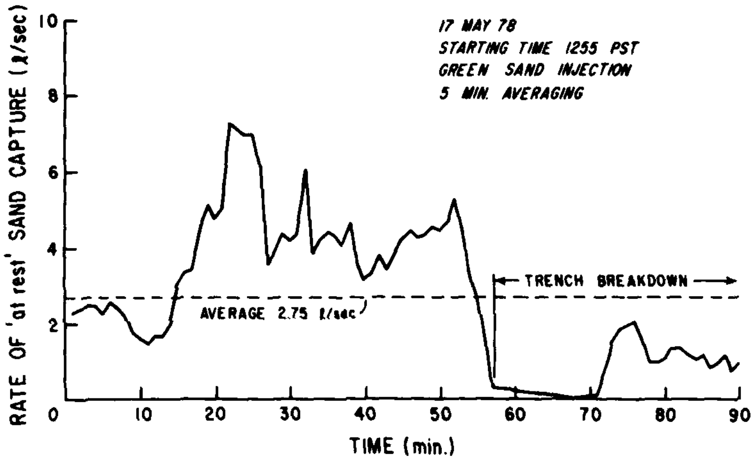

Figure 1-19. 1978 Experiment Using a 60-cm Deep Fluidizing Trench

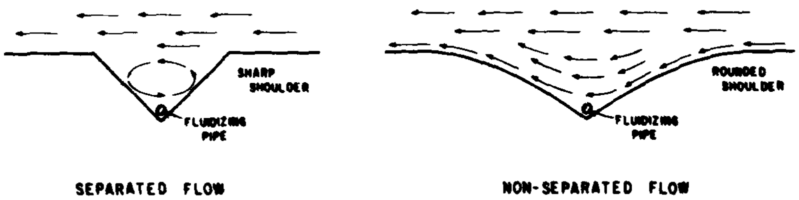

Figure 1-20. Fluidizing Trench Before and After Trench Breakdown

To avoid trench breakdown the 1979 experiment was moved further into the lagoon where flooding current amplitudes and suspended load were less. The fluidizer trench was deepened to 3m to create a flow divergence over the trench that would drop the suspended load. Figure 1-21 shows bypassing time

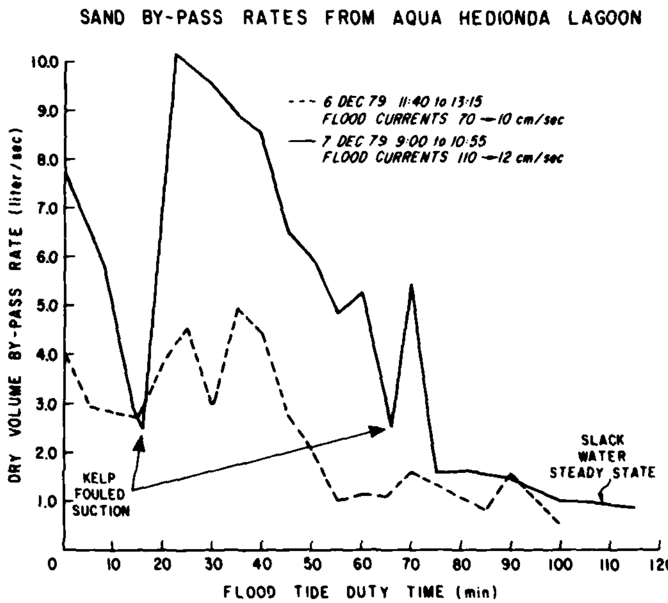

Figure 1-21. Bypassing Time Histories Without Trench Breakdown

histories in which the average total sand influx (5-6 l/sec) was exceeded for extended durations without trench breakdown at current speeds as high as 110 cm/sec. However, these experiments were plagued by extra-normal amounts of kelp and bottom debris broken loose offshore by unseasonably high waves and subsequently swept into the lagoon on flooding tide. When captured by the system and mixed into the fluidized sand, the kelp would conglomerate in masses as large as 10m in diameter, which the system had insufficient power to move. A porous screen lid over the trench and crater successfully shielded the system for a two week period but required frequent diver maintenance. Another approach tested was to simply allow a kelp fouled system to become buried. Under the added pressure of the sand overburden kelp decays anerobically to a less fibrous black mulch which the system can move again in several days to a week. With a single fluidizer trench this procedure restricts the number of bypassing cycles. To circumvent that in future bypassing schemes, multiple fluidizing trenches extending out from a single crater in a fan arrangement could be alternately cycled and allowed to bury once fouled.

OPENING A TIDAL INLET BY OPEN TRENCH FLUIDIZATION

Penasquitos Lagoon is a relatively small lagoon encompassing about 1.29×10^6 m^2 and is located approximately 10 miles north of San Diego. Past studies have shown that the normally closed lagoon inlet channel is periodically opened during times of high precipitation in which the lagoon is filled to overflowing. The lagoon then remains open until it is closed by the longshore transport of sand. Closures is enhanced when high waves coincide with neap tides. This condition results in a large influx of sand into the lagoon and causes a sand plug to form in the seaward end of the inlet channel. An overwash fan then forms behind the sand plug, filling the channel with sand.

A crater-sink/fluidization system was designed to cut a new channel across the sand plug. The system consisted of a 6-inch 100-hp dredge pump connected to a short discharge pipeline, and a 43m long spiral wound fiberglass fluidizer pipe powered by a 4-inch 30-hp water pump (Figure 1-22). The design of the pipe was based on a modified form of the analytical model developed by Bailard and Inman, (1975). The fluidizer pipe was 4 inches in diameter with 0.145 cm diameter jets angled at 45° and spaced 6.3 cm apart. The water flow rate to the pipe was 520 gpm at a pressure of 3.85×10^6 dynes/cm^2 (55 psi).

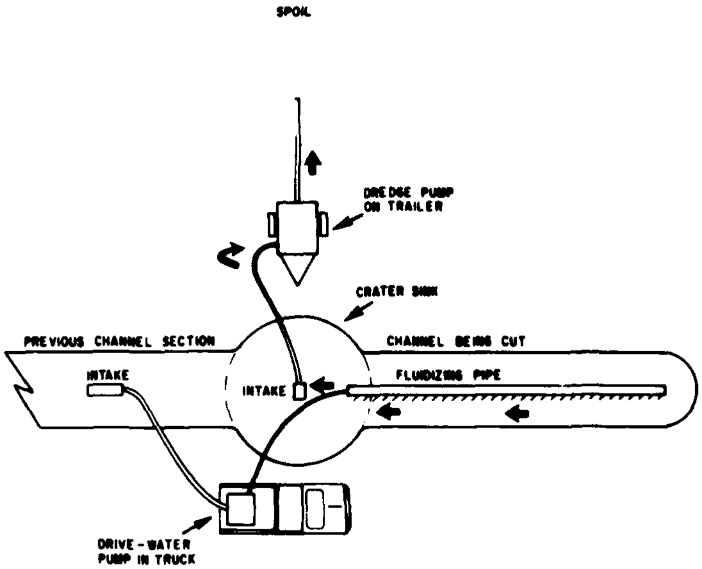

Figure 1-22. Crater Sink/Fluidization Systems.

The procedure for cutting the channel consisted of starting at the lagoon and "leap-frogging" the system across the overwash fan. The system successfully cut a 210m long channel in 5 steps (Figure 1-23) removing 600 m^3 of sand. In fine sand the fluidizer moves sand at a rate of 100 m^3 per hour; however due to the presence of extensive cobble beds, the maximum cutting rate of the system was 30 cubic meters per hour, cutting a 1.5m deep, 3.7m wide, 43m long channel segment in approximately 3.5 hours.

Figure 1-23. Channel Cut by Crater-Sink
Fluidization System

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The authors are grateful for the efforts of Dr. William Van Dorn, who was our colleague in the fine sediment studies prior to his retirement in August 1979. We have appreciated assistance and logistical support from Mr. James Dillard, technical assistant to the public works officer at Mare Island Naval Shipyard. We are further indebted to Marion Horna, plant manager, San Diego Gas and Electric Co. for the support of cable, barges, and electric power in the Agua Hedionda bypassing experiments.

SECTION 2

U. S. NAVY HARBOR MAINTENANCE DREDGING ATLAS (CONUS)*

by

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INTRODUCTION

Scripps Institution of Oceanography, sponsored by the Navy Facilities Engineering Command and the Civil Engineering Laboratory, has been working to develop sedimentation prevention systems for Navy harbors. Dredging costs have risen dramatically along with the national concern for environmental quality. Ninety percent of the material dredged by and for the Navy is contaminated with heavy metals and other potential pollutants.

The heavy metals occur naturally in rivers through the erosion of the country rock that makes up the watershed. But when these rivers begin to mix with seawater, flocculation occurs. Many Navy harbors are located along waterways where this mixing takes place, and not only is suspended sediment deposited but so are the heavy metals because of their affinity to the clayey flocs. Relocating these contaminated deposits pollutes the disposal site and imposes additional costs to measure the amount of pollution and to minimize its effect.

In the course of the research and development sponsored by the Navy Facilities Engineering Command, three approaches that show promise as cost beneficial alternatives to dredging have been developed.

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* Dredging volumes and costs contained herein are those that were presented at the symposium. These numbers are currently being revised and updated and will be published in a Civil Engineering Laboratory technical memorandum in the near future.

Areas where dredging is conducted at Navy expense are presented in this Atlas. Each Navy harbor that requires maintenance dredging in excess of 100,000 cubic yards per year is represented by a map showing locations of required dredging. The amount and type of sediment is tabulated.

NAVY HARBORS WITH HIGH SILTATION

There are 12 Navy harbors in the Continental United States that have an annual maintenance dredging burden in excess of 100,000 cubic yards per year. These are listed in Table 2-1, with the amount and type of sediment removed and the appropriate figure number. The Figures 2-1 through 2-17 show harbor configuration and the location of actual dredging.

A review of the figures shows that Navy harbors are typically located along inland waterways with the berths formed by piers extending from shore. Over two-thirds of the area of Navy harbors that require dredging with Navy funds are contained within these quiet water, cul-de-sac berths. These berths, unfortunately, are sediment settling basins where deposits build up and currents are not generated with sufficient strength to resuspend the sediment.

SEDIMENT TYPES

The bulk of sediment dredged by and for the Navy is estuarine. Typically, Navy berths are located within estuaries (tidal rivers). Sediment carried by these rivers is held in suspension while the water is fresh but, when mixed with seawater, flocculation occurs resulting in deposition with rates as high as a centimeter a day. These sediments are carried into Navy quiet water berths by eddy and tidal currents and are not subsequently resuspended.

These sediments, upon flocculation, concentrate heavy metals. Further contamination may result from sand blasting, paint removers, oil spills, and other shipyard and harbor activities.

TABLE 2-1. TABULATION OF U.S. NAVY HARBORS IN CONTINENTAL U.S. WITH LARGE DREDING BURDENS

Harbor	Annual Maint Dredging M yd^3	Reference No.	Sediment Type	Reference No.	Figure No.
Mare Island Naval Shipyard	0.4	1,3	Mud	2	2-1,2-2
Alameda Naval Air Station	0.9	1,4,3	Mud	2	2-1,2-3
Molate Point Naval Fuel Depot	0.12	3,5	Mud	2	2-1,2-4
Port Hueneme Harbor	0.19	6	Sand	6	2-5
New London Naval Submarine Base	0.1	7,8	Mud	7	2-6
Naval Weapons Station Earle	0.2	7	Mud	7	2-7,2-8,2-9
Philadelphia Naval Shipyard	0.2	4,9	Mud	4,9	2-10
Norfolk Naval Station	0.38	4	Mud	10	2-11
Charleston Naval Base & Weapons Station	3.0	4,11	Mud	4	2-12,2-13,2-14
King's Bay	0.8	12	1/2 Mud 1/2 Sand	12	2-15
Mayport Naval Station Basin	0.6	13	Mud	13	2-16
Port Canaveral	0.15	14	Sand	14	2-17
Miscellaneous	2.0	4	-		

80-2220

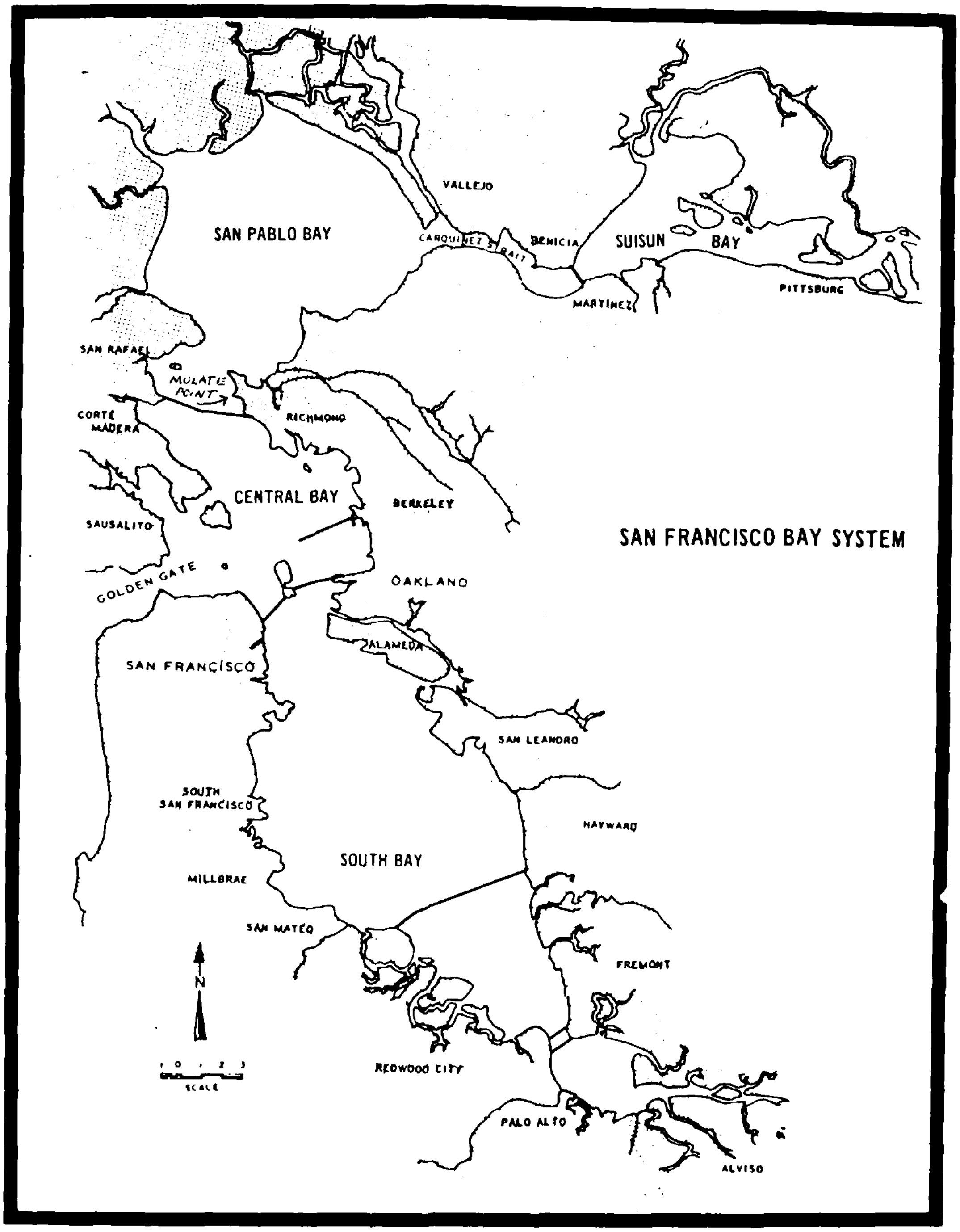


Figure 2-1. Mare Island Naval Shipyard

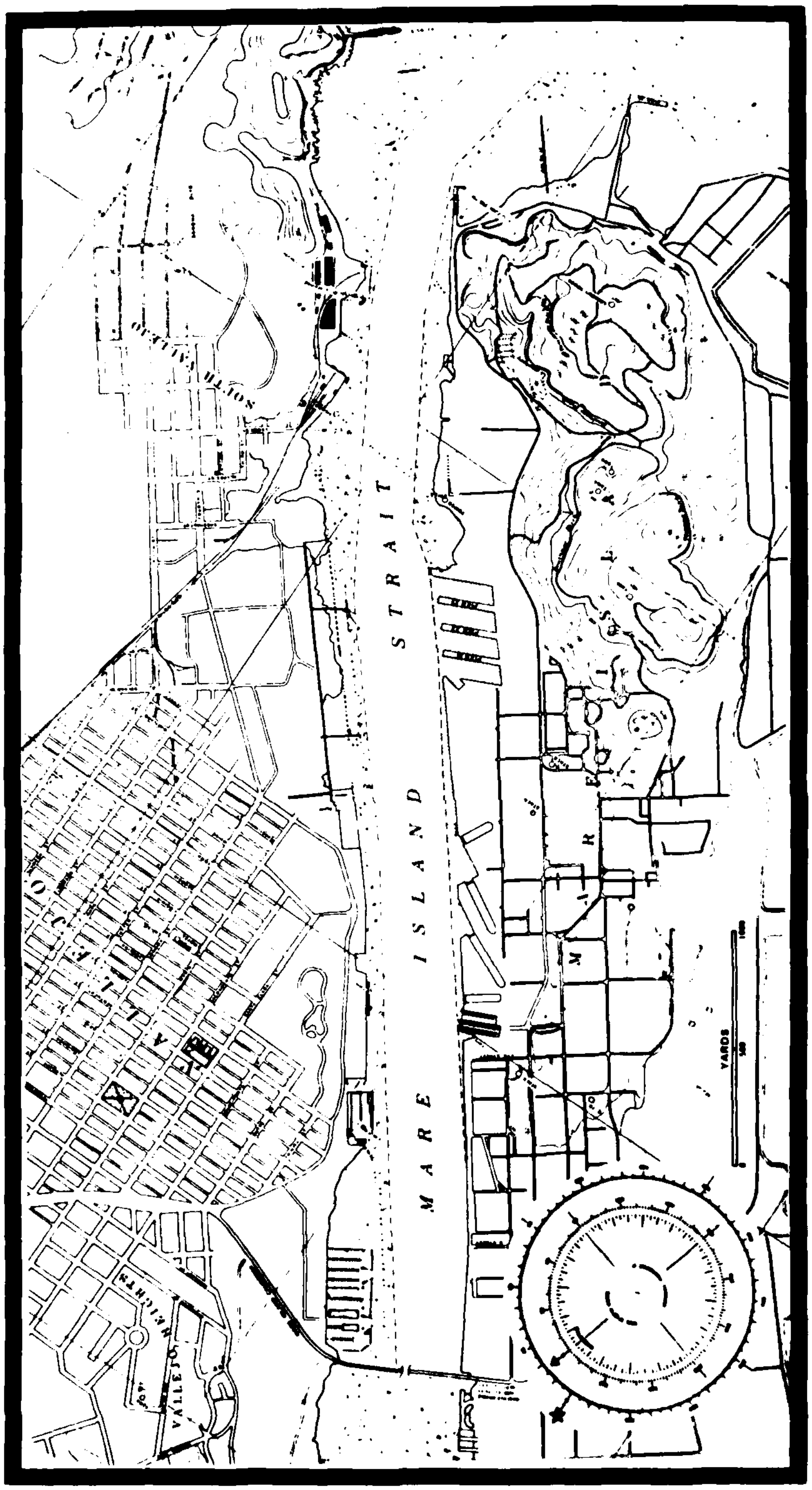


Figure 2-2. Mare Island Naval Shipyard - Strait Area

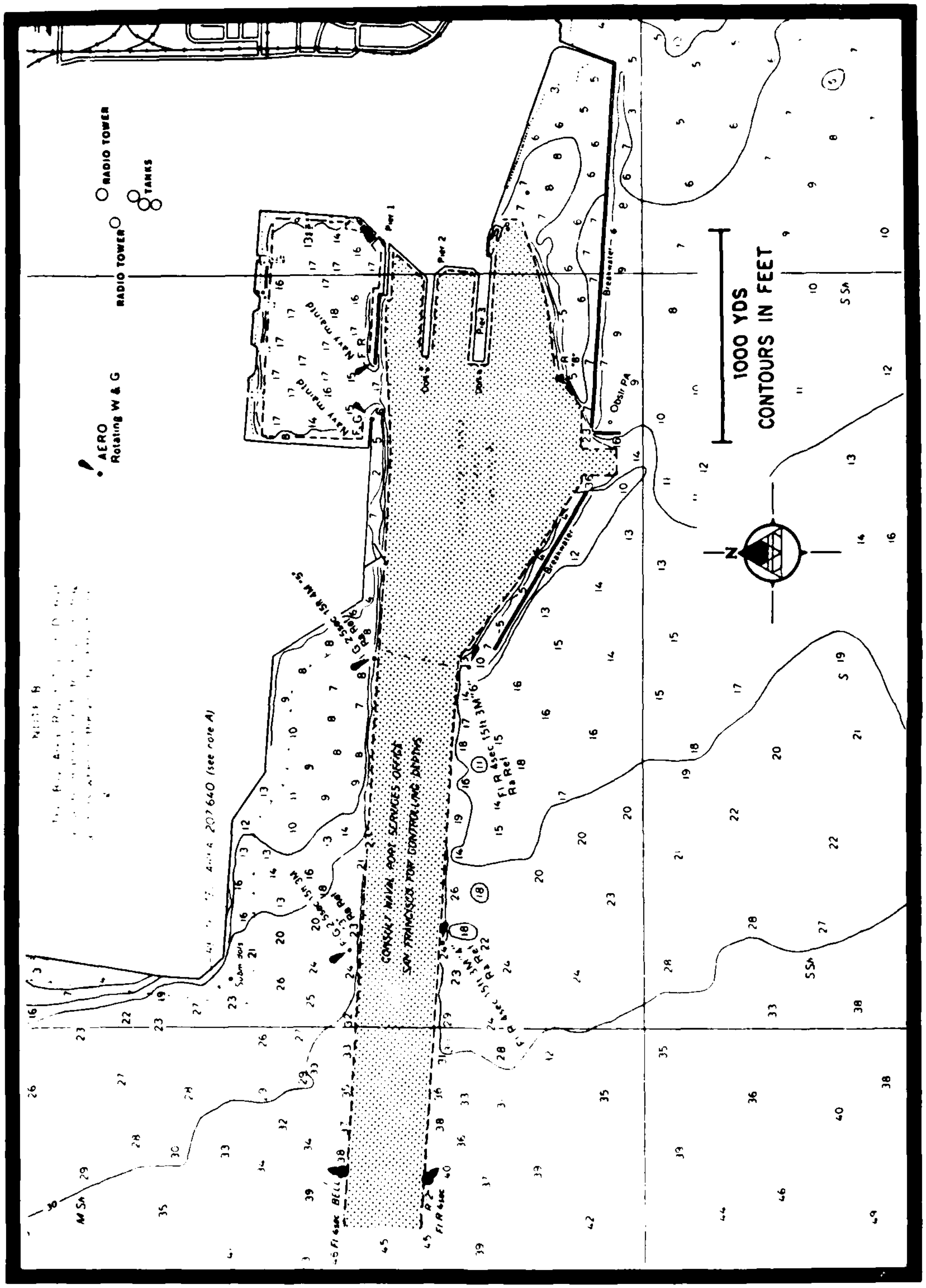


Figure 2-3. Alameda Naval Air Station

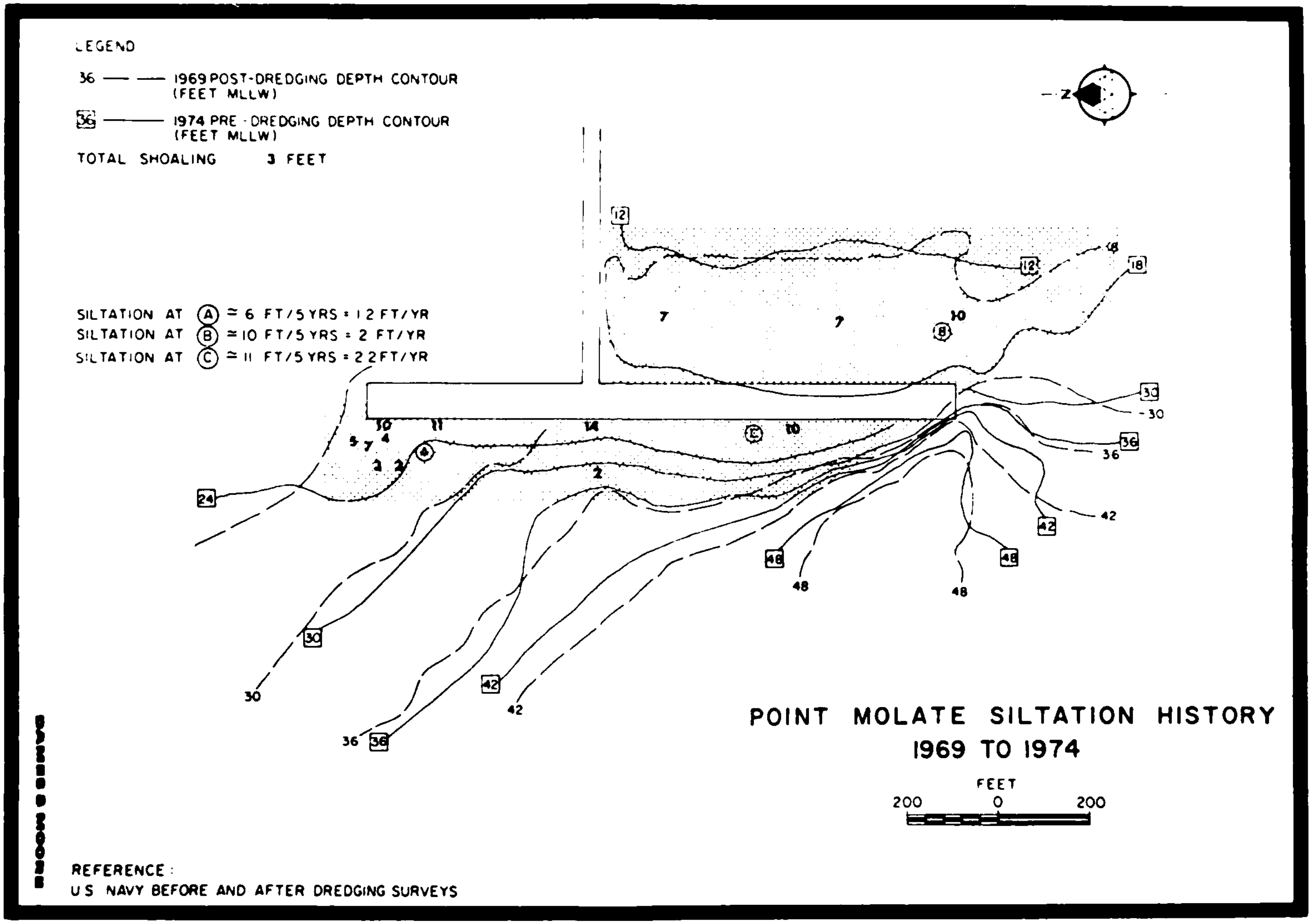


Figure 2-4. Point Molate Siltation History 1969 to 1974

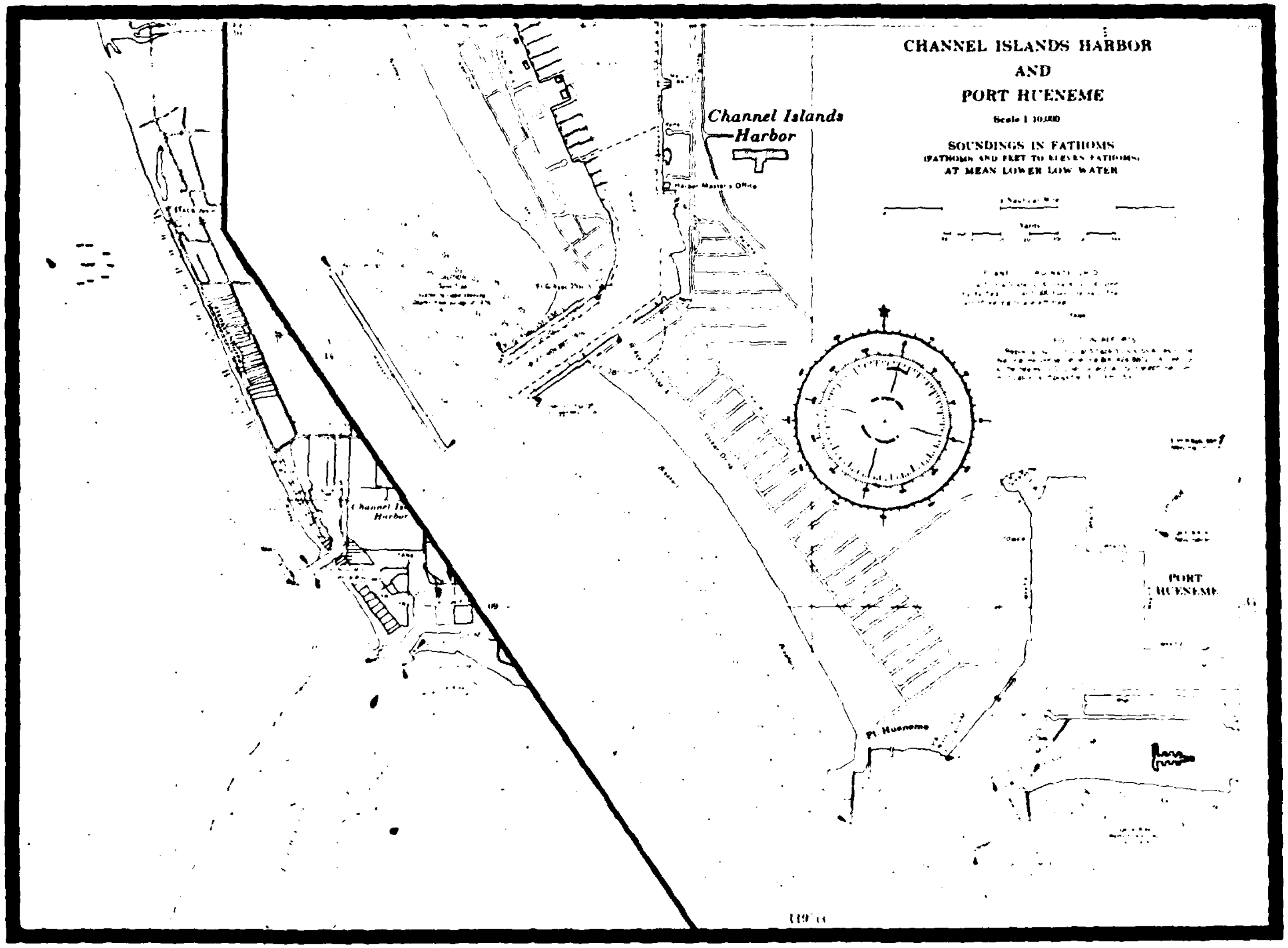


Figure 2-5. Port Hueneme Harbor

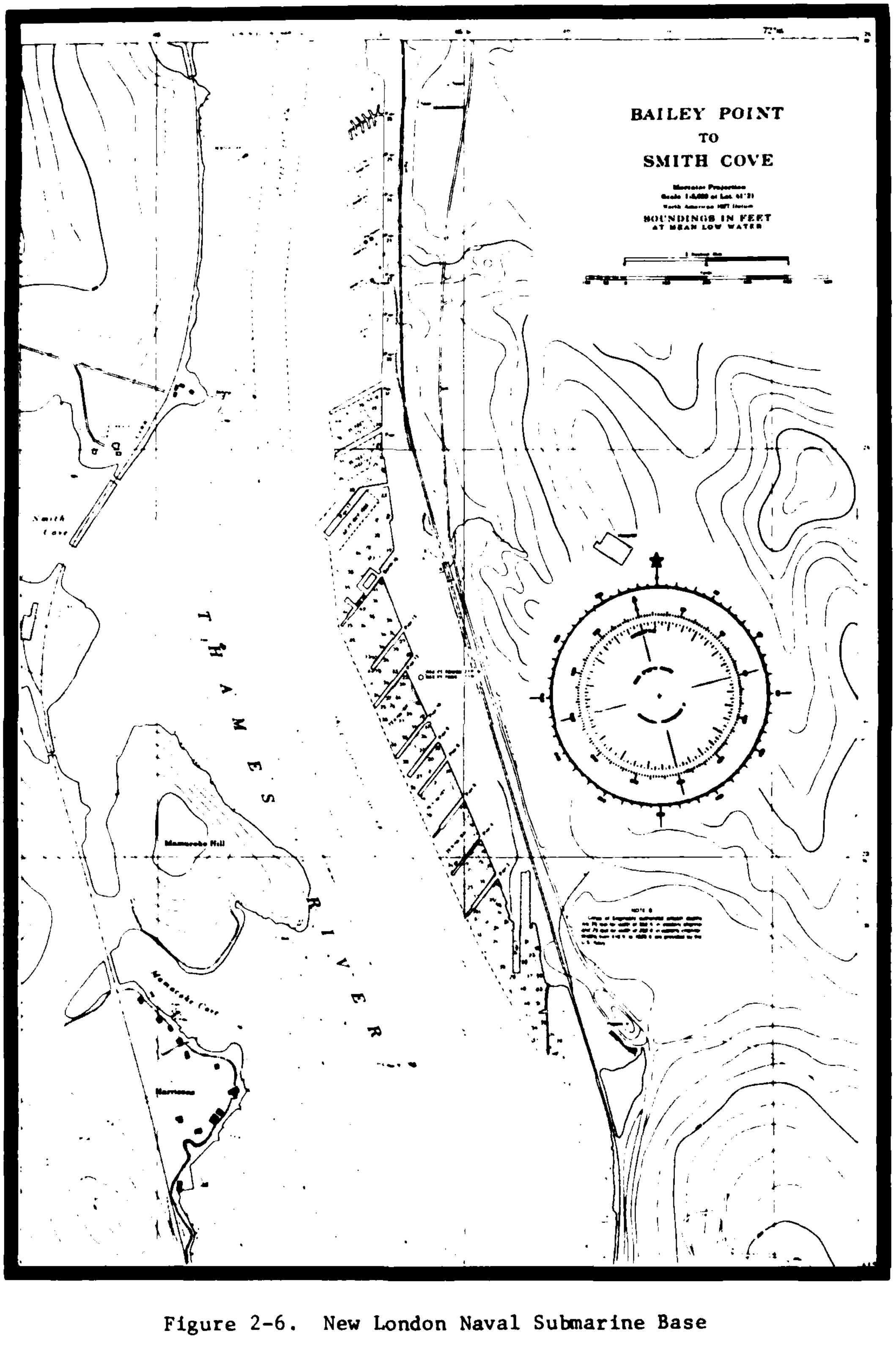


Figure 2-6. New London Naval Submarine Base

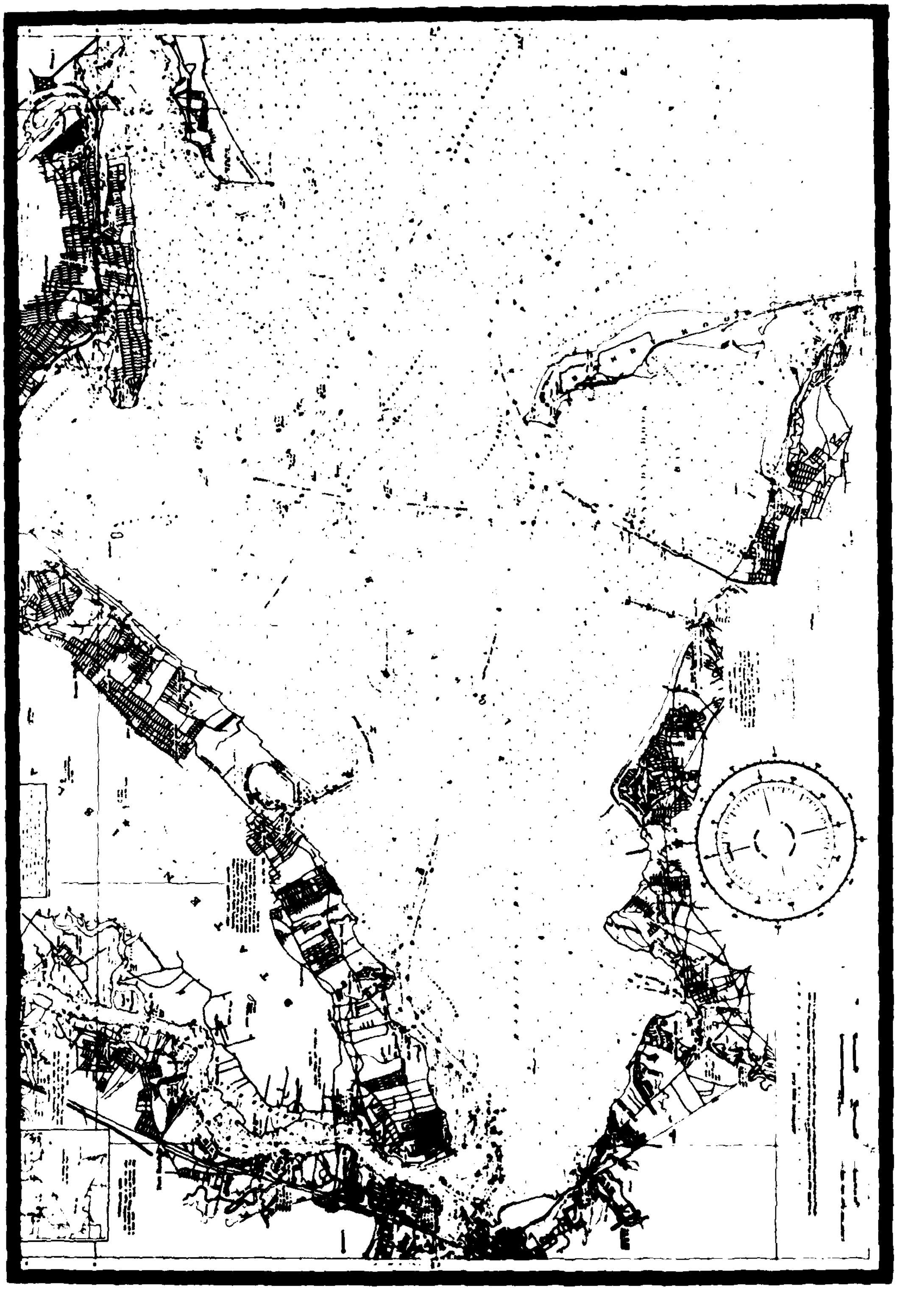


Figure 2-7. Naval Weapons Station, Sandy Hook, New Jersey

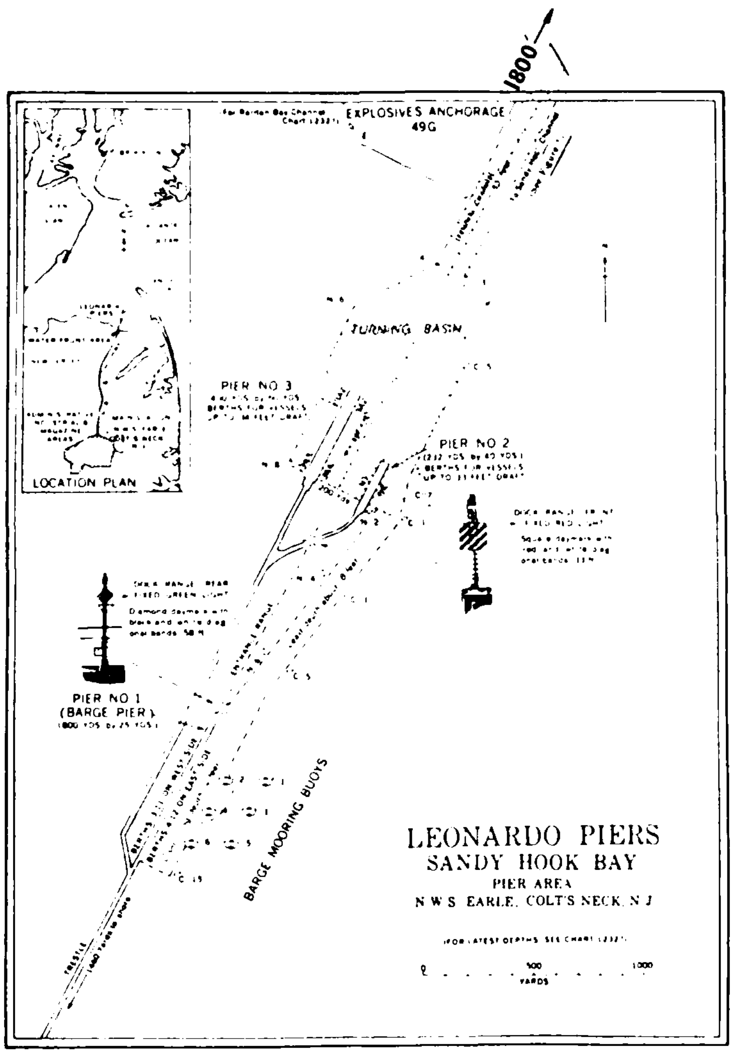


Figure 2-8. Leonardo Piers, Sandy Hook, New Jersey

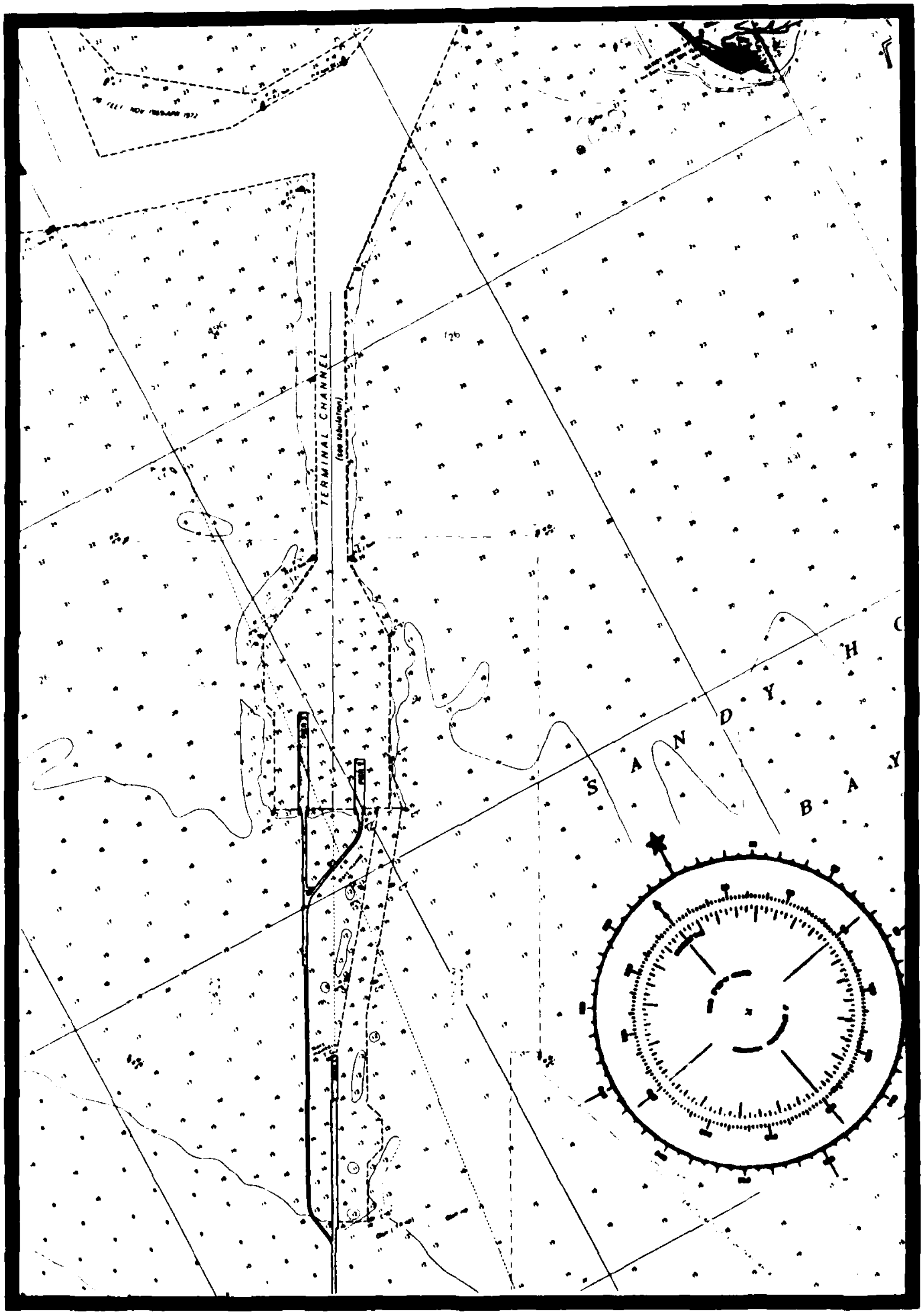


Figure 2-9. Details of Sandy Hook Channel

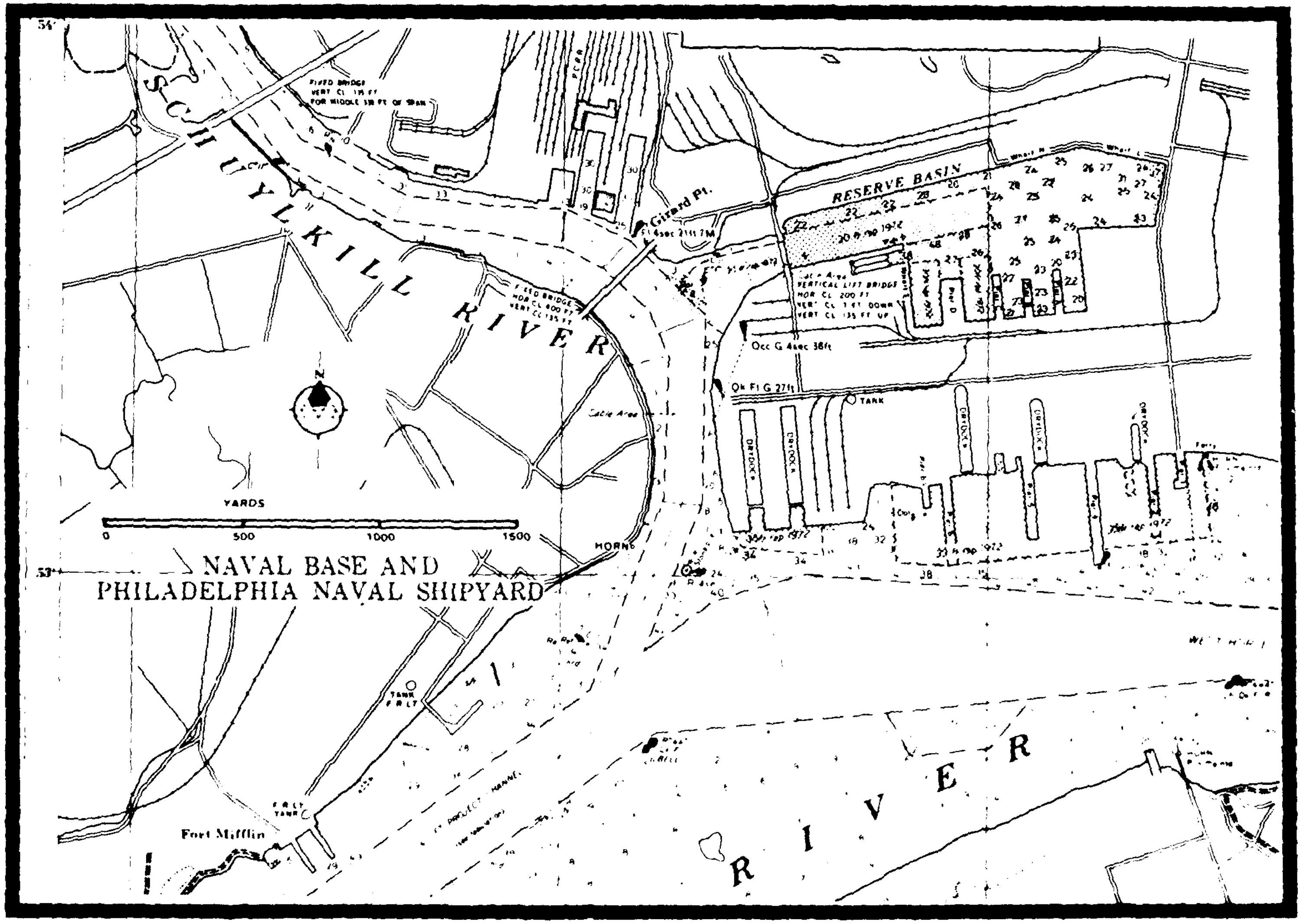


Figure 2-10. Philadelphia Naval Yard

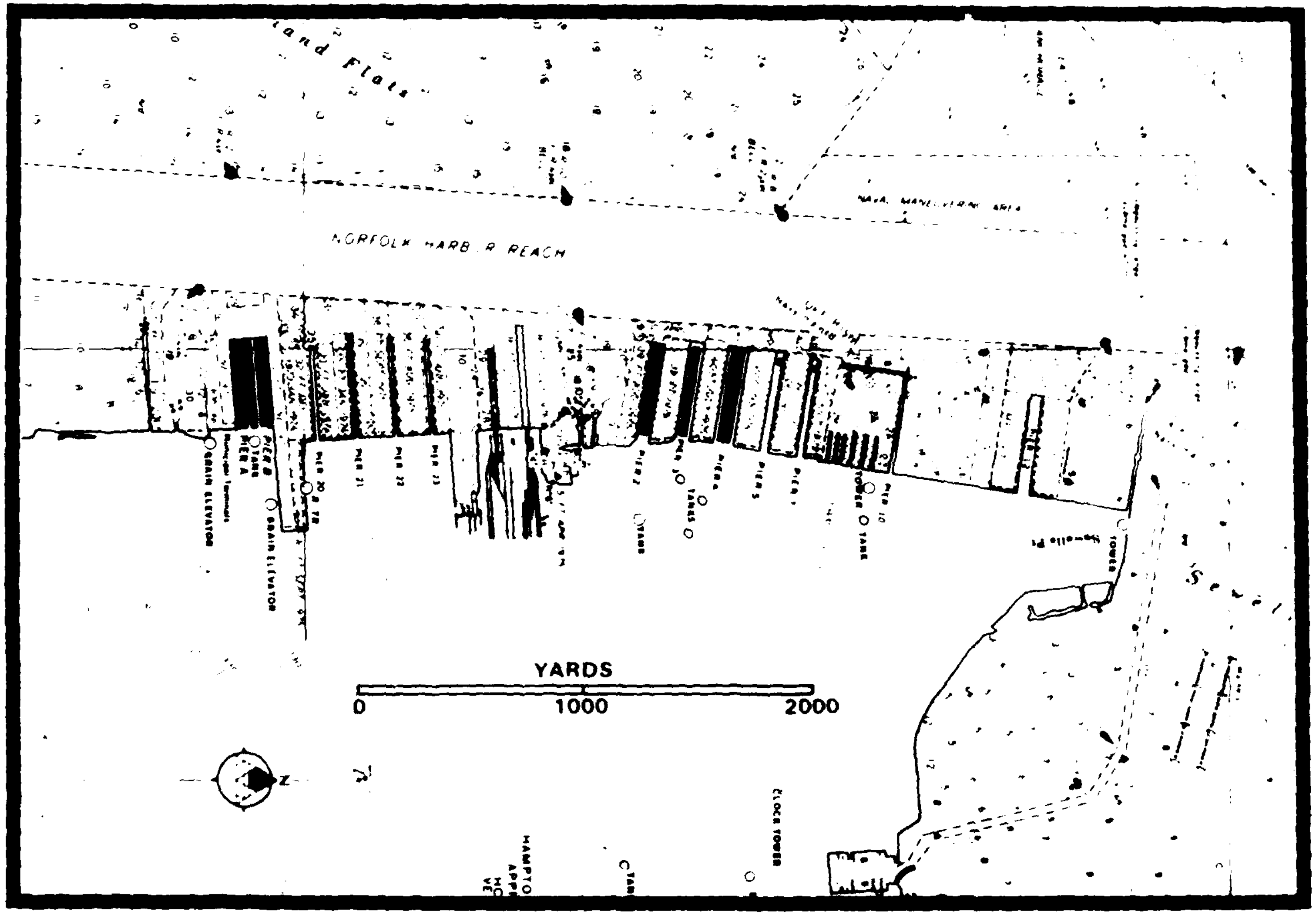


Figure 2-11. Norfolk Naval Station

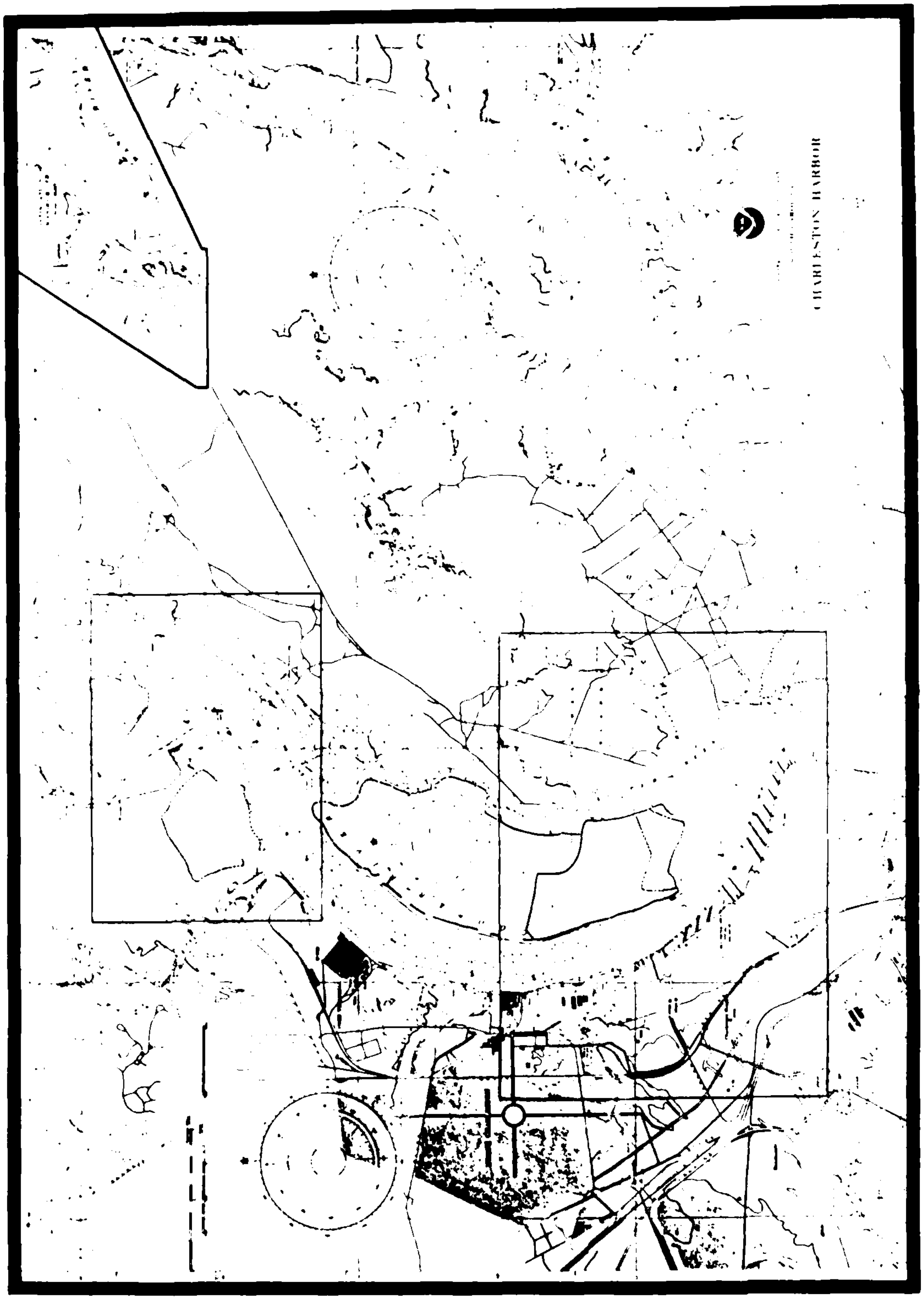


Figure 2-12. Charleston Harbor, Full View

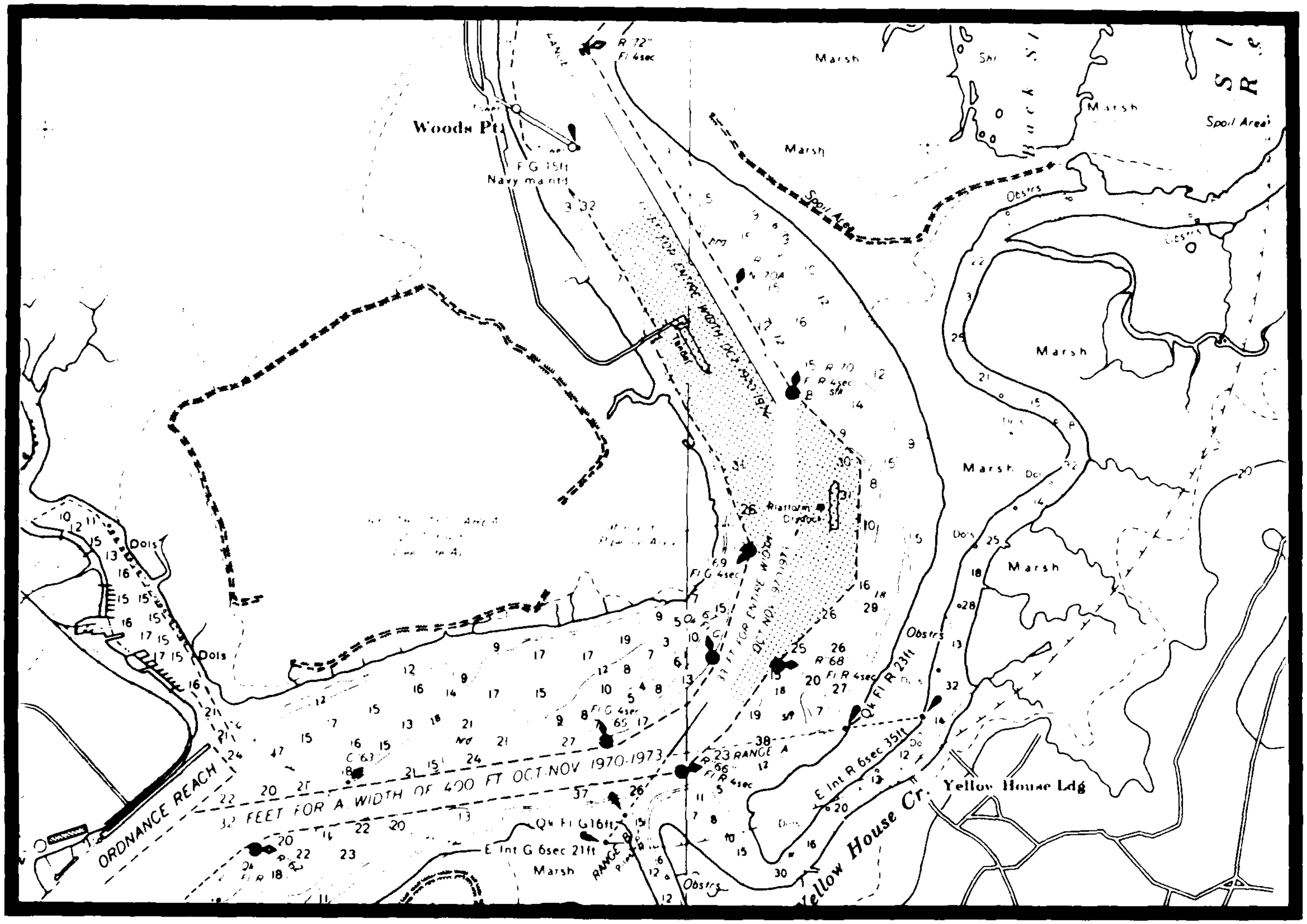


Figure 2-13. Details of Charleston Naval Base and Weapons Station, Northern Portion

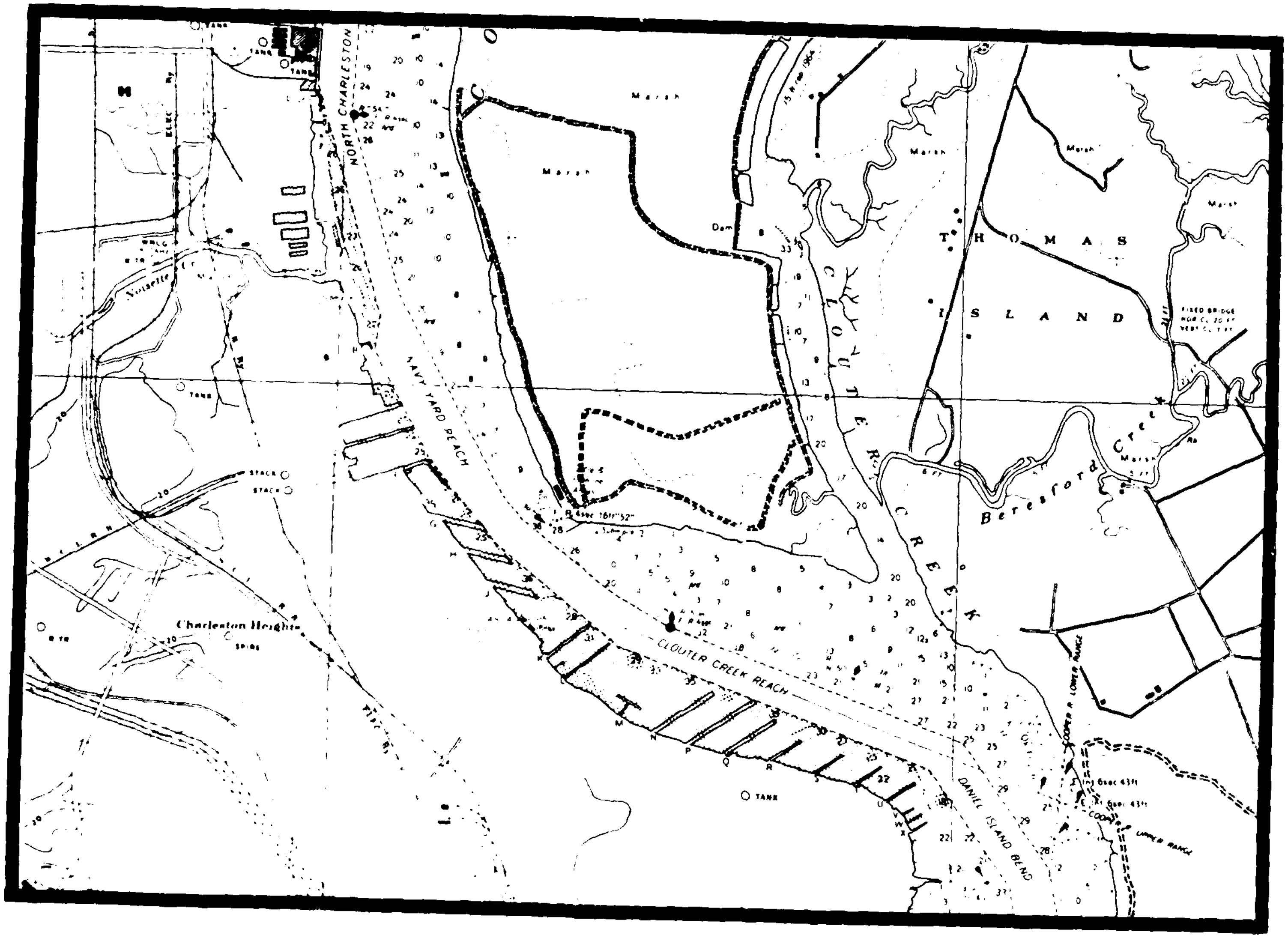


Figure 2-14. Details of Charleston Naval Base and Weapons Station, Central Portion

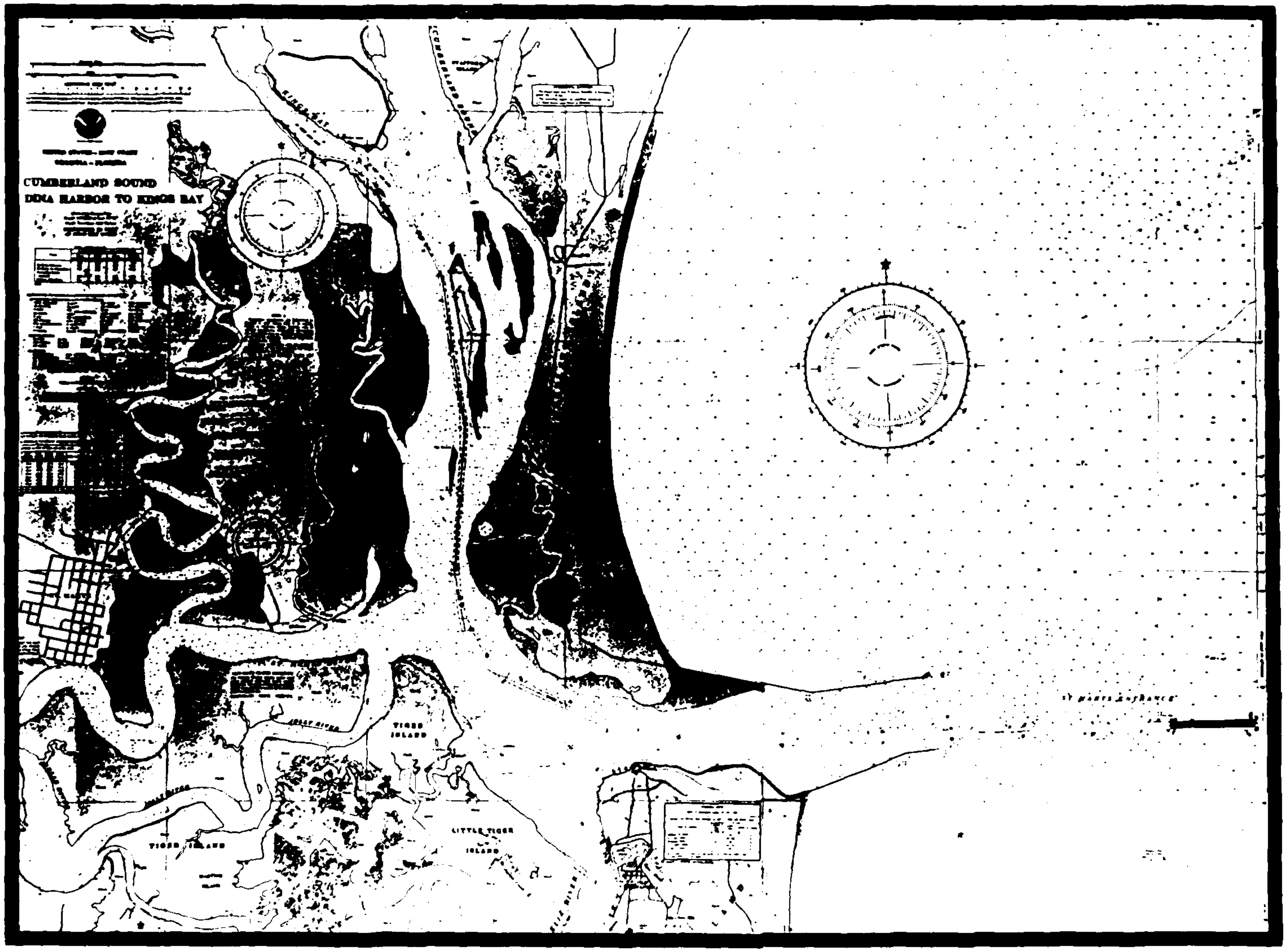


Figure 2-15. Cumberland Sound - Dina Harbor to Kings Bay

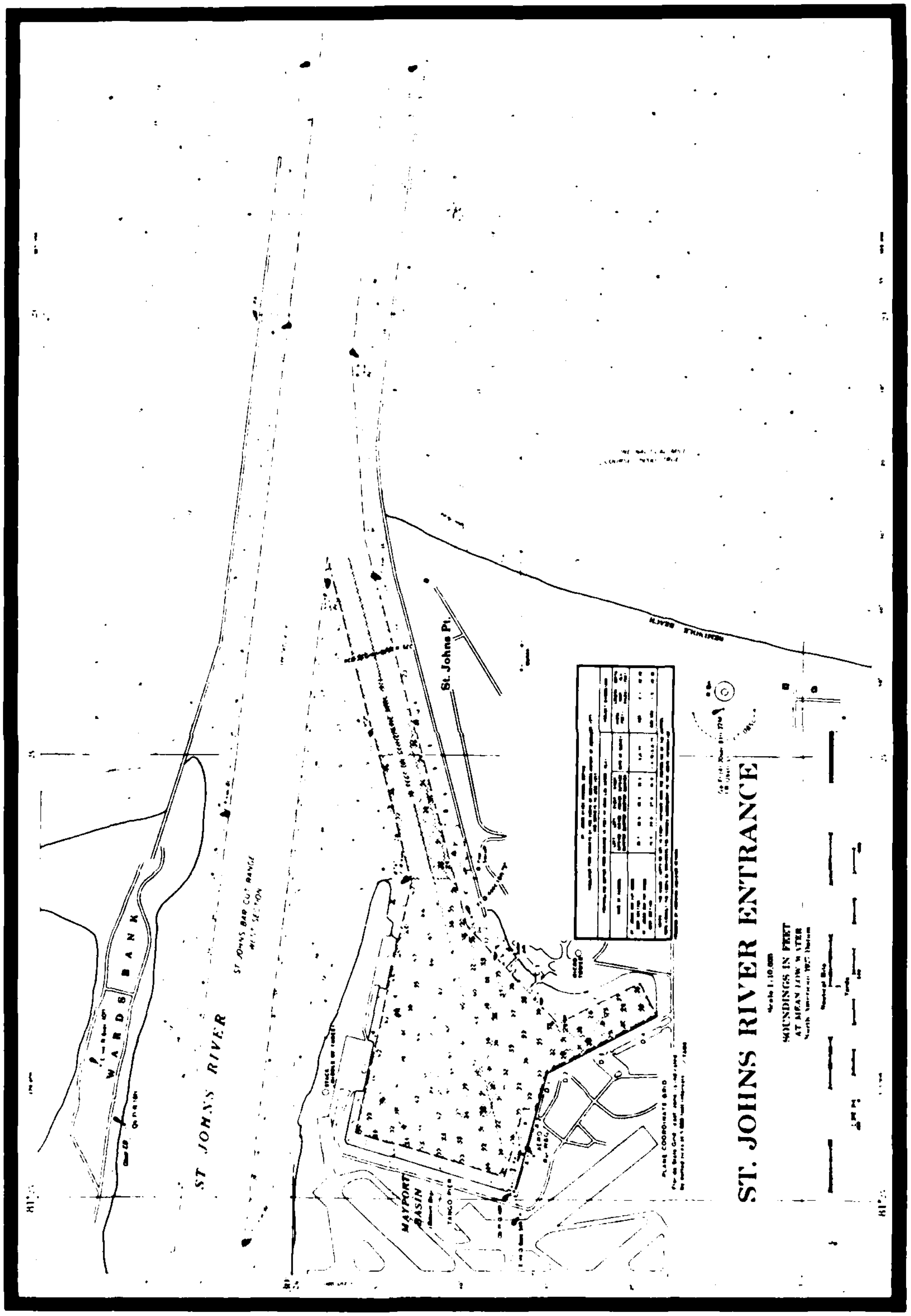


Figure 2-16. Mayport Naval Station Basin on St. Johns River

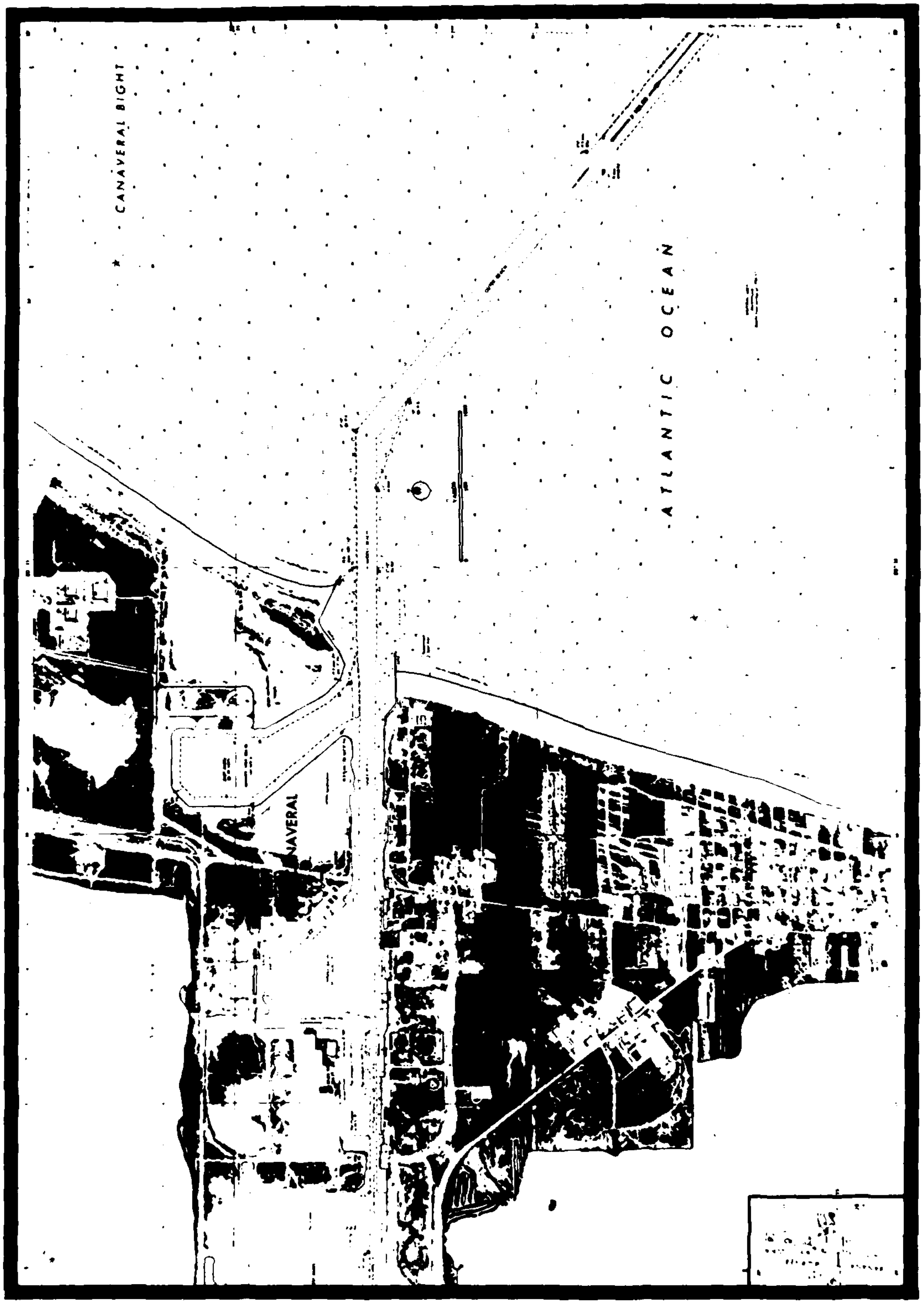


Figure 2-17. Port Canaveral, Florida

The remaining 10 percent of the material dredged with Navy funds is sand. Sand deposition occurs in high-energy environments such as in harbor entrances where jetties cross the ocean's surf zone. Energy is so high that mud size (clay and silt) sediments are washed away leaving deposits of sand. Sand is rarely contaminated because it is chemically inert.

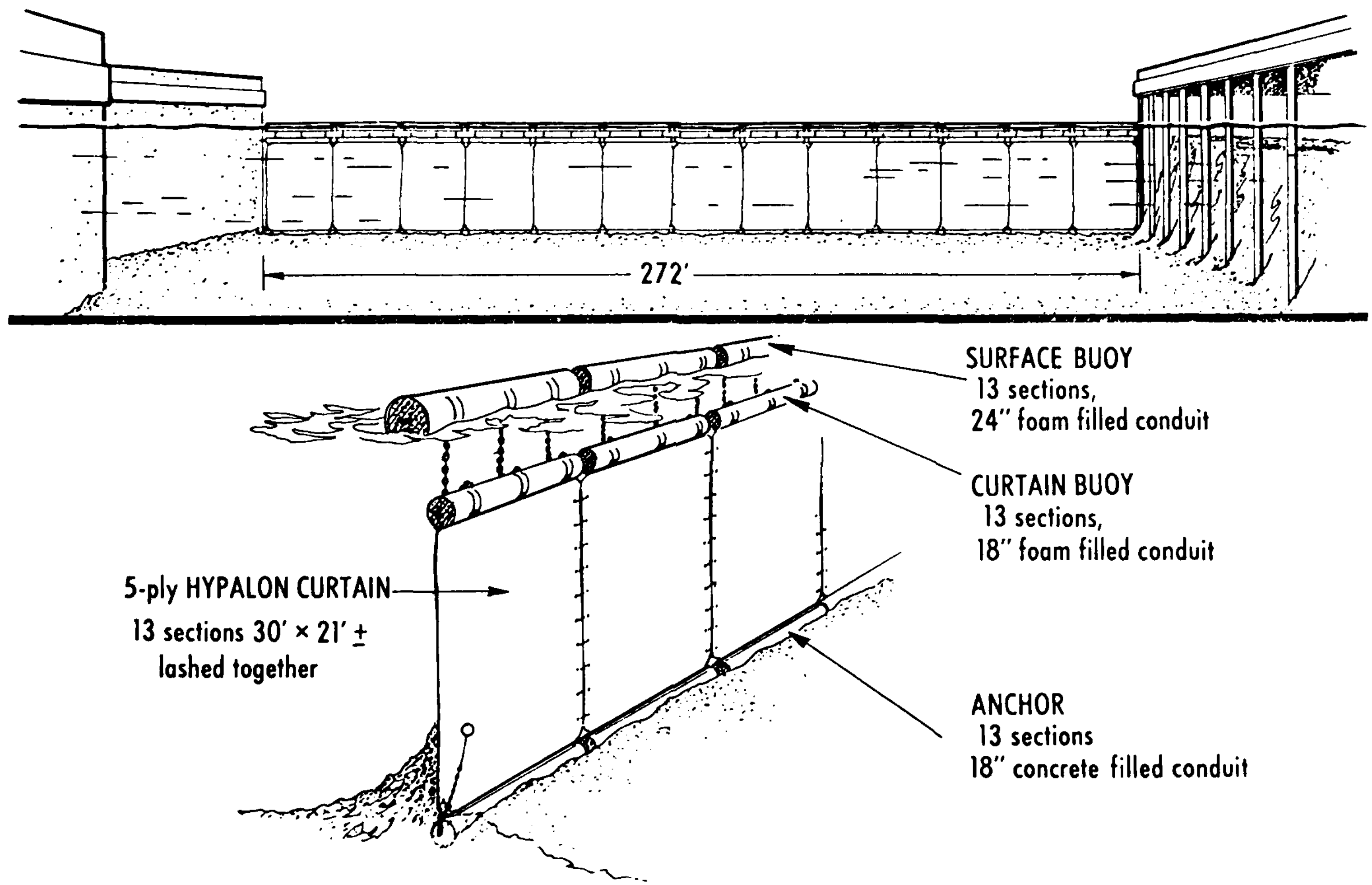
ALTERNATIVES TO DREDGING

As of April 1980, three viable systems to obviate dredging have had successful full-scale tests.

A curtain barrier was placed across the entrance to Berth 20-21N, Mare Island Naval Shipyard. The curtain is 272-feet long and 34-feet high (from the harbor floor to MLLW). The curtain prevented 80 percent of siltation over 3-week test. About 2 feet of deposition occurred outside the curtain and about 6 inches occured inside the berth (Figures 2-18 and 2-19). Since quiet water cul-de-sac berths make up over two-thirds of the Navy's dredged areas, this system has very good potential.

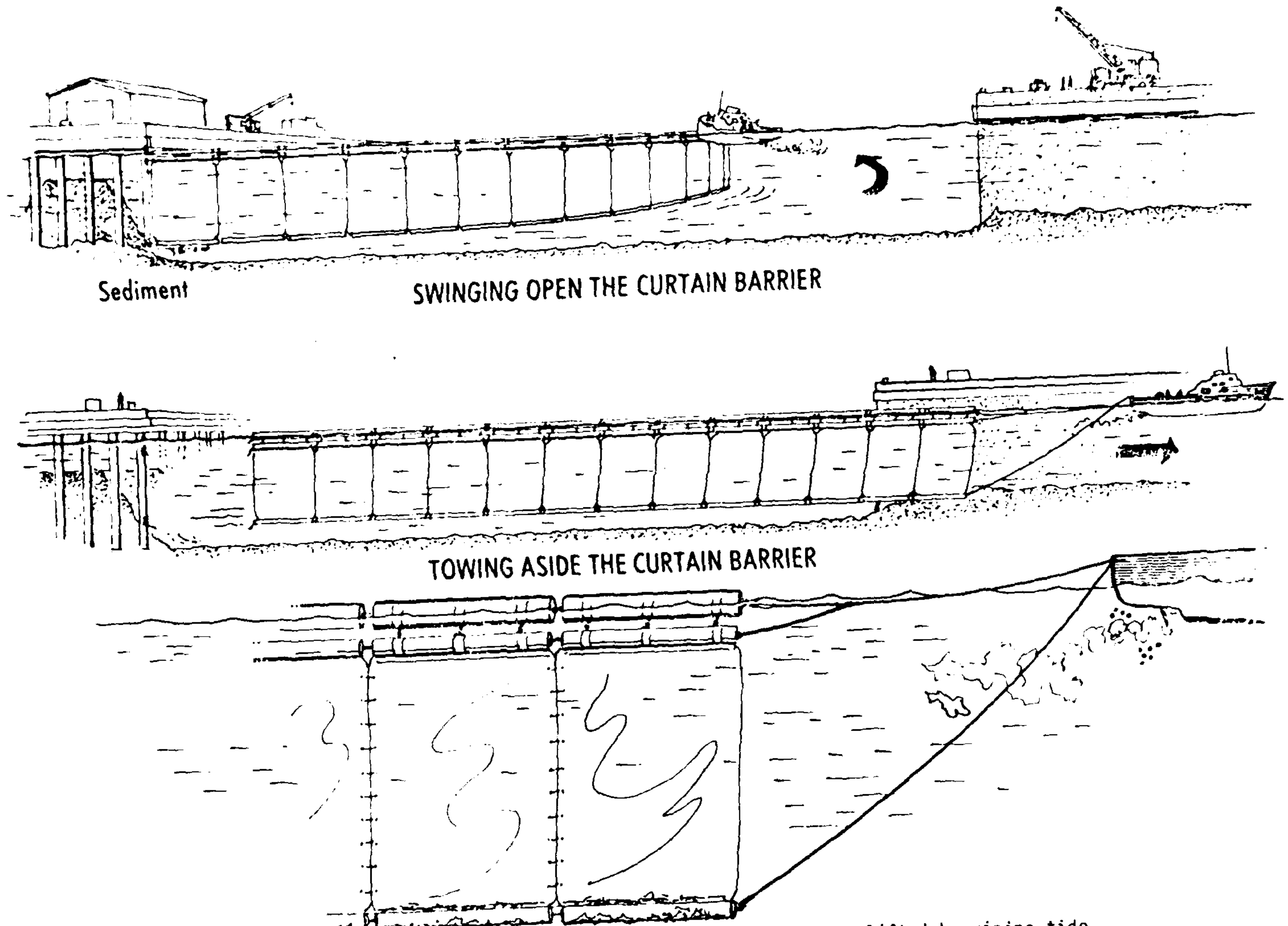
A water jet array placed along the Quay Wall at Mare Island Naval Shipyard prevented 100 percent of siltation as far as 100 feet out from the wall. This system obviates dredging and also the need to move berthed submarines in order to dredge. This system has good potential for selected sites (Figure 2-20).

Another field experiment was successful in removing sand from a shoaled lagoon inlet and pumping it down coast (Figure 2-21). This system can be used to move sand along the beach where obstacles to natural transport, such as harbor jetties interfere. This system has good potential at selected sites.



Curtain barrier viewed from within MINSY Finger Pier berth 20-21N.

Figure 2-18. Curtain Sediment Barrier



Two methods of moving the curtain barrier after lifted by rising tide.
Viewed from Mare Island Strait.

Figure 2-19. Methods of Moving Curtain Barrier

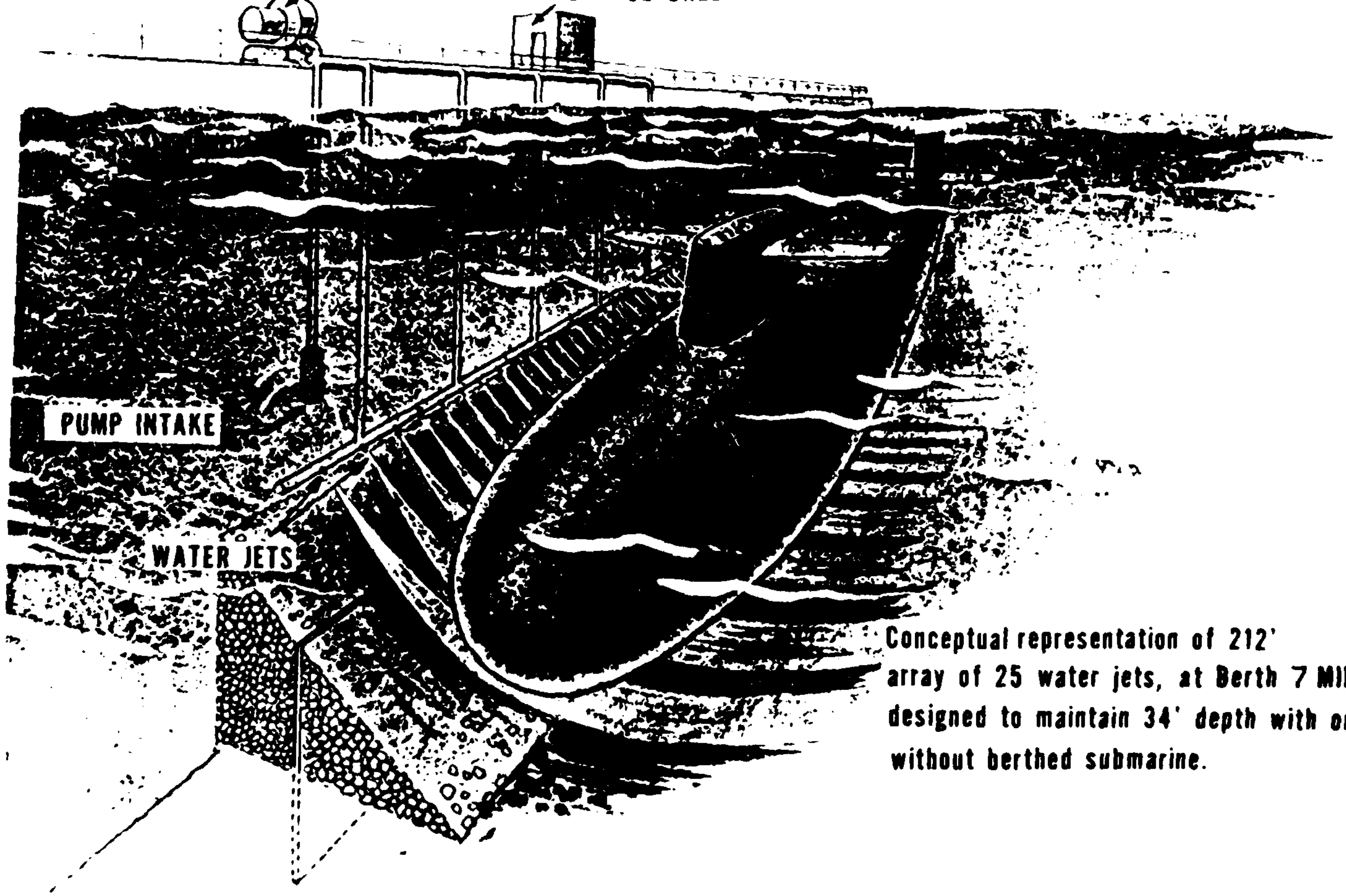


Figure 2-20. Water Jet Array

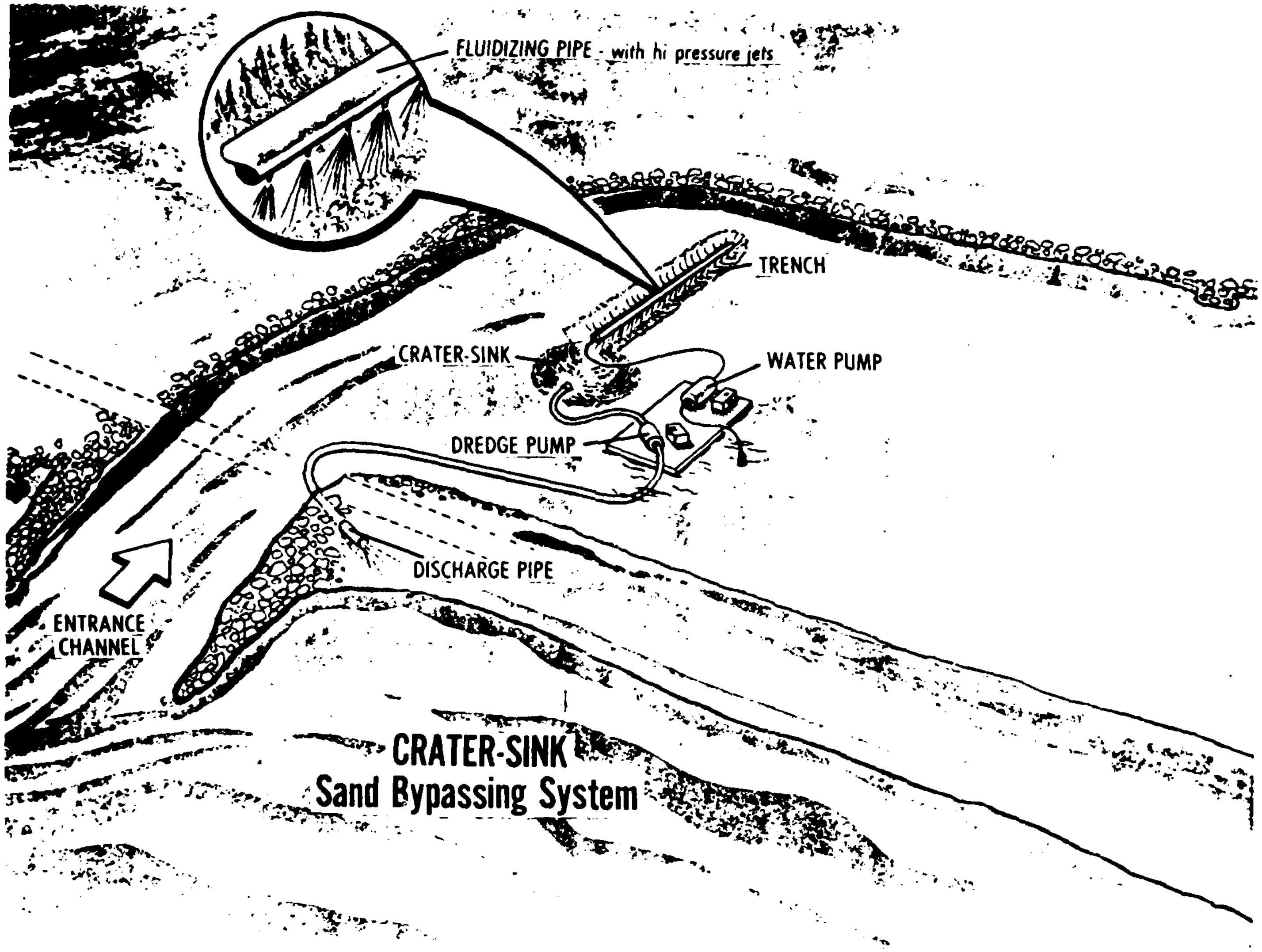


Figure 2-21. Crater-Sink Sand Bypassing System

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SECTION 3

CURRENT DREDGING PRACTICES

by

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LARGE AREA DREDGING

In order to accommodate ships with a draft deeper than the natural water depth in harbors and estuaries, channels are dredged in selected locations within a harbor. The bathymetry of the harbor or estuary before dredging represents the natural effect of sedimentation. Sediments, once deposited, are shifted by tides, river flows, storms, waves, and ship passage. Dredged channels act as catch basins trapping these shifting sediments. Once in the deeper channels, sediments are not easil dislodged by water movement, if at all. As a result of the accumulation of these sediments, the underkeel clearance of ships navigating in these channels decreases. Removal of the sediment by maintenance dredging is a necessity to enable the continuation of the flow of ship traffic.

Pier slips, turning basins, and channels are usually dredged one to two feet deeper than the desired depth for purposes of economy. Accurate control of dredging is not possible. Rather than trying to dredge exactly to a desired depth by careful manipulation of the dredging equipment and pay for the additional time involved, it is cheaper to pay for the extra foot or two of dredging. There may be some gain by this procedure insofar as the additional dredged depth may allow for additional siltation before dredging is necessary again, but a number of factors preclude this from being a generality.

Maintaining the depth of the Naval facilities and approach channels may require singly or in combination the efforts of the Corps of Engineers, private contractors and, in two harbors, Navy-owned equipment.

Described briefly below are the three major dredging methods used to remove sediments from channels, pier slips, and turning basins in the United States. These are the hopper dredge, the hydraulic cutterhead pipeline dredge, and the clamshell bucket and scow. A fourth dredging method, the bucket dredge, is described briefly but is not in use in the United States.

Hopper Dredge

A hopper dredge basically consist of a huge holding tank(s) or hopper surrounded by a ship. The tank is filled in the following fashion. A dredging head, located outboard of the ship and at the lower end of a pipe connected to a centrifugal pump, ploughs along the bottom loosening the sediment as the hopper dredge moves forward at a speed of about three knots. The head weighs on the order of 10 tons. Simultaneously, the centrifugal pump pumps the loosened sediment into the hopper. When the hopper is filled the dredge steams to a disposal area, the bottom doors an opened, and the dredge spoil contained in the hopper is released.

Hopper dredges can contain more than 11,000 cubic yards in their hoppers and can excavate material from as deep as 70 feet below the water level. The McFarland, built for the Corps of Engineers in 1967, has an overall length of 300 feet, a beam of 72 feet, a draft of 22 feet, and a hopper capacity of 3100 cubic yards.

Alternate methods of unloading the spoil are by pumping through a pipeline using a centrifugal pump or by discharging through a pipeline a short distance from the dredge. The latter method is called sidecasting. One of the newest designs in hopper dredges has the hull split open, and the entire contents can be released in less than a minute.

According to World Dredging Magazine (Feb 1980) the U.S. Corps of Engineers has 14 dredges ranging in capacity from 500 cubic yards to 8115 cubic yards. The ownership of hopper dredges in the United States is not necessarily confined to the government, however, for at least two of the larger dredging companies have their own.

Hydraulic Cutterhead Pipeline Dredge

The description of the operation of this type of dredge has been succinctly presented by Gren (1979).

> "The Cutterhead Suction Dredge, also called the pipeline dredge, is the most widely used type of dredge in the United States and is the basic tool of the private dredging industry. This type of dredge utilizes a rotating cutter on the end of the dredge ladder which physically excavates the material from its in situ condition and mixes the material with dilution water and from there it is pumped hydraulically and discharged through a stern connection to pontoon and shore pipe. The dredge is generally controlled on stern mounted spuds and is swung from one side of the channel to the other by means of swing gear. The Cutterhead Suction Dredge provides a dredging tool which under proper conditions can handle large volumes of material in an economical fashion. Equipped with the properly designed cutterhead this dredge can excavate material ranging from light silts to heavy rock properly blasted or can dig softer sedimentary rock properly blasted or can dig softer sedimentary rock in relatively thin lenses. It can effectively pump the dredged material through the floating and shore discharge lines to disposal sites. With the aid of booster pumps in the line, the material can be pumped to disposal sites located at great distances from the waterway being dredged."

> "The pipeline dredge, with its trailing discharge line, does present a navigation hazard in areas of high vessel density. As a general practice, the pipeline dredge should not be employed in dredging work in the main navigation channels wherein a danger exists to the dredge and passing vessels. In instances where a navigation channel has to be crossed, submerging the pipeline reduces this hazard. Limitations on suction pipe length and spuds for holding the dredge in position practically limit the conventional pipeline dredge to excavation depths of 60 feet. Specially designed ladders have extended this depth to 200 feet. Dredges operated in rough waters frequently utilize anchor cables in lieu of spuds."

> "Cutterhead dredges come in sizes, as measured by the diameter of the pump discharge, varying from 6 inches to 42 inches. Contractor-owned equipment in the United States today varies from 6-inch dredges with about 300 H.P. on the dredging pump to 42-inch dredges with more than 10,000 H.P. Cutter horsepower varies from 75 H.P. or less on quite small dredges to more than 2,500 H.P. on larger dredges. They can operate over a wide range of depths; even on occasion being utilized to excavate material above water level. The production rate of each size dredge may vary considerably depending on the characteristics of the material to be dredged. For example, the normal production for

a typical 27-inch dredge could range from 150 cubic yards per hour in blasted rock to perhaps 2,000 cubic yards hour in mud and soft clays."

Clamshell Bucket and Scow

The commonest and perhaps and oldest method of dredging is that of using a clamshell bucket raised and lowered by a crane mounted on a barge and filling an adjacent scow or barge. Ideally, the clamshell bucket closes tightly; however, this is usually not the case. As a consequence, sediment-ladened water is distributed throughout the water column as the bucket is raised. Environmentally, this is undersirable.

Scows vary as to dumping capability. Some are self-propelled, others must be towed to the dump site. Releasing the spoil at the dump site may be in the following ways: by opening bottom doors, tilting the barge sideways, pumping off the spoil by means of centerifugal pumps, or using a clamshell bucket which is the least desirable. A recent innovation is the split hull barge that enables dumping in less than a minute.

Though inefficient, one big advantage to using a bucket and scow is its mobility. Dredging at the base of bulkheads and fender piles can take place with no damage to equipment or piers and bulkheads. It does not have an extensive pipeline as in the cutterhead pipeline dredge and it can dredge in places that are inaccessible to the hopper dredge.

Bucket Dredge

This type of dredge is not used in the United States but, because of its frequent use in Europe (Hoffman, 1978) and other parts of the world, a brief description is presented here. A bucket dredge, also called a ladder-bucket dredge, utilizes an endless chain of buckets moving between two ladders or guides that extend into the water at an angle to the deck. In some instances the buckets have teeth welded to the cutting edge. The buckets scoop up the bottom material and dump it into a chute that overhangs a barge. The material slides along the chute into the barge. Two kinds of barges are used: one self-propelled, the other towed. Similar to the lateral movement of the head of a cutterhead pipe line dredge, the bucket dredge dredges a swath by being swung laterally by means of cables fastened to anchors located off the starboard and port sides. Bucket dredges can dredge in depths of water up to about 38m. Rates of dredging can be up to about 800 m^3/hr.

SMALL AREA DREDGING

Described below are four methods that have been used in maintaining pier slips but, owing to the relatively low rate of production, their use for dredging large areas is not feasible. These methods are: agitation dredging, Pneuma method, eductors, and the Mud Cat dredging system. The descriptions below are limited. Additional information is contained in the NAVFACENGCOM sponsored report USNA-EPRD-37 (Hoffman, 1977).

Agitation Dredging

Agitation dredging is the removal of sediment from pier slips and wharves located adjacent to shipping channels by suspending the sediment by agitation at the time of an ebbing tide. The suspended sediment is carried to the main channels by the outflowing water and thence down channel. Agitation is accomplished by dragging an I-beam or similar device behind a tug or by deflecting downward wash from boat propellers.

Because of the fact that dredging the main channels falls within the purview of the Corps of Engineers, the Corps requires reimbursement for all sediment dredged by agitation dredging. For example, in the case of Savannah River piers, Savannah, Georgia, reimbursement is at a rate of $176 per hour of dragging time. In this river, the Corps of Engineers' 1973 records indicated that approximately 450 hours of agitation dredging was performed in the Savannah Harbor.

Pneuma Dredge System

The pneuma dredge system is a dredging system that is unique. It has the capability of being able to remove sediments with a minimum of resuspension in the water column.

The pneuma system consists of four principal components.

- A pump body which consists of three cylinders bolted together. Each cylinder has only one moving part - an inlet check valve.

- A distributor to receive compressed air delivered through a single line from one or more compressors and distribute it cyclically to the three cylinders of the pump body; and to receive the used air from the cylinders and exhaust it to atmosphere.

- Air compressors which may be diesel or electrically driven.

- Compressed air delivery lines which are usually a combination of steel pipe and hose and a slurry delivery line which is a combination of steel and/or plastic pipe, and rubber hose.

The operation is as follows: the cylinders are submerged so that the inlet ports are in contact with the material to be dredged. Atmospheric pressure exists inside the empty submerged tanks. The difference between the internal atmospheric pressure and the external hydrostatic pressure at the submerged depth causes the inlet valve to open and a mixture of water and sediment to enter the cylinder. When the cylinder is filled, external and internal pressures are equal and the inlet valve closes because of its own weight. Compressed air from the distributor enters the cylinder from the top and forces the slurry into the discharge pipe which extends almost to the bottom of the cylinder. The distributor causes a cyclic operation of the tanks such that a constant flow is maintained in the discharge manifold.

To date, this method has been used to a limited extent in the United States. However, wider use of this method has been made in Japan.

Eductor Systems in Dredging

The use of eductors for dredging pier slips is useful in non-cohesive sediments. The basic eductor works on the principle of the Venturi tube. When a jet of water is constricted in a tapered tube a vacuum is created. Fluid from the surrounding environment moves towards the chamber. If the eductor rests on a sandy submarine bottom, both sand and water are sucked into the vacuum chamber and passed along with the flow. If on a flexible hose, the eductor sinks into the sand to form a crater. Fluidized sands then move laterally to the low point in the crater.

Use of an eductor system to pass littoral drift beneath two jetties to prevent beach starvation at Virginia Beach was investigated by the Corps of Engineers Waterways Experiment Station (WES). Before system installation, shoal areas at Rudee Inlet resulting from the deposition of sand, prevented the ingress and egress of boats, as well as starved the beach at Virginia Beach. (Hoffman, 1977).

The basic system consists of an eductor that sucks up sand and pumps it to a pump that pumps it through 1800 feet of pipe beneath Rudee Inlet to the north side of the jetty where longshore currents transport it northward. A crater formed around the eductor results in the movement of sand laterally to the eductor. The slope of the crater wall is about one vertical on two horizontal. Thus a crater 10-feet deep has a diameter of 40 feet. To facilitate movement, the sand in the vicinity of the eductor is fluidized by two jets located on either side of the eductor. The rate of flow through each jet pipe is 75 gallons per minute (gpm). Once the desired depth is reached, the eductors are manually moved by scuba divers to an adjacent location.

Mud Cat Dredging System

The Mud Cat is a compact, portable machine designed to hydraulically remove sediment deposited in waterways, marinas and impoundments.

In this dredging system, a hydraulically-operated boom lowers a horizontally-mounted auger-cutter assembly into the material to be excavated. The auger-cutter assembly dislodges and delivers the material to the pump suction intake. The slurry is pumped to pipeline for transmission to a remote location.

The dredge is comprised of an integrally welded platform supporting a diesel engine, a centrifugal pump, the horizontal auger-cutter assembly and the control center. The principal controls are hydraulically operated. It is easily transported from site to site and can be launched and retrieved quickly. Generally, a crane is used for this purpose. The overall dimensions are 8-feet wide, 9-feet 3-inches high, and 30 to 39-feet long depending on the model.

Prior to placing the machine in operation, an anchored cable network is rigged and a pipeline assembled. A portion of the cable is threaded through a winch mechanism which propels the machine in forward and reverse directions along a guide cable.

Materials are excavated as the dredge moves forward and backward. Several passes are normally required in the same cut to excavate underwater materials to a predetermined depth.

Limitation in equipment design has made it possible to dredge only to a depth of 20 feet.

In order to use the dredge to greater depth, the following changes would have to be made:

1. The power requirements would have to be increased.

2. Support winches would be required to raise and lower a longer boom due to added weight.

3. A submersible pump, either hydraulically or electrically powered, would have to be designed and mounted on the end of the boom directly behind the auger.

DREDGE SPOIL DISPOSAL

The disposal of spoil from dredging is becoming an increasing problem. The environmental effects of dredge spoil disposal are being examined critically for possible impacts on the environment, thus, reducing the places available for disposal.

Various places where spoil has been dumped with varying degrees of acceptance are:

1. diked-disposal areas
2. open-water dumping
3. land fill
4. non-productive wetland fill

Dike disposal areas have been constructed since the mid-1950s, when Craney Island in Norfolk Harbor was built. These are engineered structures with rip-rapped dikes on the water sides. A method of offloading the spoil is provided together with an outlet for the return of water to the adjacent water bodies. Barges and Hopper dredges can be offloaded by pumping, and cutterhead pipeline dredges can discharge directly to the area by means of pipeline. The size of these areas constructed initially depends upon the anticipated volume of spoil to be contained in the area for a selected period of years.

Where the spoil has settled sufficiently to have an adequate bearing capacity, a second peripheral dike may be constructed on top of the disposal area to contain a second "layer" of spoil. Bearing capacities of the spoil and that of the underlying in situ material must be taken into account or else structural failure will result. After the area is filled the land created is often of value economically.

Open water dumping appears to be an easy solution to the spoil disposal problem. However, the expense of steaming time to a designated dredge spoil site and the expenditure of fuel, in some cases, has led to the use of diked disposal sites (e.g.) Craney Island). Certain U.S. Navy problems have occurred in open water sites that limit the use of this practice. Dredging for deepening the Thames River leading to Groton, Connecticut to permit the passage of the SSN 688 Class submarines led to problems with environmental groups that ultimately ended up in court (Hoffman, 1977). One of the major contentions of the plaintiffs was that the dredge spoil was flowing laterally outside the designated area and blanketing bottom-dwelling organisms. They also contended that the long-term effects of pollutants, especially heavy metals, would be distributed throughout the food web and would effect man ultimately.

Another problem with open water spoil disposal occurred at the Naval Station at San Diego where the dredge sediments could not pass the bioassay test as described in COE-EPA publication "Ecological Evaluation of Proposed Discharge of Dredge Material Into Open Waters" July 1977. This test is described below.

One of the first tests instituted by the Corps of Engineers to test for the suitability of the dredge spoil was the elutriate test. Stripped of technical details, the test essentially consists of analyzing the water column at the dump site for concentrations of selected toxicants. A sample of the spoil from the dredge site is mixed with the water and agitated. The mixture is then filtered to remove suspended material and analyzed for the same toxicants as the water from the water column. If the increase in concentration is less than 50 percent, the spoil is considered to be acceptable for open water disposal at the site in question.

Inasmuch as this approach was limited to the toxicants for which analysis was made, a broader view utilized the bioassay method. In this test an organism prevalent at the dump site is contained in laboratory tanks with the spoil that was a candidate for disposal. If the mortality of the organism exceeded acceptable limits after the designated period of time, the spoil was considered to be unacceptable for disposal at the dump site.

Perhaps a more realistic approach has been proposed in a paper entitled "Application of the Biotal Ocean Monitor System to In Situ Bioassays of Dredged Material" (W. Pequegnat, 1979). In this method, selected indigenous organisms are contained in an open water area where disposal of the spoil is to take place. The containment could vary from large cages suspended in the water column to wholly-enclosed containments from the water surface to the bottom. For testing purposes, a spoil would be dumped within the containment and the effects on the organisms observed. At present, only a few tests patterned after this test have been run. However, such an approach offsets the criticism made concerning laboratory bioassay that the organisms are stressed too highly in the laboratory environment and the results do not represent the actual situation.

The third possible spoil disposal site is landfill. In many areas such an alternative is not feasible because of land values. Where sand and gravel or minerals have been strip-mined, however, depositing spoil in the depressions may have merit. Important to the feasibility of such an approach is the cost of transportation of the dredge spoil from the point of dredging to the point of disposal. Each area would probably require a different solution. In Holland, dredge spoil from Rotterdam Harbor is pumped from a receiving barge through a pipeline and distributed on polders (Hoffman, 1978).

Wetlands are commonly thought to be inviolate. This may not necessarily to true, however, for not all wetlands are biologically productive. Environmental evaluation of the impact of the disposal of spoil on a wetland may show that it can be beneficial to the ecosystem if certain other conditions are provided. Most states have laws regarding such procedures, and any action must conform to these regulations as well as to the directives of the U.S. Environmental Protection Agency and the Fish and Wildlife Service.

Environmental problems involved in the disposal of dredge spoil are too numerous to be detailed here. However, a few brief comments are made below.

Disposal of contaminated dredge spoil on land can affect the environment in three ways. Leaching of the toxicants by infiltration of precipitation can transport these toxicants to ground water aquifers. Ground water transports these toxicants to nearby streams, bays, or ocean and, thus, transfers toxicants from the disposal site in solution to a surface water body. The slow movement of ground water results in the long-time retention of these toxicants, and transport between the spoil disposal area and the surface water body to which discharge ultimately takes place may occur for a long period of time.

Runoff of precipitation from contaminated spoil piles can result in a more rapid contamination of water bodies. Furthermore, runoff can seep into the land surface en route and enter the ground water system. Even if the spoil is not contaminated, erosion can cause fine particles to move into a water body increasing its turbidity. This turbidity reduces the photosynthetic activity and, hence, decreases the biological productivity of the aqueous environment.

The third effect occurs in the atmospheric environment. Tests have shown that wind blowing over spoil piles containing polychlorinated biphenyls (PCBs) can distribute these carcinogens over a widespread area. PCBs have been reported in Antarctica as the result of this mechanism. Additionally, dumping of dredge spoil can release noxious gases, such as hydrogen sulfide, causing a temporary impact on the environment depending on the point of disposal. Hydrogen sulfide usually occurs in aqueous organic sediments due to the lack of oxygen. Anaerobic bacteria, thriving in such an environment, break down chemical salts, such as sulfates, for the oxygen necessary for metabolism. Such a condition was noted during a visit to the dredge disposal site at Kings Bay Submarine Support Facility, Georgia in January 1980.

DREDGING COSTS

The costs involved in the actual dredging process inv lve cutting of the bottom material, transportation of the spoil to the disposal site, and a fee for disposal if a diked-disposal area is involved.

The costs of cutting bottom sediments vary. The most expensive bottom sediment to dredge is tight clay; the least expensive sediment to dredge is loose sand. Ledge rock and bedrock present problems. The cutterhead pipeline dredge can sometimes dredge rock depending on its hardness. It may be necessary to drill and blast prior to excavation.

Superimposed on the above costs is the cost for mobilization and demobilization of equipment. Transportation of equipment over long distances could increase this cost measurably. Where these costs are high, it may be more economical to dredge a sizeable area to reduce the cost per yard of spoil.

In much the same fashion as other businesses, the availability of work and magnitude of the dredging job can change the cost per yard of spoil. Average costs for dredging used for budget estimates may not correspond to costs evident in the bid for work.

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Hoffman, John F. (1978). European Dredging - A Review of the State of the Art. Office of Naval Research Report R-12-78.

Hoffman, John F. (1977). An Investigation of New Methods for the Maintenance Dredging of Pier Slips and an Investigation of Selected Dredging Problems in U.S. Navy-Connected Harbors. Report prepared for Naval Material Command. USNA-EPRD-37.

SECTION 4

SEDIMENTATION CONTROL EXPERIMENTS AT MARE ISLAND NAVAL SHIPYARD

By

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INTRODUCTION

Experiments testing new engineering principles for controlling sedimentation in Navy harbors were conducted on prototype scales at Mare Island Naval Shipyard, California, over the past 5 years. Among these new principles tested are:

1. Flushing - by water jets to resuspend mud and fluid mud during ebb tide.
2. Exclusion - by full height material barrier curtains to prevent mud and organism intrusion
3. Circulation - by air chimneys (Ventra-Vac units), air bubble curtains, and water jet barriers to raise and disperse settling flocs.

Devices based on the first two principles were found to be successful and compatible with operational restrictions. Devices based in circulation had either too limited a radius of influence or enhanced mud accumulation in areas adjacent to the device.

FLUSHING SYSTEMS

Three generations of jet arrays were built and tested. The first design was mounted on the bottom at a finger pier entrance. Over a 6-month period it prevented 10,000 cubic yards of new mud deposition but was vulnerable to dragging anchors during docking operations. The two following designs operated with only minor damage from quay walls. The latest design prevented any new deposition in a 200-foot long submarine berth for over 2 months until electric drive power was lost along the waterfront. The jet array systems are tide actuated and fully automatic.

EXCLUSION SYSTEMS

Two variations of barrier curtains have been built and tested. These curtains exploit the natural occurrence of the majority of suspended sediment residing in the lower portion of the water column. By closing off the entrance to a berth to bottom water circulation, the barrier curtain prevents the influx of both new sediment and drifting organisms such as green algae colonies or hydroids. When opened three times in 2 months, the barrier curtain was found to prevent 75-80 precent of the new mud deposition. Furthermore, the vertically integrated suspended sediment load in the water trapped inside the curtain was diminished threefold, which greatly improved diver visibility. By streamlining the channel bank, the curtain was also observed to reduce mud accumulation in the next adjacent, yet unprotected berth.

Most of the curtain testing has been interrupted by frequent ship movements. Nonetheless, in 4 months, 50 percent of new accumulation has been kept out of the protected berth, representing a dredging savings of 13,000 cubic yards. It has also been learned how to open and close the curtain by a number of alternate means including winches, end-loader tractors, small tugs, and directed propeller wash. In the event of an emergency, it was demonstrated that the curtain could be dragged open without taking the time to raise its concrete anchors.

Figures 4-1 to 4-6 depict the arrangement of the full-height curtain barrier used in the Mare Island Naval Shipyard experiments. The curtain location at Pier 21 berth is shown plus some performance data. See pages 4-3 to 4-8.)

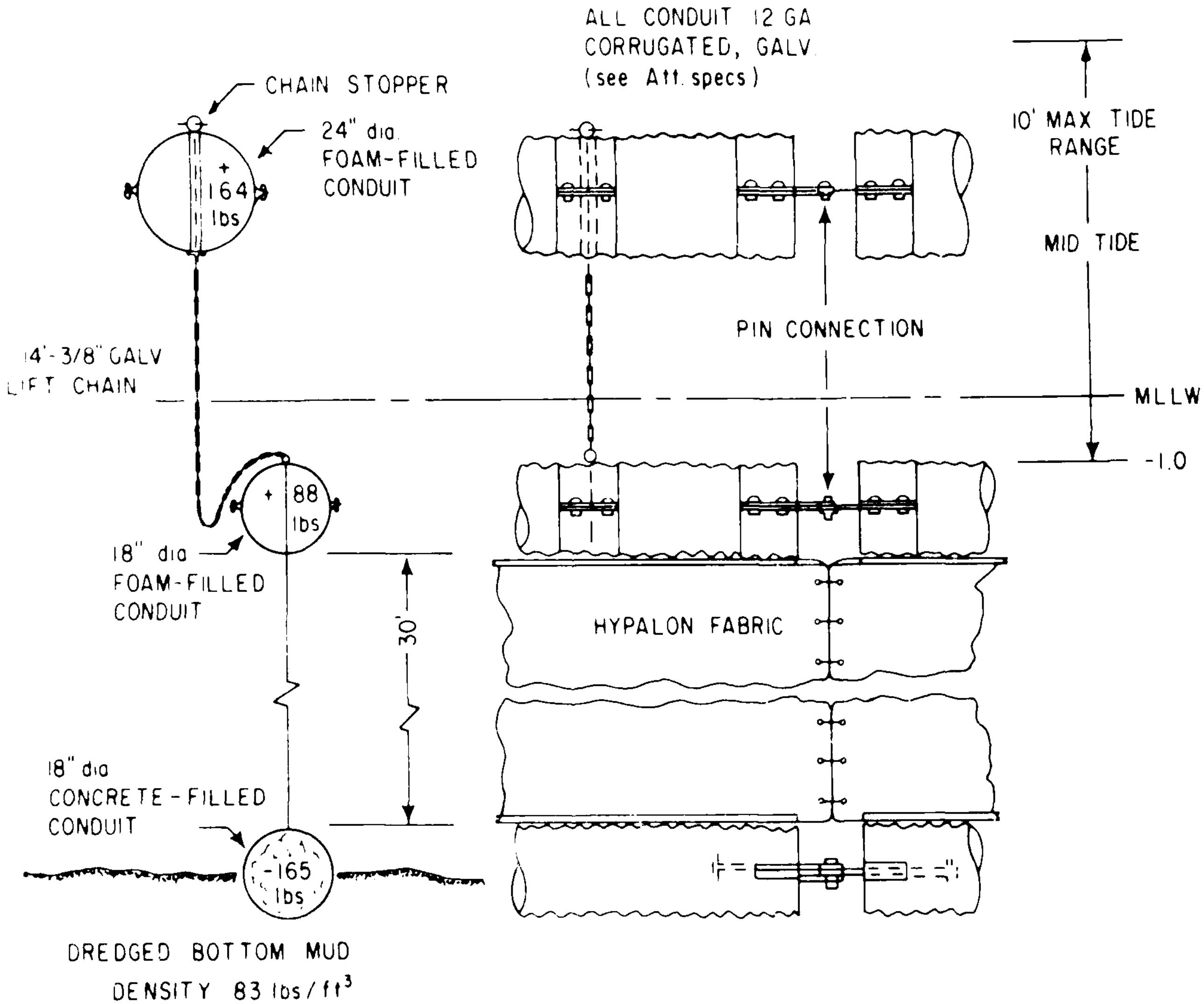

Figure 4-1. Hypalon Fabric Curtain Construction

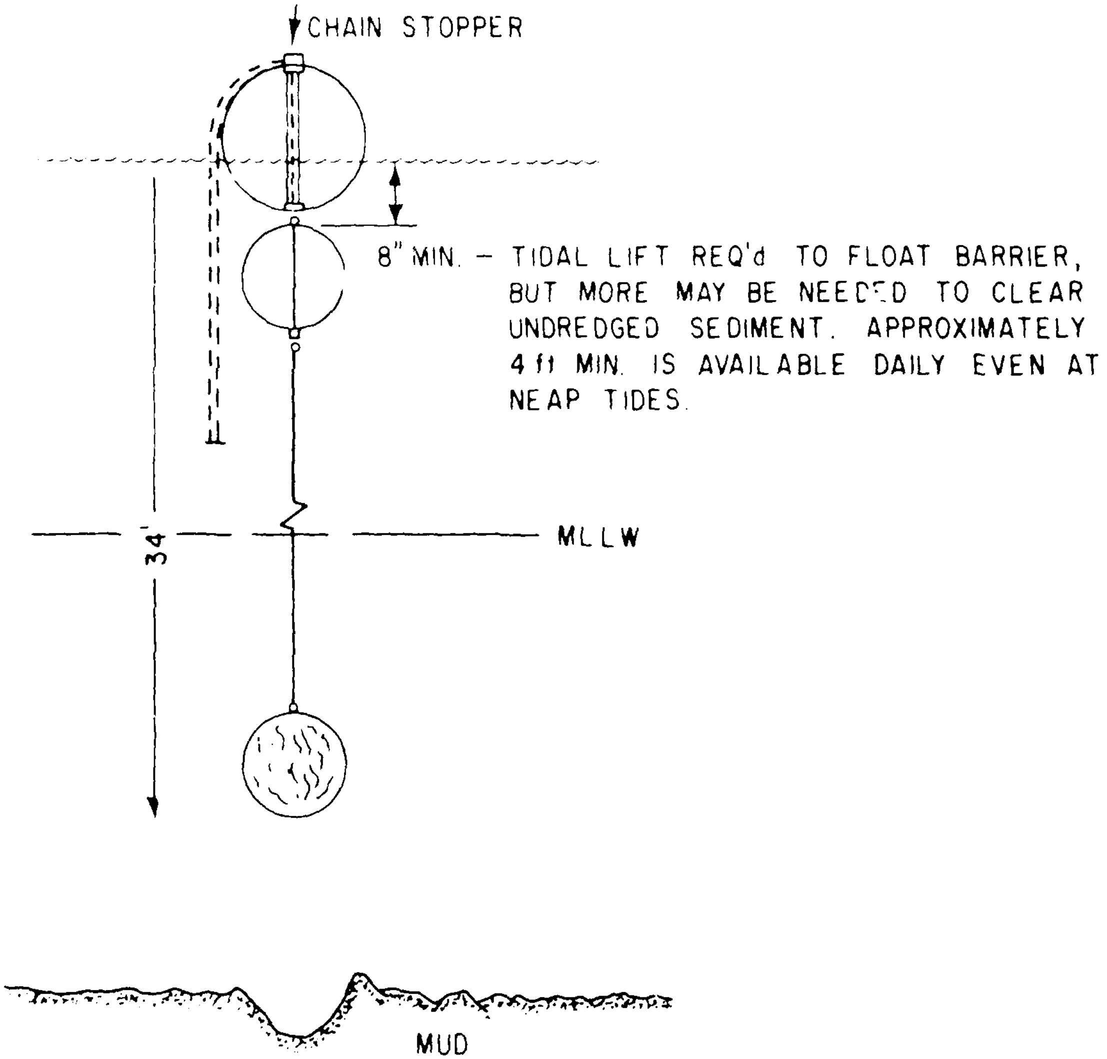

Figure 4-2. Lift Required to Float Curtain Barrier

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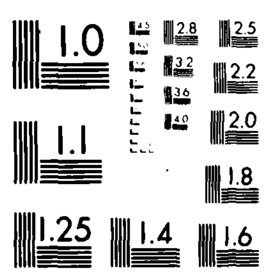

MICROCOPY RESOLUTION TEST CHART

NATIONAL BUREAU OF STANDARDS-1963-A

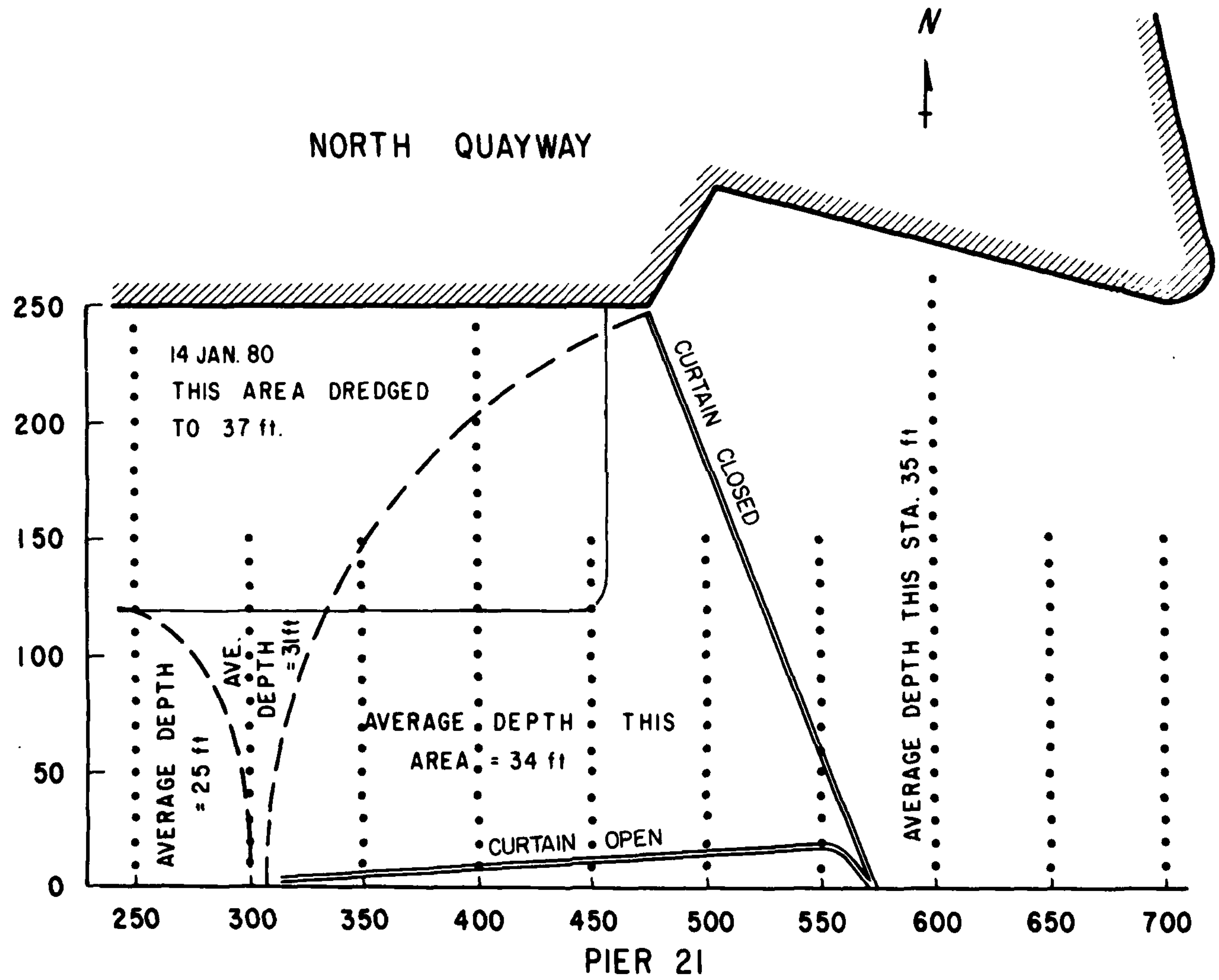

Figure 4-3. Curtain Positions at Pier 21

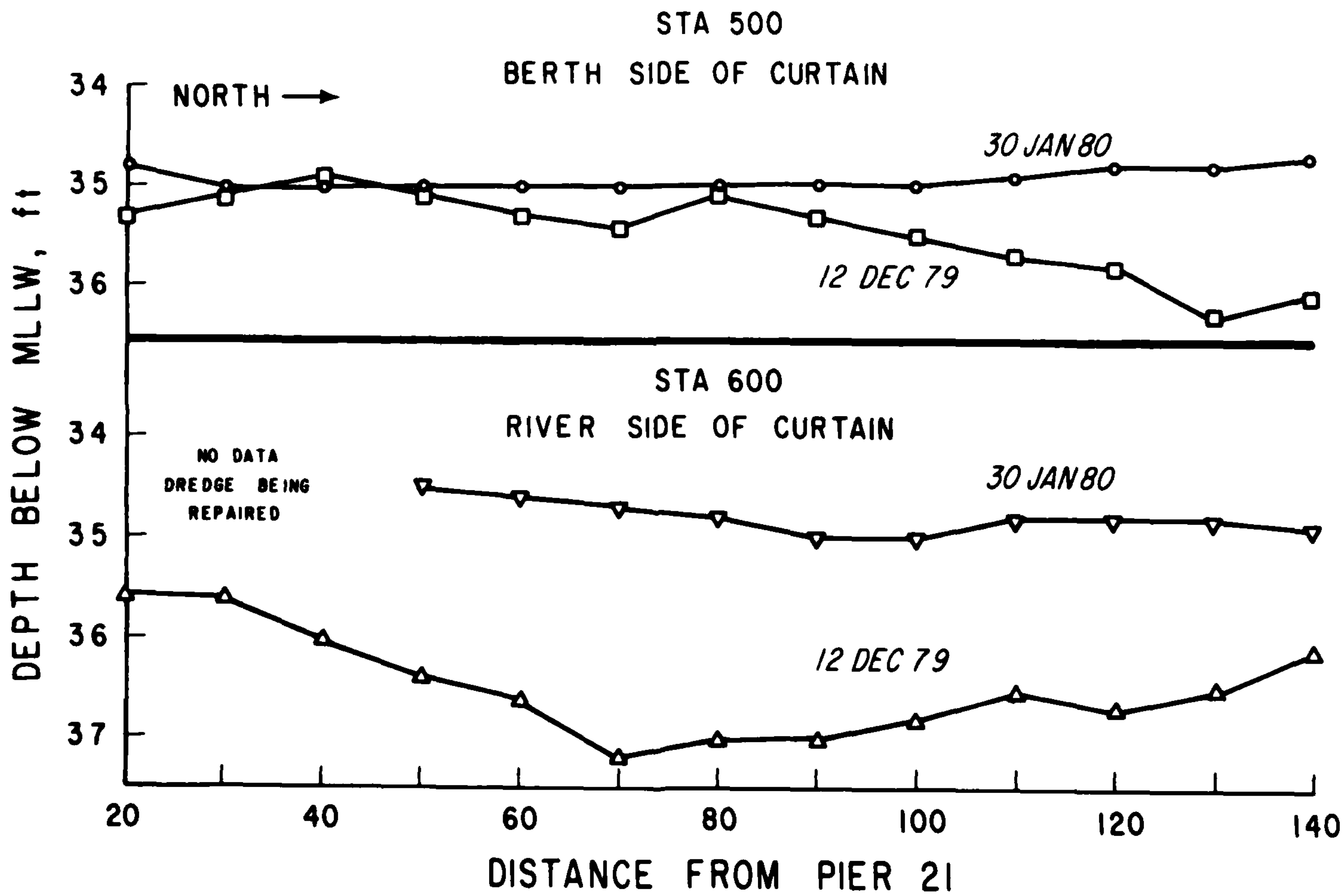

Figure 4-4. Depth Variations Across Pier 21 Berth

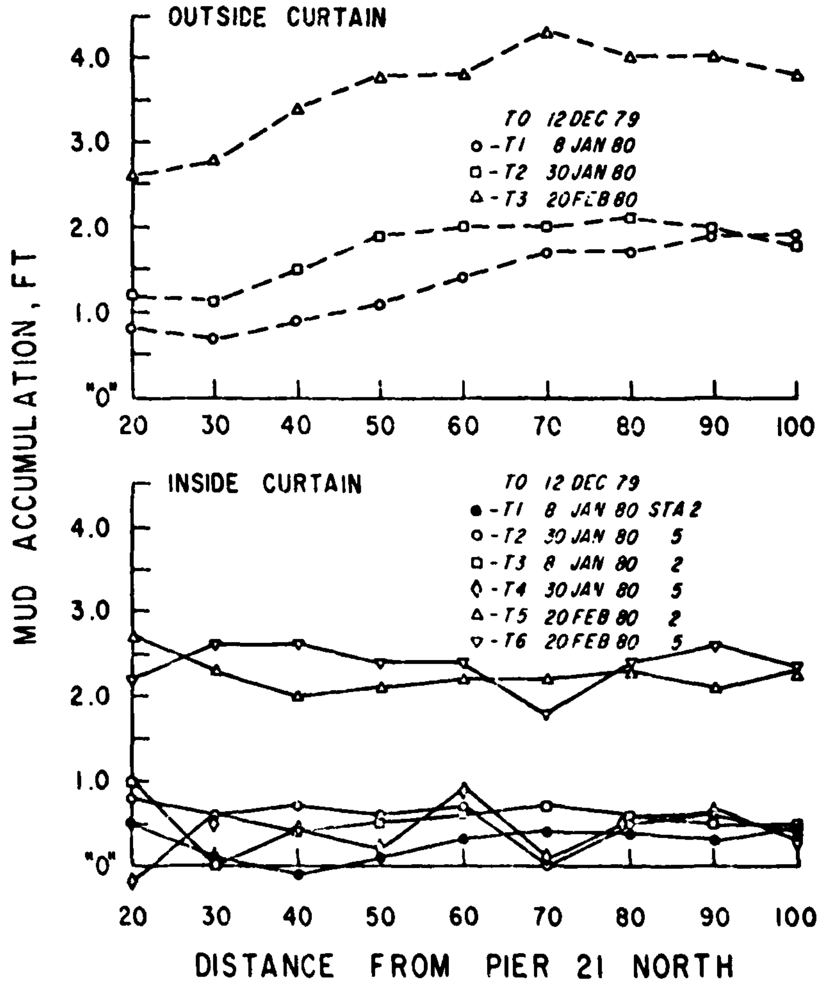

Figure 4-5. Mud Accumulation at Pier 21 During Test Period

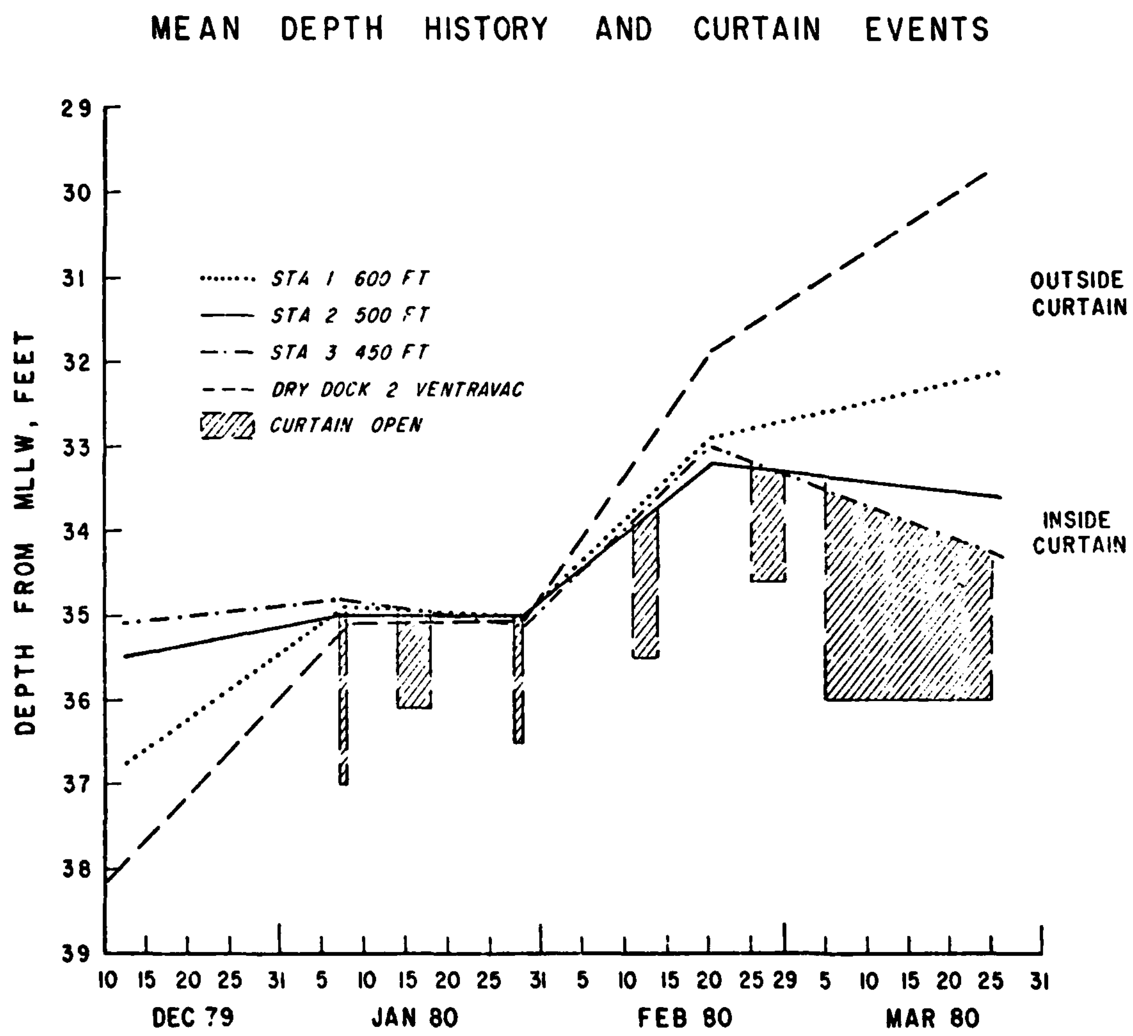

Figure 4-6. Mean Depth History and Curtain Events for 4-Month Period

PROPOSED SEDIMENT/HYDROID CURTAIN FOR MARE ISLAND AND NORFOLK

The Figures 4-7 to 4-14 provide a graphic presentation of a proposed next-generation curtain suitable for sediment/hydroid control for Mare Island Naval Shipyard and Norfolk Naval Station. The design presented is a direct result from tests run on a prototype curtain installed at Mare Island Naval Shipyard. The inputs from the personnel listed below were beneficial in preparing this design.

Scott Jenkins (Project Manager) D. Palmer (Survey and Data Reduction) P. Rohrbough (Fabrication of curtain)	Scripps Institution of Oceanography
J. Dillard (Civil Engineer) D. Campbell (Maintenance Supervisor) P. Wright (Diving Supervisor)	Mare Island Naval Shipyard

OTHER NORFOLK DEPOSITION AND ORGANISM CONTROL SCHEMES

Based upon knowledge and insight obtained as a result of research at Scripps Insitution of Oceanography Sponsored by NAVFAC and others, and a review of the Navy problem in the Sewell's Point Area, several other deposition and marine organism control schemes have been synthesized for the Norfolk carrier pier area. These schemes are depicted in Figures 4-15 though 4-17.

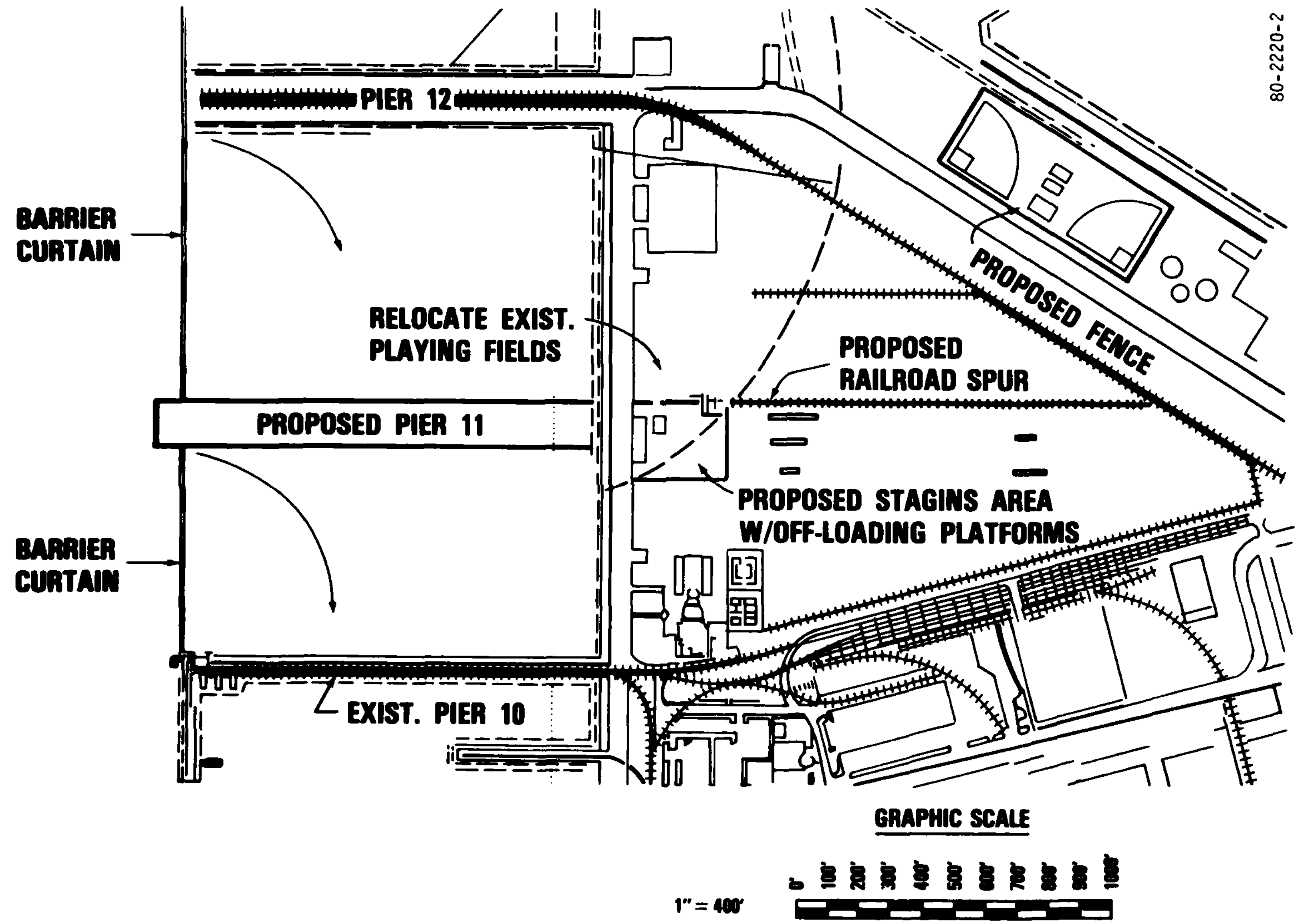

Figure 4-7. Norfolk Naval Station Proposed Pier 11 Complex with Barrier Curtain Protection Against Mud and Hydroid Intrusion

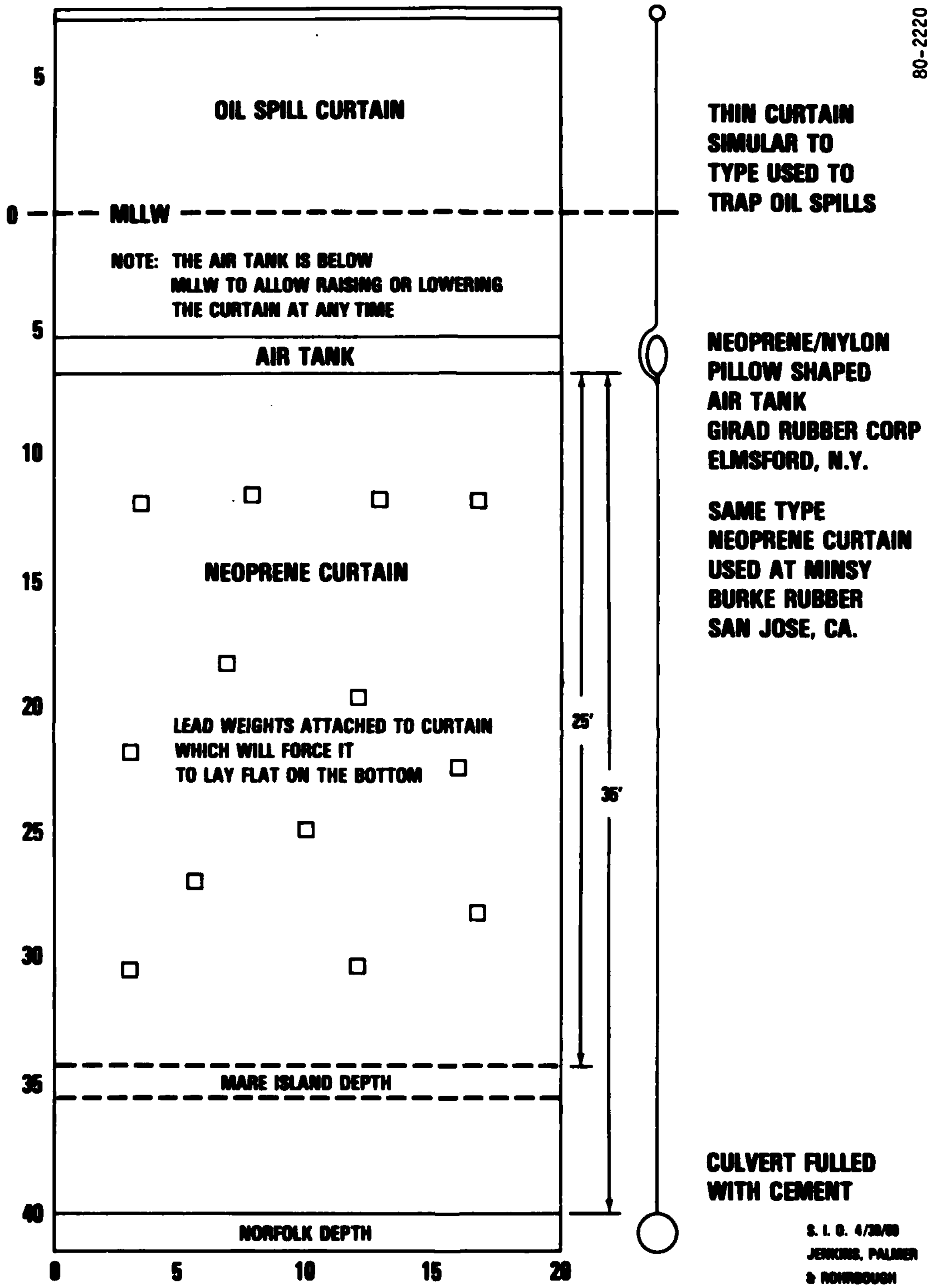

Figure 4-8. Unit Curtain Section Detail - Front View

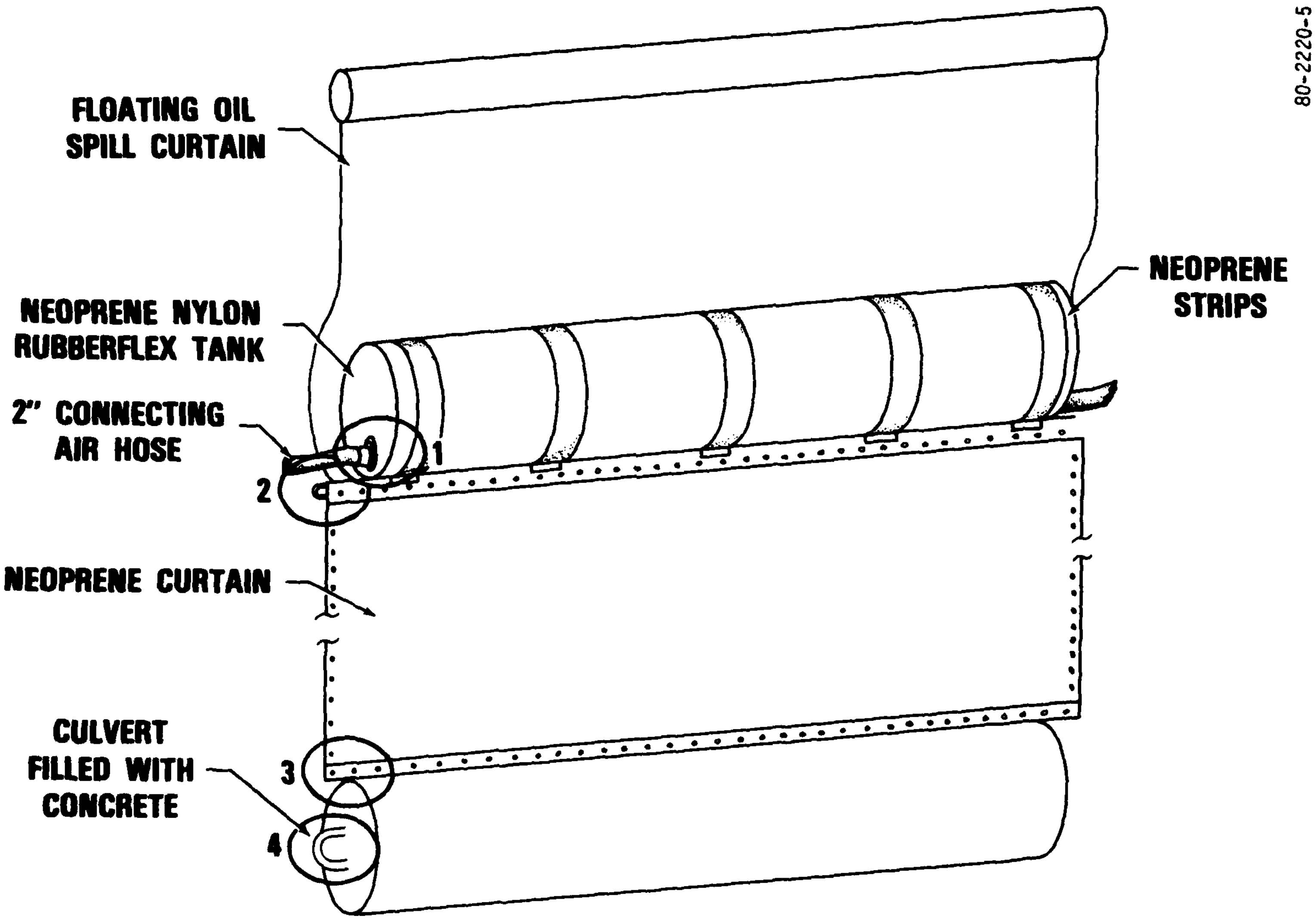

Figure 4-9. Perspective Schematic of Pneumatic Lift and Anchor Systems on A Unit Curtain Section

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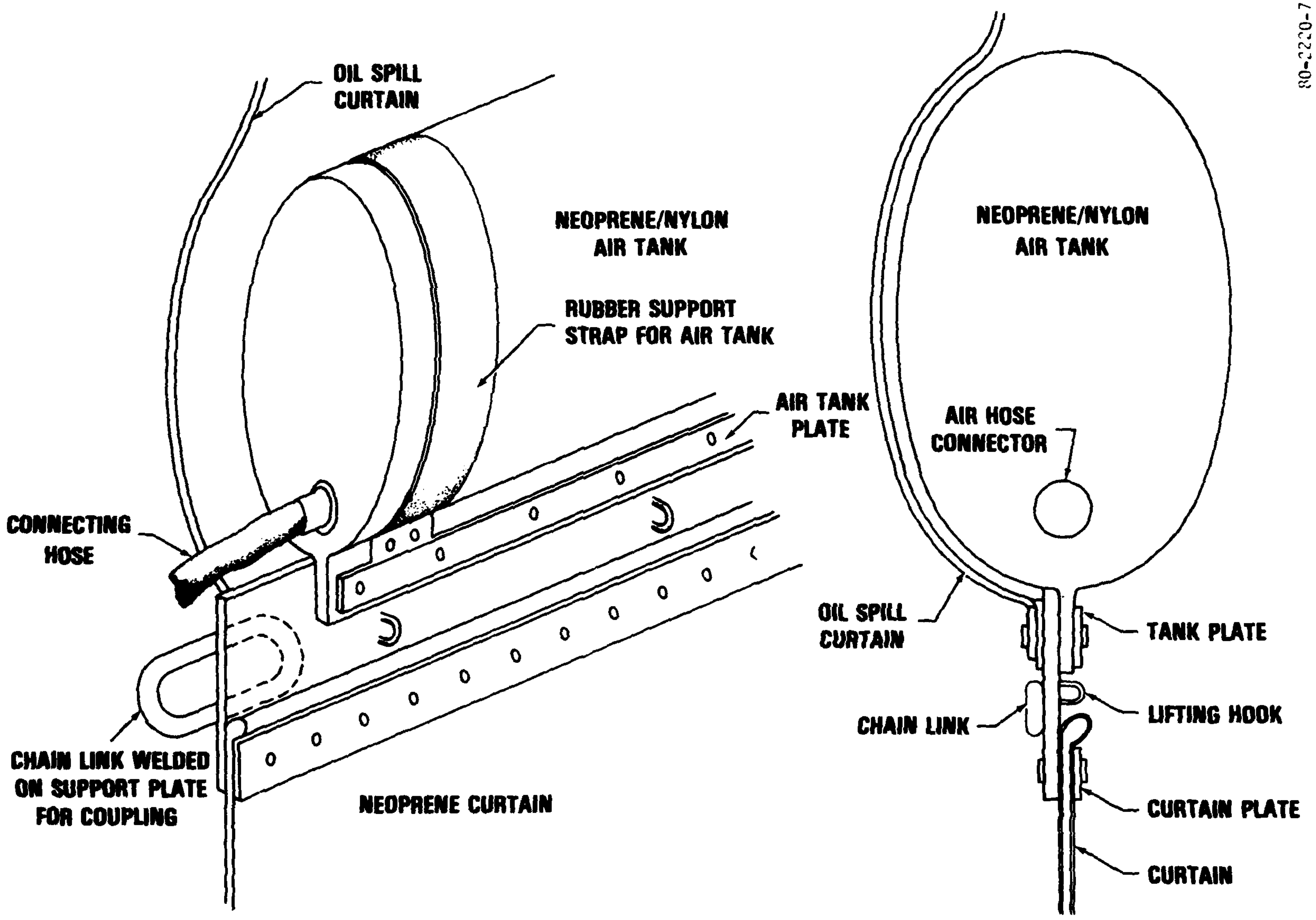

Figure 4-10. Pneumatic Lift Float Detail

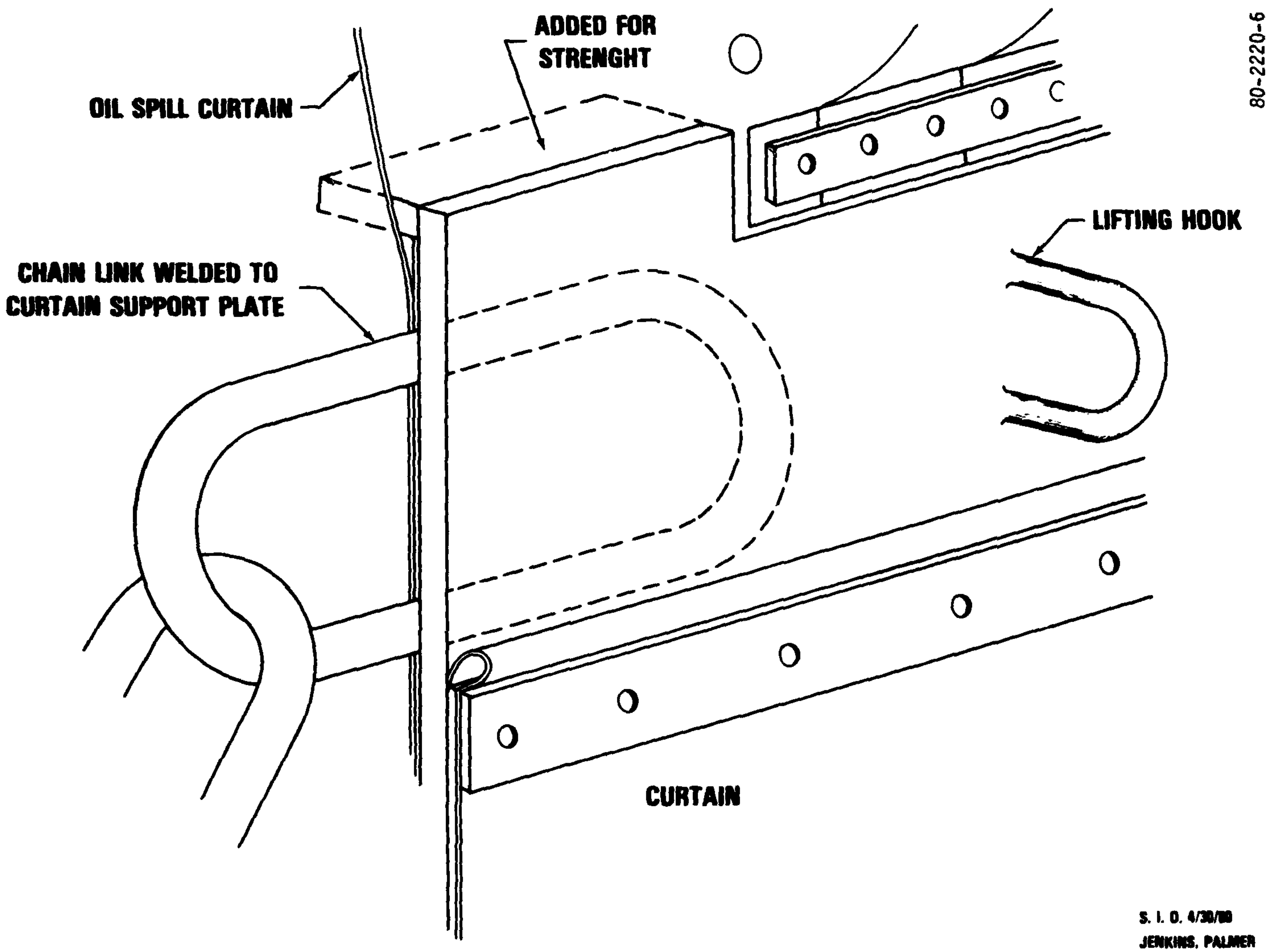

Figure 4-11. Joint Detail Between Unit Curtain Section

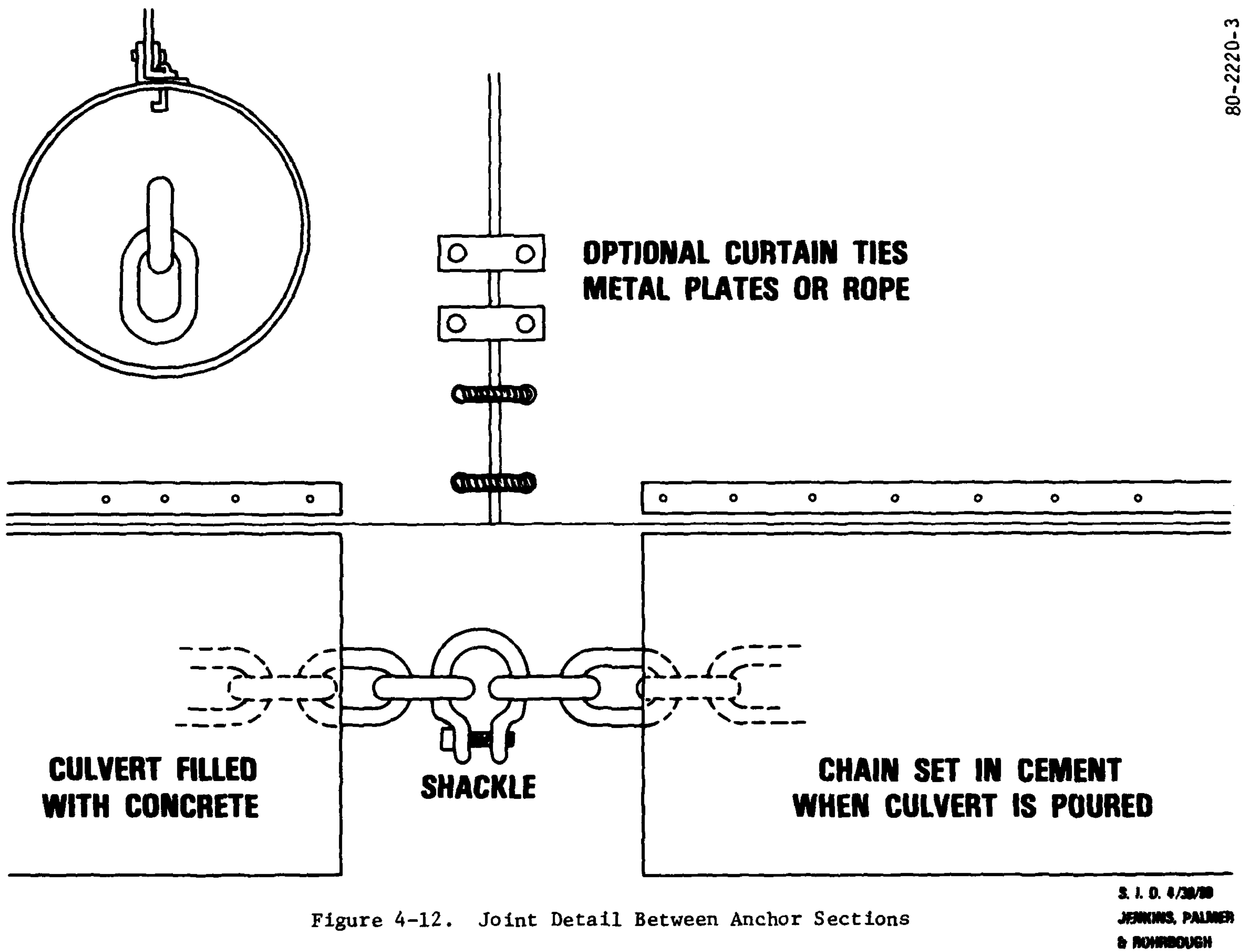

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Figure 4-12. Joint Detail Between Anchor Sections

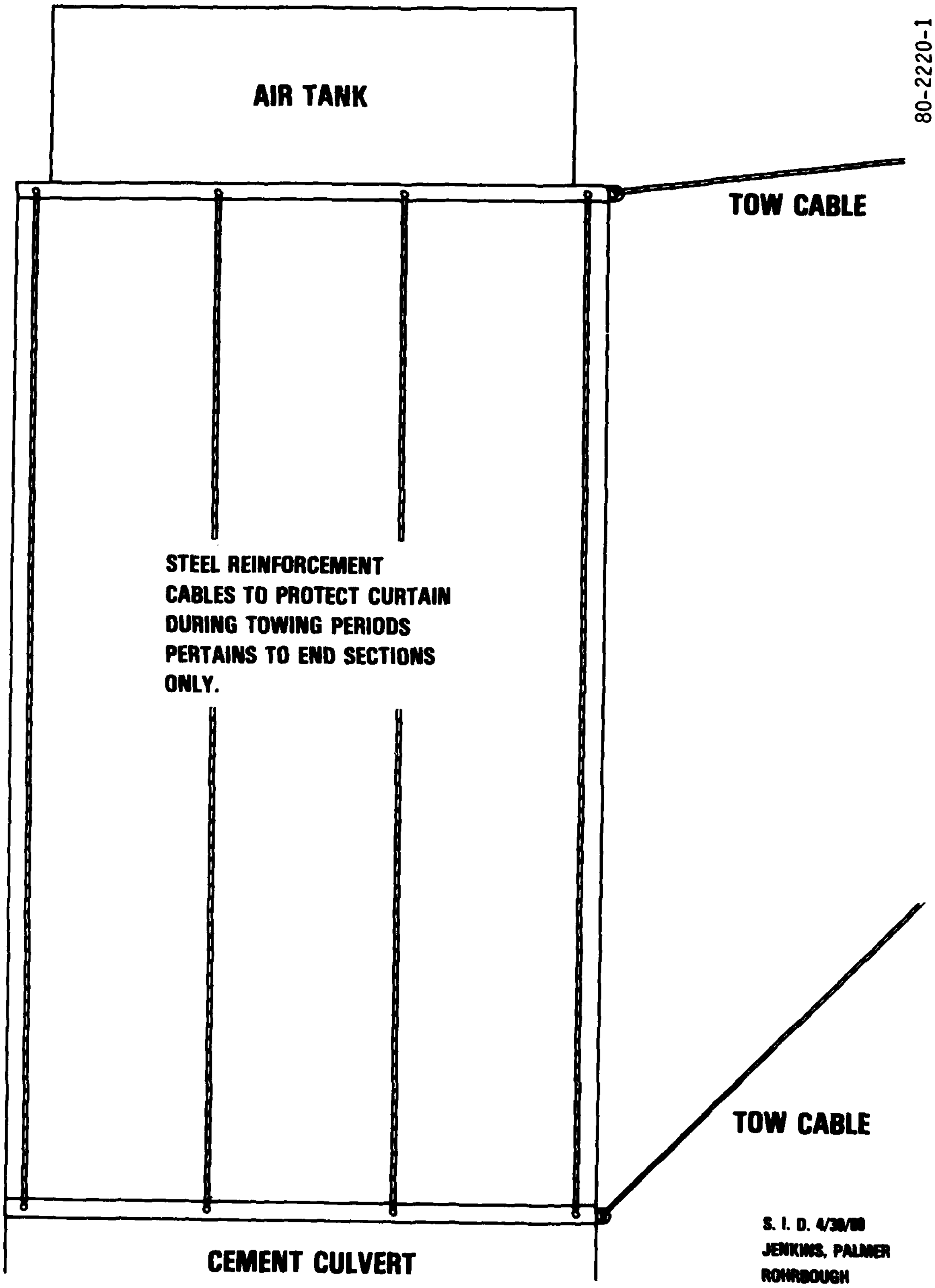


Figure 4-13. End Section Reinforcement and Towline Bridle

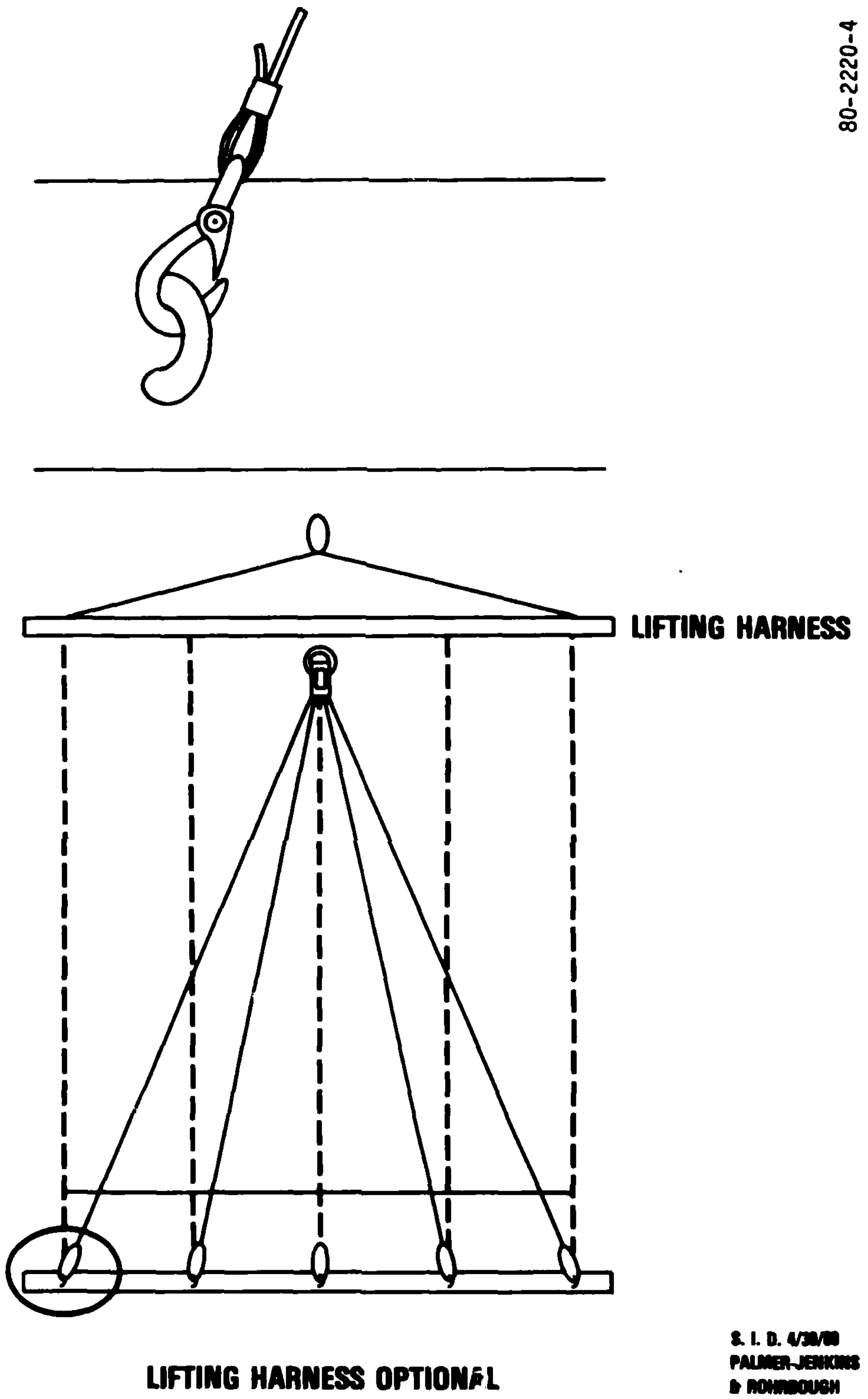


Figure 4-14. Lifting Harness for Dockside Rigging and Assembly

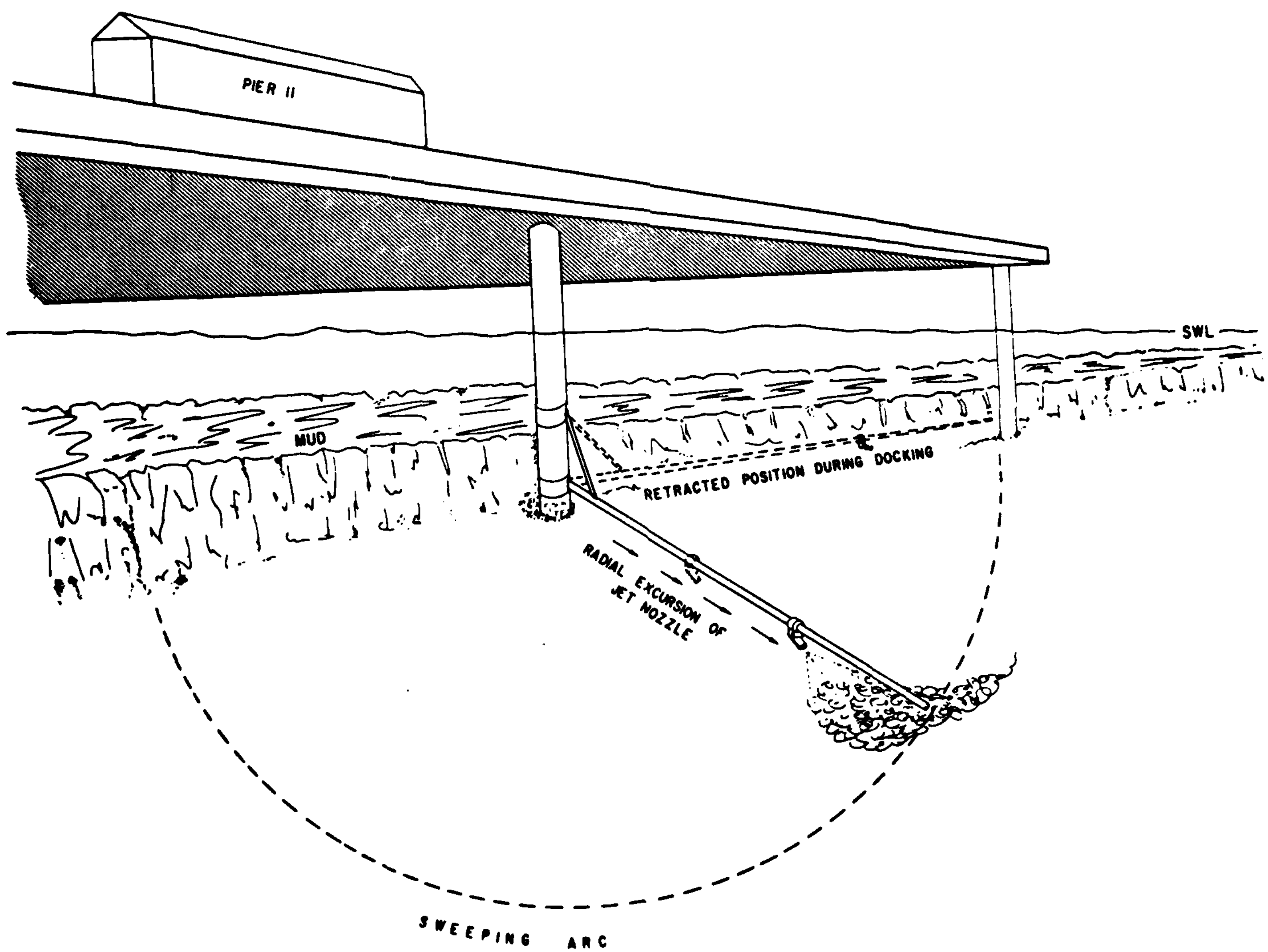
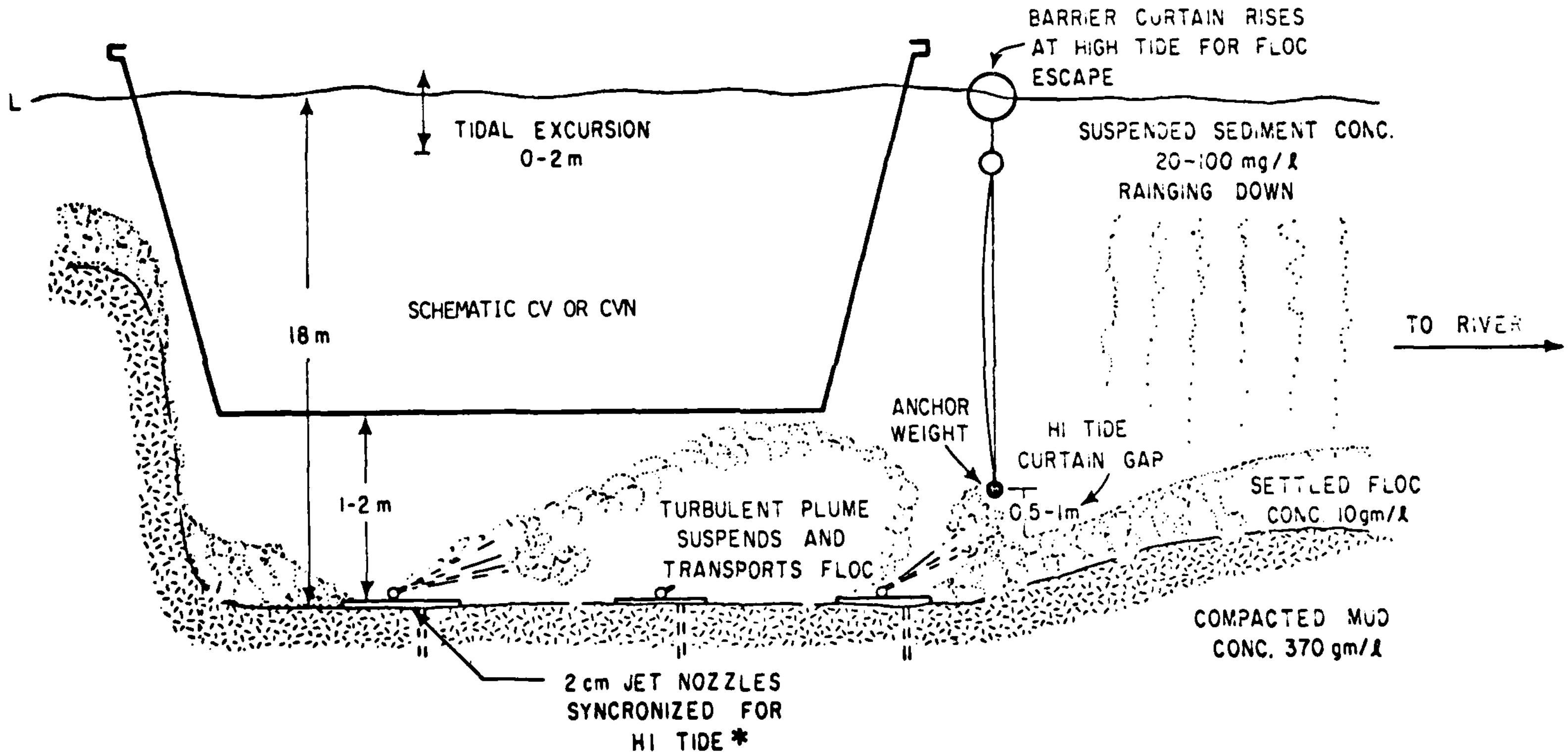
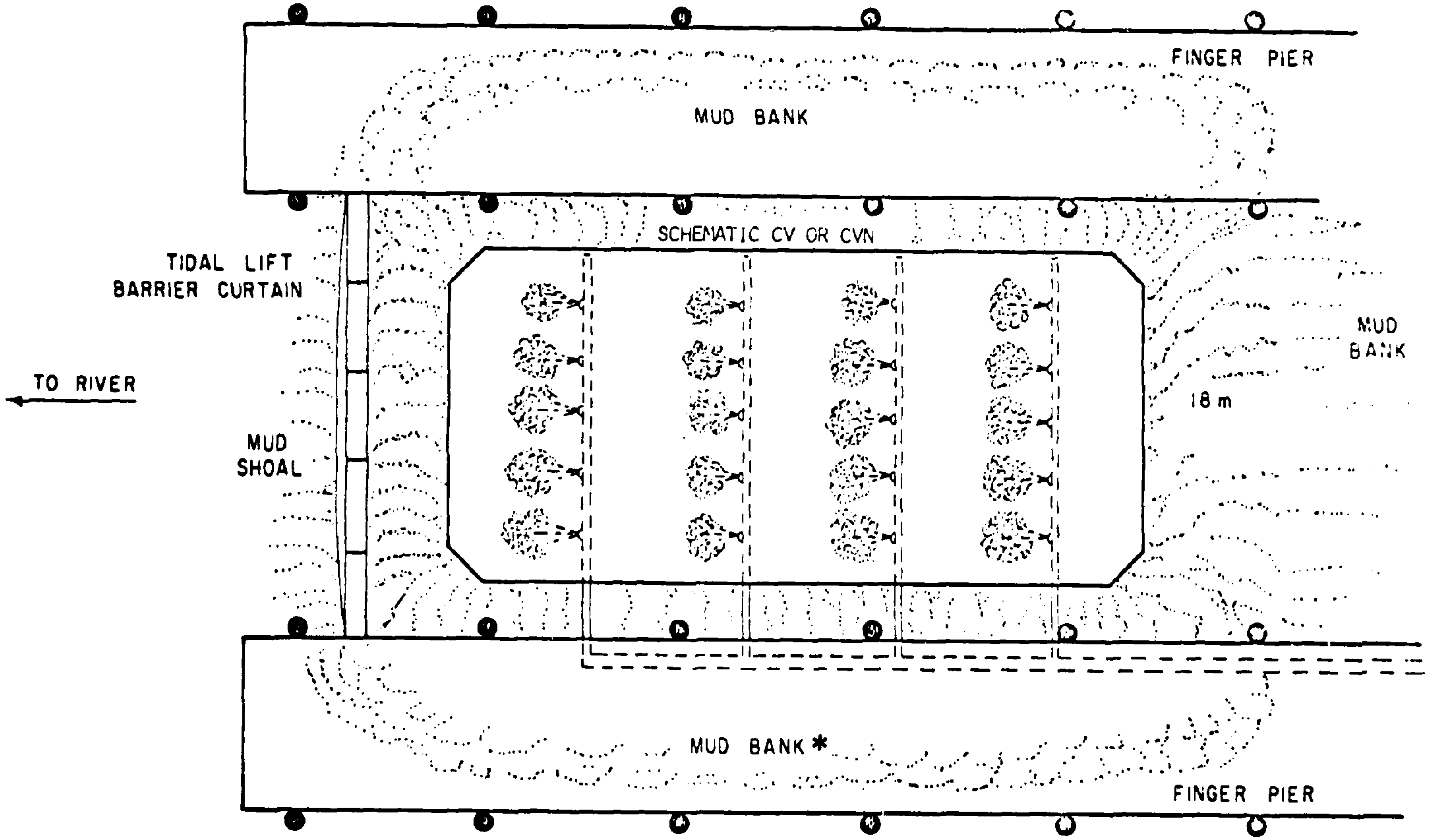


Figure 4-15. Mobile Sweeping Jet to Extend Effective Protection Radius



* (Water jets are actuated immediately after hi-tide to sweep fluid mud and organisms out under the tidal lift curtain.)

Figure 4-16. Schematic of a Water Jet Flushing System and Barrier Curtain To Protect a Carrier Dredge Hole



* The water jet flushing of the berth may be shut off and the curtain bottom fully lowered for long closure intervals.

Figure 4-17. Plan View of the Automatic Flushing Curtain System for a Carrier Dredge Hole (The mud banks provide the lateral seal to the berth up to mean lower low sea level. The water jet flushing of the berth may be shut off and the curtain bottom fully lowered for long closure intervals.)

SECTION 5

A DESIGN PROCEDURE FOR SCOUR JET ARRAYS

By

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INTRODUCTION

Navy harbors in the United States are frequently located in estuarine environments and are subject to rapid shoaling. Recognizing this problem, NAVFAC initiated a research program to develop a number of innovative sediment techniques to reduce future dredging costs to the Navy. Following a period of concept validation and experimental system development, three systems were found to be effective in reducing sedimentation in navigation and berthing areas. These systems are the crater-sink-fluidization sand bypassing system, the passive full-height barrier curtain, and the scour jet array. The purpose of this paper is to describe a rational design procedure for the latter system.

SYSTEM EQUATIONS

A scour jet array consists of a series of submerged water jets, which are positioned in front of a berth or quay wall (Figure 5-1). High pressure water is fed sequentially through each jet during ebb tide conditions, so that the scoured material is carried from the area. Laboratory experiments have shown that the shear stress distribution associated with a horizontal wall jet may be described in terms of the jet diameter and the jet discharge velocity. Moreover, the physical characteristics of a wall jet are completely described by four coupled equations with five unknowns (Figure 5-2). It follows that for a given shear stress distribution, a multitude of system states are possible. Each system state leads to different system designs, each with differing capital costs and energy costs. What is needed is a rational procedure for selecting an optimal system from among all candidates.

CONCEPTUAL JET ARRAY

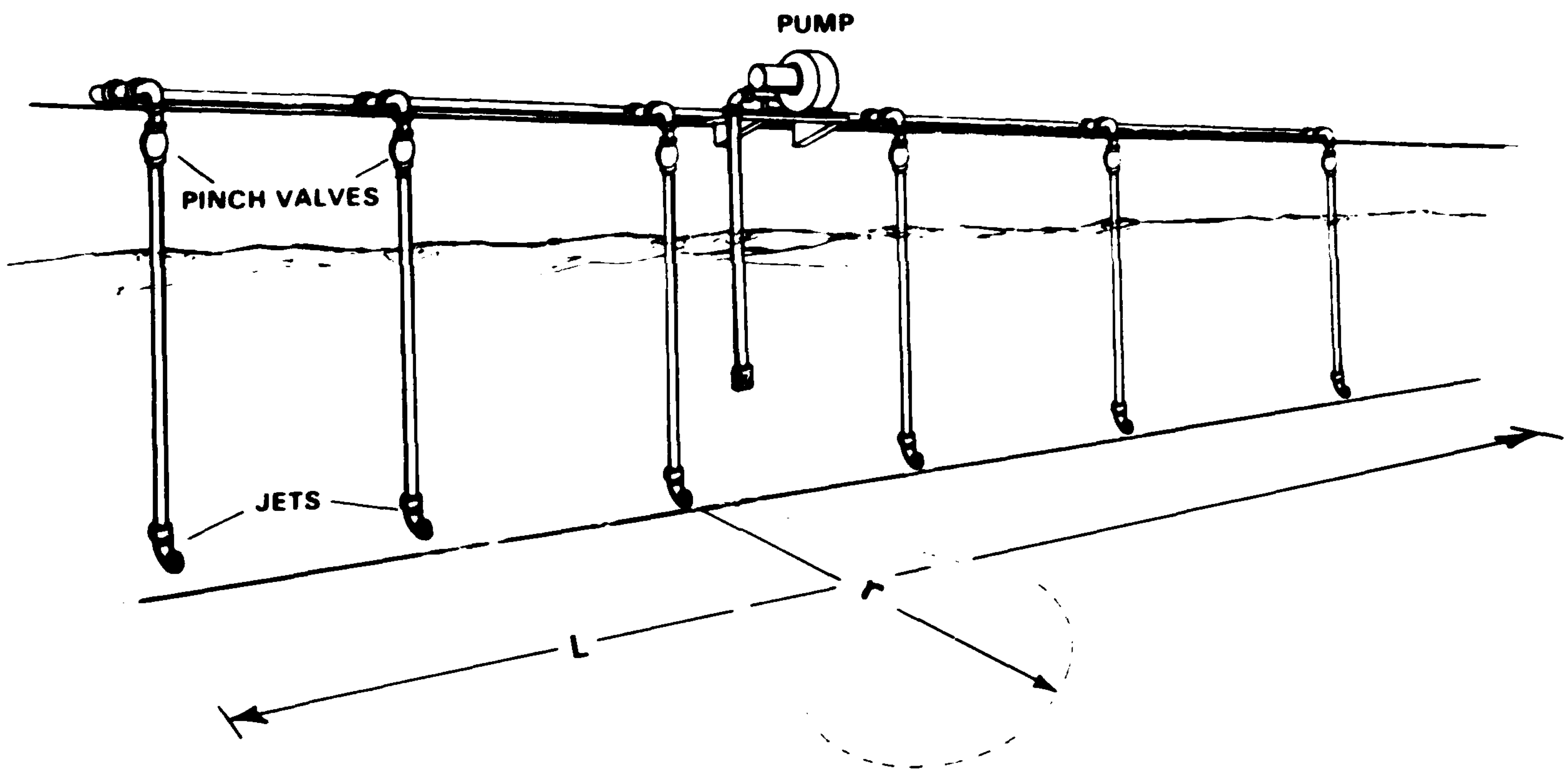

Figure 5-1. Conceptual Sketch of a Scouring Jet Array

GOVERNING PHYSICAL EQUATIONS

$$\frac{\tau_o}{\rho} = 120\, u_o^2 \left(\frac{u_o d}{\nu}\right)^{.4} \left(\frac{r}{d}\right)^{-2.4}$$

$$u_o = \sqrt{\frac{2\,\Delta p}{\rho}}$$

$$q = \frac{\pi d^2}{4}\, u_o$$

$$w = \Delta pq + \text{SYSTEM LOSSES}$$

GOVERNING ECONOMIC EQUATIONS

$$\$\ \text{COST} = \text{CAPITAL COST} + \text{ENERGY COST}$$

$$C_{ANNUAL} = \frac{C_{CAP}}{pwf} + C_{ENERGY}$$

$$\text{WHERE PWF} = \sum_{n} \left(\frac{1}{1+i}\right)^{n}$$

$$\text{SAVINGS INVESTMENT RATIO} = \frac{pwf(C_{DREDGING} - C_{ENERGY})}{C_{CAPITAL}}$$

$$\text{PAYBACK PERIOD} : \sum_{n'} \left(\frac{1}{1+i}\right)^{n'} = \frac{pwf}{SIR}$$

Figure 5-2. Governing Physical and Economic Equations for a Scouring Jet Array

For present purposes, an optimal system is defined as the system that has the lowest annual cost. More generally, for a given site, the optimal system will have the highest Savings Investment Ratio (SIR) (Figure 5-2).

OPTIMAL SYSTEM SELECTION

The above selection procedure has been incorporated into a computer code requiring specific site and system inputs (Figure 5-3). Site inputs include: array length; jet scour radius; scour shear stress; period of sedimentation; power costs; dredging costs; the required rate of return on the project; and the differential inflation rate for electricity. The system inputs include: pipe water velocity; types and costs of materials for the pipe, pump, and valves; the duty cycle time for each jet; and the system lifetime.

As an alternative to using the above described computer code, a series of graphs were developed which may be used to select a near optimal scour jet system. The figures were prepared by assuming cast iron pump and valves, steel pipe, and installed costs for these items as shown in Figures 5-4 and 5-5. In addition, it was assumed that the shear stress required to scour material is 4 $dynes/cm^2$, the jet duty cycle is 12 minutes, and the maximum water velocity in the pipes is 2.5m/sec. Finally, it was assumed that the rate of return on the project exceeds inflation by 10 percent, the differential rate of inflation of electricity is 3 percent, and the life time of the system is 10 years. Figures 5-6 through 5-9 show the savings investment ratio as a function of the array length and scour radius, for yearly dredging costs of \$4, \$8, \$12, and \$16 per square meter of bottom. As expected, all systems become more attractive (larger SIR) as the dredging costs increase. Moreover, for these general conditions, the jet scour array does not become economically attractive until dredging costs approach $\$8/m^2$. On the other hand, secondary benefits such as a reduced need for divers and/or dredging related ship movements would increase the attractiveness of such systems. The payback time for a particular SIR may be obtained from Figure 5-10. After selecting a particular array length and scour radius, Figures 5-11 and 5-12 may be used to estimate the annual and capital costs for an optimal system. Similarly, the optimal jet

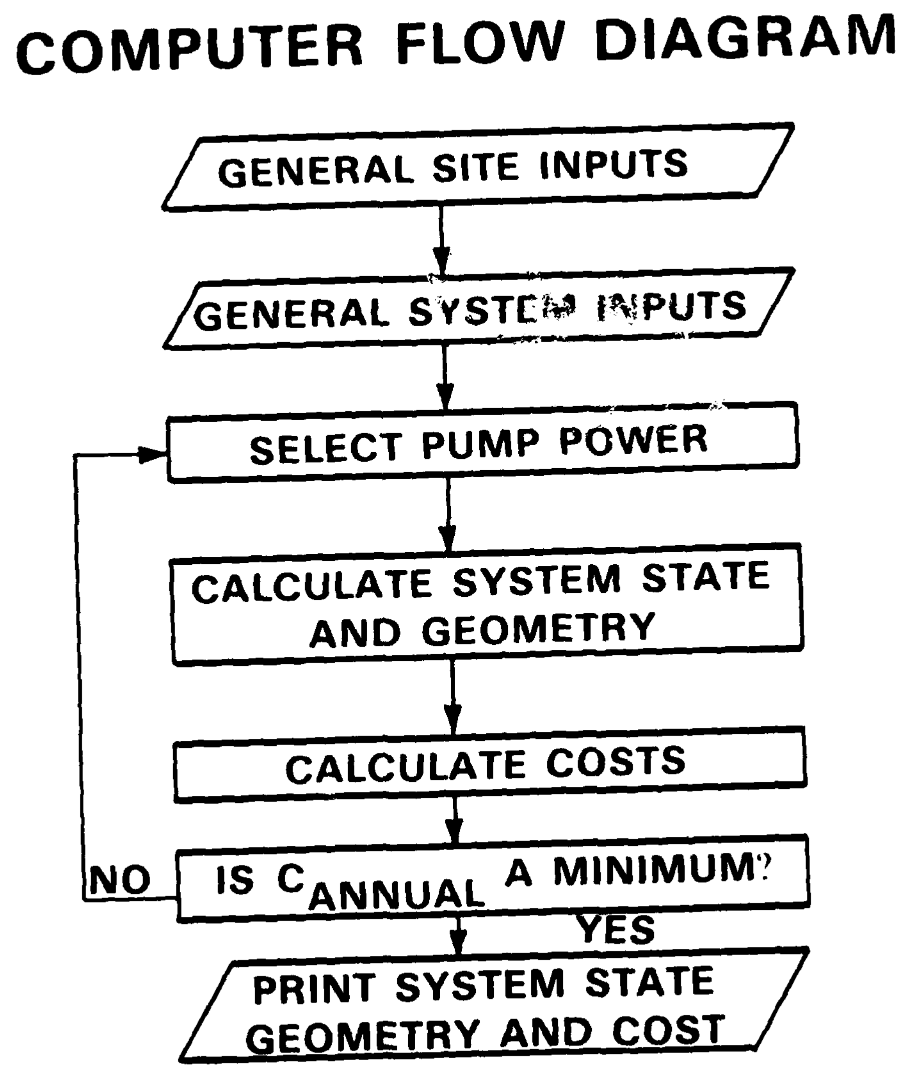

Figure 5-3. Flow Diagram for Optimization Computer Code

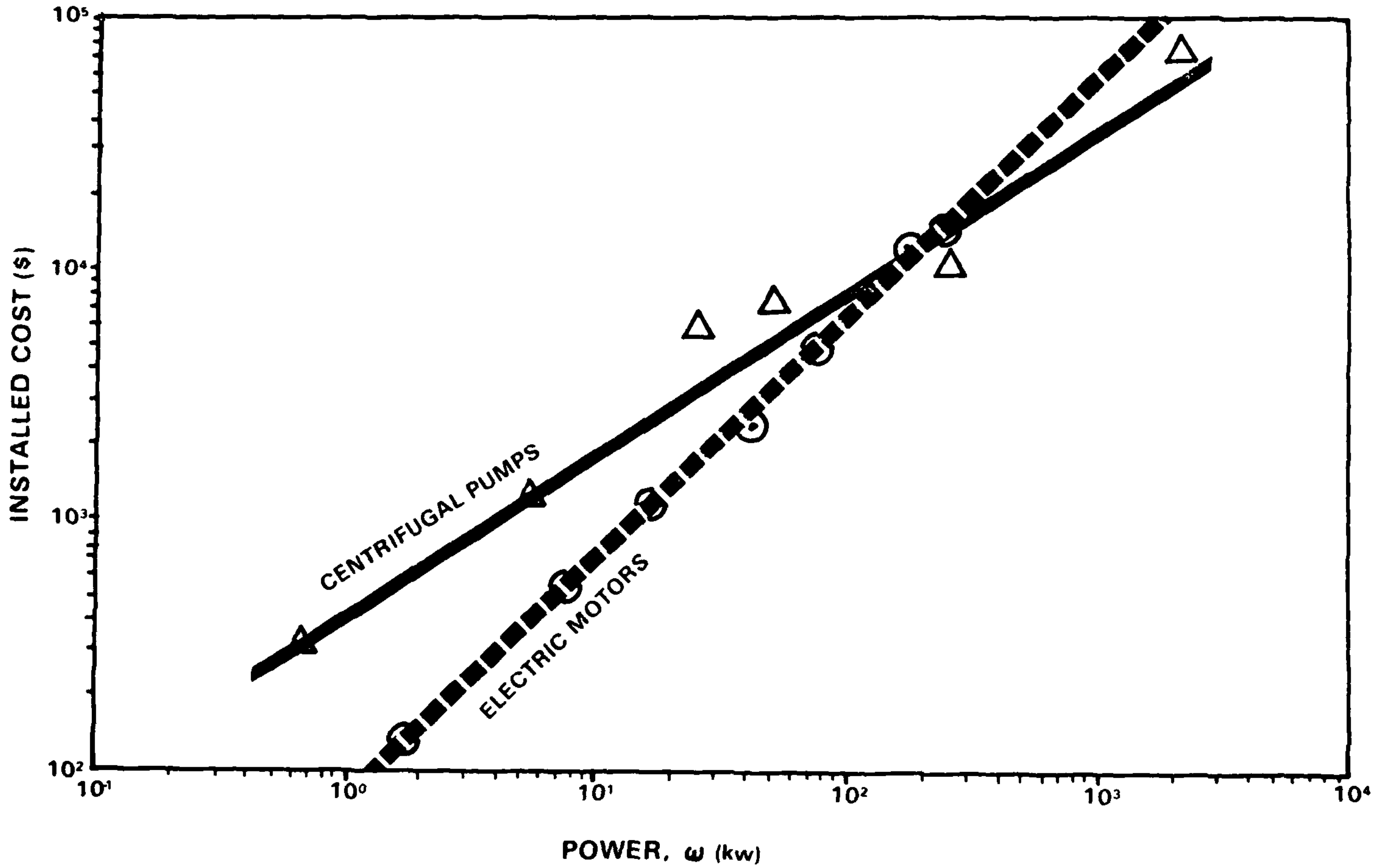

Figure 5-4. Installed Cost of Centrifugal Pumps and Electric Motors as a Function of Power

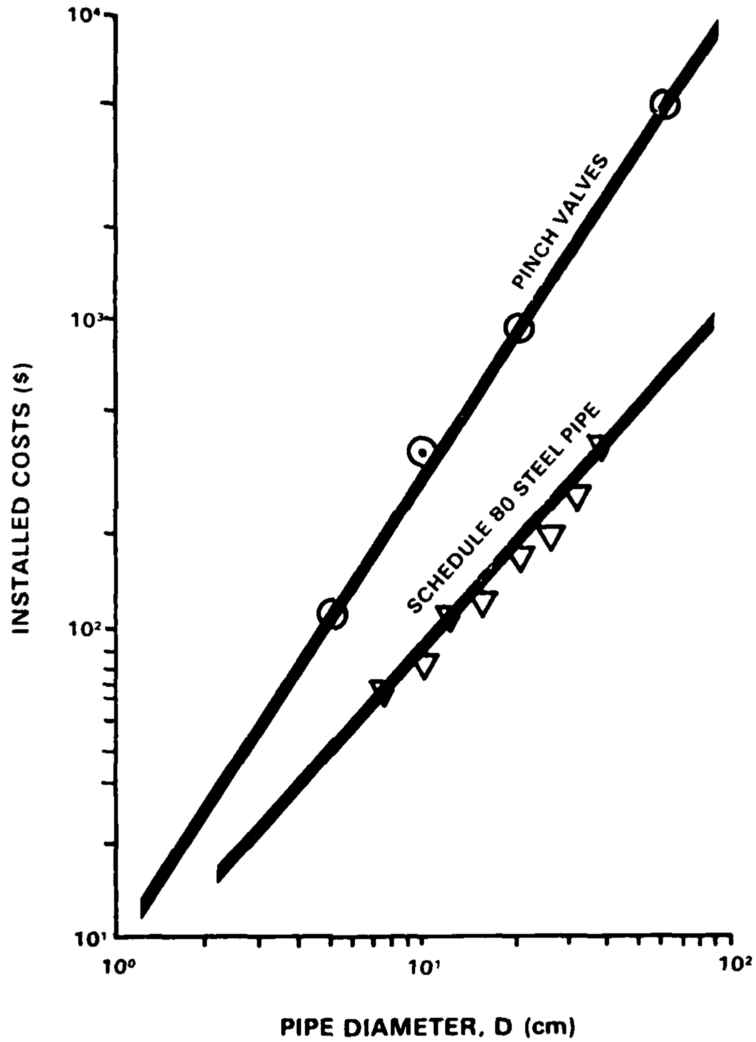

Figure 5-5. Installed Cost of Steel Pipe and Cast Iron Pinch Valves as a Function of Pipe Diameter

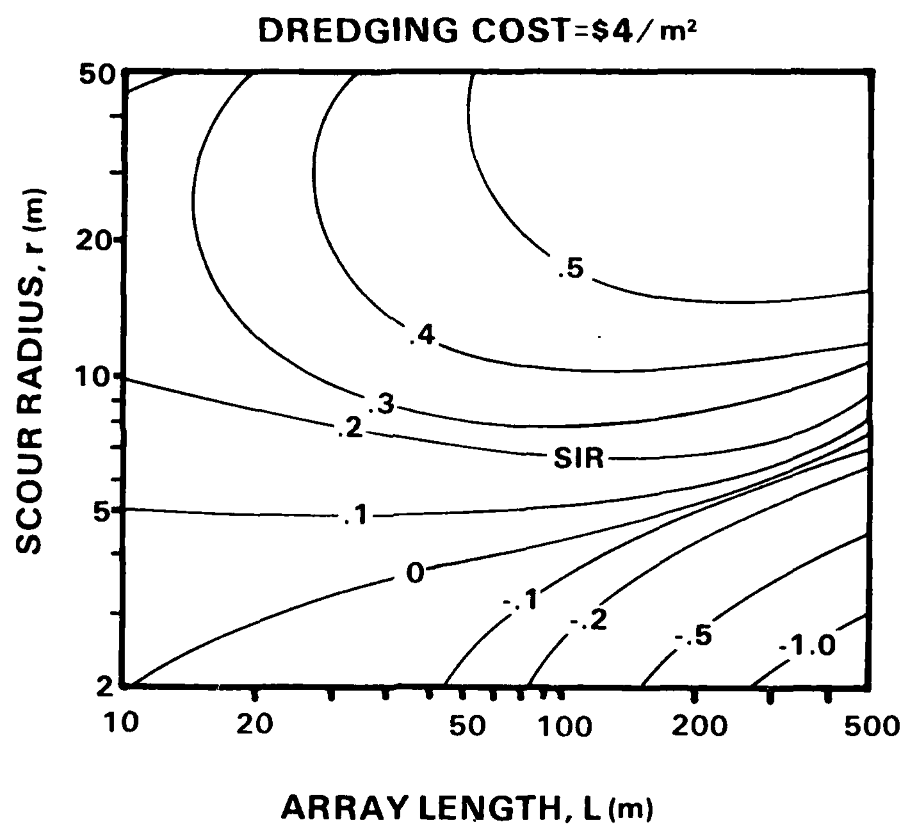

Figure 5-6. Savings Investment Ratio as a Function of Scour Radius and Array Length Assuming a Dredging Cost of $\$4/m^2$

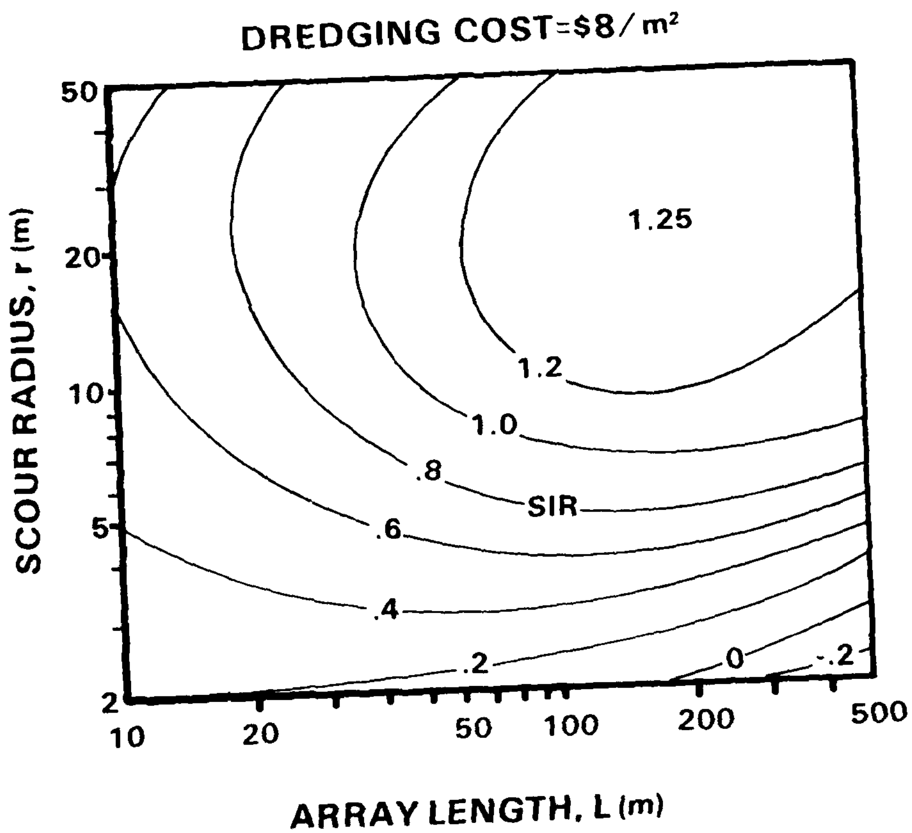

Figure 5-7. Savings Investment Ratio as a Function of Scour Radius and Array Length Assuming a Dredging Cost of $\$8/m^2$

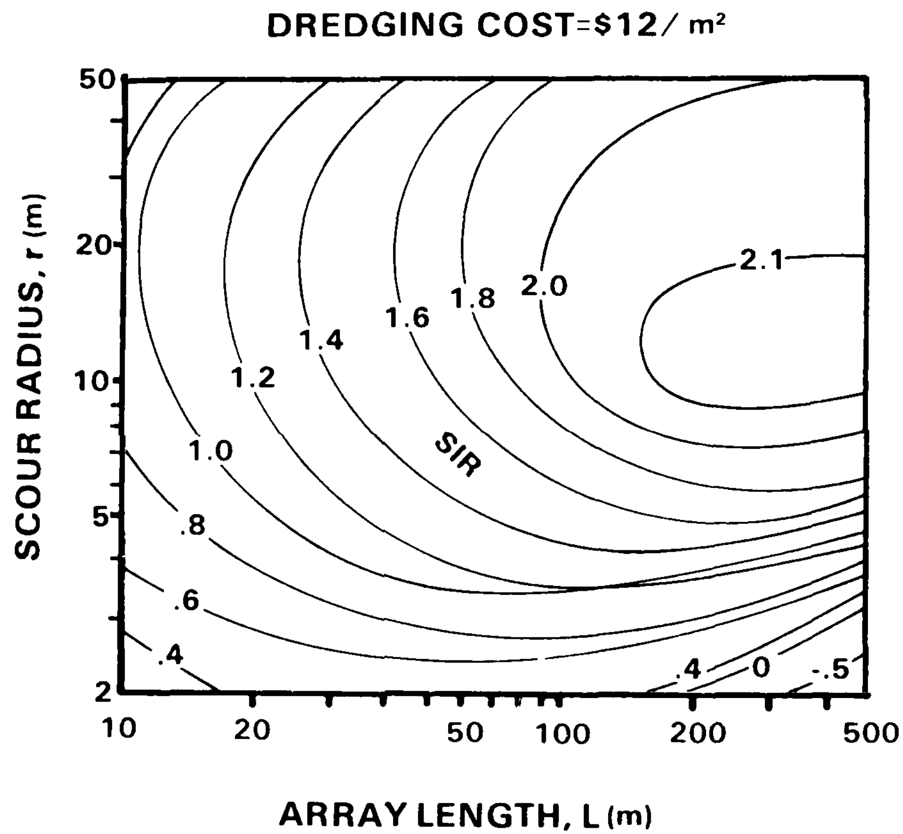

Figure 5-8. Savings Investment Ratio as a Function of Scour Radius and Array Length Assuming A Dredging Cost of $\$12/m^2$

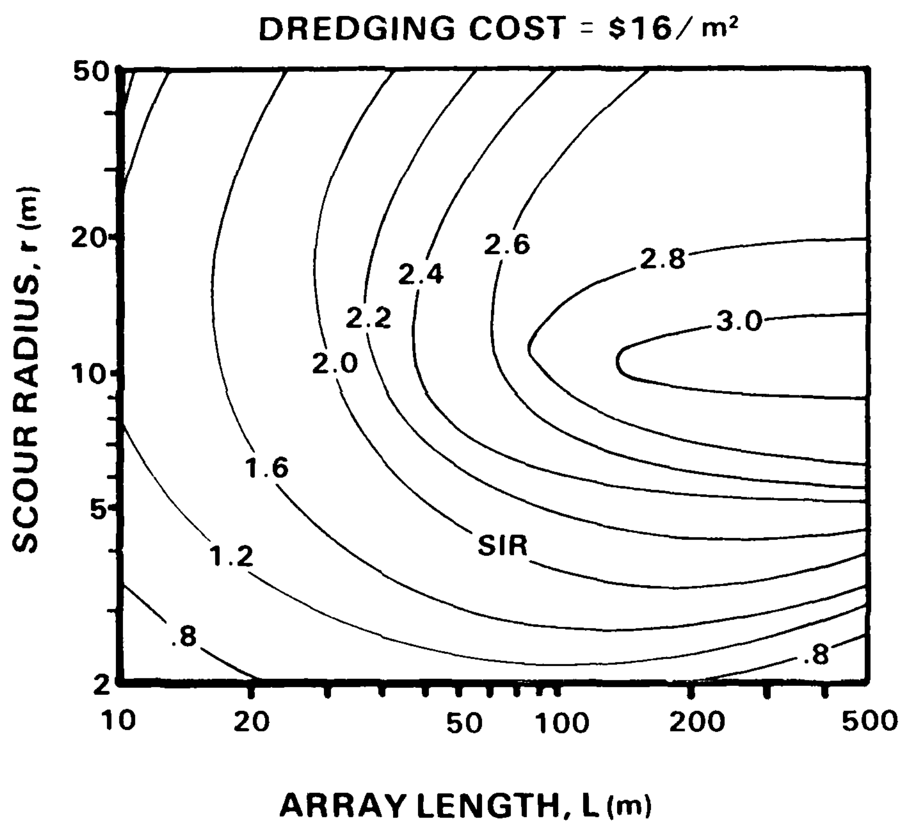

Figure 5-9. Savings Investment Ratio as a Function of Scour Radius and Array Length Assuming a Dredging Cost of $\$16/m^2$

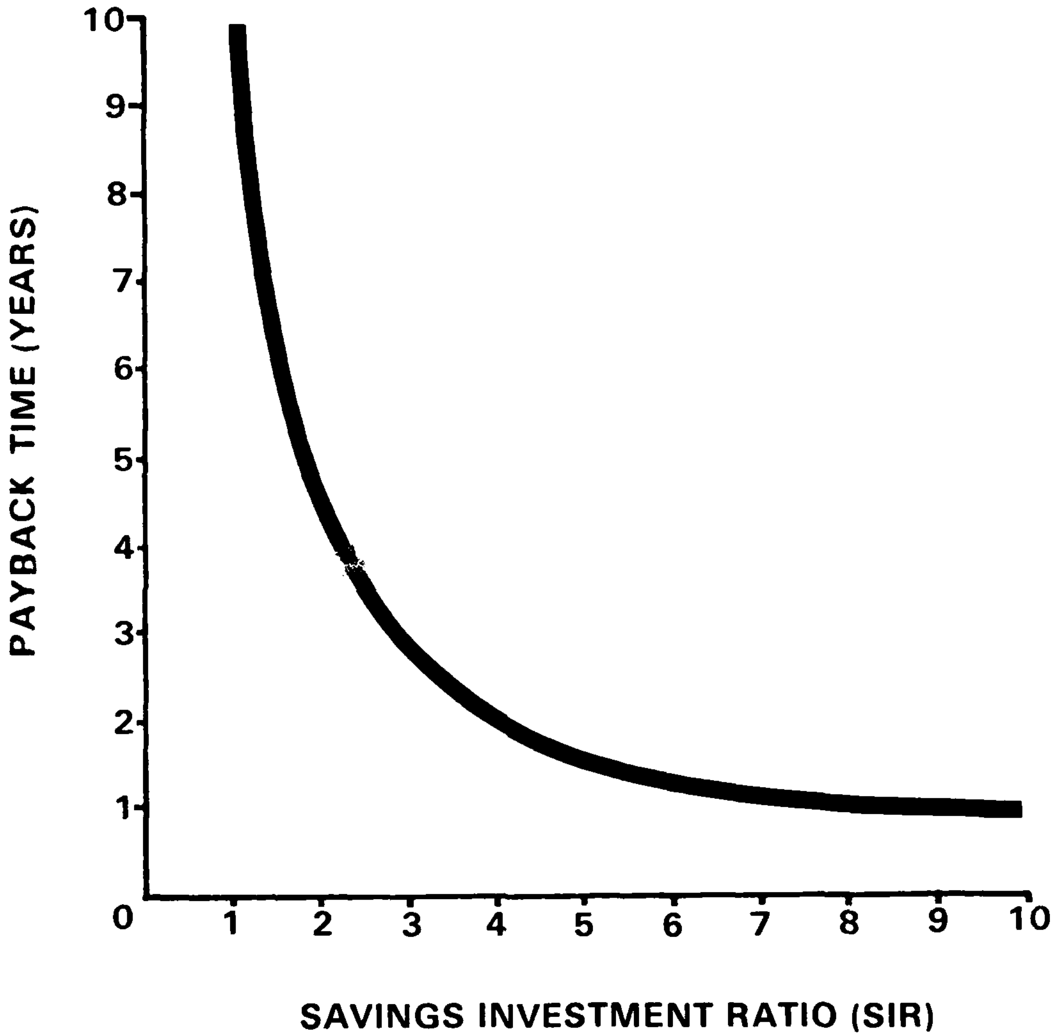

Figure 5-10. Payback Time Versus Savings Investment Ratio Assuming a 10 Year Lifetime, a Rate of Return of 10% and a Differential Inflation Rate of 3% for electricity

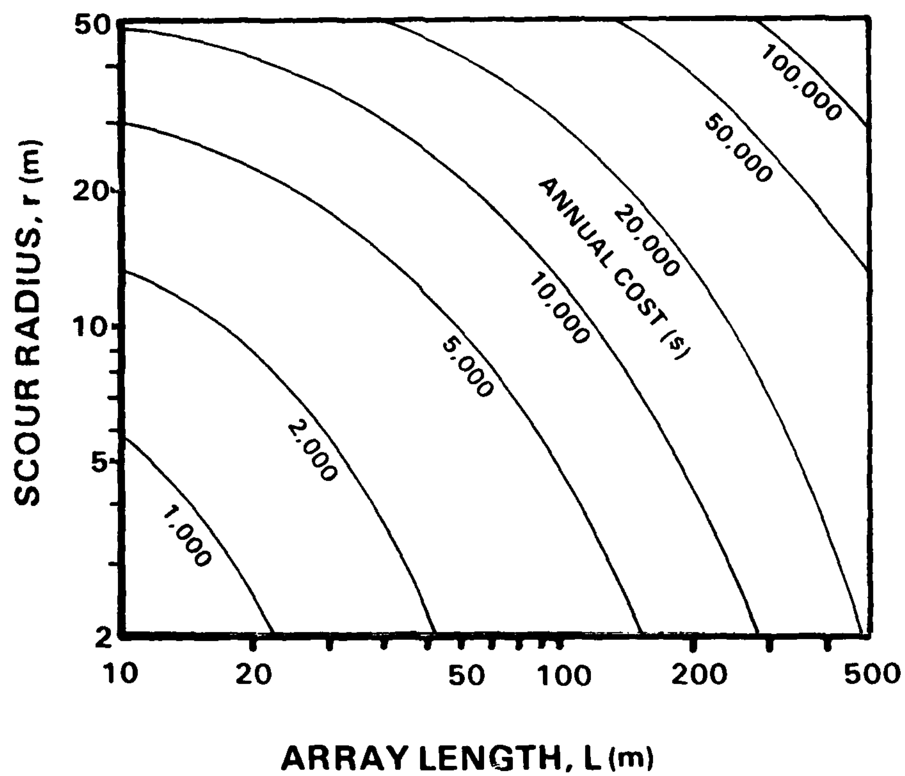

Figure 5-11. Annual Cost Versus Scour Radius and Array Length for Optimal Jet Arrays

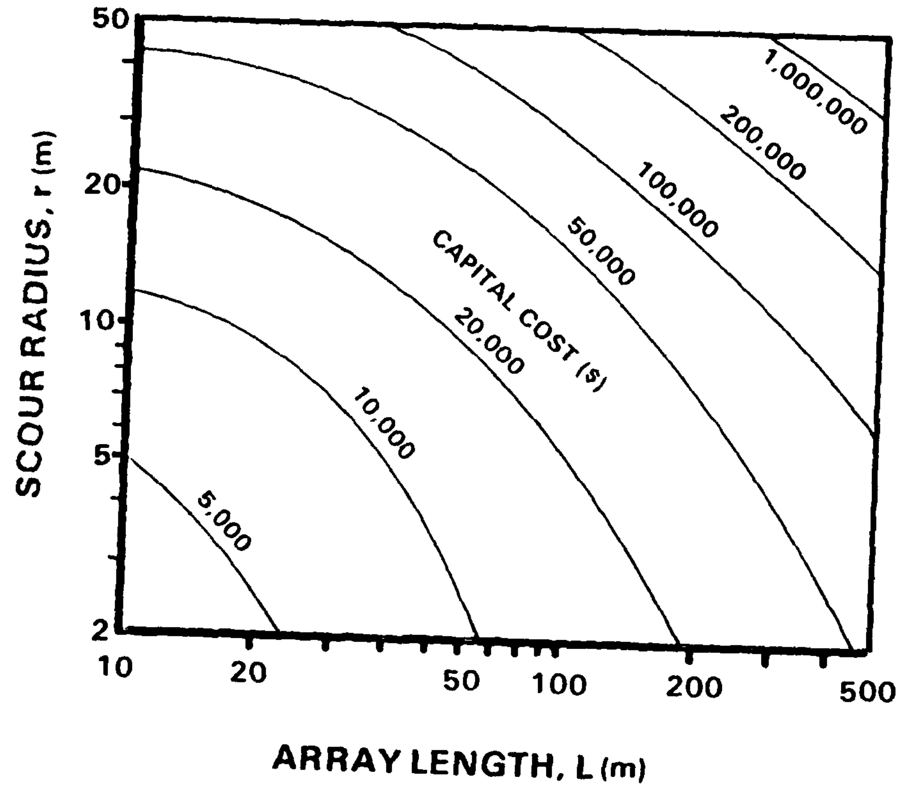

Figure 5-12. Capital Cost Versus Scour Radius And Array Length for Optimal Jet Arrays

diameter and pipe diameter may be determined from Figures 5-13 and 5-14, respectively. Finally, the total system state and the size of the required pump may be determined from the system equations given in Figure 5-15.

MINSY QUAY WALL DESIGN

As a specific example, a 760m long jet array was designed for Mare Island Naval Shipyard. The specific inputs to the computer code are shown in Figure 5-15 along with the computer output. The results suggest that the quay wall should be protected by six subsystems, each costing $63,000 with annual costs of $11,500 and an SIR of 2.46. The payback time would be 3.3 years. An actual cost-out of the system resulted in a capital cost of $59,500, and annual cost of $10,584, an SIR of 2.57 and a payback time of 3.2 years.

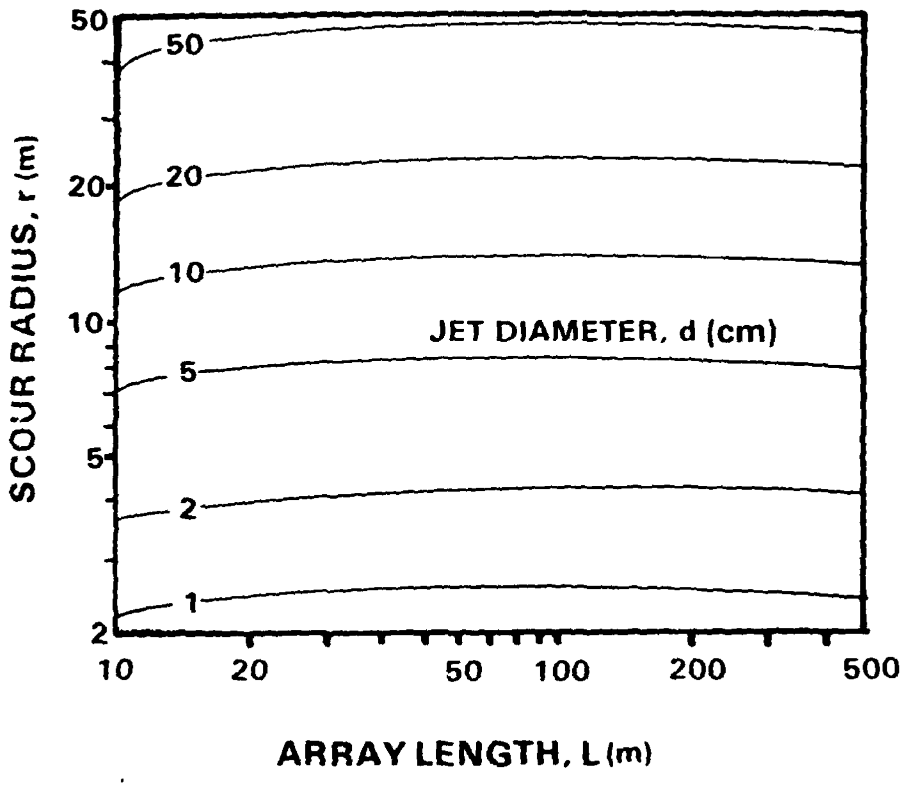

Figure 5-13. Jet Diameter Versus Scour Radius and Array Length for Optimal Jet Arrays

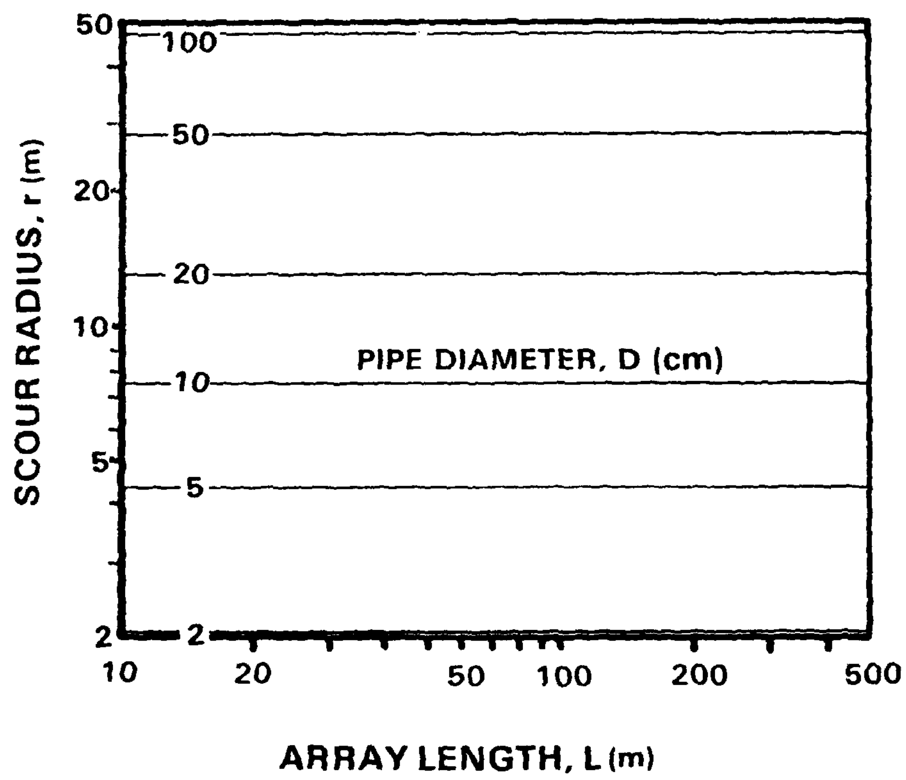

Figure 5-14. Pipe Diameter Versus Scour Radius and Array Length for Optimal Jet Arrays

SITE INPUTS

$L = 760$ m

$r = 15$ m

$\tau_o = 4$ dynes/cm²

SEDIMENTATION PERIOD	= 244 days/year
EBB TIDAL PERIOD	= 6 HR $\longrightarrow$ 28 JETS, 4.5m APART FOR 1 SUBSYSTEM; 6 SUBSYSTEMS
ELECTRIC COST	= \$.025/kw-hr
DREDGING COST	= \$12/m^2
INTEREST RATE	= 10%
DIFFERENTIAL ENERGY INFLATION	= 3%

SYSTEMS INPUTS

$U_p = 2.5$ m/sec

SCHEDULE 80 STEEL PIPE

CAST IRON PINCH VALVES

CAST IRON PUMP

ELECTRIC MOTOR

10 YEAR LIFETIME

SUBSYSTEM COMPUTER OUTPUT

C_{ANNUAL} = \$11,409

$C_{CAPITAL}$ = \$64,110

C_{ENERGY} = \$854

SIR = 2.46

PAYBACK PERIOD = 3.3 YEARS

$d = 9.88$ cm

$D = 22.1 \text{ cm} \simeq 8$ inches

$q = .0959 m^3/\text{sec} \simeq 1500$ gpm

$p = 1.30 \times 10^5 \text{ nt}/m^2 \simeq 19$ psi

$w = 12.5 \text{ kw} \simeq 17$ hp

Figure 5-15. Site and System Inputs and Resulting Computer Output for the MINSY Quay Wall Design

SECTION 6

SEDIMENT TRANSPORT AND SHOALING IN ESTUARINE AND OTHER SHALLOW-WATER AREAS

By

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INTRODUCTION

Sedimentation in estuaries and other shallow-water areas has been intensively studied for many years and the findings now comprise a literature of formidable volume and scope. Major subdivisions of the subject field include: (1) sediment texture, structure, and composition (e.g., Klein, 1970, 1972, 1977); (2) sediment sources and rates of deposition (e.g., Meade, 1972); (3) depositional/ erosional patterns, large-scale geomorphic forms, and bed forms (e.g., Hayes, 1973); (4) sediment transport processes and their mechanics in inlets, tidal flats, and channels (e.g., Ludwick, 1972, 1974, 1975a, b); (5) waves, tides, tidal currents, seiches, and estuarine circulation (e.g., Officer, 1976); (6) sediment water chemical interactions, pollution criteria for sediments (e.g., Lu and Chen, 1977); (7) sediment-biota interactions (e.g., Lauff, 1967); and (8) marinas, ports, and harbors, the environmental effects of dredging, and the disposal of dredged material (e.g., Bruun, 1973).

Sediment-moving forces acting on the bed of many estuaries or other shallow water areas are of sufficient strength to destroy or shift a pattern of shoals and channels in the course of a few days, weeks, or months; and yet many large-scale estuarine bed features either shift slowly or exhibit relative positional permanence over years or decades. To understand the past evolution and possible future development of shoaling under reversing tidal flow, one must establish by measurement the flow conditions under which the existing quasi-equilibrium is established and the departures from that equilibrium. It is the departures that determine the rates of geometric change in shoaling.

When an estuarine channel or sand body is not in equilibrium and manifests this by a gradual change in position or geometry, it is commonly observed that changes appear to be connected with local deceleration or acceleration in sediment transport rate along streamlines of the near-bed flow. Deposition or erosion can occur during ebb or flood and during all or only a part of the two phases.

PRESENT STATE OF KNOWLEDGE

In estuaries and other shallow-water areas, sediment shoals and the associated channels are known to change size, shape, and location with time. In Great Britain, particularly, with its sandy estuaries and coasts and shallow, sediment-clogged port approaches, there is a long history of concern with the shifting of sand banks and channels, especially when these latter are used by shipping. A.H.W. Robinson (1956, 1960, 1963, 1975) has deduced the existence of a 40 year, cyclical pattern of sand shoal migration across the mouth of an estuary at Teignmouth, Devonshire. Kestner (1970) has charted successive positions of bars and channels since 1845 in Morecambe Bay, Lancashire. Studies of this kind are essentially historical in method of attack and rely almost entirely on old records, charts, or series of aerial photographs. Despite their considerable merit, they tend to suffer from lack of inquiry into the root causes of the observed movements. The descriptive character of the work contains essentially no quantification of the associated fluid motions and certainly no attempted explanations that stem from mechanics of sediment transport.

A nearly separate set of scientists and engineers have been concerned with the mechanics of sediment transport. Usually, the experimental work is performed in laboratory flumes and wave tanks where conditions can be controlled. The scales of the phenomena under study are necessarily limited by the size of the available facilities. The flume and wave tank scientists have dealt primarily with grain processes or the mechanics of small bedforms and not with the evolution of large geomorphic forms, and particularly not with those subject to reversing tidal flow. Nevertheless, some of the most valuable and seminal results have originated from flume work (Shields, 1936; Schlichting, 1968; Simons et al., 1965; Einstein, 1942, 1950; Yalin, 1977; Raudkivi, 1976).

Intermediate to the method of historical study and to the method of theoretical analysis is the approach using movable-bed hydraulic models (e.g., Harleman, 1971). The wide use of these models for engineering study of estuarine bed motion has occurred because of deficiencies in the other available methods of study. In the purely historical approach, one must often assume that what has happened in the past will continue at the same rate in the future. On the other hand, a purely theoretical calculation of expected bed change in an area following a new constructional work has been deemed to be beyond the capabilities of present knowledge.

It is well known in the technology of movable-bed hydraulic modeling that the sediment particles cannot be scaled nor can the fluid turbulence be modeled. Horizontal and vertical scales are usually unequal in the model; i.e., bed-form slopes are distorted. Despite these unavoidable shortcomings, hydraulic models have been very successful for evaluating expected change in estuarine shoals and channels, particularly when a model can be verified with adequate historical data. Nevertheless, the approach is essentially ad hoc and heuristic and does not necessarily seek an advance of understanding of the processes involved.

There is yet another group of investigators who have worked chiefly from boats or on wet ground on the problems of large bed forms, tidal shoals and channels, and their movements. Characteristically, the members of this group have made measurements of fluid flow, bed forms and bed form migration vectors, directional bedding structures, sediment texture, and tracer dispersion patterns (van Veen, 1950; Klein, 1970, 1972, 1977; Lavelle et al., 1976 Coleman et al., 1971; Hayes, 1973; McCave, 1971; Smith, 1968, 1970; Stride, 1974; Swift, 1975).

One of the significant concepts that has issued from all the above work is that of flow dominance, or net flow. One actually sees and can measure ebb currents acting and transporting sediment or flood currents acting and transporting sediment, but the locations and shapes of shoals and channels come into understandable correspondence, not with these observable flows, but with the net flow or net bed sediment transport. The spatial arrangement of ebb-dominated and flood-dominated pathways has given rise to the twin notions of mutual evasion and alternation of E-dominated or F-dominated channels. A further extension of the concepts leads to the postulation of closed circulation cells in the net flow superimposed approximately on shoals. Major channels of estuaries may exhibit E-dominance on one side of the channel bottom and F-dominance on the other. Certain shoal-channel patterns are ubiquitous: the apple-tree, espalied, and parabolic patterns of van Veen; the ebb delta and flood deltas of Hayes; the cigar-shaped shoals of Smith and Stride; the double-parabolic shoals of Granat; and the zig-zag shoals of Ludwick.

A second concept of fundamental importance to the study of bed erosion and deposition is embodied in the sediment transport continuity equation (see Appendix) which can be written as

$$\frac{\partial \eta}{\partial t} = -\frac{1}{\varepsilon}\left(\frac{\partial V}{\partial t} + \nabla\cdot\vec{Q}\right) \quad , \tag{1}$$

where η is the elevation of the erodible bed above a deeper arbitrary datum plane, ε is bulk density of the bed sediment, V is the mass of sediment in suspension between the water surface and the bed per unit of horizontal area, t is time, and $\vec{Q}$ is the rate of sediment transport in units of mass per unit path width per unit of time. If, as an approximation, $\frac{\partial V}{\partial t}$ is small compared to $\nabla\cdot\vec{Q}$, or can be neglected later on because of its periodicity, and if the sediment referred to in $\vec{Q}$ is concentrated near the bed, then one may write

$$\frac{\partial \eta}{\partial t} = -\frac{1}{\varepsilon}\nabla\cdot\vec{Q} \quad , \tag{2}$$

and properly describe it as an equation of continuity for bed sediment transport. If the approximation is unjustified, the entire equation is retained.

Potentially, this equation provides the means of determining erosion or deposition rate at any number of points, or continuously, over an area if $\nabla \cdot \vec{Q}$, the divergence of the bed sediment transport, can be evaluated at each point, which is to say, if $\vec{Q}$ can be evaluated. This equation has had considerable usage beginning at least as early as 1922 with Exner in his theoretical study of sand wave migration. Many others since that tme have made use of the equation including Smith (1968, 1970), Brush (1965), Kennedy (1963), and Reynolds (1965).

Ludwick (1974, 1975) has previously attacked a portion of the above cited problem by calculating stream power, $\tau_0 u_{100}$, at 21 sites in the entrance to Chesapeake Bay from measured velocity profiles taken over complete tidal cycles. However, in this earlier work, $\vec{Q}$ was not calculated from $\tau_0 u_{100}$, but was assumed to be proportional to $\tau_0 u_{100}$. Therefore, what was mapped over the study area of estuarine shoals and channels was relative change.

In any quantitatively based study of sediment transport and deposition, use is required ultimately of sediment transport equations which can be coupled to the continuity equation (1). In the main sections of this paper that follow, four sediment transport formulations are described briefly. The sediment transport continuity equation because of its fundamental importance and because a full derivation is almost never given is presented in the Appendix.

THE SEDIMENT TRANSPORT EQUATIONS

Since the 1700's, equations for calculating sediment tranport rate from fluid flow characteristics have been under development by scientists and engineers. In addition to the primary literature sources, there are several excellent reviews of these developments (Vanoni, 1971; Graf, 1971; Raudkivi, 1967; Yalin, 1977). For the purposes of the present discussion, four formulations have been selected for presentation and evaluation. Those tested or calibrated against field data have been given preference over others which are purely formalistic. None of the four chosen is absolutely free from shortcomings.

In the process of selecting these particular formulations, care was taken to screen out any method that contained adjustable constants which can appear in sediment transport equations: those whose magnitude is known (albeit imperfectly in some instances) from laboratory, field, or theoretical work, and those that are wholly unknown. Formulations containing the latter kind of constant have been removed from consideration in this presentation.

The Shields (1936) Bed Load Transport Equation

The empirical equation of A. Shields (1936) pertains to a sediment bed on which the surface grains are in strong motion and hence no longer form a flat surface; but, instead are shaped into scales or bars of low height to length ratio. The formula is laboratory based and was developed using artificial sands of mixed particle diameters. Test sands ranged in mean diameter from 1.56 to 2.47 mm and from 1.05 to 4.25 gm/cm^3 in specific gravity. Shields regarded his equation as little more than "...an abbreviated form to express the influence of the various factors upon bed load movement." It has, nevertheless, received support from more recent studies, and it has a contemporary flavor and the implied physical reasoning is readily grasped.

The equation is

$$\frac{G}{Q} \cdot \frac{(\rho_s - \rho)g}{\rho g S} \approx 10\left[\frac{\tau_o - (\tau_o)_{cr}}{(\rho_s - \rho)gD}\right] \quad , \tag{3}$$

where G is the volume of solid mineral particles transported per unit path width per second, Q is the volume of water discharged per unit path width per second, ρ_s and ρ are solid and fluid densities, g is gravitational acceleration, S is the slope of the water surface, τ_o is the intensity of shear at the bed, $(\tau_o)_{cr}$ is the critical shear stress required for the initiation of motion of the bed sediment, and D is particle diameter.

The complete numerator on the left-hand side of the equation is the immersed weight sediment transport rate. In the denominator, $Q\rho g$ is the weight discharge rate of the transporting water. The right-hand side of the equation is the dimensionless excess shear stress.

Some substitutions in the equation produce new forms that bring out the essential features in modern terms. With the discharge equation,

$$Q = \bar{\mu}H \quad , \tag{4}$$

where $\bar{\mu}$ is the depth mean flow velocity and H is the depth of flow, the denominator on the left-hand side of the equation becomes

$$\bar{\mu}H\rho gS \quad . \tag{5}$$

The resistance equation for open channel flow is

$$\tau_o = \rho gHS \tag{6}$$

with symbols as defined above. Substituting in expression (5) yields

$$Q\rho gS = \tau_o\bar{\mu} \quad , \tag{7}$$

where the right-hand side is the "stream power" of Bagnold.

Converting G to units of mass/cm-sec

$$g_s = G\rho_s \quad , \tag{8}$$

the original equation becomes

$$g_s = 10\tau_o\bar{\mu} \frac{\rho_s}{(\rho_s - \rho)g} \left[\frac{\tau_o - (\tau_o)_{cr}}{(\rho_s - \rho)gD} \right] \tag{9}$$

This equation can be evaluated in a field program using (1) field measurements of τ_o and $\bar{\mu}$, (2) laboratory measurements of ρ_s and D, and (3) the shields diagram for $(\tau_o)_{cr}$. As explained in the pages that follow, τ_o is to be replaced by τ_o', the shear stress associated with grain roughness of the bed.

The Einstein (1942) and Einstein-Brown (1950) Equations for Bed Load Transport

The Einstein development of 1942 applies to sediment of uniform particle size or to "sediment mixtures acting like uniform sediment." In the derivation, expressions are obtained for two principal quantities: (1) the number of particles of stated size deposited per unit time per unit area of the bed; and (2) the number of particles eroded per unit time per unit of area of the bed. These two quantities are equated in obtaining the bed load formula. The first quantity is taken to be a function of the total probable travel distance of a saltating particle prior to rest, the bed load rate, and particle volume and density. The second quantity is a function of particle availability on the bed and of the turbulence intensity of the transporting fluid. This latter consideration manifests itself in the theory as the fluid. This latter consideration manifests itself in the theory as the "probability of removal" of a particle from the bed and the "time consumed by each exchange" of particles on the bed. This last notion is related intuitively by Einstein to particle diameter and settling velocity.

When the number of particles deposited (suitably expressed) is balanced against the number of particles eroded (suitably expressed), with total travel distance of a particle expressed probabilistically, the following equation results after some simplification:

$$\frac{p}{1-p} = A_* \cdot \Phi = A_* \cdot \frac{\text{(sediment transport rate)}}{(\rho_s - \rho)d_s \cdot \text{(settling velocity)}} , \qquad (10)$$

where p is the probability of particle removal from the bed, A_* is a dimensionless compound of constants evaluated experimentally by Einstein, and Φ, the intensity of bed load transport, is given by

$$\Phi = \frac{1}{F} \cdot \frac{g_s}{(\rho_s - \rho)} \sqrt{\frac{\rho}{(\rho_s - \rho)g d_s^3}} , \qquad (11)$$

where g_s is the mass rate of sediment transport per second per unit path width, ρ_s and ρ are solid and fluid density, F is a factor in the Rubey equation for settling velocity, g is gravitational acceleration, and d_s is particle diameter.

The variable p is also a function of the ratio of effective particle weight to hydrodynamic lift, and the latter is expressed using a coefficient of lift, the velocity of the fluid near the boundary, and other needed measures of gran geometry and mass. What devolves is the equation

$$p = B_* \cdot \Psi \quad , \tag{12}$$

where Ψ is the "flow intensity" of Einstein and B_* is a dimensionless compound of constants evaluated experimentally be Einstein. The factor Ψ is seen to be the reciprocal of the Shields parameter and is given by

$$\Psi = \frac{(\rho_s - \rho)gd_s}{\tau_o} \quad , \tag{13}$$

where τ_o is the boundary shear stress.

For weak or low rates of sediment transport, $\rho(1 - p)$ is approximately equal to p and

$$A_*\Phi = p = f(B_*\Psi) \tag{14}$$

which suggests the plotting of experimental data in ϕ vs. Ψ coordinates and the seeking of an organized pattern that can be fitted with a simple function containing suitable values of the constants A_* and B_*. Gilbert's data and the results of Meyer-Peter et al., were used for this purpose by Einstein. The sediment ranged in diameter from 0.0315 to 2.86 cm, and varied widely in density from that of barite to that of coal.

It was found that the experimental data were correlated well and could be fitted by

$$0.465\ \Phi = e^{-0.391\Psi} \tag{15}$$

for $\Phi \leq 0.4$, which corresponds to weak transport from the threshold of movement up to $(1/\Psi) \approx 0.232$.

The Einstein-Brown (1950) formula has been described as "...a modification developed by Hunter Rouse, M.C. Boyer, and E.M. Laursen of a formula by Einstein (1942). Its name derives from the name of the original author and the author of Chapter XII of Engineering Hydraulics (1950), where the formula first appeared." (ASCE, 1971).

In this development the coordinate space is altered to Φ_B vs. $(1/\Psi)$, the definition of Φ is modified slightly, and a relationship emerges for strong or high transport rates which fits the experimental data:

$$\Phi_B = 40(1/\Psi)^3 \quad , \tag{16}$$

and

$$\Phi_B = \frac{1}{F} \cdot \frac{g_s}{\rho_s} \sqrt{\frac{\rho}{(\rho_s - \rho)g d_s^3}} , \tag{17}$$

where d_s is the median grain diameter of mixtures. The range of applicability is from $(1/\Psi) = 0.13$ to 1.2.

In both the Einstein (1942) and Einstein-Brown (1950) formulas, the Shields parameter for dimensionless shear stress at the boundary is evaluated by measurement for a given bed and flow, and the appropriately calculated value of Φ is readily translatable into an estimated mass rate of bed load transport.

In a field study, τ_o could be determined hourly at each field station from velocity profile data. Either the 1942 or the 1950 relation could be used to calculate g_s, depending on the magnitude of $(1/\Psi)$.

The Einstein (1950) Bed Load Transport Function

This development applies not only to bed load in the strict sense, i.e., that which moves by creeping, rolling, or low level saltation up to several grain diameters, but also to the intermittently suspended load near the bed. The only sediment excluded from the theory is the "wash load," i.e., those particles that are small enough in diameter so as to spend nearly their entire aqueous history in suspension.

The chief factors that distinguish Einstein's work of 1950 from his 1942 method and from the Einstein-Brown (1950) formulation are: (1) bed sediment of mixed particle diameters is now treated, size fraction by size fraction; (2) in evaluating p, the probability of particle removal from the bed, the instantaneous lift expression now includes a random function, a normal error law, with a standard deviation of 0.5 and a mean of zero; (3) the hydraulic radius, R_h, a critical variable in the development, is now partitioned into R_h' and R_h'', viz, the radius associated with grain roughness and the radius associated with bed form roughness; and (4) an intermittently suspended load for each size fraction is now generated at the same time that the bed load is put into movement and is dependent, in part, on the magnitude of the bed load plus various other parameters of the particles and flow.

In the method repeated use is made of the quantity μ_*' which is $\sqrt{\tau_o'/\rho}$, where $\tau_o = \tau_o' + \tau_o''$, and

$$\tau_o = \Upsilon R_h S_e \quad , \tag{18}$$

where Υ is the specific weight of the fluid, R_h is the hydraulic radius of the river channel, and S_e is the slope of the energy grade line. Einstein partitions R_h into R_h' and R_h'' while holding S_e constant, but if one wishes this can be regarded equally well as a partitioning of τ_o. The deviding of R_h is done iteratively by Einstein using an experimentally determined plot Ψ'_{35} vs. $\bar{\mu}/\mu''_*$, where the former is the reciprocal Shields parameter based on d_{35}, the 35th percentile of the "finer-than" particle size frequency distribution, and where $\bar{\mu}$ is the depth mean velocity, and μ''_* is the shear velocity associated with the form resistance of the bed irregularities.

In field study, τ_o could be determined from velocity profiles taken hourly at each station. The measurement of τ_o in estuarine areas substitutes for the determination of S_e in rivers; the needed partitioning can be accomplished with adherence to the essential intent of the original Einstein method.

The new treatment of p involving, as it does, a random function which causes the lift force to fluctuate about its steady value, leads to the probability integral which is evaluated between limits based in part on the results of experiments. The value of the integral gives the percentage of time that the lift force is less than the effective particle weight. The probability of particle removal from the bed is unity minus the value of the integral.

As before,

$$\frac{p}{1 \quad p} = A_* \cdot \Phi \quad , \tag{19}$$

whence,

$$p = \frac{A_* \Phi}{1 + A_* \Phi} \quad , \tag{20}$$

and the definitive equation of the Einstein (1950) development can be written as

$$1 - \frac{1}{\sqrt{\pi}} \int_{-B_*\Psi_{*i}-(1/\eta_o)}^{+B_*\Psi_{*i}-(1/\eta_o)} e^{-t^2}\, dt = p = \frac{A_*\Phi_{*i}}{1 + A_*\Phi_{*i}} \quad , \tag{21}$$

where

$$A_* = 43.5$$

$$B_* = 0.143$$

$$\eta_o = 0.5 \tag{22}$$

$$\Psi_{*i} = f\left[\frac{(\rho_s - \rho)gd_{si}}{\tau_o'}\right]$$

and

$$\Phi_{*i} = \frac{1}{p_i} \frac{g_{sbi}}{\rho_s} \sqrt{\frac{\rho}{(\rho_s - \rho)g} \frac{1}{d_{si}}} \tag{23}$$

In these expressions d_{si} is the geometric mean particle diameter of the ith size fraction, p_i is the weight proportion of particle size d_{si} present in a bed sample, and g_{sbi} is the bed load discharge of mean size d_{si} in units of mass per unit path width per second. The bed load is confined to a layer two grain diameters in thickness.

The associated suspended load discharge, g_{ssi}, for the ith class is given by

$$g_{ssi} = g_{sbi}\left[p_r \quad I_1(\eta_o,z_i) + I_2(\eta_o,z_i)\right] \quad , \tag{24}$$

where the source theory is based on more or less conventional suspended load mechanics in which particle settling is balanced against upward directed turbulent diffusion, and it is required that sediment concentration be known at a reference level near the bed, in this instance, $2\,d_{si}$. The velocity distribution that is applied to the concentration profile is logarithmic with μ_* replaced by μ_*', the distance y above the bed is corrected for the hydrodynamic smoothness or roughness of the bed, and the equivalent sand roughness, k_s, it taken as d_{65}. The functions p_r, I_1, and I_2 derive from these considerations. The parameters η_{oi} and z_i of functions I_1 and I_2 are given as

$$\eta_{oi} = \frac{2d_{si}}{Y} \tag{25}$$

and

$$z_i = \frac{w_i}{0.4\ u_*'} \tag{26}$$

where Y is total depth and w_i is particle settling velocity. The von Karman constant, 0.4, requires reduction at high sediment concentrations (Einstein and Abdel-all, 1972).

Finally,

$$g_{si} = g_{ssi} + g_{sbi} \quad , \tag{27}$$

i.e., the total load for a particular fraction is the sum of the intermittently suspended load and the bed load, and

$$g_s = \sum_i \ g_{si} \tag{28}$$

<u>The Bagnold (1973) Formulation for Bed Load Transport Rate</u>

The sediment load which is treated in this theory is that which moves by rolling along the bottom or by saltation within a few grain diameters of the boundary. Underlying the development is the precept that the input of mechanical power from the moving fluid to a unit area of the bed can be expressed as $\tau_o \bar{u}$, the "stream power" or "fluid power of velocity," where τ_o is the shear stress at the bottom and $\bar{u}$ is the depth mean velocity of the transporting medium. It is further conceived that a constant fraction of the available power per unit area is utilized in maintaining the sediment transport, hence

$$g_{sb} = K_1 \tau_o \bar{u} \quad , \tag{29}$$

where g_{sb} is the mass rate of sediment transport per unit path width, and K_1 is a dimensional constant of proportionality, the "efficiency" in Bagnold's terminology.

In the derivation, g_{sb} is recalculated as i_b, the immersed weight of solids transported per unit time per unit path width thus yielding a dimensionally homogeneous equation in units, for example, of dyne-cm per cm^2 per second:

$$i_b = \frac{g_{sb}(\rho_s - \rho)g}{\rho_s} = K_2 \tau_o \bar{u} \quad , \tag{30}$$

where ρ_s and ρ are solid and fluid density respectively, g is gravitational acceleration, and K_2, bearing the connotation of "efficiency," is now dimensionless.

The determination of K_2 is achieved in the development that follows. A solids transport rate is written as

$$i_b = w'\bar{U} \quad , \tag{31}$$

where w' is the immersed weight of bed load solids in transit over unit area of the bed, and $\bar{U}$ is the mean travel velocity of the saltating load in the direction of flow. A coefficient of friction, $\tan\,\alpha$, is expressed as:

$$\frac{\bar{f}}{w'} = \tan\,\alpha \approx \tan\,\phi \approx 0.63 \quad , \tag{32}$$

where $\bar{f}$ is the mean opposing reactive stress offered to the flow of water by the saltating solids and is measured at the "center of thrust," near the bed, and the coefficient of friction ($\tan\,\alpha$) is averaged over time for the conditions of multiple intermittent contacts during saltation transport; $\tan\,\phi$ is a coefficient of sliding friction for granular materials, and ϕ is the "angle of repose." By substitution,

$$i_b = \frac{\bar{f}\,u_n}{\tan\,\alpha} \cdot \frac{\bar{U}}{u_n} \quad , \tag{33}$$

where u_n is the mean effective fluid velocity at the level of the "center of thrust;" the idea of "efficiency" is emobodied in the ratio $\bar{U}/u_n$, the ratio of solids travel to fluid velocity. The relation between $\bar{f}$ and τ_o is indicated by

$$\bar{f} = a\tau_o \quad , \tag{34}$$

where $0 \leq a \leq 1$ and is a measure of the fraction of the whole stress, τ_o, that is taken up in overcoming the reactive stress of the saltating load. In water transport, the factor a is said to increase slowly with u_* as given by Bagnold in the empirical expression

$$a = \frac{u_* - u_{*o}}{u_*} \tag{35}$$

where u_* is the shear velocity, $(\tau_o/\rho)^{1/2}$, and u_{*o} is the threshold shear velocity for sediment movement; and a = 0 when $u_* = u_{*o}$ and is evaluated only when $u_* \geq u_{*o}$. With substitution,

$$i_b = \frac{a\ \tau_o}{\tan\ \alpha} \bar{U} \quad . \tag{36}$$

$\bar{U}$, the solids travel rate near the bed, is always exceeded by u_n, the fluid velocity near the bed. By analogy with the relative motion for terminal settling velocity, the mean excess velocity of the fluid motion over the solids velocity, or "slip velocity" of Bagnold, is V_g, the terminal settling velocity of the solid particle. Thus,

$$\bar{U} = u_n - V_g \quad , \tag{37}$$

$$i_b = \frac{a\ \tau_o}{\tan\ \alpha} (u_n - V_g) \quad . \tag{38}$$

The velocity u_n is that of the fluid at an elevation nD above the bed, the "center of thrust" of Bagnold, where D is particle diameter and n is given by the empirical expression

$$n = 1.4 \left(\frac{u_*}{u_{*o}} \right)^{0 \cdot 6} \quad . \tag{39}$$

The distance nD is too close to the bed for practical measurement of u_n, and hence the needed velocity is calculated from the depth mean velocity of the flow, $\bar{u}$, which for a logarithmic profile corresponds to the distance 0.37 Y above the bed, where Y is the total depth. Thus

$$u_n = \bar{u} - 5.75\ u_* \log \left(\frac{0.37\ Y}{nD} \right) \quad , \tag{40}$$

and

$$i_b = \frac{a\ \tau_o}{\tan\ \alpha}\left[\bar{u} - 5.75\ u\ \ \log\left(\frac{0.37\ Y}{nD}\right) - V_g\right] \qquad (41)$$

or

$$i_b = \frac{(u_* - u_{*o})}{u_*} \cdot \frac{\tau_o\ \bar{u}}{\tan\ \alpha}\left[1 - \frac{5.75\ u_*\ \log\left(\frac{0.37\ Y}{nD}\right)}{\bar{u}} - \frac{V_g}{\bar{u}}\right] \qquad (42)$$

(Bagnold, 1973, equation 19)

and

$$g_{sb} = \frac{\rho_s}{(\rho_s - \rho)g} \cdot \frac{(u_* - u_*)}{u_*} \cdot \frac{\tau_o \bar{u}}{\tan\ \alpha}\left[1 - \frac{u_n}{\bar{u}} - \frac{V_g}{\bar{u}}\right] \quad . \qquad (43)$$

Bagnold obtained good agreement in tests of his equation 19 against sediment transport rates measured in laboratory flumes (Gilbert, 1914; Williams, 1970) after correction for side-wall effects and relative depth, Y/D, and against some Swiss river data for cobble transport. The Williams data were for 1.1-mm sand; Gilbert's data were for 0.79-mm and 7-mm mono-sized sediments.

Three difficulties were cited in applying the equation to finer-grained bed sediment: (1) the specification of the effective flow depth, Y, when bed corrugations are so large as to occupy up to one-half the total water depth; (2) the appropriate specification of the critical shear velocity, u_{*o}, when the total bed shear stress τ_o is divided between that associated with the grains, τ_o', and that associated with bed forms, τ_o''; and (3) the correct specification of n and, therefore, nD, the elevation of the "center of thrust." River data on the transport of sands 0.3 to 0.4 mm in diameter were said to be predicted reasonably well by the equation when n is scaled in hundreds rather than units.

In a field study, the following use can be made of a part of the Bagnold formalism. For use in equation (36), the Vanoni et al. (1967) relation can be used to divide τ_o into τ_o' and τ_o'' from observational data on bed form geometry, and τ_o' can then be used in equation (36) in place of τ_o. The Vanoni et al. equation is

$$\frac{1}{\sqrt{f''}} = 3.3 \log \frac{\lambda R_h}{(\Delta\bar{H})^2} - 2.3 \quad , \tag{44}$$

where f'' is a Darcy-Weisbach friction coefficient associated with the bed forms, λ is the wavelength of the bed forms, R_h is approximately equal to the water depth in the present application, and $\Delta\bar{H}$ is the mean relief of the bed forms. The value thus obtained for f'' is then used in the following expression to obtain τ_o', the bed shear associated with the grain roughness,

$$\tau_o' = \rho \left[u_*^2 - \frac{\bar{u}^2}{(8/f'')} \right] \quad , \tag{45}$$

which is equivalent to

$$u_*'^2 = u_*^2 - u_*''^2 \quad . \tag{46}$$

The Fredsøe and Engelund (1975) relation can be used to estimate $\tilde{U}$ the grain travel rate,

$$\tilde{U} = U_G = 10(u_*' - 0.7\ u_{*o}) \tag{47}$$

By substitution, using equations (30), (35), (36), and (47), the following expression can be obtained:

$$g_{sb} = \frac{\rho_s}{(\rho_s - \rho)g} \cdot \frac{(u_*' - u_{*o})}{u_*'} \cdot \frac{\tau_o'}{\tan \alpha} \cdot 10\ (u_*' - 0.7\ u_{*o}) \ . \tag{48}$$

SUMMARY

Ultimately the practical solution of problems arising from the unwanted deposition or erosion of sediment depends on understanding the mechanics of the transport of sediment by fluid. Despite the enormous complexity of the process and the fact that understanding is far from complete at the present time, some insight is gained and much of the false "lore" of erosion and deposition is removed by knowledge of sediment transport mechanics, particularly perhaps, the sediment transport continuity equation which is fundamental to the subject field. The foregoing brief review of some standard transport relationships is aimed at emphasizing the need to understand mechanics of the problem despite their present incompleteness.

APPENDIX
ON THE DERIVATION OF THE SEDIMENT CONTINUITY EQUATION

Let $\rho_b(x,y,z,t)$ = mass of sediment per unit volume of the water-sediment mixture, i.e., the bulk density, and

$$\vec{u}_s(x,y,z,t) = \hat{i}\, u_s + \hat{j}\, v_s + \hat{k}\, w_s$$

be the sediment velocity, with $\hat{i}, \hat{j}, \hat{k}$ unit vectors in the x, y, and z directions, respectively. Consider a "brick" with sides Δx, Δy, Δz centered at the point (x, y, z) with sediment flowing through it (see Figure 6-1). Now sediment is neither created nor destroyed in this "brick" so,

(mass flux of sediment in) - (mass flux of sediment out) = time rate of change of the mass of the sediment in the "brick"

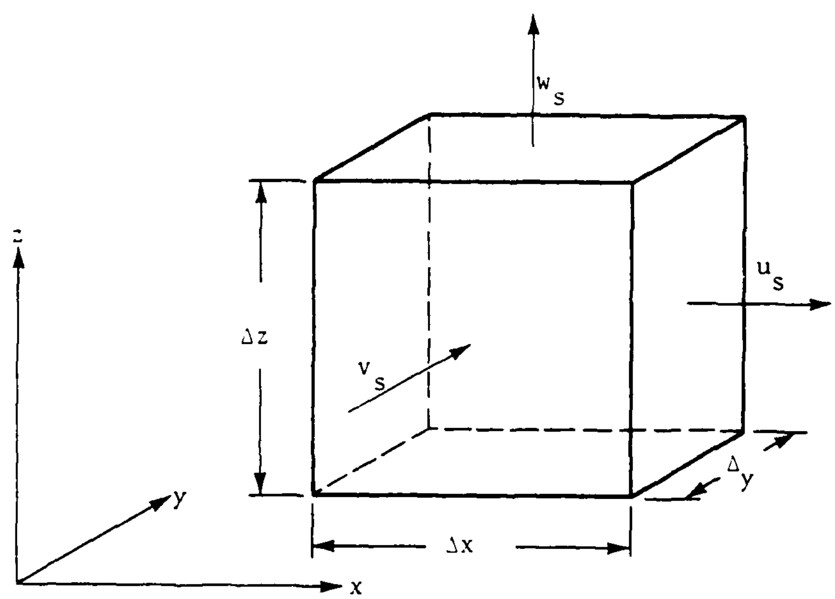

Figure 6-1. Definition Sketch for Mass Balance

The mass flux of sediment in the x direction entering the "brick" is (see Figure 6-1)

$$\rho_b\left(x - \frac{1}{2}\,\Delta x, y, z, t\right) u_s\left(x - \frac{1}{2}\,\Delta x, y, z, t\right) \Delta y\ \Delta z \quad .$$

The mass flux out for the x direction is

$$\rho_b\left(x + \frac{1}{2}\,\Delta x, y, z, t\right) u_s\left(x + \frac{1}{2}\,\Delta x, y, z, t\right) \Delta y\ \Delta z \quad .$$

Taking the difference of these two expressions, expanding ρ_b and u_s in a Taylor series about the point (x, y, z) we have

Net mass flux of sediment in the x direction

$$= \left[\rho_b\ u_s - \frac{1}{2}\,\Delta x\left(u_s\,\frac{\partial \rho_b}{\partial x} + \rho_b\,\frac{\partial u_s}{\partial x}\right) + 0\left((\Delta x)^2\right)\right] \Delta y\ \Delta z$$

$$- \left[\rho_b\ u_s + \frac{1}{2}\,\Delta x\left(u_s\,\frac{\partial \rho_b}{\partial x} + \rho_b\,\frac{\partial u_s}{\partial x}\right) + 0\left((\Delta x)^2\right)\right] \Delta y\ \Delta z \qquad \text{(continued)}$$

$$= -\left(u_s\,\frac{\partial \rho_b}{\partial x} + \rho_b\,\frac{\partial u_s}{\partial x}\right) \Delta x\ \Delta y\ \Delta z + 0\left((\Delta x)^2\ \Delta y\ \Delta z\right)$$

$$= -\frac{\partial}{\partial x}\left(\rho_b\ u_s\right)\ \Delta x\ \Delta y\ \Delta z + 0\left((\Delta x)^2\ \Delta y\ \Delta z\right) \quad . \qquad \text{(concluded)}$$

In a similar way, we find that net mass flux of sediment in the y direction

$$= - \frac{\partial}{\partial y} (\rho_b \, v_s) \quad \Delta x \; \Delta y \; \Delta z + 0 \; (\Delta x (\Delta y)^2 \quad \Delta z)$$

and net mass flux of sediment in the z direction

$$= - \frac{\partial}{\partial z} (\rho_b \, w_s) \quad \Delta x \; \Delta y \; \Delta z + 0 \; (\Delta x \; \Delta y \; (\Delta z)^2) \quad .$$

Finally, the time rate of change of the mass of sediment in the "brick" is

$$\frac{\partial}{\partial t} (\rho_b \; \Delta x \; \Delta y \; \Delta z) \quad .$$

Summing the net mass fluxes and equating the sum to the time rate of change of sediment mass in the "brick" we have

$$\begin{aligned} \frac{\partial \rho_b}{\partial t} \Delta x \; \Delta y \; \Delta z = & - \frac{\partial}{\partial x} (\rho_b u_s) + 0 \; ((\Delta x)^2 \quad \Delta y \; \Delta z) \\ & - \frac{\partial}{\partial y} (\rho_b v_s) + 0 \; (\Delta x \quad (\Delta y)^2 \; \Delta z) \\ & - \frac{\partial}{\partial z} (\rho_b w_s) + 0 \; (\Delta x \; \Delta y (\Delta z)^2) \quad . \end{aligned}$$

Dividing by the volume of the "brick," $\Delta x \; \Delta y \quad \Delta z$,

$$\begin{aligned} \frac{\partial \rho_b}{\partial t} = & - \frac{\partial}{\partial x} (\rho_b u_s) + 0(\Delta x) - \frac{\partial}{\partial y} (\rho_b \, v_s) + 0(\Delta y) \\ & - \frac{\partial}{\partial z} (\rho_b \, w_s) + 0(\Delta z) \quad . \end{aligned}$$

Finally, letting Δx, Δy, and $\Delta z \to 0$, the mass conservation equation is

$$\frac{\partial \rho_b}{\partial t} + \nabla \cdot (\rho_b \vec{u}_s) = 0 \quad . \tag{1}$$

We take a coordinate system with the z axis vertically upwards (see Figure 6-2). The origin of this coordinate system is arbitrary and we define the free surface of the water by

$$z = h(x,y,t)$$

and the bottom by

$$z = \eta(x,y,t)$$

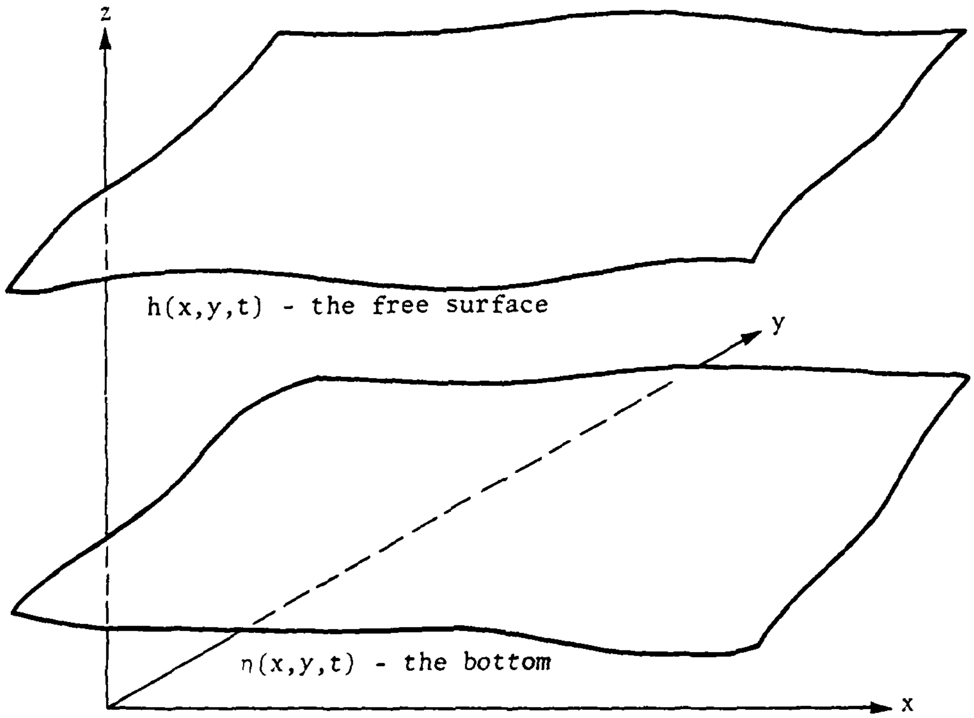

Figure 6-2. Definition Sketch

Integrate the mass conservation equation [eq. (1) above] from $z = \eta$ to $z = h$,

$$\int_{\eta}^{h} \frac{\partial \rho_b}{\partial t}\, dx + \int_{\eta}^{h} \nabla \cdot (\rho_b \vec{u}_s) dz = 0 \quad . \tag{2}$$

Using Liebnitz's rule

$$\frac{\partial}{\partial t} \int_{\eta(x,y,t)}^{h(x,y,t)} \rho_b(x,y,z,t) dz = \int_{\eta}^{h} \frac{\partial \rho_b}{\partial t}\, dz + \rho_b(x,y,h,t)\,\frac{\partial h}{\partial t} - \rho_b(x,y,\eta,t)\,\frac{\partial \eta}{\partial t} \quad ,$$

so that

$$\int_{\eta}^{h} \frac{\partial \rho_b}{\partial t}\, dz = \frac{\partial}{\partial t} \int_{\eta}^{h} \rho_b dz - \rho_b(z,y,h,t)\,\frac{\partial h}{\partial t} + \rho_b(x,y,\eta,t)\,\frac{\partial \eta}{\partial t}$$

$$= \frac{\partial V}{\partial t} + \rho_b(x,y,\eta,t)\,\frac{\partial \eta}{\partial t} - \rho_b(x,y,h,t)\,\frac{\partial h}{\partial t} \tag{3}$$

where

$$V(x,y,t) \equiv \int_{\eta(x,y,t)}^{h(x,y,t)} \rho_b\ (x,y,z,t) dz \tag{4}$$

is the mass of sediment per unit area between the bottom and the free surface.

Proceeding to the divergence terms in equation (2) we have, using Liebnitz's rule

$$\int_{\eta(x,y,t)}^{h(x,y,t)} \frac{\partial}{\partial x} (\rho_b\ u_s)dz = \frac{\partial}{\partial x} \int_{\eta}^{h} \rho_b\ u_s\ dz - \rho_b(x,y,h,t)\ u_s(x,y,h,t)\ \frac{\partial h}{\partial x}$$

$$+ \rho_b(x,y,\eta,t)\ u_s\ (x,y,\eta,t)\ \frac{\partial \eta}{\partial x} \tag{5}$$

$$= \frac{\partial Q_x}{\partial x} + \rho_b(x,y,\eta,t)\ u_s(x,y,\eta,t)\ \frac{\partial \eta}{\partial x}$$

$$- \rho_b(x,y,h,t)\ u_s(x,y,h,t)\ \frac{\partial h}{\partial x} \quad ,$$

where we have defined Q_x, the mass flux of sediment in the x direction by

$$Q_x(x,y,t) \equiv \int_{\eta(x,y,t)}^{h(x,y,t)} \rho_b(x,y,z,t)\ u_s(x,y,z,t)dz \quad . \tag{6}$$

In the same way,

$$\int_{\eta}^{h} \frac{\partial}{\partial y} (\rho_s\ v_s)dz = \frac{\partial Q_y}{\partial y} + \rho_b(x,y,\eta,t)\ v_s(x,y,\eta,t)\ \frac{\partial \eta}{\partial y}$$

$$- \rho_b(x,y,h,t)\ v_s(x,y,h,t)\ \frac{\partial h}{\partial y} \tag{7}$$

with

$$Q_y(x,y,t) \equiv \int_{\eta(x,y,t)}^{h(x,y,t)} \rho_b(x,y,z,t)\ v_s(x,y,z,t)dz \quad . \tag{8}$$

Finally, integrating directly,

$$\int_{\eta}^{h} \frac{\partial}{\partial z} (\rho_b\ w_s)dz = \rho_b(x,y,h,t)\ w_s(x,y,h,t) - \rho_b(x,y,\eta,t)\ w_s(x,y,\eta,t) \tag{9}$$

Substituting equations (3), (5), (7), and (9) into equation (2), we find, after some rearrangement, that

$$\begin{aligned} &\frac{\partial V}{\partial t} + \rho_b(x,y,\eta,t)\ \frac{\partial \eta}{\partial t} + \nabla \cdot \vec{Q} \\ &= - \left[\rho_b(\vec{u}_s \cdot \nabla)\eta\right]_{z=\eta(x,y,t)} + \left[\rho_b(\vec{u}_s \cdot \nabla)h\right]_{z=h(x,y,t)} \\ &+ \rho_b(x,y,h,t)\ \frac{\partial h}{\partial t} - \rho_b(x,y,h,t)\ w_s(x,y,h,t) \\ &+ \rho_b(x,y,\eta,t)\ w_s(x,y,\eta,t) \quad . \end{aligned} \tag{10}$$

This is the general form of the sediment continuity equation. This equation reduces to the "standard" form (where $\varepsilon \equiv \rho_b(x,y,\eta,t)$ is the bulk density at the bottom),

$$\frac{\partial \eta}{\partial t} = \left(- \frac{1}{\varepsilon}\right) \left(\frac{\partial V}{\partial t} + \nabla \cdot \vec{Q}\right) \quad , \tag{11}$$

if certain assumptions are made.

First we must assume that

$$\left[\rho_b(\vec{u}_s \cdot \nabla)\eta\right]_{z=\eta} = 0 \quad .$$

Clearly $\rho_b(x,y,\eta,t) \neq 0$, so that we require that

$$\left[(\vec{u}_s \cdot \nabla)\eta\right]_{z=\eta} = 0 \quad .$$

There is, of course, the special case that at the bottom $\vec{u}_s$ is perpendicular to the gradient of the bottom ($\nabla\eta$), but this seems to be unlikely to be true in general. Therefore, we must assume that $\vec{u}_s = 0$ or $\nabla\eta = 0$ at the bottom, or both are zero.

In general, the bottom will be sloping so that $\nabla\eta \neq 0$ and may be large at some points, at ripples for example. If there is a length scale, L, such that the small scale features have wavelengths much smaller than L and the wavelengths of the large scale features are much larger than L, then equation (10) can be Reynolds's averaged, as in the theory of turbulence, over an (L x L) area so as to, perhaps, reduce the value of $\nabla\eta$. In order to do this there must be a window in the spectrum of the bed forms. While it may be possible to reduce $\nabla\eta$ by taking a suitable spatial average, it appears that $\nabla\eta$ may be small but nonzero unless we average over such a large area that the effects of all bed forms, including shoals, are wiped out. This hardly seems to be a useful procedure.

It might be assumed that $\vec{u}_s = 0$ at the bottom. But $\vec{u}_s \neq 0$ at the bottom if the bed forms are migrating, because then a kinematic argument shows that at the bottom $\vec{u}_s = \vec{c}$, the velocity of the bed form at the bottom. Again, spatially averaging over an (L x L) area can make $\vec{u}_s$ at the bottom equal to the velocity of the large bed forms, which may be quite small, but will be, in general, nonzero.

It will be assumed here that equation (10) has been Reynolds's averaged over an (L x L) area, where L is much larger than the wavelength of the small scale bed forms and much smaller than the wavelength of the large scale bed forms. Then the magnitude of $(\vec{u}_s \cdot \nabla\eta)$ at the bottom, is equal to the product of the speed of migration of the large scale bed forms and the mean slope of these same large scale bed forms. A formal perturbation expansion can then be carried out (the details are not given here) to show that $\left[\rho_b(\vec{u}_s \cdot \nabla)\eta \right]_{z=\eta}$ is of higher order than $\frac{\partial\eta}{\partial t}$, $\frac{\partial V}{\partial t}$, and $\nabla \cdot \vec{Q}$, and so this term can be neglected.

Now consider the next three terms on the right-hand side of equation (10). These terms are, slightly rearranged

$$\rho_b(x,y,h,t) \left[\frac{\partial h}{\partial t} - w_s - (\vec{u}_s \cdot \nabla)h \right]_{z=h(x,y,t)}$$

If the suspended sediment at the free surface is moving with the water, then $w_s = w$ and $\vec{u}_s = \vec{u}$ in both cases the water velocity and the sum of these terms is identically zero because of the kinematic free surface condition that

$$w = \frac{dh}{dt} = \frac{\partial h}{\partial t} + (\vec{u} \cdot \nabla)h$$

on the free surface, h(x,y,t). It seems to be a reasonable approximation to assume that at the free surface, the sediment is moving at nearly the fluid velocity and that the term in square brackets above is very small. We also expect that ρ_b is small at the surface. Therefore, the product of these two terms is expected to be very small and can be neglected.

Finally, we expect that, at the bottom, $w(x,y,\eta,t) = 0$. Therefore, the last term on the right-hand side of equation (10) is zero.

In summary, Reynolds's averaging and a perturbation expansion show that the "standard" form of the sediment continuity equation is a locally averaged, linearized form of the general form of the sediment continuity equation. As such it is of the same type as the equations of classical linear surface wave theory.

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SECTION 7

SEDIMENT HISTORY & PREDICTIONS IN LOWER JAMES RIVER REGION

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(This paper was not made available in time for inclusion in these proceedings.)

SECTION 8

DISTRIBUTION AND HYDRODYNAMIC PROPERTIES OF FOULING ORGANISMS IN THE PIER 12 AREA OF THE NORFOLK NAVAL STATION

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INTRODUCTION

Fouling of deep draft naval vessels, in particular aircraft carriers, in the area of the Norfolk Naval Station has been a reoccurring problem since the early 60's. The principal agents of fouling have been the hydroid, Sertularia argentea and the fleshly bryozoan, Alcyonidium verrilli. The particular fouling problem encountered in the Norfolk area is not the typical case of the organisms growing attached to ship hulls but is basically a problem of sea suction and subsequent clogging of screen grates and condenser tube sheets.

To date all efforts to solve the problem have proven ineffective, partly because there has never been a clear understanding of the life history and hydrodynamic behavior of either the hydroid or bryozoan. In order to develop and make sound judgement of alternative solutions to the hydroid and bryozoan fouling problem, it is necessary to understand as completely as possible the biological and physical behavior of these organisms. This paper presents our current state of the art knowledge about hydroid and bryozoan properties as they relate to fouling of naval vessels.

THE ORGANISMS

The hydroid, Sertularia argentea, is the commonest winter hydroid in the Chesapeake Bay region. Each colony of animals is generally attached to a hard substrate, rocks, shells, piling, etc., by a stolon. Colonies can obtain lengths over 10 inches and be quite plumose, encompassing a volume equivalent to a 10 to 12 inch sphere. The integrity of the colony is maintained by a very tough chitinous polymucosaccharide sheath that is resistant to decay and breakage.

This hydroid may have an annual life cycle in the Bay area. In the early and late winter adult colonies reproduce sexually, producing a swimming larval phase that eventually sets on a suitable substrate. The newly set colonies grow until spring. When the Bay waters start to warm they become dormant and remain in a dormant state over summer. In the fall when Bay waters cool, growth ensues and, by early winter, colonies mature and reproduce sexually completing the life cycle.

Sertularia is widely distributed in the Bay and can be found growing in every major tributary. It is an estuarine species and tends to be found attached and growing at salinities of 10 to 25 percent. However, we really do not know if there are specific areas around the Bay that are major production points. In the winter when storms generate a lot of wave action the hydroid is broken free of its attachment and drifts with the currents, in a manner very similar to tumble weed. It is the movement and concentration of these loose adult colonies that creates the fouling problems for deep-draft vessels.

The bryozoan, *Alcyonidium verrilli*, is the most common winter bryozoan in the Chesapeake Bay region. Colonies of animals can be attached to a variety of hard substrates intruding the sheath of *Sertularia*. Colonies can obtain sizes larger than spheres 18 inches in diameter. The colonies are very fleshly and given structural support by a fiberous connective tissue. Unlike the hydroid, the bryozoan is prone to decay once it dies and does not tend to accumulate in the sediments.

We do not know what the life history of *Alcyonidium* is in the Bay area, but it is most likely an annual and follows a similar pattern to *Sertularia*. The bryozoan differs from the hydroid in that it is more a marine species and seems to be found growing at salinities of 20 percent or higher.

Waves and currents are also responsible for the disattachment of the bryozoan. Once free to move, they tend to concentrate in areas of reduced currents or in areas protected from wave action.

HYDRODYNAMIC PROPERTIES

The hydrodynamic properties of the fouling organisms were examined in a hydraulic flume which has a 14.6m (48 ft) by 0.9m by 0.9m (3 ft) test section. The current speed in the test section may be adjusted from 2 cm/sec[1] to 85 cm/sec. The overall uniformity of current speed versus depth is within 2-3 percent.

Once the current speed has been properly adjusted, the fouling organisms were released at the head of the test section. Movement patterns and speed were recorded. Settling velocities were determined in standing water. All tests were run in fresh water. Hydrodynamic properties of both 10 percent formaldehyde preserved and freshly collected specimens were examined.

In general it was found that both the hydroid and bryozoan were negatively buoyant and sank. The larger the colony the greater the settling or fall velocity (Table 8-1). Density was found to be approximately 1.128 g/cc for hydroids and 1.187 g/cc for bryozoans. The critical roll velocity for bryozoans seems to be about 8 cm/sec. For hydroids the critical roll velocity ranges from about 4.5 to 8 cm/sec depending on the size and condition of the colony (alive or dead). When a colony of hydroids is actively feeding it is very plumose and would present maximum surface area for movement by weaker currents. Table 8-1 summarizes the measured hydrodynamic properties of the fouling organisms.

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[1] cm/sec x 0.03281 = ft/sec
cm/sec x 0.1943 = knots
ft/sec x 0.5921 = knots

TABLE 8-1. SUMMARY OF THE HYDRODYNAMIC PROPERTIES OF THE HYDROID SERTULARIA ARGENTIA AND THE BRYOZOAN ALCYONIDIUM VERRILLI

Critical Roll or Transport Velocity Test on Smooth Floor

Hydroids

Flume Velocity		Comments*
0.05 ft/sec	1.52 cm/sec	No motion
0.07	2.13	Large plumose live col. move and stopped
0.08	2.43	No movement of dead col.
0.09	2.74	No movement of dead col., liv col. waving
0.10	3.04	Live col. waving
0.12	3.65	Move and stop large col.
0.14	4.26	Moving of large col.
0.17	5.18	Dead col. start moving
0.25	7.62	Rolling of col. starts
0.26	8.00	Rolling of preserved col.

Bryozoans

Flume Velocity		Comments**
0.18	5.48	No motion
0.20	6.09	Waving
0.22	6.70	Waving
0.23	7.01	Waving
0.25	7.62	Large col. move & stop
0.28	8.53	Large and small col. move
0.32	9.75	Some rolling of larger col.
0.48	14.63	Large col. roll, small col. slide or roll

* Large Hydroid colony is > 50 g
** Large Bryozoan colony is > 150 g

Density

Hydroid

$$\frac{152.3\ \text{g}}{135\ \text{cc}} = 1.128\ \text{g/cc}$$

Bryozoan

$$\frac{296.9\ \text{g}}{250\ \text{cc}} = 1.187\ \text{g/cc}$$

TABLE 8-1. SUMMARY OF THE HYDRODYNAMIC PROPERTIES OF THE HYDROID SERTULARIA ARGENTIA AND THE BRYOZOAN ALCYONIDIUM VERRILLI (Cont'd)

Critical Lift Velocity, on Irregular Floor, Into Water Column

Hydroids and Bryozoans	25/cm/sec / 0.82 ft/sec	

Settling or Fall Velocity

Hydroids		
wet weight live colonies		
158 g	3.42 cm/sec	0.11 ft/sec
50	2.71	0.09
29	3.71	0.12
7	1.27	0.04
wet weight preserved colonies		
40 g	5.9 cm/sec	0.19 ft/sec
20	4.5	0.15
10	3.6	0.12
5	3.0	0.10
Bryozoans		
wet weight live colonies		
278 g	6.9 cm/sec	0.23 ft/sec
130	6.5	0.21
45	8.1	0.27

TABLE 8-1. SUMMARY OF THE HYDRODYNAMIC PROPERTIES OF THE HYDROID SERTULARIA ARGENTIA AND THE BRYOZOAN ALCYONIDIUM VERRILLI (Cont'd)

Colony Velocity at Various Flume Velocities

Flume Velocity 0.23 ft/sec 7.01 cm/sec

Hydroids — Live colony Velocity cm/sec

wt.	Trial A	Trial B	Trial C	$\bar{X}$	SD
72.6 g	6.47	6.15	6.47	6.36	0.18
30.3	6.83	7.03	6.91	6.92	0.10
28.9	5.49	5.35	4.86	5.23	0.33
25.4	5.69	6.31	5.67	5.89	0.36
19.6	7.24	6.83	6.65	6.90	0.30
11.9	5.08	4.73	5.42	5.08	0.34
5.1	6.99	5.69	6.15	6.28	0.66

Flume Velocity 0.31 ft/sec 9.44 cm/sec

Hydroids — Live Colony Velocity cm/sec

wt.	Trial A	Trial B	Trial C	$\bar{X}$	SD
72.6 g	9.46	9.25	8.98	9.23	0.24
30.3	9.46	9.46	9.11	9.34	0.20
28.9	7.55	8.66	7.45	7.89	0.67
25.4	7.32	8.78	8.78	8.30	0.84
11.9	9.54	8.09	8.61	8.41	0.28

Bryozoans — Live Colony Velocity cm/sec

wt.	Trial A	Trial B	Trial C	$\bar{X}$	SD
117.5g	3.00	3.21	3.12	3.18	0.16
80.5	3.73	4.73	3.62	4.02	0.61
48.0	4.10	3.97	4.17	4.08	0.10
25.0	5.17	4.56	4.92	4.88	0.31
13.7	3.30	4.39	3.61	3.76	0.56

TABLE 8-1. SUMMARY OF THE HYDRODYNAMIC PROPERTIES OF THE HYDROID <u>SERTULARIA ARGENTIA</u> AND THE BRYOZOAN <u>ALCYONIDIUM VERRILLI</u> (Cont'd)

Colony Velocity at Various Flume Velocities (Cont'd)

Flume Velocity 0.61 ft/sec 18.59 cm/sec

Hydroids

wt.	Live Colony Velocity cm/sec Trial A	B	C	$\bar{X}$	SD
177.0 g	11.71	12.18	10.25	11.38	1.01
56.0	17.32	17.83	17.08	17.41	0.38
53.5	13.67	14.47	12.95	13.69	0.76
22.7	18.09	17.57	--	17.83	0.34
12.7	17.57	17.08	17.83	17.49	0.38
6.4	16.18	18.64	18.36	17.73	1.35

Bryozoans

wt.	Live Colony Velocity cm/sec Trial A	B	C	$\bar{X}$	SD
364.0 g	11.60	11.50	11.71	11.60	0.11
108.5	9.39	10.08	11.18	10.22	0.90
51.5	10.42	12.30	12.95	11.89	1.31
25.9	8.98	10.80	7.93	9.20	1.39
13.5	8.98	13.08	10.25	10.77	2.10

DISTRIBUTION OF FOULING ORGANISMS AT PIER 12

The distribution of fouling organisms around Pier 12 at the Norfolk Naval Base is quite variable and dynamic. Controlling factors are thought to be tidal currents and wind setup circulation. Once in the Pier 12 berthing area, sedimentation and burial of organisms play a role in keeping the organisms in the berth.

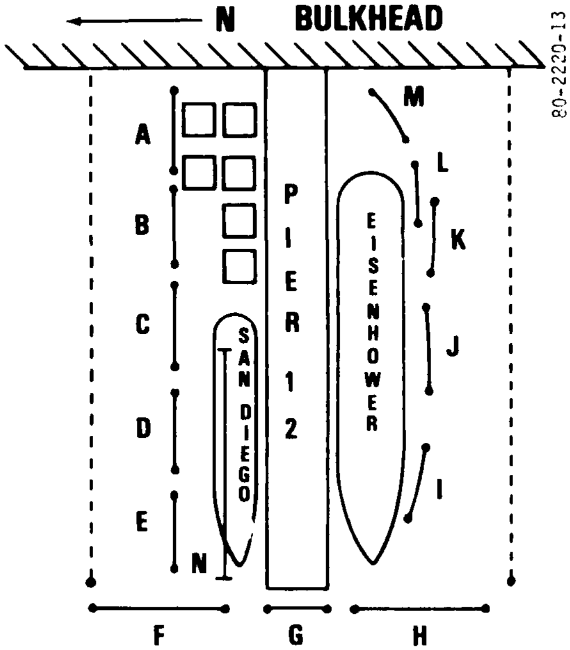
Detailed surveys were conducted in the berthing area on April 1, April 15, and May 12 using a 2 ft oyster dredge dragged for a known distance in order to get quantitive estimates of fouling organisms' densities. Results of these surveys are presented in Tables 8-2, 8-3, and 8-4. There appears to have been a substantial decline in hydroid density from April 1 to April 15 (3941 kg to 1215 kg) in the south berth of Pier 12. The mechanism that moved the hydroids out of the berth is most likely wind-driven circulation. It is also likely that a portion of the hydroids were buried in the berth. There was an increase in the percentage of buried hydroids with each survey period (70% buried April 1, 93% April 15, and 96% May 12). Navy divers have reported finding hydroids buried at least 3 ft below the sediment surface. The oyster dredge we used effectively samples only to sediment depths of 6-8 in. Therefore, while the surface concentrations of hydroids may appear to decline, there may be a net accumulation of hydroids when episodes of high sedimentation occurred.

A gradient of fouling organisms exists in the berthing area with highest densities occurring 400-500 ft from the bulkhead. It seems that the animals tend to pile up in this area on entering the berth. The highest percentages of live animals, an indication of recent recruitment, also occur in this area.

DISTRIBUTION OF FOULING ORGANISMS AROUND PIER 12 AND HAMPTON ROADS

The density of hydroids and bryozoans in Hampton Roads from April 1 to 15 was variable. While no direct comparison can be made with densities in the berthing area, because the area covered by the oyster dredge out in Hampton Roads could not quantified, there were areas, in particular Middle

TABLE 8-2. HYDROID DISTRIBUTION PIER 12
APRIL 1, 1980



Drag is 2 ft wide and was towed for 200 ft so area covered was 400 ft^2

*1 kg wet weight $\sim$ 1 gallon

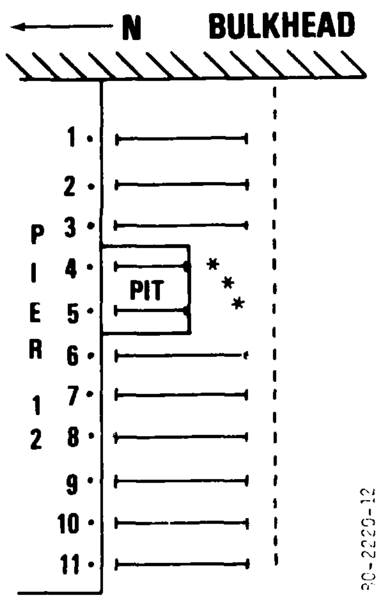
Total amount in pier area
North side 3634 kg - 7.60 g/ft^2
South side 3941 kg - 7.25 g/ft^2

	Hydroids		
Drag	Amount	% Live	% Buried
A	2.8 kg*	10	90
B	1.0	40	60
C	4.8	40	60
D	0.4	0	100
E	2.0	40	60
F	0.7	10	90
G	0.1	10	90
H	0.2	0	100
I	0 5	0	100
J	1.3	0	100
K	3.0	0	100
L	8.0	100	0
M	1.7	100	0
N**	3.2	70	30

	Bryozoans		
A	0	-	-
B	0	-	-
C	0	-	-
D	0	-	-
E	0	-	-
F	0	-	-
G	0	-	-
H	0	-	-
I	0.3	0	100
J	0	-	-
K	0	-	-
L	2.5	100	0
M	0	-	-
N	0	-	-

** Taken April 2 after San Diego left, 500 ft Drag of 8.0 kg.

TABLE 8-3. HYDROID DISTRIBUTION SOUTH SIDE PIER 12
APRIL 15, 1980



Hydroids

Drag	Amount	% Live	% Buried	Amt for Pier Seg
1	2.2 kg*	30	70	181.2
2	2.2	5	95	181.2
3	1.3	1	99	107.1
4**	2.4	10	90	199.4
5***	2.0	10	90	166.2
6	1.2	10	90	98.8
7	0.6	0	100	49.4
8	0.4	1	99	33.0
9	0.5	1	99	41.2
10	0.3	1	99	24.7
11	0.2	10	90	16.5
			TOTAL	1215.3 kg

Drag is 2 ft wide and was towed for 300 ft so area covered was 600 ft^2

* 1 kg wet weight $\approx$ 1 gallon

** Drag length in pit 130 feet

***Hydroid density in this area taken as average of drag 3 and 6

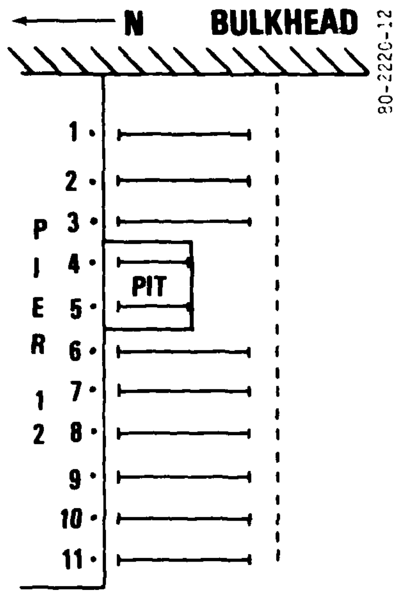
Bryozoans

Drag	Amount	% Live	% Buried
1	0.1 kg	30	70
2	0.1	5	95
3	0.05	1	99
4	0.9	5	95
5	0	-	-
6	0.1	90	10
7	0	-	-
8	0	-	-
9	0	-	-
10	0	-	-
11	0	-	-

Total volume of Hydrozoans in pier area:
- pit 366 kg
- total pier 1215.3 kg
- 30% of all Hydroids in pit
- density in pit 5 x's higher than rest of pier area

Total volume of Bryozoans in pier area:
- pit 96.9 kg
- total pier 141.9 kg
- 68% of all bryozoans in pit
- density in pit 25 x's higher than rest of pier area

TABLE 8-4. HYDROID DISTRIBUTION PIER 12
MAY 12, 1980



Hydroids

Drag	Amount	% Live	% Buried	Amt for Pier Seg
1	0.9 kg*	10	90	74.1
2	2.5	20	80	205.9
3	1.5	10	90	123.6
4**	2.3	0	100	191.1
5**	2.2	0	100	182.8
6	0.3	0	100	24.7
7	0.2	0	100	16.47
8	0.3	0	100	24.7
9	0.3	0	100	24.7
10	3.3	0	100	271.8
11	1.1	0	100	90.6
			TOTAL	1230.5 kg

Drag is 2 ft wide and towed for 300 ft, area covered was 600 ft^2

* 1 kg wet weight ~ 1 gallon

** Drag length in pit 130 ft area covered 260 ft^2

Bryozoans

Drag	Amount	% Live	% Buried	Amt for Pier Seg
1	6.7	100	0	551.9
2	0.2	0	100	16.5
3	0.1	0	100	8.2
4**	0	-	-	0
5**	0.4	0	100	33.2
6	1.5	0	100	123.6
7	0	-	-	0
8	0	-	-	0
9	0	-	-	0
10	0	-	-	0
11	0	-	-	0
				733.41 g

Ground (Figure 8-1 and Table 8-5), where it was felt that densities of hydroids exceeded densities in the Pier 12 area. It must be kept in mind that the hydroid, Sertularia argentea and the bryozoan, Alcyonidium verrilli are very abundant over the entire lower Bay during the winter months.

The origin of the hydroids and bryozoans that eventually enter Pier 12 is unknown. Current and circulation patterns in Hampton Roads are very complex, and it may be that only hydroids produced in a certain part of the James River or lower Bay serve as the primary source of fouling organisms to the pier. It is definite that the fouling organisms are not produced in the Pier 12 berths. The pier area acts only as a sink and catches drifting organisms.

RESOLUTION OF THE FOULING PROBLEM

The resolution of fouling problem at Pier 12 can only be engineered with a clear understanding of how the organisms get to the Pier 12 area and the mechanisms involved. It appears that fouling organism movement is related to extreme weather conditions. However we have no data on currents in the pier area during or after extreme weather. Where the organisms originate may also be a key to solving the problem, if there are definable areas in Hampton Roads that serve as primary sources for the organisms in Pier 12. When and at what rate the organisms grow would be helpful in predicting when to expect fouling. Correlation of the historic record of fouling incidents, biological properties (growth distribution), and hydrodynamic properties with meteorological and hydrographic conditions is necessary to predict the fate and movement of hydroids.

The solution to the fouling problem will not be simple. The organisms are too common and widely distributed to be eliminated from the Bay. Dredging the berths deeper and raking can only be considered temporary solutions. The permanent solution must be engineering and based on understanding of the dynamic processes that move the organisms in the Pier 12 area.

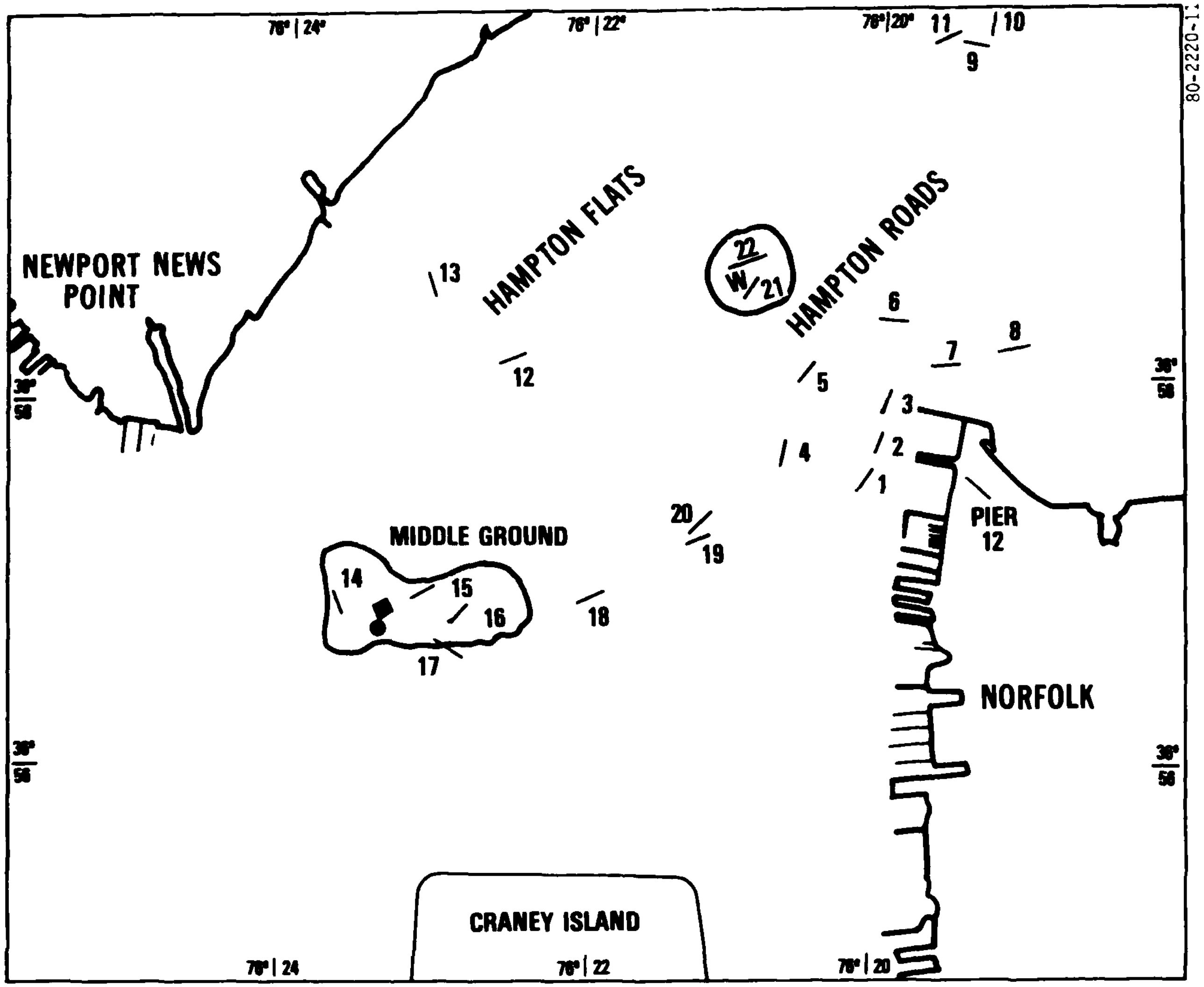


Figure 8-1. Distribution of Fouling Organisms in Hampton Roads from April 1 to 15, 1980. Quantities Collected are Given in Table 5.

TABLE 8-5. HYDROID DISTRIBUTION AROUND PIER 12
AND HAMPTON ROADS - APRIL 1 to 15, 1980

Drag*	Amount	% Live	% Buried	Comments
1	0.2 kg	50	50	No shell
2	0.9	100	0	No shell
3	0.6	100	0	Attached and growing on shells
4	2.0	50	50	No shell
5	3.0	100	0	No shell
6	0.5	100	0	Attached and growing on shells
7	3.0	100	0	Attached and growing on shells
8	0.0	--	--	Just shells
9	3.0	100	0	Mud
10	5.3	100	0	Mud
11	5.0	90	10	In dredged pit - mud
12	0.4	100	0	Attached and growing on shells
13	0.1	100	0	Attached and growing on shells
14	5.2	70	30	Shells, not attached
15	3.5	90	10	Shells, not attached
16	2.7	90	10	Attached and growing on shells
17	12.0	90	10	Attached and growing on shells
18	6.7	100	0	No shell
19	0.7	5	95	Mud
20	0.8	10	90	Mud
21	0.4	100	0	Attached and growing on shells
22	2.0	90	10	Attached and growing on shells

Bryozoans found only at the following:

3	0.2	100	0	Not attached to shells
9	0.9	100	0	Mud
10	2.3	100	0	Mud
11	2.0	90	10	Mud
17	0.3	90	10	Not attached to shells

Drags were approximately two minutes. Area covered was variable so amounts are not strictly quantitive.

* Location of Drags are numbered on Figure 8-1.

SECTION 9

CVN 68 CLASS CONDENSER FOULING

By

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Precommission Unit
USS CARL VINSON (CVN 70)
Newport News, Virginia

BACKGROUND

Propulsion plant light off on steam propelled ships in preparation for underway, propulsion plant testing, or to support catapult testing requires the operation of turbine generator condensers. The requirement exists because of the necessity to condense steam and return the water, using the main feed pumps, to the steam generators (equivalent to the boilers on conventional ships). Main feed pumps are driven by noncondensing turbines. The steam exhaust from these turbines, called auxiliary exhaust, is in turn directed to the deaerating feed tank, with excess steam "unloaded" to a turbine generator or main condenser. Proper operation of the condenser requires that it be under vacuum, both to ensure that requisite energy is extracted from the steam for proper turbine operation and to prevent overpressurization of the condenser. Maintaining a vacuum on a condenser requires uninterrupted sea water flow. Seawater flow to turbine condensers is therefore a fundamental prerequisite to any propulsion plant operation requiring significant amounts of steam.

The inability of CVN 68 Class carriers to operate seawater circulating water systems at Pier 12, Norfolk, consequently causes the following problems:

a. Underway evolutions cannot be conducted utilizing main engines. Tug assistance to anchorage is required.

b. Steam plant testing cannot be conducted in port.

c. Catapult testing cannot be conducted in port.

d. Flight operations are delayed following the underway time for up to twelve hours because catapults cannot be warmed at the pier.

e. Pre-underway electrical and electronic testing is complicated because turbine generators are not available for ship's power, requiring operation with the limited power available from the emergency diesel generators.

Clogging of condensers can occur either at the suction grating on the hull or on the tube sheet inside the condenser head itself. Clogging on the grating can be cleared either by ship's motion (assuming sufficient turbines are available to make a transit of the channel prudent) or by divers. Cleaning by divers is complicated by the difficulty experienced in locating the suctions because of minimal visibility underwater (sometimes as little as six inches) and disorientation caused by the flat and essentially featureless bottom of the hull. Cleaning of suction gratings at anchorage is further complicated by the divers' inability to maintain a given position under the ship because of tidal flow. Cleaning, therefore, is only feasible for about a 45-minute period every 6 hours at slack tide. Cleaning of the tube sheet clogging can be conducted by ship's force personnel, but is time consuming and precludes operation of the turbine during cleaning operations for up to 6.5 hours.

Clogging of condensers has been experienced at anchorage, as well as at Pier 12 making any sort of underway evolution a tenuous operation.

LOCAL ACTION TO DATE

Acting with NAVFACLANTDIV and Norfolk Naval Station, various proposals including raking the bottom, fencing the pier area, sucking sediment, etc. have been discussed and determined to be impractical, at least within current budgetary constraints. The following actions have been taken to provide some improvement on a t ıporary basis:

a. Advance maintenance dredging on the south side of Pier 12 has been conducted to a depth of 50 feet in a localized area which roughly corresponds to the area under number 2 propulsion plant. Although testing has not been conclusive, there is reason to believe that the increase of 5 feet in water depth has been helpful.

b. A 200 foot x 20 foot net has been purchased to be used as a "diaper" under two of the turbine generator condenser suctions. While the net has proved helpful in a limited sense, it is extremely awkward to rig and does not permit underway evolutions without first proceeding to anchorage.

CIRCULATING WATER SYSTEMS

The purpose of the main Circulating Water system (Figure 9-1) is to circulate sea water in order to condense steam in the main condenser and to provide cooling water to the main lubricating oil cooler. It consists basically of two suctions, a 42-inch scoop injection and a 28-inch main circulating pump suction, associated suction piping, the main condenser, lubricating oil cooler, associated discharge piping, and a 46-inch overboard discharge. The piping is seam welded 90-10 Cu-Ni. Each suction is connected to a separate sea chest with a 1-1/2 inch steaming out connection. Construction of a typical sea chest is demonstrated in Figure 9-2. The main circulating water pump is a single-stage vertical steam-turbine-driven pump (Table 9-1). It pumps water through the main condenser at a maximum flow rate of 25,000 gpm with ship speed less than approximately 10 knots. At ship speeds greater than approximately 10 knots, the scoop injection provides sufficient flow, and the steam driven main circulating pump is secured. The scoop injection can provide flow up to 56,000 gpm.

The purpose of the turbine generator circulating water system, (Figure 9-3), is to circulate sea water in order to condense steam in the turbine generator condenser and to provide cooling water to the lubricating oil cooler and the generator air cooler. It consists basically of one 16-inch sea suction with a 1-1/2 inch steaming out connection, a circulating water

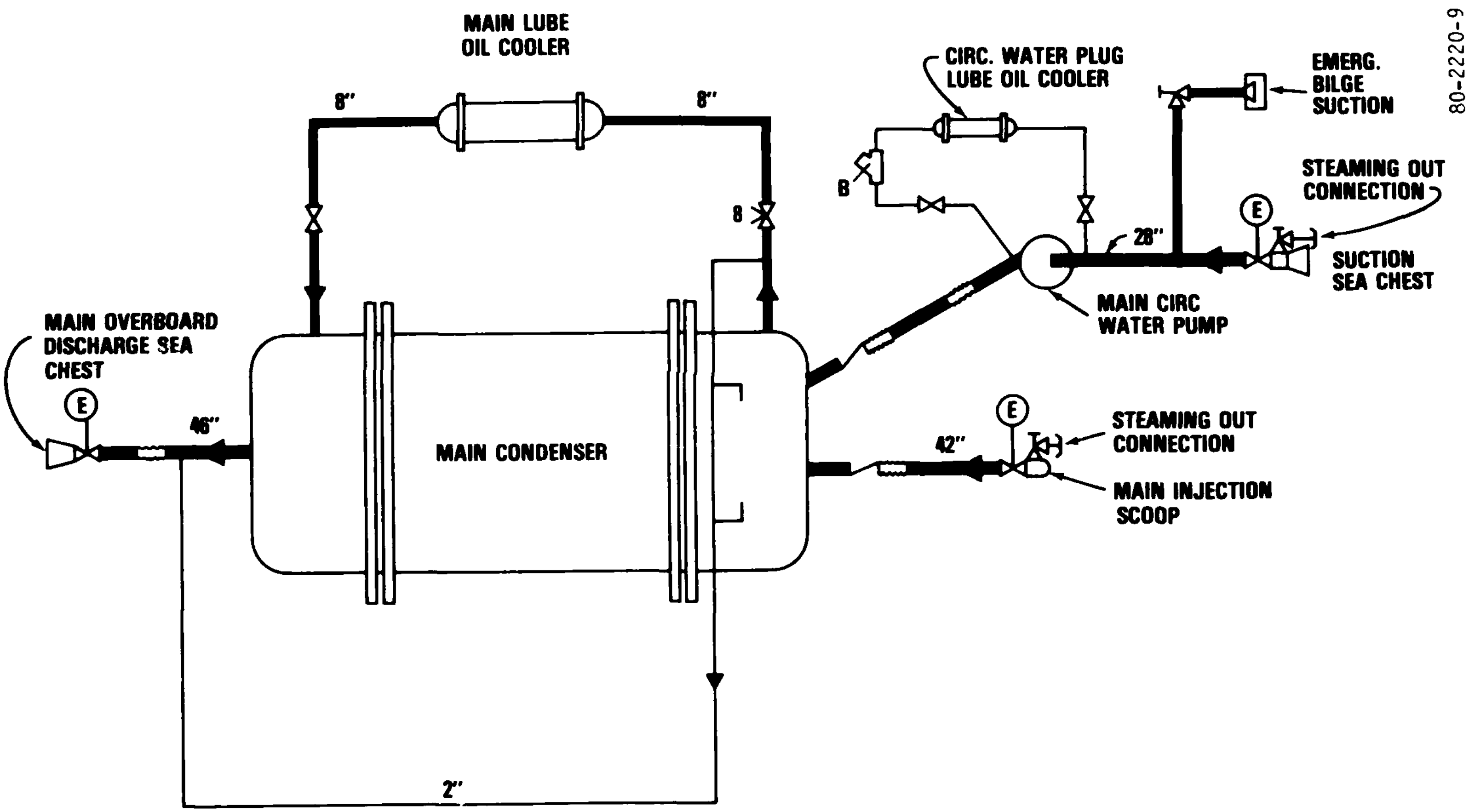

Figure 9-1. Main Circulating Water System

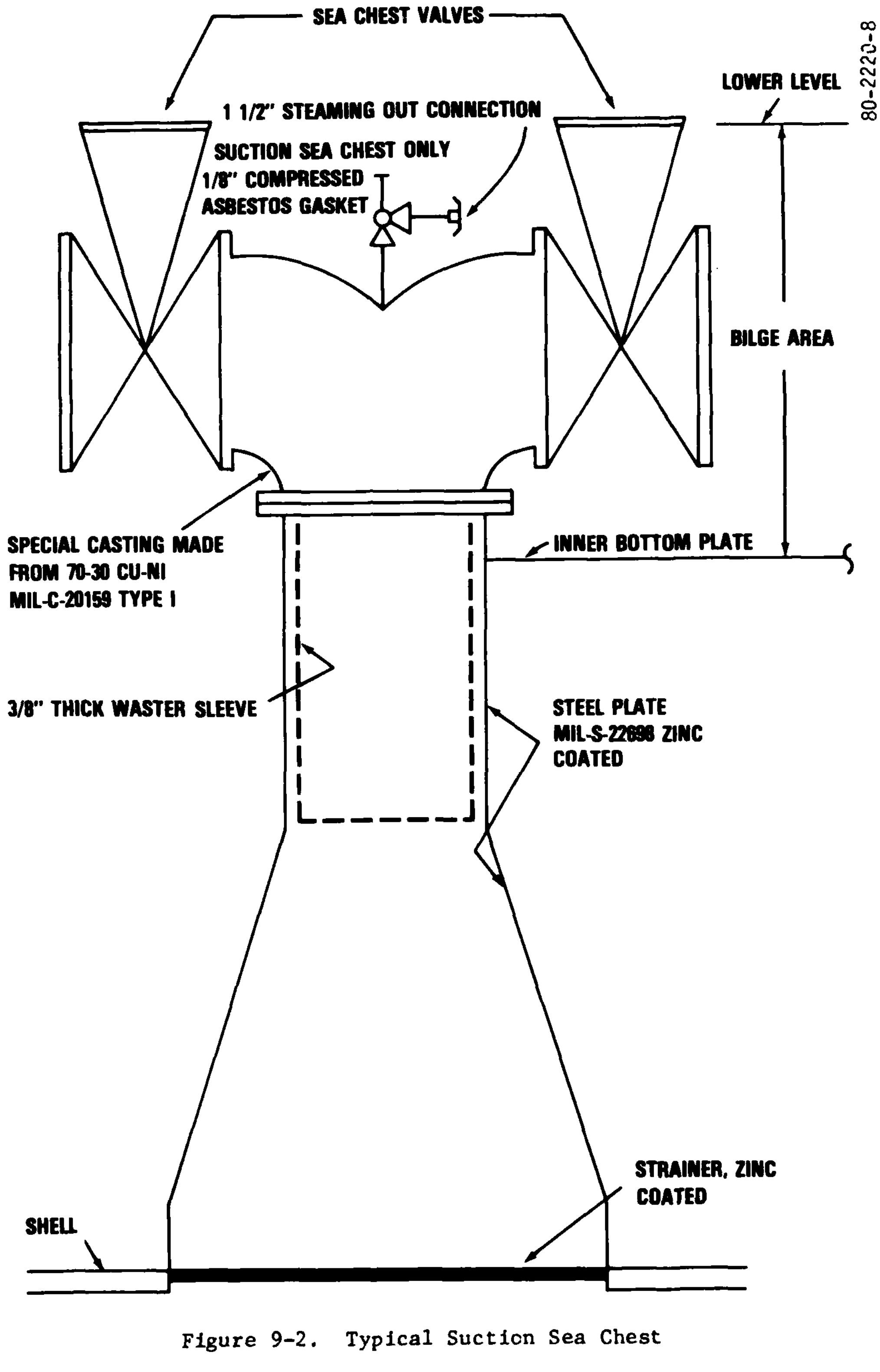

Figure 9-2. Typical Suction Sea Chest

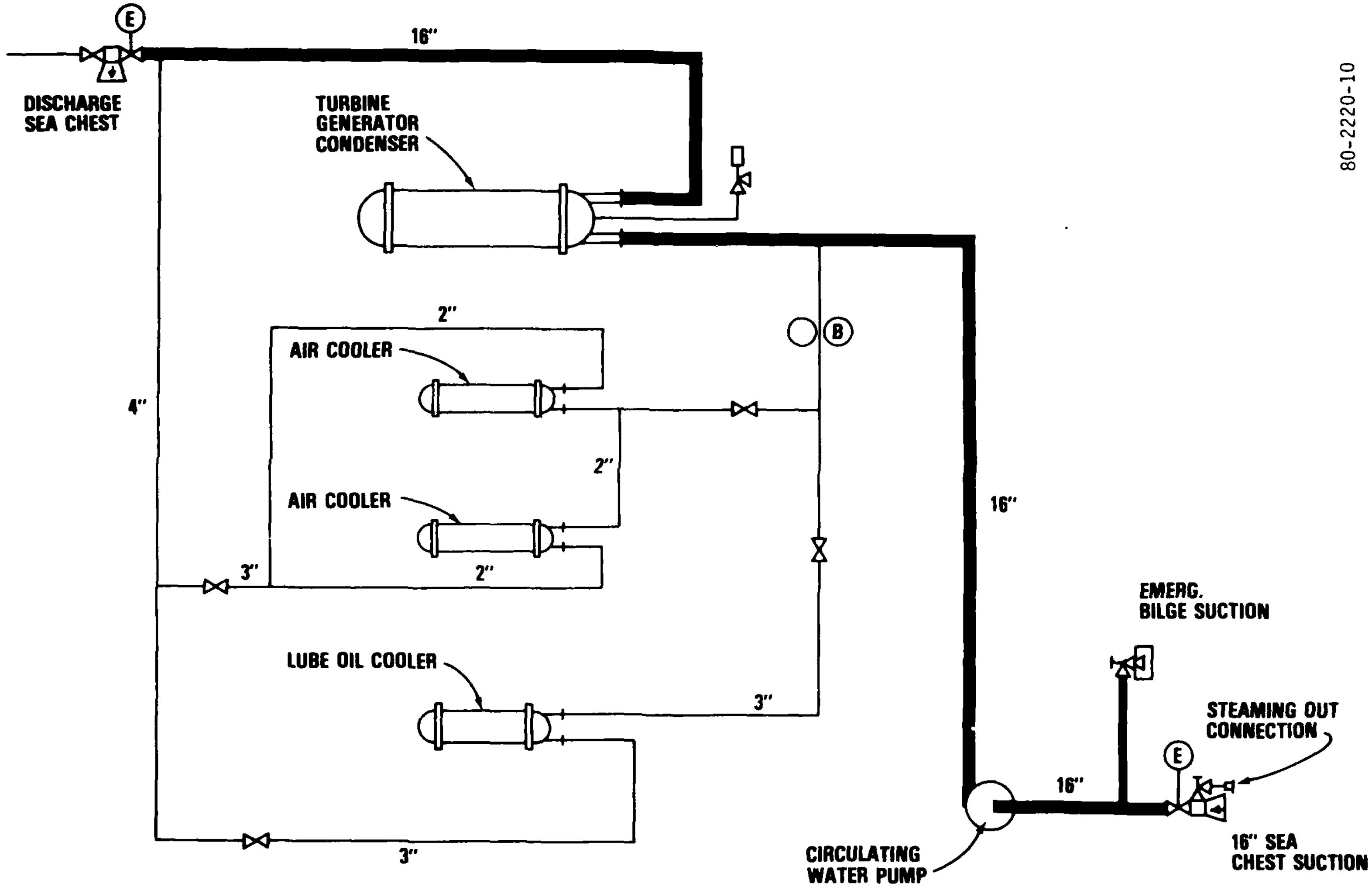

Figure 9-3. Turbine Generator Circulating Water System

pump, a condenser, and 16-inch discharge. The piping is seamless or welded 90-10 Cu-Ni. The circulating water pump is driven by a 440v, 3 phase induction motor. It is a single-stage vertical centrifugal pump with a rated capacity of about 6,000 gpm (see Table 9-1).

TABLE 9-1. PUMP SPECIFICATIONS

	Main Circulating Water Pump	TG Circulating Water Pump
Capacity (gpm)	25,000	6,000
Total Head (ft)	25	14
Speed (rpm)	760	1,175
Pump Casing	Gun-metal	70-30 Cu-Ni
Impeller	Ni-Cu	Ni-Cu

OPERATIONAL CONSIDERATIONS

Figures 9-4 and 9-5 show typical sea suction fouling on the USS EISENHOWER (CVN 69) which can occur between 1-12 hours after starting the circulating water system. Figure 9-6 is an excerpt from a message originated by the EISENHOWER which gives some insight into the operational problems that have been experienced.

The large mesh net, mentioned earlier, made of, 7/8-inch nylon. Mesh was hung under USS EISENHOWER covering two adjacient turbine generator sea suctions to determine it's ability to block Bryozoan/hydroids from condenser suction. Twenty hours of operation were conducted without clogging. For comparison, another condenser in this same vicinity, not protected by the net, became completely clogged in less than one hour. Two other turbine generators located away from the vicinity of the net were not placed into operation until well clear of Pier 12 while enroute to anchorage. Both clogged within 2 hours. Bryozoan found on condenser heads were apparently picked up from anchorage.

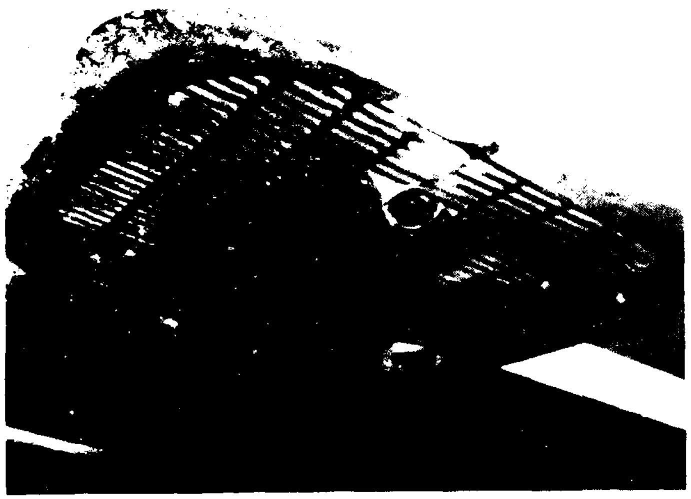

Figure 9-4. USS EISENHOWER Intake Before Starting Circulating Water System

Figure 9-5. Intake 12 Hours After Starting Circulating Water System

FM USS DWIGHT D EISENHOWER
TO COMNAVAIRLANT NORFOLK VA
UNCLAS //N09200//
CONDENSER CLOGGING 19-20 FEB 80

1. BECAUSE OF INABILITY TO ARRANGE DIVERS, THE NYLON MESH NET USED FOR JAN 80 LIGHTOFF FROM PIER 12 WAS NOT POSITIONED DURING 19-20 FEB 80 LIGHTOFF. ON AFTERNOON OF 19 FEB NR 4 COOLANT GENERATOR (4CG) WAS LIT OFF FOR GOVERNOR TESTING AND WAS OPERATED SUCCESSFULLY FOR ABOUT TWELVE HOURS BEFORE CONDFNSOR HEAD CLOGGING (VICE SUCTION CLOGGING) REQUIRED IT TO BE SECURED FOR CLEANING. ABOUT FIVE HOURS BEFORE U/W TIME NR. 3 SHIP'S GENERATOR (3SG) WAS LITOFF FOR SHIP'S POWER AND WAS OPERATED SUCCESSFULLY THROUGHOUT THE U/W EVOLUTION, BOTH 3SG AND 4 CG POSITIONS WERE LOCATED OVER AREA RECENTLY DREDGED AT PIER 12 TO A DEPTH OF FIFTY FEET.

2. IKE GOT U/W WITH 3SG ON THE LINE, REMAINING SEVEN TURBINE GENERATORS AND FOUR MAIN ENGINES WERE SECURED. AFTER BEING WELL CLEAR OF PIER 12 AND WITHIN 500 YARDS OF X-RAY ANCHORAGE ALL CIRC SYSTEMS WERE PLACED ON THE LINE. NR'S 1 AND 3 CG's CONDENSER HEADS CLOGGED WITHIN ONE HOUR (CLOGGING AGENT WAS APPROX 90 PERCENT BRYEVA) AND WERE SECURED FOR CLEANING. IKE LEFT X-RAY ANCHORAGE IN A 2CG, 2EDG LINEUP WITH NR'S 2 AND 4 SG'S BEING USED TO POWER REACTOR COOLANT PUMPS.

3. TODAY'S EXPERIENCE WOULD SEEM TO INDICATE THAT:

A. DREDGING TO FIFTY FEET PERMITS ABOUT SIX-EIGHT HOURS OF OPERATION AT PIER 12, THE SIGNIFICANCE OF THIS ABILITY WILL DEPEND UPON DENSITY OF BRYZOA AT PIER 12 WHICH WILL BE DETERMINED BY NAVFACLANTDIV SAMPLING. IF SAMPLING INDICATES THAT A HEAVY CONCENTRATION OF BRYZOA EXISTED AT PIER 12 THEN DREDGING TO FIFTY FEET WOULD APPEAR TO BE BEST ALTERNATIVE DETERMINED TO DATE.

B. BECAUSE CLOGGING HAS OCCURED DURING LAST TWO U/W EVOLUTIONS AT ANCHORAGE IT IS APPARENT THAT DEAD STICK U/W'S FROM PIER 12 ARE NOT A FINAL SOLUTION TO THIS PROBLEM.

Figure 9-6. Message from the USS EISENHOWER Giving Insight to Operational Problems

AD-A098 410 EG AND G WASHINGTON ANALYTICAL SERVICES CENTER INC R--ETC F/G 13/2
DREDGING AND SEDIMENTATION CONTROL -- 1980, PROCEEDINGS OF A SY--ETC(U)
1980 P E SHELLEY N00014-77-C-0420
UNCLASSIFIED SYMP-80 NL

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END
DATE
FILMED
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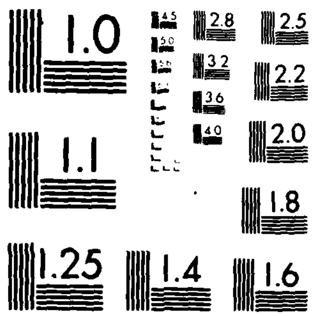
1.0
4.5
5.0
5.6
2.8
3.2
3.6
4.0
2.5
2.2
2.0
1.1
1.8
1.25
1.4
1.6

During construction, CARL, VINSON (CVN 70) has been fitted with a firemain supported sprayer head system, designed to clean the sea suction grating. This has been fitted to the number 3 ship's service turbine generator sea suction in place of the steaming out connection. Spray pattern was examined in dry dock prior to launch of VINSON. This was done using 1-1/2 inch fire hose hook-up and 90 pounds of firemain pressure. The normal configuration will be a 2-1/2 inch hose hook up with the ship's firemain pressurized to approximately 150 pounds, so that we should expect to see a more forceful flow pattern.

SECTION 10

MODELING: HOW AND WHEN TO USE

By

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INTRODUCTION

Modeling is widely employed to solve estuarine sedimentation problems. In order to decide if modeling is an appropriate approach for a particular problem, we must understand the physical processes contributing to the sedimentation problem, have a general idea of the types of remedies that might be used to solve the problem, and know the strengths and weaknesses of methods available to study the problem.

ESTUARINE SEDIMENT TRANSPORT

Estuarine sediment transport processes rank among the most poorly understood phenomena of the coastal zone, and modeling them is certainly the least precise aspect of the modeler's art. A thorough discussion of estuarine sediment transport is well beyond the scope of this paper; however, to provide a background against which to compare physical modeling methods, an outline of some important factors is presented in the following paragraphs. For more detailed presentations, see Mehta and Partheniades (1973); Krone (1972); Partheniades (1971); Ippen (1966); and Krone (1962).

Sediment characteristics and their movement within an estuary are in part functions of their source. Potential sources of estuarine sediments include: (1) the upland drainage basin; (2) the estuary itself, through erosion of banks, bottoms, and marshes; (3) the ocean; (4) municipal and industrial wastes; (5) windborne sediment; and (6) organic materials. Different sediments will tend to predominate in different parts of the estuary.

Of concern here are those fine-graned sediments (clays and fine silts) that dominate most estuaries. They are so small that they only occasionally deposit in the rivers that carry them as wash load to estuaries, where water chemistry, estuary geometry, tidal currents, and density currents combine to trap fine sediments and cause their deposition. In some locations coarser sediments are found with the fines; however, fine sediments alone cause major shoaling problems in most estuaries.

Individual clay particles will not settle under their own weight, even in still water, because their exceedingly small size allows thermal motion of water molecules to keep the particles in suspension. Only when the particles aggregate, forming porous composite particles of a number of individual particles, can settling begin. Surface electrical charges on the clay particles attract a layer of ions, making the particles mutually repulsive except at very short distaces and preventing significant aggregation. If two particles collide in spite of this repulsion, short-range attractive forces bind them tightly together into aggregates. This aggregation process, called flocculation, increases with increasing numbers of particle collisions caused by higher sediment concentration, by increased turbulence in the flow, and by the presence of dissolved salts whose ions suppress electrical repulsion between particles. At some upper limit, turbulence may hinder flocculation by breaking aggregates as rapidly as they are formed. Aggregates grow larger by colliding with individual particles and other aggregates when different settling velocities and flow velocity gradients permit them to be captured by faster-moving ones. Other aggregation processes include agglomeration by filter feeding organisms and chemical cementation. When particle aggregates have grown to a size and weight sufficient to begin settling toward the bed they become potentially depositable.

An aggregate approaching the bed can deposit if its shear strength due to interparticle bonds exceeds the shear stresses exerted on it by the shear gradient. If aggregate strength is exceeded and the bonds are broken, the resulting smaller pieces will probably be re-entrained in the flow. If an aggregate survives the high shear zone near the bed and deposits, it forms bonds with particles in the bed and is shielded from the flow by surrounding particles; thus, it tends not be eroded by the same flow conditions that allowed it to deposit. The above description of cohesive sediment behavior suggests that sediment beds that are undergoing deposition tend to do so without appreciable concurrent resuspension. This is in contrast to the live-bed concept of noncohesive sediments which is characterized by simultaneous interchange between material in transport and on the bed. The behavior of a cohesive bed with depositable sediment is a function of the bed shear stress - below a certain minimum shear, available depositable sediment will quickly deposit; above that minimum and up to some maximum shear, available sediment will deposit at a rate dependent on bed shear; above that maximum, sediment will not deposit and erosion occurs at a rate dependent upon bed shear. The critical shear stress for erosion of a sediment layer increases as the weight of overlying sediment crushes the sediment structure or as pore water is expelled, forming more particle bonds between aggregates.

Deposition of cohesive sediments is aided in several ways by estuary water chemistry. As mentioned previously, dissolved salts encourage flocculation by suppressing electrical repulsion between particles, but the importance of this effect may have often been exaggerated, since only a few parts per thousand (ppt) salts concentration as is found in many rivers is necessary to initiate flocculation. Krone (1962) found in laboratory experiments that aggregate strength was independent of salt concentration above 1.2 ppt but that median aggregate settling velocities (and by implcation, either size, shape, or density) increased with salinity up to 10 or 15 ppt. In static flocculation experiments, Sakamoto (1972) found that mean aggregate diameters increased with salinity as high as 30 ppt. He also found that illite and kaolinite aggregates exhibited increased densities in higher salinity waters by amounts several times that expected by the increased density of the entrained water. Other water chemistry effects on sediment transport may include bonding by organic constituents, water pH, and cementing of bed particles by precipitates.

Entrained water density may affect aggregate settling by the resulting slight changes in aggregate weight. Aggregates formed in low-salinity water will settle more slowly in high-salinity water until diffusion raises the entrained water salinity. Similarly, aggregates with high-salinity water entrained will more strongly resist suspension into lower-salinity surface layers.

Although dissolved salts appear to affect flocculation and settling more with increasing salinity, the effect beyond a few ppt is relatively minor compared to circulation patterns caused by the density difference between fresh and salt water. Salinity-induced density currents (predominantly upstream flow in the lower layer and predominantly downstream flow in the upper layer) is one of the most important phenomena in estuarine sediment transport. Sediments traveling downstream near the bed encounter null points, where there is no net fluid transport in either direction, and tend to be concentrated there. Sediments settling downstream of the null point are trapped in a layer with net upstream transport and are carried upstream. Thus suspended sediments are concentrated in a zone of little net transport, causing a turbidity maximum near the null point. The general area of the bottom flow predominance null point is usually a zone of heavy deposition (Simmons, 1965), though by no means is it the only zone of heavy deposition. Asymmetry of the flow's capacity to transport noncohesive sediments and erode cohesive sediments can cause the zone of heaviest deposition to be considerably upstream or downstream of the null point. Density currents also cause steep velocity gradients and flow turbulence resulting in accelerated growth of particle aggregates and therefore increase deposition.

Two additional factors, geometry and tidal flows, figure prominently in estuarine sedimentation. The most important geometric effect is the dramatic widening of the waterway that often occurs where the river enters the estuary. At this point current speeds drop considerably, and much of the noncohesive sediment load may deposit. Tidal flows add to current speeds but, because of their oscillatory nature, also provide intervals of slack currents and rapid deposition. Due to geometry, multiple channels in an estuary divert sediment and discharge in uneven ratios and experience different phasing of tidal currents, and deep channels through shallow water create pools of quiet water that trap sediment or experience strong density currents. Nonuniform geometry and manmade structures create turbulence that increases the flocculation rate. During slack-water intervals, a substantial portion of suspended sediment may deposit, requiring vigorous flows to resuspend it again. The relative scouring power of ebb versus flood flows is a determining factor in the direction of net transport at a location and in the supply of available sediment at adjacent locations.

From the preceding paragraphs, it can be summarized that estuarine sedimentation is dependent upon (1) the supply of depositable sediment and (2) flow conditions near the bed. The supply of depositable sediment is a catchall category, being a function of the character and amount of sediment, ambient water quality, and flow conditions throughout the water column. Flow conditions near the bed merely dictate whether deposition, erosion, or a stable bed will result, although they may limit what constitutes "depositable sediment." As an example of how these two criteria control sedimentation, consider first a zone of low shear stress that does not experience significant deposition either because aggregation of sediments is not occurring or because nearby deposition has exhausted the supply of depositable sediment. However, a zone may have a high average bottom shear stress but have such an abundance of depositable sediment that all of the material deposited during slack-water periods cannot be eroded during strong flow periods. Thus, alteration of either sediment supply or flow conditions can significantly change patterns of deposition and erosion.

Transport of cohesive and noncohesive sediment shares this dependence upon the balance of bed shear against sediment supply but differs in that noncohesive sediment beds may experience simultaneous large-scale erosion and deposition while cohesive sediment beds may not. They also differ in that available noncohesive sediment tends to be depositable, while available cohesive sediment may not be and require a certain level of aggregation in order to become depositable.

SOLUTION METHODS

Four primary methods are available for studying estuarine sedimentation - field methods, analytical methods, physical modeling, and numerical modeling.

Field Methods

Field methods include trial and error remedial measures, in which proposed remedial works are constructed without benefit of corroborating study, full scale experiments, such as those conducted by Scripps Institute of Oceanography at The Mare Island Naval facility, California, and field measurement of physical parameters.

Field methods are usually quite expensive, but are often indispensable. Full scale experiments can be a very useful means of checking basic designs and procedures under realistic environmental conditions that are not foreseen otherwise, and field measurements are necessary for the understanding of physical processes mentioned above plus verification of models if they used. Field methods are sometimes limited in that boundary conditions of flow and sediment supply may vary significantly during tests, preventing comparisons of one data set with another.

Analytical Methods

Analytical methods employ simple mathematical expressions for which a closed form solution can be obtained. They usually lump several processes into a empirical coefficient. An example is Manning's equation, which is a simple analytical model of the complex process of energy loss in open channel flows. A more vigorous and complete analytical model is included in the turbulent versions of the Navier-Stokes equations, but they cannot be solved by ordinary analytic means. Analytical methods are inexpensive but cannot provide many details. Their usefulness declines with increasing complexity of geometry or increasing detail of results desired.

Physical Modeling

Physical scale models have been used for many years to solve coastal hydraulics problems. Careful observance of appropriate scaling requirements permits the physical modeler to obtain reliable solutions to problems that often can be solved no other way. Physical hydraulic models of estuaries can reproduce tides and other long waves, some aspects of short-period windwaves, longshore currents, freshwater flows, pollutant discharges, some aspects of sedimentation, and three-dimensional variations in currents, salinity, density, and pollutant concentration. Present practice does not include simulation of water-surface setup and currents due to wind. Applicability of model laws and choice of model scales are dependent on which phenomena are of interest. Conflicts in similitude requirements for the various phenomena usually force the modeler to neglect similitude of some phenomena in order to more accurately reproduce the dominant processes of the situation. For example, correct modeling of tides and currents often requires that a model have different scales for vertical and horizontal lengths. This geometric distortion permits accurate reproduction of estuarine flows and is a common and acceptable practice; but it does not permit optimum modeling of short-period waves, which requires an undistorted model for simultaneous reproduction of refraction and diffraction.

Numerical Modeling

Numerical modeling is a relatively new technique that employs special computational methods such as iteration and approximation to solve mathematical expressions that do not have closed form solutions. A numerical model thus applies numerical (computational) analysis to solve mathematical expressions that describe the physical phenomena. The distinction between analytical solutions obtained by computer calculations and numerical modeling solutions may become blurred, but the distinction is a valid one that should be maintained.

Numerical models used in coastal hydraulics problems are of two principal types - finite difference and finite element. The finite difference method (FDM) approximates derivatives by differences in the value of variables over finite intervals of space or time. This requires discretization of space and time into regular grids of computation points. Finite difference methods have been in widespread use for unsteady flow problems for about 15 years. The finite element method (FEM) has only recently begun to be applied to hydrodynamic problems. This method employs piecewise approximations of mathematical expressions over a number of discrete elements. The assemblage of piecewise approximations is solved as a set of simultaneous equations to provide results at points in space and time.

Numerical models are classified by the number of spatial dimensions over which variables are permitted to change. Thus in a one-dimensional flow model, currents are averaged over two dimensions (usually width and depth) and vary only in one direction (usually longitudinally). Two-dimensional models average variables over one spatial dimension, either over depth (a horizontal model) or with width (a vertical model).

Numerical modeling provides much more detailed results than analytical methods and may be substantially more accurate, but it does so at the expense of time and money. Once a numerical model has been formulated and verified for a given area it can quickly provide results for different conditions. In addition, numerical models are capable of simulating some processes that cannot be handled in any other way. However, present models are limited by the number of dimensions and degree of resolution that are practical on available computers. They are also limited by the modeler's ability to derive and accurately solve mathematical expressions that truly represent the physical processes being modeled.

The Hybrid Model

The preceding paragraphs have described the four principal solution methods and some of their advantages and disadvantages. In practice, two or more methods are used jointly, with each method being applied to that portion of the problem for which it is best suited. For example, field data are usually used to define the most important processes and to verify a model that predicts hydrodynamic conditions in an estuary. Combining two or more methods in simple ways has been common practice for many years. Combining physical modeling and numerical modeling to provide results not possible any other way is termed a hybrid modeling method, and combining them in a closely coupled fashion that permits feedback among the models is termed an integrated hybrid solution.

Judicious selection of solution methods in a hybrid approach can greatly improve accuracy and detail of the results. By devising means to combine results from several methods, the modeler can include effects of many phenomena that would be neglected or poorly modeled by a single approach. For example, as described earlier, physical model scaling requirements for tidal flows and short-period waves conflict, and both are included in models only at the expense of imprecise modeling of one or the other. By modeling them separately and integrating the results, the modeler can predict both to the best of his modeling capability. Thus, the hybrid approach exploits strengths of each solution method while avoiding weaknesses.

PHYSICAL MODELING TECHNIQUES

Physical hydraulic models are capable of reproducing most, but not all, important hydrodynamic phenomena influencing sediment transport. Tidal, freshwater, and salinity induced densimetric currents can be modeled. A model verified to accurately reproduce observed salinities is assumed to correctly model other dispersive transport in the salinity intrusion region. Geometric influences on current directions and magnitude are modeled. Phenomena usually not modeled include wind-induced currents and locally generated wind waves, though the latter can be simulated if they are known to be important and their effect can be defined. Occasionally other influences such as ship transit may have sufficient impact to require simulation. Herrmann (1979) provides a very thorough description of physical modeling practice and theory.

Most physical model studies of estuarine sedimentation consist of sediment tracer tests in fixed-bed models, which have a bed of molded concrete. The tracer is usually light-weight plastic granules or finely-ground gilsonite. The shoaling verification for a physical model is a trial and error process to develop the test procedure that results in the most accurate reproduction of the observed location and distribution of shoaling in the field.

The calibration of a model tracer procedure typically consists of attempting to reproduce typical hydrodynamic and shoaling conditions for a period of time for which prototype hydrographic or dredging data provide an estimate of deposition and erosion in the area of interest. Adjustments are made to the model and test procedures until the tracer distribution is similar to the shoaling volume distribution observed in the prototype. Adjustment may include changes in one or several of the following:

a. Size of model sediment particles
b. Specific gravity of model sediment
c. Rate of tracer injection
d. Location of tracer injection
e. Times of tracer injection
f. Roughness element arrangement
g. Test duration
h. Tidal range
i. Freshwater discharge
j. Water salinity

In addition, it may be necessary to simulate some unusual phenomenon which exerts a major influence on the sedimentation process (e.g., the resuspension of sediments in the navigation channel resulting from ship passages). When model conditions and test procedures that satisfactorily reproduce available prototype sedimentation data are developed, the model is considered to be verified.

The gilsonite technique cannot, of course, reproduce flocculation of clay sediments. It therefore will not directly model changes in the supply of depositable sediment due to increased flocculation rates caused by geometry, structures, and other shear-producing factors. These must be simulated by a sediment injection procedure that alters the supply available for deposition. A particular tracer grain size may satisfactorily replicate the settling velocity of a particular class of sediment aggregates, but finding the proper tracer is an empirical process and not easily subject to variation over the model.

Since the model tracer is not cohesive, erosional and depositional criteria are altered from the prototype. The model currents and sediment are such that transport occurs close to the bed, and the rate of transport is proportional to the shear stress (when in excess over the critical value for initiation of motion) in contrast to that described for cohesive sediments in the prototype in which particles, once eroded, are transported at a rate dependent only on their concentration and the current speed until they approach the bed and bed shear stresses permit redeposition.

What the modeler must achieve is some correspondence between the model's ability to transport and deposit available noncohesive sediment and the prototype's ability to transport and deposit cohesive sediment. Obtaining that correspondence is intricately involved in the ten adjustments listed above. Adjustments of sediment injection location, rate, time, and duration may be necessary to obtain the proper sediment supply. However, there is a danger that the model tracer supply can be arranged so as to compensate for inaccurate hydrodynamic reproduction. Knowledge of the modeled estuary sediment sources, suspended sediment concentration patterns, and estuarine sedimentation processes must be applied to insure that the model tracer injection procedure does not force the model to reproduce prototype shoaling patterns without some correlation to the transport processes. For example, it would be reasonable to increase the injection rate in a region where a nearby flow construction could be expected to increase internal shear and thus the flocculation rate. It would not be reasonable to reduce the injection rate in an area where high current speeds in the prototype prevent deposition.

Careful attention to hydrodynamic verification is a prime requisite for avoiding errors that must be compensated for by tracer injection, but some adjustments to hydrodynamic verification conditions may be necessary to obtain shoaling verification. Changes may be necessary to obtain typical transport conditions, to improve hydrodynamic reproduction in areas between data stations, or to slightly alter the behavior of the model tracer. These changes can contribute to improved sedimentation simulation without sacrificing the model's faithful reproduction of prototype hydrodynamic behavior, but care is required. Just as with the tracer injection rate, deciding what constitutes valid hydrodynamic alterations requires knowledge of transport processes, estuary characteristics, and model behavior. For example, changing test duration, tidal range, and freshwater inflow within reasonable limits can be necessary to produce transport conditions in the model that represent typical transport conditions in the prototype. Changing ocean or inflow salinity can be used to slightly change tracer submerged weight, and thus settling velocity and erodibility. Re-arrangement of roughness elements within a reach of the model is commonly necessary since hydrodynamic verification requires only that average energy dissipation between data stations be correct. Roughness redistribution may therefore be an extension of hydrodynamic verification; however, adding substantial amounts of additional roughness is not likely to be a valid extension.

Physical model reproduction of estuarine sediment transport is imprecise and is by necessity an empirical procedure requiring considerable knowledge and judgement of the modeler. Alterations to model conditions and procedures listed in the adjustment procedures and described above can be meaningful and necessary to obtain adequate simulation of prototype behavior, but there is a real danger that they may force the model to produce desired depositional patterns without reproducing similar patterns of sediment transport. If this occurs, the model will be unable to respond to alterations in the same way as the prototype, rending it useless as a predictive tool. As model adjustment procedures become more extreme and as proposed estuary alterations have greater impact, the ability of the model to predict changes in sedimentation can become poorer. Determining the limits of reasonable model predictions is the object of a research program currently underway at the U.S. Army Engineer Waterways Experiment Station.

A consideration of importance for physical models using a fairly uniform size tracer material to represent a graded prototype material is the possible alteration in supplied sediment distribution caused by alteration of the estuary. If the distribution of sizes in the prototype shoaling material changes drastically with some modification to the estuarine system, the model will be taxed to adequately predict when verified to a fixed size fraction. There is not much that can be done to anticipate such a change and vary the model testing procedure accordingly, as the phenomena of sorting are very complex. This concept must, however, be kept in mind in interpretation of model results.

Herrmann (1979) cites numerous case studies of physical model sedimentation studies designed to solve problems of channel and harbor shoaling, dredged material disposal, design of control, structure works, etc. Reports by Letter and McAnally (1975, 1978, 1980) describe the results of follow-on studies which examine the results of physical model studies in light of later prototype behavior in order to define how well the models were able to predict estuary behavior after improvement plans were installed. In general, it was found that the reviewed models' qualitative predictions were correct but that careful interpretation of model quantitive results was necessary to avoid misleading conclusions. In addition, it was noted that a model can be pushed beyond the limits of its verification to prototype data if proposed alterations are too extensive. Careful, informed analysis of model results is a prerequisite to their beneficial use.

NUMERICAL MODELING OF ESTUARINE SEDIMENTATION

Numerical modeling of sediment transport, deposition, and erosion in estuaries in a new field of effort. Several experimental models have been developed, but in the United States only two are presently being used in a production mode to solve engineering problems. The first is STUDH, an advanced revision of SEDIMENT II (Ariathurai, MacArthur, and Krone, 1977), and the other is SERATRA (Onishi and Wise, 1978). Both employ finite element solutions of the convection-dispersion equation for sediment transport. Both models also require that hydrodynamic results be generated externally, and supplied to the sediment model as input.

The numerical model STUDH is being extensively applied by the Waterways Experiment Station, both in completely numerical and in hybrid modeling approaches. One hybrid modeling effort is that being applied to the Columbia River estuary. (McAnally, Thomas, and Letter, 1980).

The Columbia River enters the Pacific Ocean through its estuary between Oregon and Washington in the United States. The estuary is about 40 mi. (64 km) long and 2 to 9 mi. (3 to 14 km) wide. Depths range up to 100 ft (30m). Tides are mixed, with a mean diurnal range of 7.5 ft (2.3m). Mean freshwater discharge is 260,000 cubic feet per second (cfs) ($7400 m^3/s$). The wave climate at the mouth is rather severe, with wave heights over 20 ft (6m) not uncommon during the winter. The estuary's bed is predominately fine sand with a median grain size of 0.2 to 0.3 mm.

The U.S. Army Corps of Engineers maintains a 48-ft deep by 2600-ft wide (14.6m by 790m) navigation channel through the estuary entrance, which is protected by jetties. Annual maintenance dredging of the entrance amounts of about 4.8 million cubic yards (3.7 million cubic metres) has led the Corps to seek means of reducing shoaling in the entrance channel. They have sought to determine optimum jetty lengths with previous studies employing tracer tests in a fixed-bed, distorted-scale model. Recently the Corps has begun reevaluating optimum jetty length using the hybrid modeling method.

The Columbia River estuary hybrid modeling approach uses a large physical model to predict water surface elevations, current velocities, and salinity intrusion for various jetty lengths, a numerical wave refraction/diffraction model to predict wave conditions, and analytical techniques for computation of longshore current and river discharge. These hydrodynamic predictions are used to drive the numerical model STUDH for sediment transport, deposition, and erosion.

Results to date in The Columbia hybrid modeling effort have shown that the method is able to model more of the important processes and model them better than any single modeling technique or simple collection of techniques.

The Columbia application has been so successful that the Waterways Experiment Station is now applying the hybrid modeling method to estuarine sedimentation problems on the Gulf and Atlantic coasts also.

WHEN SHOULD MODELING BE USED?

Modeling should be employed as a solution method when (a) the cost or importance of the problem justifies the time and cost of modeling, and (b) the dominant processes involved are susceptible to modeling. Most estuarine construction projects are either very expensive or possess the potential for expensive side effects. If a project falls into either category, modeling should be explored as a precaution. At that point, a person knowledgeable in models can determine the appropriateness of modeling for the problem at hand and prepare a time and cost estimate for a model study of the problem. The resulting estimate can then be compared to the cost of construction or of unforseen impacts for evaluation of feasibility.

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SECTION 11

ALTERNATIVES FOR SEDIMENTATION CONTROL AT THE PIER 10-11-12 COMPLEX, NORFOLK NAVAL STATION

By

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INTRODUCTION

Norfolk Naval Station has a combined sediment and condenser fouling problem. The sedimentation rate is comparable to record rainfall conditions at Mare Island. When Pier 11 is constructed, sedimentation and hydroid problems will double. The preponderance of suspended sediment, as well as hydroid population are found near the bottom. Therefore, the principles of flushing and exclusion which have proven successful at Mare Island during record rainfall should be transferrable to the similar environmental conditions found at Norfolk.

FLUSHING

The sweeping jets which have worked so well at Mare Island Naval Shipyard may also be well-suited to sweeping both sediment and hydroids out from beneath aircraft carriers. The jet arrays have successfully prevented mud deposition out to 100 feet from the jet nozzle. The power requirements of these fixed arrays begins to become excessive for larger radii of protection as required under a CV or CVN. To extend the 5 dyne/cm^2 stress radius to protect larger vessels, especially while drawing condenser water prior to sailing, we propose deployment of a mobile sweeping jet (Figure 11-1). Each mobile jet would sweep along 180° arcs on the ends of a submerged boom attached to the finger pier. The boom and jet may be retracted under the pier during docking to prevent damage from dragging anchors. The sweeping arcs may be sequenced to either sweep hydroids and sediment clear of a moored vessel or into a crater sink for active removal.

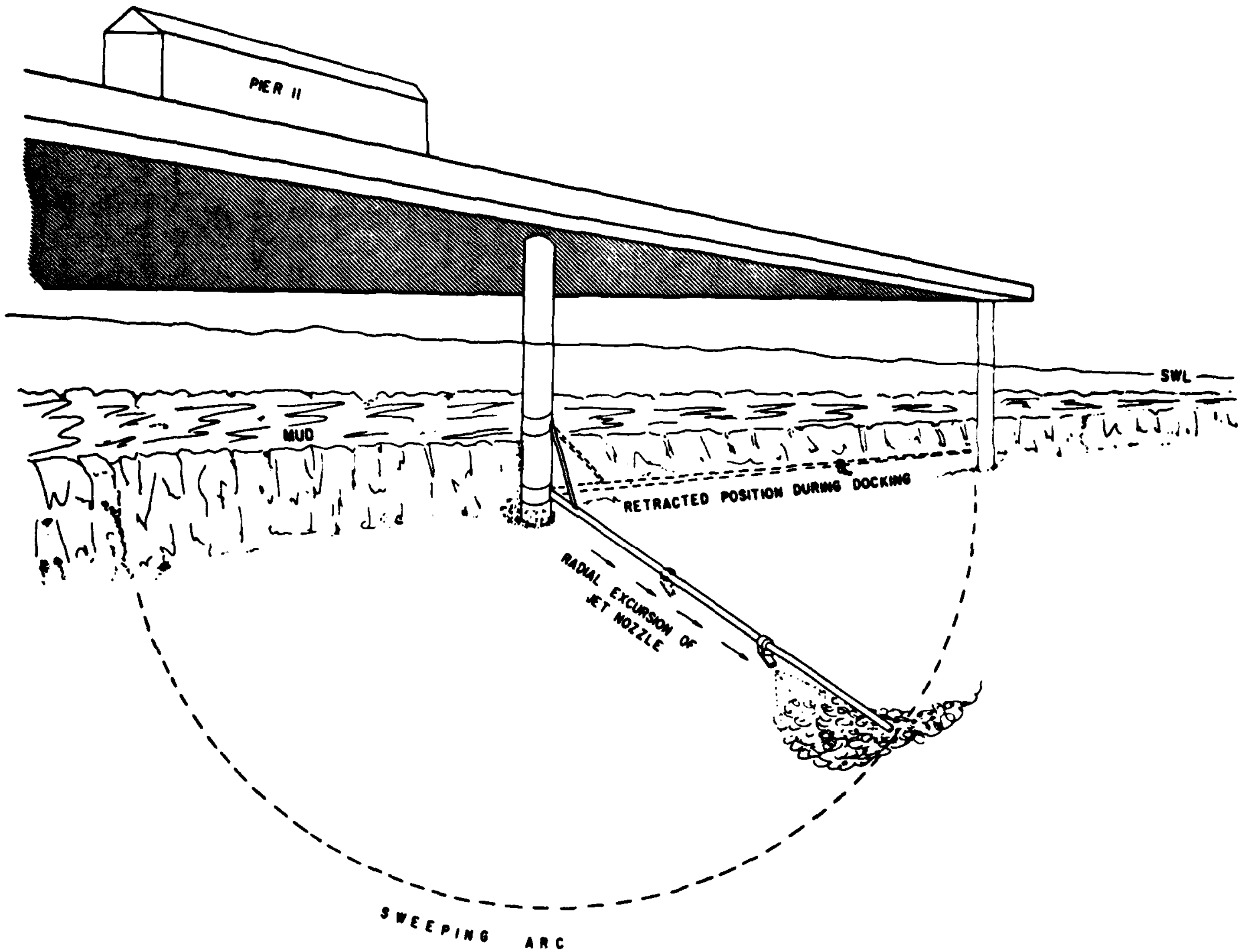


Figure 11-1. Mobile Sweeping Jet to Extend Effective Protection Radius

EXCLUSION

The large berthing areas, particularly the proposed Pier 11 complex, present an attractive application for the full height barrier curtain tested at Mare Island. Pier 11 would afford a hinge point for two curtains protecting the berths between Piers 10 and 11 and Piers 11 and 12 (Figure 11-2). These curtains would be 2 and 3 times respectively the length of the Mare Island curtain. Perhaps a certain amount of refining of the existing curtain mooring system may be necessary to repress flutter (e.g., a waving flag) of these larger curtains, and stabilization of the mud bank under Pier 12 may be required. Otherwise there are no significant environmental or operational differences known at this time that could frustrate implementation of a curtain system at Norfolk. Indeed, the pay-back time of a curtain system applied to such large, deep draft berths would be short from the mud standpoint alone, with the added benefits of hydroid protection and improved diver visibility.

EXCLUSION WITH FLUSHING

A combination of bottom mounted sweeping jets and a tidal lift barrier curtain, as represented schematically in Figures 11-3 and 11-4, offers another possibility for protecting carrier berths at Norfolk. The curtain is adjusted to rise off the bottom at high tide. As ebb tide ensues, the bottom jets blow sequentially, advecting the fluid mud out under the curtain. After flushing the berth in this manner, the curtain may be lowered and raised occasionally for maintenance flushing as required by seasonal variations in run off.

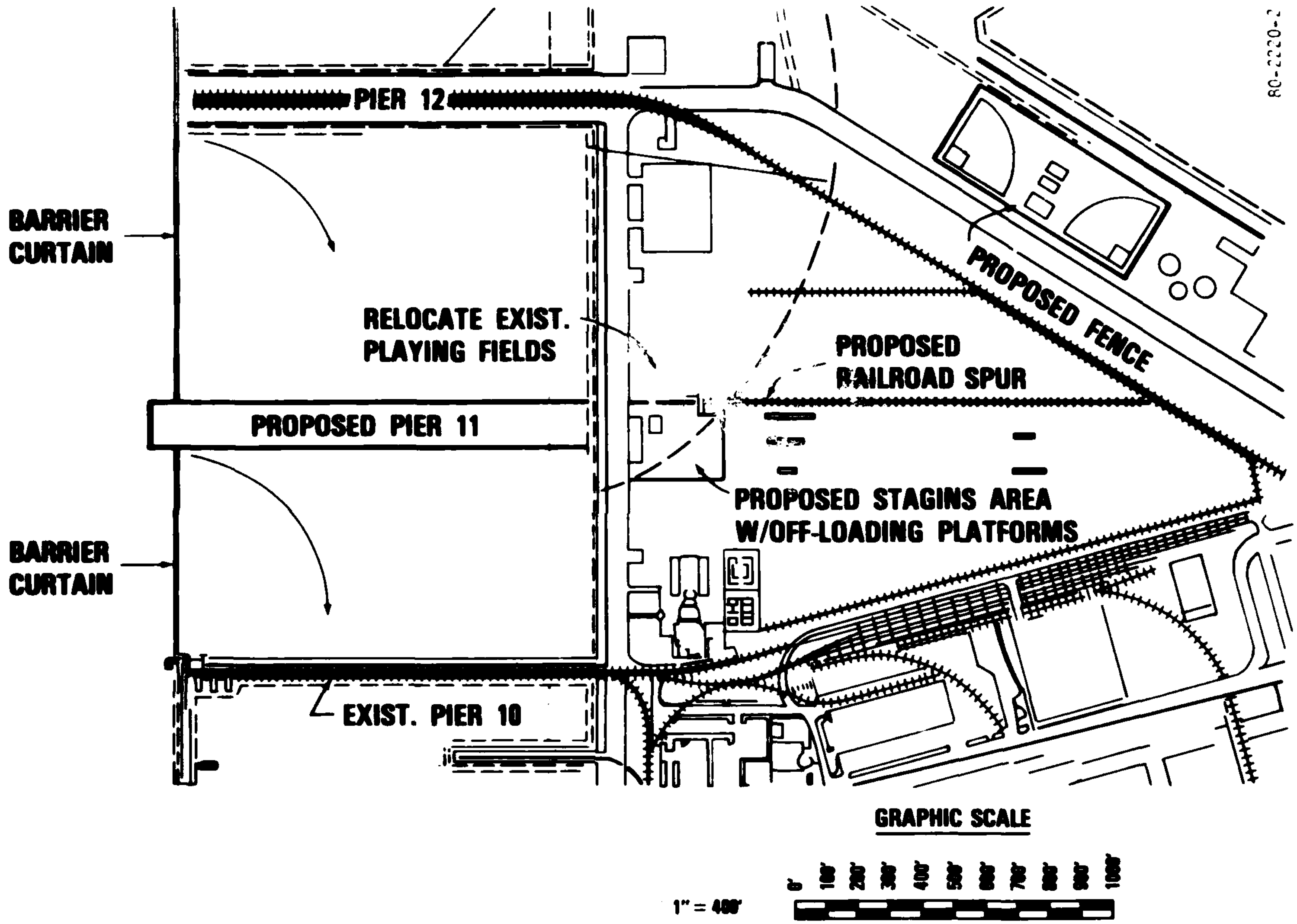
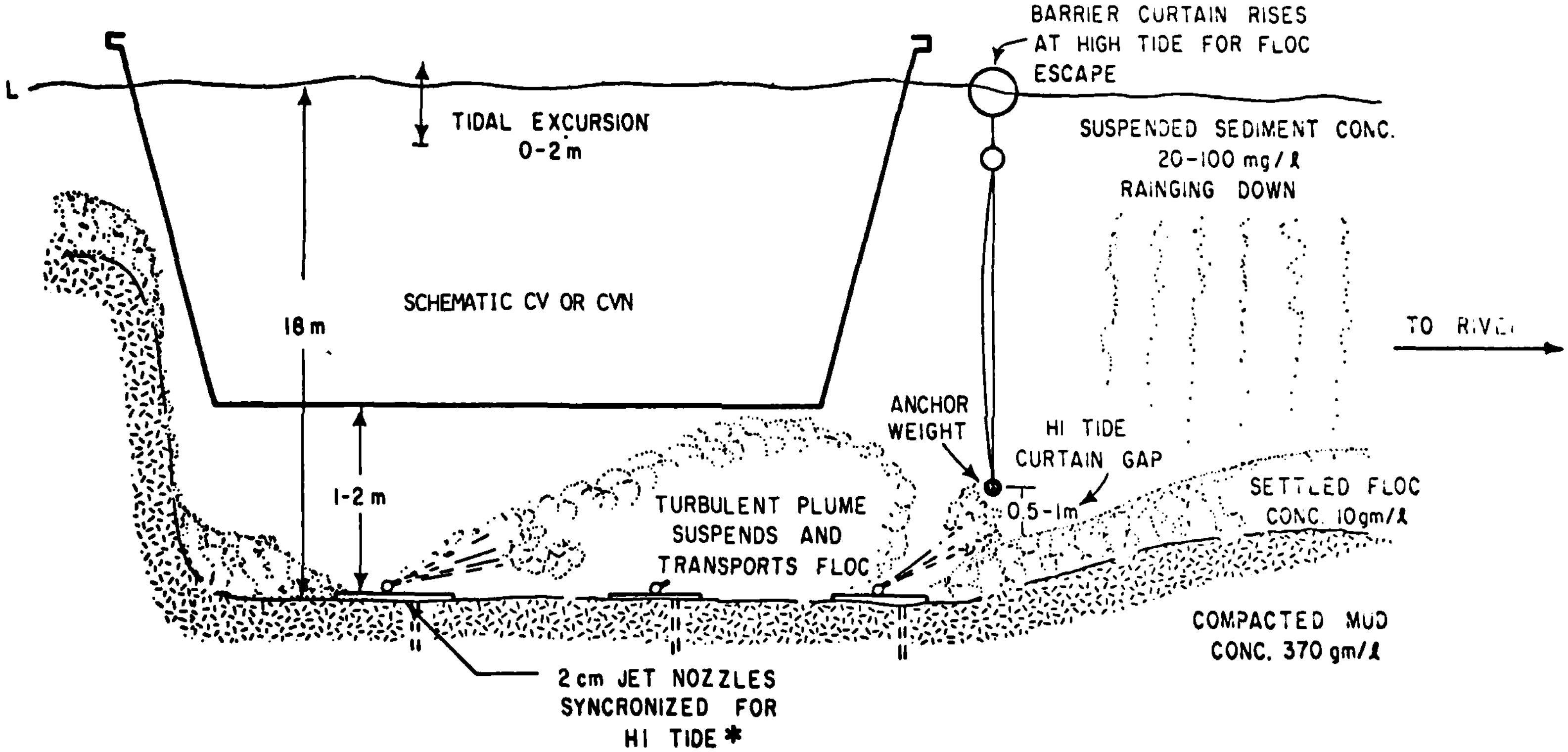
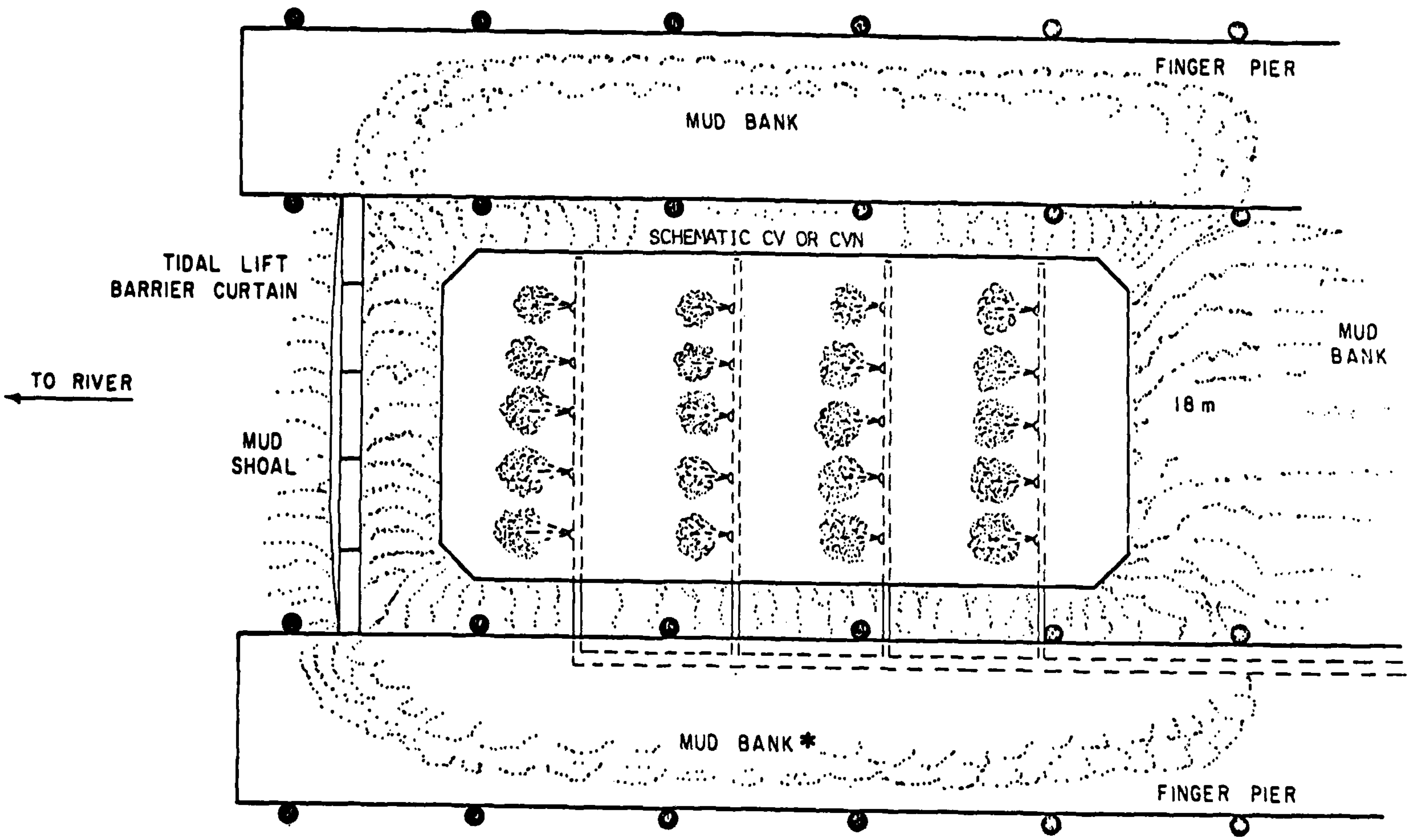


Figure 11-2. Norfolk Naval Station Proposed Pier 11 Complex with Barrier Curtain Protection Against Mud and Hydroid Intrusion



* Water jets are actuated immediately after hi-tide to sweep fluid mud and organisms out under the tidal lift curtain.

Figure 11-3. Schematic Representation of a Water Jet Flushing System and Barrier Curtain to Protect a Carrier Dredge-Hole from Sediment and Hydroid Accumulation



* (The mud banks provide the lateral seal to the berth up to mean lower low sea level. The water jets flushing of the berth may be shut off and the curtain bottom fully lowered for long closure intervals.

Figure 11-4. Plan View of the Automatic Flushing Curtain System for a Carrier Dredge-Hole

SECTION 12

MAYPORT BASIN FIELD TEST

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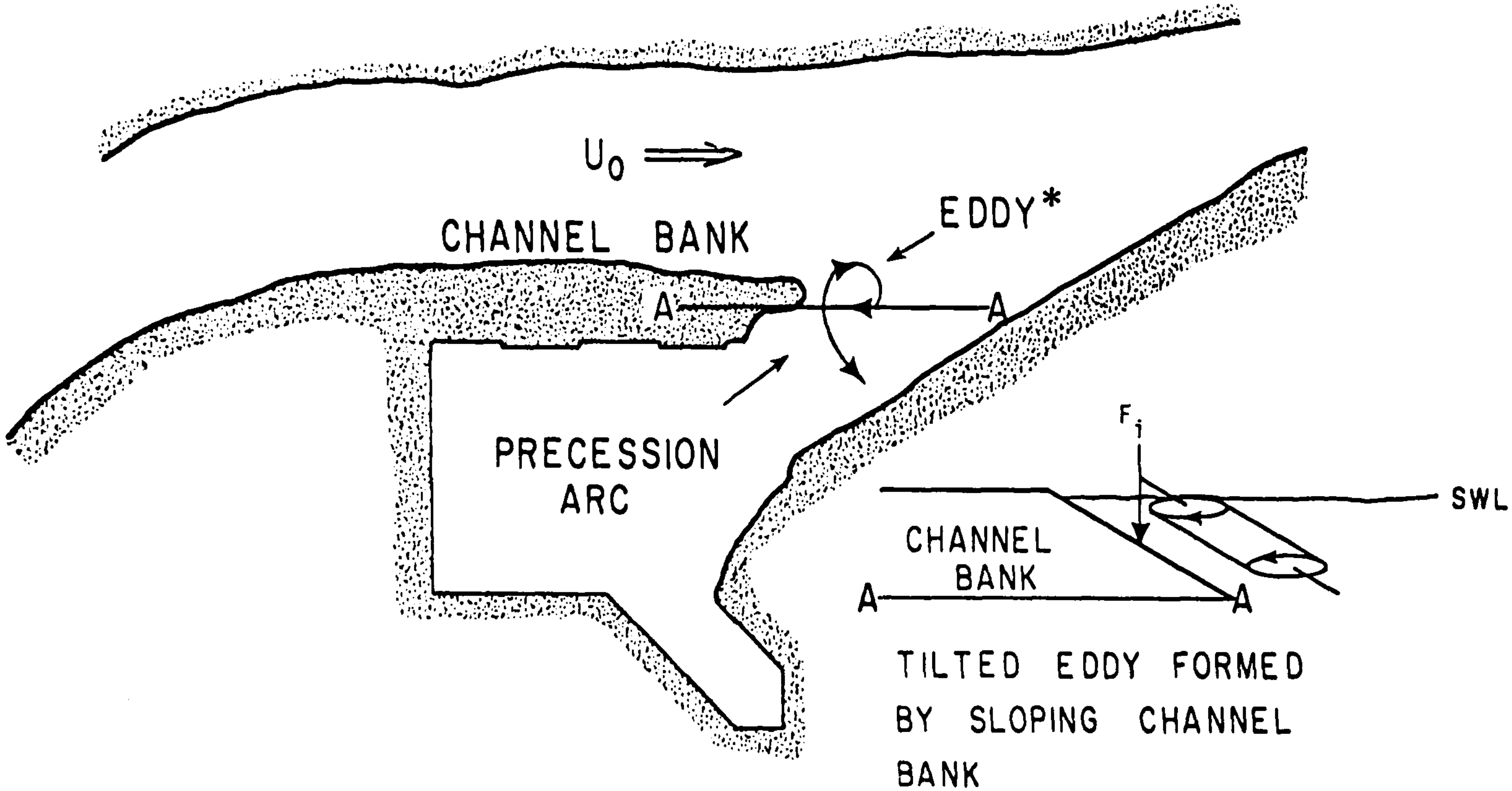
A field survey of the Mayport turning basin by the Shore and Coastal Processes Engineering (SCOPE) team uncovered a transport mechanism for fine sediments which was not considered in the COE's Mayport-Mill Cove study, see COE, WES, (1978), "Mayport-Mill Cove Model Study", Report I. It was found that the residence time of the water in the basin was sufficiently long so that even if all the suspended sediment observed fell to the bottom, the resulting accumulation would not account for the observed shoaling. The only apparent transport mechanism to account for the shoaling is the large eddies observed to form at the entrance channel during strong tidal flow. Although the SCOPE team report (Van Dorn, 1979) does not dispute the COE's recommended remedial measures, these measures were not based on an understanding of the eddy transport mechanism apparently at work in the full scale basin. In view of the tremendous costs required to implement any of the COE's recommendations, a follow up SCOPE team investigation will be made of these eddies during FY 80 to possibly discover more cost effective solutions.

The decisive question to be answered in the forthcoming field study is how the eddies' shed at the channel entrance manage to turn into the basin. Figure 12-1 presents a likely mechanism which may be exploited to solve the sedimentation problem. The channel bank at the turning basin entrance has a sloped bank from which an inclined eddy would be shed. Like a tornado, the eddy will vacuum material off the bottom, such as fluid mud and salt water, causing the eddy to have a net immersed weight, indicated by F_i. Like a tilted top, the inclined eddy will precess. For the sense of circulation, this precession will carry the eddy into the turning basin, preventing it from being carried down the St. John's River. To prevent this the immersed



* The separation of flow off the sloped bank forms a tilted eddy. The eddy intrains salt water and fluid mud giving it a net immersed weight , F_i. The immersed weight of the eddy acting on it's tilted axis causes it to precess like a top into the channel entrance where flood tides and lateral boundary friction carry it into the basin.

Figure 12-1. Schematic Illustration of an Eddy Transport Mechanism in Mayport Tu.:ning Basin weight of the eddy must be annihilated. This should be possible by the operation of a relatively inexpensive water on air jet system circulating vertically on the slope of the channel bank.

SECTION 13

TEST AND EVALUATION PLANNING

By

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Treat nature in terms of the cylinder,
the sphere, and the cone, all in perspective.
Paul Cézanne

INTRODUCTION

As can be realized by reflecting upon the presentations already made at this symposium and, especially, upon the discussions, this assembly brings a rich diversity of viewpoints to bear upon the problem of dredge sedimentation and marine organism control. And this is as it should be since, in order for our solutions to be of maximum efficiency, they must reflect the perspectives of all involved. Furthermore, planning is not a force but a process, so it is important that persons representing all operative perspectives be involved. A swamp may be viewed as an opportunity for study and learning to the naturalist who sees the elaborate ecological tapestry that nature has woven, as a challenge to the developer who sees its possibilities once it has been drained and filled, and as home to the alligator who has his own way of challenging the perspectives of either of the first two who might trespass on his domain.

I would like to begin my comments by considering three questions, the answers to which will be somewhat shaded by the perspective of the responder, recognizing that one's viewpoint is shaped by where one has been and is focused on where one is going. These questions are: Why test and evaluation (T&E); why planning; and what is the role of research, development, test, and evaluation (RDT&E)?

WHY TEST AND EVALUATION?

If there is one point upon which all attendees at this symposium can agree, it is that our level of knowledge in the subject area is imperfect at best. As noted by several of the earlier speakers, the mechanics of sedimentation are among the most complex phenomena man has attempted to deal with. A complete description of the sedimentation process remains, since it must encompass difficult subjects ranging from very complex hydrodynamics to the thixotropic world of colloidal chemistry. When one adds the biological complications as an overlay, the problem of sedimentation and marine organism control becomes formidable indeed.

Faced with such difficulties, progress is often best made through the iterative cycle of study, conjecture, and experiment, which is what test and evaluation is all about. In short, we must inquire directly of nature to get the missing information and understanding necessary to enable us to formulate workable solutions and, when we are inquiring of nature, we are engaged in test and evaluation whether it be a laboratory flume, a physical model, or a prototype scale experiment in the field that is our learning vehicle. Thus, one answer to the question of why T&E is to learn new information that will aid in the development of problem solutions and increase our understanding of the subject area.

Even when one is dealing with reasonably well understood physical phenomena, however, there is still a need for test and evaluation. Seldom is conjecture alone sufficient to determine the operability characteristics of a proposed technological solution. The horizon of concern must now be widened to include human factor considerations, possible malfunctions and the ease of repair, training requirements, and the like. In addition, the time and space scales of solution technology application are generally greater than those of initial inquiry, and must be properly accounted for, all of which require test and evaluation. Therefore, another answer to the question is to demonstrate.

Finally, even with a solution in place there is a need to monitor its performance to determine the degree of problem amelioration being obtained or, bluntly, how well it is working; degradation of performance over time; and such. This is especially true in a long-term area like sedimentation control where the goal might have been to decrease the frequency of required dredging from once every two years to once every four years. Thus, to prove performance is a third answer.

The three answers given so far to the question of why T&E, namely, to learn, to demonstrate, and to prove, could be augmented by others but should be sufficient to make the point that while not mutually exclusive, they represent somewhat differing viewpoints; in this case those of the inquirer/scientist, the designer/engineer, and the user/operator. All are necessary, but they must be integrated in a holistic fashion. Which brings us to the second question.

WHY PLANNING?

Here again, the answers tend to reflect the viewpoints of the responders. Planning is considered by some as that which makes order out of chaos and, since knowledge tends to come very slowly when the approach to inquiry is Brownian, its "raison d'etre" is self evident. With proper planning the scientific method can be implemented, as opposed to random data collection. To others, one of the main purposes of planning is to sort out and establish priorities and, thereby, *allow for the optimum allocation of resources.*

For many, the planning process is the vehicle for assuring that we do what is right as opposed to merely seeing to it that we do it right. With systematic planning, others contend, it is possible in the analysis of our results to sort out what is new information, what is confirmation of prior knowledge, and what is merely a manifestation of the information entropy that surrounds all T&E activities. Thorough planning anticipates contingencies and allows for them, it assures that all pertinent questions have been asked and that answers will be obtained, and provides the decision maker with adequate information upon which to act with an acceptable level of confidence, recognizing that nothing is certain, everything is sometimes, and indecision is a poor basis for flexibility.

Finally, planning can be used as a tool to advantage in competition for scarce resources. Those with allocation authority tend to be more favorably disposed towards those projects that are well thought out as evidenced by carefully considered execution plans. Planning is accomplished for all of the foregoing reasons, and the proper plan is one that reflects all of the applicable viewpoints in balance with one another.

WHAT IS THE ROLE OF RDT&E?

In the sedimentation and marine organism control area the question of the role or function of research, development, test, and evaluation (RDT&E) arises more readily than, say, in the development of a new weapons system. In the latter, the technological advances necessary to achieve the required capability are generally fairly clear and are understood by all involved parties; this is not so true in the former. To some, it is only necessary to dig deeper to assure that our ships have free and open access to the sea. Others hold the opinion that changes in ship design could obviate many of the current problems. To some, the problem is akin to the need for additional base housing and can be solved in much the same way while, to others, the real problem is inadequate understanding of the natural processes that must be controlled.

In the final analysis, the role of RDT&E is to reduce the risk to the decision maker to an acceptable level; it is to develop the capability for problem solution, not to implement solutions per se. The user community (i.e., those with the problems) should not necessarily expect immediate results at the outset of an RDT&E project. Such efforts are better aimed at developing long range solutions than band aids. An RDT&E project can recognize the site specific nature of sedimentation and marine organism control problems and, at the same time, provide the fundamental understanding that will allow data to be transferred to the maximum extent practicable. Thus, a role of RDT&E is to extend local capabilities and facilitate utilization of new and emerging technologies.

RDT&E is the bridge between unresolved problems and successful solutions. Hopefully, it is not a part of the former, but it cannot directly provide the latter in and of itself. There are few problems that cannot be solved if enough time and money are thrown at them. A role of RDT&E is to leverage that expenditure of resources to a measurable degree. If the expenditure of an R&D dollar only saves a dollar in solution cost, its utilitarian value is questionable. Although the expected degree of RDT&E leverage varies, values of five to ten or more are not uncommon. Given the Navy's current expenditures for dredging, the payoff potential for control technology is rather high.

Given the diverse expectations from RDT&E effort, one of its primary roles is to harmonize them, striking a balance among the presenting problems and viewpoints, and seeing that the proper trade-offs are carefully considered. And, RDT&E is the way the Navy acquires capability, which brings us to the Navy system.

THE NAVY SYSTEM

I have no intention here of synopsizing the Navy RDT&E system, the NAVFAC Command Management Plan, or even the Development Plan for this project. While these documents are important and must be understood by the direct participants, their bulk far exceeds that of these proceedings. Rather, I would like to emphasize the Navy system planning requirements for a project that is transitioning from advanced to engineering development.

To help formalize the T&E planning process that I spoke of earlier, the Navy system requires the preparation of either a Test and Evaluation Master Plan (TEMP) or a Test and Evaluation Plan (TEP). These are rather formal documents that are intended to demonstrate the thoroughness with which the planning process has been carried out. Which one is required by the Navy system depends upon the size of the effort (dollar thresholds for R&D, acquisition, etc.) and the need for Fleet services for testing (COMOPTEVFOR involvement). For the dredging and sedimentation control project, a TEP is required. In order to give the reader a sense of the kind of information that goes into a TEP, the following has been abstracted from OPNAVINST 3960.10.

PART III. INTEGRATED SCHEDULE. This part will consist of one page (may be a fold-out) displaying the integrated time-sequencing of test and evalution -- DT&E (including contractor test and evaluation, Navy preliminary evaluation, Navy technical evaluation, etc.), OT&E (IOT&E and FOT&E), PAT&E -- and related key events in the acquisition decision-making process. A legend may be used for essential explanatory notes; however, more complete information about the events on the schedule is contained in the DT&E, OT&E, and PAT&E Outlines (Parts IV, V, and VI, respectively). The following typical T&E events should be included in the integrated schedule:

1. Program milestones, such as program initiation, full-scale development, approval for service use, production, program reviews, etc.

2. Pertinent T&E data, including contractor demonstrations; laboratory tests/demonstrations; contractor acceptance tests; Navy preliminary evaluations (NPE); Navy technical evaluations (NTE); OT&E (including IOT&E, OPEVAL, and FOT&E); PAT&E; and any combined or joint testing.

3. Major resource availability requirements such as facility construction completion, target or ship schedule, range utilization, etc.

4. Key dates for issuance of test plans, reports, etc.

PART IV. DT&E OUTLINE. This part should show all planned DT&E in sufficient detail that resources can be identified, and the DA can subsequently develop detailed test plans. The near-term portion of the plan will contain more precise data; however, even the long-range portions should be as specific as possible as regards schedules and resources. Security of equipment and operations should be explicitly covered in all T&E planning. The DT&E Outline will contain the following three sections.

1. DT&E to Date. This section should contain a summary of the DT&E conducted prior to the date of the current revision of the TEMP. A brief description of actual test articles (brass-board, advanced development model, etc.) with emphasis on how the functional operational capability differed from the intended production

item should be included. DT&E events and results related to performance characteristics, critical issues, requirements levied by DSARC, etc., should be emphasized. Technical characteristics or specification requirements which were demonstrated (or failed to be demonstrated) should be addressed. Results/decisions of any program reviews should be shown.

2. Future DT&E. This section addresses all remaining DT&E commencing with the date of the current TEMP revision and extending through DT-IV. Each remaining phase of DT&E should be addressed individually, and should include the four sub-sections below:

(1) Equipment Description. This description emphasizes the functional capability and how it is expected to differ from the model tested in preceding DT&E/OT&E and the production model.

(2) DT&E Objecives. These are the specific objectives of each phase/sub-phase of planned DT&E. They are related to, but probably not the same as overall program objectives. If the program source documents (DCP/PM/NDCP/etc.) require demonstration of specific technical characteristics in a given DT phase, those characteristics are included.

(3) DT&E Events/Scope of Testing/Basic Scenarios. This sub-section includes all T&E events which will provide data with which to address the objectives. The scope of testing and basic test scenario should be described in sufficient detail that the relationship between the testing and the objectives is clearly apparent.

(4) Quantifiable Scope of Effort. This subsection contains a brief summary of the key elements of the testing expressed in quantifiable terms. The purpose is to provide perspective relative to the test effort without reference to the Resource Summary. Example: "DT-IIIA will consist of approximately 96 hours of controlled-geometry runs by two 637-Class SSNs, to verify the theoretical vertical and horizontal beam patterns of the AN/BQS-XX sonar."

3. Critical T&E Items. This section highlights any items whose availability is critical to the conduct of adequate DT&E prior to the next decision point, i.e., if the item is not available when required, the next decision point may be delayed. Critical items may be displayed on the Integrated Schedule if appropriate.

PART V. OT&E OUTLINE. The OT&E Outline is prepared by COMOPTEVFOR. It addresses all OT&E, from the early conceptual phase of IOT&E through the final phase of FOT&E. The OT&E Outline should show all OT&E in sufficient detail that resources can be identified and COMOPTEVFOR can develop test plans from it. The sections and subsections of the OT&E Outline follow the same pattern as the DT&E Outline (Part IV).

PART VI. PAT&E OUTLINE. The PAT&E Outline is the responsibility of the DA for all programs except ship and aircraft acquisition programs (normally ACAT-I's), when it is the joint responsibility of the DA and PRESINSURV. The PAT&E Outline will cover scope of testing, etc. The PAT&E Outline for ship and aircraft acquisition programs must be prepared in close coordination between the DA, PRESINSURV, and COMOPTEVFOR to prevent unnecessary testing. For ship programs, the methods of compliance with NAVMAT policies and procedures for the Ship Acceptance Test Phase and the Ship Post-Delivery Test Phase will be described herein.

PART VII. RESOURCE SUMMARY. This part contains a combined summary, in tabular form of the resources required for DT&E, OT&E, and PAT&E. Listed on the summary form are 12 resource categories, which include most of those likely to be required for T&E. Some categories may not be required for a particular program. On the other hand, additional categories may be required in special cases. These additional categories should be listed as required. For each category, show the major requirements (what, how much, how many) at the times they are required. If the tabular summary does not allow adequate space to define essential resource aspects, an additional page can be added. Resources should be shown in kind where possible, rather than in dollar terms. This part should also include the required location of each resource, and the planned disposition after completion of testing. If resources are already committed to the test program, these should be so indicated and listed. As an aid to developing the resource requirements in each of the categories listed, a brief explanation is presented.

1. Test Articles. The actual number of test articles required for each major type of T&E (DT&E, OT&E, PAT&E) should be identified. If subsystems (components, assemblies, or subassemblies) are to be tested individually, each such subsystem and the quantity required should be identified. Specifically identify requirements for preproduction prototypes, special preproduction prototypes, and production models. If a number of test systems are to be produced, indicate by serial number when each system is required.

2. Fleet RDT&E Support. The number of ship-days, aircraft-hours, and types of ships and aircraft should be estimated. If support is constrained to a specific area (Atlantic, Pacific) or to a specific ship or aircraft, so indicate. Time required for installation and removal of test systems and test-associated equipment should be indicated. A distinction should be made between dedicated, concurrent, and NIB support requirements. Include an estimate of the number of personnel who will be aboard each ship for T&E purposes, not including ship's company. Fleet RDT&E Support required solely for "target" purposes should be identified as such. (The services of non-fleet-controlled resources such as yard tugs, barges, and ancillary equipment are entered under the Support Equipment category.)

3. Test Sites/Ranges. Test sites or ranges to be used for T&E, and when they are required, should be listed. Fleet operating areas to be used as test sites will be identified by fleet nomenclature. Usage time is to be estimated in days and hours per day. When the Test site or range subdivision is identified, the normal instrumentation of that facility is expected to be available. Resource requirements for modifying existing facilities or developing new facilities will be included under this resource heading.

4. Targets. Estimates of target requirements should include type, number, when, and where the targets will be required. Targets include hulks of ships, designated targets in inventory (BOM-34, etc.), and targets that must be developed. Both target presentations and target expenditures should be shown. Special augmentation or instrumentation of targets should be indicated. Requirements for modifying existing targets or developing new targets will be included under this resource heading.

5. Special Instrumentation. Special instrumentation requirements for T&E should be identified (when and where required). Instrumentation installed at test sites, ranges, or in fleet units which will be available under normal circumstances need not be identified separately from the test site/range/fleet services. The source of the special instrumentation and the time required for installation should be identified, as should the installing activity (e.g., shipyard, tender, AMD, NARF, etc.).

6. Support Equipment. Support equipment is equipment required to conduct a test, but which is not part of the test itself; e.g., chaff may be required in a test of radar. The chaff launching device and its installation/removal would be listed as support equipment. (The chaff would be an expendable.) Support equipment should be identified by type, number required, date required and location. Support equipment which has standardized installation/removal factors or costs need only be noted as requiring installation and removal. Installation/removal time of the support equipment is to be estimated. The installing activity should be identified.

7. Installation/Removal Requirements. The installation and removal requirements for equipments, including test articles, which are actually used in tests to be conducted will be summarized. Support equipment installation and removal is a separate resource requirement and is carried under Support Equipment. If the installation and removal is initially for DT&E, and the same equipment will be used, in place, for OT&E, this will be indicated. The installing/removing acticity, estimated man-days required, and the work site should be identified.

8. Expendables. Included here are items expended during tests, not including test items or targets. Ammunition required to test a gun, missiles launched to test a launcher, chaff expended to test a dispenser, sonobuoys dropped to test an acoustic data

processor, etc., are expendables. Also included are specialized supplies not normally used by the test activity, test site, or supporting fleet unit(s). Include number and type required, and date and location required.

9. Logistics Support. Requirements for repair parts, spares, etc., in excess of normal shipboard spares or the normal support package provided with the test article should be shown. Include extra spares necessary to support other equipments used in conjunction with the test.

10. Personnel. Estimate personnel requirements in man-weeks per calendar period. Rank/rate/grade and number of personnel, and when they are required, are entered. Analytic and simulation support personnel should be tabulated separately from test personnel. Requirements for personnel other than test, analytical, or simulation support should be identified and included. If contractors are hired solely for testing, analytical, or simulation support, the man-weeks of contract support should be estimated.

11. Personnel Training. All test personnel and fleet or other source personnel who require training for the testing, including operators and maintenance personnel, are to be listed. Training of DA or COMOPTEVFOR test supervisors and observers should also be included. Identify rank/rate and number of military personnel to be trained as operators, maintenance personnel, test supervisors, and observers. Include source of personnel and when the training should be completed.

12. Planned Travel. This entry is required to permit long-range budgeting for travel and per diem by Fleet Commanders and COMOPTEVFOR. Estimate planned travel in dollar terms, by FY, subdivided by CINCLANTFLT, CINCPACFLT, and COMOPTEVFOR.

13. Other as (necessary).

PART VIII. REFERENCES. This part should list pertinent reports containing results of accomplished test and evaluation. In addition, developed test plans can be referenced for more detailed information.

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This volume records the proceedings of the First United States Navy Symposium on Dredging and Sedimentation Control, held on 22-23 April 1980 under the sponsorship of the Naval Facilities Engineering Command and the Office of Naval Research. Twelve papers dealing with various aspects of the subject area are included. They provide a general review of the state-of-the-art and findings obtained to date in the NAVFAC sponsored dredging research, development, test, and evaluation project and present a compilation of

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Block 20 Continued.

information on topics pertinent to the Navy's problems at Sewell's Point. This volume is intended to provide a useful reference by setting forth this material for the record, to provide a vehicle for sharing some of the information presented at the symposium with those who could not attend, and to focus attention on recent advances in our knowledge of the subject area.

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